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Authors

Papadopoulos, Vasileios
Murcia-Delso, Juan
Shing, P Benson

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Development of Headed Bars in Slab-Column Joints of Reinforced Concrete Slab Bridges

by Vasileios Papadopoulos, Juan Murcia-Delso, and P. Benson Shing

This paper presents results of an investigation on the development of headed bars extending from a column into the slab of a reinforced concrete slab bridge. Three full-scale slab-column specimens were tested under quasi-static cyclic lateral loading to determine the minimum embedment length required for the headed bars to develop their full tensile capacity, and the reinforcement details needed in the slab-column joint region to prevent premature anchorage failure, when a plastic hinge forms at the top of the column. The experimental results showed that for 5000 psi (34.5 MPa) concrete and Grade 60 steel, a development length equal to 11 times the bar diameter is adequate for headed bars in slab-column joints designed according to Caltrans specifications with a minimum of 2 in. (50.8 mm) of clear concrete cover. Specimens with shorter embedment lengths were able to develop the moment capacity of the columns and showed significant ductility, but exhibited moderate to severe punching cracks in the cover concrete of the slabs. Finite element analyses of slab-column assemblies showed that punching damage can be eliminated by increasing the concrete cover above the bar heads.

Keywords: cyclic loading; development length; finite elements; headed bars; large-scale testing; numerical analysis; punching; seismic design; slab bridges.

INTRODUCTION

Slab bridges are frequently used for bridges with short spans because they are economical to construct. In California, slab bridges, like other bridge structures that are designed according to the Seismic Design Criteria of the California Department of Transportation (Caltrans 2013), should have the superstructure remain elastic and plastic hinges developed at selected locations in the supporting columns during a major earthquake event. The columns should be designed so that they can sustain large inelastic flexural deformation without collapsing. Hence, for monolithic column-slab connections in a slab bridge, the longitudinal column reinforcement anchored in the deck slab must have sufficient development length to develop the full moment capacity of the columns and sustain significant plastic deformation at the top of the columns. The use of headed deformed bars for the longitudinal reinforcement can significantly reduce the required development length and, thereby, the thickness of the deck slab, and also avoid congestion that would otherwise be introduced by hooked bars.

The anchorage capacity of a headed bar is contributed by the bearing of the bar head against the concrete and the bond between the deformed bar and the surrounding concrete. Experimental studies conducted by De Vries et al. (1999) and Thompson et al. (2002) showed that the anchorage failure of a headed bar in tension is governed by three main

mechanisms: 1) side-face blowout failure of concrete when the bar is close to the edge of a concrete slab or block; 2) concrete breakout failure when the embedment length is shallow; and 3) bearing failure at the anchor head. Bars with a short embedment length compared to the lateral concrete cover such as column longitudinal bars embedded in a slab-column joint may experience breakout failures and bearing failures.

Based on the work of Thompson et al. (2005, 2006), ACI 318-14 (ACI Committee 318 2014) has the following formula in Section 25.4.4.2 to determine the minimum development length required for headed bars in tension

$$l_{dt} = \frac{0.016\psi_e f_y}{\sqrt{f'_c}} d_b \quad (1)$$

in which ψ_e shall be taken as 1.2 for epoxy-coated reinforcement and 1.0 for other cases; f_y is the specified yield strength of the reinforcing bar; d_b is the bar diameter, and f'_c is the specified compressive strength of the concrete, which shall not exceed 6000 psi (41.4 MPa). The units of Eq. (1) are pounds and inches. This empirical formula does not explicitly consider the different anchorage failure mechanisms mentioned earlier, and neglects the contribution of transverse reinforcement to the anchorage resistance of the headed bars.

Because Eq. (1) ignores the possible beneficial influence of stirrups in bar anchorage regions, it could be overly conservative for slab-column joints of slab bridges that are designed according to Caltrans MTD 20-7 (Caltrans 2016), and may call for unnecessarily large slab thickness to accommodate the required development length. For example, according to Eq. (1), with 5000 psi (34.5 MPa) concrete, a Grade 60 bar with a headed end will require a development length of $14d_b$. For a No. 9 bar, $14d_b$ is 15.8 in. (401 mm), while a multi-span slab bridge that has a span length of 30 ft (9.15 m) requires only a 16 in. (406 mm) thick slab to carry the design load according to Caltrans BDA 4-10 (Caltrans 2009) and the AASHTO LRFD Bridge Design Specifications (AASHTO 2017). This thickness is clearly insufficient to accommodate a 15.8 in. (401 mm) development length.

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Fig. 1—Reinforcement in slab-column connections.

This paper presents a study aimed to determine the minimum development length required for headed deformed bars in slab-column joints designed according to Caltrans MTD 20-7 (Caltrans 2016) and joint reinforcement details that would be required to prevent premature anchorage failure. To this end, laboratory tests were conducted on three full-scale slab-column assemblies. The test specimens had different bar sizes and embedment lengths. The experimental study was complemented by nonlinear finite element analyses. The study considered 5000 psi (34.5 MPa) concrete and Grade 60 bars, which are typically used in slab bridges in California. The head size meets ASTM A970/A970M-17 (2017) and provides a bearing area of $9A_b$ (Caltrans full-size heads), where A_b is the nominal cross-sectional area of the bar. The experimental program and results, accompanying numerical study, and resulting design recommendations are presented and discussed in the paper.

RESEARCH SIGNIFICANCE

The use of headed longitudinal column bars in slab-column joints of slab bridges can significantly reduce the required development length, and also avoid congestion that could otherwise be introduced by hooked bars. However, there is a lack of design standards for the development of headed bars in slab-column joints. The minimum development length specified in ACI 318 for headed bars could be overly conservative and may require an unnecessarily thick slab to develop column longitudinal bars. The study presented in this paper confirms that this development length can be reduced and provides recommendations to improve reinforcement details in slab-column joints of slab bridges.

SEISMIC DESIGN OF COLUMN-SLAB JOINTS IN SLAB BRIDGES

The design of the column-slab joints considered in this investigation is based on the 2014 Version of Caltrans Memorandum To Designer (MTD) 20-7 (Caltrans 2016), which has specific requirements to ensure that the superstructure of a slab bridge remains elastic and plastic deformations concentrate in the substructure during a strong earthquake. To ensure the development of the full tensile strength of the column longitudinal bars, it requires a minimum development length in the slab deck of $24d_b$ for straight bars, $19d_b$

for hooked bars, and $14d_b$ for headed bars. The headed bars need to have full-size heads, which have a bearing area nine times A_b , the cross-sectional area of the bar. The requirement for headed bars is based on Eq. (1), with a concrete compressive strength of 5000 psi (34.5 MPa), which is the minimum compressive strength expected according to Caltrans SDC (Caltrans 2013).

MTD 20-7 (Caltrans 2016) also establishes minimum quantities of longitudinal and transverse slab reinforcement as well as stirrups in the column-slab joint regions of a slab bridge. These requirements are based on the work of Ayoub and Sanders (2010). The joint region defined in MTD 20-7 is depicted in Fig. 1, which also shows typical reinforcement details. MTD 20-7 requires vertical J-bars tied to the upper reinforcing mat of the deck in the core region, which is defined as the region bounded by the column reinforcing cage, as shown in Fig. 1. These J-bars are to carry tensile forces induced by the headed bars, thus helping to prevent breakout failure. Additional vertical stirrups and longitudinal reinforcement are required for the outer part of the joint region, which extends horizontally a distance equal to one half of the slab depth from each side of the column cage in both directions. These requirements were derived with a strut-and-tie model (Ayoub and Sanders 2010).

TEST PROGRAM

Three slab-column assemblies were tested to determine the minimum development length required for headed column longitudinal bars in the joint region of a slab bridge, with the slab-column joint reinforced according to the requirements of MTD 20-7 (Caltrans 2016). These tests were also to examine if the joint detailing requirements in MTD 20-7 need to be improved. The slab-column joint under investigation and the test setup are shown in Fig. 2 and 3, respectively. The slab-column assemblies were tested in an upside-down position with two edges of the slab hinge-supported. Each specimen had a constant vertical load applied to the column, and quasi-static cyclic lateral displacements applied at the top of the column. The test setup mimicked the loading condition for a slab bridge when it is subjected to a lateral seismic force in the transverse direction, as shown in Fig. 2. The test assembly represented a portion of a three-span continuous slab bridge with a span length of 30 ft (9.14 m) and a slab thickness of

Fig. 2—Slab-column joint investigated. (Note: 1 ft = 304.8 mm; 1 in. = 25.4 mm.)

Fig. 3—Slab-column test setup. (Note: 1 ft = 304.8 mm; 1 in. = 25.4 mm.)

16 in. (406 mm). The hinge supports represented the lines of inflection of the deflected shape of the slab when the superstructure was subjected to a lateral seismic force transverse to the direction of the bridge. These lines were assumed to be midway between two adjacent columns in a bent. The top of the column in the specimens was pin-connected to a servo-hydraulic actuator, representing the point of inflection of the deflected shape of the column. The height of the column was selected to provide a shear-span ratio of 6. Figure 4 shows a picture of the test setup.

Test specimens

The specimens were designed according to the AASHTO LRFD Bridge Design Specifications (AASHTO 2017) and the Caltrans Seismic Design Criteria (SDC) (Caltrans 2013), as well as the additional requirements by Caltrans for slab bridges, which were MTD 20-7 (Caltrans 2016), and Bridge Design Aids (BDA) 4-10 (Caltrans 2009). Each of the slab-column assemblies had a 2 ft (610 mm) diameter cast-in-place column with a height of 12 ft (305 mm), measured from the top surface of the slab in the specimen to the elevation at which the lateral load was applied. The main differences among the test specimens were the size of the longitudinal reinforcing bars in the columns and the thickness of the slab at the slab-column joint. Table 1 summarizes the

Fig. 4—View of Specimen #1 and test setup.

design details of Specimens #1, #2, and #3. The specimens were tested in sequence, and the reinforcement details of the second and third specimens were based on the test results from the first specimen. To ensure that the desired information and the goal of the study could be achieved with three test specimens, a nonlinear analysis was conducted prior to each test using a detailed finite element model similar to that described in a later section. Detailed descriptions of the design of the specimens and finite element analyses can be found in Papadopoulos et al. (2015).

For Specimen #1, eight No. 9 (28 mm) headed bars were used for the longitudinal reinforcement of the column. The thickness of the slab was 16 in. (406 mm). The embedment length for the headed bars was $9.8d_b$, measured from the inner surface of the head to the top surface of the slab in the specimen orientation. This length was the maximum embedment length practically feasible for a 16 in. (406 mm)

slab if the head of a bar had to be above the bottom mat of reinforcement in the slab (that is, below the top mat of reinforcement in an actual deck slab), as shown in Fig. 1. A pre-test nonlinear finite element analysis had indicated that this length would be sufficient to develop the tensile strength of the headed bars. Figure 5 shows the column reinforcement and Fig. 6 shows the slab reinforcement for Specimen #1. The slab reinforcement complied with the requirements of MTD 20-7 (2014), and there was a 2-in. (51 mm) cover concrete over the top and bottom reinforcement. A picture of the slab reinforcement during the construction is shown in Fig. 7.

Specimen #2 had six No. 10 (32 mm) headed bars for the longitudinal reinforcement of the column. The total cross-sectional area of the longitudinal reinforcement was almost the same as that for Specimen #1. The thickness of the slab was kept at 16 in. (406 mm). The embedment length provided for the headed bars was the same as that in Specimen #1, which is $8.7d_b$ for the No. 10 (32 mm) bars. The

Table 1—Design details of slab-column specimens

Design characteristic	Specimen #1	Specimen #2	Specimen #3
Column diameter	24 in. (610 mm)		
Confined core diameter from outer edge of hoops	20 in. (508 mm)		
Column height	12 ft (3.66 m)		
Column longitudinal bars	8 No. 9 (28 mm)	6 No. 10 (32 mm)	6 No. 10 (32 mm)
Longitudinal steel ratio	1.77%	1.68%	1.68%
Column hoops	No. 5 (16 mm) @ 3.5 in. (89 mm)		
Transverse steel ratio	1.77%		
Slab thickness at slab-column joint	16 in. (408 mm)		19 in. (483 mm)
Embedment length of longitudinal bars	11 in. (279 mm), $9.8d_b$	11 in. (279 mm), $8.7d_b$	14 in. (356 mm), $11d_b$

Fig. 5—Column reinforcement for Specimen #1. (Note: 1 in. = 25.4 mm.)

purpose of Specimen #2 was to establish the lower limit of the development length. The reinforcement of the slab of Specimen #2 was the same as that of Specimen #1, except that it had four additional vertical stirrups at positions next to the column cage, as shown in Fig. 8. These four vertical stirrups were not required according to MTD 20-7, but were shown to be useful by the test results from Specimen #1.

Similar to Specimen #2, Specimen #3 had six No. 10 (32 mm) headed bars for the longitudinal reinforcement of the column. However, as shown in Fig. 9, a 3 in. (76 mm) deep drop cap was added to the 16 in. (406 mm) thick slab, providing an embedment length of $11d_b$ for the headed bars. This specimen was to check if structural performance could be improved, in terms of the severity of punching cracks in the slab, with the increase of the development length. The reinforcement for the drop-cap region was determined according to MTD 20-7 (Caltrans 2016).

Material properties and pre-test strengthening of Specimen #1

The concrete for the slabs had a specified 28-day compressive strength of 4000 psi (27.6 MPa), a slump of 4 in. (102 mm), and a maximum aggregate size of 1 in. (25.4 mm). The concrete for the columns had a specified compressive strength of 4500 psi (31.0 MPa). For each specimen, the slab was cast first, and the column was cast one week later. The specimens were to be tested when the strength of the concrete in the slab was close to 5000 psi (34.5 MPa). It was desired that the strength of the column concrete would be equal to or higher than that of the slab concrete so that the bar anchors would not be under tested. The actual strengths of the concrete measured on the day of each test are presented in Table 2. All the reinforcement was Grade 60, complying with ASTM A706/A706M-16 (2016). Each of the column longitudinal reinforcing bars had a full-size head complying with ASTM A970/A970M-17 (2017). Results from material tests on the steel reinforcement are presented in Table 3.

The compressive strength of the column concrete in Specimen #1 was way below the target minimum of 5000 psi

Fig. 6—Plan view of slab reinforcement for Specimen #1. (Note: 1 ft = 304.8 mm; 1 in. = 25.4 mm.)

Fig. 7—Slab and column reinforcement of Specimen #1 before concrete casting.

(34.5 MPa). This decrease of concrete strength could compromise the ductility of the column and might also slightly reduce its moment capacity, thus reducing the load demand on the slab-column joint. To circumvent this problem, the lower 4 ft (1219 mm) of the column was retrofitted with external steel hoops, as shown in Fig. 4, to provide passive confinement. The hoops were made of Grade 50 steel, and were 1 in. (25.4 mm) wide and 3/16 in. (4.8 mm) thick. They were spaced at 4 in. (102 mm) on center. The hoops were fabricated in halves and welded together in place. Their inner diameter was 0.5 in. (12.7 mm) larger than the diameter of the column. A fluid high-strength grout was injected into the gaps between the column and the hoops. Material tests conducted on steel coupons showed a yield strength of 54 ksi (372 MPa) and a tensile strength of 70 ksi (483 MPa).

Table 2—Compressive and tensile strengths of concrete on day of test

Specimen	Region	Compressive strength of concrete, ksi (MPa)	Splitting tensile strength of concrete, ksi (MPa)
#1	Column	3.2 (22.1)	0.46 (3.2)
	Slab	5.0 (34.5)	0.35 (2.4)
#2	Column	4.8 (33.1)	0.41 (2.8)
	Slab	4.9 (33.8)	0.45 (3.1)
#3	Column	5.0 (34.5)	0.43 (3.0)
	Slab	4.5 (31.0)	0.43 (3.0)

Instrumentation

Electrical resistance strain gauges were attached to select column and slab reinforcing bars of the specimens. Strain gauges were placed at different elevations on the column longitudinal bars located on the north and south sides of each specimen (which was loaded in the north-south direction, as shown in Fig. 3) to monitor the strain distributions along these bars, including strains along the embedment length in the joint region. The strain gauges were placed on the longitudinal ribs of the bars to avoid disturbing the transverse ribs, which could affect the bond characteristics.

Displacement transducers and inclinometers were installed to measure the lateral displacements and rotations of the column, and the rotations of the slab. Vertical displacement transducers were mounted near the base of the column to measure the rotation of the column base with respect to the slab caused by bar slip.

The reader is referred to Papadopoulos et al. (2015) for detailed instrumentation schemes. Locations of sensors will be identified in figures in which the sensor data are presented.

Fig. 8—Plan view of slab reinforcement for Specimen #2. (Note: 1 ft = 304.8 mm; 1 in. = 25.4 mm.)

Fig. 9—Slab with drop cap in Specimen #3. (Note: 1 ft = 304.8 mm; 1 in. = 25.4 mm.)

Loading protocol

In each test, the column was subjected to a constant vertical load of 70 kip (311.4 kN) to simulate the gravity load on the column from the superstructure. This resulted in an axial compressive stress of 3.5% of the target compression strength of the concrete at the base of the column. The vertical load was applied by two post-tensioned rods, as shown in Fig. 3. The forces in the post-tensioned rods were controlled by center-hole hydraulic jacks placed on top of the steel beam sitting on the top of the column. These rods passed through holes in the steel beam and the slab, and were anchored to the bottom of the slab with a hinge mechanism.

Under the constant vertical load, the top of the column was subjected to fully reverse lateral displacement cycles using a

220 kip (979 kN) load capacity, 48 in. (1219 mm) stroke actuator, as shown in Fig. 3. Initially, the specimen was subjected to four fully reversed force-controlled load cycles, with amplitudes of 25, 50, 75, and 100% of the lateral load F_y' that caused the first yield of the longitudinal reinforcement at the base of the column. The specimen was then subjected to fully-reversed displacement-controlled load cycles with increasing ductility demands of 1, 2, 3, 4, and so forth, until the lateral load resistance dropped significantly. There were two cycles at each ductility level. The ductility demand was defined as $\mu = \Delta / \Delta_y$, in which Δ was the lateral displacement of the column at the level of the centerline of the horizontal actuator, and Δ_y was the effective yield displacement, which was calculated as

Table 3—Yield and tensile strengths of steel reinforcement

Specimen	Reinforcement	Bar size, No. (mm)	Yield strength, ksi (MPa)	Tensile strength, ksi (MPa)
#1	Column longitudinal bars	9 (28)	69.0 (476)	99.5 (686)
	Column hoops	5 (16)	67.5 (465)*	91.2 (629)
	Slab longitudinal bars	8 (25)	66.5 (459)	91.5 (631)
	Vertical ties in slab	5 (16)	67.5 (465)	96.0 (662)
#2	Column longitudinal bars	10 (32)	64.5 (445)	94.2 (650)
	Column hoops	5 (16)	69.6 (480)*	92.2 (636)
	Slab longitudinal bars	8 (25)	68.2 (470)	98.2 (677)
	Vertical ties in slab	5 (16)	61.0 (421)	85.0 (586)
#3	Column longitudinal bars	10 (32)	71.1 (490)	90.1 (621)
	Column hoops	5 (16)	64.0 (441)*	97.9 (675)
	Slab longitudinal bars	7 (22)	64.5 (445)	96.5 (666)
	Slab longitudinal bars	8 (25)	67.6 (466)	97.5 (672)
	Vertical ties in slab	5 (16)	63.2 (436)	90.1 (621)

*Stress at 0.002 strain (no yield plateau).

Fig. 10—Lateral force-versus-drift ratio curves for Specimens #1 and #2. (Note: 1 kip = 4.45 kN.)

$$\Delta_y = \frac{F_y}{F'_y} \Delta'_y \quad (2)$$

in which Δ'_y and F'_y were the lateral displacement and load at which the extreme tensile column longitudinal bar reached the yield strain for the first time, respectively; and F_y was the expected ultimate strength of the specimen. To determine the loading protocol, F'_y and F_y were calculated with finite element analyses using models that are described in a later section, and Δ'_y was taken as the average of the absolute maximum displacements measured in both loading directions in Cycle 4 of the test.

TEST RESULTS

Global response and general observations

The lateral load-versus-drift curves for Specimens #1 and #2 are compared in Fig. 10, while the same comparison for Specimens #2 and #3 is provided in Fig. 11. The drift ratio is defined as the lateral displacement of the column at the point of load application with respect to the column base

Fig. 11—Lateral force-versus-drift ratio curves for Specimens #2 and #3. (Note: 1 kip = 4.45 kN.)

(in the test configuration) divided by the vertical distance between the load application point and the column base. The displacement and the load are positive when they are towards north. The ductility demand level indicated is calculated with Eq. (2) using the values of Δ'_y , F'_y , and F_y actually measured in the respective tests to calculate the effective yield displacement Δ_y . The effective yield displacement calculated for Specimen #1 is 2.5 in. (64 mm), while that for Specimens #2 and #3 is 2.0 in. (51 mm).

As shown in Fig. 10 and 11, all three specimens exhibited ductile behavior due to the formation of plastic hinges near the base of the columns (the side connected to the slab). However, Specimen #2 exhibited a more rapid load degradation than Specimen #1 after reaching the peak resistance, and more pinched hysteresis curves. The less-severe load degradation exhibited by Specimen #1 can be attributed to the external confinement applied to the column, which helped to alleviate the crushing of concrete in the plastic-hinge region. The pinching of the hysteresis curves for Specimen #2 was caused by the more severe slip of the column longitudinal

bars in the anchorage region. Specimen #2 had the shortest embedment length of $8.7d_b$ among the three. As shown in Fig. 11, Specimens #2 and #3 exhibited very similar rate of load degradation, which was caused by the crushing of concrete in the plastic-hinge region, but Specimen #3 had the least pinched hysteresis curves compared to the other two specimens. This is consistent with the fact that Specimen #3 had the longest embedment length of $11d_b$.

Figure 12 shows the damage on the bottom surface of the slabs for Specimens #1, #2, and #3. Specimen #2, which had the shortest embedment length, experienced the most severe damage with severe concrete spalling on the bottom surface during the last cycle at Ductility 6. The spalling was limited to the cover concrete and was caused by the punching action of the anchor heads when the bars were in compression. Parts of the slab longitudinal reinforcement and the hooks of some J-bars and vertical stirrups were exposed. These punching cracks initiated during the displacement cycles at Ductility 1. During the last cycle at Ductility 6, the strain measured in a longitudinal bar of the slab right below the head of a column bar exceeded -0.012 . This strain was most likely caused by bar bending under the punching action of the headed bar. Moderate punching cracks were observed on the bottom surface of the slab of Specimen #1. At the end of the test, the maximum relative residual displacement measured across the punching cracks was 0.2 in. (5 mm). The slab of Specimen #3, which had the longest embedment length among the three, experienced very limited cracking on the bottom surface. The maximum relative displacement measured across the punching cracks during loading at Ductility 6 was 0.021 in. (0.5 mm), and did not increase at Ductility 7.

Figure 13 shows the damage on the top surface of the slabs (in the test configuration) at Ductility 6 for Specimens #1, #2, and #3. In all three specimens, flexural cracks were observed in early cycles at a ductility level of 1 or 2. As the ductility demand increased, additional cracks, radiating from the column, appeared and propagated towards the free edges of the slab as shown in Fig. 13. In addition, in all three specimens, a circular crack developed around the column at 2 to 4 in. (51 to 102 mm) away from the column surface due to the pullout forces exerted by the longitudinal bars of the column. The opening of the cracks increased in latter cycles.

Figure 13 also shows the damage of the columns at Ductility 6. In Specimen #1, the spalling of concrete in the plastic-hinge region was limited because of the external steel confinement. The steel hoops yielded in tension. The column of Specimen #3 had more severe concrete spalling than that of Specimen #2. This can be attributed to the reduced slip of the headed longitudinal bars within the joint region of the slab of Specimen #3, which resulted in more severe curvature demand in the plastic-hinge region of the column.

Strain penetration in bar anchorage region

Figure 14 plots the strains in the headed bars in the anchorage region at the extreme south face of the columns for all three specimens. Horizontal loads were applied to the columns in the north-south direction. The depth of the strain gauges shown in the graphs are measured from the top

(a) Specimen #1

(b) Specimen #2

(c) Specimen #3

Fig. 12—Damage at bottom face of slab (test configuration) at end of tests.

surface of the slab (in the test configuration). The deepest gauge for each specimen was 1 in. (25 mm) away from the anchor head. The plots show that the columns in all three specimens were able to develop the full moment capacity with the maximum tensile strains in the extreme reinforcing bars greater than 0.01, which is way beyond the yield strain. In all three specimens, the reinforcing bars developed plastic strains all the way to the anchor head. This indicates that the bar forces were developed by the bearing action of the heads on the concrete. However, Specimen #3 had by far the lowest strain developed near the anchor head, indicating

(a) Specimen #1

(b) Specimen #2

(c) Specimen #3

Fig. 13—Damage at base of column and top face of slab at $\mu = 6$.

better bonding of the bar to the concrete. Specimen #2 had the lowest tensile strength near the surface of the slab, probably due to the more severe slip of the bar.

Strains in J-bars and vertical stirrups

Figure 15 shows the strains developed in the J-bars placed in the core region of the slab-column joints for all three specimens. The strains were measured with gauges located at their midheight. In Specimen #1, the J-bars developed large tensile strains, with the maximum reaching the yield level. These bars were always subjected to tension whether the column was pushed towards the north or the south—that

is, regardless of the fact that the longitudinal headed bars next to the J-bars were subjected to tension or compression. This indicates that the J-bars were engaged to resist both the pullout and punching forces exerted by the headed bars after cracking occurred in the joint region of the slab. Similar tensile strains were measured in the J-bars of Specimen #2, although slightly smaller. In Specimen #3, the strains barely reached half of the yield strain, indicating that the J-bars were not engaged as significantly as those in Specimens #1 and #2. This is most likely due to the fact that the breakout and punching cracks in Specimen #3 were less severe compared to the other specimens.

Figure 16 shows the strains in the stirrups in the slabs outside the column steel cage. The fluctuation of these strains is similar to that observed for the J-bars. The stirrups in Specimen #3 developed very small strains, way below the yield level. In Specimens #1 and #2, only the stirrups right next to the column cage developed large strains, showing that those farther away had little contributions to the behavior of the slab-column joint. In Specimen #2, Stirrup V4, which was not present in Specimen #1, developed the largest strain, reaching the yield level. This indicates that the four additional Stirrups V4, V7, V10, and V13 (refer to Fig. 16), which were closest to the column cage in Specimen #2, were most significantly engaged in resisting the forces exerted by the headed bars. They were, therefore, more important than the other stirrups.

NUMERICAL STUDY

Finite element model

A three-dimensional finite element (FE) model was developed to simulate the behavior of the slab-column specimens, and to further investigate the anchorage mechanism of the headed bars and their interaction with the reinforcement in the slab-column joints. Figure 17 briefly describes the FE model developed in Abaqus (Simulia 2017). For concrete, the microplane model developed by Caner and Bažant (2013) has been implemented and used for the analyses. The column longitudinal bars are modeled with beam elements using a steel material law developed by Papadopoulos and Shing (2015), which is based on the Menegotto-Pinto model and is able to account for the low-cycle fatigue of reinforcing bars. Bond slip between the longitudinal bars and concrete is considered, using the bond-slip model of Murcia-Delso and Shing (2015). The modeling of the anchorage capacity of headed bars was validated with the tests of De Vries et al. (1999) and Choi et al. (2002). The reader is referred to Papadopoulos et al. (2015) for a detailed description of the constitutive models and the modeling method.

The FE model has been validated by the column-slab assembly tests presented in previous sections. Figure 18 presents the numerical results obtained for Specimen #2. The deformed FE mesh presented in the figure shows significant punching damage below the headed bars, similar to that observed in the test. It can also be observed that the lateral load-versus-drift ratio hysteresis curves from the FE analysis (FEA) closely match the test results. The strains measured in the J-bars and vertical stirrups have also been replicated by the FE model reasonably well, as shown in Fig. 19.

Fig. 14—Strains in headed bars along their embedment length. (Note: 1 in. = 25.4 mm.)

Fig. 15—Strains in J-bars in slab. (Note: 1 in. = 25.4 mm.)

Effectiveness of concrete cover thickness

To investigate the influence of the concrete cover below the bar head (in the specimen configuration) on the punching resistance, a FE analysis has been conducted. In the FE model, the headed bars in Specimen #2, which experienced the most severe punching damage at the bottom of the slab, are moved away from the bottom face of the slab by $2d_b$, thus reducing the bar embedment length from $8.7d_b$ to $6.7d_b$ and increasing the concrete cover by $2d_b$. This is referred

to as Specimen #2B. As shown in Fig. 20, the increased concrete cover significantly reduces the punching damage. In spite of the smaller embedment length, the lateral load capacity of the column was fully developed, the ductility of the column was not compromised, and the hysteresis curves were less pinched as compared to Specimen #2. This indicates that punching resistance is the controlling factor and the development length required to develop bar tension can be as low as $6.7d_b$.

Fig. 16—Strains in vertical stirrups in slab. (Note: 1 in. = 25.4 mm.)

Fig. 17—FE model of Specimen #1.

Effectiveness of shear reinforcement in slab-column joint

The importance and effectiveness of the shear reinforcement in the slab is shown by two additional FEA of Specimen #3, in which the amounts of the vertical stirrups and J-bars are varied. In one case, identified as Specimen #3B, all the J-bars and vertical stirrups are removed. In the second case, identified as Specimen #3C, only the J-bars and the first row of vertical stirrups closest to the column cage are retained, while all other vertical stirrups are removed. As shown in Fig. 21, Specimen #3B, in which all the shear reinforcement is removed, experiences severe punching damage induced by the headed bars. The lateral load capacity of the column is developed, but the hysteresis curves are significantly more

pinched with more severe load degradation than Specimen #3. Specimen #3C has similar behavior as Specimen #3. The FEA confirms the experimental observations that the vertical stirrups in the second row and farther away the column cage have little influence on the behavior of a slab-column joint.

CONCLUSIONS

An experimental study on the embedment length of headed bars anchored in slab-columns joints of RC slab bridges was presented in this paper. The experimental data are complemented by nonlinear finite element analyses to obtain a better understanding of the influence of different design variables. The study has shown that, for 5000 psi (34.5 MPa) concrete and Grade 60 steel, an embedment length of $11d_b$

Fig. 18—FEA of Specimen #2. (Note: 1 kip = 4.45 kN.)

Fig. 19—Strains in J-bar and vertical stirrups in FEA of Specimen #2.

Fig. 20—Variation of concrete cover thickness below headed bars. (Note: 1 kip = 4.45 kN.)

Fig. 21—Variation of amount of shear reinforcement in slab-column joint. (Note: 1 kip = 4.45 kN.)

is adequate for headed bars in slab-column joints designed according to MTD 20-7 (Caltrans 2016) with a minimum of 2 in. (50.8 mm) of clear concrete cover. This development length has been adopted by Caltrans in the most recent version of MTD 20-7 (Caltrans 2016).

The laboratory tests and FEA conducted in this study indicated that the J-bars and vertical stirrups specified in MTD 20-7 for slab-column joints of slab bridges help to improve the anchorage capacity of the headed bars extending from the columns into the slab, and can thus reduce the required bar development length. However, the FEA results showed that only the J-bars and the vertical stirrups in the first row away from the column cage are needed for the development of the headed bars in the slab, while the amount of vertical stirrups in the second row or farther away, as required by MTD 20-7, can be significantly reduced. They are only required to provide the necessary shear strength for the slab.

Furthermore, the experimental and numerical results showed that the required bar development length is governed by the punching failure of the cover concrete in the slab rather than the breakout failure of concrete induced by bar tension. Hence, the thickness of the cover concrete over the bar head is also an important consideration for design. For the same reason, it is important that the bar heads are underneath the top reinforcement mat in the slab. For a deck slab that has a 2 in. (51 mm) cover concrete over the top reinforcement mat, or 4 in. (102 mm) over the bar heads, a minimum development length of $11d_b$ is required to prevent severe punching cracks. This embedment length is also sufficient to develop the full tensile strength of the bar with little deterioration of the anchor capacity under severe cyclic loading. The FEA results have shown that the bar development length can be reduced to $6.7d_b$ without jeopardizing the anchorage performance when the cover concrete over the bar heads is increased from 4 to 6.5 in. (102 to 165 mm).

Furthermore, the slab-column joint region should have an adequate amount of J-bars and vertical stirrups, as discussed earlier. Further experimental investigation is needed to verify the numerical findings and to study the effect of other design parameters, including the concrete cover over the bar heads, the size of the bar head, and the amount of vertical stirrups in the slab-column joint. In addition, studies are needed to determine the minimum development length required to prevent the pullout failure of headed bars in a slab-column joint as a function of the design parameters.

AUTHOR BIOS

Vasileios Papadopoulos received his BS from the National Technical University of Athens, Athens, Greece, and his MS and PhD from the University of California, San Diego (UC San Diego), La Jolla, CA.

ACI member **Juan Murcia-Delso** is an Assistant Professor at the University of Texas at Austin, Austin, TX. He received his BS and MS from the Technical University of Catalonia, Barcelona, Spain, and his PhD from UC San Diego. His research interests include the nonlinear behavior and seismic performance of reinforced concrete structures.

ACI member **P. Benson Shing** is a Professor of structural engineering at UC San Diego. He received his BS, MS, and PhD from the University of California, Berkeley, Berkeley, CA. His research interests include seismic design and the behavior and performance of concrete and masonry structures, including large-scale testing and analytical modeling.

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