Pull-Out Performance of Eccentrically Spliced Longitudinal Headed Bars for Precast Beam-Footing Connections

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Abstract

For common detailing of footings in steel or precast concrete structures, longitudinal reinforcement of foundation beams is bent horizontally and spliced with reinforcement of cast-in-place footings to insure an adequate juncture for load transfer. In this study, instead of bending longitudinal reinforcement bars of both, beams, and footings, headed reinforcement bars are adopted. By doing so, a discontinuity region is created where longitudinal bars of footings become eccentric to those of beams that are embedded in the footings. To allow developing forces in longitudinal bars of beams flow to longitudinal bars of footings, a set of reinforcing ties is provided between them. As such setting of headed reinforcement bars is not common, thorough investigations have been carried out. In this paper, the pull-out performance of eccentrically spliced longitudinal headed bars with different detailing of transverse reinforcement, proposed for precast beam-cast-in-place footing connection, is discussed based on an experimental investigation. A method, based on the friction shear theory, for the strength evaluation of such arrangement is suggested.

1. Introduction

In common practice, when precast concrete or steel columns are adopted, or when avoiding congestion problems, longitudinal bars of foundation beams are horizontally bent around columns and spliced to longitudinal bars of footings to insure an adequate juncture for load transfer from beams to footings (Fig. 1). Such way of doing is not cost effective. Furthermore, to reduce construction tasks, use small capacity cranes at sites, shorten construction time of buildings and avoid reinforcement congestion problems at their foundations, simple construction methods are required. To satisfy such requirements, the authors have proposed a simple structural system for foundations, which consists of precast foundation beams and cast-in-place footings.

As an alternative to bending longitudinal reinforcement of foundation beams and footings, the authors conceived their precast beams with protruding longitudinal headed bars at their ends (Fig. 2). To use small capacity cranes at sites, each beam is partitioned, transversely, into 2 or 3 prefabricated sub-beam elements of smaller width, as depicted by discontinuous lines in the plan view of Fig. 2. By adopting straight headed bars, a discontinuity region is created, where the longitudinal bars of the footings become laterally eccentric to those of the beams that are embedded in the footings. To allow developing forces in the longitudinal bars of the beams flow to the longitudinal bars of the footings, providing an appropriate set of reinforcing ties becomes a challenge, though various ways and combinations that satisfy detailing regulations can be implemented. For instance but not limited to: in case of normal shear reinforcement, setting up lapped U-shaped bars, or one side headed, or two sides headed bars, and in case of slanted shear reinforcement, setting up above and below, respectively, the upper and lower longitudinal steel bar layers of each side (compression and tension), and if any obstacles exist, such slanted bars can be partitioned and then lap spliced within the footing along the beam width. As the proposed foundation system and its connections are intended for moderate to high seismic regions, their cyclic behavior in terms of energy dissipation and ductility capabilities should fulfill design criteria. Thus, its principal failure mode, as to seismic loading, should be of ductile type.

For regulations, whereas some requirements, such as the minimum ratio for confining reinforcement and limit for concrete cover, specified in some codes, like in AJJ (2018, 2010) and ACI 318-19 (ACI 2019), for non-contact splicing (spaced splicing) of longitudinal reinforcement can generally be fulfilled for common detailing of structural elements, the limit specified for the spacing of non-contact splices might not be satisfied when the arrangement of longitudinal reinforcement becomes uncommon where non-alternated longitudinal reinforcement and/or columns of large size (width) be adopted, as illustrated in Fig. 2.

When non-contact splicing was investigated experimentally, as in the studies carried out by Chamberlin (1952, 1958) on bent beams, Sagan et al. (1991) on tensioned wall-like elements and Hamad and Mansour (1996) on bent slab elements, the transverse spacing of spliced bars was within the limits specified by regula-
tions and the bars were arranged in a way that force transfer occurred between the spliced bars without being intensely affected by the developed forces in the other adjacent bars.

When eccentric and/or wide beam elements were dealt with in some studies like the ones carried out by Rafaele et al. (1992), Burak and Wight (2004), Shin and Lafave (2004), Lam et al. (2011) and Luk and Kuang (2012), because longitudinal beam bars of the studied connections were mainly anchored in the beam-column joint’s core or placed closely at beam ends, issues related to region/reinforcement discontinuities were not addressed. Furthermore, when headed bars were used as in the studies carried out by Thompson et al. (2003a, 2003b, 2006), Ishikawa et al. (2004), Lee and Yu (2009), Yang et al. (2010) and Tagawa et al. (2011), headed bars in opposite directions were arranged alternately and closely within the considered connection zones. Whereas such arrangement of headed bars addressed the issue of reinforcement discontinuity (non-contact), eccentricity of sets of headed bars and stress flow between such eccentric reinforcement sets have not sufficiently been investigated.

As the proposed setting of headed reinforcement bars is not common and not explicitly presented by design regulations, thorough investigations have been carried out. To ensure a good beam-footing connection, the effects of the amount of reinforcement ties, splicing length, lateral eccentricity between longitudinal reinforcement of a foundation beam and those of a footing, and the amount of headed reinforcement bars on its seismic performance have been studied on different stages. This paper reports the results of a preliminary experimental investigation phase, dealing with the pull-out performance of eccentrically spliced longitudinal headed bars with different detailing of transverse reinforcement and suggests a simple evaluation method for the strength capacity.

2. Test outline and specimens

To grasp the behavior of the proposed beam-footing connection and observe the likely developing failure mechanisms under cyclic bending, at first, a preliminary pull-out test of eccentrically spliced longitudinal headed bars with different arrangements of transverse rein-
forcement was carried out.

2.1 Specimens and materials

Specimens constructed for testing, do not include the concrete part of the precast beams. The specimens represent only a part of the footing, which includes the longitudinal bars protruding from the precast beams that would undergo a tensile force due to bending of the beams (Fig. 2). The geometry, detailing and main characteristics of the specimens are shown in Fig. 3 and Table 1. Specimens were 815 mm wide, 200 mm thick and 400 mm long, and provided with a stub (815 mm x 650 mm x 200 mm) that should be fixed to a rigid base for loading. A sufficient concrete cover was adopted to avoid any spalling close to the anchor heads and any premature shear bond failure. As illustrated in the plan view of Fig. 2, the distance between the closest eccentric longitudinal bars of the beam and footing was decided, according to the beam width (herein 250 mm) and, mainly, the size of the column (herein 300 mm). Such spacing was beyond the requirement prescribed in AJJ (2010, 2018) and ACI 318-19 (ACI 2019) which would be the minimum of 150 mm and one fifth the splicing length of the non-contact bars. Splicing length of the eccentric longitudinal bars in all the specimens was fixed to 20 times the diameter of the beam longitudinal bars. These eccentric bars were tied by transverse reinforcement, distributed along the splicing length with a shear reinforcement ratio of 0.8% (relative to the area of Section C in Fig. 3) and satisfying the minimum requirements of AJJ (1996). When slanted shear reinforcement was added, half of the amount of the transverse reinforcement was considered. Conditioned by the size of the footing, the inclination from the beam axis of the slanted bars was 50 deg. for the specimen No-2 and 62 deg. for the specimens No-4 and No-5. When footing longitudinal reinforcements in the orthogonal direction were added, only 2/3 of the amount of the footing longitudinal bars in the specimen’s axial direction were considered, assuming a less reinforced footing in the orthogonal

Fig. 3 Outline of specimens and detailing (strain gauge location shown by red and green disk marks).
direction. All the specimens (specimens No-1 to No-5) were designed to experience shear failure, except one (specimen No-6), which was designed to experience yielding of beam longitudinal steel bars and would be recommended for construction in actual buildings. Whereas, in common practice, normal grade reinforcements are used, in this test, to prevent any premature yielding before shear failure during loading, high grade steel bars (SD685 type) were used in the specimens No-1 to No-5 as beam longitudinal reinforcement instead of normal grade steel bars (SD390 or SD345). Normal concrete was used to construct all the specimens. Table 2 lists the characteristics of materials.

The parameters investigated in this phase of the study concerns the effect of slanted shear reinforcement (comparison of specimen No-1 with specimen No-2 and specimen No-4 with specimen No-5), effect of longitudinal reinforcement of footings in the transverse direction (comparison of specimen No-1 with specimen No-3) and effect of alternating some of beam longitudinal reinforcement with footing longitudinal reinforcement (comparison of specimen No-1 with specimen No-4 and specimen No-2 with specimens No-5 and No-6).

### Table 2 Characteristics of specimens.

| Specimen | No-1 | No-2 | No-3 | No-4 | No-5 | No-6 |
|----------|------|------|------|------|------|------|
| Footing longitudinal reinforcement | 2×6D16 \( \text{(SD390)} \) | | | | | |
| Footing transverse shear reinforcement | 2D6@40 \( \text{(SD295)} \) | | | | | |
| Beam longitudinal reinforcement | 8D16 \( \text{(SD685)} \) | 8D16 \( \text{(SD390)} \) | | | | |
| Slanted shear reinforcement | 0 | 8D6 \( \text{(SD295)} \) | 0 | 8D6 \( \text{(SD390)} \) | | |
| Longitudinal bars of footing in orthogonal direction | 0 | 4D16 \( \text{(SD390)} \) | 0 | | | |
| Alternation of opposed headed bars of beam and footing | Not Implemented | | Implemented | | | |

### Table 2 Material properties (Laboratory test results).

| Material          | Concrete | Steel                      |
|-------------------|----------|----------------------------|
|                   | D6 \( \text{(SD295)} \) | D16 \( \text{(SD685)} \) | D16 \( \text{(SD390)} \) |
| Compressive strength (MPa) | 40.9 | - | - | - |
| Young’s Modulus (MPa) | 28 000 | 190 600 | 189 000 | 192 900 |
| Tensile yield strength (MPa) | - | 357.2 | 738.9 | 442.0 |
| Tensile strength (MPa) | 2.71 | 529.1 | 928.4 | 619.2 |
| Ultimate elongation (%) | - | 12.4 | 13.4 | 21.9 |

2.2 Loading procedure and instrumentation

The loading setup is shown in Fig. 4. Specimen’s stub was fixed by high strength bolts to a rigid floor of the testing facility. The loading setup was accommodated for tensile forces to be applied vertically through the protruding beam longitudinal reinforcement, which were fixed at their end to a loading beam. The loading beam was pushed away using two symmetrically-placed hydraulic jacks. A non-reversed cyclic tensile loading pattern with three successive cycles followed by a monotonic phase until failure was planned. Each loading cycle was to be performed twice. The amplitudes of loading cycles were related to the nominal yield force level \( P_y \) of the beam longitudinal bars of normal grade (SD390). They were, successively, \( F_1 = P_y/3 \), \( F_2 = 2P_y/3 \) and \( F_3 = P_y \).

Specimens were instrumented internally and externally. Linear variable displacement transducers (LVDTs) were installed to measure the total and relative displacements at different locations, particularly at the

![Fig. 4 Loading setup and loading pattern.](image-url)
concrete upper face on the axis of each specimen (“CF” location in Fig. 4) and at the lower face on the axis of the loading steel beam (“SF” location in Fig. 4). Limited number of electric resistance strain gauges were placed at several locations on the longitudinal and transverse reinforcement (Fig. 3). The hydraulic jacks were fitted with load cells. Signals from the LVDTs, strain gauges and load cells were processed through a computerized data acquisition system.

3. Test results

Under the applied loading, several differences had appeared on the specimens, in terms of crack progression, reinforcement deformation and failure type. Test results showed that using only distributed shear reinforcement to tie the eccentrically spliced longitudinal reinforcement did not bring satisfaction as a ductile failure did not occur and when slanted shear reinforcement was added, they slightly improved the performance of the eccentric connection, whereas the failure mode was unchanged, remained brittle and occurred before yielding of the beam longitudinal reinforcement. However, when some of the beam longitudinal bars were alternated with those of the footing, the performance was greatly improved and a ductile behavior could be developed by combining slanted shear reinforcement, proving that the proposed arrangement of eccentric reinforcement of the specimen No-6 would be an adequate solution without increasing considerably the amount of reinforcement in the footing. Furthermore, the adopted value of splicing length (20d, d: longitudinal bars diameter) of headed bars proved to be adequate for the proposed arrangement of eccentric reinforcement.

3.1 General observations and response description

Behaviors and failure modes of the tested specimens occurred as expected. All the specimens experienced shear failure and yielding of their footing longitudinal reinforcement, except the specimen No-6, which experienced a ductile failure and yielding of its beam longitudinal bars without any yielding of the footing longitudinal reinforcement. Figure 5 shows the relationship between the tensile strength (applied load) and displacement of the concrete upper surface on the axis of each specimen during test (“CF” location in Fig. 4). For relatively small displacements, all the specimens showed almost similar responses to the applied load until reaching the peak of loading cycle P = F1, since then several

![Fig. 5 Load-displacement of upper concrete surface relationships of tested specimens.](image-url)
differences had been observed, especially between the specimens No-1, No-2, and No-3.

For all the specimens, the load transfer was achieved through bond at first and then through compression struts that developed from the anchor heads of the beam longitudinal bars to those of the footing longitudinal bars and shear failure was triggered when transverse reinforcement close to the anchor heads of the footing longitudinal bars reached their yield strength, except for the specimen No-6. Measured tensile strengths of the specimens are listed in Table 3.

The performance of the specimen No-1 was not satisfactory, as its shear capacity \( P_{\text{max, No-1}} \) did not reach the defined tensile yield strength \( P_y = 620.0 \) kN of the beam longitudinal bars (8D16, SD390), based on the nominal yield stress of normal steel bars (SD390), besides that it showed the lowest shear capacity among all the specimens. The presence of transverse reinforcement (some footing longitudinal bars in the orthogonal direction), as arranged in the specimen No-3, brought a slight improvement. Specimen No-3 showed a shear capacity \( P_{\text{max, No-3}} = 619.3 \) kN, just 1.15 times that of the specimen No-1. However, by just alternating some of the beam longitudinal bars with those of the footing, the specimen No-4 showed a high shear capacity \( P_{\text{max, No-4}} = 855.4 \) kN, 1.58 times that of the specimen No-1. Furthermore, inserting slanted shear reinforcement improved the performance of the specimens. Due to the presence of such reinforcement, the specimen No-2 sustained all the cyclic loading levels better than the specimen No-3. Furthermore, the specimens No-2 and No-5 showed, respectively, slightly high shear capacities \( P_{\text{max, No-2}} = 624.4 \) kN, \( P_{\text{max, No-5}} = 949.5 \) kN, 1.16 times and 1.11 times that of the specimens No-1 and No-4. Eventually, when the reinforcement arrangement included both, the slanted shear reinforcement and the alternated longitudinal reinforcement, as in the specimen No-5, the shear capacity \( P_{\text{max, No-5}} = 949.5 \) kN was, respectively, 1.76 times and 1.52 times that of the specimens No-1 and No-2.

By introducing a lower steel grade for the beam longitudinal reinforcement, the specimen No-6 showed a very stable behavior, in comparison to the specimen No-5. The performance of the specimen was reverted from a non-ductile behavior to a ductile one, where the shear failure was prevented by a primary yielding of the beam longitudinal bars, with a considerable safety margin as to shear failure. Whereas the shear capacity of the specimen No-2 \( P_{\text{max, No-2}} = 624.4 \) kN exceeded slightly the defined tensile yield strength \( P_y \) and that of the specimen No-4 \( P_{\text{max, No-4}} = 855.4 \) kN was 1.38 \( P_y \), the shear capacity of the specimen No-5 \( P_{\text{max, No-5}} = 949.5 \) kN was 1.53 \( P_y \) and 1.35 times the measured tensile yield strength \( P_{\text{my, No-5}} = 700.9 \) kN of the specimen No-6. The calculated tensile yield strength \( P_{\text{my, No-6}} = 703.7 \) kN, based on the actual yield stress of steel bars obtained by tensile test and shown in Table 2, was slightly higher than the measured one \( P_{\text{my, No-6}} = 700.9 \) kN.

Recorded displacement at the concrete upper surface (“CF” location in Fig. 4) related to the displacement of the loading steel beam (“CS” location in Fig. 4), which includes the elongation of the beam longitudinal steel bars, for all the specimens are illustrated in Fig. 6. These displacements were all measured relatively to the upper face of the fixed concrete stub (considered as a reference line). The relationships show that, for the specimens No-1—No-5, the displacement of the loading steel beam increased with an almost constant rate proportionally to the displacement at the concrete upper surface, which was basically due to shear cracking, whereas for the specimen No-6 the rate increased suddenly soon after the beam longitudinal reinforcement reached yielding and, consequently, the displacement at the concrete upper surface had become almost unchanged, letting the deformation to be concentrated in the beam longitudinal reinforcement outside the concrete part.

### 3.2 Damage progression and failure conditions

For a better comprehension of the damage undergone by all the specimens and of their failure mechanisms during...
the test, crack patterns of the specimens are illustrated by the photos and drawings in Fig. 7 at the peak of the loading cycle $P = F_2$ and at maximum strength ($P = P_{\text{max}}$).

Cracks progression during loading was almost similar for all the specimens within the loading range $0 < P \leq F_1$, where only cracks perpendicular to the axis of each

![Graph showing displacement of steel beam and concrete surface](image)

**Fig. 6** Displacement of steel beam – displacement of upper concrete surface relationships.

![Crack evolution images](image)

**Fig. 7** Crack evolution of specimen at the peak of the loading cycle $P = F_2$ and at maximum strength.
specimen occurred below the beam anchor heads (at the level of the upper surface of the fixing concrete stab). Shear cracks were observed in all the specimens at around the peak of the second loading cycle \((P = F_2)\), except for the specimen No-1, where it appeared soon after the perpendicular cracks. Since the loading level \(P = F_2\) in all the specimens, the compressive strut formed clearly, where the crack shape suggested that the applied load was mainly resisted by the compressive strut action between the anchor heads of the longitudinal bars of the beam and footing, inducing a transverse displacement of the footing side-parts away from the beam part. Concomitantly, the transverse shear reinforcement resisted the applied load and provided a clamping action on the developed cracks but when these transverse reinforcements reached their yield level, the cone shape became evident and when some shear cracks between the most adjacent longitudinal bars of the beam and footing joined, the load ceased to increase, except in the specimen No-6 where the cracks did not sufficiently develop to form a cone shape and a clear failure surface.

Alternation of the opposed bars of beams and footings reduced the crack progression, as it can be seen in the compared figures of the specimens No-1 and No-4, at similar loading levels. Except the cracks forming the compressive struts that were described previously, no clear cracks forming compressive struts between the exterior beam bars and their closest footing bars on the footing side-parts were observed. Therefore, some vertical cracks close to the beam exterior bars (beam bars between footing bars on the footing side-parts) that appeared at the loading level \(P = F_2\) seemed to be an origin of a possible failure surface between the beam exterior bars and their closest footing bars on the footing side-parts, suggesting an additional resistance mechanism that would explain the high strengths of the specimens with alternated longitudinal bars.

Slanted shear reinforcement restrained, beyond the loading level \(P = F_2\), the crack progression as it can be seen in the compared figures of the specimens No-1 and No-2 and the specimens No-4 and No-5. By crossing the developed cracks, the tensile forces in the slanted reinforcement resisted the applied load.

Addition of the footing longitudinal reinforcement (D16 bars) in the orthogonal direction, restrained slightly the crack progression, as it can be seen in the compared figures of the specimens No-1 and No-3, but was not very effective as such reinforcement (D16 bars) were not spread along the splicing length and concentrated close to the anchor heads of the beam longitudinal bars. Whereas the resistance of such reinforcement to the applied load might be through the dowel action, the latter would likely be reduced due to the effect of bending at the high loading level of the footing side-parts.

### 3.3 Strains in reinforcement

Measured strains of the longitudinal reinforcement and their distributions on the most interior adjacent longitudinal bars of beams and footings are shown in Fig. 8 at the peak of the loading cycles \(P = F_2\) and \(P = F_3\) and at maximum strength \((P = P_{\text{max}})\). It is worth reminding that the loading level \(P = F_3\) was not reached in the case of the specimens No-1 and No-3, and consequently their corresponding lines are not drawn in the figures. Yielding never occurred on the side of anchor heads in all the bars (SD390 and SD685) of all the specimens.

The presence of the slanted shear reinforcement reduced the strains at the locations close to the anchor heads of the longitudinal bars of the beams and footings and on the longitudinal bars of the footings at the bottom of the specimens, as it can be seen for similar loading levels in the compared figures of the specimens No-1 and No-2 and the specimens No-4 and No-5. Similar observation was made, respectively, when the transverse bars (longitudinal bars of the footing in the orthogonal direction) were used and when alternation of longitudinal reinforcement of beams and footings was implemented, as it can be seen for similar loading levels in the compared figures of the specimens No-1 and No-3 and the specimens No-1 and No-4.

Measured strains of the longitudinal reinforcement at different locations confirmed the expected flow of forces from the longitudinal reinforcement of beams to those of footings and showed that the strength difference between the specimens depended on the configuration of reinforcement. Larger strains developed along longitudinal bars of both beam and footing when their locations were farther from the anchor heads, revealing that bond forces along the longitudinal bars as well as the action of friction and transverse reinforcement participated in transferring forces besides anchor forces. Considering the measured (applied) load at different levels, the material characteristics of the longitudinal bars listed in Table 2 and assuming similar forces occurring at similar locations of similar longitudinal bars, the estimated strain of the footing longitudinal bars at the bottom of the footing (above the stub) when compared to the measured strain on the most interior footing bar for each of the specimens No-1, No-2 and No-3, showed that the measured strain after the load level \(P = F_1\) (after the pull out crack occurred) was higher than the estimated one, suggesting two properties: 1) the load transfer would likely be more significant for the most interior footing bars and gradually decrease for those on the far side of the footing, and 2) the most interior footing bars, in comparison to those on the far side of the footing, would be subjected to additional tensile forces due to a likely bending of the footing side-parts. Similar deduction was made for each of the specimens No-4, No-5 and No-6 after the load level \(P = F_2\).

### 4. Strength capacity evaluation

#### 4.1 Model and assumed conditions

An attempt was made to evaluate the capacity of the tested specimens, whether beam longitudinal bars yield,
or shear failure occurs, based on the observation during the test of the specimens and using the existing theory.

When yielding of beam longitudinal bars is expected before reaching the shear strength capacity, the beam longitudinal bars’ tensile yielding strength \( P_{y,cal} \) can be evaluated using Eq. (1). Therefore, the shear strength capacity \( P_{cal} \) can be evaluated using Eq. (2).

\[
P_{y,cal} = A_{s,b} \cdot f_{y,b}
\]

where \( A_{s,b} \) and \( f_{y,b} \) are, respectively, the beam longitudinal bars’ total cross section (mm\(^2\)) and yield strength (N/mm\(^2\)).

Shear strength of the specimens was investigated using a simple model, considering shear friction along a part of presumed failure surfaces, as well as tensile forces of the slanted bars and dowel bearing forces of the transverse bars crossing the presumed failure surfaces. To that end, failure surfaces were assumed between the spliced longitudinal reinforcement, as shown in Fig. 9. Omitting the deformation level, the axial (compression) and tangential (shear) stresses developing along each failure surface were assumed to be the result of different cumulative actions, as illustrated in Figs. 10 and 11. The first action was the compression stress action of the transverse and slanted shear reinforcements. The second one was the compression stress caused by a component of the simplified strut action between the anchor heads. The third one was the compression stress caused by the bending action of the footing side. The fourth one was the component of the tensile force of the slanted bar action. The fifth one was the dowel bearing action of the transverse reinforcement. The transverse bars of specimen No-3, representing the longitudinal bars of footings in the orthogonal direction, were not included in evaluating the friction, as such reinforcement were concentrated and not distributed along the failure surfaces. It is worth mentioning that the first three actions were assumed to develop only on the non-inclined parts of the failure surfaces, as the inclined parts would, from a certain level, provide a negligible resistance due to cracking. Based on such assumptions, the shear strength \( P_{cal} \) of each specimen would be evaluated using Eq. (2).

\[
P_{cal} = P_f + P_s + P_d
\]

where \( P_f \), \( P_s \) and \( P_d \) are, respectively, the shear strength due to friction, slanted bar tensile force and transverse bar dowel bearing force.

Shear strength \( P_f \) due to friction is evaluated in this study by Eq. (3) in which the shear stresses developing along each failure surface were assumed to be

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**Fig. 8 Strain distribution on the most adjacent beam-footing longitudinal bars of specimens.**
limited by the minimum of the ultimate concrete shear strength ($\tau_u$). Whereas no bond failure was observed during testing, for design purposes, instead of Eq. (3), Eq. (3-1) might be considered as a proposal to limit a likely large plastic penetration of the beam longitudinal bars into the footing part and preventing bond strength degradation along the laterally compressed bars by using Eq (8), which evaluates the ultimate shear bond stresses ($\tau_{u3}$) along steel bars when subjected to compressive stresses. The compressive stresses are assumed to originate from the component of the strut action and from the footing side bending action as well, as described by Eq. (9).

$$P_f = \sum_i b \cdot L_{ci} \cdot \min(\tau_{u1,ci}, \tau_{u2,ci})$$

(3)

$$P_f = \sum_i b \cdot L_{ci} \cdot \min(\tau_{u1,ci}, \tau_{u2,ci}, \tau_{u3,ci})$$

(3-1)

where

- $n$ is the total number of failure surfaces,
- $i$ is the index for the failure surface number,
- $b$ is the width (mm) of the failure surfaces,
- $L_{ci}$ is the actual anchor length (mm) (or the bond + wedge length part) assumed under compression stresses of the failure surface $i$, as given by Eqs. (4) and (5),

$$L_{ci} = L_s - L_d = L_s - (S_{sp,bf} - \phi_\alpha) / \tan \beta$$

(4)

$$L_{ci} = L_s - L_d = L_s - (S_{sp,bf} - \phi_\alpha) / \tan \beta$$

(5)

where $L_s$ is the splice length (mm), counted as the internal length between anchor heads of beam longitudinal bars and anchor heads of footing longitudinal bars, $S_{sp,bf}$ are the spacing (mm) between the beam longitudinal bars and the closest footing longitudinal bars, $\phi_\alpha$ the exterior diameter (mm) of the anchor head, $\beta$ the stress flow angle (deg.) as to the beam axis, assumed here = 60 deg. based on the average value observed on the specimens presented in this paper.

![Assumed failure surfaces](image1)

Fig. 9 Assumed failure surfaces.

![Assumed actions](image2)

Fig. 10 Assumed actions on failure surfaces for specimens No-1 to No-3.
value is close to the value of 55 deg. observed in the test of Thompson et al. (2003b).

\[ \tau_{u1,i}, \tau_{u2,i} \text{ and } \tau_{u3,i} \] concrete ultimate shear strengths evaluated, respectively, as given by Eqs. (6) (AIJ 2002), (7) (AIJ 2002) and (8) (AIJ 1999).

\[ \tau_{ul,i} = 0.3 \cdot F_c \] (6)

\[ \tau_{u2,i} = \mu \cdot (\rho_{as} \cdot f_{sy} + \rho_{ps} \cdot f_{sy} \cdot \sin \theta + \sigma_0) \] (7)

\[ \tau_{u3,i} = 0.7 \cdot (1 + \sigma_0 / F_c) \cdot F_c^{2/3} \] (8)

where

\[ F_c \] is the compressive strength of concrete (N/mm\(^2\)),

\[ \mu \] the friction coefficient along the compressed failure surface. A value of 0.5 was assumed for the calculation.

The experimental work carried by Tassios and Vintzeleou (1987) on specimens with concrete strengths 16, 30 and 40 N/mm\(^2\), showed that the friction coefficient ranged from 0.55 to 0.35 for monotonically loaded specimens with rough interfaces. These values are slightly below the value 0.6 suggested in AIJ (2002) and in ACI (2019).

\[ \rho_{as} \text{ and } \rho_{ps} \] are, respectively, the shear steel ratios of the transverse reinforcement and slanted reinforcement counted as to the compressive failure surface,

\[ f_{sy} \] and \( f_{sy} \) are, respectively, the yield strengths of the transverse and slanted reinforcements,

\[ \theta \] is the angle (deg.) of the slanted reinforcement as to the beam axis,

\[ \sigma_0 \] is a compressive stress (N/mm\(^2\)) along a part of the failure surface length, resulting from two cumulative actions of different origins. The first stress (\( \sigma_{0,i} \)) results from the component of the strut action. The second stress (\( \sigma_{0,b} \)) results from the bending action of the footing side.

Both action components are assumed to hold on the same section of the failure surface. The compressive stresses \( \sigma_0, \sigma_{0,i} \) and \( \sigma_{0,b} \) are, respectively, given by Eqs. (9), (10) and (11). The later equation is derived from the yield bending moment of beams (AIJ 1999).

\[ \sigma_{0} = \sigma_{0,i} + \sigma_{0,b} \] (9)

\[ \sigma_{0,i} = \gamma \cdot \min(A_{s,f} \cdot f_{sy,f} \cdot A_{b} \cdot f_{sy,b}) \cdot \tan \alpha_i / (b \cdot L_a) \] (10)

\[ \sigma_{0,b} = 0.9 \cdot (A_{s,f} \cdot f_{sy,f} \cdot d_f) / L_a / (b \cdot L_a) \] (11)

where \( \gamma \) is a reduction factor for the yield strength of longitudinal reinforcement (≈ 0.75, based on the measured strains close to the anchor heads of the longitudinal reinforcement),

\[ A_{s,f} \text{ and } A_{b} \] are, respectively, footing and beam longitudinal bars’ total cross sections (mm\(^2\)),

\[ f_{sy,f} \text{ and } f_{sy,b} \] are, respectively, footing and beam longitudinal bars’ yield strengths (N/mm\(^2\)),

\[ d_f \] is the lever arm (mm) of the footing longitudinal bars considered by bending, counted from the footing side,

\[ \alpha \] is the simplified strut inclination (deg.) as to the beam axis, as given by Eqs. (12)-(14)

\[ \tan \alpha_i = (S_{sp,b} + S_{sp,b}) / L_a \] (case of No-1 – No-3) (12)

\[ \tan \alpha_i = S_{sp,b} / L_a \] (case of No-4 – No-6) (13)

\[ \tan \alpha_i = S_{sp,b} / L_a \] (case of No-4 – No-6) (14)

Shear strength \( P_s \), due to the slanted bars’ tensile force is evaluated by Eq. (15).

\[ P_s = \rho_{ps} \cdot f_{sy} \cdot \cos \theta \cdot b \cdot L_a \] (15)

Fig. 11 Assumed actions on failure surfaces for specimens No-4 to No-6.
Shear strength $P_d$ due to the transverse reinforcement’s dowel bearing force is evaluated by Eq. (16) (AIJ 2002). It is worth mentioning that the dowel action of the slanted reinforcement is not considered, as their tensile action was fully counted.

$$P_d = 1.65 \cdot A_{ds} \cdot \sqrt{f_y \cdot f_{yd}} \cdot (1 - \phi^2)$$  \hspace{1cm} (16)

where, $A_{ds}$ is the total cross section area ($\text{mm}^2$) of the transverse reinforcement, $f_{yd}$ is the yield strength ($\text{N/mm}^2$) of the transverse reinforcement, $\phi$ is a reduction factor counting for the effect of bending on the dowel action (assumed = 0.9).

4.2 Comparison of calculation and test results

Based on the assumed model and stress conditions, the evaluated strengths of the specimens are listed and compared to the test results in Table 3 and Fig. 12. The figure also shows the strength values when considering only the friction part ($P_f$). As ACI (2019) allows the cumulation of friction and dowel forces that develop on a same failure plan, whereas AIJ (2002) does not, and for a better understanding of the calculated strengths, the different constituent values of the strengths are listed in the table. Whereas the estimated value corresponding to the component of the tensile slanted bars may be considered close to the actual value, the values corresponding to the friction and dowel could not be easily identified.

For the specimen that failed by beam longitudinal bar yielding, the evaluated tensile yield strength was similar to the test value, where the ratio of the maximum test value to the calculation value was 1.05. For the specimens that failed in shear, the evaluated strengths $P_{cal}$ obtained by cumulation of the different components ($P_f$, $P_s$ and $P_d$) were higher than the test maximum values, where the ratio of the maximum test value to the calculation value ranged between 0.77 and 0.93 and the lowest ratios were those of the specimens with slanted reinforcement. The strength values, when only the friction component $P_f$ was considered, resulted in a higher safety than the values of $P_{cal}$ when all components were considered, where the ratio of the maximum test value to the calculation value ranged between 1.02 and 1.21.

Whereas the spacing between the longitudinal reinforcement of footing and beam considered in this study was close to the upper limit of 150 mm prescribed in AIJ (2010, 2018) and ACI-318-19 (ACI 2019), when the spacing becomes enormously larger than this value, there would be a need for larger amount of distributed transverse reinforcement or some reinforcement should be concentrated on the side close to the anchor heads of the footing longitudinal reinforcement. Therefore, for the suggested evaluation to be confirmed for larger spacing, there would be a need for an investigation to decide about a critical value for the spacing when using the proposed arrangement. It is worth mentioning that the 150 mm maximum spacing is added in the regulations because most research available on the lap splicing of modern deformed bars was conducted with reinforcement which was within this spacing (ACI 1972).

5. Conclusions

This paper addressed the issue of reinforcement discontinuity by non-contact splicing of eccentric sets of headed longitudinal bars. The stress flow of such arrangement and its enhancement were investigated experimentally, and an evaluation method was suggested. The study dealt with the pull-out performance of eccentrically spliced longitudinal headed bars with different detailing of transverse reinforcement. The parameters investigated in this preliminary study concerns the effect of slanted shear reinforcement, effect of longitudinal reinforcement of footings in the transverse direction and effect of alternating some of beam longitudinal reinforcement with footing longitudinal reinforcement.

![Fig. 12 Comparison of strength capacities of test and calculation.](image-url)
Test results showed that using only distributed shear reinforcement to tie the eccentrically spliced longitudinal reinforcement, adding some footing longitudinal reinforcement in the orthogonal direction, or adding slanted shear reinforcement, the performance of the eccentric connection was slightly improved, whereas the brittle failure mode was not affected. However, when some of the beam longitudinal bars were alternated with those of the footing, the performance was greatly improved and a ductile behavior could be developed by combining slanted shear reinforcement, proving that the proposed arrangement of eccentric reinforcement would be an adequate solution without increasing the amount of reinforcement in the footing.

The suggested evaluation model, based on the friction shear theory, seems encouraging as it could predict the failure modes and the obtained strength values would be acceptable when only the friction component is considered. This preliminary study is intended as a basis for further investigations on precast foundation beam-footing connections, where more refined evaluation method would be expected.

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