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Seismic Design of Reinforced Concrete Beam-Column Joints with Headed Bars

by Thomas H.-K. Kang, Myoungsu Shin, Nilanjan Mitra, and John F. Bonacci

Section 12.6 provisions of ACI 318-08 detail the development of headed and mechanically anchored deformed bars for the first time in the Code series. Prior to this, Joint ACI-ASCE Committee 352 published design recommendations for headed reinforcement used in reinforced concrete beam-column joints (ACI 352R-02). However, both ACI 318-08 and 352R-02 are based on quite limited experimental research. Given this concern, these ACI standards and recommendations were evaluated using an extensive database encompassing most available test data for reinforced concrete beam-column joints with headed bars subjected to reversed cyclic loading. The primary objectives of this study are to document the experimental investigations in a uniform format; provide a detailed review for the test data; and, finally, propose design guidelines to supplement ACI 352R-02 and 318-08 on the subject of headed bars anchored in beam-column joints.

Keywords: beam-column joints; headed bars; reinforced concrete; seismic.

INTRODUCTION

Headed deformed reinforcing bars (referred to as "headed bars" hereafter) are becoming increasingly popular as longitudinal and transverse reinforcement for relatively large reinforced concrete structures that are exposed to extreme loads such as earthquakes or blasts. The use of headed bars often provides an adequate solution to steel congestion, particularly at beam-column joints, outrigger beam-column connections, or pile-footing connections of heavily reinforced buildings and infrastructures.

A combination of the bond between concrete and steel (along the length of the bar) and head bearing contributes to the anchorage capacity of a headed bar, somewhat like the combination of the bond and bearing of the hook that works for a conventional hooked bar. Previous research[1,2] identified that proportions of the bond and bearing that contribute to the anchorage capacity are approximately equal for headed bars used in beam-column joints subjected to relatively small deformations. The bearing of the head, however, provides a greater portion of resistance as the bond deteriorates due to intensive cracking at the beam-joint (or column-joint) interface as well as inside the joint core.

New code provisions for headed bars have been added to the 2008 edition of ACI 318[3] (Sections 12.6.1 and 12.6.2), which detail the development of headed and mechanically anchored deformed bars in tension. These provisions include requirements for development length, material properties, reinforcing bar size, and net bearing area of the head (A_{brg}), as well as clear concrete cover and bar spacing. Here, the net bearing area A_{brg} is defined as the gross head area A_{head} minus the larger of the obstruction area A_{obs} or the bar area A_b (refer to Fig. 1 and also ACI 318-08,[3] Sections 2.2 and 3.5.9).

The calculated development length l_{dt} in tension for headed bars specified by ACI 318-08[3] was determined based solely on the results of tests conducted at the University of Texas at Austin, as reported by Kang.[4] The l_{dt} equation in ACI 318-08[3] results in the development lengths of approximately 30 and 80% of those required for straight and hooked bars, respectively. Certain limitations (Section 12.6.1) are specified based on the lower bound of the data used for the establishment of l_{dt} (Section R12.6). These include data from tests of headed bars in lap splices,[5] single-headed bars embedded in beams,[6,7] and headed bars subjected to pullout,[8] as reported by Kang.[4] These specimens[5-8] are less prone to steel congestion problems and have a much lower degree of concrete confinement than typical beam-column joints have. Previous data from beam-column joint subassembly tests[1,2,9] have not been included in this data set. As a result, some of the limitations set forth in Section 12.6.1 appear to be overly strict for beam-column joints, particularly with regard to clear bar spacing.

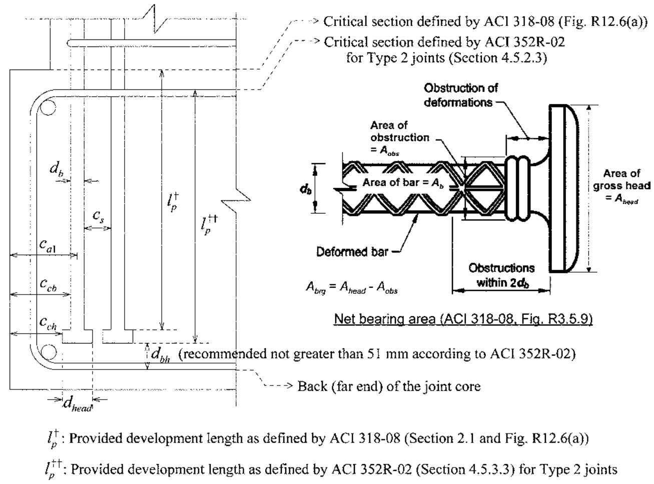

Fig. 1—Defined notation for various dimensions (definitions: refer to Notation section).

This can be a significant limitation, as the beam-column joint region mostly involves the use of headed bars that are needed to reduce reinforcing congestion, and also because typical bar clear spacing in a beam or column ranges from only $1d_b$ to $3d_b$ (as per Sections 7.6 and 12.2.2 of ACI 318-08[3]) versus the limit of $4d_b$ prescribed in Section 12.6.1(f). Currently, no provisions exist in Chapter 21 regarding headed bars (versus hooked or straight bars) used in beam-column joints of structures exposed to low to high seismic hazard. Therefore, for both seismic and nonseismic designs, the overly strict requirements of Section 12.6 must be

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followed. Given this concern and the need to supplement ACI 318-08[3] on the subject of headed bars anchored in beam-column joints, this study assembles a comprehensive data set of experimental studies of beam-column joints with headed bars and assesses the applicability and feasibility of the design criteria required in Section 12.6.1 of ACI 318-08.[3]

As an added objective, Joint ACI-ASCE Committee 352, Joints and Connections in Monolithic Concrete Structures, agreed to update its recommendations related to headed bars (Section 4.5.3 of ACI 352R-02[10]). As part of these efforts, a task group within this committee has compiled most available test data concerning headed bars terminating in reinforced concrete beam-column joint subassemblies. All of these specimens were subjected to considerable inelastic lateral displacement reversals; thus, the review focuses on cases where moderate-to-high seismic risk exists. Because a force mechanism for knee-type joints is quite different from that for other types (for example, interior, exterior, and roof-interior connections), the discussion for a special case of knee joints is not included in this paper (refer to the report by Kang[11]). The primary objectives of this study are to document all these test results in a uniform format and conduct a detailed review of the data to support the updating of the limited ACI 352R-02[10] recommendations as well as ACI 318-08[3] Code provisions.

RESEARCH SIGNIFICANCE

ACI 318-08[3] and other standards do offer practical provisions for the design of beam-column joints with headed bars, although some detailing restrictions set forth in Section 12.6.1 of ACI 318-08[3] (for example, maximum f_c' and minimum clear bar spacing) are not feasible for application to joint design. Relatively few details are given in ACI 352R-02[10] due to a substantial lack of experimental data on beam-column joints involving headed bars, particularly those under inelastic deformation reversals. There are only a few available reports on such seismic tests published in English.[1,2,9,12,13] This research directly addresses the aforementioned concerns by assembling and reviewing most available international data of beam-column joints subjected to reversed cyclic loading.

SUMMARY OF ACI STANDARDS AND RECOMMENDATIONS

Design approaches for headed bars or headed anchors given in ACI standards and recommendations are briefly summarized in this section. Current efforts of ACI and Joint ACI-ASCE committees on these subjects are also introduced.

In 2008, new provisions for headed bars were added in the ACI 318. Sections 12.6.1 and 12.6.2 detail the development of headed bars and the limiting conditions for use of headed bars. ACI 318-08[3] also introduces new provisions (Section 3.5.9) for obstructions or interruptions of the bar deformations, which should not extend more than $2d_b$ from the bearing face of the head. ASTM A970/A970M-07,[14] "Standard Specification for Headed Steel Bars for Concrete Reinforcement," should also be satisfied by the requirements of Section 3.5.9.

While not directly applicable to headed deformed bars, ACI 318-08[3] Appendix D (which adopts recommendations of ACI 349-01[15] Appendix B) and ACI 355.1R-91[16] provide guidelines for the design of plain headed bars and headed anchors, bolts, or studs (headed anchors, hereafter) in concrete. In ACI 318-08[3] Appendix D, the concrete capacity design (CCD) methodology is used to determine the anchorage capacity of headed anchors installed in mass plain concrete. In the CCD method, no bond stress is assumed along the length of a bar, and the concrete is assumed to be unconfined.

Design guidelines for headed bars in beam-column joints were incorporated into the 2002 edition of the ACI 352R[10] report on the basis of both monotonic[8,9,17] (or repeated[9]) and reversed cyclic tests.[1,9] This report recommends the development length for headed bars along with some other specifics such as the location of heads and the amount of head-restraining reinforcement required to prevent prying action of headed bars placed near the concrete-free surface. ACI 352R-02[10] defines two different development lengths of headed bars as functions of $(f_y d_b/\sqrt{f_c'})$ for Type 1 and Type 2 beam-column connections. A Type 2 joint is defined to have sustained strength under deformation reversals into the inelastic range, whereas a Type 1 joint is defined as a joint designed with no consideration of significant inelastic deformation. The critical section for Type 2 joints is defined to be located at the outer edge of joint transverse reinforcement, and at the joint-member interface for Type 1 joints (Fig. 1).

Furthermore, as the concrete bearing capacity is substantially higher in the diagonal compressive strut, ACI 352R-02[10] (Section 4.5.3.2 and Fig. 4.9) recommends that a head be located within 2 in. (51 mm) from the back of the joint core (refer to Fig. 1). For details of the head, ACI 352R-02[10] refers to ASTM A970/A970M-98,[18] where the net bearing area A_{brg} was recommended to be greater than $9A_b$. The current version of ASTM A970/A970M[14] (2007) no longer specifies a minimum A_{brg}.

To provide the state-of-the-art information on headed reinforcement, ACI Committee 408, Development and Splicing of Deformed Bars, and ACI Committee 439, Steel Reinforcement, are jointly preparing a new report on Headed Ends for Anchorage and Development of Reinforcing Bars. In this report, a broad overview of mechanical anchorage and headed bars is provided, including definitions, historical development, and descriptions of various types of headed end devices, as well as previous research and applications. This report refers to ACI 352R-02[10] for the use of headed bars in beam-column joints.

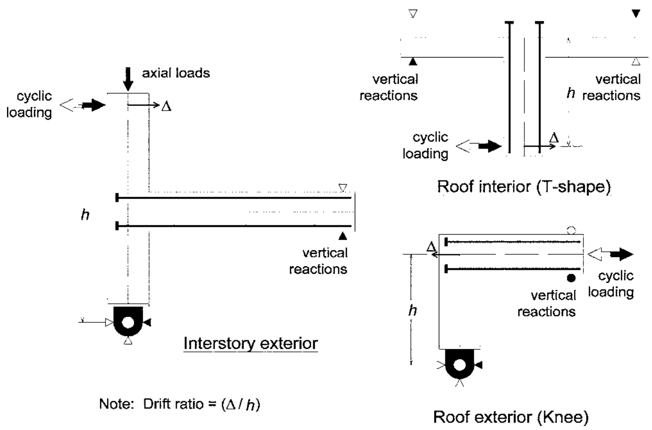


Fig. 2—Schematic diagrams of investigated beam-column joint subassemblies with headed bars.

SUMMARY OF EXPERIMENTAL RESEARCH

In this section, most prior seismic testing programs and test results are summarized, outlining the range of various factors that impact on seismic joint performance.

The dataset assembled in this work includes experimental data from a total of 93 reinforced concrete beam-column connection tests carried out by approximately 22 different groups of investigators around the world (Table A.1 in the Appendix*). The specimens were subjected to quasi-static reversed cycling loading to simulate seismic forces. The tests include 69 interstory exterior connections, 17 (T-shaped) roof-interior connections, and seven knee connections (refer to Fig. 2), all of which rely on member reinforcement terminating in the joint region. Most of the beam-column joint specimens tested by Bashandy[9] (except one) are not included because they do not have beams and were subjected to repeated loading in the same drift direction (not reversed cyclic loading).

For the exterior connections, headed bars were employed for top and bottom beam reinforcement while they were used for the column reinforcement in the roof-interior connections. Most of the subassemblies were planar without any transverse beam and slab; only a small number of the exterior connections included one or two transverse beam(s) framing perpendicular to the main beam into the column (refer to Table A.1 in the Appendix). Two of the exterior connections[19] had one beam at each of the two principal directions of the rectangular-shaped column (that is, a corner column with two orthogonal beams), and they were loaded in a combination of the two directions. Of the exterior connection studies, one[20] investigated the performance of headed bars used in a wide beam-column connection in which some of the headed bars were anchored in a transverse beam outside the joint. One[13] was an eccentric exterior connection. Almost half of the specimens had multiple layers of headed bars in the beam(s) or the column (Table A.1 in the Appendix).

The main test variables included the development length for headed bars, clear cover to headed bars, type of anchoring devices, and head size, as well as the compressive strength of concrete and joint failure mode. The development length provided for headed bars ranged widely from $6d_b$ to $23.7d_b$, when measured from the joint-member interface. In most specimens, the net head bearing area A_{brg} was 2.6 to 8 times the reinforcing bar area A_b. The tested compressive strength of concrete ranged from approximately from 3.5 to 20 ksi (24 to 138 MPa), and it was higher than 10 ksi (69 MPa) in approximately 1/4 of the specimens. The tested clear bar spacing c_s in a layer varied from $1.2d_b$ to $7.6d_b$, which was typically not the design parameter varied among the specimens in each program.

The performance of headed bars used for beams and/or columns, terminated in the joint cores, was investigated for all types of joint failure modes including beam or column hinging, joint shear failure, and bar bond-slip. Other investigated design variables include the amount of beam and/or column bars, the amount of joint transverse reinforcement, the type of reinforcing steel, and the level of column compression. The tested yield strength of steel ranged from 43 to 148 ksi (297 to 1020 MPa), and was higher than 100 ksi (690 MPa) in approximately 1/3 of the specimens. Approximately 1/2 of all specimens were tested with large-diameter headed bars (No. 8 to 11; d_b = 25 to 36 mm). The preapplied column compression varied from 0 to 12% of the column gross section area times the measured concrete compressive strength ($f'_{c,meas}$). Details of the test parameters used in the investigated beam-column connection specimens are documented in Table A.1 in the Appendix.

In the following sections, the primary factors influencing the action of headed bars in beam-column joints will be considered one at a time with respect to trends in the data with an aim toward improving existing recommendations for each.

DISCUSSION OF TEST DATA AND RELEVANCE TO ACI 318-08 AND 352R-02

Failure modes

All the investigated joint subassemblies with headed bars showed inelastic hysteretic behavior with some strength and stiffness degradations under reversed cyclic loading. The tested specimens are categorized into three different groups in terms of failure modes established by the writers as follows: Category I: member flexural hinging followed by modest joint deterioration; Category II: member flexural hinging followed by joint failure; and Category III: joint failure prior to member flexural hinging. The definition of "joint failure" includes cases with joint shear failure, significant bond slip, and a combination of both joint shear distress and bond slip. In this study, Category I specimens are considered to exhibit "satisfactory seismic joint performance," while specimens in the other two categories exhibit "unsatisfactory seismic joint performance." The performance indexes applied by the authors for classification include: 1) the ratio of measured peak moment to nominal moment capacity (M_p/M_n); 2) drift ratio at the point of 20% drop from the peak lateral load ($\delta_{0.8peak}$); 3) ratio of strain in the headed bar at the joint-member interface to yield strain; and 4) joint shear distortion during approximately 3.0% drift cycles, where M_n is estimated following ACI 318-08[3] procedures. It is noted that the framework is used to organize data for analysis and that the main findings are not overly sensitive to the numerical benchmarks.

Joint failure was assumed to occur prior to flexural hinging (Category III) if the ratio of (M_p/M_n) was less than 1.0 and no bar yielding was monitored by strain gauges. The headed bars used for five specimens (No. 6 to 10; Kiyohara et al.[21]) yielded at relatively large drift levels (approximately 4%); however, M_p did not exceed M_n and the specimens exhibited a reduction in lateral stiffness. Thus, these specimens

*The Appendix is available at **www.concrete.org** in PDF format as an addendum to the published paper. It is also available in hard copy from ACI headquarters for a fee equal to the cost of reproduction plus handling at the time of the request.

are included in Category III. The rest of the Category III specimens (refer to Table A.1 in the Appendix) experienced a significant drop in strength after reaching the peak load values corresponding to M_p less than M_n. A variety of design parameters appeared to cause the poor joint behavior, including relatively large joint shear demand (4 of 15; refer to Columns (23) and (24) of Table A.1 in the Appendix) combined with a substantial lack of confinement by joint transverse reinforcement (15 of 15; refer to Column (25) of Table A.1 in the Appendix), or a lack of headed bar embedment length (8 of 15; refer to Columns (8) to (11) of Table A.1 in the Appendix). The specimens with the shorter embedment lengths are discussed in detail later in this paper.

The drift ratio and the joint shear distortion were used to differentiate between Category I and Category II. If the specimen exhibited less than a 20% reduction in strength until 3.5% drift, and did not exceed 1.2% of joint shear distortion (if reported) until 3.5% drift cycles, the joint was considered to have exhibited satisfactory seismic performance and was classified as Category I. These benchmark values (or similar values) have been used or accepted by ACI Committee 374[22] (ACI 374.1-05, Section 9.1.3) and by one of the authors[2,12] or other investigators.[23,24] For the specimens in Category I, the displacement ductility μ was always larger than 2 (group average equals 5.5), where μ is defined as ($\delta_{0.8peak}/\delta_y$). For the specimens in Category II, joint failures occurred also after flexural yielding, but with much less ductility (group average equals 2.6).

Based on the slip measurements (if reported), bar slips were limited to less than 0.04 in. (1 mm) at 2% drift ratio for the specimens[2,21,25-27] in Category I except three[28] (0.08 to 0.12 in. [2 to 3 mm]). Further, the performance of the specimens[2,12,13] evaluated based on the ACI 374.1-05[22] criteria, including pinching indexes, was acceptable; thus, the large drift ratios recorded from these Category I specimens were not due to excessive bar slips and associated stiffness reduction. The contribution of bond slips to the drift ratio could not be small, but it would not affect the overall effectiveness of the head anchorage.

The joint failures (Category II) resulted mainly from substantial joint shear distortion, along with moderate bond deterioration of reinforcement within the joint. Even after bond deterioration, head bearing resistance was maintained with a relatively small loss. Some degree of pinching (bond slips) is common for reinforced concrete beam-column joints that are part of moment frames when subjected to cyclic loading, and it is tolerable. The bar stress determined from measured strain just in front of the head dropped by only 0 to 30% (average $\Delta\sigma_h/\Delta\sigma_{h\_max}$ = 10%) of the peak value for specimens in all failure mode categories for which bar strain data were reported (refer to Column (30) of Table A.1 in the Appendix). This indicates that for these specimens, anchorage (that is, bearing) of the headed bar was not significantly deteriorated throughout the test.

Ten of the 27 Category II specimens appeared to be adversely affected by a lack of proper bond development (P1 to P4,[20] No. 5,[21] AH8-2-45,[29] 2S-2, 2S-0 and WN-ST,[30] and SN-U[31]). This is based on the reported observations[20,21,29-31] and the data exhibiting an abrupt increase in bar slip,[21] or a sudden drop in lateral load with little residual strength.[29] Also, modest joint shear distortion[30,31] (refer to Column (28) of Table A.1 in the Appendix) or sufficient joint confinement[20] (refer to Column (25) of Table A.1 in the Appendix) indirectly indicates that bond deterioration was rather a major cause of poor performance of these connections. These specimens are discussed in detail in each of the following subsections.

Development length for headed bars in beam-column joints under cyclic loads

Development length l_{dt} equations for headed bars in both ACI 318-08[3] and 352R-02[10] are functions of ($f_y d_b/\sqrt{f'_c}$) (refer to ACI 318-08,[3] Section 12.6.2 and ACI 352R-02,[10] Section 4.5.3). The difference is only in the constants. Certain limitations such as clear concrete cover, bar spacing, and head size should be applied to ensure adequate development and anchorage of a headed bar (refer to Sections 12.6.1 and R12.6). From all of the tests for which the relevant data were reported, it was observed by the authors that the portion of bond contribution to the anchorage capacity of headed bars was large at small drift levels. As the bond deteriorated with increasing drifts, however, the head bearing played a significant role in anchorage during the inelastic stage (after approximately 2.5 to 4% drift). This implies that both bond along the development length and head bearing ultimately contributed to surviving substantial lateral drifts and achieving satisfactory joint behavior, but by different extents at various drift levels. In light of this behavior, it appears reasonable either that a term associated with head bearing be included in the development length equation as a parameter or that certain requirements relating to head bearing are specified to achieve the desired connection behavior in the inelastic range of the deformation.

Figure 3 illustrates the provided development length for headed bars used in the investigated Category I specimens and Category II and III specimens that were affected by improper bond development, compared with the values required by ACI 318-08[3] and 352R-02.[10] The ACI 318-08[3] equation resulted in conservative estimations for the Category I specimens, which exhibited satisfactory seismic performance. On the other hand, most of the Category I (37 of 44) specimens are located on the left side of the ACI 352R line, indicating that the ACI 352R equation corresponds quite well with the data. These findings apply to both Category I specimens with a single layer of headed bars and those with multiple layers.

Figure 3 also illustrates that most Category II and III specimens that were affected by improper bond development did not satisfy either ACI 318-08 provisions or 352R-02 recommendations for development length, indicating that a designer has a proper tool to rule out these bond-slip failures. For the specimens that exhibited premature failures (Category II and III) despite complying with the ACI 352R-02[10] development requirement (refer to Columns (10) and (11) of Table A.1 in the Appendix),[21,25,28,29,32-34] the primary failure mode was joint shear failure (not shown in Fig. 3 for clarity). This is based on the reported observations and the data in Table A.1 in the Appendix. In conclusion, an examination of the experimental data indicates that the ACI 318-08[3] equation gives somewhat conservative l_{dt} requirements for headed bars in joints, and that the current ACI 352R[10] recommendation for development length l_{dt} appears reasonable for both single and multiple layers of headed bars anchored in the Type 2 beam-column joint. Thus, an amendment of the ACI 352 defined l_{dt} for the Type 2 joint is not recommended until sufficiently detailed investigations are carried out.

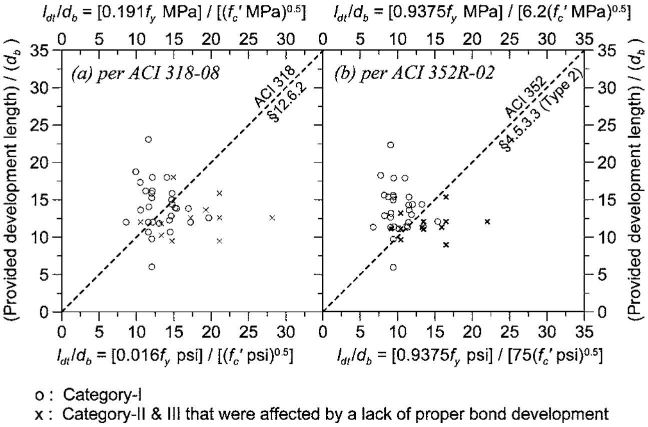


Fig. 3—Comparison between provided and required development lengths.

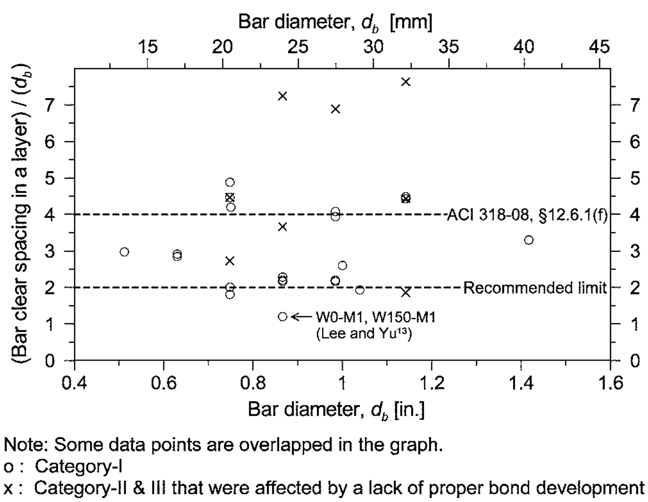


Fig. 4—Bar clear spacing in a layer versus bar diameter.

Headed bar clear spacing

ACI 318-08,[3] Section 12.6.1(f), specifies that the minimum clear spacing between headed bars should be $4d_b$. This specified minimum clear spacing c_s is significantly larger than the value required for beam reinforcement (= $1d_b$ per Section 7.6.1) or column reinforcement (= $1.5d_b$ per Section 7.6.3) terminating without heads, and even larger than those having been used in practice for both hooked and headed bars ($1d_b$ to $3d_b$). According to Section R12.6 of ACI 318-08[3] and the report by Kang,[4] the minimum limit of $4d_b$ was determined based on the lower bound values obtained from 10 lap splice tests[5] and two pullout tests.[8] Given that the minimum spacing c_s of $4d_b$ has been developed based on limited data (which have not involved beam-column joints), it is suggested that the requirement be reviewed through further research on additional experimental investigations.

The ACI 352R-02[10] recommendations do not provide guidelines for clear spacing between headed bars in a layer; therefore, the clear bar spacing specified for conventional reinforcing bars would also be used for headed bars as per ACI 318-08, Sections 7.6.1 and 12.2.2, where the bond capacity is known to be affected by the clear bar spacing, when less than $2d_b$. For the clear spacing not less than $2d_b$, bond may not be a serious issue for all types of bars (hooked, headed, or straight). The clear bar spacing between headed bars may affect the concrete breakout capacity "near the head." Figure 4 ('x' marks) illustrates that there is no apparent relationship between the clear bar spacing and seismic bond performance.

For the 44 specimens falling under Category I, the clear bar spacing was less than $4d_b$ in 33 specimens and close to or even less than $2d_b$ in nine specimens, with the average and the lowest spacing being $2.8d_b$ and $1.2d_b$, respectively (refer to Fig. 4). For the Category I specimen tests, there were no apparent anchorage splitting cracks, side-face blowout failure, or concrete breakout failure; and there were no data providing evidence that bearing or pullout failure occurred. Moreover, these kinds of failures were not reported from the tests of Category II and Category III specimens with clear bar spacing less than $4d_b$. Further, it is shown that the small clear bar spacing did not adversely affect the drift ratio measured at a drop to 80% of the peak lateral load (refer to Columns (14) and (27) of Table A.1 in the Appendix), which is considered as one of the seismic performance indicators in this study. Therefore, it is concluded that there was no influence of the clear bar spacing, if not less than $2d_b$, on the lateral force resistance of the tested beam-column connections.

Based on these observations of the available experimental database, a recommended limit of $2d_b$ is proposed to be used for the design of beam-column joints instead of the current limit of $4d_b$. The Category I data[26,35,36] that met the ACI 318 requirements for l_{dt}, (A_{brg}/A_b), and clear cover to the bar (c_{sb}) can be considered to lower the minimum clear spacing c_s to $2d_b$ even in general (ACI 318-08, Section12.6.1). The proposal was made during public discussion period on ACI 318-08.[3] ACI Committee 318 responded that the clear bar spacing issue would need to be addressed as new business in the next code cycle and the committee encouraged future research. For headed bars with (A_{brg}/A_b) greater than 8 (circular head) or 10.5 (square head) and without any obstructions, the clear bar spacing of $2d_b$ causes overlapping of the heads. In this case, it is recommended to stagger the heads (Section R12.6) as required to fit all bars within available width and maintaining a clear distance between heads.

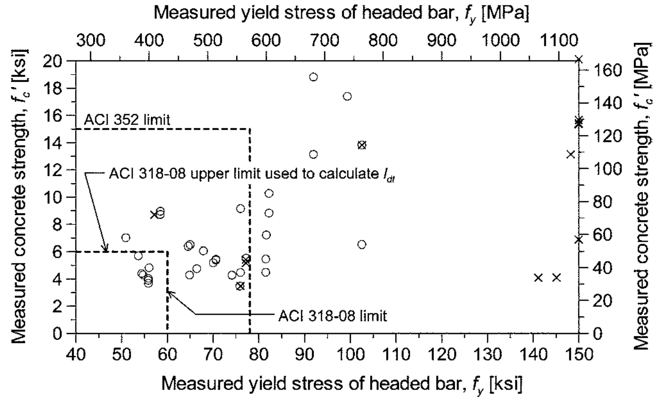
Limitation for concrete and reinforcing bars

According to Sections 12.6.1 and 12.6.2 of ACI 318-08,[3] the specified yield strength of a headed bar (f_y) and the value of f_c' used to calculate l_{dt} should be limited to 60 and 6 ksi (420 and 41 MPa), respectively. On the other hand, the ACI 352R-02[10] recommendations are valid for f_y up to 78 ksi (540 MPa) per ASTM A970/A970M,[14,18] and for f_c' up to 15 ksi (100 MPa). For the Category I specimens, the measured yield strength of steel varied from 51 to 103 ksi (352 to 710 MPa) (Fig. 5). In particular, 16 of these specimens had high-strength steel with f_y higher than 78 ksi (420 MPa). For concrete, the measured compressive strength on the testing day ranged from 3.7 to 18.8 ksi (25.4 to 130 MPa). These test results of satisfactory joint behavior (Category I) support the ACI 352R-02[10] recommendations that the use of high-strength steel having f_y up to 78 ksi (420 MPa) and high-strength concrete having f_c' up to 15 ksi (100 MPa) be permitted for both Type 1 and 2 joints with headed bars.

Only one[31] of 44 specimens with $f'_{c,meas} \geq 6$ ksi (41.4 MPa) experienced a substantial bond slip, although they met the ACI 318 requirement for l_{dt} (Table A.1 in the Appendix and Fig. 3(a)), possibly due in part to the lack of confinement (refer to Column (22) of Table A.1 in the Appendix). In general, the use of f_c' values of 6 to 15 ksi (41.4 to 103 MPa) for l_{dt}

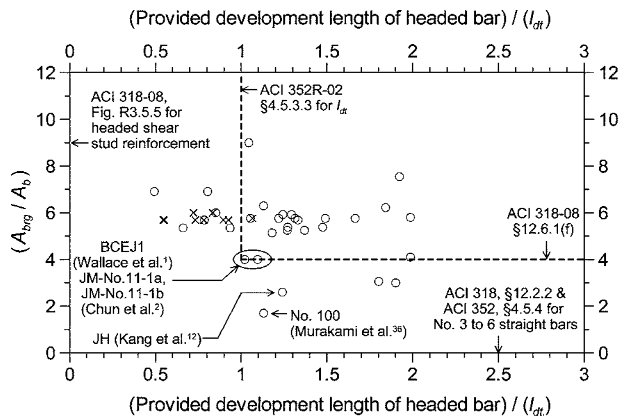
appears warranted. It is additionally noted that the performance was unsatisfactory for all beam-column joints with very high-strength steel (f_y > 140 ksi [815 MPa]), likely due to shorter development lengths than those required by ACI 318-08 (or 352R-02) (Fig. 5 and Table A.1 in the Appendix). The joint design may not be feasible (very large dimension) if the development length requirement would be satisfied. Future research efforts will be needed to study the use of high-strength materials in beam-column joints.

In both ACI 318-08[3] and 352R-02,[10] the maximum allowable size of the headed bar is a No. 11 (d_b = 1.41 in. [36 mm]), and only normalweight concrete is permitted when headed bars are employed as reinforcement (Section 12.6.1(c)). On the contrary, ASTM A970/A970M[14,18] allows the use of No. 14 (d_b = 1.69 in. [43 mm]) and No. 18 (d_b = 2.26 in. [57 mm]) headed bars, which are common for bridges and nuclear plants. From surveying the considered test database, the maximum headed bar diameter used for the Category I specimens was 1.41 in. (36 mm) (Fig. 4), which remained within that specified by ACI 318-08[3] and 352R-02.[10] There were no available data for lightweight concrete, that is, normalweight concrete was used for all specimens. Due to this lack of data, no recommendations to allow or limit use of a larger bar size and lightweight concrete are possible from this study.



Note: Some data points are overlapped in the graph.
o : Category-I
x : Category-II & III that were affected by a lack of proper bond development

Fig. 5—Tested concrete and steel properties.



Note: i) Provided development length & l_{dt}; measured from the outer edge of joint transverse reinforcement to the outside edge of the head
ii) Some data points are overlapped in the graph.
o : Category-I
x : Category-II & III that were affected by a lack of proper bond development

Fig. 6—Head size versus provided development length.

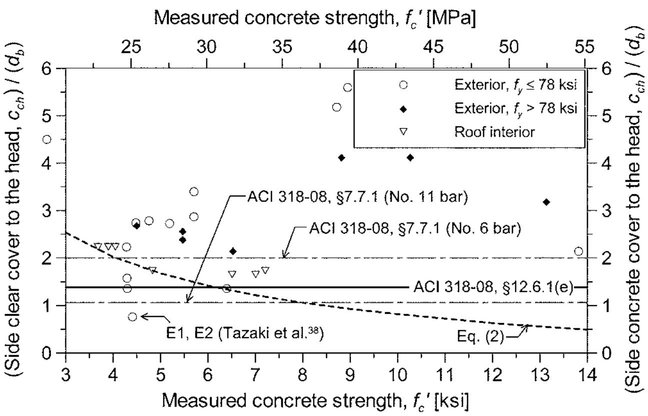
Head size

As discussed previously, both the development length and head size determine the anchorage capacity of a headed bar. After considerable bond deterioration (at approximately 2.5 to 4% drift), anchorage relies in large part on the head bearing acting against the concrete. Therefore, the head size should be large enough to ensure that no pullout (due to local crushing) eventually occurs at the face of the head during this stage. The larger head size, however, does not necessarily warrant a shorter development length needed to ensure adequate bond behavior at low-to-moderate drift levels (up to 2.5%) (refer to 'x' marks in Fig. 6).

ACI 352R-02[10] recommends A_{brg} be at least $9A_b$ by referring to the 1998 version of ASTM A970/A970M,[18] whereas ACI 318-08[3] requires A_{brg} be at least $4A_b$. Two different types of heads were used in the investigated tests: 1) heads without a sleeve connection, and 2) heads with a sleeve connection. In the first case, A_{brg} is calculated as the difference between A_{head} and A_b, while in the second case, A_{brg} may be taken as a smaller value equal to A_{head} minus A_{obs} (refer to Fig. 1 for definitions). In this paper, as the information on A_{obs} is not available for a vast majority of the tests in the literature, A_{brg} is approximately taken as A_{head} minus A_b, conservatively (leading to slight overestimation of the head size associated with a given performance). The value of A_{obs} is often difficult to obtain, as the obstruction is not always circular in cross section and this information may not be provided by the manufacturer; however, in a design case, ignoring the obstruction is not recommended by ACI 318-08[3] (Section R3.5.9).

The specimens falling in Category I reached more than 3.5% drifts with modest strength degradation (less than 20% of the maximum strength). Eight of the 44 Category I specimens possessed A_{brg} equal to or smaller than $4A_b$, with the lowest value of $1.7A_b$ (Fig. 6). Three[1,2] of these eight specimens had a combination of a small A_{brg} (not greater than $4A_b$) and a small development length ($l_p \approx l_{dt}$), where l_{dt} is the ACI 352R recommended development length of a headed bar in a Type 2 joint. Also, two[12,36] of the eight specimens had A_{brg} values of $1.7A_b$ and $2.6A_b$ with relatively small l_p values (refer to Fig. 6). For these five specimens, no signs of concrete breakout or pullout were identified (as reported in the literature).

Strain measurements also indicated that the reduction in bearing resistance was modest (average $\Delta\sigma_h/\Delta\sigma_{h\_max}$ = 7%) for Category I specimens whose data are available (refer to Column (30) of Table A.1 in the Appendix). Further, head bearing drop began to occur beyond 3% drift ratio except one (No. 3[26] at 2% drift). For the Category II and Category III specimens with shorter development lengths than required, however, bond anchorage performance was not satisfactory despite the large head size ($A_{brg}/A_b \geq 5.7$) (Fig. 6, and Column (30) of Table A.1 in the Appendix). Based on the results in this subsection, a minimum head size of (A_{brg}/A_b = 4) is feasible for headed bars terminating in beam-column joints, provided that the development length of the bar complies with ACI 352R-02.[10] Perhaps, a size of (A_{brg}/A_b = 3) will even be allowed for the seismic design of beam-column joints (pending the outcome of the Joint ACI-ASCE Committee 352's deliberations). It is noted that ACI 318-08, Chapter 12 does not consider seismic loading; thus, the findings are of value in updating ACI 352R-02 and ACI 318-08, Chapter 21.



Note: i) Data with c_{ch} smaller than that given by §12.6.1(e) are 0-1, 0-3, 0-4 (Tekeuchi et al.[33]) and E1, E2 Tazaki et al.[38]).
ii) Some data points are overlapped in the graph.

Fig. 7—Side clear cover to the head for Category I specimens.

Table 1—Comparison between Section 12.6.1(e) and 7.7.1(c) of ACI 318-08

Bar size	d_b, in. (mm)	A_b, in.^{2} (mm^2)	c_{cb}, in. (mm)	$c_{ch}^{\dagger}$, in. (mm)	$c_{ch}^{\dagger\dagger}$, in. (mm)
No. 6	0.75 (19)	0.44 (284)	1.50 (38)	1.04 (26)	1.50 (38)
No. 7	0.88 (22)	0.60 (387)	1.75 (44)	1.22 (31)	1.50 (38)
No. 8	1.00 (25)	0.79 (510)	2.00 (51)	1.38 (35)	1.50 (38)
No. 9	1.13 (29)	1.00 (645)	2.26 (57)	1.56 (40)	1.50 (38)
No. 10	1.27 (32)	1.27 (819)	2.54 (65)	1.76 (45)	1.50 (38)

Note: c_{cb} is clear cover to the headed bar per Section 12.6.1(e); $c_{ch}^{\dagger}$ is clear cover to outermost part of the head per Section 12.6.1(e), for circular heads with $(A_{brg}/A_b) = 4$; $c_{ch}^{\dagger\dagger}$ is clear cover to the outermost part of the head per Section 7.7.1(c).

In U.S. design practice, beam-column joint dimensions are typically determined based on joint shear requirements rather than those for bar development length (unless high-strength steel is used). In addition, the headed bar is recommended to extend to the far face of the joint core (Section R12.6 of ACI 318-08[3] or Section 4.5.3.3 of ACI 352R-02[10]; refer to Fig. 1). Thus, embedment lengths in the joints are typically approximately 1.5 to $2l_{dt}$, which leads to less bearing demand placed on the head (then, head size may not be a significant concern for beam-column joints).

Side clear concrete cover

The minimum clear cover required by Section 7.7 of ACI 318-08[3] is primarily intended to protect reinforcement against extreme weather and/or fire. Following Sections 7.7.1(c) and R7.7, the clear cover to the outermost part of the head (c_{ch}) should not be less than 1.5 in. (38 mm) (refer to Fig. 1). At the same time, for the purpose of preventing side-face blowout, ACI 318-08,[3] Section 12.6.1(e), sets a lower limit for the clear cover to the headed bar (c_{cb}) as $2d_b$ (refer to Table 1). Both requirements of Sections 12.6.1(e) and 7.7.1(c) are, in general, not difficult to meet for headed bars anchored within a beam-column joint, if adequate clear cover is also provided for the joint transverse reinforcement based on Section 7.7.

ACI 352R-02[10] does not provide explicit recommendations for minimum clear cover to the head. Rather, ACI 352R-02[10] specifies the minimum amount of restraining reinforcement engaging the headed bar just before the head which is needed to produce the strength of $0.25A_sf_y$ for a Type 1 joint or $0.5A_sf_y$ for a Type 2 joint, where A_s is the headed bar area near the free surface. This head-restraining reinforcement should be provided for all headed bars adjacent to a free face of the joint, such as beam bars in a joint having any free vertical face (for example, interstory corner joint), and top beam bars in an exterior joint with a discontinuous column. When c_{cb} is greater than $3d_b$, ACI 352R-02[10] allows reducing the amount ($A_{s,rst}$) of the head-restraining reinforcement and recommends that the reduced amount be estimated by considering the resisting force R_{lb} to lateral bursting according to ACI 349-97[37] as

$$R_{lb} = 4\sqrt{f_c'}\ (\text{psi})\,(c_{a1})^2\pi = A_{s,rst}f_y; \quad (1)$$

$$R_{lb} = 0.33\sqrt{f_c'}\ (\text{MPa})\,(c_{a1})^2\pi = A_{s,rst}f_y;$$

where c_{a1} is equal to $(c_{cb} + 0.5d_b)$ or $(c_{ch} + 0.5d_{head})$ (refer to Fig. 1). Note that the resisting force R_{lb} equation is no longer available in the 2001 version of ACI 349.[15] The head-restraining reinforcement, however, is not required in any case by ACI 318-08.[3] This is based on the results of the tests[5-7] showing that reinforcement placed transverse to the headed bar did not increase the anchorage strength.

Figure 7 depicts the tested range of side clear cover to the head (c_{ch}) for the Category I specimens, along with comparisons to Sections 7.7.1 and 12.6.1(e), and Eq. (D-17) of ACI 318-08.[3] The side clear cover c_{ch} required to prevent side-face blowout can be estimated by setting Eq. (D-17) of ACI 318-08[3] equal to the maximum bar force of $1.25A_sf_y$ and solving for c_{a1} ($= c_{cb} + 0.5d_b$; refer to Fig. 1) as

$$N_{sb} = 160c_{a1}\sqrt{A_{brg}}\,\lambda\sqrt{f_c'}\ (\text{psi}) = 1.25A_sf_y; \quad (2)$$

$$N_{sb} = 13.33c_{a1}\sqrt{A_{brg}}\,\lambda\sqrt{f_c'}\ (\text{MPa}) = 1.25A_sf_y$$

where N_{sb} is the nominal side-face blowout strength of headed anchors, λ is assigned as 1 for normalweight concrete, and A_{brg} is set equal to a lower-bound value of $4A_b$ for Eq. (2). The previous equation was developed for cases where the concrete is unconfined; thus, it is conservative to apply it to well-confined beam-column joints.

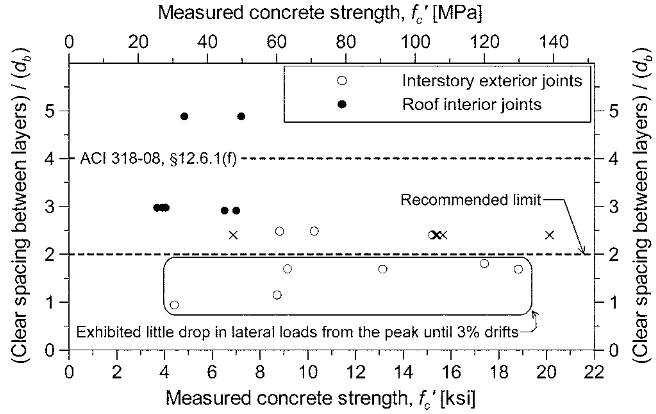
In five of the Category I specimens, c_{ch} was smaller than the values given by Section12.6.1(e), as shown in Fig. 7. For all the interstory exterior joints including those five, no horizontal head-restraining reinforcement was provided, as it was not a common practice. Side-face blowout or spalling of the side clear cover, however, was not observed in any of the Category I joints. The absence of side-face blowout failures was also supported by strain data measured in joint hoops.[1,34] The hoop strains were below 2500 μs until the drift exceeded 3%, indicating that the satisfactory behavior was attributed in part to good lateral confinement of the joint core near the head. The head was located entirely (for exterior joints) or partly (for roof-interior joints) within the core. Furthermore, the joints classified as Category II or Category III did not experience side-face blowout, nor did the joints with closely spaced beam bars adjacent to free vertical faces of the joint.[2,28,29,35,38,39] Only three roof-interior joints[30] in Category III experienced moderate spalling of the concrete cover on the free face of the column, likely due to significant interfacial debonding; however, there was no apparent side-face blowout failure.

Based on the results showing that side-face blowout is not a concern, the requirement of Section 12.6.1(e) of ACI 318-08[3] can also be applied for headed bars terminating in beam-column joints. Furthermore, a design recommendation is proposed such that horizontal head-restraining reinforcement is not required for headed beam bars adjacent to a free vertical face of an interstory joint, provided that the requirement of Section 12.6.1(e) of ACI 318-08[3] is met, and that the lateral confinement is supplied by closed joint hoops and by at least one beam member covering at least three quarters of the column width.

Multiple layers of headed bars

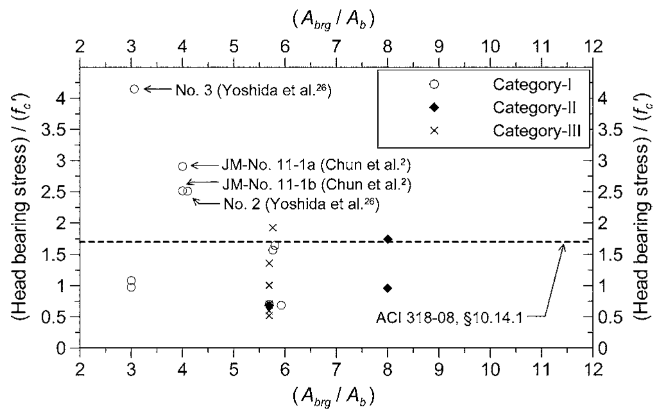
Of the 93 beam-column joint subassemblies, a total of 22 interstory exterior specimens had two layers of top beam bars, with clear spacing between the layers ranging from $0.9d_b$ to $2.5d_b$ (Fig. 8). A total of 17 roof-level interior specimens had multiple column headed bars adjacent to free vertical faces of the columns, with clear bar spacing ranging between $2.9d_b$ and $7.6d_b$ (Fig. 8). As discussed, none of these specimens exhibited side-face blowout. The clear spacing c_l between the layers was smaller than $2d_b$ for nine specimens (JM-2[2]; AH12-8-series and AH8-6-45[29]; No. 1 and No. 2[35]; E1 and E2[38]; J1 and J2[39]) in Category I, and even smaller than $1d_b$ for two specimens (E1 and E2[38]). These specimens exhibited little drop in lateral loads from the peak value until 3% drift ratios (refer to Fig. 8). For the specimens in Category I with the strain gauge data available,[2,25,39] it was reported that head bearing resistance was maintained with minimal loss (average $\Delta\sigma_h/\sigma_{h\_max}$ = 10%; refer to Column (30) of Table A.1 in the Appendix). On the other hand, the specimens of Category II and III with multiple layers of headed bars failed in joint shear (non-bond failure), if they had an embedment length greater than required by ACI 318-08 and 352R-02.

Maximum head bearing stress ($p_{brg} = A_b n f_{y,meas}/A_{brg}$) was estimated to be $4.1f_c'$ for the 11 Category I specimens for which bar strain data were monitored (Fig. 9), where A_b is the cross-sectional area of a headed bar, $f_{y,meas}$ is the measured yield stress of steel, and n is the ratio (maximum of 1.0) of the maximum strain measured in the bar just before the head to the yield strain ε_y of the headed bar. The estimated bearing stresses were quite higher than that permitted ($1.7f_c'$) by ACI 318-08, Section 10.14.1, but close to those ($2f_c'$ to $5f_c'$) monitored during CCT node tests[6] with (A_{brg}/A_b) ranging from 3 to 5. The higher concrete bearing stress in front of the closely spaced heads can be attributed to well-confined concrete (refer to Column (25) of Table A.1 in the Appendix) as well as compressive strut action. Based on the results from nine Category I specimens with c_l less than $2d_b$ and a higher bearing stress capacity of the joint core than the unconfined concrete, multiple layers of headed bars are suggested to be allowed with a minimum clear spacing of $2d_b$ between the layers.



Note: Some data points are overlapped in the graph.
o and ●: Category-I
x : Category-II & III that were affected by a lack of proper bond development

Fig. 8—Beam-column joints with multiple layers of headed bars.



Note: Some data points are overlapped in the graph.

Fig. 9—Head bearing stress versus net bearing area for all categories.

SUMMARY AND RECOMMENDATIONS

A detailed review of previous research on the use of headed bars in reinforced concrete beam-column joints subjected to quasi-static reversed cyclic loading is presented in this paper. The investigated database comprises most available experimental tests on this subject around the world, including those conducted in the U.S., Korea, Japan, and Taiwan (Table A.1 in the Appendix). The test database was assessed to evaluate the new ACI 318-08, Section 12.6, requirements for applications in beam-column joints and to supplement the current ACI 352R-02 report. The recommendations and conclusions that can be drawn based on this review include the following:

1. The development length l_{dt} for headed bars in beam-column joints that ACI 352R-02 recommends corresponds to the experimental data, while the l_{dt} specified by ACI 318-08 is relatively much more conservative for headed bars in beam-column joints. Therefore, the equation of ACI 352R-02 can be included in Section 21.7.5 of ACI 318-08.

2. Test results indicate that bond along the length of the headed bar and bearing of the head both contributed to the anchorage capacity, but by different extents at various drift levels. Thus, either minimum head size should be specified to ensure the desired nonlinear joint behavior, or a term associated with head bearing may be considered in the development length equation for further detailed investigation.

3. Based on the review of the previous data, the net bearing area of a head is suggested to be at least three times the bar area for the design of beam-column joints. The data of beam-column joints subject to cyclic loading provide a means to update both ACI 352R-02 and ACI 318-08, Chapter 21.

4. For the beam-column joint design, the minimum clear spacing between headed bars can be reduced to $2d_b$ from

$4d_b$, which is required by ACI 318-08. This is based on the observation of no influence of headed bar clear spacing (c_s) on the lateral resistance of the beam-column joint, with c_s ranging from $1.2d_b$ to $7.6d_b$. The data from beam-column joints that performed satisfactorily and met the ACI 318 requirements for l_{dt}, (A_{brg}/A_b) and c_s can be considered to lower the minimum clear spacing c_s to $2d_b$ even in general (ACI 318-08, Section 12.6.1). Also, multiple layers of headed bars can be used for beam-column joints with a minimum clear spacing of $2d_b$ between the layers.

5. The test results are consistent with the ACI 352R-02 limitations on f_c' (up to 15,000 psi [100 MPa]) and f_y (up to 78 ksi [540 MPa]). Based on the review of the results, the ACI 318 limits of f_y ($\leq$ 60 ksi [410 MPa]) and the value of f_c' ($\leq$ 6000 psi [41 MPa]) used to calculate l_{dt} could also be expanded.

6. The ACI 318-08 requirements of the minimum side clear covers to the head (c_{ch} = 1.5 in. [38 mm]) and to the bar ($c_{cb} = 2d_b$) can be applied to headed bars used in beam-column joints. There were no side-face blowout failures observed in any groups of the investigated specimens with a minimum c_{cb} of $1.5d_b$.

7. The horizontal head-restraining reinforcement is not necessary for headed beam bars adjacent to a free vertical face of an interstory joint, provided that the aforementioned clear cover requirements are met, and that the lateral confinement is supplied by closed hoops within the joint and by at least one beam member covering at least 3/4 of the column width.

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NOTATION

A_b = bar area
A_{brg} = net bearing area of head
A_g = gross cross-sectional area of column
A_{head} = gross area of head
A_{obs} = cross-sectional area of obstruction
A_{sh} = area of joint transverse reinforcement in principal direction within hoop spacing s_h
$A_{s,rst}$ = amount of head-restraining reinforcement
b_j = effective joint width (ACI 352R-02)
c_{a1} = distance from center of headed bar to edge of concrete
c_{cb} = clear cover to bar
c_{ch} = clear cover to outermost part of head
c_l = clear layer spacing for multiple layers of headed bars
c_s = clear bar spacing in a layer
d = effective member depth
d_b = bar diameter
d_{bh} = distance measured from far end of joint core to back of head
d_{head} = head diameter
f_c' = specified concrete compressive strength
$f_{c,meas}'$ = measured concrete compressive strength
f_y = specified yield stress of headed bars
$f_{y,meas}$ = measured yield stress of headed bars
h = joint depth (refer to Fig. R21.7.4 of ACI 318-08)
h'' = joint core width
l_{dt} = development length required for headed bar
l_p = development length provided
M_n = nominal moment capacity calculated using specified tension force and assumed internal level arm of $0.9d$
M_p = measured peak moment of member
N_{sb} = nominal side-face blowout strength
n = ratio of maximum strain measured in bar just before head to ε_y
P = applied column axial force
p_{brg} = estimated maximum head bearing stress
R_{lb} = resisting force to lateral bursting
s_h = joint hoop spacing
V_n = nominal joint shear capacity calculated based on ACI 352R-02
V_p = maximum joint shear demand applied during testing
$\Delta\sigma_h$ = drop in bar stress from $\Delta\sigma_{h\_max}$ monitored during testing
$\delta_{0.8peak}$ = drift ratio at drop to 80% from peak lateral load
δ_y = drift ratio at first bar yielding (measured)
ε_y = yield strain
γ_j = maximum joint shear distortion during approximately 3.5% drift cycles
ρ_h = $A_{sh}/s_h h''$
$\rho_h^{ACI,2}$ = minimum value recommended by ACI 352R-02 for a Type 2 joint
σ_{h_max} = maximum bar stress measured just before head

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Appendix: Table A.1–Test data of beam-column joint subassemblies with headed bars subjected to reversed cyclic loading

Authors	I.D	Type	F.M.	f'_c [ksi]	f_y [ksi]	d_b [in]	$l_p^{\dagger}$ [d_b]	$l_{dt}^{\dagger}$ [d_b]	$l_p^{\dagger\dagger}$ [d_b]	$l_{dt}^{\dagger\dagger}$ [d_b]	d_{bh} [in]	A_{brg}/A_b	c_s [d_b]	c_l [d_b]	c_{cb} [d_b]	d [in]	M_n ["-k]	M_p ["-k]	$P/(A_g f'_c)$	b_j [in]	h [in]	V_n [kips]	V_p [kips]	$\rho_h/\rho_h^{ACI,\ 2}$	δ_y	$\delta_{0.8peak}$	γ_j	σ_{h_max}/f_y	$\Delta\sigma_h/\sigma_{h\_max}$
[1]	[2]	[3]	[4]	[5]	[6]	[7]	[8]	[9]	[10]	[11]	[12]	[13]	[14]	[15]	[16]	[17]	[18]	[19]	[20]	[21]	[22]	[23]	[24]	[25]	[26]	[27]	[28]	[29]	[30]
Wallace et al.[1]	BCEJ1	Ext.	I	5.2	70	1.00	13.9	15.2	13.0	11.9	2.6	4.0	2.6	S.L.	Trans.	22.8	3859	5000	0	18.0	18.0	350.1	261.7	1.19	0.015	0.048*	N.A.	N.A.	N.A.
	KJ16	Knee	N.A.	5.4	71	0.63	22.6	15.2	22.2	11.9	2.5	11.4	3.3	S.L.	5.8	13.8	1500	1300	0	13.5	16.0	190.3	104.9	1.08	0.017	0.035	N.A.	N.A.	N.A.
	KJ17	Knee	N.A.	5.5	71	0.63	22.6	15.2	22.2	11.9	2.5	11.4	3.3	S.L.	5.8	13.8	1508	1460	0	13.5	16.0	191.4	118.1	1.08	0.012	0.06	N.A.	1.00	N.A.
	KJ18	Knee	N.A.	5.5	77	0.79	18.1	15.2	17.8	11.9	2.5	7.0	2.4	S.L.	4.6	13.7	2483	2250	0	13.5	16.0	192.9	181.7	1.08	0.017	0.04	N.A.	1.00	0
Chun et al.[2]	JM-1	Ext.	I	8.9	58	0.87	17.3	10.5	15.6	8.2	2.9	3.0	2.2	S.L.	6.1	17.3	2328	2965	0.03	16.7	19.7	373.9	165.6	0.27	0.005	0.068*	0.001	0.50	0
	JM-2	Ext.	I	8.7	58	0.87	17.3	10.5	15.6	8.2	2.9	3.0	2.2	1.2	5.7	17.3	4204	4983	0.03	16.7	19.7	369.0	300.2	0.27	0.009	0.04	0.011	0.44	0.25
	JM-No.11-1a	Ext.	I	4.8	66	1.42	12.3	14.4	11.5	11.3	2.9	4.0	3.3	S.L.	3.4	17.1	4567	4894	0	21.7	20.5	366.9	264.6	1.04	0.016	0.075	0.003	0.83	0
	JM-No.11-1b	Ext.	I	4.8	66	1.42	12.3	14.4	11.5	11.3	2.9	4.0	3.3	S.L.	3.4	17.1	4567	4779	0	21.7	20.5	366.9	258.3	1.04	0.018	0.065	0.006	0.72	0
	JMT-No.11-1a	Knee	N.A.	6.1	68	1.42	11.2	12.5	10.1	9.8	2.0	4.0	6.6	S.L.	5.6	16.1	3841	3239	0	23.6	18.9	278.0	212.6	0.96	N.A.	0.03	0.003	0.42	0
	JMT-No.11-1b	Knee	N.A.	6.1	68	1.42	11.2	12.5	10.1	9.8	2.0	4.0	6.6	S.L.	5.6	16.1	3832	3151	0	23.6	18.9	278.0	206.3	0.96	0.02	0.03	0.025	0.42	1.00
	JMT-No.11-2a	Knee	N.A.	6.1	68	1.42	11.2	12.5	10.1	9.8	0.8	4.0	6.6	S.L.	5.6	16.1	3903	3664	0	23.6	18.9	278.0	240.1	0.96	0.019	0.035	0.003	0.57	0.11
	JMT-No.11-2b	Knee	N.A.	6.1	68	1.42	11.2	12.5	10.1	9.8	0.8	4.0	6.6	S.L.	5.6	16.1	3947	3991	0	23.6	18.9	278.0	261.7	0.96	0.024	0.042	0.002	0.44	0
Bashandy[9]	Specimen	Ext.	I	4.3	65	0.98	12.8	14.7	12.0	11.4	1.1	9.0	4.1	S.L.	2.7	15.5	1273	1593	0	11.5	15.0	135.6	114.3	1.40	0.009	0.053	0.005	1.00	N.A.
Kang et al.[12]	JH	Ext.	I	4.2	70	0.75	15.0	14.7	14.3	11.5	4.5	2.6	4.2	S.L.	3.6	19.4	2055	1713	0	17.7	17.7	244.2	128.5	0.47	0.006	0.036*	0.001	N.A.	N.A.
Lee and Yu[13]	W0-M1	Ext.	I	4.5	69	0.87	14.4	14.8	13.6	11.5	2.7	5.1	1.2	S.L.	9.9	16.0	2067	2777	0.10	18.0	16.0	230.5	170.0	1.10	0.01	0.08	N.A.	N.A.	N.A.
	W150-M1#	Ext.	I	5.2	69	0.87	14.4##	14.8##	13.6##	11.5##	2.7##	5.1	1.2	S.L.	3.0	16.0	2067	2777	0.10	14.4	16.0	199.2	172.0	1.10	0.01	0.08	0.001	N.A.	N.A.
Matsushima et al.[19]	H	Ext.	II	4.4	80	0.98	11.8	10.4	11.0	17.2	3.2	N.A.	4.1	S.L.	2.3	15.2	2212	2631	0.11	14.8	15.7	184.0	165.9	0.39	N.A.	0.035	0.030	0.72	0
	Hs	Ext.	II	4.4	80	0.98	8.0	10.4	7.2	17.2	3.2	N.A.	4.1	S.L.	2.3	15.2	2212	2471	0.11	14.8	15.7	184.0	155.9	0.39	N.A.	0.035	0.030	N.A.	N.A.
Ishida et al.[20]	P1	Ext.	II	3.5	76	0.87	13.6	30.9	13.0	19.3	3.0	N.A.	3.7	S.L.	Trans.	14.0	3686	3950	0.12	15.7	15.7	219.4	236.8	1.61	0.012	0.015	N.A.	0.90	N.A.
	P2	Ext.	II	3.5	76	0.87	13.6	30.9	13.0	19.3	3.0	N.A.	3.7	S.L.	Trans.	14.0	3686	3950	0.12	15.7	15.7	219.4	236.8	1.61	0.01	0.03	N.A.	0.65	N.A.
	P3	Ext.	II	3.5	76	0.87	13.6	30.9	13.0	19.3	3.0	N.A.	3.7	S.L.	Trans.	14.0	3686	4399	0.12	15.7	15.7	219.4	263.8	1.61	0.012	0.03	N.A.	0.75	N.A.
	P4	Ext.	II	3.5	76	0.87	13.6	30.9	13.0	19.3	3.0	N.A.	3.7	S.L.	Trans.	14.0	3686	4681	0.12	15.7	15.7	219.4	280.7	1.61	0.013	0.03	N.A.	0.80	N.A.
Kiyohara et al.[21]	No.1	Ext.	II	13.8	103	1.14	12.6	14.8	12.1	11.5	6.3	5.7	4.4	S.L.	2.9	21.3	7790	9833	0	19.7	21.7	601.3	430.2	0.39	0.017	0.04*	0.018	N.A.	N.A.
	No.2	Ext.	II	13.8	103	1.14	12.6	12.0	12.1	9.4	6.3	5.7	4.4	2.4	2.9	21.3	11685	11746	0	19.7	21.7	601.3	513.9	0.26	0.017	0.04*	0.018	N.A.	N.A.
	No.3	Ext.	I	6.5	103	1.14	12.6	19.7	12.1	15.4	6.3	5.7	4.5	S.L.	2.9	21.3	5843	6856	0	19.7	21.7	413.2	300.0	0.69	0.013	0.04*	0.011	N.A.	N.A.
	No.4	Ext.	I	13.8	103	1.14	15.9	14.8	15.3	11.5	2.6	5.7	4.4	S.L.	2.9	21.3	7790	10524	0	19.7	21.7	601.3	460.4	0.39	0.017	0.08*	0.008	N.A.	N.A.
	No.5	Ext.	II	13.8	103	1.14	9.5	14.8	9.0	11.5	9.8	5.7	4.4	S.L.	2.9	21.3	7790	8876	0	19.7	21.7	601.3	388.3	0.39	0.015	0.033	0.018	N.A.	N.A.
Kiyohara et al.[25]	No.6	Ext.	III	15.4	150	1.14	12.6	21.1	12.1	16.5	6.3	5.7	1.9	2.4	2.9	20.3	18583	13792	0	19.7	21.7	635.0	639.1	0.39	0.04	0.04*	0.012	0.61	0.17
	No.7	Ext.	III	20.1	150	1.14	12.6	17.2	12.1	13.5	6.3	5.7	1.9	2.4	2.9	20.3	18583	14350	0	19.7	21.7	725.6	664.9	0.26	0.04	0.04*	0.013	0.68	0.27
	No.8	Ext.	III	6.9	150	1.14	12.6	28.1	12.1	22.0	6.3	5.7	1.9	2.4	2.9	20.3	18583	9647	0	19.7	21.7	424.1	447.0	0.69	0.04	0.04*	0.017	0.53	0.12
	No.9	Ext.	III	15.4	150	1.14	15.9	21.1	15.3	16.5	2.6	5.7	1.9	2.4	2.9	20.3	18583	16264	0	19.7	21.7	633.8	753.6	0.39	0.04	0.04*	0.012	0.45	0.21
	No.10	Ext.	III	15.7	150	1.14	9.5	21.1	9.0	16.5	9.8	5.7	1.9	2.4	2.9	20.3	18583	12836	0	19.7	21.7	640.1	594.7	0.39	0.04	0.04*	0.013	0.89	0.21
	No.11	Ext.	II	15.0	100	1.14	12.6	14.8	12.1	11.5	6.3	5.7	1.9	S.L.	2.9	21.3	9738	10391	0	19.7	21.7	626.3	454.6	0.39	0.018	0.04*	0.014	0.58	0.21
	No.12	Ext.	I	15.2	100	1.14	15.9	14.8	15.3	11.5	2.6	5.7	4.4	2.4	2.9	20.3	11134	13686	0	19.7	21.7	631.1	634.1	0.39	0.02	0.04*	0.008	0.60	0.15
Yoshida et al.[26]	No.1	Ext.	I	5.5	81	0.75	13.8	9.3	14.4	7.2	2.0	5.8	2.0	S.L.	3.2	14.0	1571	1979	0	11.8	13.8	144.4	127.8	0.50	0.02	0.04	0.007	0.64	0.15
	No.2	Ext.	I	5.5	81	0.75	13.8	9.3	14.4	7.2	2.0	4.1	2.0	S.L.	3.2	14.0	1571	1993	0	11.8	13.8	144.4	111.1	0.50	0.02	0.04	0.006	0.69	0.20
	No.3	Ext.	I	4.5	81	0.75	13.8	10.2	14.4	8.0	2.0	3.1	2.0	S.L.	3.2	14.0	1571	1961	0	11.8	13.8	130.9	109.2	0.61	0.02	0.04	0.007	0.70	0.18
Adachi et al.[28]	J30-12-0	Ext.	II	4.5	76	0.98	12.0	17.2	11.4	13.5	4.9	5.4	2.2	S.L.	3.5	11.8	2299	3490	0.06	15.7	17.7	224.1	258.8	0.71	N.A.	0.032	N.A.	N.A.	N.A.
	J30-12-P1	Ext.	I	4.5	76	0.98	12.0	17.2	11.4	13.5	4.9	5.4	2.2	S.L.	3.5	11.8	2299	3513	0.06	15.7	17.7	224.1	260.5	0.71	N.A.	0.045	N.A.	N.A.	N.A.
	J30-12-P2	Ext.	I	4.5	76	0.98	12.0	17.2	11.4	13.5	4.9	5.4	2.2	S.L.	Trans.	11.8	2299	3569	0.06	15.7	17.7	224.1	264.7	0.71	N.A.	0.062	N.A.	N.A.	N.A.
	J60-12-0	Ext.	II	9.1	76	0.98	12.0	12.2	11.4	9.5	4.9	5.4	2.2	1.7	3.5	14.4	4214	4845	0.04	15.7	17.7	320.2	276.4	0.35	N.A.	0.033	N.A.	N.A.	N.A.
	J60-12-P1	Ext.	II	9.1	76	0.98	12.0	12.2	11.4	9.5	4.9	5.4	2.2	1.7	3.5	14.4	4214	5139	0.04	15.7	17.7	320.2	293.2	0.35	N.A.	0.034	N.A.	N.A.	N.A.
	J60-12-P2	Ext.	I	9.1	76	0.98	12.0	12.2	11.4	9.5	4.9	5.4	2.2	S.L.	Trans.	14.4	4214	5320	0.04	15.7	17.7	320.2	303.5	0.24	N.A.	0.067	N.A.	N.A.	N.A.

Masuo et al.[29]	AH12-2-45	Ext.	II	18.8	148	0.98	12.0	13.8	11.3	8.6	4.9	5.8	6.9	S.L.	2.5	15.3	2727	4032	0.02	12.8	17.7	373.2	225.4	0.28	0.013	0.03	N.A.	N.A.	N.A.
	AH12-2-40	Ext.	II	18.8	148	0.98	12.0	13.8	11.3	8.6	3.0	5.8	6.9	S.L.	2.5	15.3	2727	3772	0.02	12.8	15.7	331.7	210.9	0.30	0.019	0.028	N.A.	N.A.	N.A.
	AH12-2-45A	Ext.	II	18.8	148	0.98	10.0	13.8	9.3	8.6	6.9	5.8	6.9	S.L.	2.5	15.3	2727	3998	0.02	12.8	17.7	373.2	223.5	0.28	0.016	0.03	N.A.	N.A.	N.A.
	AH8-2-45	Ext.	II	13.1	148	0.98	12.0	16.9	11.3	10.6	4.9	5.8	6.9	S.L.	2.5	15.3	2727	3603	0.03	12.8	17.7	311.8	201.4	0.42	0.02	0.03	N.A.	N.A.	N.A.
	AH12-8-45	Ext.	I	18.8	92	0.98	12.0	13.8	11.3	8.6	4.9	5.8	2.2	1.7	3.5	14.4	6753	8064	0.02	15.7	17.7	459.3	485.3	0.37	0.01	0.04	N.A.	N.A.	N.A.
	AH12-8-40	Ext.	I	18.8	92	0.98	12.0	13.8	11.3	8.6	3.0	5.8	2.2	1.7	3.5	14.4	6753	7883	0.02	15.7	15.7	408.3	474.4	0.37	0.012	0.04	N.A.	N.A.	N.A.
	AH12-8-45B	Ext.	I	18.8	92	0.98	12.0	13.8	11.3	8.6	4.9	5.8	2.2	1.7	3.5	14.4	6753	8550	0.02	15.7	17.7	459.3	514.5	0.37	0.012	0.04	N.A.	N.A.	N.A.
	AH8-6-45	Ext.	I	13.1	92	0.87	13.6	16.9	12.9	10.6	4.9	5.8	2.2	1.7	4.0	14.4	5065	6302	0.02	15.7	17.7	383.7	379.2	0.46	0.012	0.04	N.A.	N.A.	N.A.
Tasai et al.[32]	No. 6	Ext.	III	7.5	105	0.98	12.0	8.8	11.4	6.9	3.0	8.0	2.1	S.L.	2.4	15.5	4204	3725	0	14.8	15.7	242.0	224.2	0.91	0.01	0.06*	0.020	0.55	0
	No. 7	Ext.	II	7.5	105	0.98	12.0	8.8	11.4	6.9	3.0	8.0	3.7	S.L.	2.4	15.5	2102	2265	0	14.8	15.7	242.0	137.1	1.00	0.02	0.03	0.010	1.00	0
Takeuchi et al.[33]	0-1	Ext.	I	6.4	65	0.98	10.7	9.7	11.3	7.6	4.5	5.8	3.9	S.L.	2.2	15.6	2126	2458	0.10	14.8	15.7	223.1	133.3	0.96	0.005	0.05	0.008	0.90	0
	0-2	Ext.	II	8.8	85	0.98	10.7	8.1	11.3	6.4	4.5	5.8	3.9	S.L.	2.2	15.6	2804	2897	0.10	14.8	15.7	262.2	156.2	0.67	0.01	0.033	N.A.	N.A.	N.A.
	0-3	Ext.	I	4.3	55	0.98	10.7	13.7	11.3	10.7	4.5	5.8	3.9	S.L.	2.2	15.6	1807	1927	0.10	14.8	15.7	183.0	103.9	1.92	0.005	0.05	N.A.	N.A.	N.A.
	0-4	Ext.	I	6.4	65	0.98	12.0	9.7	12.6	7.6	3.2	5.8	3.9	S.L.	2.2	15.6	2126	2591	0.10	14.8	15.7	223.1	139.7	0.96	0.006	0.05	0.003	0.90	0
	0-6	Ext.	III	6.4	104	0.98	10.7	9.7	11.3	7.6	4.5	5.8	2.3	S.L.	2.2	15.6	4597	3481	0.10	14.8	15.7	223.9	187.8	0.96	N.A.	0.03	0.040	N.A.	N.A.
	0-7	Ext.	III	9.0	104	0.98	10.7	8.1	11.3	6.4	4.5	5.8	2.3	S.L.	2.2	15.6	4597	4106	0.10	14.8	15.7	264.7	221.4	0.67	N.A.	0.03	N.A.	0.96	0.30
Kato[35]	No.1	Ext.	I	8.8	82	0.87	16.2	12.1	15.4	12.1	3.5	5.3	2.3	2.5	4.9	14.5	4361	5744	0	15.7	18.7	331.8	376.3	1.62	0.013	0.04*	N.A.	N.A.	N.A.
	No.2	Ext.	I	10.3	82	0.87	16.2	11.2	15.4	11.2	3.5	5.3	2.3	2.5	4.9	14.5	4361	5582	0	15.7	18.7	358.1	365.6	1.62	0.011	0.08*	N.A.	N.A.	N.A.
Murakami et al.[36]	No. 100	Ext.	I	5.7	54	0.63	14.1	11.7	13.2	11.7	2.2	1.7	2.8	S.L.	3.7	14.3	857	1031	0.04	11.8	11.8	126.4	62.9	0.26	N.A.	0.08	N.A.	N.A.	N.A.
	No. 101	Ext.	I	5.7	54	0.63	14.1	11.7	13.2	11.7	2.2	6.3	2.8	S.L.	3.7	14.3	857	1066	0.04	11.8	11.8	126.4	65.1	0.26	N.A.	0.083*	N.A.	N.A.	N.A.
	B8-M	Ext.	I	4.3	74	0.75	11.8	13.0	11.1	13.0	2.2	6.0	4.5	S.L.	3.1	14.2	1248	1395	0.06	11.8	11.8	109.5	85.6	0.34	N.A.	0.06	N.A.	N.A.	N.A.
	B7-M	Ext.	I	4.3	74	0.75	11.8	13.0	11.1	13.0	2.2	6.0	9.7	S.L.	3.1	14.2	832	1242	0.06	11.8	11.8	109.5	76.2	0.34	N.A.	0.07	N.A.	N.A.	N.A.
	No. 102	Ext.	III	5.7	137	0.75	11.8	11.7	11.1	11.7	2.2	2.1	2.7	S.L.	3.1	14.2	3079	1957	0.04	11.8	11.8	126.4	120.1	0.26	N.A.	0.04	N.A.	N.A.	N.A.
	No. 103	Ext.	III	5.7	137	0.75	11.8	11.7	11.1	11.7	2.2	5.8	2.7	S.L.	3.1	14.2	3079	1524	0.04	11.8	11.8	126.4	93.5	0.26	N.A.	0.055	N.A.	N.A.	N.A.
	No. 104	Ext.	III	5.7	137	0.75	11.8	11.7	11.1	11.7	2.2	13.4**	2.7	S.L.	3.1	14.2	3079	1793	0.04	11.8	11.8	126.4	110.0	0.26	N.A.	0.05	N.A.	N.A.	N.A.
	M8D16	Ext.	III	4.1	145	0.63	14.1	13.3	13.2	13.3	2.2	6.0	2.8	2.2	3.7	14.3	4639	1793	0.06	11.8	11.8	107.1	109.4	0.36	N.A.	0.04	0.026	N.A.	N.A.
	M4D19	Ext.	III	4.1	145	0.75	11.8	13.3	11.1	13.3	2.2	6.0	2.7	S.L.	3.1	14.2	3258	1688	0.06	11.8	11.8	107.1	103.5	0.36	N.A.	0.04	0.024	N.A.	N.A.
	M3D19	Ext.	III	4.1	145	0.75	11.8	13.3	11.1	13.3	2.2	6.0	4.5	S.L.	3.1	14.2	2443	1676	0.06	11.8	11.8	107.1	102.8	0.36	N.A.	0.04	0.026	N.A.	N.A.
	M2D22	Ext.	III	4.1	141	0.87	10.2	13.3	9.6	13.3	2.2	6.0	9.7	S.L.	2.6	14.1	2115	1676	0.06	11.8	11.8	107.1	103.4	0.36	N.A.	0.02	0.021	N.A.	N.A.
Tazaki et al.[38]	E1	Ext.	I	4.4	55	1.04	9.8	10.3	9.7	12.1	0.8	6.9	1.9	0.9	1.7	9.8	823	951	0.08	11.8	11.8	111.1	91.6	0.40	0.005	0.06	0.008	N.A.	N.A.
	E2	Ext.	I	4.4	55	1.04	6.0	10.3	5.9	12.1	4.7	6.9	1.9	0.9	1.7	9.8	823	951	0.08	11.8	11.8	111.1	91.6	0.40	0.015	0.06	0.007	N.A.	N.A.
Nakazawa et al.[39]	J1	Ext.	I	17.4	99	0.75	15.3	22.4	15.0	12.0	1.8	5.9	1.8	1.8	3.7	11.7	3225	3391	0	12.6	14.2	282.6	254.5	0.81	0.02	0.05*	0.001	N.A.	N.A.
	J2	Ext.	I	17.4	99	0.75	15.9	22.4	15.6	12.0	1.3	5.9	1.8	1.8	3.7	11.7	3225	3344	0	12.6	14.2	282.6	251.0	0.81	0.02	0.058	0.001	0.71	0
Ishibashi et al.[27]	T345-30-4S	Roof Int.	I	4.8	56	0.75	18.0	27.6	17.9	12.1	1.5	5.4	4.9	4.9	2.5	13.5	1116	1495	0	13.8	15.7	180.9	111.9	0.66	0.01	0.065	0.009	N.A.	N.A.
	T345-30-3N	Roof Int.	I	4.8	56	0.75	18.0	27.6	17.9	12.1	1.5	5.4	4.9	4.9	2.5	13.5	1116	1488	0	13.8	15.7	180.9	111.3	0.44	0.01	0.053	0.008	N.A.	N.A.
	T490-45-4S	Roof Int.	I	7.2	82	0.75	18.0	22.5	17.9	14.1	1.5	5.4	4.9	4.9	2.5	13.5	1612	2098	0	13.8	15.7	221.1	156.9	0.44	0.01	0.053	0.010	N.A.	N.A.
	T490-45-3N	Roof Int.	I	7.2	82	0.75	18.0	22.5	17.9	14.1	1.5	5.4	4.9	4.9	2.5	13.5	1612	2083	0	13.8	15.7	221.1	155.9	0.29	0.01	0.04	0.011	N.A.	N.A.
Ishibashi and Inokuchi[30]	2S-2	Roof Int.	II	5.2	77	1.14	18.0	23.9	16.4	14.9	1.7	N.A.	7.2	7.2	2.5	20.3	5740	6272	0	19.7	23.6	401.5	312.4	0.55	0.01	0.03	0.004	N.A.	N.A.
	2S-0	Roof Int.	II	5.2	77	1.14	18.0	23.9	16.4	14.9	1.7	N.A.	7.2	7.2	2.5	20.3	5740	6165	0	19.7	23.6	401.5	307.1	0.55	0.01	0.03	0.003	N.A.	N.A.
	WN-ST	Roof Int.	II	5.4	77	1.14	18.0	23.9	16.4	14.9	1.7	N.A.	7.2	7.2	2.5	20.3	5953	6272	0	19.7	23.6	410.9	312.4	0.55	0.01	0.03	0.005	N.A.	N.A.
Shimizu et al.[31]	SN-U	Roof Int.	II	8.7	57	0.87	23.7	19.5	21.7	8.6	1.7	N.A.	7.2	7.2	2.5	20.2	5846	6591	0	19.7	23.6	520.5	329.7	0.37	0.01	0.03	0.004	N.A.	N.A.
Hattori et al.[34]	T-1	Roof Int.	II	6.1	57	0.75	15.8	11.6	16.4	11.6	3.0	4.1	2.9	2.9	2.0	12.8	1715	N.A.	0	13.8	15.7	203.2	N.A.	0.46	0.005	0.03	0.028	N.A.	N.A.
	T-2	Roof Int.	II	3.5	43	0.75	15.8	11.6	16.4	11.6	3.0	4.1	2.9	2.9	2.0	12.8	1298	N.A.	0	13.8	15.7	153.6	N.A.	0.61	0.013	0.03	0.028	N.A.	N.A.
	T-3	Roof Int.	II	8.7	71	0.75	15.8	17.0	16.4	17.0	3.0	4.1	2.9	2.9	2.0	12.8	3013	N.A.	0	13.8	15.7	242.9	N.A.	0.57	0.01	0.03	0.028	N.A.	N.A.
	T-4	Roof Int.	II	6.1	57	0.75	15.8	11.6	16.4	11.6	3.0	4.1	2.9	2.9	2.0	12.8	1715	N.A.	0	13.8	15.7	203.2	N.A.	0.46	0.008	0.03	0.016	N.A.	N.A.
	T-5	Roof Int.	I	6.5	65	0.63	18.8	9.9	18.3	9.9	3.0	6.2	2.9	2.9	2.5	12.8	1076	N.A.	0	13.8	15.7	210.1	N.A.	0.76	0.008	0.04	0.002	N.A.	N.A.
	T-6	Roof Int.	I	7.0	51	0.63	18.8	9.9	18.3	9.9	3.0	6.2	2.9	2.9	2.5	12.8	1076	N.A.	0	13.8	15.7	217.9	N.A.	0.76	0.01	0.04	N.A.	N.A.	N.A.
	T-7	Roof Int.	I	3.7	56	0.51	23.1	11.6	22.3	11.6	3.0	7.5	3.0	3.0	3.2	12.8	607	N.A.	0	13.8	15.7	158.0	N.A.	1.22	0.002	0.04	N.A.	N.A.	N.A.
	T-8	Roof Int.	I	3.9	56	0.51	23.1	11.6	22.3	11.6	3.0	7.5	3.0	3.0	3.2	12.8	607	N.A.	0	13.8	15.7	162.6	N.A.	0.61	0.005	0.04	N.A.	N.A.	N.A.
	T-9	Roof Int.	I	4.0	56	0.51	23.1	11.6	22.3	11.6	3.0	7.5	3.0	3.0	3.2	12.8	607	N.A.	0	13.8	15.7	165.6	N.A.	0.61	0.01	0.04	0.008	N.A.	N.A.

Conversion: 1 ksi = 6.8948 MPa; 1 in. = 25.4 mm; 1 in^2 = 645.16 mm^2; 1 in.-kips = 0.113 kN-m; 1 kips = 4.4482 kN. F.M. = Failure Mode; I = Category–I (member flexural hinging followed by modest joint deterioration); II = Category–II (member flexural hinging followed by joint failure); III = Category–III (joint failure prior to member flexural hinging); N.A. = Not Available; S.L. = Single layer of headed bars; Trans. = with Transverse beams; $l_p^{\dagger}$ & $l_{dt}^{\dagger}$ = development length provided (p) & required (dt) per ACI 318-08; $l_p^{\dagger\dagger}$ & $l_{dt}^{\dagger\dagger}$ = development length provided (p) & required (dt) per ACI 352R-02 (Type 2); $^{\#}$ = eccentric connection with an offset of a half of the beam width; $^{\#\#}$ = average values for staggered headed bars; * = at least (i.e., testing was stopped prior to 20% drop from the peak); ** = one bearing plate was used for a group of heads.