SEISMIC PERFORMANCE OF HIGH-STRENGTH RC BEAM-COLUMN JOINTS USING HEADED BARS UNDER HIGH P-δ EFFECTS

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ABSTRACT

Based on effects of gravity, axial forces of columns at lower stories of a high-rise building usually are remarkable. The axial forces of the exterior or corner columns caused by a lateral seismic force may vary depending upon the applied direction of the seismic force. Therefore, a second-order moment from P-δ effects may be huge for the beam-column joints of the lower stories of high-rise buildings. This study is to focused on experimental seismic performance of beam-column joints using high-strength materials, 70 MPa for concrete and 690 MPa for main reinforcement with threaded surface deformation, respectively. In this paper, not only the P-δ effect but also the shear effect of panel zones for the beam-column joint is considered as the study parameter and there are four beam-column-joint specimens are carried out. A grouting threaded anchor device, called as Plate Nuts, is used at end of beam main reinforcements for anchorage in the beam-column joints. For the main bras of the beams, related requirements of headed bars according to the ACI 318-14 Code are adopted, but their net spacing is 2𝑑. Three testing conditions of axial force of column including two fixed loads, 0.1 and 0.45 𝑓 covid, and a variable load from 0 to 0.5 𝑓 covid with different lateral deformation are conducted. Test results show that shear cracks at the panel zone can be restrained as the P-δ effect increases. In addition, no shear failure occurring in panel zone for all specimens is appeared, even though the specimen with high shear demand. The results also indicate that all specimens have well seismic deformation capacity with at least 4% radian story drift ratio.

Keywords: beam-column joints; headed bars; high-strength reinforced concrete (HSRC); P-δ effect

1. INTRODUCTION

A beam-column joint is one of the most complex force-acted elements in a moment resisting frame (MRF). Its panel zone located an intersection area of beam and column is subjected to significant shear under actions of lateral earthquake forces. Under a combining effect of gravity and seismic loads for building structures, the axial forces of exterior columns in the seismic MRFs usually are varied with quantity and applied direction of overturning moment caused by seismic forces. In order to assess real seismic strength-deformation performance of beam-column joints at lower stories of a high-rise building, a second order P-δ effect, product of the axial force P and corresponding horizontal story drift δ, should be considered. This is because the beam-column joints located at lower stories for a building, especially

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for a high-rise building, the additional moment induced by the P-δ effect is huge.

For high-rise buildings, the axial forces of columns in lower stories, in general, are more significant caused by gravity loads. Under combining seismic loads, the axial force of an exterior column located on two peripheral frames of buildings perpendicular to the direction of seismic loads is most significantly increased or decreased depending on acting direction of the seismic load. The axial forces of interior columns near the center of mass of the buildings may have relative less variation due to shorter distances between the columns and neutral axis of the overturning moment caused by seismic loads. Therefore, a real beam-column joint of high-rise building may always be subject to the P-δ effect produced by its axial force P and corresponding relative story drift δ. The secondary P-δ moment of the columns in the lower stories of high-rise building may be quite remarkable. Based on the modern reinforced-concrete design codes in American and Taiwan (ACI 318, 2014 and CPAMI, 2011), the design axial force of column may reach 0.5 f_c A_g high. Therefore, the relative high P-δ effect is possible for columns in lower stories of high-rise buildings. This study is to focus on seismic performance of HSRC beam-column joints not only with different P-δ effects but also with various shear demand-to-capacity ratios of panel zone.

In the past, due to limitations on test facilities of large scale structures in the world, related experimental seismic studies of full-size beam-column joint considering large P-δ effects lacked. In recent years, these related experimental studies, like large scale or full-scale specimens, make possible to be conducted since a multi-function testing facility with large scale and huge capacities of vertical force and horizontal displacement, called as MATS (Multi-Axial Testing System), was established by the National Center for Research on Earthquake Engineering (NCREE) in Taiwan (Lin, et.al., 2017). The MATS facility has a large test space with 5 meters in height.

Since 2009, the NCREE actively launched an integral research and development project for high-strength reinforced concrete (HSRC) structural system in Taiwan, called as “Taiwan New RC Project”. Its application scope on strengths of materials are between 70 to 100 MPa for concrete and 690 MPa of yield strength for main reinforcement. The Taiwan New RC Project is attempted to apply for high-rise buildings. Two guidelines of design and construction for the high-strength reinforced-concrete buildings in Chinese were published in 2017 (CSSC, et.al. 2017a, 2017b).

Figure 1 Threaded bars, threaded-type coupler and threaded-type end-anchorage device

1.1 Net spacing of headed bars

For an exterior beam-column joints, discontinuous main bars of beam members usually are anchored
into the panel zone by 90-degree hooks traditionally. Nevertheless, the Grade 690 MPa steel bar is not allowed to bend proposed by the design and construction guidelines for HSRC structures in Taiwan (CSSE, et al., 2017a, 2017b) due to its characteristics of hard bending and unfavorable welding. Consequently, this grade steel made as threaded deformation on its surface, as shown in Figure 1 (a), is proposed. The threaded bar also is convenient to be spliced by threaded-type couplers grouting high-strength mortar, see Figure 1 (b) and to be anchored by threaded-type end-anchorage devices grouting high-strength mortar, see Figure 1 (c). In this paper, the end-anchorage device called “Plate Nut” made by Tokyo Tekko Co. Ltd. is used as a head of main bar for anchorage. For the latest ACI 318-14 Code (ACI, 2014), a net spacing of $3d_b$ between the headed bars is stipulated in its seismic design requirements, where $d_b$ is diameter of the headed bars. However, this requirement of the $3d_b$ net spacing causes an adverse condition on placing reinforcements in practices compared to $1d_b$ net spacing with the traditional standard hook. Past researches (Lin et al., 2016, Lin and Chi, 2011, Hwang et al., 2016) focused on the net spacing of headed bars in exterior beam-column joints had confirmed that the seismic performance of the beam-column joint using headed bars as beam’s main bars with $2d_b$ net spacing to anchor in panel zone was similar to that using traditional standard hooks with the same spacing of $2d_b$ to anchor in panel zone. Therefore, the net spacing of $2d_b$ is used to design the specimens in this study as well. This result that the minimum net spacing of the headed bars should not be less than $2d_b$ had been adopted by the design guideline for HSRC structures in Taiwan (CSSE, et al., 2017).

1.2 Development Length of Headed Bars in ACI 318-14 Code

Until now, there are no design provisions for anchorage used by headed bars in Taiwan’s Structural Concrete Design Code (CPAMI, 2011). However, the ACI 318 Code had provided design requirements of the headed bars since the Year 2008 Version. A development length of the headed bars in tension stipulated by the Year 2014 Version is same as that of the Year 2008 Version and is shown in EQ (1), and not less than $8d_b$ and 150 mm.

$$l_{dt} = 0.192 \frac{f_y}{f'_c} d_b \quad \text{(MPa)} \quad (1)$$

where $f'_c$ and $f_y$ are the compressive strength of concrete (MPa) and yield strength of reinforcement (MPa), respectively, and $d_b$ is the diameter of the headed bar (mm). However, there are some limitations to be made to limit strengths of materials and design details. Those include yield strength of the headed reinforcement not exceeding 420 MPa, concrete compressive strength not more than 70 MPa and with normal-weight concrete bearing area of head not less than $4d_b$ (sectional area of the steel bar), size of the headed bar not more than #11, clear cover of the headed bar not less than $2d_b$, etc.

1.3 Shear Demand to Capacity Ratio of Panel Zone

To avoid a beam-column joint from shear plastic hinging at panel zone before flexural plastic hinges occurring at beam ends, the panel zone should provide sufficient shear capacity against shear demand. The demand shear is caused by two reverse plastic moments of beams framed the joint along the direction of seismic loads. Actually, the horizontal demand shear of beam-column joint without P-δ effect can be obtained by the sum of the possible maximum tension from tensile reinforcements of the two beams subtracting its column shear, $V_{col}$. According to a basic seismic design concept of strong column-weak beam and seismic design provisions of ACI 318 or Taiwan RC design codes, the tensile reinforcements of beam’s ends those connected into beam-column joints should provide inelastic deformation capacity and develop at least 1.25 times nominal yield tensile force of tensile reinforcements of the beam, $1.25A_f f_y$. Therefore, the horizontal demand shear, $V_{h,u}$, of an exterior beam-column joint can be estimated by Eq. (2).

$$V_{h,u} = 1.25A_s f_y - V_{col} \quad (2)$$

where $A_s$ is the larger sectional area of top and bottom longitudinal tension reinforcements of the beam; $V_{col}$ is column shear that can be obtained to assume a beam-column joint sub-assembly with both top
and bottom inflection points located at the two middle points of top and bottom columns, respectively, and the beam end(s) forming full flexural plastic hinge(s).

Regarding the shear capacity of beam-column joint, according to seismic design requirements of ACI 318-14 or the Taiwan code, the nominal shear capacity, $V_{jh,n}$ of panel zone of beam-column joint should be determined by Eq. (3).

$$V_{jh,n} = \gamma \sqrt{f_y} b_y h_c$$  \hspace{1cm} (3)

where $\gamma$ is a shear capacity coefficient of panel zone which depending on confined conditions of beam-column joint, $\gamma$ equal to 1.0 for exterior beam-column joints required by the ACI 318-14 code; $b_y h_c$ represents an effective shear resisting area of the panel zone; $b_y$ is the effective width of the shear area and $h_c$ is the overall depth of the column in plane along the direction subjected to shear.

According to basic strength design provisions, a reduced shear strength capacity of $\phi V_{jh,n}$ for the panel zone of the beam-column joint shall be larger than and equal to $V_{jh,u}$, see Eq (4). Here, a ratio of shear demand to capacity, $V_{jh,u} / V_{jh,n}$ in the panel zone was defined as shear DCR in this paper.

$$\phi V_{jh,n} \geq V_{jh,u}$$  \hspace{1cm} (4)

where $\phi$ is equal to 0.85 based on seismic design requirements of ACI 318-14 or the Taiwan code.

2. EXPERIMENTAL PLAN

This study concentrates on seismic performance of HSRC beam-column joints not only with high shear demand of panel zone but also with high P-\(\delta\) effects. In addition, the seismic performance of the beam-column joints using headed bars as beam main bars with a net spacing of 2$d_b$ to anchor into their panel zones is also investigated.

In this study, four specimens of exterior beam-column joints are conducted. The compressive strength of concrete is 70 MPa, and the main and stirrup reinforcements adopt SD690 and SD790 steel bars, respectively, where the surface deformation of the SD 690 steel bar was threaded type, see Figure 1(a). Mechanical properties of the SD 690 and SD 790 are presented in Table 1. The head of the headed bars in this study uses the grouting end-anchorage devices (Plate Nuts), shown in Figure 1(c). The embedded length of beam longitudinal reinforcements within the panel zones of the beam-column joints determined by the Eq (1) and the net spacing of 2$d_b$ for the headed bar are adopted to anchor in their panel zones for all specimens. Two design parameters which include axial force and the shear demand to capacity ratio, DCR, are concerned. Three axial force conditions of constant 0.1 $f_y A_g$, constant 0.45 $f_y A_g$ and varied from 0 to 0.5 $f_y A_g$ are considered. Two shear DCRs of 1.02 and 0.76 are adopted for the four specimens.

<table>
<thead>
<tr>
<th>Grade</th>
<th>$f_y$ (MPa)</th>
<th>$f_u$ (MPa)</th>
<th>$f_{ua}/f_{ya}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>SD 690</td>
<td>690-815</td>
<td>$\geq$ 860</td>
<td>$\geq$ 1.25</td>
</tr>
<tr>
<td>SD 790</td>
<td>$\geq$ 790</td>
<td>$\geq$ 930</td>
<td>--</td>
</tr>
</tbody>
</table>

- $f_y$ nominal yield strength
- $f_u$ nominal ultimate strength
- $f_{ya}$ actual yield strength determined by tensile test
- $f_{ua}$ actual ultimate strength determined by tensile test

Based on the two design parameters, the four specimens are named by LAHV, LAMV, HAHV, HAMV and VAMV, respectively, in which the specimen titles of HAMV and VAMV use the identical specimen.
For the specimen title, the former two letters described axial force degree of the specimen column. The LA or HA means that the specimen is subjected to a low or high constant axial force of column (0.1 $f_c A_g$ or 0.45 $f_c A_g$), respectively, and VA is to apply varied axial force of column with different lateral displacements. The latter two letters represent the shear demand to capacity ratio (DCR) in panel zone without considering the P-δ effects, The MV or HV has medium (0.76) or high (1.02) DCR, respectively. The HAMV and VAMV were the same specimen. The specimen was called as HAMV, when its column was subjected to a constant and high axial force of 0.45 $f_c A_g$ during a cyclic loading test of 0 to +/- 3% radian of the interstory drift ratio. It was called as VAMV, when the column was applied varied axial forces from 0 to 0.5 $f_c A_g$ during a cyclic loading test of +/- 4% to +/- 8% radian. The different types of loading history during testing are showed in Figure 2. The cross-section of beam and column for all specimens and design parameters were presented in Table 2.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>LAHV</th>
<th>LAMV</th>
<th>HAHV</th>
<th>HAMV VAMV</th>
</tr>
</thead>
<tbody>
<tr>
<td>Material</td>
<td>$f_c'$ (MPa)</td>
<td>70</td>
<td>Steel Bar</td>
<td>SD 685</td>
</tr>
<tr>
<td>column</td>
<td>$b_c \times h_c$ (mm)</td>
<td>600 x 600</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Main bars</td>
<td>16-#8</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Beam</td>
<td>$b \times h$ (mm)</td>
<td>400 x 700</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Top Bars $(\rho)$</td>
<td>4+4-#8 (0.017)</td>
<td>4+2-#8 (0.013)</td>
<td>4+4-#8 (0.017)</td>
</tr>
<tr>
<td></td>
<td>Bott. Bars $(\rho)$</td>
<td>4+4-#8 (0.017)</td>
<td>4+2-#8 (0.013)</td>
<td>4+4-#8 (0.017)</td>
</tr>
<tr>
<td>Joint</td>
<td>Net spacing</td>
<td></td>
<td></td>
<td>2 $d_b$</td>
</tr>
<tr>
<td></td>
<td>Stirrup</td>
<td></td>
<td></td>
<td>#4 @ 100 mm</td>
</tr>
<tr>
<td></td>
<td>Type</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Anchor Length of BM bars $L_{dt}$</td>
<td>390 mm</td>
<td>15.4$d_b$</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$\frac{V_{j,h,u}}{V_{j,h,v}}$</td>
<td>1.02</td>
<td>0.76</td>
<td>1.02</td>
</tr>
<tr>
<td></td>
<td>$R_m = \frac{\sum(M_a)_u}{\sum(M_a)_b}$</td>
<td>2.28</td>
<td>2.91</td>
<td>2.28</td>
</tr>
<tr>
<td></td>
<td>Axial Force</td>
<td>Constant</td>
<td>Constant</td>
<td>Constant</td>
</tr>
<tr>
<td></td>
<td>$0.1f_c A_g$</td>
<td>$0.1f_c A_g$</td>
<td>$0.45f_c A_g$</td>
<td>$0.45f_c A_g$</td>
</tr>
</tbody>
</table>

Notes:
- $L_{dt}$: anchorage length of headed bars,
- $\rho$: tension reinforcement ratio of beam,
- $V_{j,h,u}$ / $V_{j,h,v}$: shear demand to capacity ratio at joints.

The design of the beam-column joint specimens is corresponding to a hypothetical prototype structure, the beam span and story height are 8.2m and 3.7m, respectively. The column height of the beam-column joint specimens between top and bottom hinge bearings is 3700 mm, and the beam length of the specimens from the center of column to the center of the actuator at the beam end is 4100 mm, shown in Figure 3. The dimensions of column and beam for all specimens are 600 x 600 mm and 400 x 700 mm, respectively, shown in Table 2. The MATS Facility is adopted to do the tests in this study. During the experiment, displacement control procedure is used to apply horizontal cyclic displacements to the column’s bottom end and zero displacement is applied at the free end of beam, see Figure 3. Figure 4 indicates the actual experimental setup. The axial and shear forces of specimen column, the shear forces of specimen beam, the horizontal and vertical displacements of specimen column and the vertical displacements of specimen beam can be obtained from the MATS facility. The panel zone deformation
and the local deformation at beam end near panel zone were measured by the NDI optical measurement system. The measured points are shown in Figure 5

Figure 2 Lateral displacement history and axial force for all specimens
3. DISCUSSIONS OF RESULTS
3.1 Failure Modes

Actual failure situations of all specimens after completion of experiment are shown in Figure 6. There are two kinds of failure mode to be observed. Figure 6(a) shows significant shear failure at the panel zone of Specimen LAHV which the column is subjected to lower axial force ($0.1A_f$) and the panel zone is resisted high shear force (DCR=1.02). However, a slightly flexural plastic hinge is formed and found. The other three specimens were failed at ends of their beams after developing fully flexural plastic hinges, shown in the Figure 6(b), (c) and(d). The photos also show that there are two types of crack to be developed at their panel zones of the three specimens, HAHV, LAMV, and VAMV, developed fully flexural plastic hinges. For the specimen HAHV, one pattern of vertical cracks in the panel zone roughly along column’s longitudinal reinforcements is examined due to higher axial force (Figure 2(b)). For Specimens LAMV and VAMV, the other pattern of diagonal cracks in their panel zones is shown in Figure 6(c) and (d) caused by lower axial force. It is well known that the diagonal cracks in panel zone of beam-column joint results from its relative high shear force under acting by the flexural plastic hinge of its beam end(s). Therefore, the Specimen LAHV, subjected to lower column axial force and higher panel zone shear, causes significant shear failure at its panel zone. On the contrary, the Specimen HAHV which it is subject to higher axial force in the column and as high panel zone shear demand as the Specimen LAHV didn’t observe the diagonal cracks at panel zone and fully developed flexural plastic hinge at the beam end. Comparing the test results of Specimens LAHV and HAHV, the higher axial force seems to help enhance shear capacity of panel zone. On the other hand, for the two specimens with medium shear at the panel zone, LAMV and VAMV, the shear cracks of the panel zones are found at panel zone of the Specimen LAMV (Figure 6(c)) with lower constant column axial force and the Specimen VAMV (Figure 6(d)) with varied column axial force when it is subjected to lower column axial force. Based on the crack forming of panel zones in these four specimens, obviously, it is demonstrated that an increase in column axial forces of the beam-to-column joint make helpful for improving concrete shear capacity of panel zone.

Figure 6 Failure modes of specimens
Figure 7 Relationships between beam moment and interstory drift ratio

Figure 8 Relationships between the column shear and interstory drift ratio
3.2 Behaviors of Strength and Deformation

Figure 7 plots the relationship between the beam end moment and interstory drift ratio for all four beam-column joint specimens, where the interstory drift ratio is the lateral drift of column divided by column height between top and bottom hinge supports. For all specimens, when the interstory drift ratio is up to 4% radian, the integrated strength responses are similar. Their decay of envelope strength and significant strength deterioration during the three repeat cycles are not observed. At this time, the Specimen LAHV do not fail in shear at its panel zone yet. As the interstory drift ratio reaches 6% radian, the beam moment of all specimens develops to their maximum strength. During the load cycles of this peak interstory drift ratio, due to the panel zone’s shear failure, the specimen LAHV has a significant strength decrease in the second cycle comparing with the first cycle. For the other three specimens, their strength deteriorations of beam moment in the second cycle are limited to compare with the first cycle. This is because these three specimens develop fully flexural plastic hinges of the beams. Therefore, these two kinds of failure mode of the shear are failed at panel zone and the flexural yielded at beam end for the beam-column joint specimens are able to be distinguished by the different strength-decayed phenomena mentioned above. These test results indicate that all specimens may provide stable seismic performance without strength deterioration before the drift ratio of 4% radian.

3.3 Anchorage effects of headed bars

As mentioned above, the headed bars made by the grout-filled threaded anchor device (Plate Nuts) are anchored into the beam-column joints with 2 $d_b$ of clear spacing and the embedded length of Eq (1) for all specimens. These experimental results with high-strength reinforced-concrete materials confirm that all specimens provide sufficient seismic performances of at least 4% radian of interstory drift ration. This study indicates that the threaded bars using the details of clear spacing of 2 $d_b$ and anchored length of Eq (1) embedded into joints can perform well seismic capacities for the high-strength RC beam-column joints. This conclusion is same as one of normal-strength RC beam-column joints found by Lin et al. in 2016 (Lin et al., 2016).

3.4 P-δ effects

Figure 8 presents relationships between column shear and drift, relative displacement of top and bottom ends of column δ, for all specimens. The column shears are calculated by beam shear, column axial force and with/without drift between top and bottom hinge supports of column based upon force balance relationship. In figure 8, both black dashed and red solid lines are the shear responses of columns without and with the P-δ effects, respectively. The green solid lines show the horizontal shear of column which are caused by the P-δ effects, i.e. product of axial force and drift of column, during testing. The axial forces are constant for Specimens LAHV, HAHV, LAMV and HAMV during testing, so their column shears due to P-δ effects are linear. For Specimen HAMV, its column shear exhibit as a 2nd-order curve during the testing with axial force varied linearly. To examine the Specimens HAHV and VAMV with high axial force especially, a phenomenon is observed obviously that the directions of column’s shear force and displacement are reserved as the drift (δ) of column is increasing. The larger the P-δ effect, the more significant the phenomena are.

4. CONCLUSIONS

1. Test results indicate that all specimens have well seismic deformation capacities of 4% radian of interstory drift ratio without strength deterioration on their enveloped strength curves. There are two failure modes including shear failure within panel and flexural hinge at beam end to be found, but these failures occur far away the interstory drift of 4% radian.

2. Comparing the Specimens LAHV and HAHV with an identical shear demand at the panel zone, the failure mode transfers from panel zone’s shear failure of Specimen LAHV with lower axial force to beam end’s flexural plastic hinge of Specimen HAHV with higher axial force. These results demonstrate the axial force of column is helpful to enhance its panel zone shear capacity for the
beam-column joint. It is implied that as the axial force of column is high, the beam-column joint is capable of well seismic performance not to be affected by the high P-δ effect.

3. The test results also show that the threaded headed bars anchored into beam-column joint by the grout-filled anchorage devices (Plate Nuts) with 2 \( d_b \) of clear spacing and Eq (1) of anchored length are able to provide sufficient anchorage performance for the high-strength reinforced-concrete materials of 70 MPa concrete and SD 690 reinforcement.

4. Based on the test results of Specimens HAHV and VAMV with high axial force, a phenomenon is observed obviously that the directions of shear force and displacement are reserved as the drift (\( \delta \)) of column is increasing. The larger the P-δ effect, the more significant the phenomena are.

5. ACKNOWLEDGMENTS

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