ANCHORAGE BEHAVIOR OF 90-DEGREE HOOKED BEAM BARS IN REINFORCED CONCRETE WALL-BEAM INTERSECTIONS

Osamu Joh, Yasuaki Goto and Atsunori Kitano
Graduate School of Engineering, Hokkaido University, Japan

Abstract

Experiments were conducted to determine anchorage performances of 90-degree hooked beam bars in a joint at which a beam intersects a structural wall at right angles. Seven wall-beam joint specimens that had some variations in the arrangements of wall reinforcement and L-shaped beam bar anchorage were used in the experiments. All of the specimens subjected to pullout loading on the beam bars failed in beam bar anchorage. The anchorage strength increased as the amount of wall reinforcements increased, especially when the wall reinforcement was arranged in the direction of thickness as tie bars. The following results were obtained: (1) the inclusion of many tie bars made the zone of transmission stress become wider and the anchorage strength increase, (2) the anchorage strength of hooked bars without tie bars increased by only 10% even though spacing of wall bars was reduced by 50%, and (3) accurate estimation of anchorage strength was possible by considering the dowel action of wall bars.

1. Introduction

In a reinforced-concrete structure, when a beam is connected to one face of a structural wall at right angles, the main bars of the beam are anchored into the wall usually with 90-degree hooks. We have been experimentally investigating anchorage behaviors of steel bar hooks arranged in exterior beam-column joints with a rotated T shape for a middle story of a building [1] and with a reversed L shape for a roof story [2], and we have proposed formulas for evaluating anchorage strengths. Based on our previous experimental results, failure modes of a 90-degree hooked-bar anchorage in a beam-column joint can be divided into three types: 1) side split failure, in which the concrete covering the bend portion of the beam bars in a joint peels away, leaving dish-shaped depressions on both sides of the joint; 2) local compression failure, in which a small amount of concrete adjoining the inside part of the 90-degree-bent portion of each beam bar is crushed; and 3) raking-out failure, in which a concrete block,
approximately the same size as the inside dimensions of the hooked bar, is raked out toward the beam side of the column, and all beam bars simultaneously lose their resistance. Raking-out failure is caused by the use of many beam bars and/or a short horizontal development length inside the joint.

From the results of our previous experiments, it became clear that the anchorage strength of the raking-out failure mode depends on the number of hoops in the joint and the horizontal development length \(L_{dh}\). This length is the lateral distance between an inside face of the column and an outside face of beam bar tails and it consists of horizontal bar length of a straight portion in the joint, radius of bar bent and diameter of the bar. Judging from the anchorage behaviors in a beam-column joint, it can be easily predicted that the mode of anchorage failure of a beam-wall joint depends on raking-out and that the anchorage strength of hooked beam bars in this joint is lower than that in a beam-column joint because the depth of the wall is smaller than that of the column and there is no lateral reinforcement in the depth direction. Most structural design codes, however, do not prescribe any special requirement for reinforcement.

The purpose of this study was to clarify anchorage behaviors of 90-degree hooked beam bars in a beam-wall joint intersecting each other at right angles. For this purpose, experiments in which specimens were subjected to pullout loading on the beam bars were carried out. The results were compared with results of anchorage strengths in beam-column joints obtained from our previous experiments.

2. Experiment

2.1. Test specimens

Nearly actual sized specimens, which were simulated tensile beam bars with 90-degree hooks anchored into a beam-wall joint, were used (see Figure 1). Beam concrete or compressive beam bars was not used in order to simplify the production of specimens. The dimensions of the specimens were identical: 2700 mm in height (the distance between two reaction points on the wall being 2300 mm); 900 mm and 250 mm in wall width and thickness, respectively; and 250 mm and 600 mm in imaginary beam width and depth, respectively. The beam bars were two threaded deformed bars of 19 mm in nominal diameter \(d_b\), the wall bars were deformed bars of 16 mm or 19 mm in diameter, and the tie bars were deformed bars of 10 mm in diameter. These bars were arranged in the specimens as shown in Figure 1 and in Table 1.

A total of seven variables were used: 1) two horizontal development lengths \(L_{dh}\) (distance from the critical section of the beam to the center of the tail), 2) two vertical development lengths \(L_{dv}\) (distance from the tip of the tail to the center of the lateral beam bar), and 3) three spacings of wall-tie bars. Specimen WA25-1A was a standard specimen, and the other specimens differed from the standard specimen in only one or two test variables.

2.2. Mechanical properties of materials

Only beam bars made from high-strength steel were used in order to generate anchorage failure. Wall bars and tie bars made from normal-strength steel were generally used, but
those made from high-strength steel were used for specimens having high anchorage strength in order to avoid premature failure in yielding of the bars. Concrete strength varied from 34.5 MPa to 41.2 MPa. The mechanical properties of the steel bars and typical concrete used in the specimens are shown in Table 2. The aggregate used was crushed stone with a maximum diameter of 20 mm, which was normal size to match the actual scale of specimens.

Table 1. Specification of specimens

<table>
<thead>
<tr>
<th>Specimen</th>
<th>beam-bar (2-D19)</th>
<th>wall-bar spacing</th>
<th>wall-tie bar (D10)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$L_{db}$</td>
<td>$L_{dv}$</td>
<td>grade</td>
</tr>
<tr>
<td>WA25-1A</td>
<td>155</td>
<td>295</td>
<td>SD490</td>
</tr>
<tr>
<td>WA25-3A</td>
<td>155</td>
<td>539</td>
<td>SD490</td>
</tr>
<tr>
<td>WB25-1A</td>
<td>83</td>
<td>295</td>
<td>SD490</td>
</tr>
<tr>
<td>WA25-1B</td>
<td>155</td>
<td>295</td>
<td>SD490</td>
</tr>
<tr>
<td>WA25-1AP2</td>
<td>155</td>
<td>295</td>
<td>SD490</td>
</tr>
<tr>
<td>WA25-1AP3</td>
<td>155</td>
<td>295</td>
<td>SD685</td>
</tr>
<tr>
<td>WA25-1BP2</td>
<td>155</td>
<td>295</td>
<td>SD685</td>
</tr>
</tbody>
</table>

[Notes] $D$: diameter of reinforcing bar, $a/b$: distinction of strength, $D16$: deformed bar of 16mm in normal diameter
2.3. Instrumentation for loading and measuring

Tensile load \((T_b)\), controlled so as to distribute the increasing pull-out displacement equally between two beam bars, was supplied horizontally to both bars. In order to simulate a moment diagram in actual beam-wall joints, the following loading system was used. An apparatus supported by a pin and an apparatus by a roller were set on the compression zone of an imaginary beam and on the reflection point at the bottom of the wall, respectively, and another load \((Q_w)\) applied to the top of the wall was controlled at \(Q_w = 0.19 \, T_b\) so as to generate the same shear forces in the upper and lower parts of the wall. Reaction \(R_1\) was generated in the compression zone of the imaginary slab cross-section by a steel plate with a height of 180 mm and a width of 900 mm, and reaction \(R_2\) was generated at the bottom of the wall by a steel plate with a height of 100 mm and a width of 900 mm, as shown in Figure 1.

Using a steel frame for measurement attached at the top and bottom reflection points of the wall, relative displacements were measured in the direction of out-plane of the wall and lateral displacement of each beam bar was also measured as relative displacements from the mid depth of the wall, as shown in Figure 2.

3. Test results

3.1. Behavior of Failure

Figure 3 is a schema showing a typical crack pattern that appeared on the inside faces of specimens. Failure was dominated by three types of cracks: radially oriented UV and DV cracks caused by the formation of a shallow cone, the top of which was located at the position of the beam bars, due to expansion of the wall; and a C crack that formed along the circumference on the side opposite the tails. The C crack was a shearing crack oriented inward from the bent part of the bar and upward at an angle of 45-degrees. The properties of this crack are the same as those of a diagonal crack occurring in a beam-
column joints due to raking-out anchorage failure. Figure 4 shows photographs of specimens taken after the loading test.

3.2. Relationship between load and displacement
The existence of tie bars and the number of wall bars affect the relationship between load and displacement as shown in Figure 5.

(1) Specimen WA25-1A (with no tie bars and with wall bar spacing of 200 mm): First, the initial stiffness in the relationship was decreased by the occurrence of WF cracking (shown by ▲ in Figure 5). Next, the stiffness in the relationship was greatly decreased again by the occurrence of UV cracking (V in Figure 5). Finally, the occurrence of C crack led to maximum strength or to strength just before maximum strength.

(2) Specimens WA25-1AP2 and -1AP3 (with tie bars and wall bar spacing of 200 mm): The stiffness of each specimen was decreased remarkably by the occurrence of WF cracking, and the occurrence of C crack led to maximum strength.

(3) Specimens WA25-1BP2 and -1BP2 (with tie bars and wall bar spacing of 100 mm): Since the small spacing of tie bars and wall bars generated a truss mechanism that could effectively transmit local compressive stress from the bar bent to the supporting points, anchorage strength increased after the occurrence of C cracking. These specimens reached maximum strength due to the occurrence of TV cracking (shown by ◇ in Figure 5) or EH cracking (shown by ◆ in Figure 5).

<table>
<thead>
<tr>
<th>Specimen</th>
<th>$\sigma_b$ (MPa)</th>
<th>$T_d$ (kN)</th>
<th>$T_d'$ (kN)</th>
<th>Crack at $T_d$ (kN)</th>
<th>$T_u$ (kN)</th>
<th>$T_u'$ (kN)</th>
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<td>116 124</td>
<td>WF</td>
<td>153 163</td>
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<td></td>
</tr>
<tr>
<td>WA25-3A</td>
<td>38.9</td>
<td>105 105</td>
<td>WF</td>
<td>155 155</td>
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<td></td>
</tr>
<tr>
<td>WB25-1A</td>
<td>38.8</td>
<td>51 51</td>
<td>WF</td>
<td>93 93</td>
<td></td>
<td></td>
</tr>
<tr>
<td>WA25-1B</td>
<td>38.8</td>
<td>96 96</td>
<td>WF</td>
<td>172 179</td>
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<td></td>
</tr>
<tr>
<td>WA25-1AP2</td>
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<td>WF</td>
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<td></td>
</tr>
<tr>
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<td>37.8</td>
<td>140 142</td>
<td>WF</td>
<td>224 228</td>
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<td></td>
</tr>
<tr>
<td>WA25-1BP2</td>
<td>38.3</td>
<td>121 122</td>
<td>WF</td>
<td>377 380</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
3.3. **Strength at degradation of initial stiffness**
The test results of strengths are shown in Table 3. The value \( T_d \) is the strength at degradation of initial stiffness (shown by ▲ or ▼ in Figure 5) expressed by total tensile force in the beam bars, and the value \( T_u \) is maximum strength. The values \( T_d' \) and \( T_u' \) were normalized by the average compressive strength of concrete (=39.1 MPa) using Equation (1) in order to eliminate the differences in concrete strength \( \sigma_B \) among the specimens. All of the specimens except WB25-1A had almost the same strength \( T_d' \). This means the strength at degradation of initial stiffness does not depend on the spacing of wall bars and tie bars, but it is proportional to the lateral development length.

\[
T_d' \text{ or } T_u' = T_d \text{ or } T_u \times \sqrt{\frac{39.1}{\sigma_B}} \tag{1}
\]

3.4. **Factors affecting maximum strength**

(1) Spacing of wall bars
In the standard specimen (WA25-1A) with no tie bars and with wall bar spacing of 200 mm, the resistance against pull-out loading on the beam bars is thought to consist mainly of sliding resistance of the concrete on a plane along the shear crack (C crack) and a dowel action caused by wall bars intersecting the crack plane. When the spacing of the wall bars was decreased to 100 mm (the same as that of specimen WA25-1B), the enhancement of maximum strength was limited to about 10% because the wall bars were not confined in the direction of wall thickness due to the absence of tie bars. However, when the spacing was decreased and the tie bars were added (as in specimen WA25-1AP), maximum strength was greatly enhanced.

(2) Number of tie bars
The relationship between normalized maximum strength \( T_u' \) and tie-bar ratio \( P_b \) is shown in Figure 6. The tie-bar ratio is the ratio of the total cross-sectional area of tie bars arranged in a unit area of the wall surface. The maximum strength increased in proportion to \( P_b \) for each wall bar spacing, as shown by ▲ for spacing of 100 mm and
The maximum strength increased because the route of shear stress transmission through both the truss and strut mechanisms (see Figure 6) was formed by the inclusion of tie bars, because the tie bars could transmit shear stress from the inside to the outside of the wall.

### 3.5. Performance of tie bars

Stress distributions of tie bars in some specimens are shown in Figure 7. When the tie bars were arranged with a smaller spacing (100 mm), the effective area of stress transmission through the tie bars spread widely around the beam bar tails by generating the truss mechanism. When the tie bars were arranged with a larger spacing (200 mm), the tie bars near the tips of the tails did not work effectively. Thus, the effective area of tie bars with spacing of 200 mm was smaller than 300 mm within the tail side.

### 3.6. Validity of our previously proposed equation for anchorage strength

We previously proposed the following equation for estimating anchorage strength in a beam-column joint that has failed in raking-out [3]:

\[
\text{cal}T_u = k_N (\text{cal}T_c + \text{cal}T_w),
\]

where \( \text{cal}T_c \) is the horizontal resistance of concrete \( (= k_h \cdot k_c \cdot b_{ew} \cdot L_{j'} \cdot \sigma_e) \), \( \text{cal}T_w \) is the horizontal resistance of lateral reinforcement \( (= k_w \cdot k_b \cdot a_{ew} \cdot \sigma_{wy}) \), and \( \sigma_{wy} \) is the axial stress modification factor \( (= 1 + 0.020 \sigma_o) \).

The definitions of the terms in the above equation are as follows:
- \( k_c \): effective factor for sliding resistance of concrete
  - \( = 0.85 \) when the bar tail is bent toward the beam compressive zone, and
  - \( = 1.20 \) when the bar tail is bent toward the wall-supporting point

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![Figure 7. Distributions of wall-tie bar stress \( T_t \) (kN)](image_url)
\( k_w \): effective hoop stress factor
(\( = 0.8 \) when the bar tail is bent toward the beam compressive zone, and
\( = 0.9 \) when the bar tale is bent toward the wall-supporting point)
\( k_b \): effective factor by the thickness of cover concrete (\( = 1.0 \) when \( C_c \leq 0.8 L_{dh} \))
\( b_{ces} \): effective column width (\( = b_c + 0.53 h_c \))
\( L_{d1}' \): diagonal crack length measured along the horizontal plane (\( = L_{dh} - d_b - C_c \))
\( a_w \): total cross-sectional area of lateral reinforcement crossing failure planes
\( b_1 \): distance between extreme beam bars
\( C_c \): total thickness of cover concrete on both sides
\( D_c \): depth of beam
\( d_b \): diameter of beam bar
\( \sigma_o \): column axial stress (not larger than \( 0.08 \sigma_B \))
\( \sigma_e \): sliding strength of concrete (\( = \sqrt{\sigma_B} \))
\( \sigma_{wy} \): yield strength of lateral reinforcement

A comparison of the calculated and experimental results is shown in Table 4 and Figure 8. The values of the variables in Equation (2) were calculated on the basis of the assumption that the tie bars within the effective area of the walls (the walls of the specimens considered as being wide columns) correspond to lateral reinforcement of the equation. In case in which tie bars were used, Equation (2), which is the sum of concrete resistance (\( calT_c \)) and lateral reinforcement resistance (\( calT_w \)), could be used to estimate anchorage strength of beam-wall joints, since the ratio of calculated values to experimental values for all of the specimens with tie bars (except for specimen WA25-1AP2, which had the smallest tie-bar ratio) were in the range of 0.85 to 0.96. However, further study is needed to determine the reason for the minimum value (0.85) in specimen WA25-1AP3, in which tie bars were alternately spaced at 100 mm and 200 mm. When tie bars were not used, the calculated values (\( expT_u \)) were less than the experimental values (\( expT_u \)) because these calculated values were obtained by only the concrete resistance \( T_c \) and the dowel action of wall bars was disregarded in the calculation. Lateral resistance by the dowel action maybe is estimated as the difference \( T_\Delta \) between the calculated and experimental strengths of WB25-1A (\( T_\Delta = 92.6 = 86 \) MPa), which had the smallest value of \( calT_c \) among all specimens because it had the smallest horizontal

<table>
<thead>
<tr>
<th>Specimen</th>
<th>( calT_c )</th>
<th>( calT_w )</th>
<th>( calT_u )</th>
<th>( expT_u )</th>
<th>( exp/cal )</th>
<th>( calT_u \Delta )</th>
<th>( expT_u )</th>
<th>( exp/cal )</th>
</tr>
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<tbody>
<tr>
<td>WA25-1A</td>
<td>60</td>
<td>—</td>
<td>60</td>
<td>153</td>
<td>2.55</td>
<td>146</td>
<td>1.05</td>
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<td>—</td>
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<td>155</td>
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<td>150</td>
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<td>172</td>
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<tr>
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<tr>
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<tr>
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<td>391</td>
<td>377</td>
<td>0.96</td>
<td>377</td>
<td>0.96</td>
<td></td>
</tr>
</tbody>
</table>

\( \text{Table 4 Comparison of calculated and experimental values} \) [kN]
development length \( L_{dh} \). Using modified strength \( \text{cal}_{Tu} \), which is the sum of the original calculated value \( \text{cal}_{Tu} \) and the dowel resistance \( T_{d} \), the ratios of \( \text{exp}_{Tu} \) to \( \text{cal}_{Tu} \) for specimens without tie bars and for specimen WA25-1AP3 ranged from 0.92 to 1.15, and the experimental and calculated values showed good agreement. However, the ratio for WA25-1B, which had a large number of wall bars, was the largest value (1.15) because the effect of the number of wall bars was disregarded.

The phenomena mentioned above indicated the following results: in the case of no tie bars, dowel resistance predominated in total resistance and it depended on the number of wall bars; in the case of using many tie bars, resistance of tie bars at yielding predominated; and in the case of using few tie bars, both of the resistances needed to be considered to estimate anchorage strength.

4. Conclusions

The anchorage behavior of beam-wall joints using wall panels with 90-degree hooked beam bars at right angles as study specimens was examined. The test variables of the specimens were horizontal and vertical development lengths, lateral reinforcement ratio of tie bars, spacing of wall bars. The conclusions based on the experimental results are as follows,

(1) The inclusion of many tie bars in walls around beam bars, especially around tails, made shear-stress transmission zone become wider and anchorage strength increase, because of the generation of the truss mechanism.

(2) The anchorage strength of hooked bars without tie bars increased by only 10% even though the spacing of wall bars was reduced to by 50%, but the strength was greatly enhanced by inclusion of tie bars.
(3) Our previously proposed equation for estimating anchorage strength in beam-column joints could be applied to estimation of anchorage strength in beam-wall joints with tie bars, dowel resistance caused by wall bars crossing shear crack planes needed to be included in the calculation in order to get good agreement with experiment results.

Acknowledgements

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References