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# Analysis on efficiency of slot-cutting around RC frame joint for "strong column and weak beam" engineering design

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**Abstract**

A numerical model was established with Adina software for beam-column joint on a reinforced concrete frame. The results for the traditional joint and that for the joint with slot around were compared. It is shown that the stress in the reinforce bar for slab close to beam will increase significantly with the applied top drift in horizontal direction, and less for far from the beam. It is also shown that for the joint with slot around, the yielding time of column bar will shift to a later time, while number of cracks on column becomes less. The time interval between the occurrence of plastic pivot on beam and on column becomes longer. The occurrence of column plastic pivot on the first floor would be delayed, which exhibits a failure type closer to the ductile mode.



*Keywords*: frame joint; strong column and weak beam; plastic pivot; yield; enlarging factor for end moment; engineering design.

## 1. Introduction

The principle named "strong column and weak beam" has been specified in the design code for frame design[1]. It is expected that when a frame structure is subjected to earthquake action, all the plastic pivots will appear at the ends of beams before columns, and then after the yielding of the column ends on the first floor, the whole structure will collapse gradually. On the contrary, for the "strong beam and weak column" frame, the plastic pivots will appear on column ends before beams, which will result the abrupt collapse of the structure. It is clear, then that there are two defensive protections for the "strong column and weak beam" structure. During a destructive earthquake, a large amount of energy could be dissipated with the elasto-plastic deformation in the two protections. While, the earthquake disaster investigation and experiment results[2]-[3] have proven that in the collapsed frame with cast-in-situ concrete, most of the plastic pivot will still appear on the column ends, instead of the beam ends, which is the converse behavior of the original expectation. For example, in Wenchuan earthquake, much of the plastic pivot appeared at the column ends, resulting significantly heavier destruction for the columns[4]. It is obviously clear that the object for energy dissipation and collapse delay have not been realized.

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During a destructive earthquake, if it is ensured that the plastic pivots could appear on beams before columns, and most of the input energy be dissipated in deformation, the beams will become an effective protection for the columns. Then, it may be expected that the collapse probability will present a substantial reduction.

It is generally considered that the main factor resulting the unexpected "strong beam and weak column" could be attributed to the reinforce bar in slabs longitudinal to the beam. Up to now, much research work have been devoted to investigate and to solve this problem. For example, in ACI design code, according to the research accumulation in 15 years dated from 1980, from ACI 318-1999, it is specified that the influence of slab should be taken into account in evaluating the resisting the lateral drift in cast-in situ reinforced frame. In ACI 318-08[5], 12.6.2, it is specified that the bending resisting capacity of column should follow $\Sigma M_{nc} \geqslant 1.2 \Sigma M_{nb}$, and more over, in cast-in-situ slab, the reinforce bar within the effective width of the flange should be counted in computation for $M_{nb}$.

where:

$M_{nc}$-Summary of the column bending resisting capacity, include of upper column end and lower column end, either in clockwise direction or in counter clock direction. It should be computed according to the specification about eccentric compression members, taking the minimum bending resisting capacity under the axial forces resulted from gravity load, and the lateral load in respectively the same direction as the bending resisting capacity.

$M_{nb}$-Summary of the beam bending resisting capacity, taking the beam end moment at outer surface across the beam. While for the cast-in-situ slab, which forms a T beam, the contribution to the summary from reinforce bar within the effective width of flange $b_f$ should be taken consideration. $b_f$ could be computed as follows:

1) $b_f$ should be less than 1/4 span of the beam;

2) to take the hanging length of flange:

a) For a T type beam, it may take the less one between 8 times slab thick and 1/2 net space between beams;

b) For an L type beam, it may take the least one of 6 times slab thick, 1/2 net space between beams and 1/12 span.

It is demonstrated from experiment results that for a T type beam with its flange being within the tensile area, the contribution to beam bending resisting capacity is closely related to the value of inter-storey drift. A large contribution will correspond to a large inter-storey drift[2]. In ACI318-08, the specification about $h_f$ corresponds to the inter-storey of 2%.

In design practice for reinforced concrete frame in China, it is a traditionally approach to multiply the rigidity coefficient of the rectangle cross-section of beams with a factor of 2.0 or 1.5, in order to consider the enhancement of slabs to beams[6]. This will lead to the increase in the computed internal forces and reinforcement in beams, while this extra part of beam reinforcement is lay in the rectangle part of the beam, and the slab reinforcement is still set within the effective width of flange, which, in fact, leads to the enhancement of beam bending resisting capacity. It is clear that the slab reinforcement play the dominant role in the unexpected converse from "strong column and weak beam" to "strong beam and weak column", which introduces the deviation for the expectation of "strong column and weak beam".

In 2009, through the analysis for a 6 floor frame, Su You-po and Zhang Yu-min[4] suggested that the specification will not be satisfied for every joint if the slab reinforce bars are to be taken in to account to the beam bending resisting capacity. The influence to the internal joint is the most significant. Further more, the hardening of beam and slab reinforce bar after yielding will even strength this tendency, which will worsen the case. It is clear that the slab reinforce bar should be taken into consideration.

At present, it is rather a difficult task to make retrofit to "strong beam and weak column" structures, either to the dimension of cross-section or to increase the reinforce ratio for columns. It has become an important subject under intensifying investigation[7-10]. In this paper, according to the modified improvement for "strong column and weak beam" suggested by Prof. Su You-po, a numerical analysis model was established to evaluate the efficiency of "strong beam and weak column". With this method, slots will be cut in the slab around the joints, which practically cut the reinforce bars near the beams to suppress their contribution to beam bending resisting capacity relatively.

## 2. Structural analysis model

For the numerical model, a 6 floor 2x3 span frame structure was designed according to the present design code, with a even distribution of mass and rigidity along height and on every floor, fortifying earthquake intensity 8, II class ground, II earthquake resisting grade for frame. In order to reduce the amount of computing work, the numerical model was established with scale 1:2, for a single interior truss from the frame with slab thick 100mm, beam span 3600, beam cross-section 150x300mm, storey height 1950mm, exterior column 220x220mm, interior column 300x300mm, as is shown in Fig.1. The amount of reinforcement was computed with SATWE in PKPM08, with the rigidity coefficient factor 2.0 for interior joint and 1.5 for exterior. The influence of filler wall was neglected.

The cross-section and reinforcement are shown in Fig.2, with concrete grade C30, as is given in table 1, stirrupΦ10@100 for exterior with the intensified area along the total height, Φ10@100/150 for interior column with the intensified area 500mm, Φ10@100/150 for beam with the intensified area 700mm.

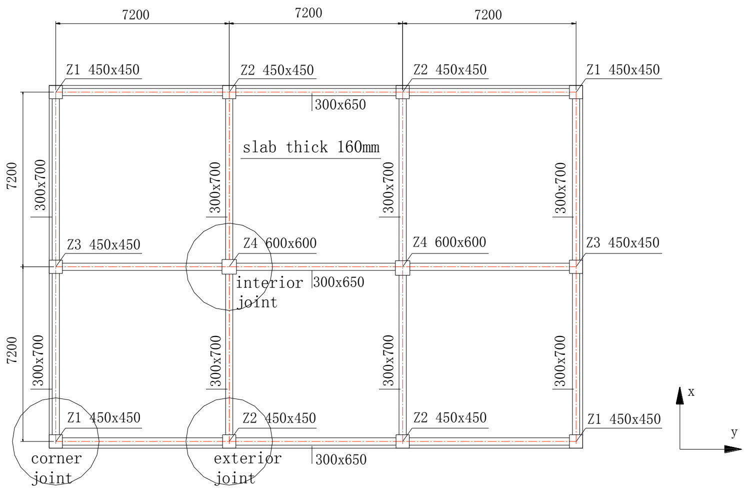



Fig.1. Structure plane

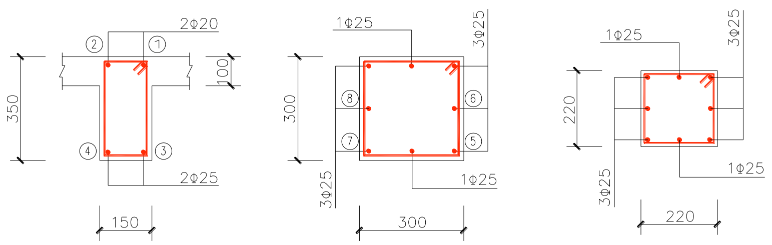



Fig.2. Reinforcement of columns and beams. (a) Frame beam; (b) Interior column; (c) Top

Table 1. Parameters for concrete material

| density t/m$^3$ | Poisson ratio | Yong's /Pa | uni-axial tension /Pa | residual stress/Pa | maximum stress/Pa | strain in maximum stress/Pa | ultimate stress/Pa | ultimate stress/Pa |
|---|---|---|---|---|---|---|---|---|
| 2.4 | 0.2 | 2.2e9 | 1.27e6 | 6e5 | 1e7 | 0.00164 | 7e6 | 0.0038 |

Table 2. Parameters for reinforce bar

| reinforce bar | grade | Yong's/ MPa | Poisson ratio | yield point | hardening modulus/ MPa | hardening type |
|---|---|---|---|---|---|---|
| beam column | HRB335 | 2.1e5 | 0.3 | 335 | 1.0e3 | isotropic |

| | | | | | | |
|---|---|---|---|---|---|---|
| slab | HPB235 | 2.0e5 | 0.3 | 235 | 1.0e3 | isotropic |
| stirrup | HPB235 | 2.0e5 | 0.3 | 235 | 1.0e3 | isotropic |

The numerical model was formulated with ADINA software. The specified material model for concrete was prescribed in the software which enables it to simulate the cracking, crushing and softening of concrete. The bilinear elasto-plastic model was assigned to reinforce bar. The software could to provide the stickiness for reinforce bar to concrete. The dynamic-implicit algorithm was introduced for an unconditional stable object at larger load steps and fine accuracy. A quasi-static displacement load was applied to truss top along X direction, with step length 1cm, till the appearance of concrete crushing or breaking off of computation.

## 3. Stress distribution for reinforce bar in slab

Form the disaster investigation after Wenchuan Earthquake in 2008, it is generally considered that the collapse will most probably occur at joints on the second floor, only a few occurs beyond the third floor, resulted from the possible defects in structure because of undesirable design or construction procedure. Analysis work was thus concentrated to the joints on second floor of the structure in Fig.1.

The stresses in the slab bars at top and bottom for the second floor are given in Fig.3. d is distance from the centroid of bar to the lateral side of the beam. δ/H is the normalized horizontal drift at structure top.

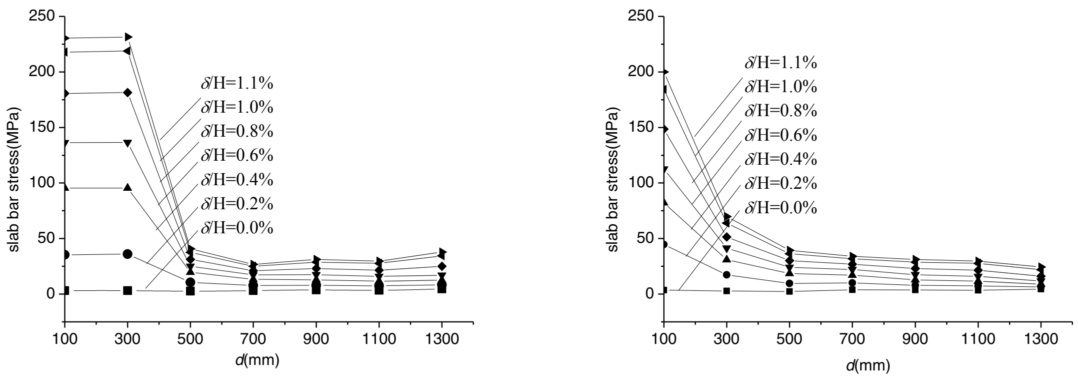



Fig.3. Changing of the stress in the reinforce bar for slab with the applied top displacement. (a) Top; (b) Bottom

It is shown in Fig.3 that with the increasing of horizontal drift, the bar stresses will also increase, while the uneven distribution indicates that only part of the slab bars contribute to the beam resistance to bending. Since the stresses showed significantly increase near the beam, it is clear that slab bar near the beam make larger part of the contribution to the increase of beam resistance to bending. When the horizontal drift δ/H increase to 1.1%, the slab bar stresses on slab top at distance 100mm and 300mm from the beam will increase to 230.4Mpa and 231.5Mpa respectively, close to its yielding point, as is shown in Fig.3(a). The slab bar stresses on slab bottom are similar to the slab top, as is shown in Fig.3(b). This part of contribution has not been specified in the present Chinese design code, which seems should not be neglected in evaluating the effect of “strong column and weak beam”. It is also shown that slab bar stresses tends to be a constant beyond 500mm from the beam, indicating the independence from distance. It could be concluded, at this moment, that slab bars beyond 500mm from beam will make no contribution to beam resistance to bending, but only to slabs. Wang Su-guo, from South China Polytechnic University, has suggested the effectiveness width flange of beam from the analysis results with ABAQUS soft ware, which shows the approximately coincidence to the results in this paper[11-12].

## 4. Analysis for the efficiency of “strong column and weak beam”

In order to suppress the contribution of slab bar as has been demonstrated in the previous section, slots are cut in the slab around the joint. In fact, one or more of the slab bars in a quadrant around the joint will be cut off, to remove their influence through axis forces, and the specified "strong column and weak beam" may be expected, as is shown in Fig.4. In order to investigate the efficiency of slot cutting to "strong column and weak end", the axial forces of the beam bars and column bars for the traditional joints and the joints with slots were picked out and were plotted vs. the drift, and then were compared. The efficiency of slot cutting was evaluated according to the appearance order of plastic pivot resulted from yielding of reinforce bar respectively.

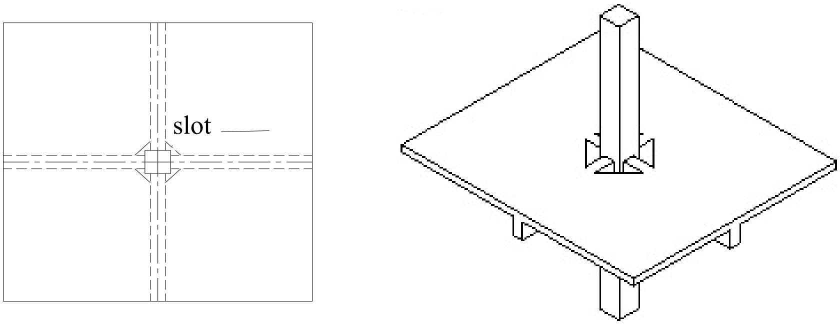



Fig.4. Model of beam-column joint with slot around. (a) Interior joint plane; (b) Model of interior joint

When subjected to horizontal drift quasi-static loading, tensile stresses will appear at a single side of both the upper column end and lower column end, and on both the top and bottom of the right beam and the left beam respectively. Since tensile stress of reinforce bar is critical to the formation of plastic pivot, attention was concentrated to the tensile bars and cracks in concrete. The stresses vs. horizontal drift are plotted in Fig.5, and the numbering of the bars is shown in Fig.2.

*4.1. Traditional joint-no slot.*

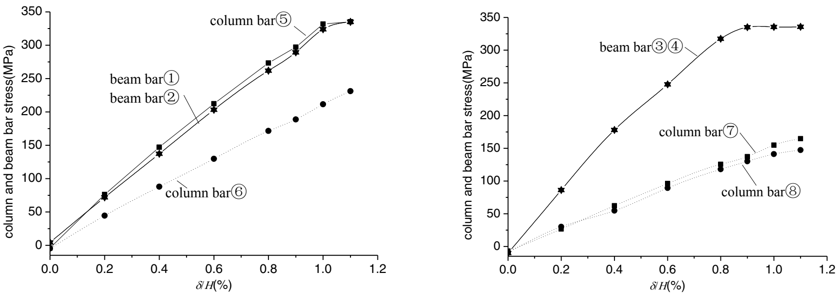



Fig.5. Bar stresses for traditional joint vs. horizontal drift at top. (a) Bars in upper column and right beam; (b) Bars in lower column and left beam

Table 3. Column and beam bar stress for traditional joint vs. horizontal drift. /MPa

| Bars / δ/H(%) | 0.0 | 0.2 | 0.4 | 0.6 | 0.8 | 0.9 | 1.0 | 1.1 |
|---|---|---|---|---|---|---|---|---|
| Right beam bar① | 4.3 | 72.0 | 137.2 | 203.4 | 261.9 | 289.4 | 323.9 | 335.1 |
| Right beam bar② | 4.3 | 72.0 | 137.2 | 203.4 | 261.9 | 289.4 | 323.9 | 335.1 |
| Left beam bar③ | -10.1 | 86.3 | 178.0 | 247.8 | 317.4 | 335.1 | 335.4 | 335.7 |
| Left beam bar④ | -10.1 | 86.3 | 178.0 | 247.8 | 317.4 | 335.1 | 335.4 | 335.7 |
| U column bar⑤ | -4.2 | 76.3 | 147.2 | 212.2 | 273.4 | 297.2 | 331.8 | 335.2 |
| U column bar⑥ | -4.8 | 44.3 | 87.8 | 129.7 | 171.4 | 188.8 | 211.5 | 231.1 |
| L column bar⑦ | -7.3 | 26.6 | 62.3 | 96.1 | 125.7 | 137.2 | 155.0 | 164.8 |
| L column bar⑧ | -9.2 | 30.1 | 54.6 | 89.3 | 118.0 | 130.1 | 141.2 | 147.3 |

*Figures underlined indicate yielding stress

It is seen from Fig.5 that stresses for the reinforce bar at upper column end is rather large. For example, the stresses in column bar ⑤ are almost the same as the stresses in the beam bar ① and ②, with the yielding time being almost the same. While on the other hand, the stresses in column bar ⑥ is relatively small. The bottom of the upper column is the most dangerous cross-section. The tensile stress in beam bar ① and ② are completely the same.

When the top drift reaches 1.1%, the stress in the top bars for the right beam will completely yield, as is shown in Table3. A large amount of concrete cracks appeared which may be regarded as the plastic pivot at the beam. One of the column bars ⑤, also reaches yielding when the top drift reaches 1.1%. It is then seems that the plastic pivot on beam appeared at the same time as the column, but the difference small. It is hard to say that the traditional joint will be of “strong column and weak beam”, as is expected, because of many other affecting factors, such as the discreteness and the uncertainty of construction.

### *4.2. Slot cutting joint-the side length 200mm.*

When the side length of the slot is 200mm, as shown in Fig.4, the slab bar within the slot are cut off, one for top and the other for bottom, in both directions, for a single quadrant. The stresses in the column bar and beam bar are shown in Fig.6.

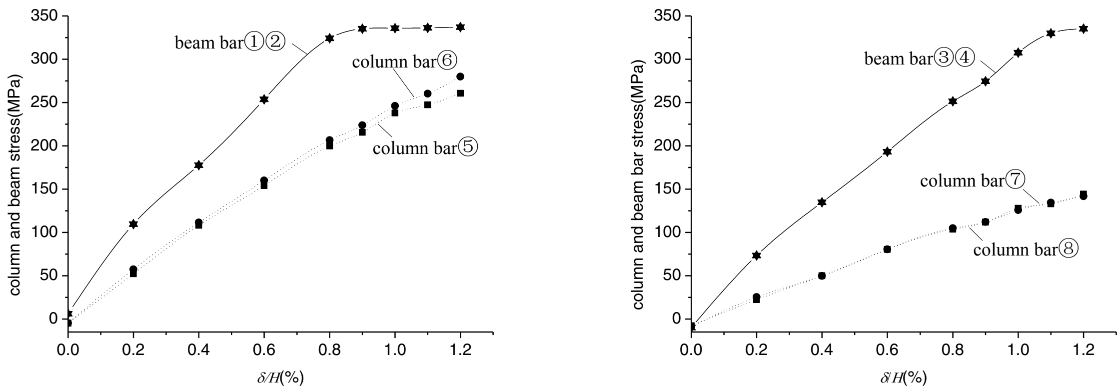



Fig.6. Bar stresses for joint with slot of side length 200mm vs. horizontal drift at top. (a) Bars in upper column and right beam; (b) Bars in lower column and left beam

Table 4. Column and beam bar stress for joint with slot of side length 200mm vs. horizontal drift. /MPa

| Bars / δ/H(%) | 0.0 | 0.2 | 0.4 | 0.6 | 0.8 | 0.9 | 1.0 | 1.1 | 1.2 |
|---|---|---|---|---|---|---|---|---|---|
| Right beam bar① | 6.2 | 109.7 | 177.7 | 253.7 | 324.1 | 335.2 | 335.8 | 336.4 | 337.2 |
| Right beam bar② | 6.2 | 109.7 | 177.7 | 253.7 | 324.1 | 335.2 | 335.8 | 336.4 | 337.2 |
| Left beam bar③ | -9.8 | 73.3 | 134.8 | 193.2 | 251.5 | 274.6 | 307.6 | 330.0 | 335.2 |
| Left beam bar④ | -9.8 | 73.3 | 134.8 | 193.2 | 251.5 | 274.6 | 307.6 | 330.0 | 335.2 |
| U column bar⑤ | -4.7 | 52.3 | 108.1 | 154.1 | 199.8 | 215.5 | 237.8 | 247.4 | 260.6 |
| U column bar⑥ | -4.7 | 57.4 | 111.5 | 160.1 | 206.5 | 223.8 | 246.0 | 260.1 | 279.9 |
| L column bar⑦ | -7.9 | 22.3 | 50.0 | 80.2 | 103.6 | 111.9 | 127.8 | 133.0 | 144.1 |
| L column bar⑧ | -8.7 | 25.4 | 49.8 | 80.4 | 105.0 | 112.0 | 125.7 | 134.4 | 141.8 |

*Figures underlined indicate yielding stress

It is seen in Fig.6 that under the same lateral drift, there is significantly increase in beam bar stress for the joint with slots comparing to the traditional joint. The yielding took place when δ/H is 0.9%. The maximum stress for beam bars ①② increase by about 16%. Moreover, the number of the yield elements increase to 4 instead of 1 for the traditional joint, indicating the enlargement of dimension of the plastic pivot, which is a desirable behavior of the possible collapse procedure.

When the horizontal drift is 1.1%, stress of the column bar ⑤ decrease to 247.4MPa from the previous 335.1 MPa of the traditional joint. Then, when the horizontal drift is 1.2%, the maximum stress for the upper column is 260Mpa. Although for the same horizontal drift, the stress in column bar ⑥ exhibited a little increase, the maximum stress is only 279.8Mpa, far less than yielding point. The stresses in column bar ⑦⑧ decreased by 13% and 4% respectively.

When the horizontal drift reached 1.2%, with the crushing of some concrete, the computation procedure encountered non-convergence, but column bar ⑤⑥⑦⑧ still suffered no yielding. The column bar stresses decreased significantly. The appearance of plastic pivots at column ends and collapse of the columns were delayed effectively, which means the decrease in collapse possibility of the whole structure. Slot cutting (cutting off slab bar in fact) could weaken the enhancement of the beam resistance to bending.

### *4.3. Slot cutting joint-the side length 300mm.*

When the side length of the slot is 300mm, the slab bars, two for top and two for bottom surface, in both directions for a single quadrant, were cut off. A more significant efficiency may be expected. The stresses in the column bars and the beam bars are shown in Fig.7.

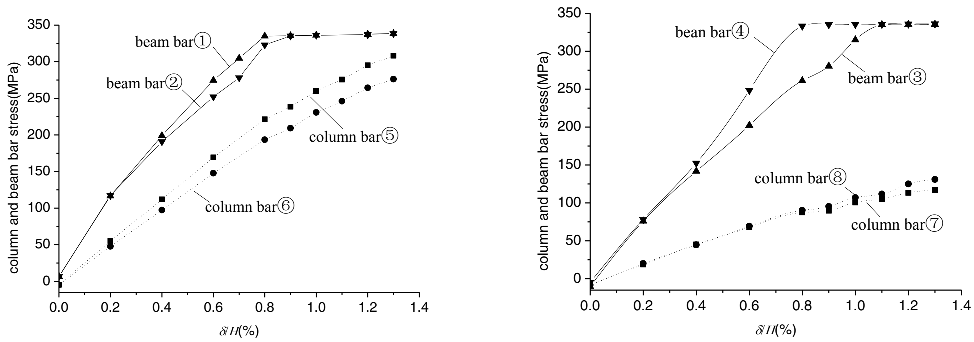



Fig.7. Column and beam bar stress for joint with slot of side length 300mm vs. horizontal drift at top. (a) Bars in upper column and right beam; (b) Bars in lower column and left beam

Table 5. Column and beam bar stresses for joint with slot of side length 300mm vs. horizontal drift. /MPa

| Bars / δ/H(%) | 0.0 | 0.2 | 0.4 | 0.6 | 0.8 | 0.9 | 1.0 | 1.1 | 1.2 | 1.3 |
|---|---|---|---|---|---|---|---|---|---|---|
| Right beam bar① | 6.7 | 116.6 | 199.0 | 274.6 | 335.1 | 335.5 | 336.3 | 336.9 | 337.6 | 338.3 |
| Right beam bar② | 6.4 | 117.5 | 190.8 | 252.0 | 322.9 | 335.3 | 336.2 | 336.8 | 337.7 | 338.4 |
| Left beam bar③ | -10.2 | 76.0 | 141.8 | 202.3 | 261.1 | 280.3 | 314.9 | 335.0 | 335.2 | 335.4 |
| Left beam bar④ | -4.8 | 38.0 | 152.4 | 248.4 | 333.2 | 335.1 | 335.4 | 335.5 | 335.8 | 336.0 |
| U column bar⑤ | -4.7 | 55.1 | 111.9 | 169.3 | 221.3 | 238.4 | 259.8 | 275.7 | 295.0 | 308.3 |
| U column bar⑥ | -4.7 | 47.7 | 97.3 | 147.6 | 193.5 | 209.1 | 230.6 | 246.0 | 264.4 | 276.3 |
| L column bar⑦ | -7.9 | 18.9 | 45.0 | 68.1 | 87.5 | 89.8 | 100.5 | 105.2 | 113.2 | 116.7 |
| L column bar⑧ | -8.3 | 20.0 | 44.6 | 69.3 | 90.0 | 95.2 | 107.0 | 111.7 | 124.9 | 130.8 |

*Figures underlined indicate yielding stress

It is seen from Fig.7 and Tab.5 that when the horizontal drift is 0.8%, the beam bar①②at the right beam end began to yield. The stress for both the upper column and the lower column bars will decrease for the same horizontal drift. When the drift is 1.1%, the stresses for the column bar ⑤⑥ exhibited a little variations comparing to side length 200mm, but they were still far less than yielding stress. The stresses for the column bar ⑦⑧ decreased by 21% and 17% respectively, which presented more significant efficiency.

It must be mentioned that the slot can only have influence to suppress the beam resistance to bending for a T cross-section with flange in the tensile area. If a quasi-static load is imposed in a single direction, only the beam in one of the two sides is affected. For the beam in another side, the flange is in pushing area, the beam bar ③④ are in tensile state, the slot in the flange does not have any effect. For the traditional joint,③④ yielded when drift is 0.9%; for slot with 200mm side length,③④ yielded at drift is 1.1%; for slot with 300mm side length, ③④ again yielded at 0.9%. Anyhow, the practical earthquake action is in reciprocating movement, the slot at both side will correspond to the beam for the two sides and play their role in effect. The beam bars at the top surfaces for both sides will yield as stated above. The stress and strain behavior of beam bars at the bottom surface does not has any effect to “strong column and weak beam” behavior.

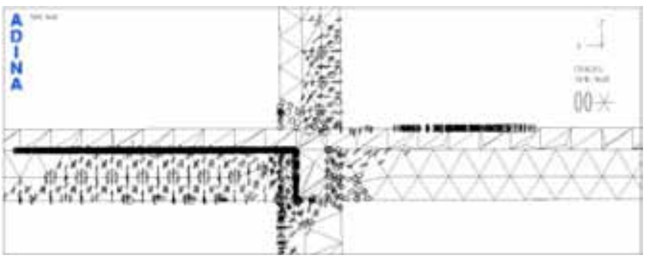
To sum up, the stresses for the beam bars and the column bars are shown in Table 6.

Table 6. Summary of bar stresses in beam

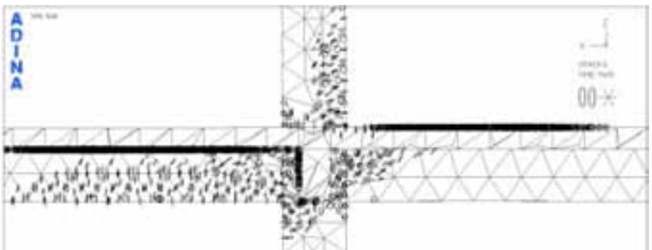
| joint type | top drift when beam yielding(%) | column bar stress when beam yielding (MPa) | | | |
|---|---|---|---|---|---|
| | | upper column | | lower column | |
| traditional joint (no slot) | 0.9 | 335 | -- | 164 | -- |
| slot with side length 200mm | 0.9 | 223 | decrease 33% | 115 | decrease 30% |
| slot with side length 300mm | 0.8 | 221 | decrease 34% | 90 | decrease 45% |

It is clear that after slot cutting, the yielding of column bars will shift to a later moment. When the beam bars began to yield, the stresses in the column bars will be significantly lower than traditional joints, and they are also far less than yielding stresses. Slot cutting can improve the failure mode of beams and columns, by adjusting the appearance of plastic pivots. It can establish more efficient protection in multiply folds, in expecting the desirable “strong column and weak end” effect.

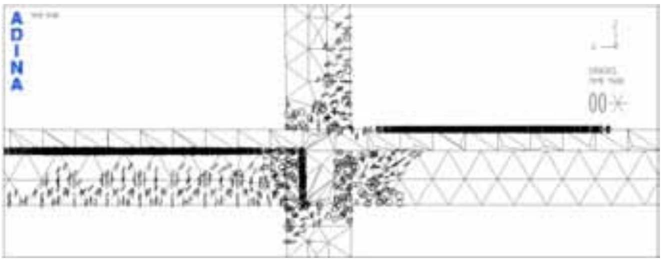
*4.4. Concrete cracks.*




(a) Traditional joint (no slot)




(b) Joint with slot of side length 200mm




(c) Joint with slot of side length 300mm

Fig.8. Cracks in the beam-column joints when the beam start to yield

The information about cracks of column and beam concrete are shown in Fig.8.The amount of cracks on columns exhibits decrease, while the cracks on beams seems experience little increase in the tensile area, which coincides with the computation results for the column and beam bar stresses.

*4.5. Seismic behavior of column on the first floor.*

Since the failure of column on the first floor is unavoidable, what can be done is to postpone its occurrence as late as possible, or to postpone its occurrence later than their occurrence on beams. It is seen from Tab.7 that the column bars on the first floor began to yield at drift of 0.8% for the traditional joints, while for the joints with slots (side length 200mm or 300mm), the column bars began to yield at drift is 1.0%, which exhibits an effective delay and increase in structure ductility.

Table 7. Comparison for bar stresses at column on the first floor/MPa

| joint type | bars / δ/H(%) | 0.6 | 0.8 | 0.9 | 1.0 | 1.1 | 1.2 | 1.3 |
|---|---|---|---|---|---|---|---|---|
| traditional joint | col.ext.1 | 230.6 | 302.7 | 330.7 | 335.3 | 335.5 | — | — |
| no slot | col.ext.2 | 255.5 | 333.1 | 335.2 | 335.6 | 335.9 | — | — |
| side length | col.ext.1 | 227.0 | 294.7 | 321.4 | 335.3 | 335.7 | 336.3 | — |
| 200mm | col.ext.2 | 221.3 | 286.6 | 312.2 | 335.2 | 335.6 | 336.1 | — |
| side length | col.ext.1 | 202.1 | 263.1 | 286.1 | 323.0 | 335.2 | 335.6 | 335.9 |
| 300mm | col.ext.2 | 215.2 | 276.0 | 299.6 | 335.0 | 335.3 | 335.7 | 336.0 |

## 5. Conclusion

(1) For the traditional cast-in-situ reinforce concrete frame joint, the stresses in slab bars will experience significantly increase with the horizontal drift. The increase is larger near the beam and is smaller away from the beam. This shows an uneven distribution and ineligible enhancement in the beam resistance to bending.

(2) Slot-cutting on slab around joints (cutting off the slab bars in fact) could lead to later appearance of plastic pivots on column ends to enlarge the time interval between plastic pivots on beam ends and on column ends, which shows rather efficiency for “strong column and weak column”.

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