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EXPERIMENTAL STUDY ON T-SHAPED BEAM-COLUMN JOINTS ANCHORED BY ATTACHMENTS OF SCREW NUTS TO COLUMN REINFORCEMENTS

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SUMMARY

Several specimens were tested to find out an optimum design, while mechanical devices for anchor, shear strength ratio of the joints and additional shear reinforcements into the joints were varied as experimental variables. Failure of concrete and yielding of steel of a series of specimens was observed and a reinforcing method was applied for another series in consideration of simplifying the construction practices. It had been confirmed that more than 0.04 radian of relative story displacement angle could be produced under severe alternative load reversals while over 80 % of load bearing capacities remaining. Thus ductility up to plastic deformation range could take place under the conditions as follows: The ratio of joint shear strength should be more than 1.4 which was calculated using the past experimental equation of the force transfer into the joint at the time of column yielding. Not only hoops of column but also stirrups of beam must be arranged as more than 0.3 % of joint shear reinforcements (the minimum requirements for joint design). Screw nuts must be attached as mechanical devices and confined by additional spiral reinforcements along the development length, because the confinement of concrete does not enough surround the bar which is located at a corner of the column section.

INTRODUCTION

In our country, deformed reinforcing steel bars which have ribs as thread of screw have been frequently used for reinforced concrete construction (hereinafter referred to as screw steel bar). At the same time both connecting reinforcements together and anchoring them into concrete have been also executed putting splice nuts and anchor nuts on the 'screw' steel bars instead of laps and hooks.

Super high-rise reinforced concrete buildings higher than 60 m should be designed using high strength material, such as steel bars as strong as 390 N/mm^2 at yield point and concrete as stronger than 36 N/mm^2 of compressive strength. The high strength steel bar with larger than 38 mm of diameter won't be bent in a smaller radius. Moreover it is nearly impossible for the hooked or bent steel bars to be arranged densely within the exterior beam-column joints. Consequently the ends of longitudinal reinforcements of beams and columns are set up with mechanical anchors to be embedded within the joint concrete.

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The beam-column joints at the roof floor are not usually designed supposing yielding mechanism of the partial frame, because the calculated response under severe earthquake motion would not be so great there. But many constructors will wonder if they can apply newly developed mechanical anchors for the construction of a building which has been designed basically without any mechanical devices. There are two reasons why they cannot be free to use these devices. First, development length of reinforcement in a joint is apt to be shorter than the structural requirement. Secondly ductility of the partial frame after yielding of members is not generally guaranteed as reserve power which is required to be able to acquire the official appraisal from the Japanese official organization.
Recently many construction methods have been developed concerning mechanical anchor for development of the longitudinal bar into the joint. The paper chooses a mechanical device of screw nut as shown in **Figure 1**. The steel bar has ribs of screw thread. A screw nut is put on the end of the bar and has a flange to transfer bearing stress from the bar to concrete. After setting the nut epoxy resin adhesive is poured into space between the nut and the bar to fix play.

When people use the mechanical devices for the development of beam bar into the joint they expect anchorage failure as shown in **Figure 2** and design the development length based on the current experimental equation indicated by Kubota [1]. He defined a concrete failure mode of crush and coming off outside of a joint and proposed a tensile strength equation of a steel bar when it was pulled out from a reinforced concrete beam. When compressive strength of concrete is stronger than 60 N/mm^2 the tensile strength can be calculated as more than 1000 N/mm^2 by the equation where the development length is longer than 12 times of the bar diameter. Because the strength exceeds one of the steel which they use for the general constructions in Japan, I wondered whether the strength can be displayed or not, under severe shear failure of the joint concrete.

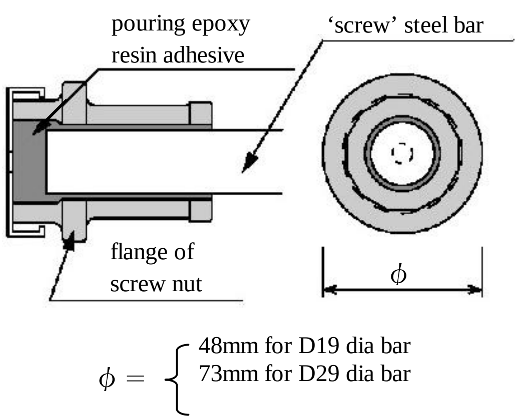


Figure 1. **Mechanical device**

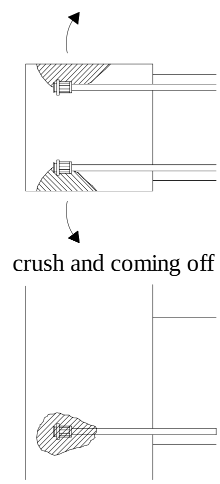


Figure 2. **Failure mode**

The Architectural Institute of Japan [2] recommended the bar arrangement detail in a T-shaped beam-column joint on the top floor (hereinafter referred to as T-joint). In 1979 it had not so strict recommendation, but it recommended more precise detail of bar arrangement around a T-joint in a revised edition published in 2003. When the development of column reinforcement is bent to be attached by a 180 degree hook, the steel bar may touch the other reinforcement due to high density of steel. It is the reason why placing bars will become difficult (see **Figure 3. (A)**). On the other hand space between column reinforcement and beam reinforcement will be large, and confinement round the beam reinforcement will reduce and bond failure will be apt to take place. Thus situation of bar arrangement will be improved by mechanical anchorage system and the Institute's recommendation about arrangement of several confinement steel bars in the joint which are shaped in inverse U (see **Figure 3. (B)**). In the recommendation the inverse U shaped confinement should have the same diameter as column reinforcement and transfer stress of splice, when full strength of column reinforcement is produced at the critical section. Because thus recommendation is not realistic for building construction they will give up keeping the recommendation and then plan to change the structural design (projecting short columns on the roof floor) or to adopt a new construction method which has been appraised by the official organization.

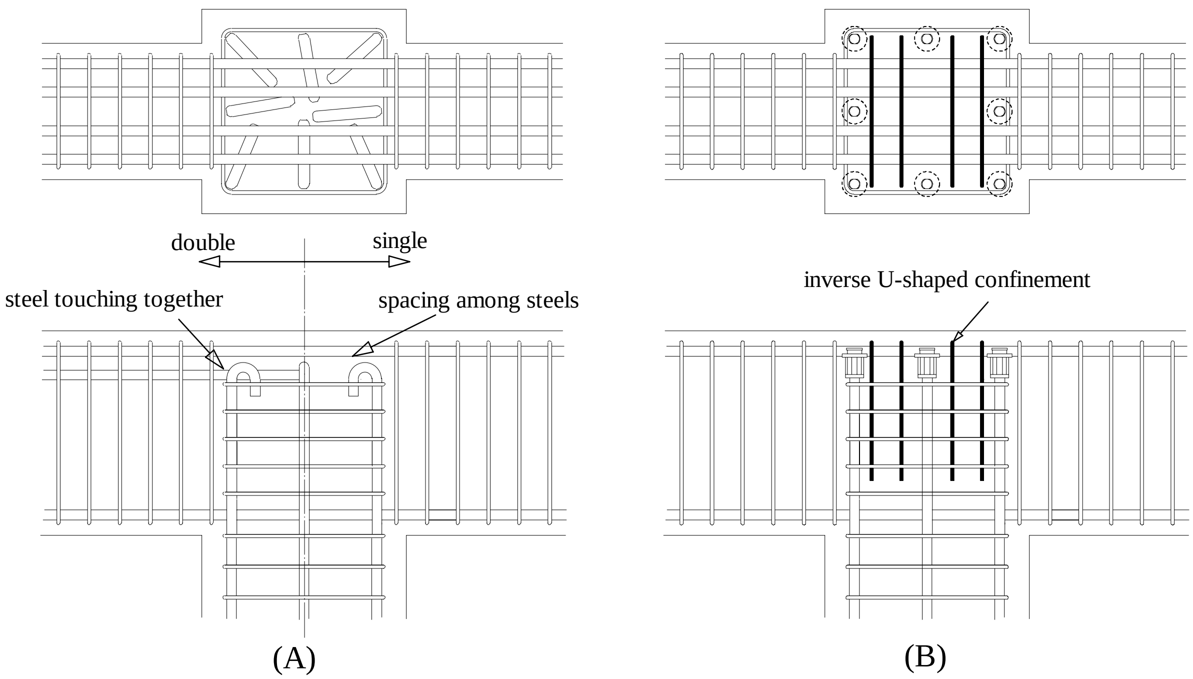


Figure 3. Bar arrangement detail in T-joint

The paper describes three series of dynamic tests which were conducted to investigate the anchorage behavior in the joints. Lateral force and internal force transfer to the frame as shown in **Figure 4**. First the exterior beam-column joints located at the middle floor (hereinafter referred to simply as exterior joints; see **Figure 4**) were tested to get the ultimate tension of the beam bars. Secondly reduced sized T-joints were tested to prove effective confinement by a large amount of vertical lateral reinforcements. Finally large scaled T-joints were tested to acquire prototype of practical bar arrangement.

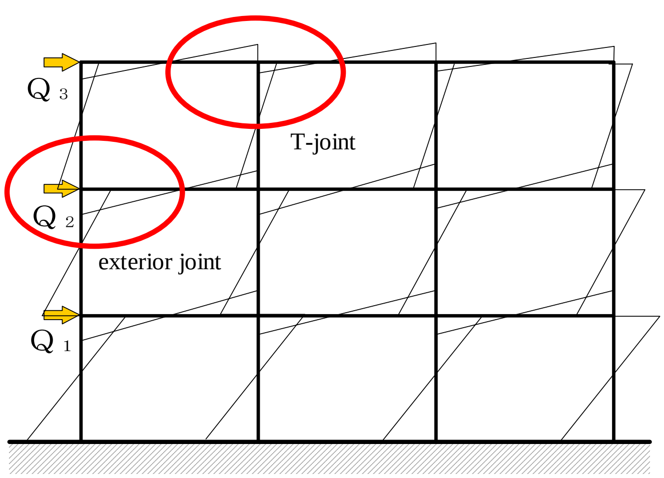


Figure 4. Internal force and joint

THE ULTIMATE TENSION OF BEAM BARS IN EXTERIOR JOINTS

Object of tests

Kubota [1] proposed an equation concerning the tensile strength of beam reinforcement at development (Psu) as follows:

Psu=Sum(fsu*at*beta) (1)

where fsu (N/mm^2) is the maximum tensile stress of a beam reinforcement which is located nearest to outside of the joint and has made concrete crush and coming off, at (mm^2) is area of each beam reinforcement and beta(=0.8) is reduction factor.

fsu=k1*k2*k3*Sigmastd (2)

where k1 (=1.0) is an effective factor concerning the bearing area ratio of development device, and k2 is an effective factor concerning concrete cover thickness of the development reinforcement as follows:

k2=0.96+0.01*(C_0/db) (3)

where , C_0 (mm) is distance from concrete surface to sectional center of the nearest beam reinforcement and db (mm) is nominal diameter of the reinforcement.
k3 is an effective factor concerning lateral reinforcement ratio (Pwjc) of a couple of hoops between which the beam reinforcements are put, should be calculated as follows:

k3=62.5*Pwjc-1.21*Pwjc*(Fc-27.2)+1 (4) when Pwjc is equal to or less than 0.04
k3=1.25-0.0051*(Fc-27.2) (4') when Pwjc is more than 0.04

Pwjc=2*aw/ (bc*x) (5)

where aw (mm^2), bc (mm) and x (mm) is defined in **Figure 5**.
Sigmastd (N/mm^2) is the maximum tensile stress of the bar pulled out of concrete and calculated by the experimental equation as follows:

Sigmastd= 101*$Fc^{0.5}$ (6)

where Fc (N/mm^2) is concrete design strength and ranges between 21 N/mm^2 and 60 N/mm^2.

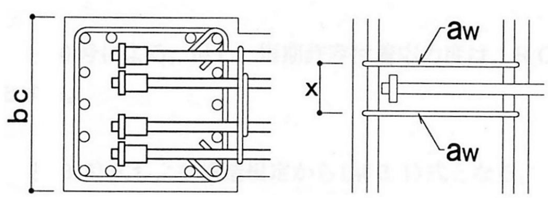


Figure 5. Detail of a couple of hoops

The goal of these series of tests is to verify the tensile strength of beam reinforcement at development which is expected by the abovementioned equation. The exterior joints will be subjected to severe shear forces and the concrete will fail. From the information of these tests we can understand the considerable reduction and the deterioration of the strength of development under severe shear failure.

Test specimens and testing method

Four size reduced specimens were provided for the tests. Each specimen was a partial frame of an exterior joint. In order to make joint concrete crush and come out, yield strength of beam reinforcements had been raised from 490 to 850 N/mm^2 by quenching, because steel bars having such high strength were usually not available. Test specimens had two experimental variables, that is, material strength and amount of beam reinforcements. Concrete strength was 36 N/mm^2 for two specimens having amount of beam reinforcements of 3-D19 that means three tension reinforcements with nominal diameter of 19 mm and 60 N/mm^2 for another two specimens having 4-D19. Two specimens which were reinforced with stronger reinforcements had five-cornered diagonal reinforcements vertically surrounding the joint because they could prevent from large joint shear deformation after beam yielding. The performance of the diagonal reinforcements has been proved by Ishibashi [3]. Every specimen had a couple of hoops between which a beam development is placed. This indicates that a reinforcement ratio of 0.4 % might be adopted in the current structural design. Kubota [1] recommended the development length in the joint to be more than 3/4 times of column depth (D), but there were the length of 12 times of bar diameter (db) in the plots of experimental data. The latter length was adopted for specimens as a strict condition.

the dimensions and steel bar arrangements of four test specimens which were named as E, F, G, and H respectively are shown in **Table 1** and **Figure 6**. And mechanical properties of materials for the specimens are also shown in **Table 2** for concrete at the time of loading and in **Table 3** for reinforcements.

Table 1. Demensions and reinforcements (for exterior joints)

specimen	E	F	G	H
beam				
section	288mm×400mm			
effective depth	360mm			
beam reinforcement (standard)	4-D19 (SD850)	3-D19 (SD850)	4-D19 (SD490)	3-D19 (SD490)
Pt	1.10%	0.82%	1.10%	0.82%
stirrup	4-D10@125 (SD295A)	3-D10@125 (SD295A)	4-D10@125 (SD295A)	3-D10@125 (SD295A)
Pw	0.79%	0.59%	0.79%	0.59%
span length	1875mm			
column				
section	350mm×350mm			
effective depth	310mm			
column reinforcement (standard)	14-D19 (SD390)	12-D19 (SD490)	14-D19 (SD390)	12-D19 (SD490)
Pt	1.2%	1.1%	1.2%	1.1%
hoop	4-D10@110 (SD295A)			
Pw	0.74%			
span height	2000mm			
joint				
hoop	2-D10 (SD295A) 4sets			
diagonal reinforcement	D13 (SD685)	D13 (SD685)	–	–
concrete strength	60N/mm^2	36N/mm^2	60N/mm^2	36N/mm^2

Pt = tension reinforcement ratio

Pw = web reinforcements ratio

Table 2. mechanical properties of reinforcements (for exterior joints)

diameter standard	yield strength (N/mm^2)	tensile strength (N/mm^2)	elongation (%)	young's modulus $\times 10^5$ (N/mm^2)	strain at yielding ($\times 10^{-6}$)
D10 (SD295A)	513	603	15	1.87	2744
D13 (SD685)	794	1051	13	2.13	3730
D19 (SD390)	439	665	20	1.95	3020
D19 (SD490)	559	762	17	1.95	3352
D19 (SD850)	1018	1018	8	1.95	5214

Table 3. mechanical properties of concrete (for exterior joints)

specified design strength	age (day)	compressive strength (N/mm^2)	young's modulus $\times 10^4$ (N/mm^2)	splitting strength (N/mm^2)
60 (E type)	36	66.1	3.80	4.78
36 (F type)	30	47.1	3.30	4.00
60 (G type)	41	58.4	3.67	3.46
36 (H type)	35	34.3	3.03	2.48

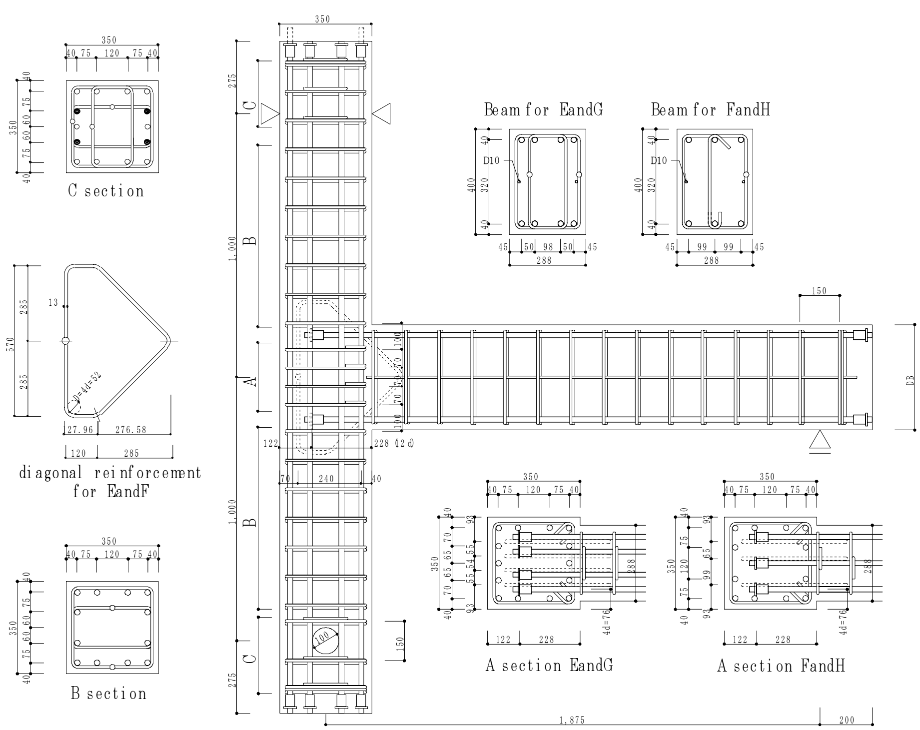


Figure 6. Bar arrangement for exterior joints

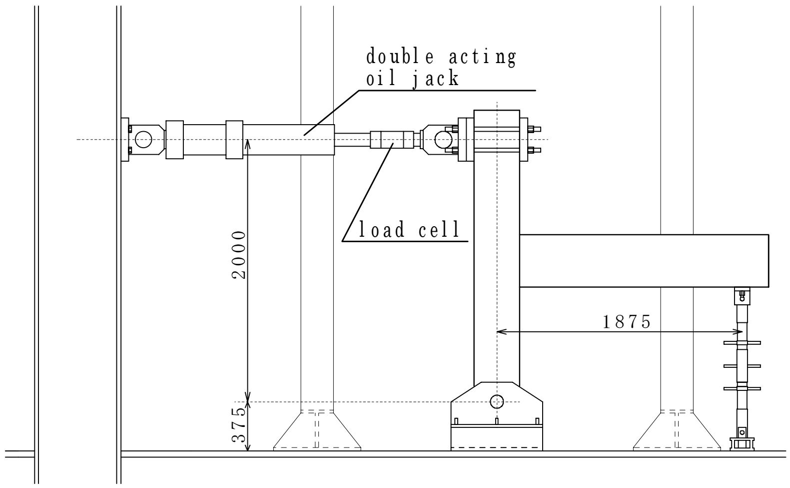


Figure 7. **Loading setup for exterior joints**

Loading setup for an exterior joint is shown in **Figure 7**. Positive loads were given in the direction of the oil jack pushing out and alternative reversal loads were controlled by relative story displacement angle (R) which increased step by step, 2 cycles loading for R=0.005 radian, 3 cycles for R=0.01 radian, 3cycles for R=0.02 radian, 2 cycles for R=0.04 radian and finally loading until R=0.08 or recognition of the reduction of load bearing capacities.

Cracks and concrete failure were observed as well as partial deformations outside of specimens and strain of reinforcements and slippage of the development of beam reinforcements were also measured.

Before the tests using material testing results the load bearing capacities of each specimen was calculated. The ultimate bending strength of beam was calculated to use common moment arm of 0.9 time of effective depth, shear strength of a joint was followed the equation from A.I.J. [4] and strength of development was based on the equation (1). Failure mechanisms were predicted to be beam yielding for G and H, shear failure of the joint for E and failure of development of beam reinforcements for F.

Test results and discussion

Relationship between load and relative story drift of each specimen is shown **Figure 8**. Specimen E had been designed to fail in shear of the joint, nevertheless it could not bear enough the calculated load of development failure. Specimen F had been designed to fail in development while the relationship led to show exceeding a little the load. Specimen G and H showed more ductile relationship than E and F because of beam yielding.

As for the condition of development of beam reinforcements into the exterior joint, following the current requirements of the mechanical anchor system, the beam developments should be put between a couple of hoops which have the reinforcement ratio of 0.4 %. The straight development length which was defined as distance from column face to inside of screw nut flange was shorter than the required length; nevertheless specimens were prevented from failure of development. Therefore it is possible for the exterior joint to be connected by the beam reinforced with reinforcements of high strength steel when follows the current requirements.

From test results, when shear transference to the joint was so great that the hoop might yield along the beam development and shear deformation of the joint increased too much, failure of development might

occur. In this case it would be recommended to reduce shear force transfer into the joint or to add hoops and tie reinforcements at the location of the beam developments.
It was confirmed that crush and coming out of joint concrete took place locally, when bearing failure was produced around the screw nut on the beam development nearest to concrete surface. At that time the maximum axial force of beam reinforcement was verified by the current experimental equation while the failure was accompanied with severe shear failure of concrete.
When the exterior joint failed exceedingly in shear, large slippage by pulling out might occur along the development without local crush and coming out of joint concrete. Therefore it is important to design an exterior joint not to fail in shear first and then to fail in bending secondly and never to fail in development.

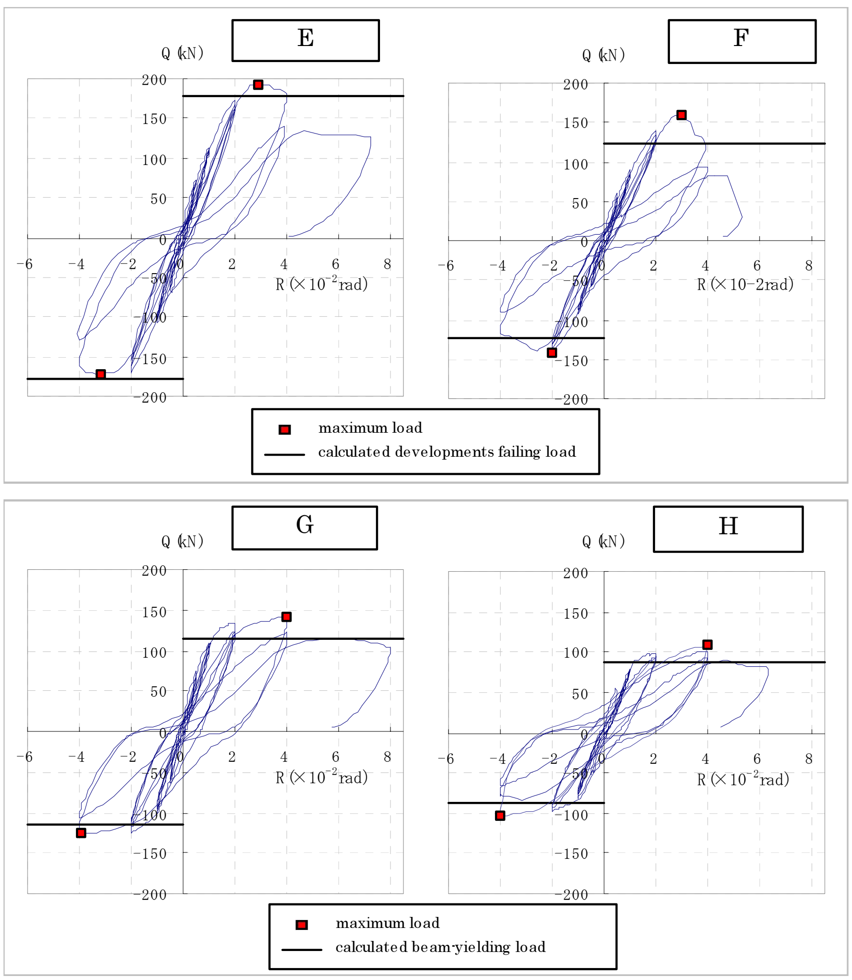


Figure 8. Relationship between Road and relative story drift

EFFECTIVE VERTICAL LATERAL REINFORCEMENTS IN SIZE REDUCED T-JOINTS

Object of tests

When a T-joint on the top of a building was designed to fail in bending of the column, following two strength factors was supposed to acquire the same ductility as the other partial frame.

1. Strength ratio of the joint should be enough, desirably more than 1.1 times of the joint shear strength predicted from the equation of A.I.J. [4].
2. Adding to horizontal lateral reinforcements of hoops and ties, inverse U and U-shaped vertical lateral reinforcements should be arranged surrounding whole joint concrete core. Consequently joint shear strength would be kept and development strength of beam reinforcement would be hereby raised up.

The ultimate relative story displacement angle of a partial frame is usually required by more than 1/50 in the common structural design guideline. The object of the tests was supposed to keep 0.8 times of load bearing capacities (hereinafter referred to as 0.8Qcmax) until the relative story displacement angle reached 1/25 (hereinafter referred to as the ultimate angle).

Test specimens and testing method

Four size reduced specimens were provided for the tests. Each specimen was a partial frame of a T-joint and designed to fail in bending of column. Test specimens had three experimental variables, that is, concrete strength and steel strength of beam/column reinforcements and amount of horizontal lateral reinforcements in the joint. Assortment of these variables is shown in **Table 4** and **Table 5**. From calculation using material testing results (see **Table 6** and **Table 7**) shear strength ratio of the joint was confirmed to be larger enough to avoid severe shear failure of the joint.

Dimensions and steel bar arrangements of four test specimens which were named as T345-30-4S, T345-30-3N, T490-45-4S and T490-45-3N respectively are shown in **Figure 10**.

Loading setup for a T-joint is shown in **Figure 9**. A T-joint was set in bottom up in the contrary direction to placing concrete direction. Alternative reversal loads were controlled by relative story displacement angle (R) which increased step by step, 2 cycles loading for R=0.005 radian, 2 cycles for R=0.01 radian, 2 cycles for R=0.02 radian, 2 cycles for R=0.03 radian, 2 cycles for R=0.04 radian, 2 cycles for R=0.06 radian and finally loading until recognition of the reduction of load bearing capacities.

Cracks and failure of concrete were observed as well as partial deformations outside of specimens, the strain of reinforcements and slippage of the development of column reinforcements were also measured.

Test results and discussion

Relationship between load and relative story drift of each specimen is shown **Figure 11**.

All specimens failed in bending of columns at the deformation amplitude R=0.01, and reduced stiffness by R=0.02 but the load was renewed as the maximum value until R=0.03 where the load reached bearing capacities.

The capacities continued to decrease while the deformation amplitude increased over R=0.04. The reduction was larger when horizontal lateral reinforcement ratio and shear strength ratio of the joint were smaller.

Before a specimen reached the capacities the development of column reinforcements did not fail. When the capacities reduced the development of column reinforcements failed while the development devices slipped largely from concrete. The device located at sectional corner of column slipped destructively in the both direction of pulling out and pushing out. Compared with this, the device located inside of column section failed only in the direction of pushing out. It would be suggested that both confinement of concrete volume and compressive stress block from beam bending moment influenced on the development behavior.

The largest strain of horizontal lateral reinforcements was measured at the measuring point where development devices were nearest and development failed after the specimen reached capacities.

The largest strain of vertical lateral reinforcements was about 2000 micro in the elastic range and the reinforcements confined increasing the crack width. But there was not measuring point of strain appearing sudden increase when the development failed.

The ultimate angle of 1/25 radian was produced at the first cycle of load reversals when shear strength ratio of the joint was more than 1.35 and horizontal lateral reinforcement ratio was more than 0.30 % and vertical lateral reinforcement ratio was 0.37 % . It could be suggested that T-joint was ductile enough to apply for the common design criteria of the ultimate angle of 1/50 when the above-mentioned condition was satisfied. Nevertheless the condition proposed here might give considerable difficulties to construction practices.

Table 4. Demensions and reinforcements of beam and column

specimen	conrete strength	span L(beam) × H (column)	column: column reinforcement [Pg]	column: column reinforcement standard	column: hoop [Pw]	column: hoop standard	column: cover mm	column: development length mm	beam: beam reinforcement [Pt]	beam: beam reinforcement standard	beam: stirrup [Pw]	beam: stirrup standard
T345-30-4S	30	3400mm × 1000mm	8-D19 [1.44%]	SD390	3-S6@60 [0.40%]	KSS785	57	342 (18db)	4-D19 [1.06%]	SD295A	3-S6@60 [0.40%]	KSS785
T345-30-3N												
T490-45-4S	45			SD490	3-S6@40 [0.60%]				4-D19 [1.06%]	SD390	3-S6@40 [0.60%]	
T490-45-3N												
2S-2	40	5100 × 1500	8-D29 [1.43%]	SD490	3-S10@90 [0.39%]	RB785	87	522 (18db)	4-D29 [1.17%]	SD490	3-S10@90 [0.39%]	KSS785
2S-0												
WN-ST												
SP-ST												

Pg = gross reinforcement ratio

Pt = tension reinforcement ratio

Pw = web reinforcements ratio

Table 5. Reinforcements and strength ratio of T-joint

specimen	joint: hoop [Pwjh]	joint: hoop standard	joint: vertical lateral reinforcements [Pwjv]	joint: vertical lateral reinforcements standard	joint strength ratio (Vpu/Vmu)
T345-30-4S	3-S6(5sets) [0.38%]	KSS785	2-D10(3sets) [0.37%]	SD295A	1.46
T345-30-3N	2-S6(6sets) [0.30%]				
T490-45-4S	3-S6(5sets) [0.38%]				1.35
T490-45-3N	2-S6(6sets) [0.30%]				
2S-2	3-S10(5sets) [0.37%]	RB785	2-S10(4sets) [0.22%]	SD295A	1.10
2S-0			–	–	
WN-ST	2-S10(5sets) [0.25%]	RB785	2-S10(6sets) [0.34%]	RB785	1.15
SP-ST					

Pwjh = horizontal lateral reinforcement ratio in the joint

Pwjv= vertical lateral reinforcement ratio in the joint

Table 6. Mechanical properties of steel of T-joint

specimen	diameter standard	yield strength (N/mm^2)	tensile strength (N/mm^2)	elongation (%)	young's modulus $\times 10^5$ (N/mm^2)	strain at yielding ($\times 10^{-6}$)
T345-30-4S T345-30-3N T490-45-4S T490-45-3N	S6 (KSS785)	1123	1196	10	1.97	5691
	S10 (KSS785)	1069	1163	9	2.14	4996
	D10 (SD295A)	396	531	29	1.93	2063
	D19 (SD295A)	336	507	27	1.94	1727
	D19 (SD345)	386	577	21	1.85	2060
	D19 (SD390)	427	640	22	1.9	2291
	D19 (SD490)	563	765	17	2.06	2866
2S-2 2S-0	D10 (SD290A)	415	594	20	1.81	4635
	S10 (KB785)	961	1146	10	1.95	7340
	D29 (SD490)	532	731	22	1.97	2930
WN-ST SP-ST	S10 (KB785)	944	1079	10	2.01	6874
	D29 (SD490)	536	740	24	1.96	3117

Table 7. Mechanical properties of concrete of T-joint

specimen	specified design strength	age (day)	compressive strength (N/mm^2)	young's modulus $\times 10^4$ (N/mm^2)	splitting strength (N/mm^2)
T345-30-4S T345-30-3N	30	24 ,30	33.3	2.54	2.9
T490-45-4S T490-45-3N	45	33 ,35	49.7	2.91	3.3
2S-2 2S-0	40	33 ~ 56	35.7	2.86	4.23
WN-ST SP-ST	40	35 ~ 39	38.5	3.38	4.58

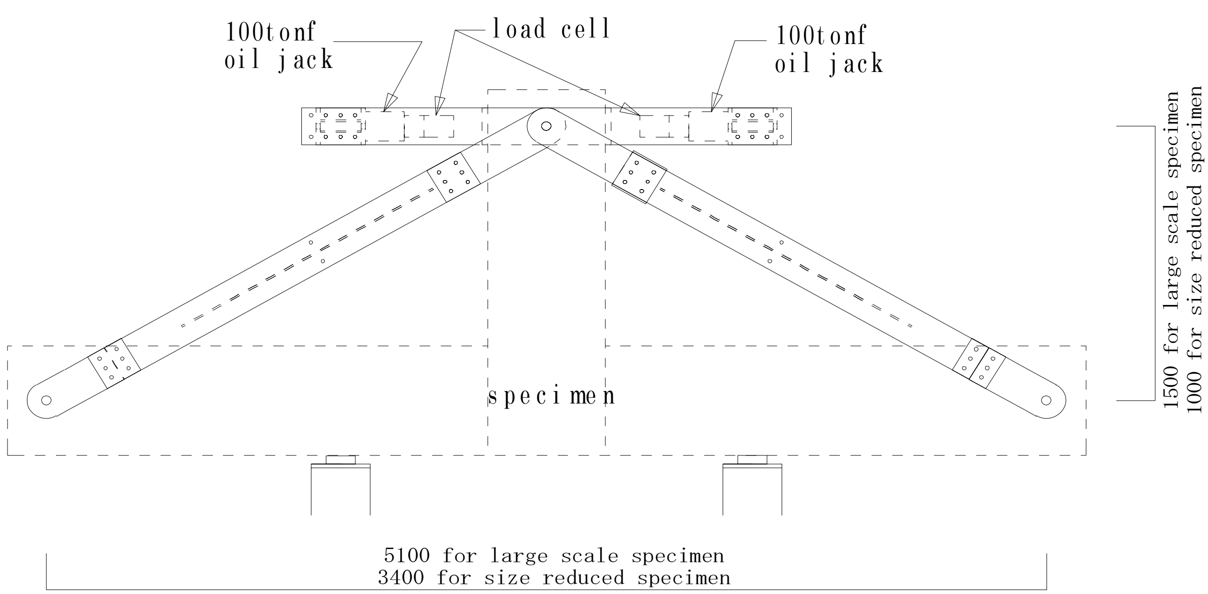


Figure 9. Loading setup for T-joint specimen

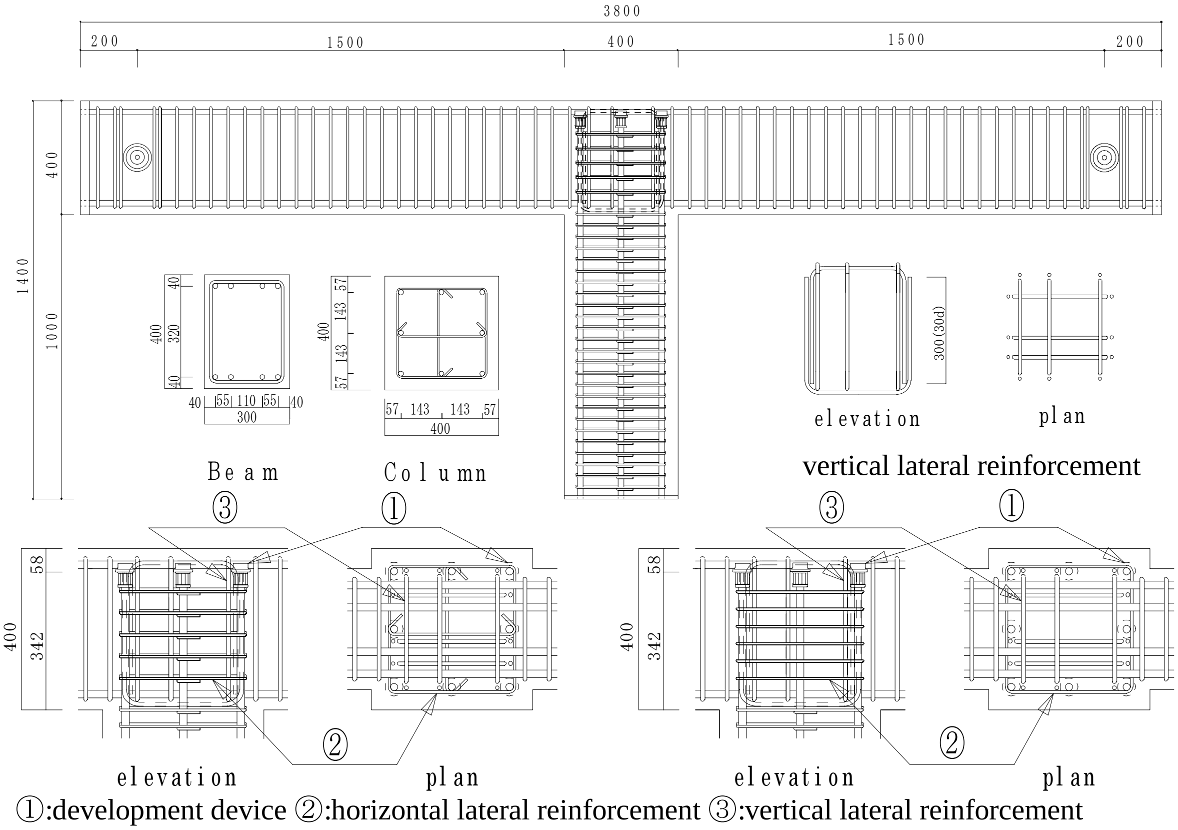


Figure 10. Bar arrangement for size reduced T-joint

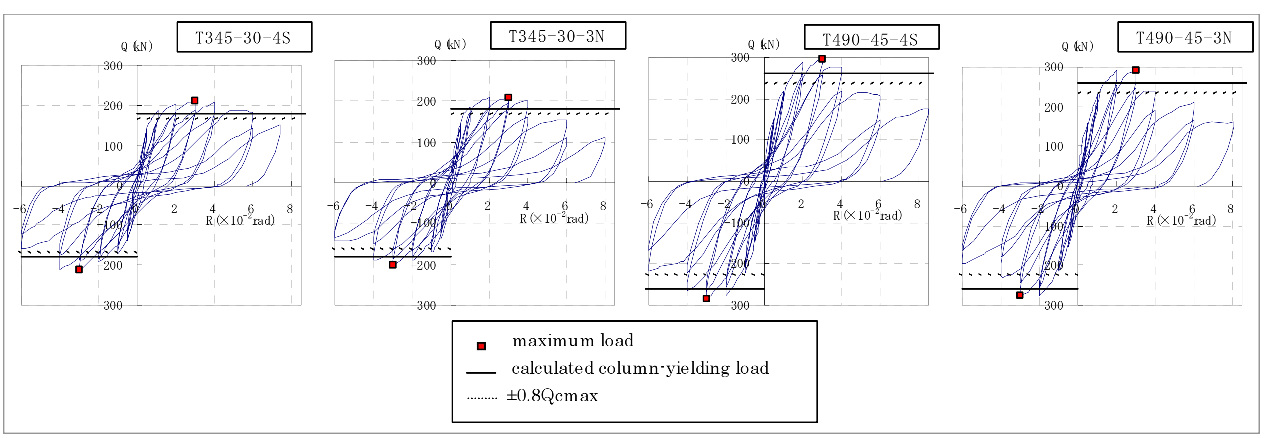


Figure 11. Relationship between load and relative story drift (for size reduced specimen)

PROTOTYPE OF PRACTICAL BAR ARRANGEMENT IN LARGE SCALE T-JOINTS

Object of tests

Compared with tests of size reduced T-joints, column section was enlarged to be 60 cm square and ratio of beam width to column width reduced to make the condition on development of the column reinforcement located at column sectional corner strict. This change could provide more realistic situation for the T-joint and simultaneously prove safety of more practical and simpler bar arrangement.

Large scale T-joint specimens were made from fewer amounts of reinforcements than previous tests. In this case, object of the tests was also supposed to keep 0.8Qcmax until the ultimate angle.

Test specimens and testing method

Four large scale specimens were provided for the tests. Each specimen was a partial frame of a T-joint and designed to fail in bending of column. Test specimens had three experimental variables, that is, method of bar arrangement and amount of horizontal or vertical lateral reinforcements in the joint. Assortment of these variables is shown in **Table 4** and **Table 5**. From calculation using material testing results (see **Table 6** and **Table 7**) shear strength ratio of the joint was confirmed to be larger to avoid severe shear failure of the joint.

Dimensions and steel bar arrangements of four test specimens which were named as 2S-2, 2S-0, WN-ST and SP-ST respectively are shown in **Figure 12**.

Concrete strength was 40 N/mm^2 and standard yield strength of steel was 490 N/mm^2.

Specimen 2S-2 had hoops and ties of horizontal lateral reinforcements which was fewer than size reduced specimens because of mitigation of hardness of construction practice. Specimen 2S-0 omitted vertical lateral reinforcements while adding development devices at the location of bottom beam reinforcements. Specimen WN-ST had double arrangement of development devices and stopped using the inverse U and U-shaped vertical lateral reinforcements and arranged stirrups of beam into the joint. Specimen SP-ST adopted stirrups and stopped using double devices, and spiral steel coils were put on along the development length.

Loading and measurement and observation were the same way as size reduced specimens.

Test results and discussion

Relationship between load and relative story drift of each specimen is shown **Figure 13**.

All specimens failed in bending of columns at the deformation amplitude R=0.01, and reduced stiffness by R=0.02.

Specimens 2S-2, 2S-0 and WN-ST lost the load bearing capacities during load reversals of R=0.02 and R=0.03. Specimen SP-ST gradually reduced the capacities during increase of deformation amplitude up to R=0.06 and the ultimate angle was 1/25.

The double devices put on specimen 2S-0 and WN-ST did not seem effective to protect pushing out of the development.

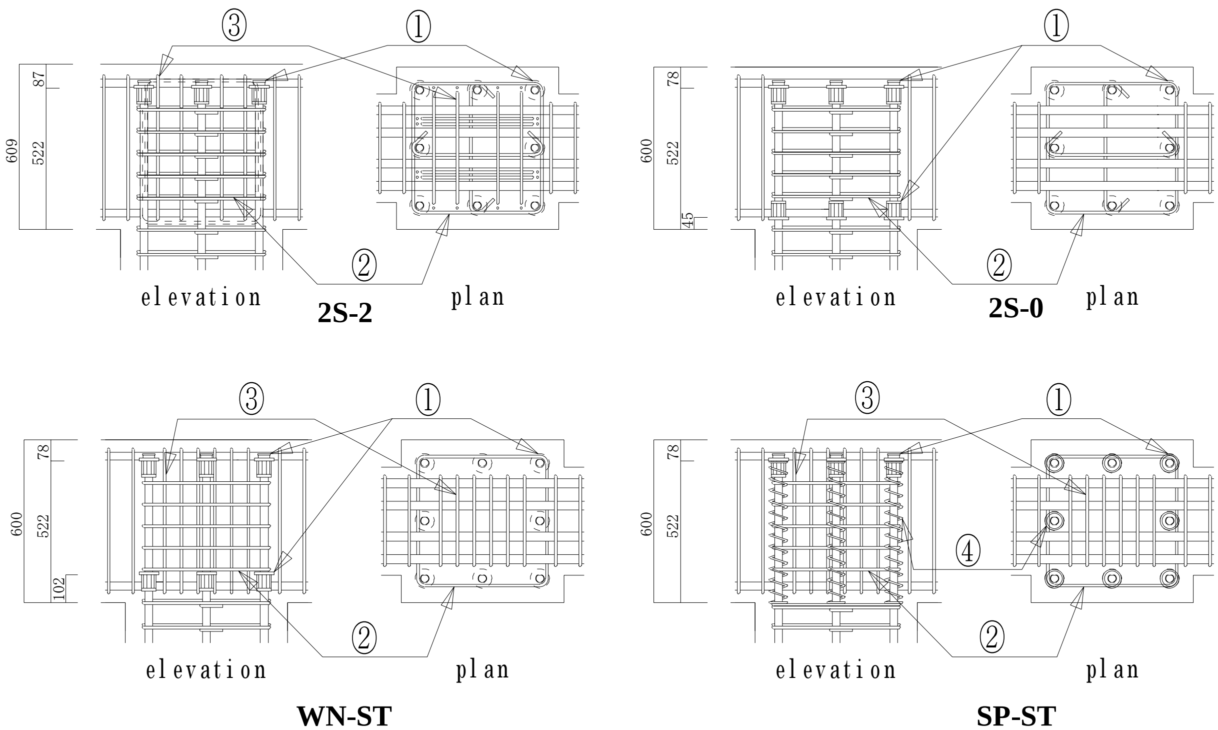
Vertical lateral reinforcements arranged to the specimens except 2S-0 prevented from inflation of beam reinforcements.

Specimens of 2S-2, 2S-0 and WN-ST got bond failure along the development length, but specimen SP-ST did not produce that failure because of reinforcing effect of spiral steel coils.

Before specimens did not reach the capacities bond strength of development reinforcement was large enough.

After bearing the maximum load the slippage of developments took place larger and led to failure in specimens of 2S-2, 2S-0 and WN-ST.

The ultimate angle of 1/50 radian was produced when shear strength ratio of the joint was more than 1.10 and horizontal lateral reinforcement ratio was more than 0.25 % and vertical lateral reinforcement ratio was 0.22 % . Under these conditions the ultimate angle of 1/25 radian could not be produced except specimen SP-ST.



①:development device ②:horizontal lateral reinforcement ③:vertical lateral reinforcement
④:spiral reinforcement

Figure 12. Detail of bar arrangement of large scale T-joint

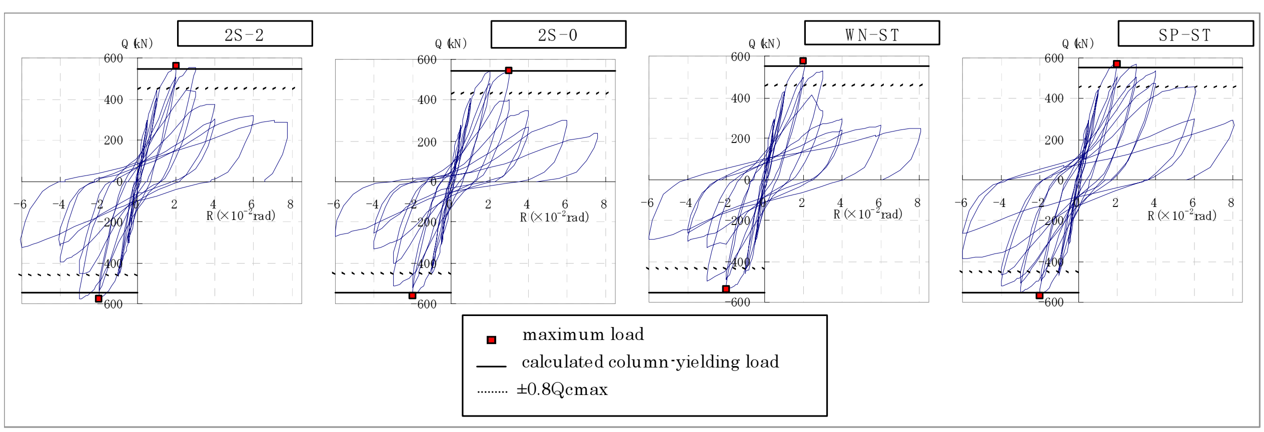


Figure 13. Relationship between load and relative story drift (for large scale specimen)

CONCLUSIONS

Three series of dynamic experiments have been conducted to investigate mechanical anchor performance of longitudinal bars embedded within the beam-column joints which are located at the side or on the top of a building. The test results and its discussion presented herein have led to the following conclusions:

1. Anchorage of beam reinforcements in the exterior beam-column joint
Anchorage of beam reinforcements which are terminated in the joints with screw nuts is to be designed based on the current equation and the recommendation of bar arrangement. In this case both confinements of concrete and horizontal lateral reinforcements are satisfactorily effective on the development of the beam reinforcements.

2. Confinement around the bar located at column section corner
In the T-shaped beam-column joint on the top of a building, anchorage performance of the longitudinal column reinforcement located at the column sectional corner would be controlled by confinement of concrete and lateral reinforcements. The test showed us severe anchorage failure under lower anchorage strength than the value calculated by the preceding equation, when the partial frame produced large plastic deformation after column yielding and the joint failed in shear.

3. Vertical lateral reinforcements in the T-shaped beam-column joint on the top of a building
Vertical lateral reinforcements in the T-shaped beam-column joint on the top of a building should be arranged surrounding whole joint concrete like a cage and have more than 0.4 % of reinforcement ratio while the joint is having more than 0.3 % of horizontal lateral reinforcements too. When the vertical lateral reinforcements are reduced to half amount or extension of beam stirrups to the joint, we cannot get enough ductility.

4. Shear strength ratio of the joint
When the condition above-mentioned in **3.** is satisfied and shear strength of the joint is more than 1.4 times of the working force at column yielding, the partial frame of a T-shaped beam-column joint can acquire such ductility as deforms over yielding by 0.04 radian of relative story drift while maintaining 80 % of load bearing capacities.

5. Setting of spiral reinforcements around the developments
When spiral reinforcements are set around the straight developments of column longitudinal bars in the joint the partial frame of T-shaped beam-column joint deforms enough beyond column yielding without failure along the development in spite of severe shear cracks in the joint panel. This improvement was also effective under the condition of extension of beam stirrups in the joint instead of vertical lateral reinforcements surrounding whole joint concrete like a cage.

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