FURTHER STUDIES ON THE BEHAVIOR OF REINFORCED CONCRETE BEAM-COLUMN JOINT UNDER REVERSED CYCLIC LOADING

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SUMMARY

The cyclic loading tests of 36 beam-column joints of reinforced concrete frames were carried out to study the behavior of the joint core and the main factors influencing the shear strength of the joint core, those include the confinement effect of intersecting beams, the shear and compression stress ratios, the hoop content, the anchorage slip of longitudinal beam bars in the core, and the relocation of plastic hinges on the beams.

Based on the experimental results, design suggestions are presented.

INTRODUCTION

In recent years, quite a lot of joint damage of reinforced concrete frames occured during severe earthquakes in the world. The damage of reinforced concrete frame buildings in Tangshan city and colliery compound showed typical examples during Tangshan earthquake 1976 (Photo 1,2,). These earthquake lessons call attention to the proper design of beam-column joint in multistory frame structures.

Recent investigations about the aseismic behavior of beam-column joint give quite different design suggestions (Ref.2,3), such as the shear strength contributed by the concrete portion of the joint core and the effect of orthogonally intersecting beams. In order to get further understanding of joint core behavior and establish the seismic design approach of the joint, a series of beam-column joint tests under reversed cyclic loading were carried out.

OUTLINE OF THE TESTS

Thirty six joint specimens were divided into four groups. The first group (11 specimens) was prepared to study the confinement effect on the

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joint cores by intersecting beams; the second group (13 specimens) and the third group (6 specimens), to study the effects of compression and shear stress ratios, hoop content and the slip of longitudinal beam bars on the concrete shear strength at the joint core; the fourth group (6 specimens), for studying the effect of relocating beam plastic hinges on the behavior of joint core. In addition, photoelastic experimental study was also carried out to study the stress distribution in joint core at elastic stage.

The tests were conducted on a static testing platform (Fig.1). A constant axial loading was applied on the top of the column according to the assigned compression stress ratio and the reversed cyclic static loadings were applied on both ends of the beams. The loading sequence is shown in Fig.2.

THE BEHAVIOR AND FAILURE PROCESS OF THE JOINT CORES

Under the cyclic loading, the remarkable failure mode of the joint core is shear-compression failure. In the elastic stage, the shear force in the core is mainly carried by the concrete portion acting as diagonal concrete strut. The photoelastic experiment verifies this mechanism, there exists diagonally parallel equivalent lines of principal compressive stress, the constitution of diagonal compression strut is clearly shown in Fig.3, as axial compression force increases, the range of equivalent stress lines or the width of the diagonal struts increases accordingly. When the principal tensile stress reaches the ultimate tensile stress of the concrete, through cracks occur along both diagonals, this is the stage of diagonal crack, with moderate hoop content the shear force acting at the core approximates to 60–70% of the ultimate shear capacity of the core. The diagonal cracks lead to obvious degradation of the joint stiffness.

Under subsequent cyclic loading, the longitudinal bars in the beam yield, multiple parallel diagonal cracks occur at the core, and the hoops yield progressively, the joint core concrete is sliced by the parallel cracks of both direction into rhombic blocks, the fracture stage is reached. The shear sliding between the blocks is resisted by the interlock of aggregates and dowel action of hoops and column bars. As shown by tests, with rational design of hoops, the maximum load-bearing capacity of the joint core can still be maintained, but the rigidity of the joint degenerates successively.

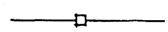
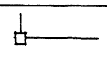
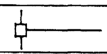
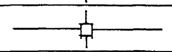
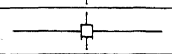
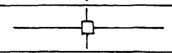
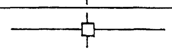
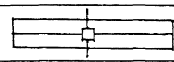
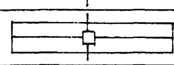
With subsequent loading, the interlock and dowel actions are deteriorated by the progressive fracture and crushing of the blocks, the load-bearing capacity of the joint core decreases and final failure is reached. Fig.4 shows the P(load) $-\gamma$ (shear angle) envelope curve of the joint core, it explains from the begining of fracture stage to failure, the joint core can stand a number of loading cyclic without degradation of load bearing capacity, the deformation is 3—4 times enlarged. Based on the philosophy of "fracture without collapse", the process of failure provides the possibility of taking the fracture stage as ultimate state in designing beam-column joint of ordinary frame structures. For special structures, while cracking at the joint core is not allowed, the diagonal crack stage will be taken as ultimate state in design.

THE CONFINEMENT EFFECT OF INTERSECTING BEAMS ON THE JOINT CORE

The coefficient ψ is specified in the formulation for calculating the concrete shear strength of the joint core to express the confinement effect of the orthogonally intersecting beams. 11 specimens with 5 different joint types were tested to measure the values of ψ. The width of the intersecting beam section is the same as the frame beam, and its depth is 4/5 of the frame beam.

Values of ψ measured at different stages are shown in Table 1 .

Table.1

Specimen no.	Type of specimen	Coefficient of confining effect, ψ		
		Diagonal cracking stage	Fracture stage	Adopted value
I, I−1, I−2		1	1	1
II *		1.067	1.078	1
II − 1		1.026	1.078	1
II − 2		1.189	1.126	1
III *		1.177	1.439	1
III − 1		1.502	1.666	1.5
III − 2✧		1.989	2.435	2
IV − 1		1.476	2.203	1.5
IV − 2		1.570	2.295	1.5

* Constant loads applied at the ends of orthogonally intersecting beams.
✧ Relocation of beam plastic hinge by cutting off extra bars

In the case of applying a constant load at the ends of the intersecting beams, vertical cracks occur at the column surfaces in contact with the intersecting beams, the confinement effect of the intersecting beams on the joint core is thus reduced. (specimen III). Test results of specimens IV-1 and VI-2 indicate that the ψ values of the interior column joints with floor slab are greater than those without slabs, this might compensate the above mentioned degenerating effects of the intersecting beams. Considering the above facts, following suggestion has been made: in case of two way beams, the width of the intersecting beams not less than half width of the column, and its depth not less than 3/4 of the frame beam, the value of ψ equals 1.5. As indicated by the results of apecimens II, II-1, for exterior joint with beams in three directions, there is no evident confinement effect, the value of ψ is suggested to be 1.0. As to specimen III-2, because the plastic hinges are shifted away from the joint, the confinement effect on the joint core increases, the ψ value of 2.0 is suggested for this case.

THE EFFECT OF AXIAL COMPRESSION ON CONCRETE SHEAR STRENGTH IN THE JOINT CORE

The test results indicate that within certain range of the compression stress ratio (σ_0/R_a), the increase of the axial compressive force will not only increase the cracking strength of the concrete in the core, but also its ultimate shear strength. σ_0 is the compressive stress at the joint and R_a is the axial comoressive strength of concrete.

The values of σ_0/R_a of SJ-1-2 and SJ-1-4 are 0.25 and 0.61 respectively. The Q_h - Δ curves (Fig.5, Q_h is the shear force carried by the concrete, Δ is the deflection at the beam end) of those two specimens show that the cracking strength of specimen SJ-1-4 is 80% higher than that of SJ-1-2, and the ultimate strength is 46.6% higher than SJ-1-2, but the deformation capacity of specimen SJ-1-4 is smaller than that of SJ-1-2, and the decending slope of the curve over the peak Q_h value is greater than that of SJ-1-2. So the increase of the axial force can increase the concrete shear strength in the core, but reduce the ductility.

According to the testing results, expression $\sqrt{1+6\frac{\sigma_0}{R_a}}$ is suggested to respresent the axial force effect on the concrete shear strength of core. The test result of specimen SJ-2 indicates, when σ_0/R_a =0.702 the shear strength of the core concrete reduces obviously. Therefore, the upper limit value of σ_0/R_a has to be limited not greater than 0.6 in the design.

THE EFFECT OF SHEAR STRESS RATIO AND HOOP CONTENT ON THE JOINT CORE CONCRETE SHEAR STRENGTH

The shear stress at the core (expressed by shear stress ratio τ/R_a) and the hoop concrete $\mu_t R_g/R_a$ are important factors affecting the shear strength of the concrete in the joint core. μ_t is the volumetric hoop ratio in the joint core, R_g is the tensile strength of the hoop. The effect can be represented by factor m in the calculation of core concrete shear strength Q_h, and the formulation of Q_h can be expressed as: $Q_h = m\psi b_j h_j R_a\sqrt{1+6\frac{\sigma_0}{R_a}}$

b_j h_j : the effective width and depth of joint core section.

In general, the shear stress ratio will increase while hoop ratio increases. The testing results indicate that, when $\mu R_g/R_a$ is near 0.1, m gets higher value. While $\mu R_g/R_a \leqslant 0.3$, m value reduces, correspondingly, while $\tau/R_a \leqslant 0.25$, m gets higher value, while $\tau/R_a > 0.25$, m also reduces (Fig.6). The reasons of the reduction of m value can be known from Fig.7, adquate detailing of the hoop and reasonable control of shear stress ratio will make the concrete and the hoop reach their ultimate strength simultaneously (fracture stage). In other words, when concrete reaches its ultimate shear strength, the hoops yield(Fig 7a, SJ-1-2), at this time,

$$m = \frac{Q_h}{\psi b_j h_j R_a\sqrt{1+6\frac{\sigma_0}{R_a}}} \qquad \mu_t R_g/R_a = 0.074. \qquad \tau/R_a = 0.19$$

If hoop ratio (or shear stress ratio) is very high, the concrete will fail before the yielding of hoops (Fig 7b, SJ-2-2), in this case, when hoops yield, the shear force of the concrete has already reduced from the maximum value Q_h to $0.6Q_h$, then,

$$m = \frac{0.6Q_h}{\psi b_j h_j R_a\sqrt{1+6\frac{\sigma_0}{R_a}}} \qquad \mu_t R_g/R_a = 0.28, \ \tau/R_a = 0.25$$

In order to develop the full strength of the concrete and hoops, the hoop ratio has to be limited, this can be performed by limiting the shear stress ratio.

According to Fig 6, the shear stress ratio and m value are suggested as follows: $\tau/R_a \leqslant 0.25$, $m=0.09$.

$0.25<\tau/R_a \leqslant 0.35$, $m=0.07$.

when $\tau/R_a>0.35$, the cross section of the column has to be enlarged.

THE EFFECT OF THE SLIP OF BEAM BARS AND THE RELOCATION OF THE BEAM PLASTIC HINGES ON THE BEHAVIOR OF BEAM-COLUMN JOINT

In interior beam-column joints, following the yielding of beam bars, slip will occur at the joint core, the shear strength of joint core, the moment resistant capacity and ductility of the beams are greatly reduced.

It was known that the effect of bar slipping can be mitigated by shifting the beam plastic hinge a distance away (not less than the beam depth) from the column face (Ref.4). In order to get further understanding, six specimens of interior beam-column joints were tested.

Two measures were taken for shifting the beam plastic hinge by cutting off or cross bending of extra bars (Fig.8(a) specimen SJ-4-B, and Fig.8(b) specimen SJ-4-A). The relocation of the plastic hinge can be realized, when the moment resisting capacity of the beam at column face is at least 1.25 times that at the plastic hinge. In calculating the moment resisting value at the plastic hinge, the effect of the cross bending bars must be considered, test shows the maximum stress of the bending bar at the cross point is about 80% of its yielding strength. In order to warrant adequate ductility of the relocated plastic hinge, the shear stress at the hinge should be limited to 0.1 Ra. All flexural bars at the hinge should be restrained by hoops, which are designed to take the total shear force, and its spacing should not be more than 10 cm or 4d (d is the least diameter of flexural bars).

The method of cross bending extra bars gives higher moment and shear resisting capacity at the plastic hinge and less defect of shear sliding in vertical direction. (photo 3,4) Beam bar slipping may be prevented by shifting beam plastic hinges, the shear resisting strength of the joint core concrete is 30% increased, and its shear deformation is about 65% reduced. The joint rigidity and the behavior of energy dissipation are much improved (Fig.9(a),(b)).

Based on the test results, in designing the joint core the coefficient of confinement effect can be taken as 2, and the factor m of concrete shear strength may be taken as 0.1.

CONCLUSION

In the seismic design of beam-column joints, the shear force taken by the joint core concrete can be evaluated as follows:

1. For ordinary frame structures, the shear force taken by joint core concrete can be calculated by the following formula:

$$Q_h = m\psi b_j h_j R_a \sqrt{1+6\sigma_0/R_a}$$

m: shear strength factor of joint core concrete;

For $\tau/R_a \leqslant 0.25$, m=0.09

For $0.25<\tau/R_a \leqslant 0.35$, m=0.07

the average shear stress in the joint core should be limited to 0.35Ra

ψ: coefficient of confinement of intersecting beams.

a. for interior joint with 2-way beams $\psi=1.5$

b. same as above except beam plastic hinges are relocated $\psi=2$

c. all other conditions $\psi=1$

Width of beams not less than half width of column, depth of intersecting beam not less than 3/4 depth of the main beam.

The yielding stress of hoop steel may be used to calculate the shear force taken by hoops.

2. For frame structure while cracks at joint core are not allowed, Q_h value may be calculated as follows: $Q_h=0.063\psi b_j h_j R_a\sqrt{1+6\frac{\sigma_0}{R_a}}$

Not more than 70% of the hoop yielding stress should be used to calculate the shear force taken by the hoops, in addition the average shear stress in the joint core should be limited to 0.2Ra.

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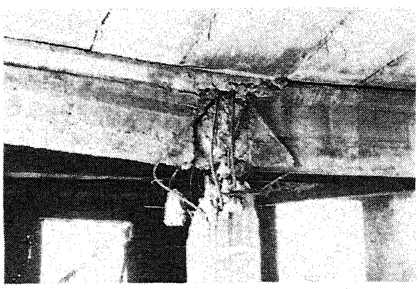
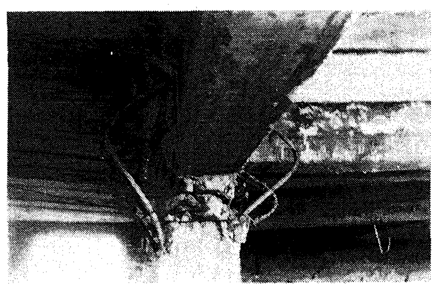


Photo(1) Earthquake damage of beam-column joint The New Hua Hsin Hotel, Tangshan.



Photo(2) Earthquake damage of beam-column joint Washing plant, Tangshan Colliery.

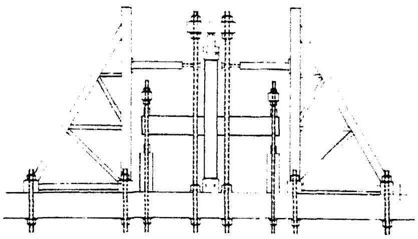


Fig.1 Test equipment.

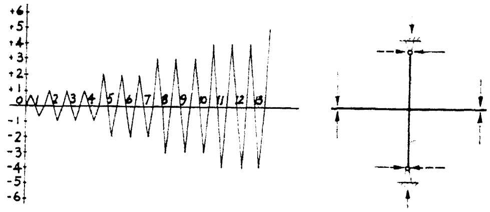


Fig.2 Loading sequence.

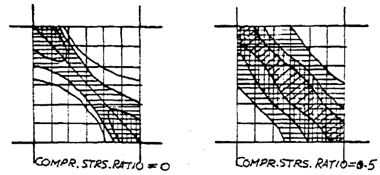


Fig.3 Variation of equivalent lines of principal compressive stress with the compressive stress ratio at joint core.

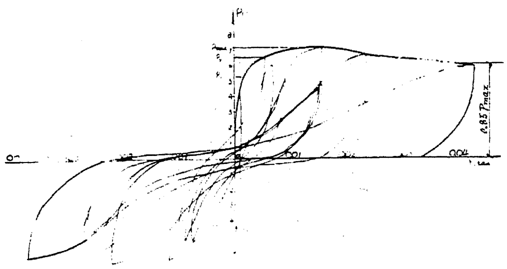


Fig.4 P-γ skeleton curve, specimen Ⅲ-1.

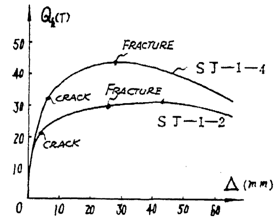


Fig.5 Q_h — Δ relationship.

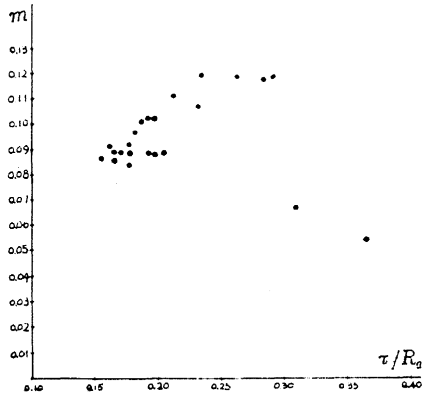


Fig.6 m — τ/R_a relationship.

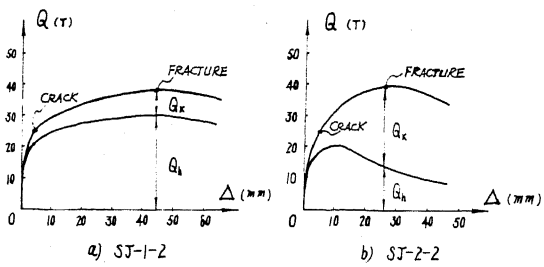
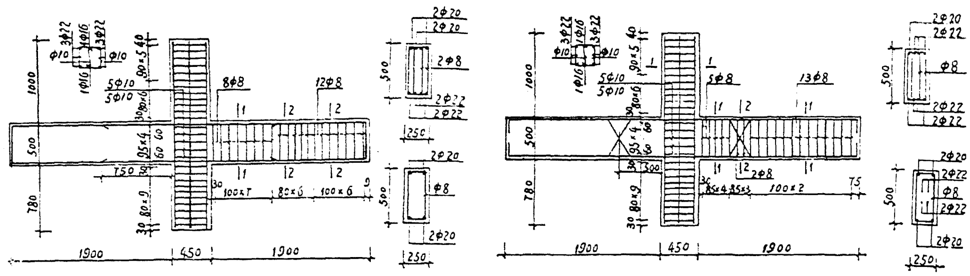


Fig.7 Q_h, Q_k — Δ Curves.



(a) (b)

Fig.8 (a) Reinforcement details, specimen SJ-4-B
(b) Reinforcement details, specimen SJ-4-A

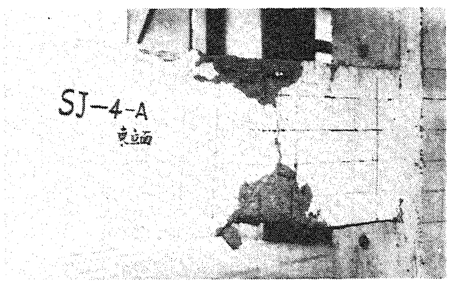


Photo.3.

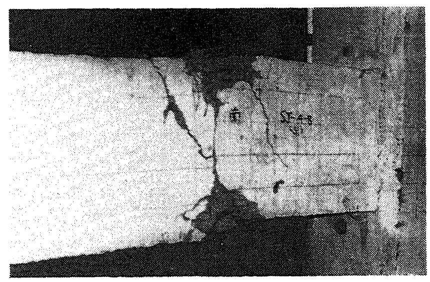
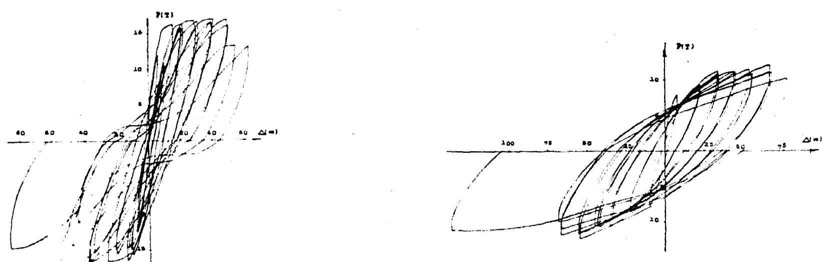


Photo.4.



(b)

(a)

Fig.9 Hysteresis loops.
(a) SJ-2-A Plastic hinge at beam end.
(b) SJ-4-A Plastic hinge relocated.