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Autor: Yousef, Ahmed M. / Yakimov, Ivan

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High-Strength Concrete Beam-Column Joints under Seismic Loading

Noeuds de cadres en béton à haute résistance sous charges sismiques

Rahmenknoten aus hochfestem Stahlbeton unter Erdbebenbelastung

Ahmed M. YOUSEF
Dr. Eng.
Univ. for Arch. and Civil Eng.
Sofia, Bulgaria

Ahmed M. Yousef, born 1961, received his B.Sc. and M.Sc. in Structural Engineering at the Faculty of Engineering, El-Mansoura University, Egypt, where he is working, as assistant lecturer. Since June 1992, he is pursuing, his study towards the Ph.D. degree at Sofia University.

Ivan YAKIMOV
Associate Professor
Univ. for Arch. and Civil Eng.
Sofia, Bulgaria

Ivan Yakimov, born 1935, is Associate Professor at the Department of Concrete Structures, Univ. for Architecture and Civil Engineering, Sofia, Bulgaria. His research activities are in the area of reinforced and prestressed concrete struc-tures, as well as composite construction design.

SUMMARY

The results from reversed cyclic loading tests on six exterior beam-column joints constructed with high strength concrete are presented. Three of the tested specimens were reinforced with crossed inclined bars within the joint core instead of the intermediate vertical column bars. The primary variables were the percentage of the transverse reinforcement in the joint, the amount of crossed reinforcement bars and the ratio of the column-to-beam flexural capacity. The results showed an excellent joint behaviour.

RÉSUMÉ

L'article présente les résultats de recherches expérimentales sur six éprouvettes de noeuds de cadres réalises en béton à haute résistance, soumis a un chargement cyclique. Trois éprouvettes out été arme avec des barres inclinées et croisées au noyau de noeud, au lieu de barres centrales longitudinales de la colonne. Les premières quantités variables étaient la section transversale des barres inclinées et le pourcentage de l'armature transversale au noeud. Le résultats ont montré un comportement excellent de toutes éprouvettes.

ZUSAMMENFASSUNG

Die Forschungsergebnisse für 6 Probekörper von externen Rahmenknoten unter zyklischer Belastung, die aus hochfestem Beton hergestellt worden waren, werden vorgestellt. Drei von diesen Probekörpern waren statt gerader Längsstäbe in der Säule mit geneigten kreuzenden Längsstäbe im Rahmenknotenkörbe bewährt. Die Querschnittfläche der kreuzenden Längsstäbe, das Prozent der Querbewehrung im Rahmenknotenkörben und das Verhältnis der Säule- und Balkentragfähigkeiten waren die ersten Variablen. Die Ergebnisse haben ein vorzügliches Verhalten des Rahmenknotens gezeichnet.

1. INTRODUCTION

Guidelines for designing beam-column joints of reinforced concrete ductile moment resisting frames were recommended by ACI-ASCE Committee 352 [1]. The design provisions stipulated in chapter 21 of ACI 318-89 (revised 1992) building code [2] were largely based on the work of the committee 352. Tests providing the basic data for these provisions were conducted on connections with concrete compressive strength (f_c) mostly less than 40 MPa. With the commercial availability of high strength concrete (HSC) with compressive strengths approaching 120 MPa [3], many questions have been raised regarding the applicability of these design provisions for HSC.

The 352 recommendations require a minimum flexural strength ratio at the joint (M_R) of 1.4 and a maximum allowable joint shear stress in the form of $\gamma\sqrt{f_c}$, where joint shear stress factor γ is a function of the joint type and the loading conditions. The required joint transverse reinforcement, which is calculated independently of these two variables, can cause congestion of the joint especially when using HSC. The reported studies on the seismic behaviour of HSC beam-column joints showed that these joints have sufficient energy dissipating capacity and load carrying capacity [4],[5]. On another hand, Tsonos, Tegos, and Penelis [6] suggested the use of crossed inclined bars within the joint core in order to improve the performance of Type 2 exterior beam-column joints.

The main objectives of this experimental study were to investigate the effect of joint trnsverse reinforcement on the seismic behaviour of exterior beam-column joints constructed with HSC (f_c varied between 69 and 77 MPa) and to examine the efficiency of using inclined reinforcing bars within the joint core as a method of reducing the quantity of joint transverse reinforcement required by the ACI-ASCE Committee 352.

2. TEST PROGRAM

2.1 Test Specimens

Six reinforced concrete beam-column subassembles were constructed with HSC. According to the classification of the 352 recommendations, the configuration of the specimens qualifies them as corner joint. all specimens were cast flat. After the reinforcing cage was placed into the form, concrete was placed and internally vibrated. The specimens were removed from the form after seven days and were moist-cured for 28 days. The minimum age of the specimens at the time of testing was four months.

All specimens have the same dimensions in three series. Each of them consists of two specimens, one conventionally reinforced in the joint region (C-Specimens) and one specimen reinforced with inclined crossed bars (X-Specimens) as shown in Fig. 1. The specimens of each series differed only in reinforcement details at the joint region as given in Table 1. The X-Specimens were reinforced with four crossed inclined bars bent diagonally across the joint core as shown in Fig.1, instead of the four intermediate longitudinal bars in the columns of the X-Specimens. In addition, the joint transverse reinforcement of the second specimen of each series was always less than that of the first one. For all specimens, the transverse reinforcement in the joint was extended to the column at 50 mm spacing. The beam shear reinforcement included R8 rectangular hoops with a 135-deg standard hook. The maximum allowable spacing being one-fourth of the beam effective depth according to the seismic provisions of ACI Building Code [2].

According to the 352 recommendations, the maximum joint shear stress for a corner joint constructed with ordinary strength concrete is ($0.083\,\gamma\sqrt{f_c}$ MPa) where joint shear stress factor γ is equal to 12, and the concrete compressive strength f_c was limited to 41 MPa. The design shear stress factor of the specimens was kept below 12 as given in Table 1. The joint shear stresses were calculated when the specimens were designed assuming that strain hardening will increase the tensile strength in the beam longitudinal reinforcement by 15% over the measured tensile yield strengths. The recommendations set a lower limit of 1.4 for the flexural strength ratio (M_R), defined as the

Table 1 Summary of test program

Specimes	f_c (MPa)	A_{sb}	A_{vc}	A_{ic}	A_{xc}	Joint hoops	ρ_{sh}%	ρ_{ACI}% Eq. (1)	ρ_{ACI}% Eq. (2)	M_R	γ	$N/f_c A_g$
JC1	73.3	2D14	4D12	4D10	-	3R6/3	0.71	4.8	2.5	1.4	6.2	0.15
JX1	77.1	2D14	4D12	-	4D10	3R6	0.47	5	2.7	1.4	6.2	0.15
JC2	71.1	2D14 +1D10	4D14	4D12	-	2R8/3	0.94	3.8	2	1.4	8.1	0.06
JX2	69.2	2D14 +1D10	4D14	-	4D12	2R8	0.63	3.7	2	1.4	8.1	0.06
JC3	69	2D14	4D16	4D14	-	3R6	0.47	4.5	2.4	2.4	6.2	0.15
JX3	69.7	2D14	4D16	-	4D14	2R6	0.35	4.5	2.4	2.4	6.2	0.15

Note: © D10, D12, D14, D16=Deformed bars with yield strength f_y = 532.9, 397.6, 446.1, and 392.1 MPa respectively.
© R6, R8=Plain bars with diameters 6, 8 mm, f_y=260.3, 315.6 MPa respectively. © 3R6/3=Three hoops with crossties.
© ρ_{sh} % = Actual joint transverse reinforcement ratio= (A_{sh} / sh'') © $N/f_c A_g$= Axial load intensity Index.

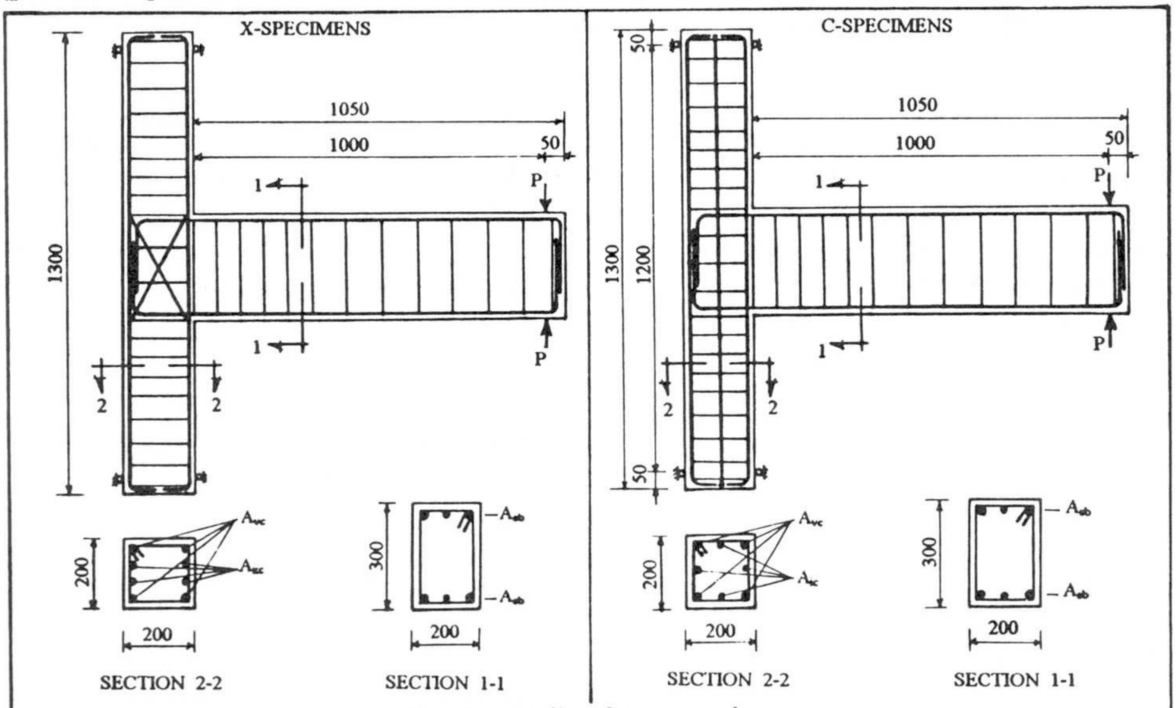

Fig 1 Details of test specimens

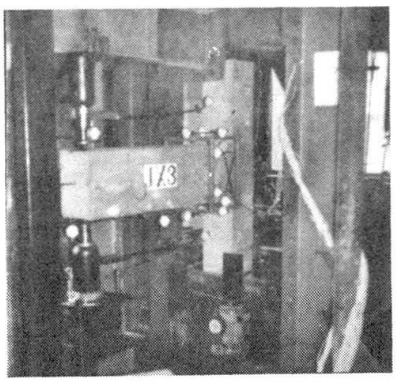

Fig 2 General view of loading setup

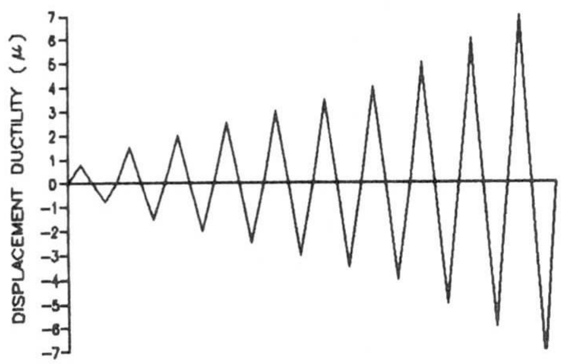

Fig 3 Cyclic load sequence used in the tests

sum of the flexural capacities of the columns to that of the beam at the connection region. For the tested specimens, M_R was equal to 1.4 or 2.4.

In order to insure adequate confinement of the joint core, the recommendations require that the total cross-sectional area of hoops and crossties A_{sh} to be calculated from the following equations:

$$A_{sh} = 0.30\, sh'' \,(f_c / f_{yh})\,(A_g/A_c - 1) \quad (1)$$

but not less than
$$A_{sh} = 0.09\, sh'' \,(f_c / f_{yh}) \quad (2)$$

where A_{sh}= total cross-sectional area of hoops and crossties in each set; A_g= gross area of column section; A_c= area of column core bound by stirrups; f_c= compressive strength of concrete cylinder; f_{yh}= yield strength of stirrups; h''= core dimension of the column; and s = spacing of stirrups.

Satisfying these requirements results in joints that are congested with reinforcement and impractical to construct. For all specimens, the joint transverse reinforcement ratio ρ_{sh} was kept well below the requirements of the Committee 352 as shown in Table 1.

For all the six specimens, the terminating beam longitudinal bars were hooked within the transverse reinforcement of the joint using 90 deg standard hook. Due to the lack of data about the required development length in HSC, the required length was calculated using the equation recommended by the Committee 352 for ordinary strength concrete. The provided length measured from the critical section defined by the recommendations was little less than that required (about 135 mm).

2.2 Materials

A concrete mix using ordinary portland cement and limestone coarse aggregate with maximum dimension of 12 mm was used. The mix proportions for 1 m^3 of concrete consisted of 545 kg cement, 55 kg fly ash, 1100 kg coarse aggregate, 635 kg sand, 165 kg water and 20 liter superplastizer. The slump of the mix was approximately 55 mm. The average cylinder concrete compressive strength after 28 days for each specimen is presented in Table 1. The minor difference in concrete compressive strength was assumed to have no significant influence on the test results.

2.3 Loading Setup and Instrumentation

The specimens were tested with the column portions placed vertically in the steel frame as shown in Fig. 2. An axial load was applied to the column and kept constant throughout each test as given in Table 1. The free ends of the beams were subjected to several slow load reversals simulating very severe earthquake loading by two 120 KN hydraulic jacks. The typical loading sequence is shown in Fig. 3. The first cycle was load controlled in the elastic range up to 75% of the theoretical flexural strength of the beam, as calculated on the basis of the measured material strengths, and was followed by a series of deflection controlled cycles in the inelastic range until a displacement ductility factor (μ) equal to 7. Approximately 36 electrical strain gauges were bonded to the reinforcing steel at the critical locations near and within the joint region of each specimen. In addition, sufficient instrumentation was provided to monitor the deflections of the beam and the deformations in the potential plastic hinge region.

3. DISCUSSION OF TEST RESULTS

3.1 General Performance and Effect of Inclined Bars

The overall behaviour of the specimens is best described by plots of the applied load versus the displacement of the beam at the load point as shown in Fig. 4. All the HSC specimens exhibited excellent ductile hysteretic response until the loading was terminated at storey drift equal to 4.9% (μ=6) for specimens JC2 and JX2, and 4.2% (μ=7) for specimens JC1, JX1, JC3, JX3. The flexural strength ratio M_R of the specimens was equal to 1.4 or more as required by the 352 Committee. Therefore, all the specimens failed due to flexural hinging at the end of the beam. Only

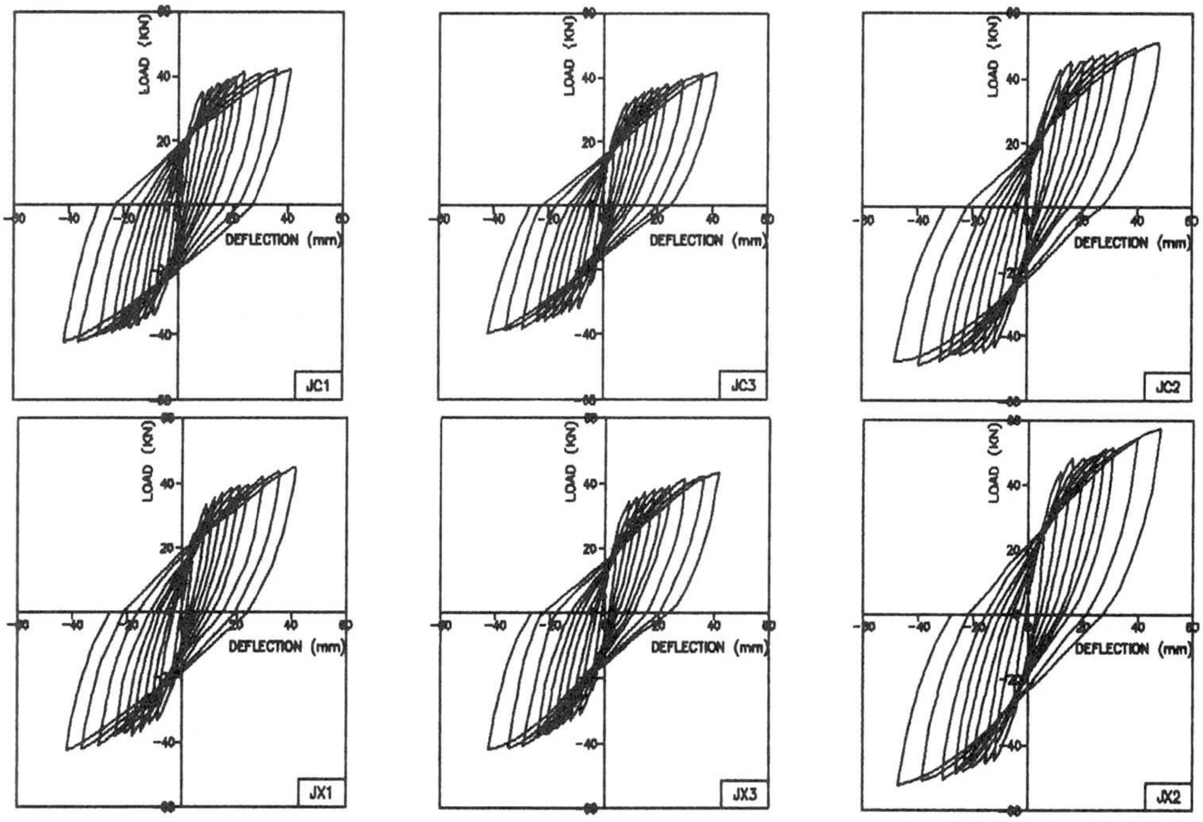

Fig. 4 Load-displacement response for the specimens

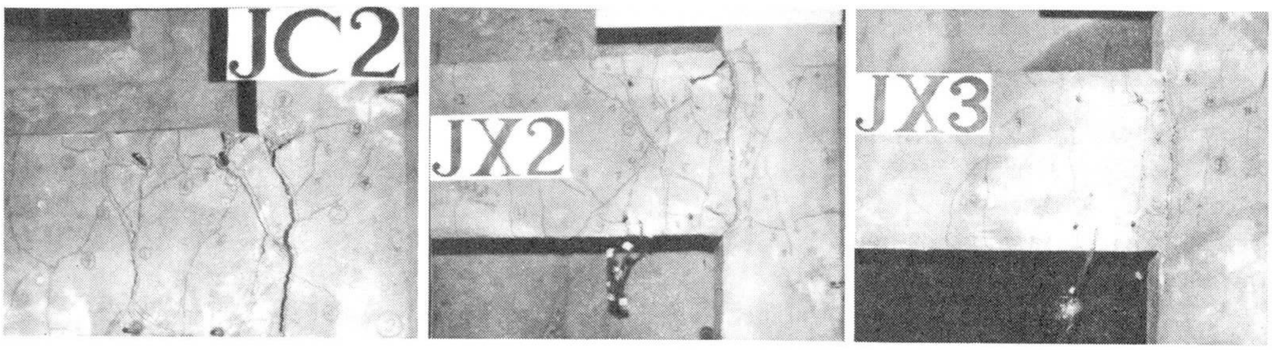

Fig 5 Cracking in the joint region for three specimens at the end of the test

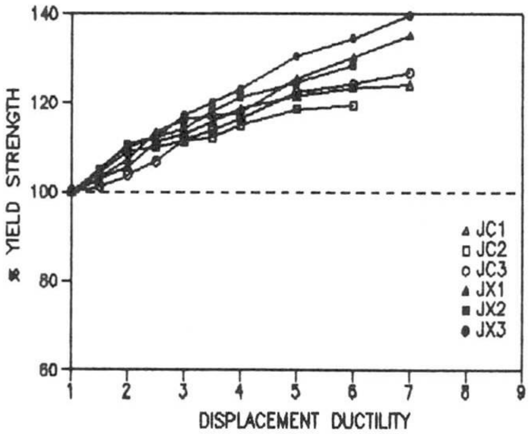

Fig. 6 Cyclic load-carrying capacity of the specimens

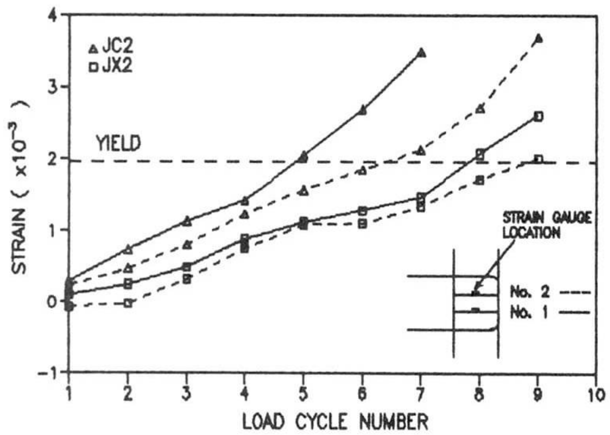

Fig. 7 Maximum strain in joint core hoops of specimens JC2 and JX2 during the positive loading runs of each cycle

minor cracking occurred in joint region for specimens JC1, JX1, JC3, and JX3, while in specimens JC2 and JX2, which had the largest joint shear stress, there were many more and larger cracks as shown in Fig. 5. In order to compare the performance of X-Specimens and C-Specimens a plot of percent yield strength versus the displacement ductility is shown in Fig. 6. The yield load and displacement for each specimen were measured from the strain gauge data of the beam longitudinal reinforcement at the face of the column. The presence of inclined crossed bars within the joint core of X-Specimens enhanced its load carrying capacity by more than 10% compared with that of C-Specimens. Specimen JX2 with 4D12 inclined bars had less load carrying capacity than JC2 with 4D10 inclined bars. This showed that increasing the amount of inclined bars had a significant improvement when the joint shear stress was low.

3.2 Effect of Joint Transverse Reinforcement

The required joint transverse reinforcement using Eq. (1) and Eq. (2), as recommended by the committee 352, is linearly proportional to f_c. For joints constructed with HSC (f_c >50 MPa), these equations lead to very large amounts of confinement steel, as shown in Table 1, and it is practically impossible to construct a joint with such amount of stirrups. For the C-Specimens, $\rho_{sh} \approx 0.2$ to 0.5 of ρ_{ACI} required by Eq. (1). However, all C-Specimens behaved very satisfactory. Plots of strains measured on the joint stirrups for specimens JC2 and JX2 is shown in Fig. 7. Yielding of the joint hoops and crossties was recorded from cycle 5 onwards (from μ=3). However, in non of the specimens did the cover concrete of the back of the column opposite to the beam separate from the joint core. The tests showed that the major part of joint shear was resisted by the diagonal compression strut even after severe load reversals, and using HSC increased proportionally its strength. Fig. 7 demonstrates that the hoops of C-Specimens accepted more tension than that of X-specimens despite the fact that C-Specimens had greater amount of joint hoops. This was because the crossed inclined bars accepted more shear stress than the intermediate longitudinal column bars.

4. CONCLUSIONS

From the results of this experimental study, the following can be concluded:

1. Exterior beam-column joints constructed with high-strength concrete (f_c = 69 to 77 MPa) showed excellent ductile behaviour up to 4.9% storey drift.
2. The minimum amount of transverse reinforcement required by the ACI-ASCE Committee 352 recommendations for Type 2 HSC exterior beam-column joints may be safely reduced by at least 50%, for the cases with γ <12 in combination with $M_R \geq 1.4$, without loss of ductility requirements.
3. Exterior beam-Column joints with crossed inclined reinforcing bars performed considerably better than those with conventional reinforcement. The presence of inclined reinforcing bars within the joint core also resulted in a reduction of the joint transverse reinforcement to only about 33% Of the requirements of the Committee 352 recommendations.

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