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NEW DETAILS OF HSC BEAM-COLUMN JOINTS FOR REGIONS OF LOW TO MODERATE SEISMICITY

Jing LI[1], Hoat Joen PAM[2] and Francis Tat Kwong AU[3]

SUMMARY

An experimental investigation was conducted to examine the feasibility of two kinds of new joint detailing for low to moderate seismic risk regions. Particular attention was paid to beam-column joints cast of high-strength concrete (HSC). Joints without transverse reinforcement (E detail) commonly adopted in low to moderate seismic risk regions have been proven unsatisfactory for strength and ductility requirements under seismic loading, while conventional joints containing transverse reinforcement (H detail) commonly adopted in seismic regions cannot avoid reinforcement congestion. Two kinds of new joint details, i.e. AD (adding diagonal steel bars in the joint) and CD (bending some of the beam longitudinal reinforcement bars diagonally up and down in the joint) are proposed in this study to satisfy the limited ductility requirements for low to moderate seismic risk regions. Four half-scale beam-column joint specimens cast of HSC containing respectively E detail, H detail, AD detail and CD detail were fabricated and tested under reversed inelastic cyclic displacement excursions. The test results showed that, depending on the reinforcement detailing within the beam-column joints, different failure modes occurred. Units E and CD could hardly reach the limited ductility range and the joint suffered severe inelastic damage, while Units AD and H behaved excellently with the joint relatively well intact. It is concluded that the detail of Unit AD is suitable for joints in regions of low to moderate seismicity due to ease of fabrication compared to Unit H.

INTRODUCTION

The introduction of silica fume and superplasticizers makes it relatively easy to produce ready mixed high-strength concrete (HSC). The development and application of HSC have greatly increased in the last several decades all over the world. HSC is not only applied to offshore concrete structures and bridges, but also to high-rise buildings, prefabricated or precast elements, pavements, etc. When the strength of concrete gets higher, some of its characteristics and engineering properties become different from those of

[1] Lecturer, Department of Civil Engineering, South China University of Technology, Guangzhou, P.R.C. Email: jingli@graduate.hku.hk

[2] Associate Professor, Department of Civil Engineering, The University of Hong Kong, Hong Kong, P.R.C. Email: pamhoatjoen@hku.hk

[3] Associate Professor, Department of Civil Engineering, The University of Hong Kong, Hong Kong, P.R.C. Email: francis.au@hku.hk

normal-strength concrete (NSC) as described by the report of ACI Committee 363 [1]. These differences in material properties may have important consequences in terms of the structural behavior and the design of HSC members. Linear extrapolation of the existing design equations to cover a higher range of concrete strengths may lead to either unsafe and therefore potentially dangerous design, or overconservative and hence unduly restrictive design. Hence, further research of the design on HSC members should be conducted.

In 1975 Park and Paulay [2] clarified that beam-column joints can be the critical regions in reinforced concrete frames designed for inelastic response to seismic loading. NZS 3101 [3] and ACI 352 [4] propose conventional joint detailing, which uses transverse reinforcement (Detail H) to ensure ductile behavior in the joint. The full ductility requirements in beam-column joints to sustain severe post-elastic deformations will result in sophisticated reinforcement detailing as well as reinforcement congestion. As a result, this type of joint detailing is not adopted by many countries located in low to moderate seismic risk regions.

No design and detailing provisions for beam-column joints are required in BS 8110 [5], which is adopted in many regions with low to moderate seismic risk. Therefore, no transverse reinforcement is installed in the beam-column joint (Detail E). Hanson [6] and Uzumeri [7] showed that this kind of joint was potentially critical to the safety of structures in resisting seismic attack, even for low to moderate earthquakes.

Consequently, limited ductility requirement, which is between elastic and ductile requirements, is appropriate for beam-column joints in low to moderate seismic risk regions. According to the New Zealand Loading Standard NZS 4203 [8] and Concrete Design Standard NZS 3101 [3], the displacement ductility factor μ should be more than 1.25 and less than or equal to 3 for limited ductility requirement.

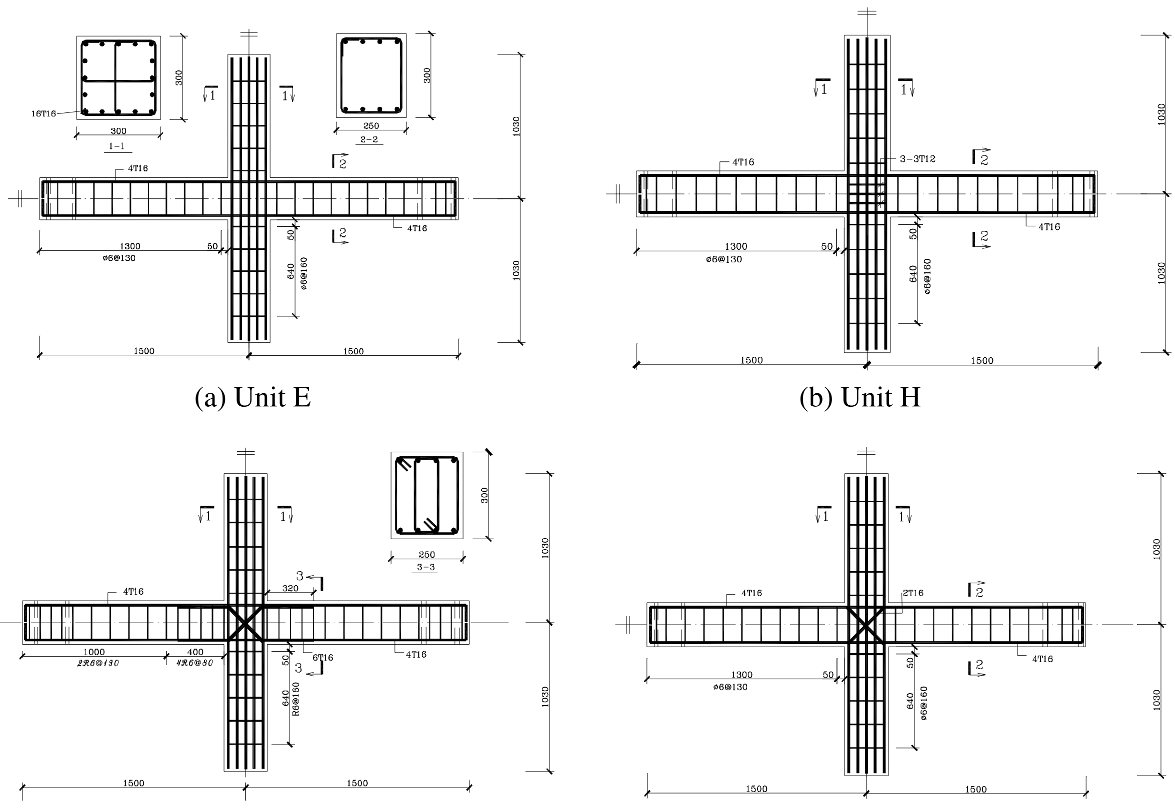
The authors proposed two kinds of beam-column joint detailing to satisfy the limited ductility requirement and avoid reinforcement congestion, i.e. adding diagonal steel bars (Detail AD) and bending diagonally the beam longitudinal steel bars (Detail CD) in the joint. Experimental investigation was conducted to examine the seismic behavior and the feasibility of these new joint details.

EXPERIMENTAL PROGRAM

Four half-scale beam-column joint specimens constructed with HSC containing respectively E detail, H detail, AD detail and CD detail were tested under reversed inelastic cyclic displacement excursions. All the columns were subjected to zero axial load.

Test units and material properties

The dimensions and reinforcement layout of the beam-column joints are shown in Fig. 1. All the units had dimensions of 250mm×300mm and 300mm×300mm respectively for the beam and column cross-sections, 3000mm and 2060mm respectively for the beam length and column height. Two types of steel were used for the reinforcement of the test units, i.e. high yield deformed steel, symbolized by T, for the longitudinal bars of the columns and the beams, as well as the transverse reinforcement of the joint; and mild steel round bars, symbolized by R, for the stirrups in the columns and the beams. The longitudinal reinforcement details of the beam and column in all the units were identical, i.e. the beams contained 4ϕ16mm deformed high yield steel bars (4T16) installed at each of the top and bottom faces, while the columns contained 16T16 bars distributed evenly around the perimeter of the column core section. As shown in Fig. 1a, Unit E contained no joint reinforcement apart from the longitudinal steel bars of the column and the beam. Unit H, which was designed according to NZS 3101 [3], contained 3 sets of steel hoops each with one crosstie (Fig. 1b). Diagonal steel bars in the form of "obtuse Z" were installed in two opposite directions of the joint of Unit AD. The length of the horizontal tail projecting out from each side of the column face was set at about one-half of its full flexural development length as defined in NZS 3101 [3], i.e. 20 times its bar diameter, and the diagonal component was more or less in line with the diagonal direction of the joint (Fig. 1c). Extra transverse steel was added within the beam plastic hinge zones of Unit AD to avoid premature shear failure associated with the flexural strength enhancement caused by the tail end of the diagonal bars. The diagonal steel bars of Unit CD were bent from the beam longitudinal steel bars (Fig. 1d). The detailed calculations of the reinforcement of these test units are described in [9].



(a) Unit E

(b) Unit H

(c) Unit AD

(d) Unit CD

Fig. 1 Test units

Normal weight concrete was used for the test units. Some cement was replaced by silica fume, and superplasticiser was added to get an ample slump. The measured compressive strengths of concrete cubes at 28-day and the date of testing are listed in Table 1. The properties of steel reinforcement, including yield strength f_y, Young's modulus E_s and ultimate strength f_u, are listed in Table 2.

Table 1 Concrete compressive strength

Test Units	f_{cu} at 28-day (MPa)	f_{cu} at test (MPa)
E	81.0	88.2
H	82.2	82.7
AD	73.4	79.7
CD	79.4	82.5

Table 2 Steel properties

Steel	f_y (MPa)	E_s (MPa)	f_u (MPa)
R6	374	203,000	491
T12	533	204,000	648
T16	557	207,000	650

Test set-up, instrumentation and test procedure

The test set-up is shown schematically in Fig. 2. Each unit was tested in a self-reaction steel-loading frame. The column was held in place by two hinges, each attached respectively to the top end and bottom end of the column, which simulated the inflection points. Reversed cyclic quasi-static loading simulating earthquake forces were applied by a pair of 500kN MTS servo-hydraulic actuators, Jack 1 and Jack 2, each with a built-in load cell for measuring the applied vertical forces.

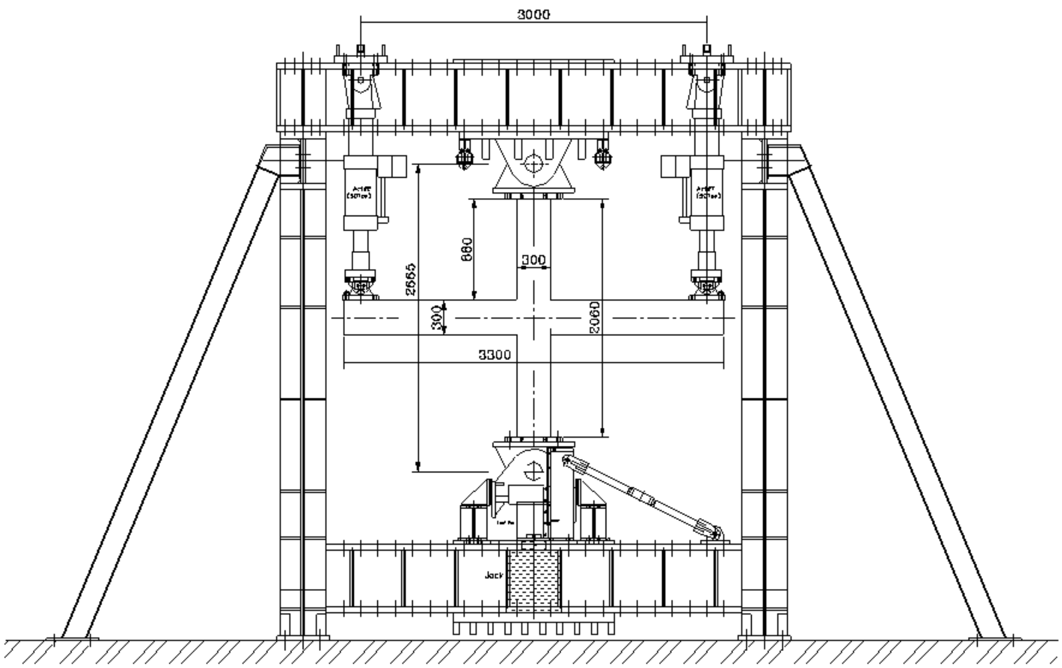


Fig. 2 Schematic test set-up

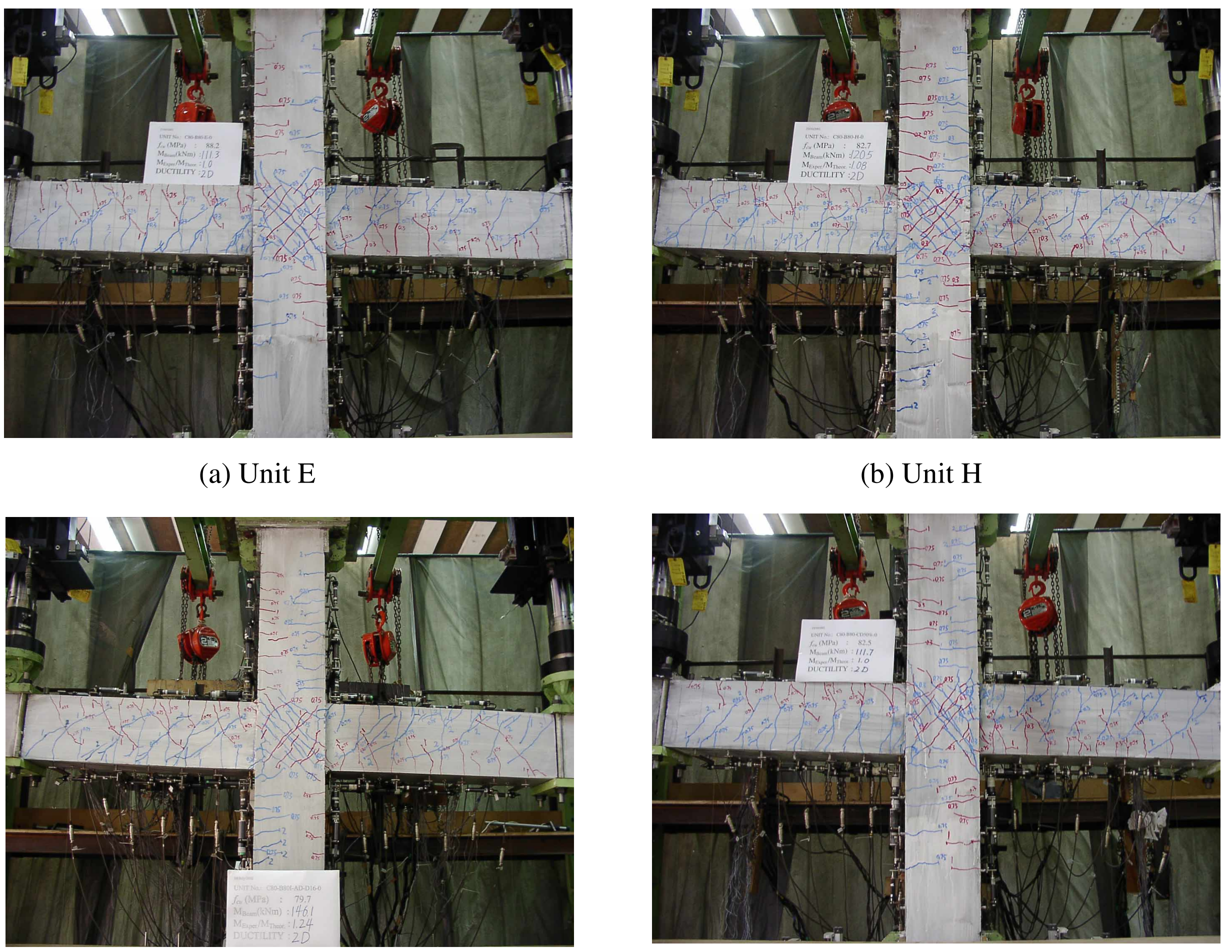
About 60 linear variable differential transducers (LVDTs) were installed on each test unit to measure the beam deflections, beam curvatures, column curvatures and joint distortions. All the LVDTs were connected to a data logger, which transferred the data to a computer. In addition, about 100 electrical strain gauges were installed in each unit to monitor strain variations of the selected reinforcement bars in the beams, the columns and the joint. The details and results are discussed and presented elsewhere [9].

Load control was applied in the test for the first cycle with the peak load set at 75% of the theoretical strength of the test unit. The average peak displacement obtained for this cycle is referred to as $0.75\Delta_y$, from which the nominal yield displacement Δ_y can be easily calculated. From the second cycle onwards, displacement control was adopted. At the second cycle, the peak displacements were $+\Delta_y$ and $-\Delta_y$ for respectively the positive and negative half cycles. The displacement ductility factors, μ, at this stage were respectively equal to +1 and –1, in which μ is defined as the ratio of peak displacement to the nominal yield displacement. From the third cycle onwards, μ was increased by 1 for every two cycles until the load carrying capacity was less than 80% of the measured maximum load.

TEST RESULTS AND DISCUSSION

Test observations

In all the units, diagonal cracks occurred in the joint during the first loading cycle. When the displacement ductility factor μ reached 2 for the first time, the cracks in the joints of Units E and CD widened considerably and also concrete crushing appeared on the columns near these joints (Figs. 3 a and d). For Unit H, concrete crushing happened in the region of the beam close to the joint panel, while at the other beam one wide crack appeared at the tension edge (Fig. 3c). The maximum crack width in the joint of Unit E was 2.2mm, while that of the other units was about 0.7mm. Fig. 3 shows that at μ=2, the behavior of Unit AD was the best.



(a) Unit E

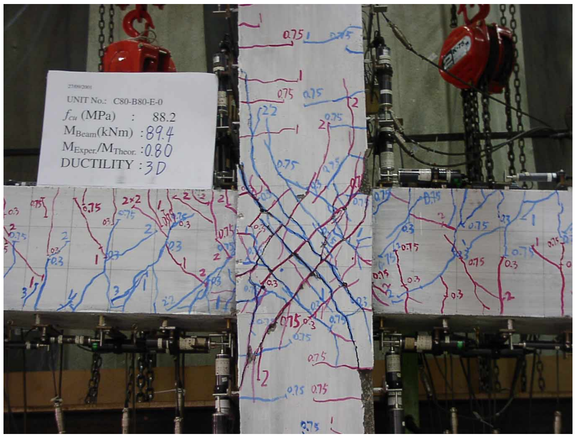
(b) Unit H

(c) Unit AD

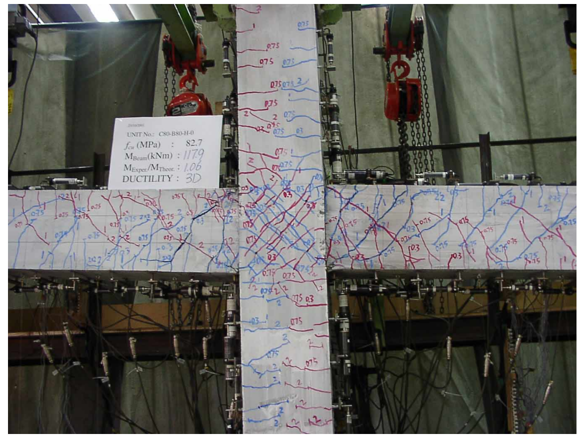
(d) Unit CD

Fig. 3 Crack patterns of the units at μ=2

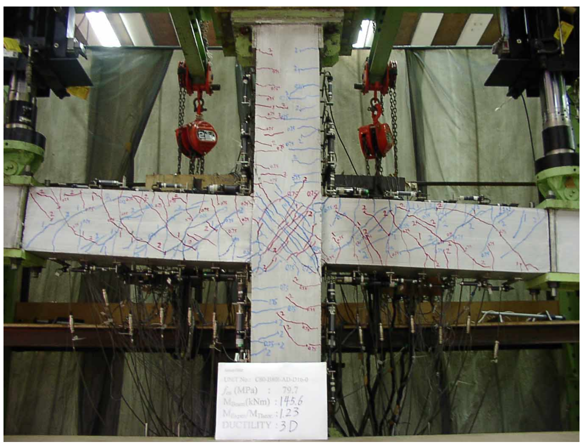
At the peak of the first positive cycle of μ=3, concrete spalled from the upper and bottom columns as well as the joint panel of Unit E, while the condition of Unit CD was even worse (Figs. 4 a and d). Unit AD still maintained the best condition so far, with no large cracks appearing on the unit (Fig. 4c), while the condition of Unit H was quite satisfactory, except for two apparent shear cracks in the beams (Fig. 4b).



(a) Unit E



(b) Unit H



(c) Unit AD

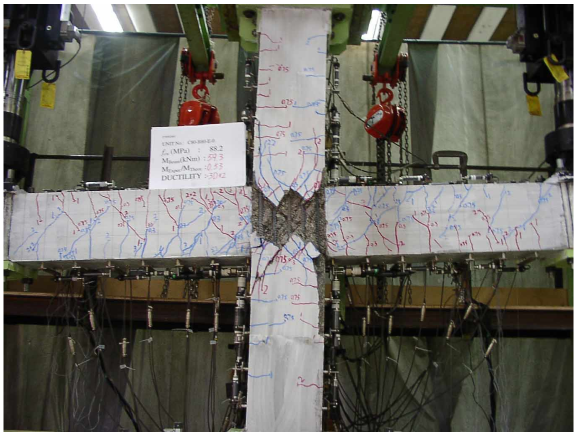
(d) Unit CD

Fig. 4 Crack patterns of the units at μ=3

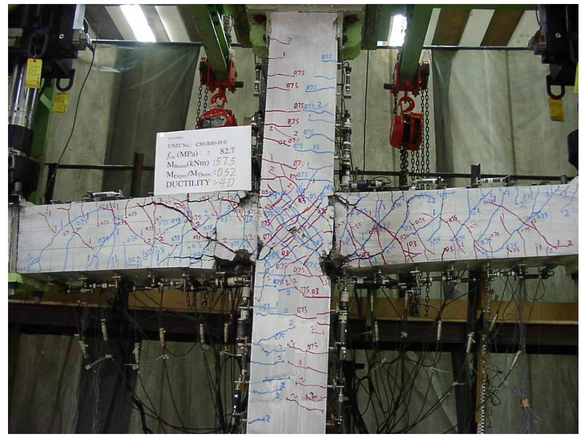
During the second half cycle of μ=-3, concrete in the joint panel of Unit E continued to spall, while the load carrying capacity dropped considerably until the test was stopped. Unit H, Unit AD and Unit CD were stopped respectively at μ =-4, -5 and –5. Fig. 5 shows that, except for Unit H that was dominated by beam failure, the others were dominated by joint failure. With reference to the crack pattern at μ =3, Unit AD can be considered to satisfy the limited ductility requirement.

Hysteretic loops

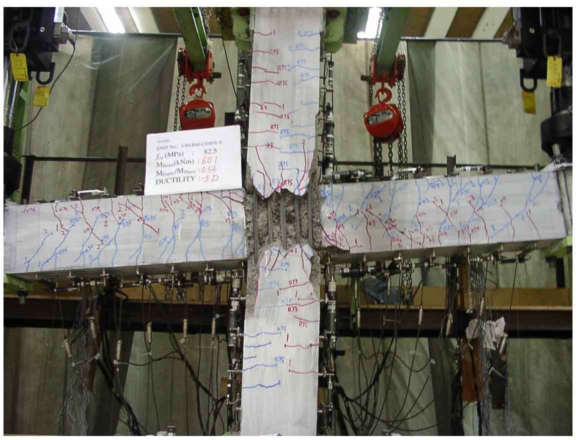
The column shear force – drift hysteresis loops for the units are shown in Fig. 6. Unit E and Unit CD only managed to reach the respective theoretical capacity at the first positive peak of cycle of μ=2, beyond which the strength degraded at every second cycle of the same ductility factor. As a result, bending part of beam longitudinal reinforcement up and down in the joint cannot improve the joint shear strength. The strength enhancements of Unit H and Unit AD were 6% and 26% respectively. This strength enhancement in Unit AD was due to the contributions of the horizontal tails of the diagonal steel bars on the beam moment capacity and the diagonal steel bars on joint shear capacity.



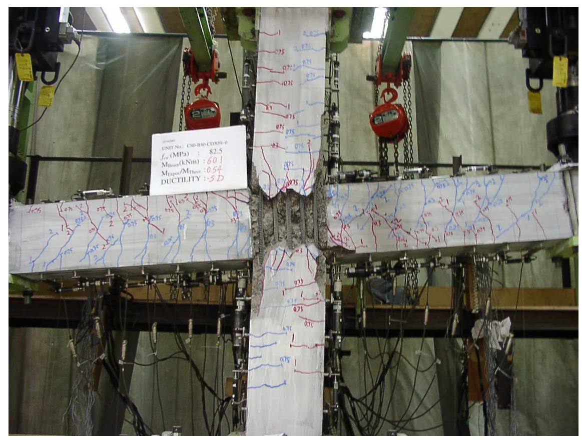
(a) Unit E at μ=-3×2



(b) Unit H at μ=-4

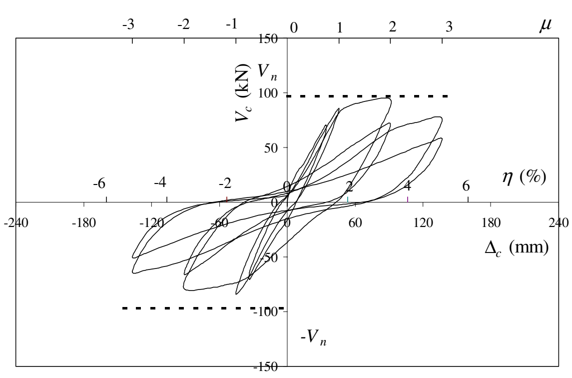


(c) Unit AD at μ=-5

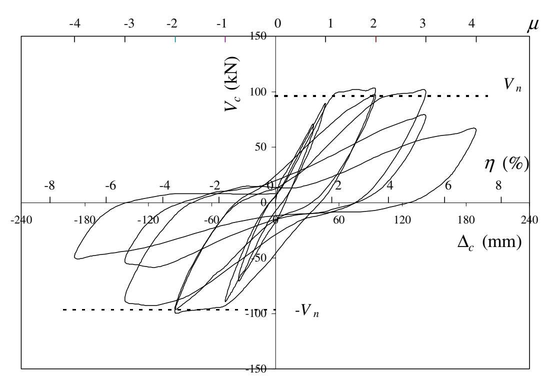


(d) Unit CD at μ=-5

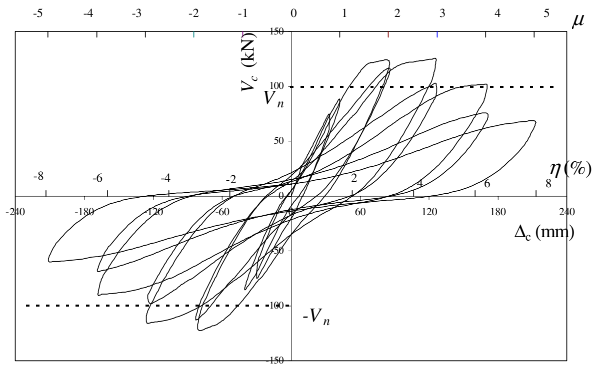
Fig. 5 Test units at failure



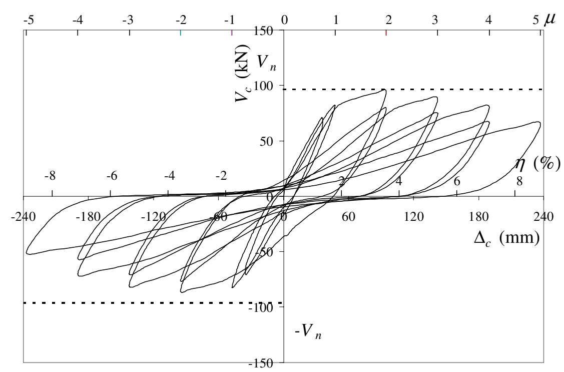
(a) Unit E



(b) Unit H



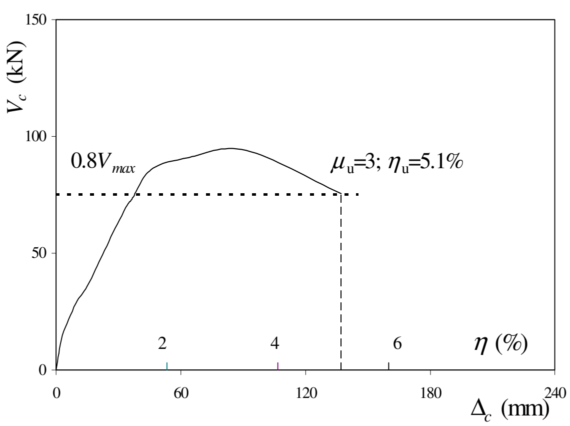
(c) Unit AD



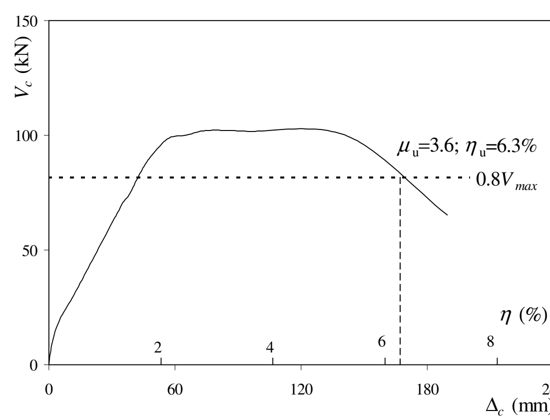
(d) Unit CD

Fig. 6 Column shear force – drift hysteresis loops

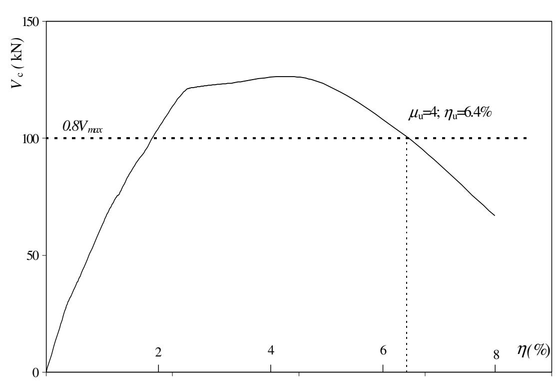
Failure was assumed to occur at 80% of the maximum load carrying capacity achieved during the test. The displacement ductility factor at failure, which is also termed the ultimate displacement ductility factor, μ_u, and the ultimate column drift ratio, η_u, are obtained from interpolation on the envelope of the respective force-displacement response, which are shown in Fig. 7. It can be observed from the figure that at failure, the units can be arranged from the least to the most ductile in the following sequence: Unit E with μ_u=3 or η_u=5.1%, Unit H with μ_u=3.6 or η_u=6.3%, Unit AD with μ_u=4 or η_u=6.4% and Unit CD with μ_u=4.4 or η_u=7.7%.



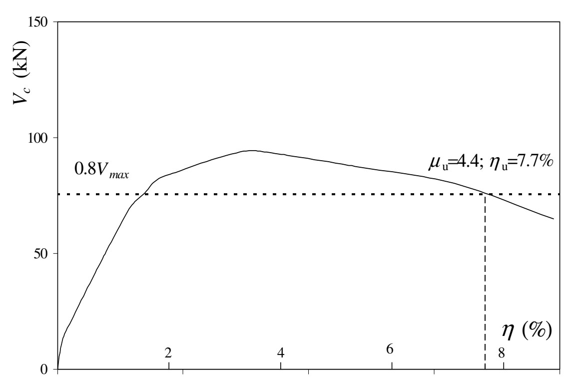
(a) Unit E



(b) Unit H



(c) Unit AD



(d) Unit CD

Fig. 7 Envelopes of the column shear force – drift hysteresis loops

The beam nominal moment capacity, the ratio of the measured maximum column shear strength to the nominal strength (V_{max}/V_n) and the ultimate displacement ductility factor and drift ratio of each specimen are shown in Table 3.

Table 3 Summary of test results

Test Unit	M_n (kNm)	Failure mode	V_{max}/V_n	At failure	
				μ_u	η_u (%)
Unit E	114.6	Joint brittle failure	0.97	3.0	5.1
Unit H	114.2	Beam failure	1.06	3.6	6.3
Unit AD	117.9	Joint ductile failure	1.26	4.0	6.4
Unit CD	114.2	Joint ductile failure	0.98	4.4	7.7

Strength degradation

Fig. 8 shows the strength degradation of all the units. The maximum load carrying capacity in all the units happened at the peak of the first positive loading cycle of μ=2, except in Unit AD, which happened at the peak of the first positive loading cycle of μ =3. It could be observed that even with joint failure, Unit AD obtained the largest maximum load carrying capacity enhancement and possessed the best load retention ability. At the limited ductility level, i.e. μ lying between 2 and 3, Unit AD performed the best followed by Unit H, Unit CD and Unit E. It should be noted that all the specimens have always rebounded in strength when they underwent a new path of inelastic displacement excursion towards a higher ductility factor. This rebound in strength happened in the most stable manner in Unit CD.

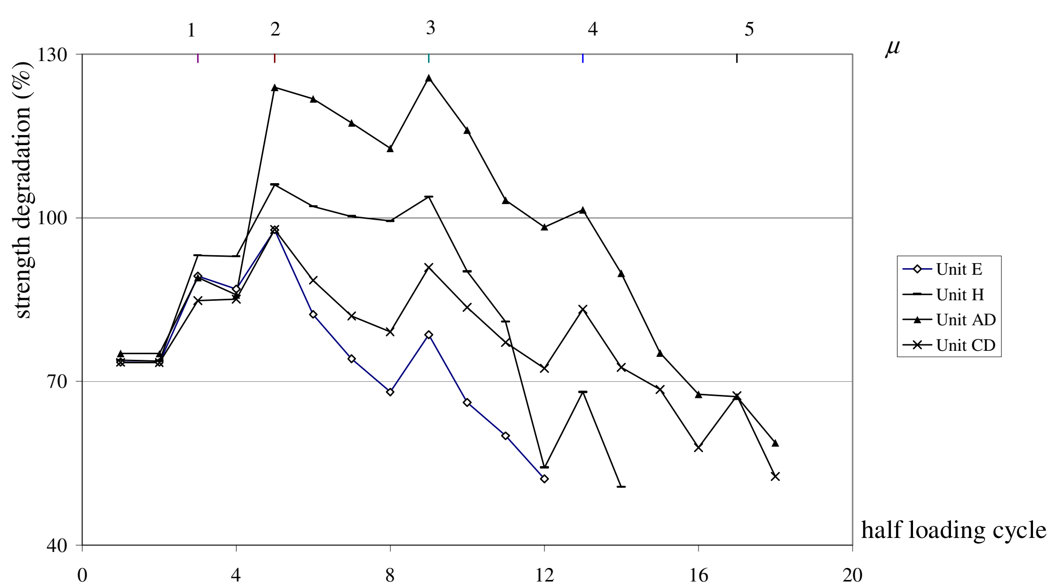


Fig. 8 Strength degradation

Energy dissipation

Energy dissipation is a very important characteristic in seismic resistant design of structures. Taking in each unit the energy absorption under the first loading cycle as the reference value E_a, the energy dissipation index, I_E, can then be defined as the ratio of the energy absorbed during any full loading cycle E_b to E_a,

$$I_E = \frac{E_b}{E_a} \tag{1}$$

Comparison of the energy dissipation indexes among the units is shown in Fig. 9. If the unit's performance is evaluated based on the energy dissipation capacity during the whole test, Unit AD and Unit CD performed the best. Although these two units failed in the joint, their energy absorption capacities were stable; therefore the failure pattern of these two units is considered as ductile joint failure. The rest of the units can be arranged from good to bad in terms of their energy dissipation in the following order: Unit H and Unit E.

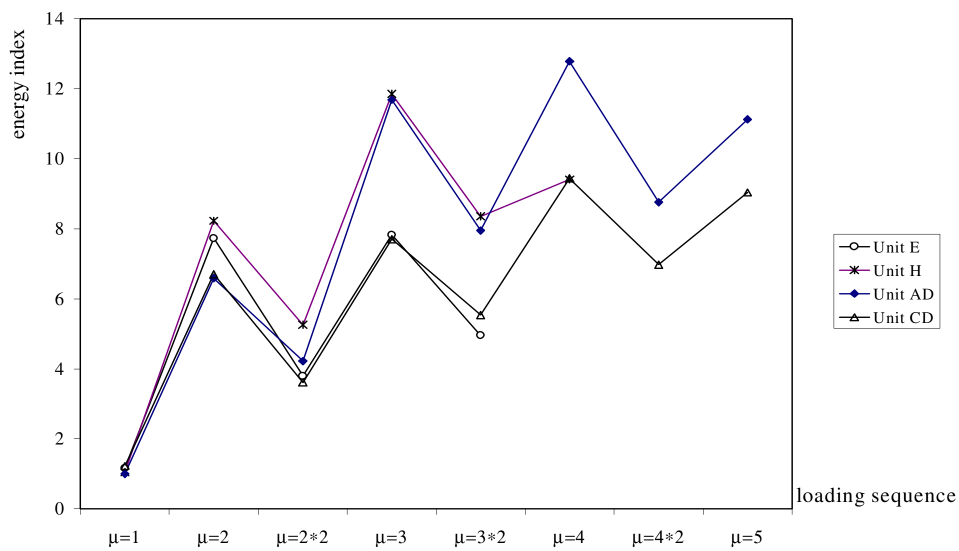


Fig. 9 Energy dissipation

CONCLUSIONS

Four HSC beam-column joints have been fabricated and tested to investigate their behavior at limited ductility levels. They include the newly proposed beam-column joint details (Units AD and CD), those commonly adopted in low to medium seismic regions (Unit E) and the conventional joints commonly adopted in seismic regions (Unit H). Based on test observation and results, the following conclusions are drawn.

1. Under reversed cyclic loading, the load carrying capacity of Unit E could reach its theoretical nominal capacity. However, the ductility is not acceptable even for limited ductility levels. Brittle joint failure, which is not preferred in a reinforced concrete frame structure, occurred.
2. Although Unit AD cannot avoid joint failure at the final stage, the displacement ductility factor is much improved. In addition, the load carrying capacity was enhanced by 26%. Hence, this kind of joint detailing can satisfy the limited ductility requirements.

3. Although Unit CD did not experience enhancement of the joint shear strength, its displacement ductility factor was also much improved, and it was the largest among all the four units.
4. With reference to strength degradation and energy dissipation, Unit AD performed the best. Therefore, Detail AD is appropriate for beam-column joints in low to moderate seismic risk regions.

ACKNOWLEDGEMENTS

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