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REHABILITATION OF NONDUCTILE BEAM-COLUMN JOINT USING CONCRETE JACKETING

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SUMMARY

Simulated seismic load tests on reinforced concrete one-way interior and exterior beam-column joints with substandard reinforcing details typical of low-rise buildings constructed in Taiwan are described. These substandard reinforcing details of the beam-column joints are mainly lack of transverse reinforcement and inadequate beam bar hooks bent outwards the joint core. RC jacketing offers a versatility for retrofitting those deficient buildings, which considers a simple way not only in construction but also in design. This paper discusses ongoing recent research works carried out at National Central University, Taiwan. The improvement in performance of the joints rehabilitated with RC jacketing is demonstrated. Experimental investigation on four full-scale one-way beam-column sub-assemblages and comparison of measurement with recent predicted models are included in the study.

INTRODUCTION

In Taiwan, many joint shear failures were found in reinforced concrete (hereafter called as RC) buildings, especially for low-rise apartments, during 1999 Chi-Chi earthquake [1]. The reason for the failure was mainly due to low concrete strength, no horizontal shear reinforcement, and/or inadequate reinforcing details in joints, as shown in Fig.1.

Nowadays, retrofitting of structures has been undertaking in Taiwan after structural damage caused by the Chi-Chi earthquake or because existing structures were required to comply with newly seismic code provision [2]. Several retrofit techniques suggested in the literatures [3-5], including the use of concrete jackets, bolted steel plates, and FRP sheets, were considered in the structural upgrading, especially for columns and beam-column joints in the moment-resisting frames. The purpose of the rehabilitation is to prevent columns or joints from a brittle shear failure, and shift the failure towards a beam flexural hinging mechanism that is a more ductile behavior.

Among these retrofit techniques, the RC jacketing applied to columns was widely used in Taiwan after 1999 earthquake. This is because concrete jacketing is more consistent with as-built RC structures than the other retrofit materials, such as steel or FRP jacketing, and the deficient beam-column joints can be

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easily repaired as well. Meanwhile, the retrofit scheme is simple, easy, and thus cost-saving, comparing with others.

In this paper, two analytical models [6-7] for assessing shear strength of as-built RC beam-column joints were adopted to evaluate the non-ductile joints retrofitted with RC jacketing. A test program comprising four tests on interior and exterior joints was carried out to observe the seismic behavior and to examine the accuracy of prediction obtained from those models.

MODELS USED IN THE PREDICTION OF TEST UNITS

Many evaluation models can be used for predicting the shear strength of beam column joints. Nevertheless, so far, the theoretical evaluation on the retrofitted joints is still underway. Thus, two of previous models for assessing the shear strength of existing joints were chosen for the study. Special concern will be focused on the predicted shear strength of the joints with RC jacketing.

Joint-Shear-Degradation Model (Hakuto Model)

Hakuto et. al. [6] based on tests on as-built interior beam column joints without joint shear reinforcement to propose an assessment model for joint shear strength. He suggested that for beam-column joints without shear reinforcement the maximum probable horizontal joint shear force that can be resisted is:

$$V_{jh} = v_{ch}\, b_j \mathrm{h} = \mathrm{k}\sqrt{f_c^{'}}\ \sqrt{1+\frac{N^*}{A_g k\sqrt{f_c^{'}}}}\ b_j \mathrm{h} \tag{1}$$

where v_{ch} (in MPa) = nominal horizontal joint shear stress carried by a diagonal compressive strut crossing the joint, b_j (mm) = effective width of the joint, h(mm) = depth of column, A_g (mm^2) is the gross area of column, N^* (in Nt) is the axial load on columns. The degradation of joint shear strength is expressed in terms of k in Fig.2.

Softened Strut-and-Tied Model (SST Model)

The model proposed by Hwang et. al. [7] was derived starting with the basic concept of strut-and-tie model (or truss model) [8] for force equilibrium. Then, strain compatibility and constitutive laws of cracked reinforced concrete were introduced into the equilibrium to set five equations and finally solve five unknown parameters. In the model, concrete softened laws were taken as given by Zhang and Hsu [9].

More simplified solution was further revised using 449 measured data available in the authors' and other existed experiments of beam-column joints, deep beam, corbels, and squat walls. The purpose of this is to obtain a simple evaluation of the nominal diagonal compressive strength C_{dn}, presented in equation (2), for engineers.

$$C_{d,n} = K\zeta\ f_c^{'} A_{str} \tag{2}$$

$$\xi \approx 3.35/\sqrt{f_c^{'}} \le 0.52 \tag{3}$$

where ζ is the softening coefficient of concrete, approximately estimated by equation (3). A_{str} is the effective area of diagonal compressive strut. K is the strut-and-tie index with horizontal and vertical ties, defined in detail by the study [7].

TEST PROGRAM

Details of the test units

Four full-scale one-way beam-column joint units were constructed in the test. Two were interior joints and two were exterior ones, as depicted in Fig.3. Each unit was cast in the horizontal plane. Concrete compressive cylinder strengths were 20 MPa for as-built units and 41 MPa for jacketing concrete. The yield strengths of reinforcing bars #3 (D10), #6 (D19), and #8 (D25) were 412 MPa, 520 MPa, and 461 MPa, respectively.

The as-built joint units JI1 and JE1 were made as was typical of pre-1980s construction in Taiwan, in which no horizontal shear reinforcement was arranged. The beam bar hooks of exterior joint unit JE1 bent toward joint core were one of reinforcing types found in Taiwan. The other type of beam bars bent away from joint core, also seen in Taiwan, was not described in the study.

The thickness of 100 mm of RC jacketing for units JIR1 and JER1 was taken into account, differing from a practical thickness of at least 150 mm. For the purpose of lowering labor cost, no anchor bolt was installed in the column before casting concrete jacketing. This is one of goals to observe the performance of jacketed joints without embedding anchors. However, special care on the premature bond slip in the interface between new and old joint core has to be taken.

When estimating joint shear strength of the jacketed units, a consistent concrete strength in the joint cores was adopted by means of a root-mean rule, stipulated in equation (4) [3].

$$A_j \sqrt{f'_{c,j}} = A_1 \sqrt{f'_{c,1}} + A_2 \sqrt{f'_{c,2}} \tag{4}$$

where A_j is overall joint core area in the case of jacketed units. $f'_{c,j}$ is the weighted average concrete strength. A_1 is the gross area of existing column. $f'_{c,1}$ is the concrete strength in existing column at joint. A_2 is the area of column jacket included in the joint core, equaling to A_j - A_1. $f'_{c,2}$ is the concrete strength in jacketed column at joint.

For all the rehabilitated units, the failure mechanism of strong-column and weak-beam was taken into account. That is the demand for joint shear imposed into the core is lower than the capacity when the flexural strength of beams reached.

Test set-up

Fig. 4 shows the test set-up and test sequence. During testing, load controlled cycles were initially imposed to the units to find the secant stiffness and horizontal displacement at 75% of the estimated flexural capacity of the beam in each direction of loading. Displacement controlled cycles were then applied to the units when loaded beyond the elastic range. These cycles were controlled in terms of the displacement ductility, μ, which is defined as the ratio of the applied horizontal displacement Δ to the displacement at first yield of beams Δ_y. The displacement at first yield is defined as 4/3 times the horizontal displacement observed in the load-controlled cycles to 75% of flexural strength capacity of beams. Horizontal displacements were measured at the point of application of loading.

TEST RESULTS

General behavior

The hysteretic loops of observed horizontal shear force against displacement for all test units are shown in Figs. 5 to 8. Also shown in dashed lines of these loops are the theoretical strengths calculated when the beam hinges adjacent to the column established. The displacement ductility and story drift ratio are indicated in these figures as well. At the end of testing, crack patterns were marked as displayed in Figs. 5 to 8. It is obvious for as-built units JI1 and JE1 that joint shear failure occurred as crushing of concrete compressive struts at joint were found. At the same time, the measured maximum strength did not reach the theoretical value due to beam hinging mechanism. Testing was then stopped when horizontal shear was loaded down to 80% of maximum measured strength.

For rehabilitated units JIR1 and JER1, from hysteretic loops represented in Figs. 6 and 8, the failure mode of strong column and weak beam instead of joint shear failure was verified. It is clear that the imposed joint shear is lower than the joint shear capacity. Meanwhile, the yield strain measured at the steel reinforcement of beam ends indicated that the occurrence of plastic hinges at beams was confirmed. However, the displacement ductility for both units is different. Unit JER1 shows a better ductile behavior than JIR1, which is up to μ=4. It implies the jacketed exterior joint has a better seismic performance. It is noted that no shear reinforcement and dowel anchors between new and old concrete joint are provided in retrofitting, as mentioned previously. The reasons for the jacketed interior joint JIR1 with non-ductile failure could be attributed to the premature debonding occurred in the interface between new and old concrete of the joint region. However, detailed observation on the debonding phenomena is necessary for the future study.

Comparison between measurement and prediction

To compare with predicted values resulting from two models mentioned previously, measured joint shear is then obtained by means of the data reduction from the loads measured on the column tip and beam ends (see Fig. 4). Meanwhile, the difficulty in deciding the concrete strength for the joint shear capacity assessed by equations (1) and (2) is perceived. This is because the joint concrete strength for new and old casting is not consistent. In the following discussion, two concrete strengths were adopted into the model prediction, i.e. 20 MPa for as-built concrete strength and 31 MPa for root-mean strength suggested by equation (4).

For as-built joints shown in Fig. 9, SST model only predicts maximum joint shear strength whereas Hakuto model evaluates the whole path of the shear strength. It is clear for units JI1 and JE1 that the maximum joint shear strengths predicted by SST model are much closer to the measured values.

For the retrofitted joints, in comparison between JIR1 and JER1, the shear strength predicted by SST model is more reasonable than that estimated by Hakuto model. Considering the effect of concrete strength, prediction using as-built concrete strength (i.e. lower concrete strength) instead of root-mean one (i.e. higher concrete strength) resulted in a better approach to the measured joint strength. This was because the consistency of joint could not maintain all the way for the joint subjected to loading. Thus core joint came from lower concrete strength gradually became softened and discrepant with jacketed joint made from higher concrete strength, which might cause the interfacial debonding occur.

CONCLUSIONS

The test results indicate that the seismic performance of typical exterior and interior beam-column joints of Taiwan pre-1980s low-rise building frames without transverse reinforcement in the joint cores would be poor in a severe earthquake.

The jacketing of beam-column joints with new reinforced concrete was identified as a useful technique for enhancing the stiffness, strength, and ductility of poorly detailed as-built beam-column joint regions. The technique, however, is very labor-intensive and the placement of the new joint core hoops or anchors, passing through holes to be drilled in the existing beams, is difficult.

Comparing with interior joints, exterior joints with RC jacketing obtained a significantly better seismic performance. The reason for the jacketed interior joints having early non-ductile behavior is due probably to incomparable strut reaction took place at premature ductility μ=2 in the jacketed joint region, so called as debonding occurred in the interface between new and old concrete in the joint.

The predicted joint shear strength using softened strut-and-tie (SST) model obtained a satisfactory agreement on the measured values. The tendency of the shear degradation attained from the measured values of as-built interior and exterior units was similar to the evaluation made by Hakuto model. However, the accuracy of prediction on the retrofitted joints using two recent models needs further experimental verification. Special care of the premature debonding effect on the joint shear strength and related ductility should be taken.

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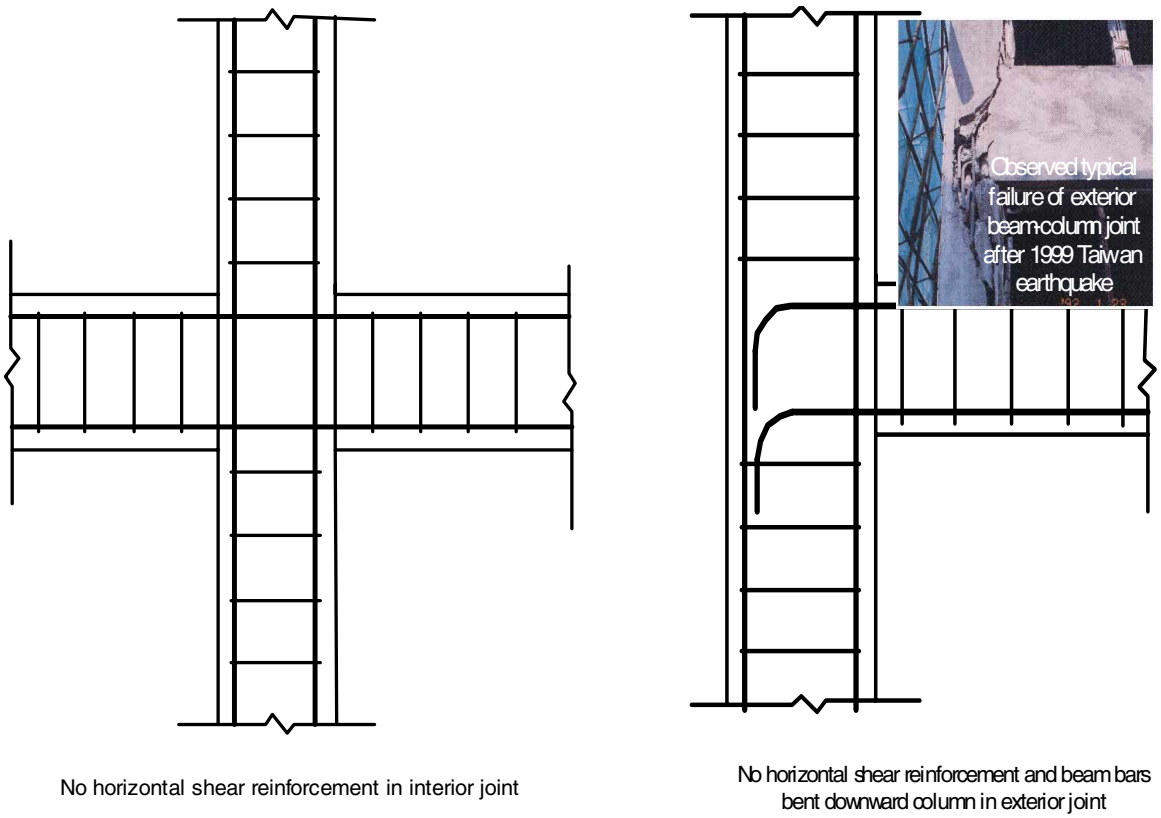

Fig.1 Deficiencies in beam-column joints

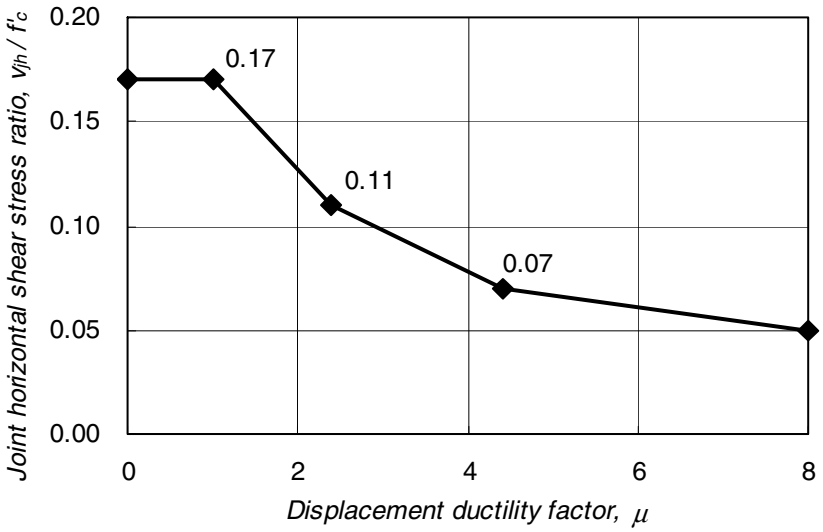

Fig.2 Degradation of concrete shear resisting mechanism of interior beam-column joints [6]

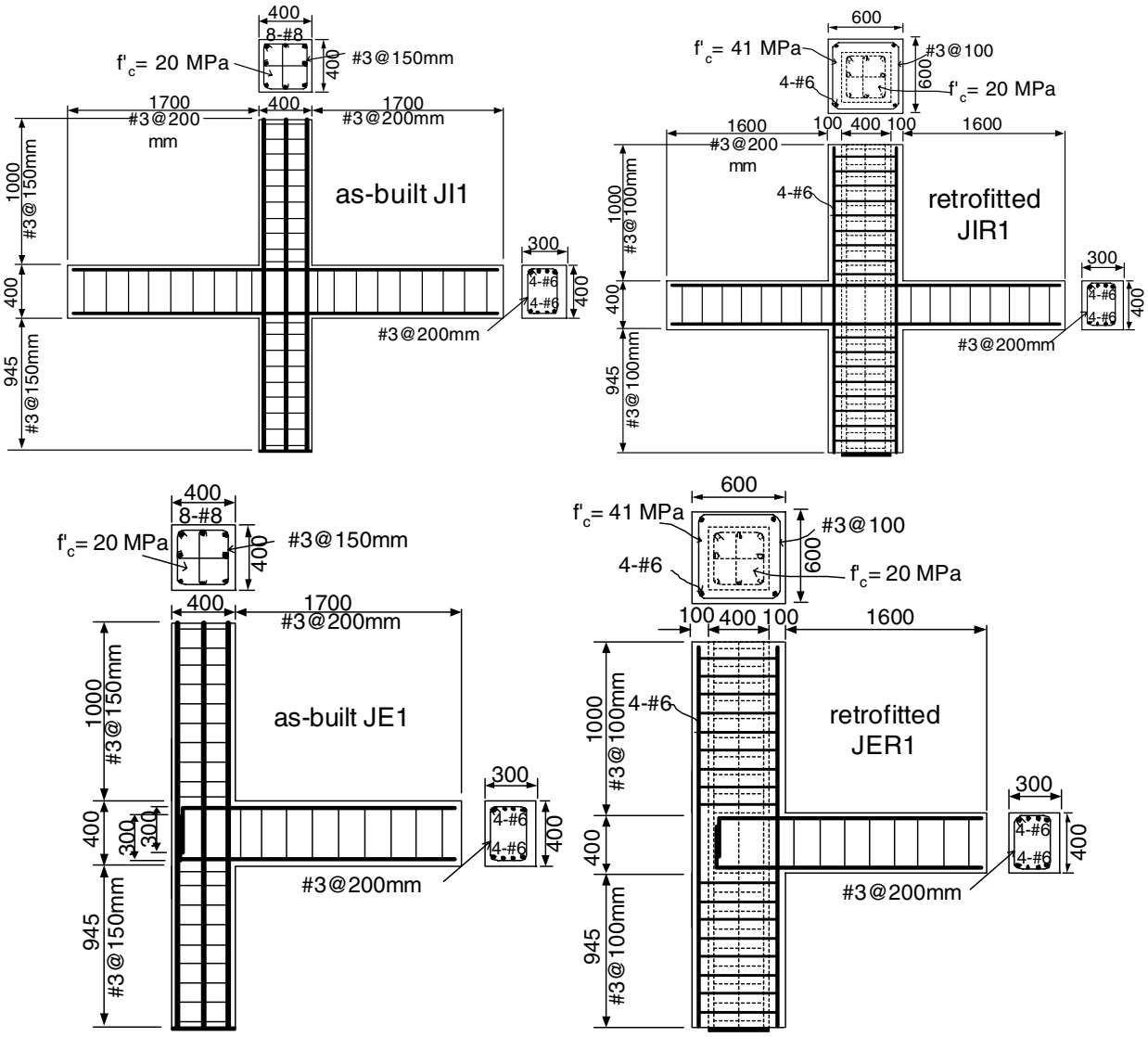

Fig.3 Reinforcing details of test specimens

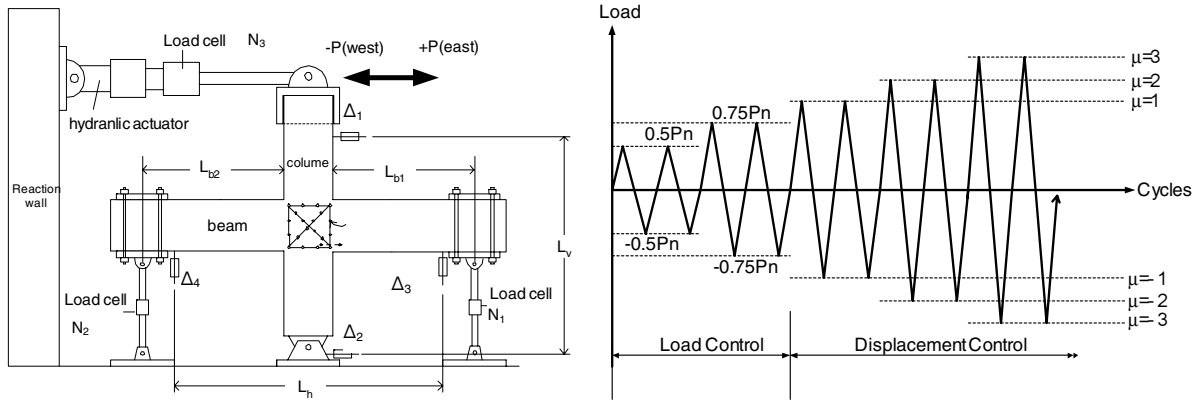

Fig.4 Test set-up and loading sequence

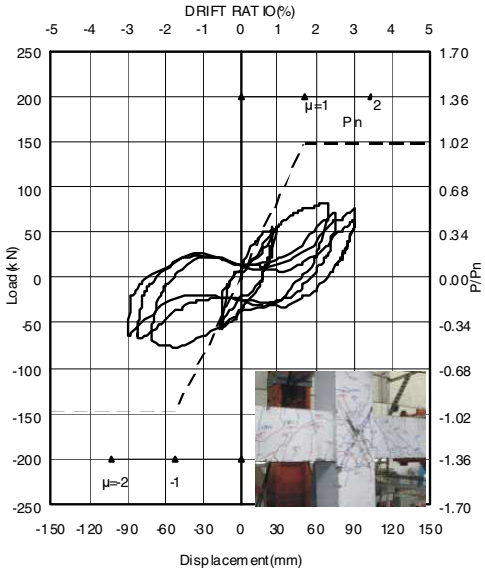

Fig.5 Test results of Unit JI1

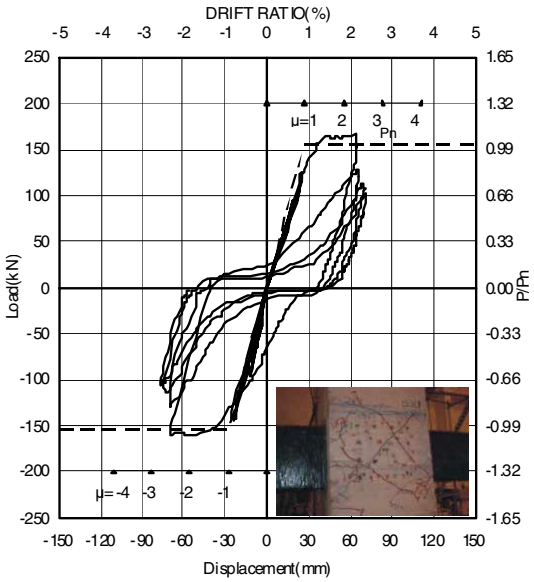

Fig.6 Test results of Unit JIR1

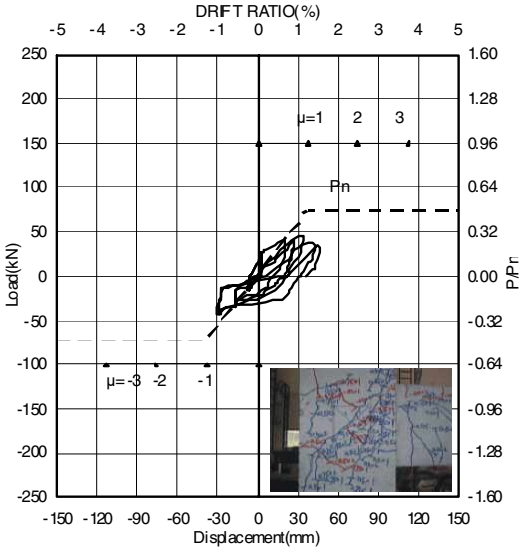

Fig.7 Test results of Unit JE1

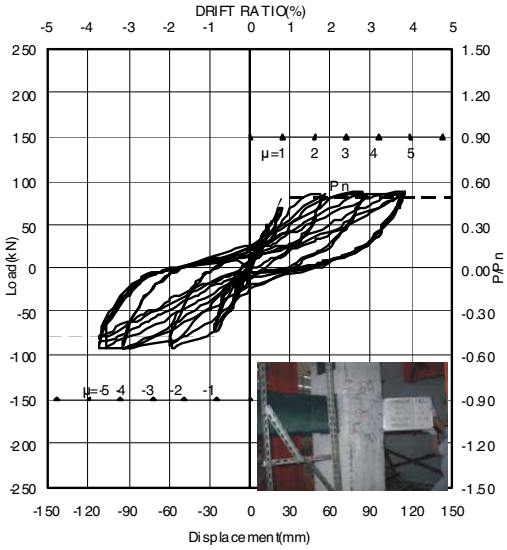

Fig.8 Test results of Unit JER1

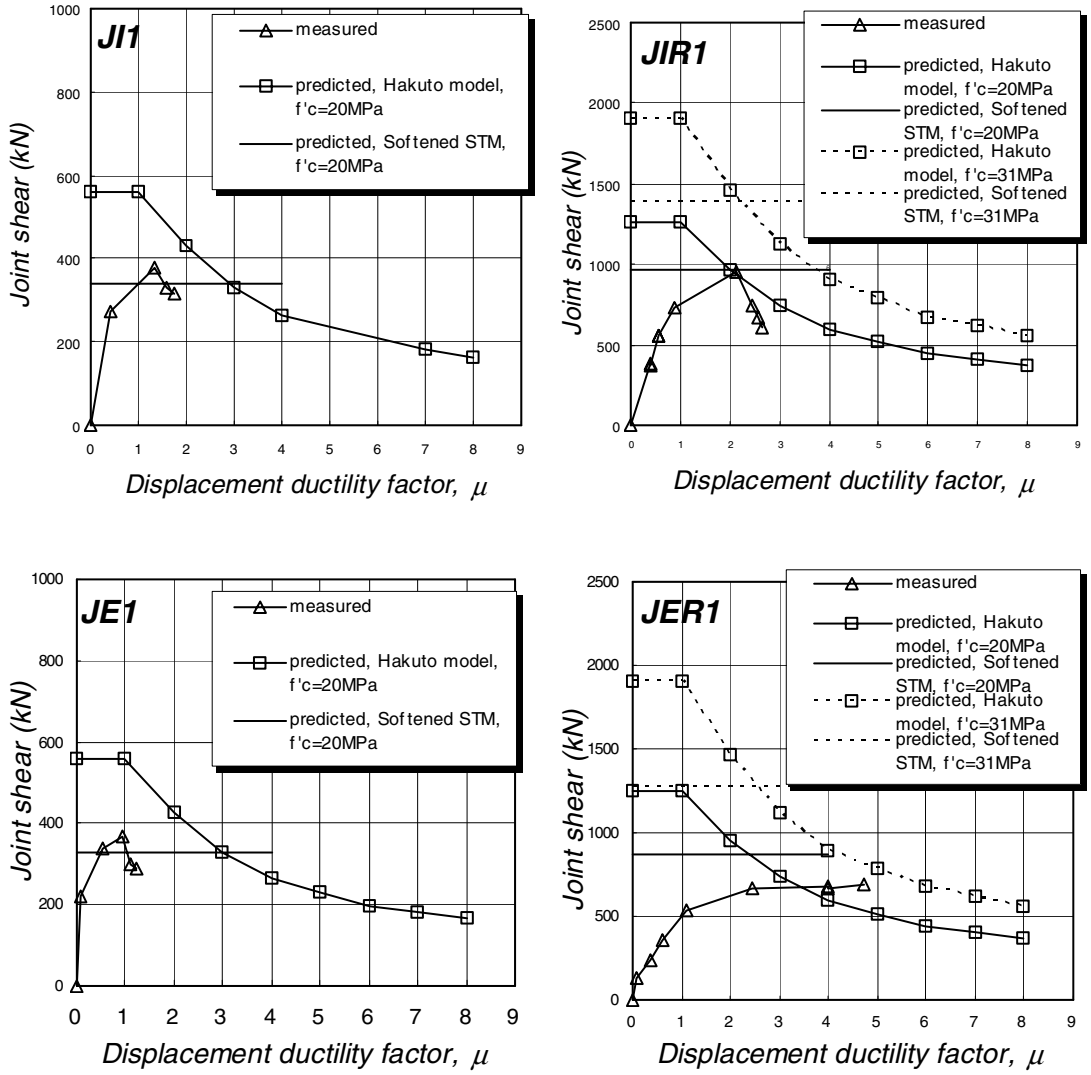

Fig.9 Measured and predicted joint shear for test units.