ENG 202-(EQC 1991/15)

Strengthening and/or Repair of Existing Reinforced Concrete Columns

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ENG
202

FINAL REPORT TO THE EARTHQUAKE AND WAR DAMAGE COMMISSION ON THE RESEARCH PROJECT 91/15

"STRENGTHENING AND/OR REPAIR OF EXISTING REINFORCED CONCRETE COLUMNS

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SUMMARY

A research project No. 91/15 entitled "Strengthening and/or Repair of Existing Reinforced Concrete Columns" funded by the Earthquake and War Damage Commission, was commenced at the University of Canterbury in 1991. EQC granted $32,000 for the project. This project is the second phase of the project "Development of Methods for the Strengthening and/or Repair of Existing Reinforced Concrete Columns" which was funded by EQC in 1989. This second stage of the project was supervised by Professor R Park and Dr H Tanaka. A Master of Engineering student, Mr Shigeru Hakuto from Japan, conducted the experimental work during 1991 and 1992.

The second phase of the project has involved a literature review of recent techniques for strengthening reinforced concrete frames including the beam-column joint region and seismic load tests conducted on two full scale beam-column joint units strengthening by jacketing.

RESULTS ACHIEVED

A literature review was conducted of the available procedures for assessing the seismic resistance of existing reinforced concrete frames and of retrofitting reinforced concrete beam-column joint regions of buildings, before and/or after damage caused by severe earthquakes, in order to establish methods to extend the life of existing reinforced concrete structures.

A typical reinforced concrete building which was designed in the late 1950s was assessed. The building exists in Christchurch. As with many building structures designed to early codes prior to about 1970, the reinforcing details are adequate for gravity and wind loads but some of the details are inadequate for earthquake forces. Earthquake design codes of that period did not specify capacity design nor detailing procedures which ensure adequate strength and ductility in the event of a major earthquake. With regard to the earthquake resistance of the building studied, the assessment showed that some of the columns have inadequate longitudinal reinforcement for flexural strength and inadequate transverse reinforcement for shear strength and ductility, and that the beams have inadequate transverse reinforcement for shear resistance. Also, the beam-column joints have no horizontal shear reinforcement and the diameter of the longitudinal beam reinforcement passing through the joints is such that significant loss of anchorage of reinforcement would occur in that region.

Two full-scale replicas of a beam-column joint region of the perimeter frame of the building have been constructed and subjected to simulated seismic loading in the Structures Laboratory of the University of Canterbury. The beams had cross sections of 500 mm x 300 mm and the columns had cross sections of 300 mm x 460 mm. The reinforcement in the members and joints was as in the as-built structure, and hence did not meet the requirements of the current New Zealand concrete design code NZS 3101:1982 in many ways.

One of the beam-column joint replicas was tested as-built subjected to simulated seismic loading. The test confirmed that the performance of the beam-column joint region would be poor in a major earthquake, mainly due to the lack of shear reinforcement and poor anchorage of longitudinal beam bars in the beam-column joint core. The damaged (tested) beam-column replica and the other undamaged (not tested) beam-column joint replica were then retrofitted by jacketing with new reinforced concrete, to increase the strength and ductility of the existing frame. Both retrofitted beam-column joint replicas were then tested subjected to simulated seismic loading and performed in a very satisfactory manner.

PUBLICATIONS ARISING FROM THE PROJECT

Copies of reports on the progress of this second phase of the project are submitted with this report.

The Master of Engineering student, Mr S Hakuto, has now transferred his enrolment to that for a Doctor of Philosophy degree and is commencing the third phase of the project funded by EQC, namely Project No. 93/102 "Retrofitting of Existing Reinforced Concrete Building Frames". Technical papers based on Projects 91/15 and 93/102 will be published as the work progresses.

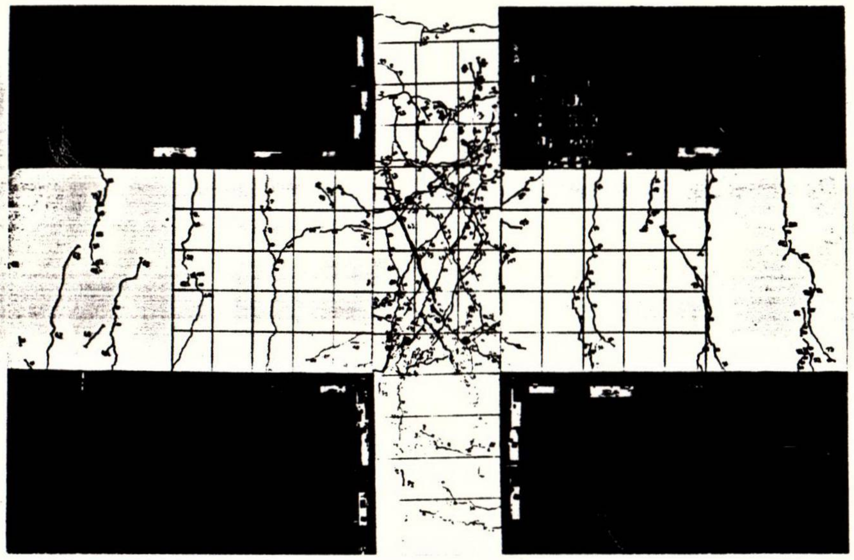
CONCLUSIONS

The project has resulted in a review of the assessment and retrofitting procedures available for existing reinforced concrete building frames. Also, test results have been obtained which indicate that the concrete jacketing technique can be used for retrofitting beam-column joint regions. This retrofitting method may be used for extending the life of existing reinforced concrete structures by strengthening techniques and the repair of damage arising from major earthquakes.

ACKNOWLEDGEMENTS

The financial supporters of the Earthquake and War Damage Commission is gratefully acknowledged.

April 1993



Photograph

As-built 1950s reinforced concrete beam-column joint unit after being subjected to simulated seismic loading illustrating shear and bond failure of the joint core.

2 June '92

Assessment of a building designed in the late 1950's

A reinforced concrete moment resisting frame designed in the late 1950's is assessed with current New Zealand design codes(NZS 4203:1984 and NZS 3101:1982). The selected structure is that of a seven storey office building, with 5 spans in x-direction and 3 spans in y-direction. An elevation in x-direction is shown in Fig.1. The typical floor plan with shear walls is shown in Fig.2 and frame A and B line in x-direction are assessed. In this assessment, however, the shear walls are omitted and the structural plan is somewhat simplified. Foundation consists of foundation beams and reinforced concrete piles.

[1] Material Strength

Originally specified material strength is assumed as follows:

specified concrete strength f'_c=20MPa
specified yield strength of steel f_y=275MPa(Grade 275)

Note that all reinforcement other than stirrups, ties and hoops are assumed to be deformed bars.

[2] Ductile Detailing

In NZS3101, it is specified that in the plastic hinge regions, the maximum permitted centre to centre spacing of the transverse reinforcement be less than

(for column)

min(d/5, 6d_b, 200mm)

(for beam)

min(d/4, 6d_b, 150mm)

where d is the effective depth for beam or
the least lateral dimension of column section
d_b is the longitudinal bar diameter.

If these ductile detailing requirements are met, the displacement ductility is assessed to be 6. Otherwise, the ductility is assumed to be the value between 1 and 4.

Fig.3 shows typical sections of columns and beams. Spacing of transverse reinforcement commonly used are 305mm or 230mm for columns and 381mm for beams respectively. These values are approximately 3~4 times greater than those required by the

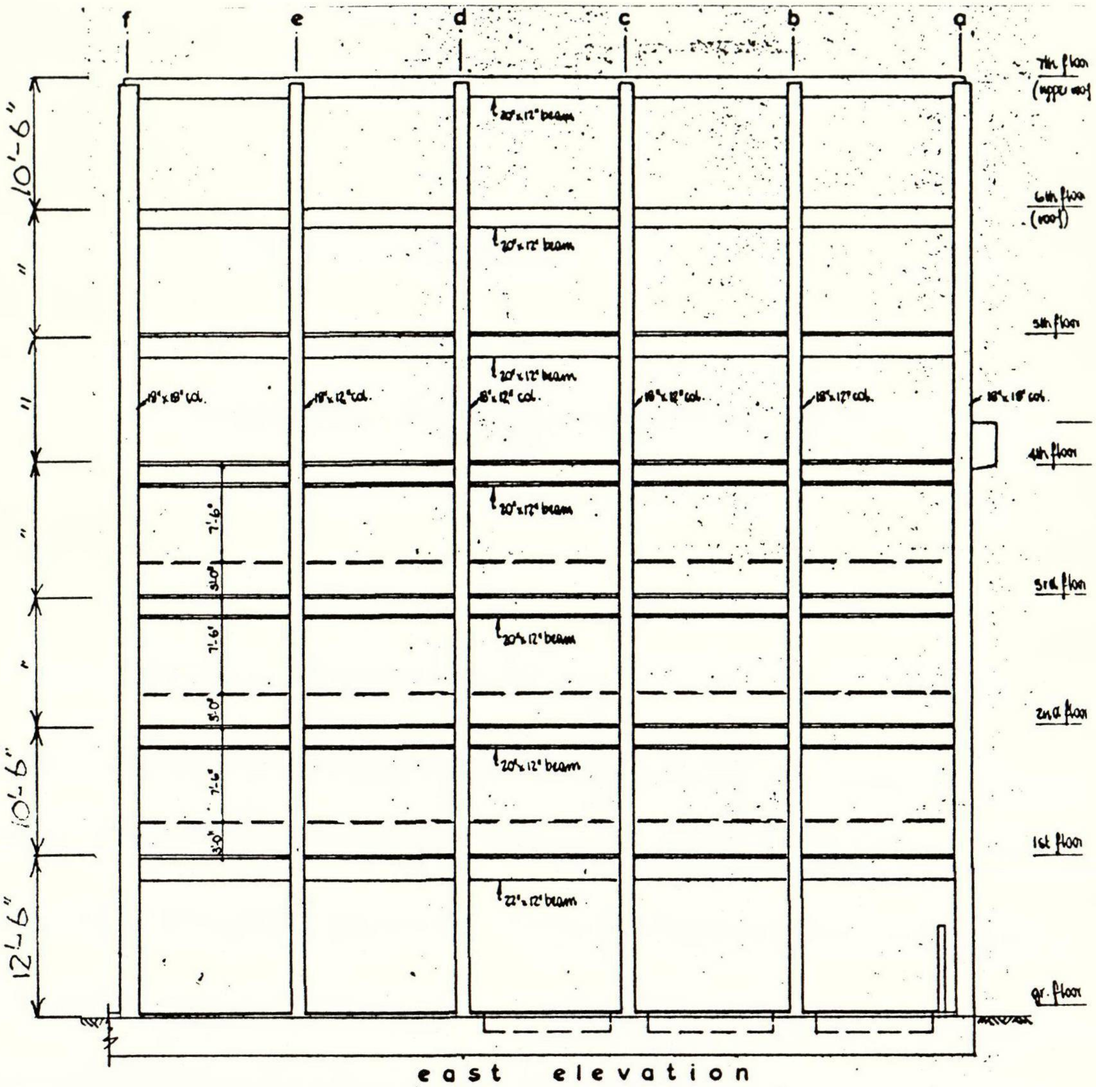


Fig. 1 Elevation (X-direction)

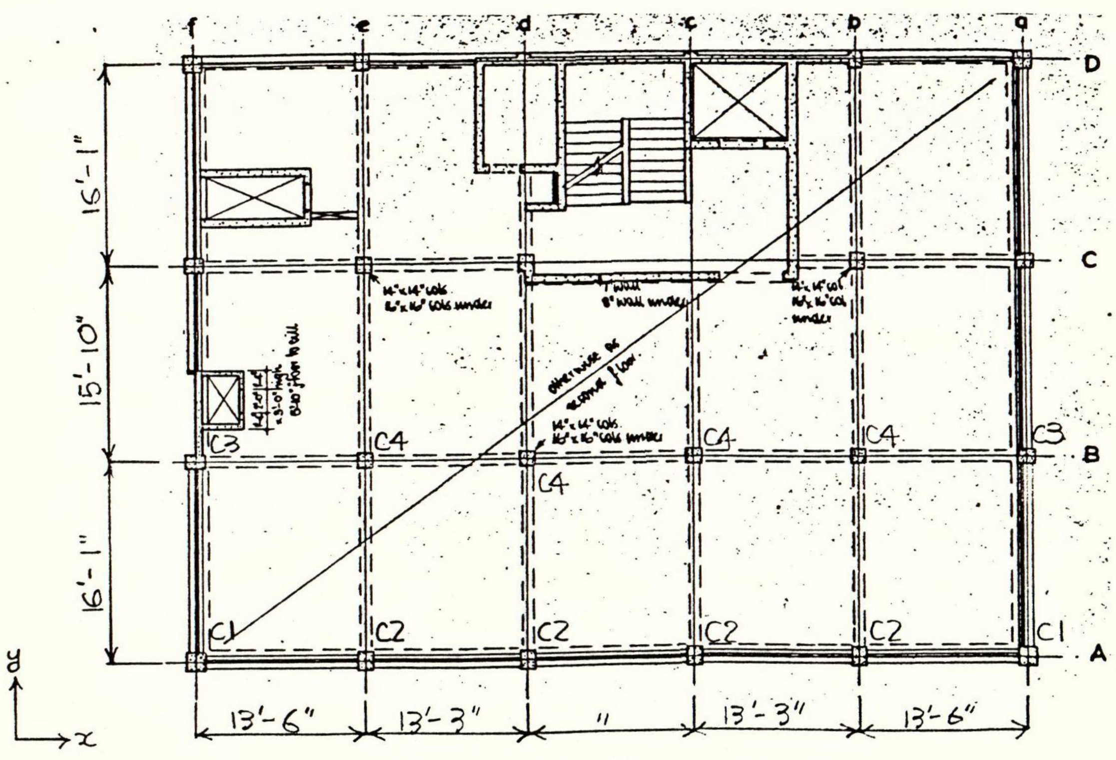


Fig. 2 Typical Floor Plan

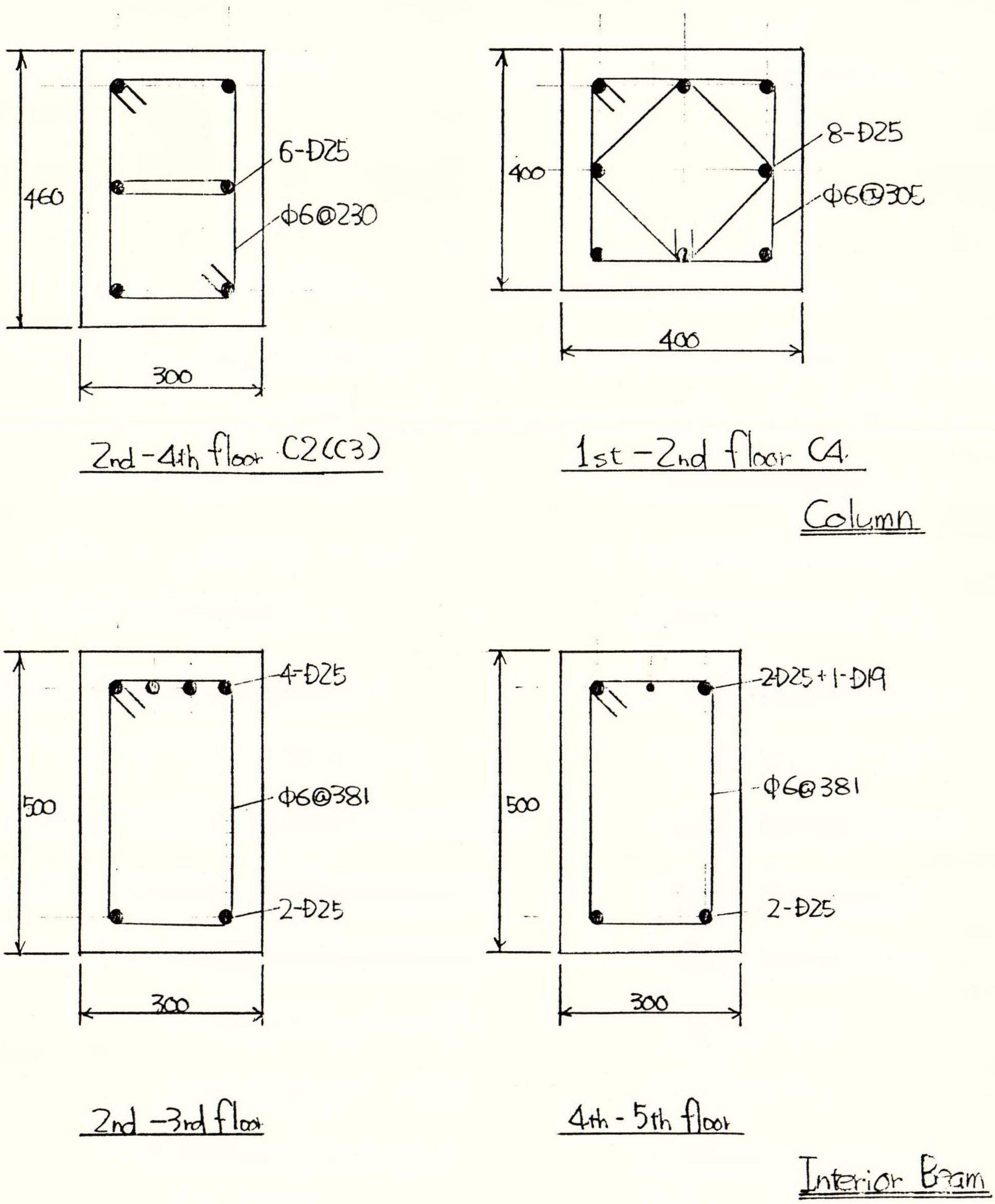


Fig. 3 Typical Sections of Columns and Beams

current code requirements. Hence the displacement ductility of each member may be assessed to be up to 2 although in potential hinge regions of the exterior beams of lower storeys, much closer spacing is provided.

[3] Calculation of Member Strengths

Concrete compressive strength of $1.5f'_c$, which considers strength increase with age, is used for calculation of member strengths.

Flexural Strength

The flexural strength is calculated according to the ACI method using a maximum compressive strain in concrete of 0.003. In this calculation, it is assumed that all main bars are stressed to $1.15f_y$. The contribution of floor slab to enhancement of the flexural strength of beams, which depends on the displacement ductility level and slab bar anchorage, is considered as the lesser of one quarter of the span of the beam and one half of the span of the slab, transverse to the beam under consideration. The amount of slab bars considered for this assessment is 700mm^2 for interior beams and 350mm^2 for exterior beams.

Shear Strength

The shear strength is calculated by NZS3101.

non-ductile shear strength : $V_{nd}=(v_c b_w d+\dfrac{A_v f_{yh} d}{s})$

where $v_c=v_b=(0.07+10\rho_w)\sqrt{f'_c}$: for beams

$v_c=v_b\left[1+\dfrac{3Pu}{Agf'_c}\right]$: for columns

ρ_w : longitudinal tension steel ratio

s : spacing of transverse reinforcement

A_v : area of transverse reinforcement

f_{yh} : specified yield strength of transverse reinforcement

b_w : web width

ductile shear strength : $V_d=\dfrac{A_v f_{yh} d}{s}$

for columns when the average compression stress exceeds $0.1\ f'_c$ the shear stress carried by the concrete v_c is taken as follows:

$$v_c=4v_b\sqrt{\frac{P_e}{f'_c A_g}-0.1}$$

where P_e : minimum design compressive force

These shear strengths, which are dependent on displacement ductility, will be used to identify the failure mode of each member.

[4] Classification of Failure Modes and Collapse Mechanism

Assuming the development of the flexural strengths calculated above at critical sections of beams and columns, the shear force(V_f) corresponding to the flexural strengths are calculated. By comparing V_f with the non-ductile and ductile shear strengths, the failure modes of the members are classified into three categories, as follows:

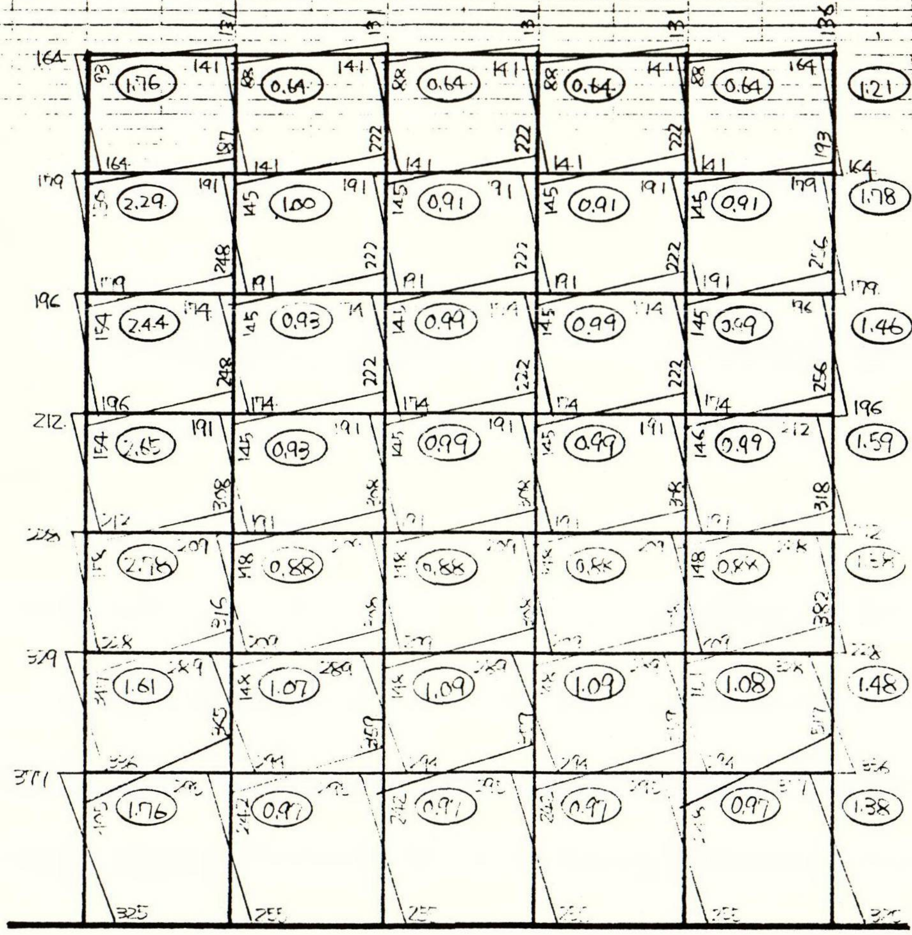
$V_f > V_{nd}$:	Brittle shear failure	$\mu=1$
$V_{nd} > V_f > V_d$:	Limited ductile with shear failure	$2<\mu<4$
$V_f < V_d$:	Ductile flexural	$\mu>4$

The ratios of the shear strengths by NZS3101 to the shear forces corresponding to the flexural strengths are

frame A line(Fig.2)	beams :	V_{nd}/V_f=1.05~2.16,	V_d/V_f=0.08~0.52
	columns :	V_{nd}/V_f=0.89~1.60,	V_d/V_f=0.13~0.59
frame B line(Fig.2)	beams :	V_{nd}/V_f=0.89~2.09,	V_d/V_f=0.12~0.69
	columns :	V_{nd}/V_f=0.69~1.55,	V_d/V_f=0.14~0.87

No members are assessed to be ductile flexural as failure mode. Most columns and beams are categorized into limited ductile with shear failure. However, the beams and columns in lower storeys are classified to be brittle shear failure, especially for frame B line(Fig.2). It should be noted that these category can be applied only when the flexural strengths are developed at both ends of members.

To obtain the moment and shear distribution at the formation of the complete mechanism, a simplified limit analysis is conducted. The flexural strength of the member governed by the shear failure mode($V_f > V_{nd}$) is reduced proportionally to match the shear strength. By extrapolating moment diagram, associated nodal moments are calculated. The sum of the column moments and the beam moments so obtained are compared to determine the moment and shear distribution at the formation of the mechanism. When the sum of the beam moments is smaller, half of this sum is substituted for the upper and lower column moments. In Fig.4, the ratios of flexural strengths of columns to those of beams so obtained are shown. These strength ratios except top storey are derived as follows:



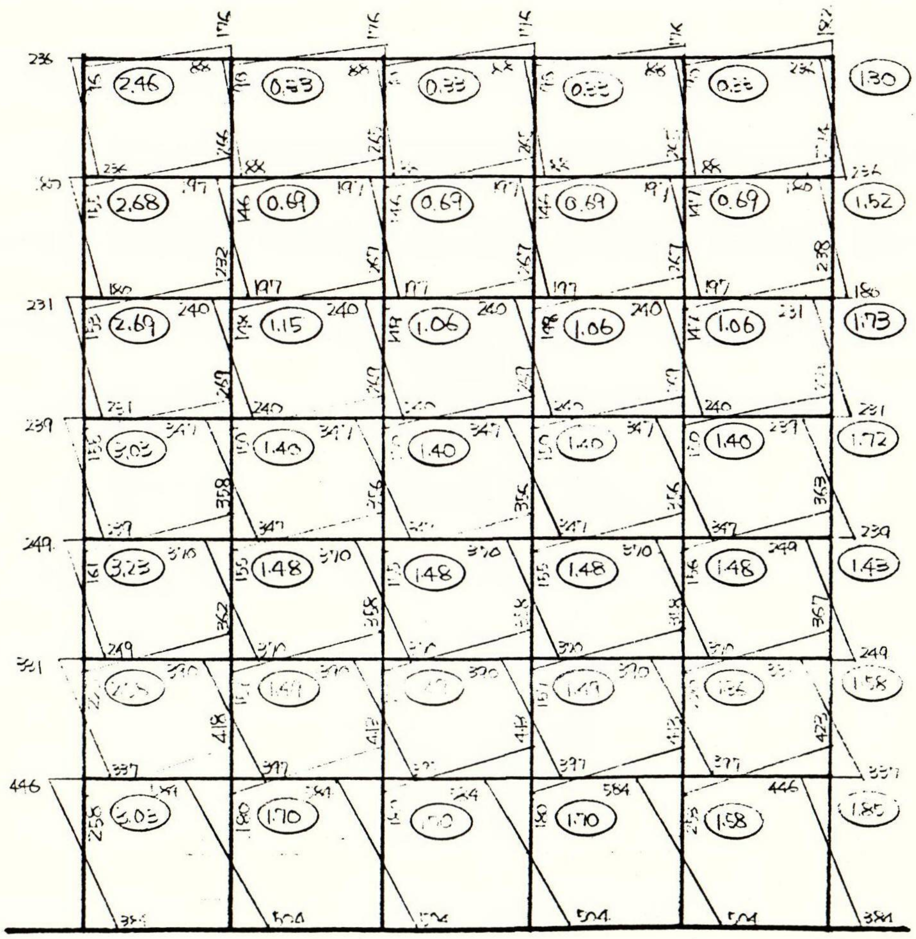


Fig.4 The Ratios of flexural Strengths of columns to those of beams.

For Frame A line : 0.88~2.78
For Frame B line : 0.69~3.23

Fig.5 shows the moment and shear distribution at the collapse mechanism which is shown in Fig.6. The collapse mechanism is composed of flexural hinges of beams and columns, and of shear failure of beams. In this assessment, column shear failure which could result in total collapse, is not identified. As mentioned above, however, shear strength of members degrades as displacement ductility increase. Hence the flexural mode may shift to brittle shear failure mode at higher ductilities more than 2. It may be reasonable to assess the displacement ductility of flexural hinges up to 2 when considering the poor ductile detailing.

To investigate whether column sidesway mechanism can be expected, a sway-potential index S_p by Priestley et al, is calculated. The index S_p is obtained by the following equation.

$$S_p=\frac{\sum_{i=1}^{j}(\Sigma M_{Bn,i})+\sum_{i=1}^{j}(\Sigma M_{Bn+1,i})}{\sum_{i=1}^{j}(\Sigma M_{Cn,i})+\sum_{i=1}^{j}(\Sigma M_{Cn+1,i})}$$

where $\Sigma M_{Bn,i}$: sum of beam moment capacities(left + right) at the joint centroid of joint i, level n

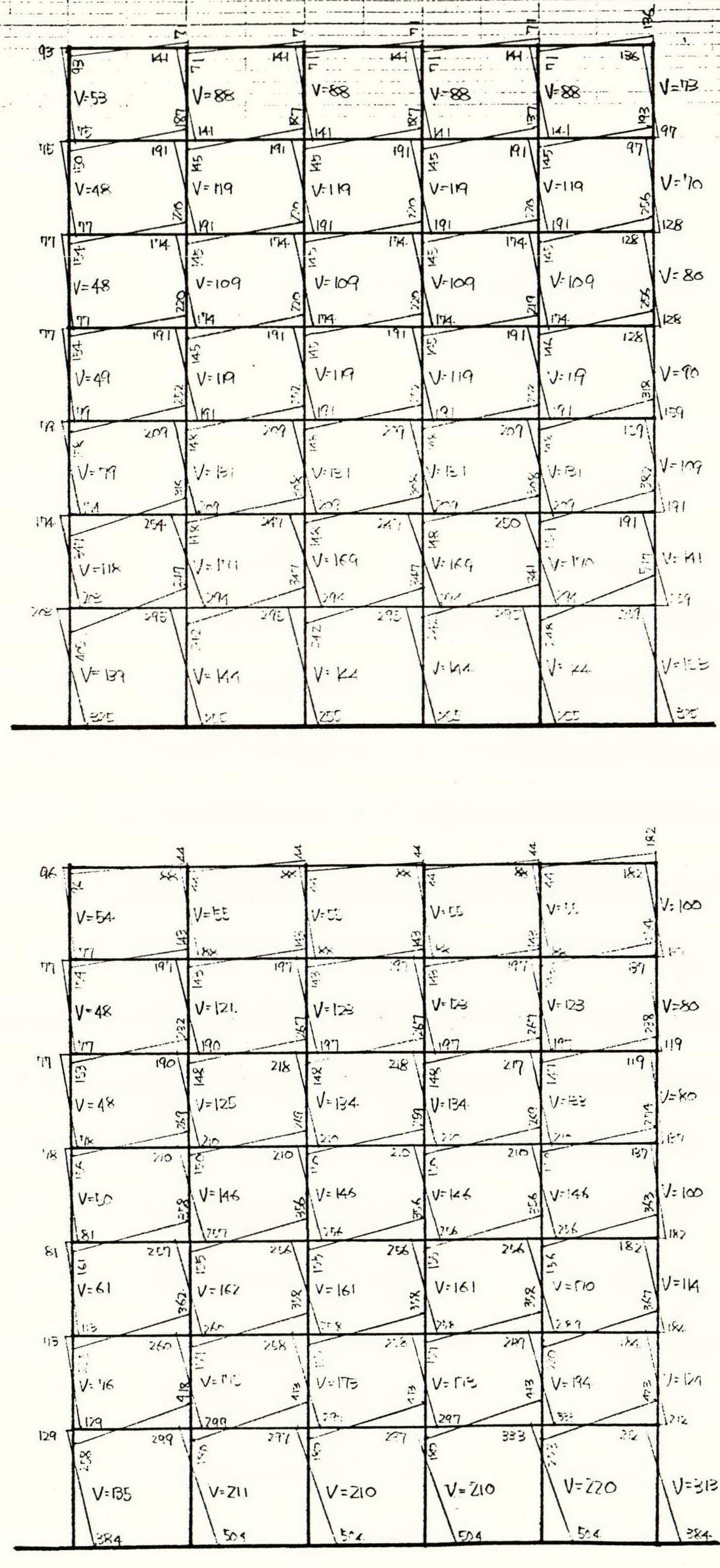
$\Sigma M_{Cn,i}$: sum of column moment capacities(upper + lower) at the joint centroid of joint i, level n

It was suggested that column sidesway mechanism be expected when $S_p>0.85$. The value of S_p so calculated is 0.72~0.85 for frame A line and 0.56~0.87 for frame B line respectively. According to the criteria($S_p>0.85$), column sidesway mechanism is expected for the 6th storey of frame B line only.

Base shear capacity at the collapse mechanism shown in Fig.5 is calculated.

Base shear capacity=4310KN=0.26W

where W : total weight of the building assuming average weight of 8KPa



A-Frame

B-Frame

Fig.5 Moment and Shear distribution at the collapse mechanism

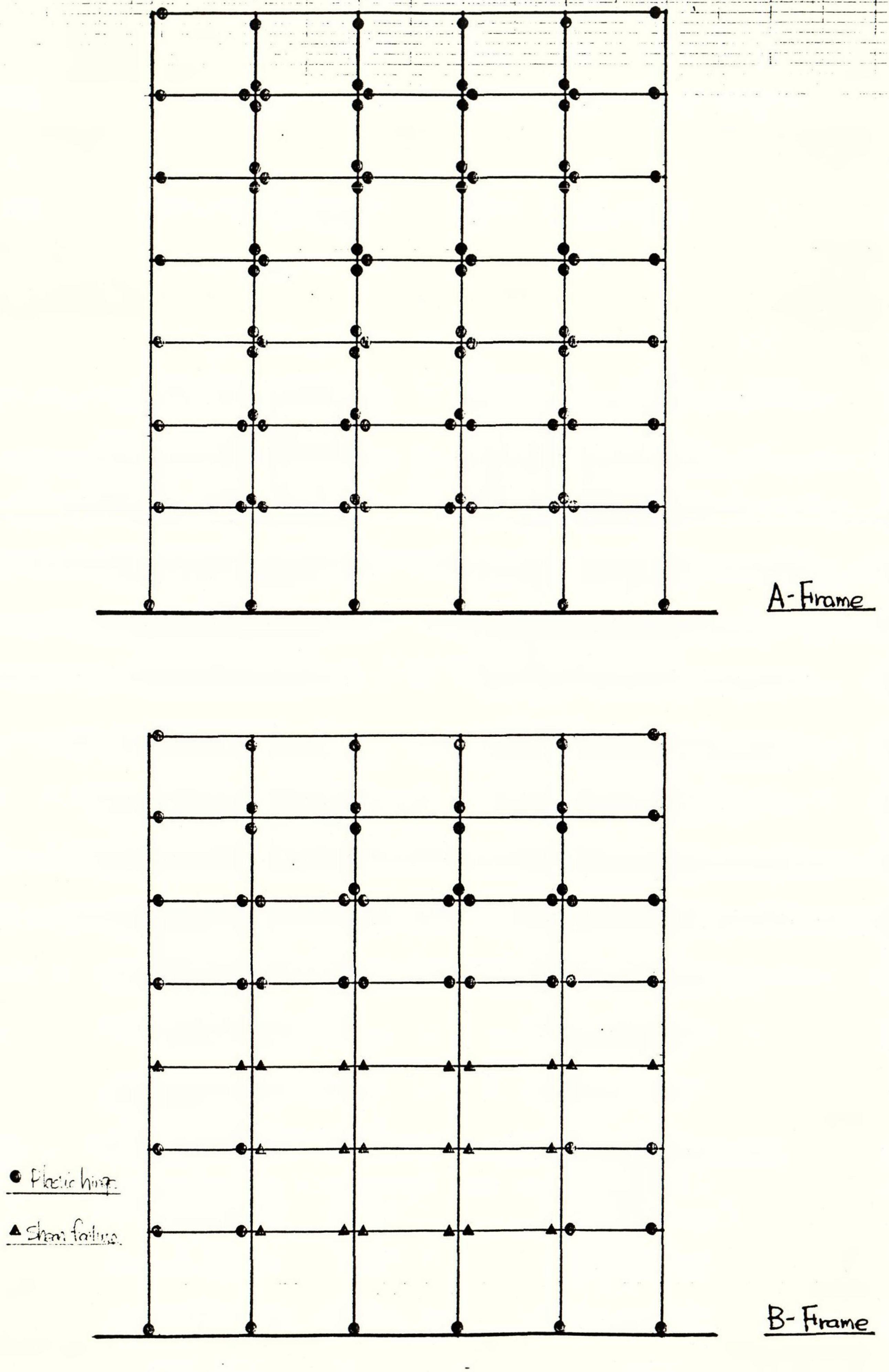


Fig.6 Collapse Mechanism

According to NZS4203, every building shall be designed and constructed to withstand a total horizontal seismic force(V) in each direction under consideration in accordance with the following formula.

$$V = C_d W$$

where $C_d = CRSM$

R : Risk Factor=1.0

M : Structural Material Factor=0.8

C : Basic Seismic Coefficient

Christchurch Zone B(intermediate subsoils)

Assuming Period=1.0(sec)

From Fig.3 in NZS3101, C=0.08

S : Structural Type Factor

For elastically responding structure(S=5), C_d=0.32>0.26 (N.G.)

Although the level of ductility of the structure, that is an appropriate value for S, is not certain, the required value for S could be obtained from the following equation.

$$S = \frac{0.26}{CRM} = 4.1$$

The required level of ductility, μ for the structure to meet the horizontal seismic force by current code is expressed by the following equation.

$$\mu = \frac{4}{SM} = 1.22$$

At this stage, the collapse mechanism and base shear capacity assuming the strength of the beam-column joints to be infinite is derived.

[6] Beam-Column Joints

A typical feature of the beam-column joint of the building compared with current code is no transverse reinforcement and the relatively small size of the columns and hence excessive slip of bars through the joint core and joint shear failure may be expected.

Anchorage of Longitudinal Reinforcement

To keep bond stresses to an acceptable level, the diameters of longitudinal bars d_b passing through a joint core are limited by NZS3101 as follows:

For beams : $\frac{h_c}{d_b} \geq \frac{f_y}{12}$

For columns : $\frac{h_b}{d_b} \geq \frac{f_y}{15}$

where f_y : specified yield strength of logitudinal bar

d_b : diameter of longitudinal bars

h_c : column overall depth

h_b : beam overall depth

For interior column(C2, C4 in Fig.2) of the building, the ratio of the column/beam width to the beam/column bar diameter is

For beams : $\frac{f_y}{23} \leq \frac{h_c}{d_b} \leq \frac{f_y}{17}$

For columns : $\frac{f_y}{15} \leq \frac{h_c}{d_b} \leq \frac{f_y}{14}$

Hence slip of beam bars could be expected.

To investigate the possibility of bond degradation, an index, called "beam bar bond index" by Kitayama et al, is used. The average bond stress u_b over the column width for simultaneous yielding of the beam reinforcement in tension and compression at the two faces of the joint is expressed as follows:

$$u_b = f_y(d_b/h_c)/2$$

If the bond strength is assumed to vary with the square root of the concrete compressive strength f'_c, the feasibility of bond degradation may be expressed by a bond index, BI, defined as

$$BI = u_b/\sqrt{f'_c}$$

The bond deterioration is more likely to occur for a higher index value. For interior beam-column joint(C2, C4 in Fig.2) of the building, the bond indices are obtained as follows:

$$1.6 < BI < 2.1$$

where f'_c=30MPa

The above equation to limit the ratio of the column width to the beam bar diameter by NZS3101 restricts the index value to be smaller than 1.1. However, the bond indices values of the interior beam-column joint show 45%~91% higher values than that limited by NZS3101. Hence at higher ductilities, bond deterioration is expected for the beam bars.

Shear Strength

The horizontal joint shear force is calculated as follows:

$$V_{jh}=\frac{M_{BL}+M_{BR}}{j_d} - V_c$$

where M_{BL}, M_{BR} : positive and negative beam moment at the formation of collapse mechanism respectively(Fig.5)

V_c : column shear force at the formation of collapse mechanism

j_d : internal lever arm between resultant forces

The corresponding joint shear stress is

$$v_{jh}=\frac{V_{jh}}{b_j h_c}$$

where b_j : effective width of joint

The joint shear stresses of interior beam-column joints so calculated are 5.8~8.7MPa ($1.1\sqrt{f'_c}$~$1.6\sqrt{f'_c}$), which are 71%~105% of the maximum shear stress permitted by the current code(i.e. $1.5\sqrt{f'_c}$).

For a beam sidesway mechanism with a ductility capacity less than 2, the approach assessing effective shear stress and strength was proposed by Priestley. According to this assessment, the effective shear force is expressed by the following equation.

$$V'_{jh}=\frac{M_{BR}}{j_d} - V_c$$

This equation assumes that compression force of the beam and column moment can be transmitted to the joint by a diagonal compression strut and the joint is assessed under reduced shear force. In conjunction with any effective axial stress on the joint, f_a, the principal tension stress, f_t, can be derived as follows:

$$f_t=\frac{f_a}{2}-\sqrt{\left[\frac{f_a}{2}\right]^2+v'^2_{jh}}$$

where f_t is negative for tension and f_a is positive for axial compression

v'_{jh} : effective joint shear stress

To determine whether joint shear cracking will develop, the principal tension stress is compared with the stress $0.3\sqrt{f'_c}$. The effective joint shear stresses so obtained are between $0.46\sqrt{f'_c}$~$0.70\sqrt{f'_c}$ and hence joint shear cracking can be expected.

Results of the Assessment

By using the current New Zealand design codes, the building designed in the late 1950's was assessed as follows:

[1] Displacement ductility of each member is expected to be up to 2.

[2] The collapse mechanism is composed of flexural hinges of beams and columns and of shear failure of beams. According to a sway-potential index, column sidesway mechanism is not expected except the 6th storey of frame B line.

[3] To meet the base shear force by current code the required ductility level of the structure is 1.22

[4] The beam-column joint was assessed to be critical region since bond deterioration of the beam bars and deterioration of joint shear strength due to the shear crack are expected. However, the ductility level when bond of beam bars and shear strength of beam-column joints deteriorate is still uncertain

20 November '92

Hakuto. S

Retrofitting of Reinforced Concrete Moment Resisting Frames

Two full scale replicas, i.e., original specimens, of the beam-column regions of a reinforced concrete moment resisting frame designed in the late 1950's were constructed. One original specimen (O1) was tested subjected to simulated seismic loading to investigate its behaviour. After testing, the specimen O1 was retrofitted by using concrete jacketing of both beams and columns(specimen R1) and retested. The other original specimen was retrofitted using the same method except providing no joint horizontal hoops(specimen R2) and tested to permit a comparison of the effect of the joint hoops on the seismic performance.

This report presents the test results briefly. Theoretical strengths based on the measured material properties as well as the measured stiffness and maximum strength of each specimen are described first. Secondly, crack pattern, hysteresis loops and strain distributions of reinforcement for each test unit are shown to provide information about their behaviour during the test. Finally, based on the test data, two limiting conditions are identified for the joint behaviour without horizontal shear reinforcement.

Test Results

Material Properties

Table 1 Measured Concrete Properties

		slump (mm)	at 28days f'c(MPa)	before testing f'c(MPa)	after testing f'c(MPa)	after testing f't(MPa)	after testing f'r(MPa)
Specimen O1	original unit	55	34	41 (107days)	45 (114days)	4.2 (114days)	5.2 (114days)
Specimen R1	original unit	55	34	42 (175days)	43 (189dyas)	3.9 (189days)	
	jacketing unit	150	40	54 (42days)	59 (56days)	3.8 (56days)	
Specimen R2	original unit	125	35	43 (182days)	42 (186days)	4.1 (186days)	6.1 (186days)
	jacketing unit	180	48	61 (38days)	60 (42days)	4.5 (42days)	4.0 (42days)

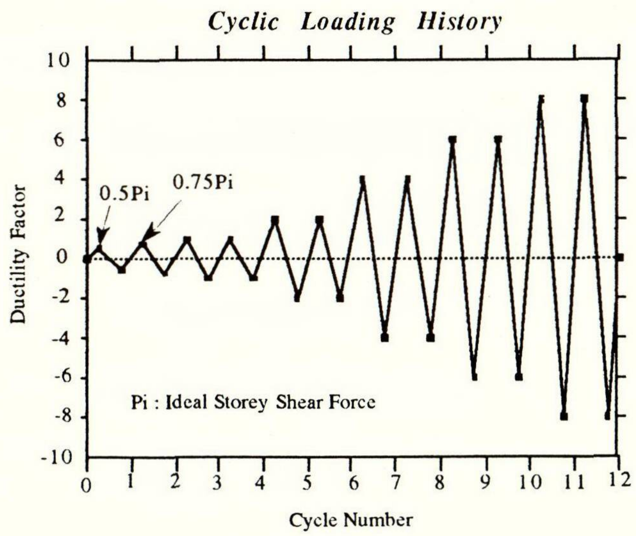
Notes : f'c=compressive strength of 100mm dia. x 200mm concrete cylinder
f't=split cylinder tensile strength
f'r=modulus of rapture of 120 x 120 x 480mm concrete beam under two-point loading

Table 2 Measured Reinforcing Steel Properties

Grade of Steel	Grade 300				Grade 430	
Bar Size	R6	D10	D12	D24	HR16	HD24
Yield Strength, fy(MPa)	338	332	302	325	436	461
Ultimate Strength, fu(MPa)	463	448	422	481	599	613
Elongation(%)	20	27	30	28	20	19

Note: R6 = plain round bar of 6mm diameter
D10 = deformed bar of 10mm diameter
HR16 = plain round high strength bar of 16mm diameter
HD24 = deformed high strength bar of 24mm diameter bar

Cyclic Loading History



Theoretical Ideal Flexural Strengths

For Original Specimen(O1)

$M_{i,column}$=120KNm(f'_c=41MPa)

$M_{i,beam}(+)$=132KNm(f'_c=41MPa)

$M_{i,beam}(-)$=246KNm(f'_c=41MPa)

The strength ratio of total ideal strength of columns to that of beams is

$\sum M_{i,column}/\sum M_{i,beam}=0.69$

For Retrofit Specimen(R1)

$M_{i,column}$=592KNm(f'_c=54MPa)

$M_{i,beam}(+)$=180KNm(f'_c=48MPa)

$M_{i,beam}(-)$=387KNm(f'_c=54MPa)

$\sum M_{i,column}/\sum M_{i,beam}=2.10$

For Retrofit Specimen(R2)

$M_{i,column}$=598KNm(f'_c=61MPa)

$M_{i,beam}(+)$=182KNm(f'_c=52MPa)

$M_{i,beam}(-)$=389KNm(f'_c=61MPa)

$\sum M_{i,column}/\sum M_{i,beam}=2.11$

Ideal Storey Shear Strength

The ideal storey shear strength when the column/beam hinge mechanism is developed is

${}_{O1}P_i$=89KN (column hinge mechanism)

${}_{R1}P_i$=217KN (beam hinge mechanism)

${}_{R2}P_i$=218KN (beam hinge mechanism)

Stiffness

Measured stiffness(0.75P_i)

${}_{O1}K$=2.38KN/mm

${}_{R1}K$=16.7KN/mm (${}_{R1}K/{}_{O1}K$=7.0)

${}_{R2}K$=20.3KN/mm (${}_{R2}K/{}_{R1}K$=1.2)

Measured yield displacement

${}_{O1}\Delta y$=37.4mm (storey drift angle, R=1.20%)

${}_{R1}\Delta y$=13.0mm (R=0.41%)

${}_{R2}\Delta y$=10.7mm (R=0.34%)

Strengths

Measured maximum strength

$_{O1}P_{max}$=89KN (=1.0$_{O1}P_i$)

$_{R1}P_{max}$=231KN (=1.1$_{R1}P_i$)

$_{R2}P_{max}$=223KN (=1.0$_{R2}P_i$)

Failure Mode

For original specimen(O1)

During the loading cycle, 0.5Pi, bond splitting cracks developed along the beam bars in the joint(see Fig.4.1.3(a)). As shown in Figs.4.1.4 and 4.1.5, premature bond deterioration of both beam and column bars was clear, resulting in a significant softening in the hysteresis curves(see Fig.4.1.2).

Joint diagonal tension cracks initiated during the loading cycle, 0.75Pi(see Fig.4.1.3(b)).

After developing column hinge mechanism(see Figs.4.1.4 and 4.1.5), one dominant diagonal tension crack opened wide(see Fig.4.1.1). Hysteresis loops indicated the strength and stiffness degradation due to the joint distress(seeFig.4.1.2).

For Retrofit specimen(R1)

Beam hinge mechanism was developed(see Figs.4.2.1 and 4.2.4).

During the second cycle of loading to displacement ductility factor, DF, 8, beam bottom bars(2-D12) buckled and fractured.

Joint diagonal tension cracks initiated during the loading cycle, 0.75Pi(see Fig.4.2.3(b)). However, those cracks did not open wide.(maximum crack width was 0.5mm)

The maximum tensile strain of new joint horizontal hoop(6-HR16) was 929^{μ}, which corresponds to the steel bar stress of 186MPa.(see Fig.4.2.5)

For Retrofit specimen(R2)

Beam hinge mechanism was developed(see Figs.4.3.1 and 4.3.4).

Joint diagonal tension cracks initiated during the first cycle of loading to DF of 4, (see Fig.4.2.3(d)) and opened wide up to the maximum crack width of 3.6mm

As shown in Fig.4.4, "strong beam-weak column" response of the Original specimen(O1) could be shifted to "strong column-weak beam" behaviour of the Retrofit specimens(R1, R2).

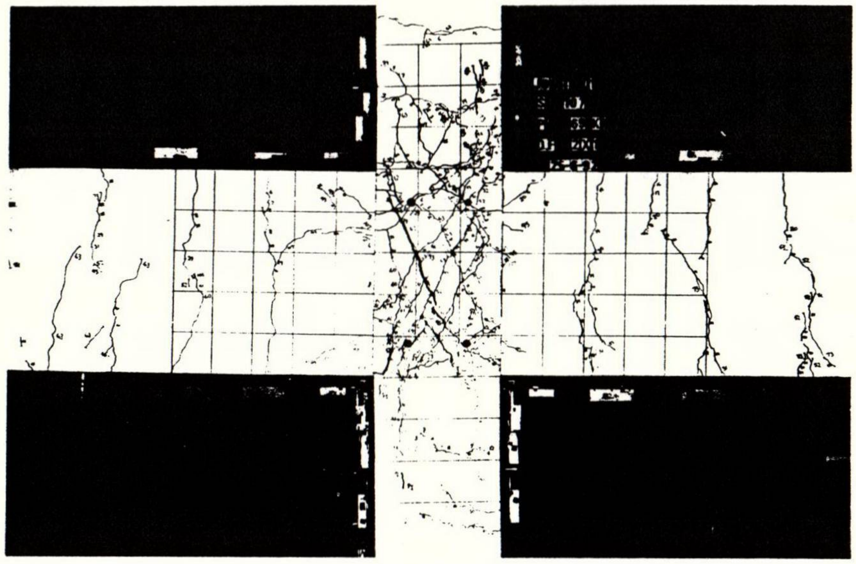


Fig.4.1.1 Observed Cracking of Original Specimen(O1)
at first cycle of DF=2

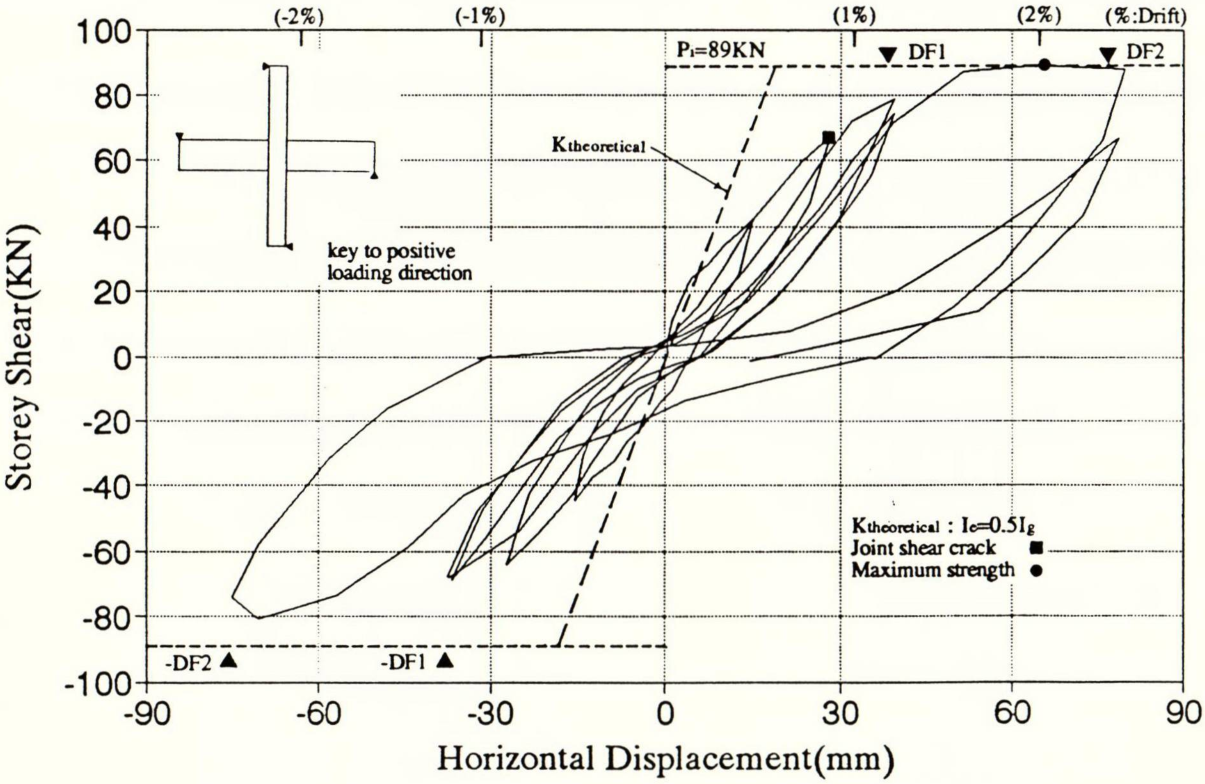
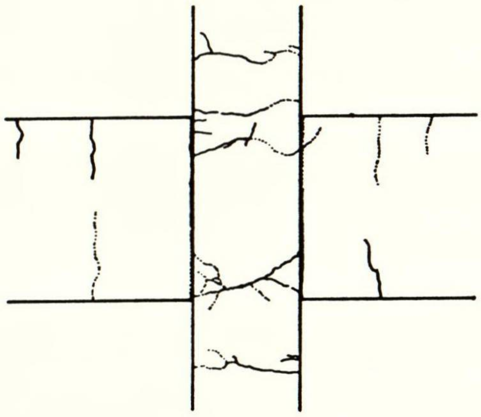
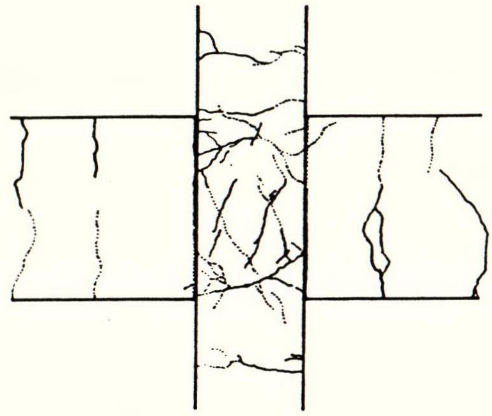


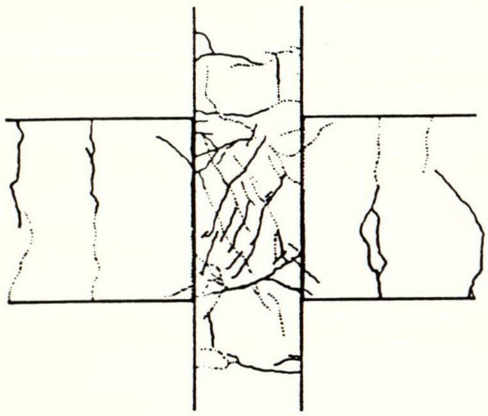
Fig.4.1.2 Storey Shear versus Horizontal Displacement Relationship
for Original Specimen (O1)



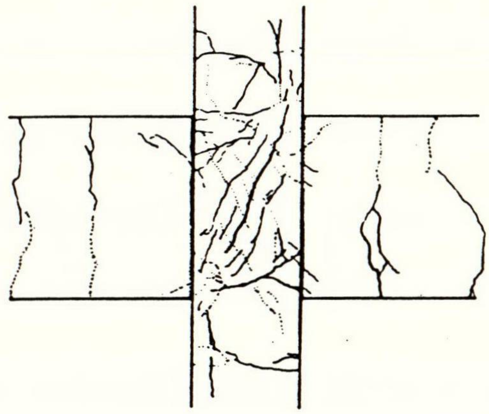
(a) at the peak of 0.5Pi
(R=0.48%)



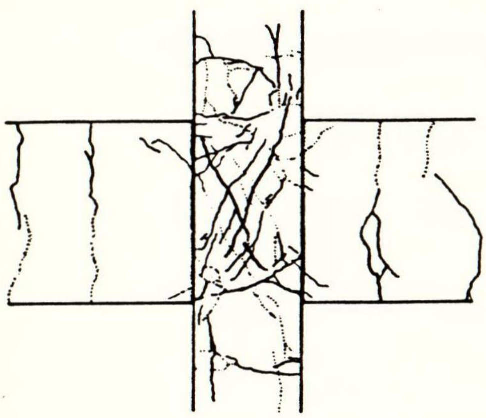
(b) at the peak of 0.75Pi
(R=0.88%)



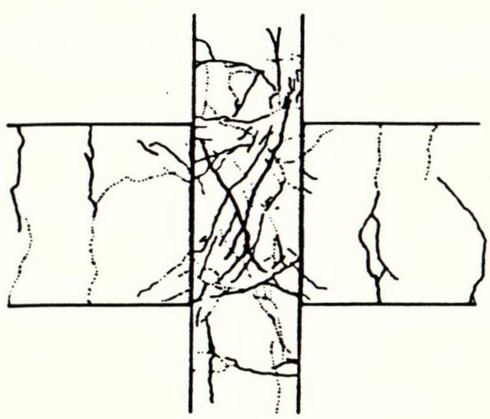
(c) at the peak of second cycle, DF=1
(R=1.17%)



(d) at the peak of first cycle, DF=+2
(R=2.34%)



(e) at the peak of first cycle, DF=-2



(f) at the peak of second cycle, DF=+2

Fig.4.1.3 Observed Cracking (Original Specimen O1)

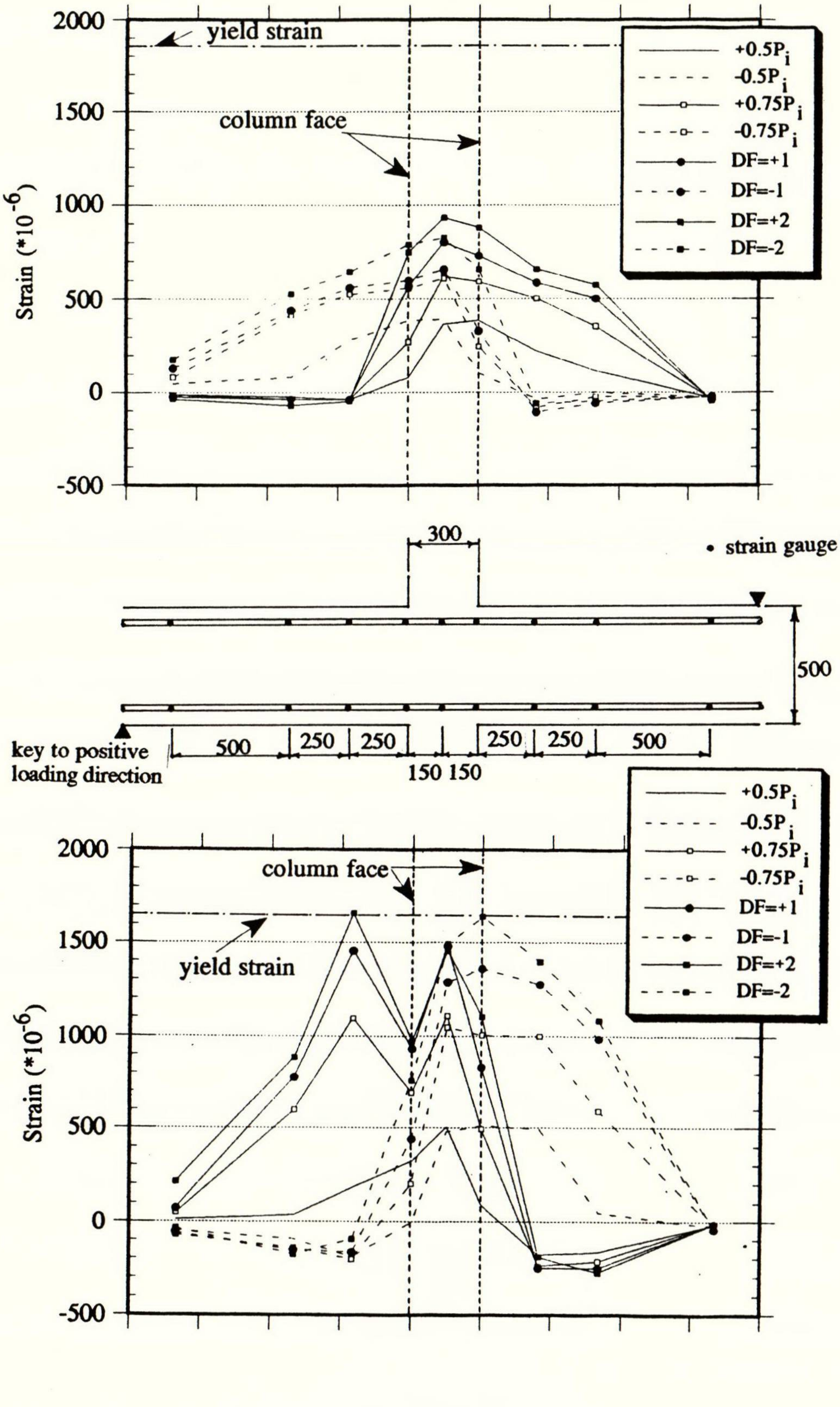


Fig.4.1.4 Strain Distributions of Beam Bars(O1)

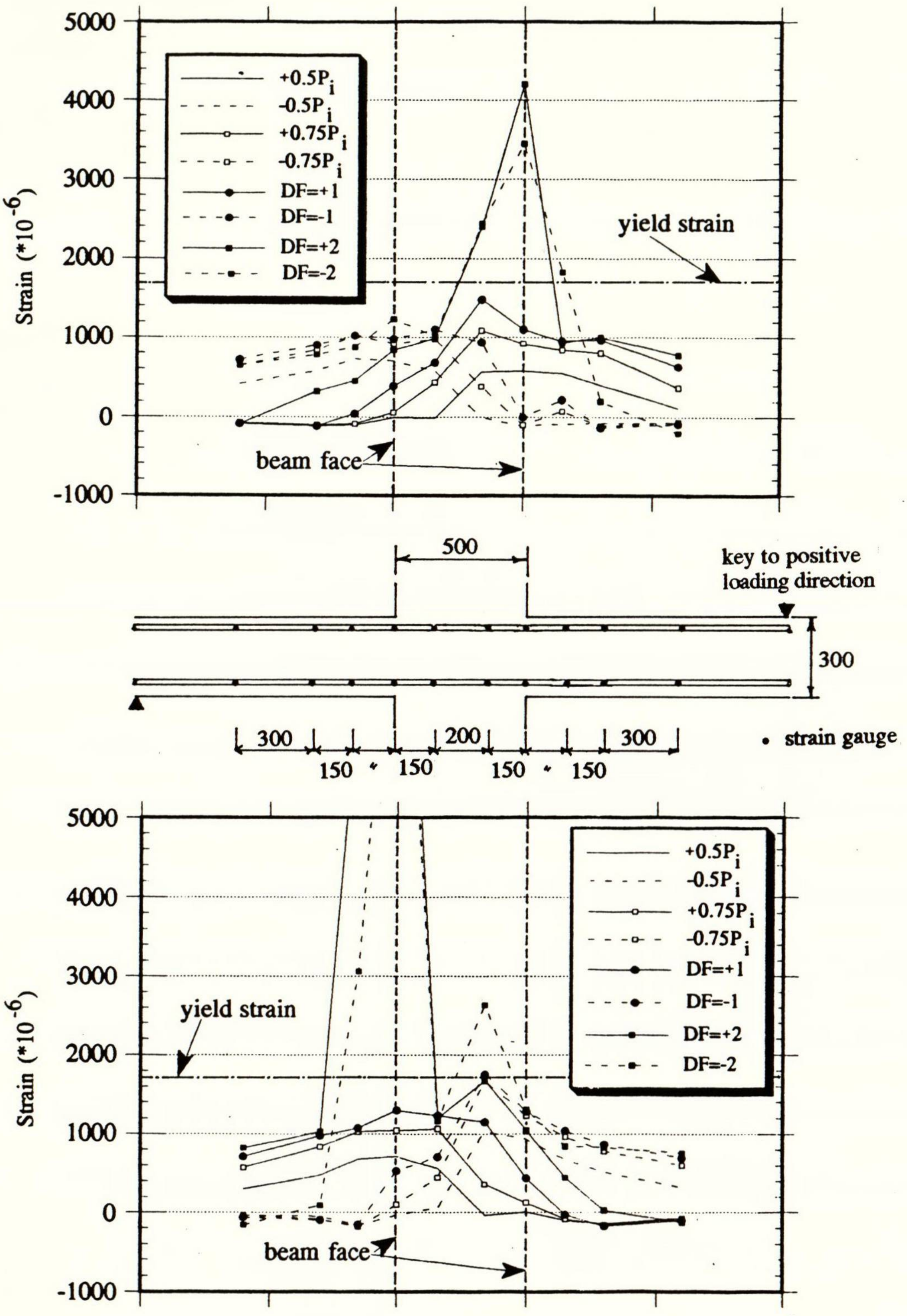


Fig.4.1.5 Strain Distributions of Column Bars(O1)

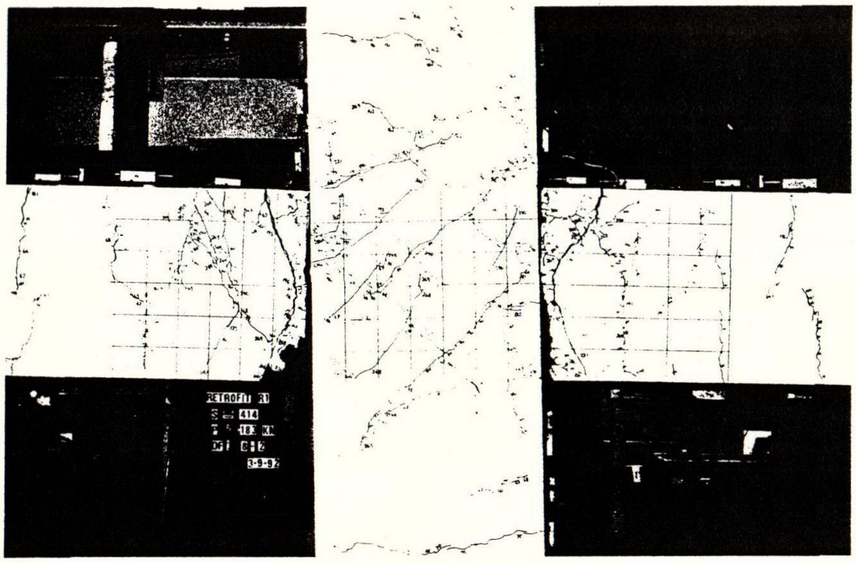


Fig.4.2.1 Observed Cracking of Retrofit Specimen(R1)
at second cycle of DF=-8

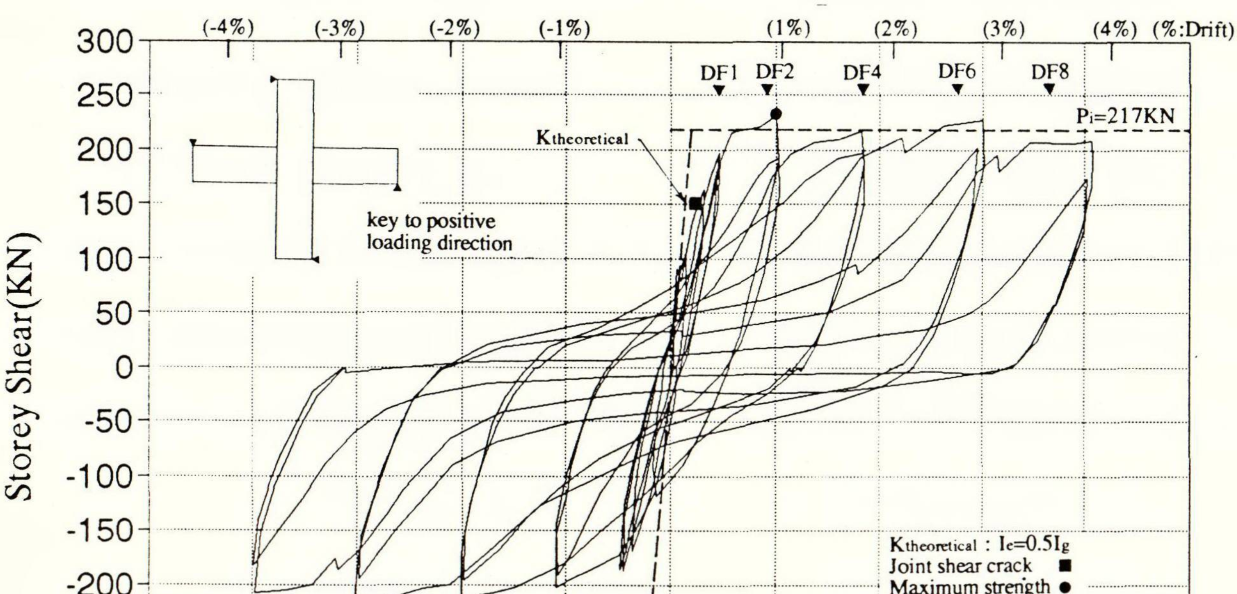
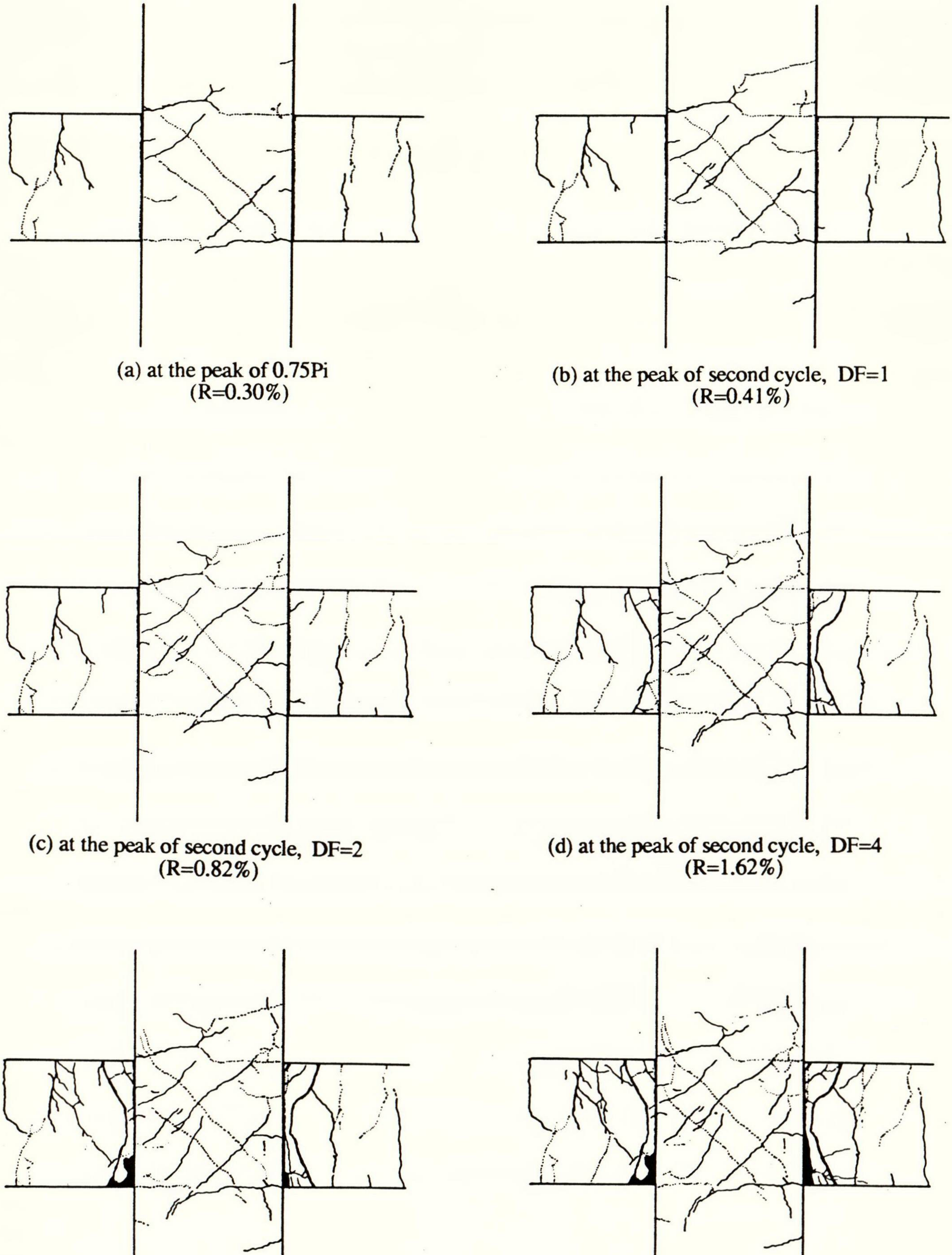


Fig.4.2.2 Storey Shear versus Horizontal Displacement Relationship
for Retrofit Specimen (R1)



(a) at the peak of 0.75Pi
(R=0.30%)

(b) at the peak of second cycle, DF=1
(R=0.41%)

(c) at the peak of second cycle, DF=2
(R=0.82%)

(d) at the peak of second cycle, DF=4
(R=1.62%)

(e) at the peak of second cycle, DF=6
(R=2.44%)

(f) at the peak of second cycle, DF=8
(R=3.25%)

Fig.4.2.3 Observed Cracking (Retrofit Specimen R1)

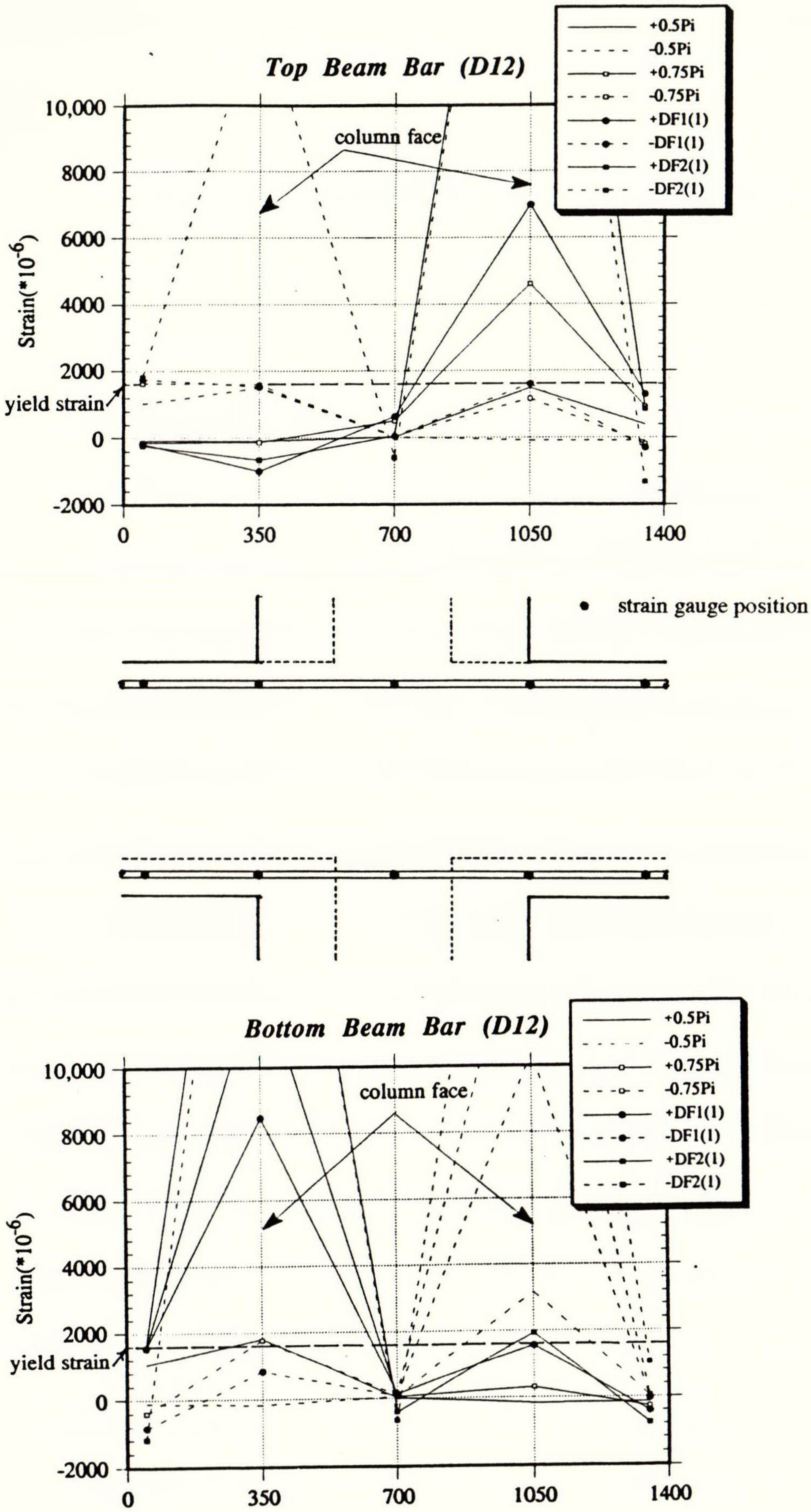


Fig.4.2.4 Strain Distributions of Beam Bars(R1)

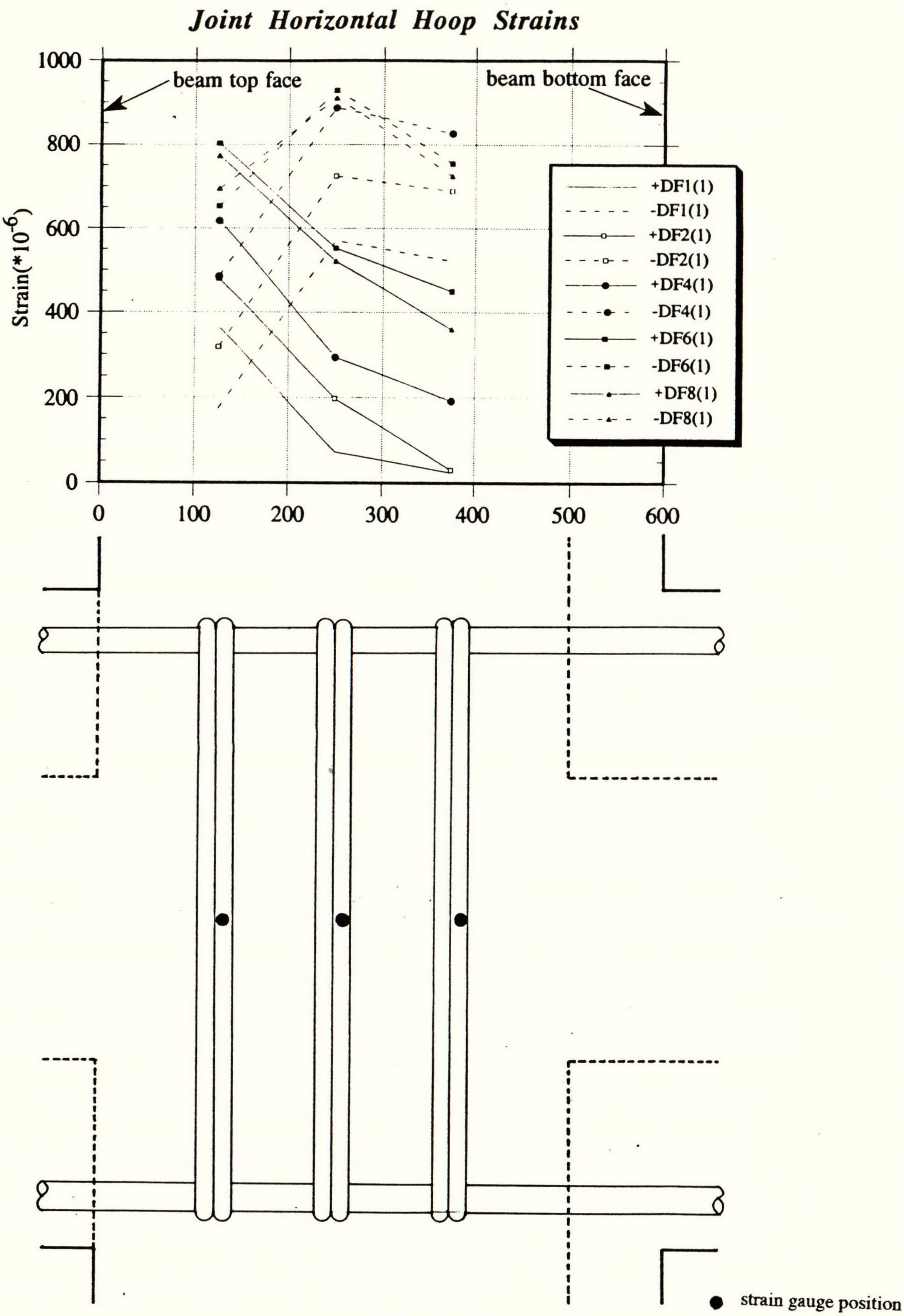


Fig.4.2.5 Strain Distribution of Joint Horizontal Hoops

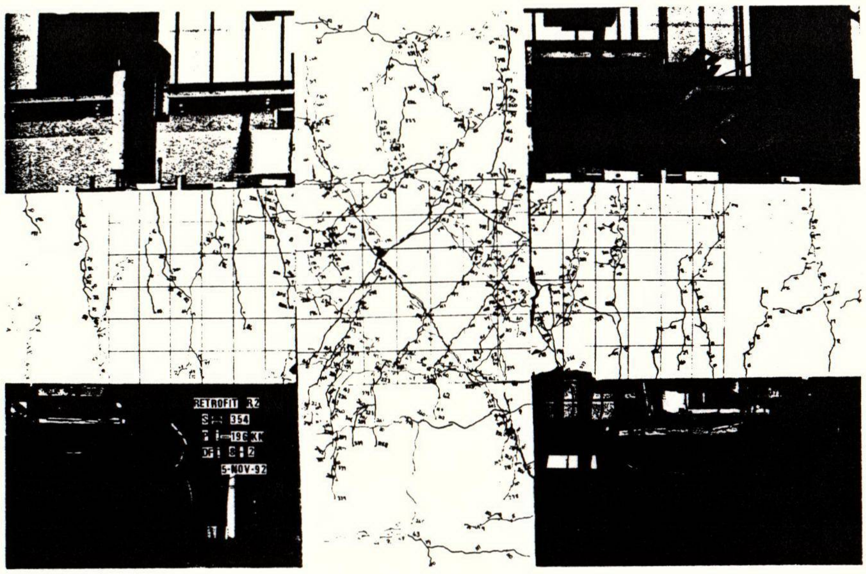


Fig.4.3.1 Observed Cracking of Retrofit Specimen(R2)
at second cycle of DF=-8

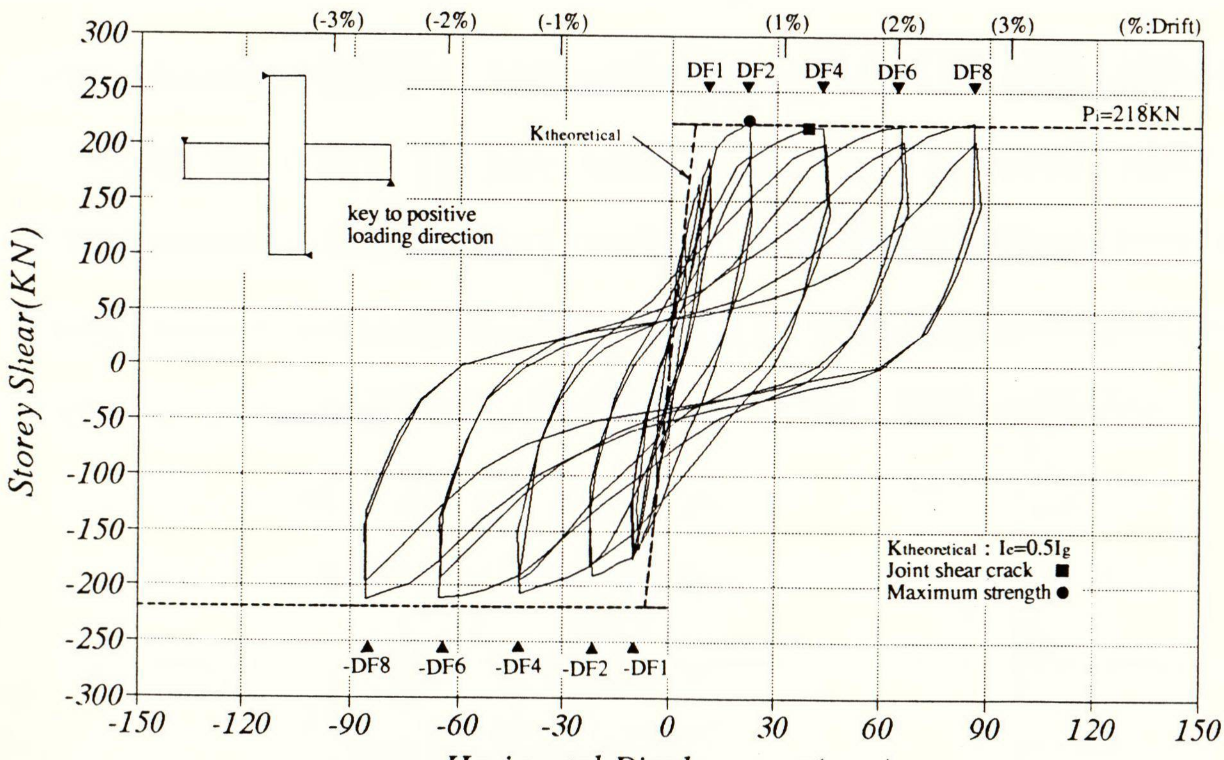
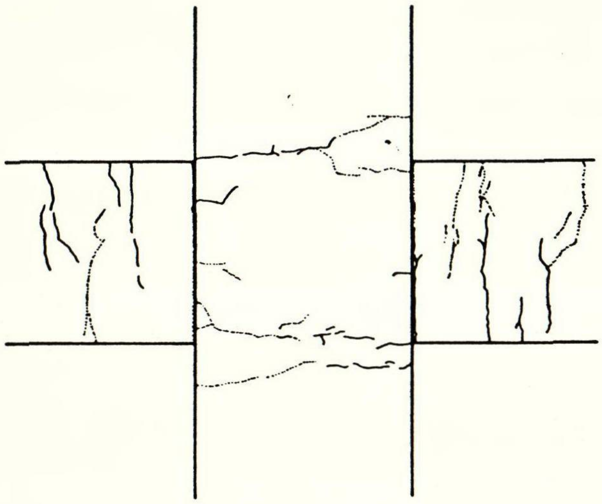
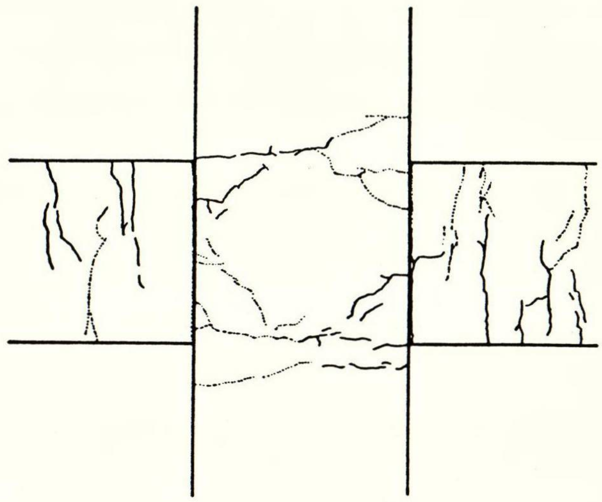


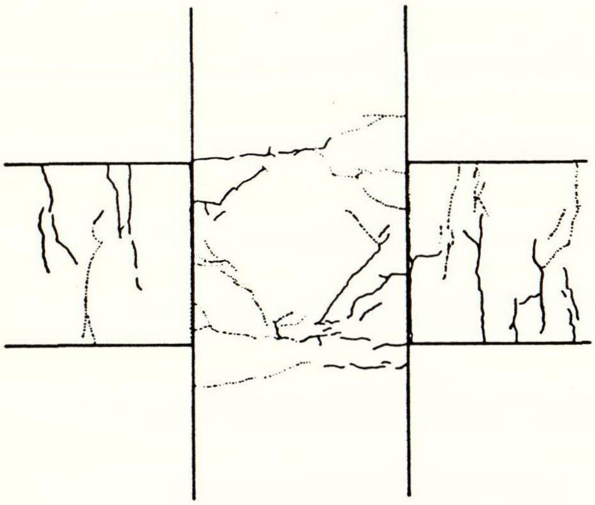
Fig.4.3.2 Storey Shear versus Horizontal Displacement Relationship
for Retrofit Specimen (R2)



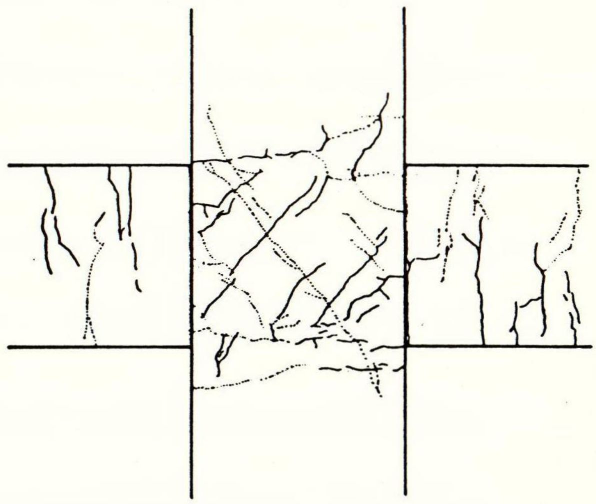
(a) at the peak of 0.75Pi
(R=0.25%)



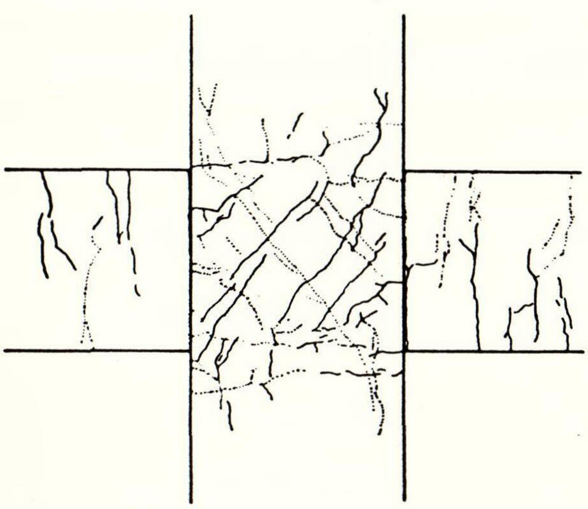
(b) at the peak of second cycle, DF=1
(R=0.34%)



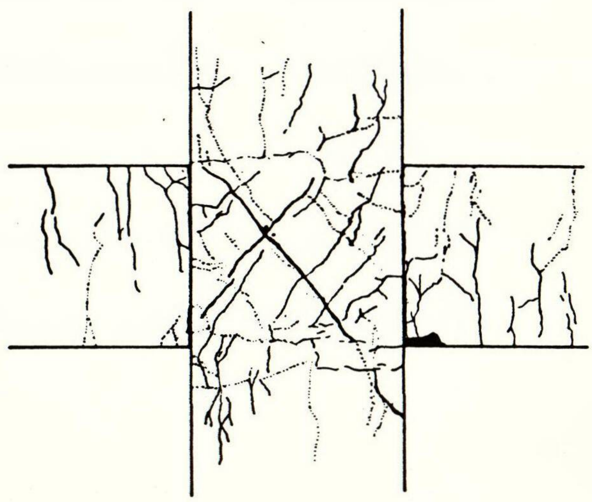
(c) at the peak of second cycle, DF=2
(R=0.67%)



(d) at the peak of second cycle, DF=4
(R=1.34%)



(e) at the peak of second cycle, DF=6
(R=2.01%)



(f) at the peak of second cycle, DF=8
(R=2.68%)

Fig.4.3.3 Observed Cracking (Retrofit Specimen R2)

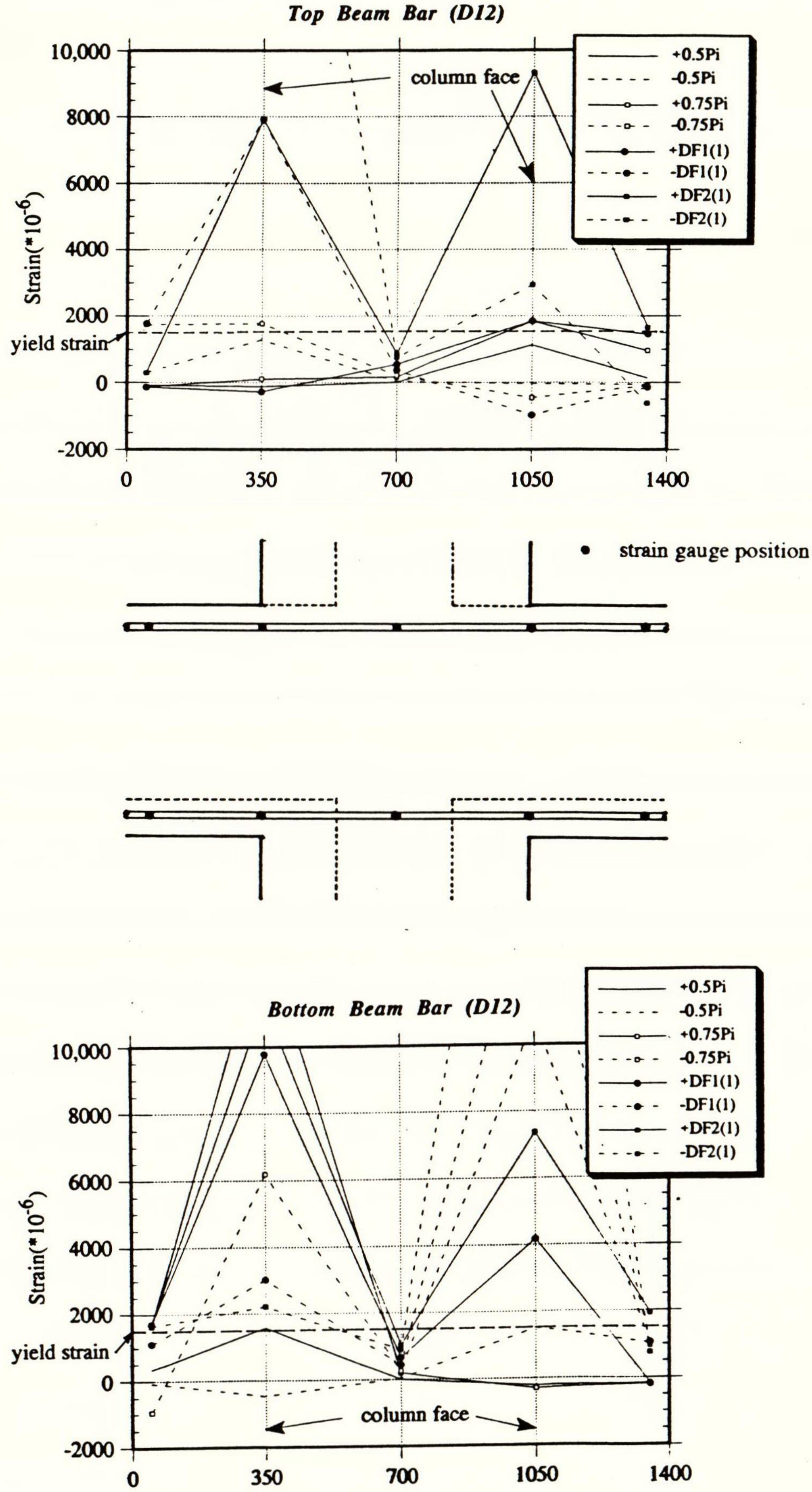


Fig.4.3.4 Strain Distributions of Beam Bars(R2)

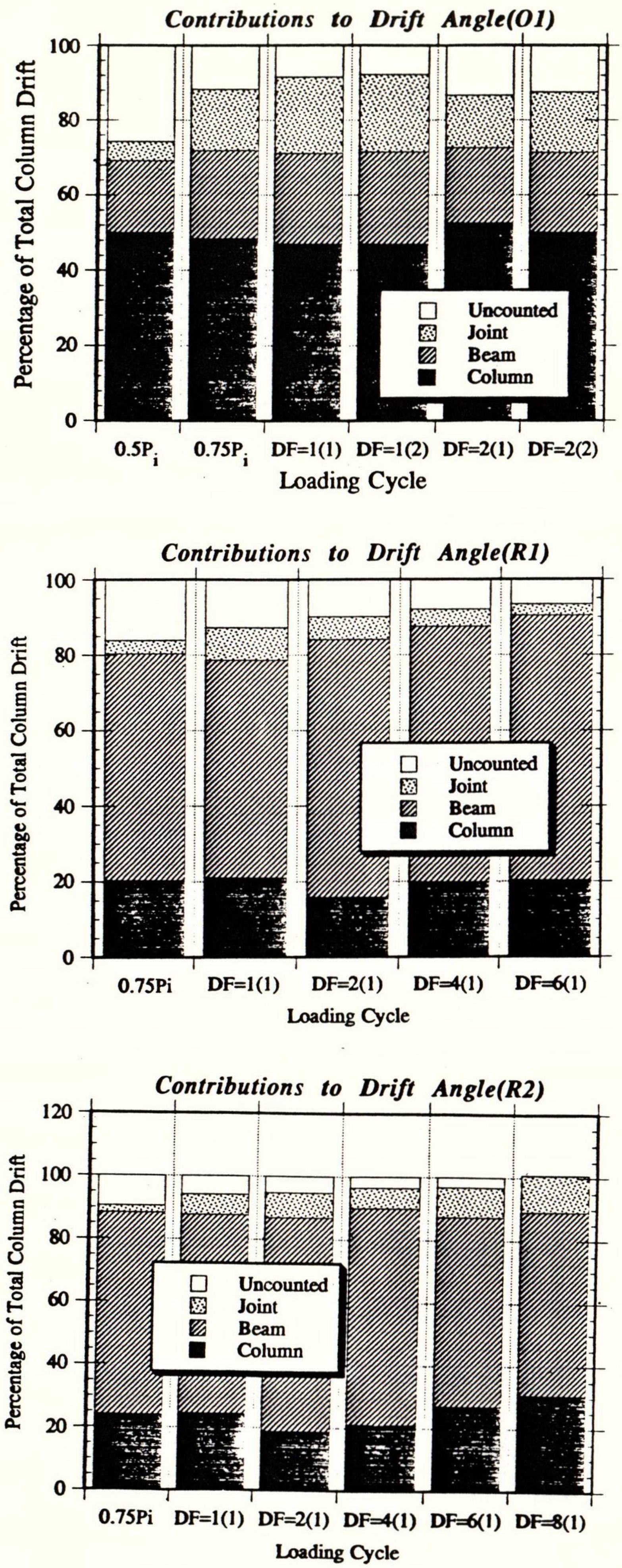


Fig.4.4 Components to Horizontal Displacement

<u>Joint Shear Stress</u>

Joint shear stress(v_{jh}) induced in the joint at developing both the joint diagonal tension cracks($_{cr}v_{jh}$) and maximum storey shear strength($_{max}v_{jh}$) were calculated as follows:

$$v_{jh}=(C'_s+C'_c+T-V_c)/b_jh_c \qquad \text{where } C'_s+C'_c=T'$$
$$=(T'+T-V_c)/b_jh_c$$

where the notation is as shown in Fig.4.5. Steel forces(T', T) until the beam bars yielded were calculated by using the wire strain gauge readings of beam bars at the column face. After yielding, it was assumed that the beam bar stress was the measured yield stress, f_y, up to the displacement ductility factor of 1 and that if the displacement ductility factor exceeded 1, the beam bar stressed 1.15 times the measured yield stress, $1.15f_y$.

When calculating the joint shear stress at developing the joint diagonal tension cracks, the effective joint shear stress(v'_{jh}) proposed by Priestley may be used. The effective joint shear stress was calculated as follows:

$$v'_{jh}=(T-V_c)/b_jh_c \qquad \text{where } T>T'$$

It should be noted that the effective joint shear stress could not be applied to the Retrofit specimen, R2, since the joint diagonal tension cracks initiated during the loading cycle of DF=4 as mentioned above.

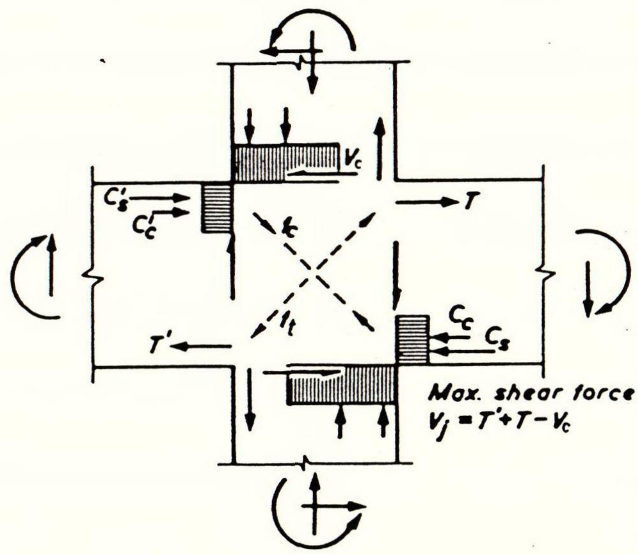


Fig.4.5 Actions and stress resultants of interior joint

Joint shear stress so obtained were

For Original Specimen(O1)

${}_{cr}v'_{jh}=1.52\text{MPa}=0.24\sqrt{f'_c}$ ${}_{cr}v_{jh}=2.66\text{MPa}=0.41\sqrt{f'_c}$

${}_{max}v_{jh}=3.88\text{MPa}=0.61\sqrt{f'_c}$

(where $f'_c=41\text{MPa}$)

For Retrofit Specimen(R1)

${}_{cr}v'_{jh}=1.10\text{MPa}=0.15\sqrt{f'_c{}^*}$ ${}_{cr}v_{jh}=1.88\text{MPa}=0.27\sqrt{f'_c{}^*}$

${}_{max}v_{jh}=2.03\text{MPa}=0.29\sqrt{f'_c{}^*}$

(where $f'_c{}^*=50\text{MPa}$)

For Retrofit Specimen(R2)

$({}_{cr}v'_{jh}=1.16\text{MPa}=0.16\sqrt{f'_c{}^*})$ ${}_{cr}v_{jh}=2.06\text{MPa}=0.28\sqrt{f'_c{}^*}$

${}_{max}v_{jh}=2.05\text{MPa}=0.27\sqrt{f'_c{}^*}$

(where $f'_c{}^*=56\text{MPa}$)

The joint region of the retrofit specimens consisted of two different concrete. A weighted average concrete strength($f'_c{}^*$) was used. The average concrete strength was given by

$$A_R f'_c{}^* = A_o f'_{c1} + A_j f'_{c2}$$

where A_R : joint area($=b_j h_c$)

A_o : cross section area of existing column

A_j : cross section area of jacketing column

f'_{c1} : concrete compressive strength of existing column

f'_{c2} : concrete compressive strength of jacketing column

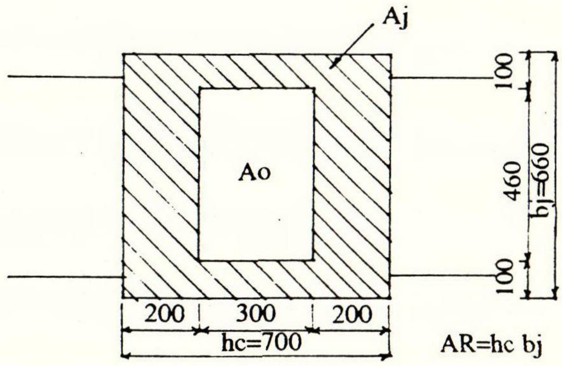


Fig.4.6 Cross section of the joint (specimen R1 and R2)

Based on the limited test data, two limiting conditions were identified for the seismic behaviour of the joint without horizontal shear reinforcement. At shear stress level of less than $0.3\sqrt{f'_c}$, the joint did not fail in shear and up to displacement ductility factor of 8, the joint did not affect the ductility of the beam adjacent to the joint which developed flexural plastic hinge. The stress level, $0.3\sqrt{f'_c}$, also corresponded to the joint shear stress level at initiating the joint diagonal tension cracking. It should be noted that the calculated joint shear stress is identical to the principal tension stress since no axial load was applied to the test specimens.

Joint shear failure was observed when joint shear stress level of $0.6\sqrt{f'_c}$ was developed and the hysteresis loops indicated rapid strength degradation and pinching. The joint behaviour with no joint horizontal reinforcement may be expressed as shown in Fig.4.7.

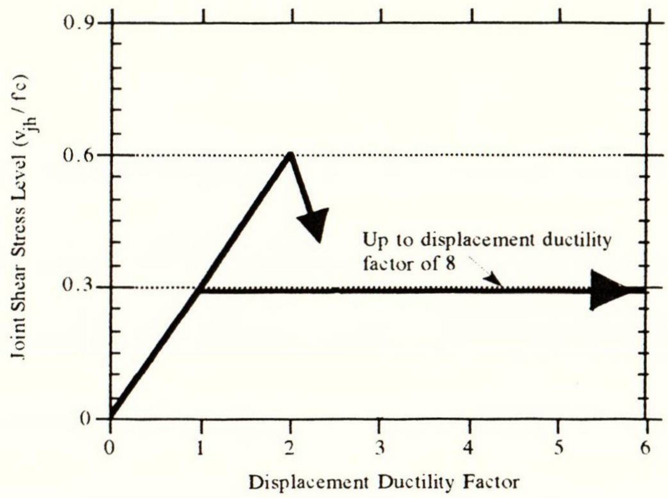


Fig.4.7 Joint behaviour

Comments

According to the test results, concrete jacketing was identified as a useful technique for enhancing the stiffness, strength and ductility of the existing frame. Even when no joint horizontal shear reinforcement was provided, a reduction in joint shear stress down to $0.3\sqrt{f'_c}$ by enlarging the column cross section resulted in almost the same behaviour of the specimen with joint hoops. However, concrete jacketing of both columns and beams will require extensive labour efforts even if placing joint horizontal shear reinforcement is eliminated. In order to develop more economical retrofit method, some test using concrete jacketing of column alone may be required.

During the elastic loading cycle, bond deterioration along both the beam and column main bars in the joint of the original specimen was found from the splitting cracks along the beam bars and strain distributions of both beam and column main bars. This is partly due to the relatively small column depth and the absence of the joint reinforcement as confinement. This bond deterioration caused a significant softening in the hysteresis loops. When compared with the theoretical initial stiffness(for example using $I_e=0.5I_g$), the measured stiffness was about 50% of the theoretical value. This is also the case of the retrofit specimens(R1,R2) which development length for column and beam bars are adequate by the code requirement. Even in the elastic loading cycle, slip of the beam and column main bars in the joint is believed to exist, attributing to the loss of initial stiffness. When assessing the existing frame, the effect of slip on the elastic response as well as inelastic response should be taken into account.