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SEISMIC UPGRADE FOR REINFORCED CONCRETE COLUMNS BY STRENGTHENING THE CROSS SECTIONAL CENTER

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SUMMARY

During the past major earthquakes, many column failures have been observed in reinforced concrete buildings with little shear walls such as pilotis where there are many openings in lower floors. The failure of such columns are in general a direct result of insufficient strength or ductility and can lead to the severe damage or even collapse of the building. The methodology we proposed herein is to increase the RC column ductility by strengthening the column cross section through embedding a small sectional area of steel. The embedded reinforcement is designed to resist the axial forces and, therefore, prevent the non-ductile failure of such a column. In order to verify the performance of such a seismically upgraded column during an earthquake, we have conducted both experimental tests and analytical analyses. In the experiment, we have tested 24 specimens of such a seismically upgraded RC column. In addition, we have performed analytical analysis to theoretically reconfirm the behavior of such a column. Based on both experimental tests and analytical results, we have concluded that embedding a small sectional area of steel into the cross sectional center of RC column is an effective seismic upgrade methodology. It can reduce the development of cracks at the middle of column and, therefore, achieve the ductile behavior of such a column without weakening its strength.

INTRODUCTION

Many reinforced concrete structures have suffered damages in the past earthquakes due to the shear failure of reinforced concrete members. Building code and regulations have taken measures to prevent this kind of failure of reinforced concrete structure by enforcing more stringent requirement in the strength of the shear reinforcement and horizontal load carrying capacity in the RC structures.

Nevertheless, in the 1995 Great Kobe earthquake, piloti-like reinforced concrete structures that were designed according to the current seismic design requirement have suffered severe damages due to the following reasons. (1) The horizontal stiffness and strength in the piloti floor are significantly smaller than the floors above and, therefore, during the shaking of the earthquake, the first floor has yielded prior to the stories above and caused

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the concentration of excessive horizontal forces at the piloti floor; (2) In general, the stories above the first piloti floor are multi-story shear wall and, therefore, the RC columns in the piloti floor have been acted upon by large axial forces due to the overturning moment,. These large axial compression forces can cause the concrete compression failure and the rebars buckling failure. As these failures progress, they cause the sudden structural collapse as seen in the case where the member sizes are reduced.

As a result, it is necessary to strengthen these reinforced concrete columns in order to build the piloti-like structures. For such a purpose, we have proposed to embed a steel member into the center of the column cross-section as a reinforcing element. This method of seismic strengthening has advantage of easy-construction and low-cost.

In order to study the effect of this seismic strengthening, we have conducted experimental research. In the experiment, we have tested reinforced concrete columns with shear span ratio of 2.5 and varied as the parameters the axial force ratio, tie ratio, central reinforcement ratio and central reinforcement anchor length. In addition, we have developed the truss model considering the stress distribution and analyzed the central strengthened RC columns, and as the result understood the dynamic behavior of such a central strengthened RC columns during the dynamic horizontal loading.

2. EXPERIMENTAL PROGRAM

The configuration of the test specimen is as follows. The cross sectional size is 20cm x20cm, the length is 100cm, and the ratio of shear span to the depth is 2.5. To investigate the dynamic behavior of reinforced concrete columns with central reinforcement, we have chosen the following parameters:

(1) two axial force ratios, 0.2 and 0.4
(2) 9mm round ties with two different tie ratios (Pw), Pw=1.28% at 50mm spacing and Pw=0.85% at 75mm spacing
(3) central reinforcing elements include the following
    CH5-type : flanged (H-shaped) steel in the strong axis direction (H-50x50x5x7)
    CH3-type : flanged (H-shaped) steel in the strong axis direction (H-30x30x5x7)
    CI5-type : flanged (H-shaped) steel in the weak axis direction (H-50x50x5x7)
    CP-type : steel tube (42.7x3.5)
    CPG-type : steel tube (42.7x3.5) with mortar infill
    CB-type : deformed bar
    CB4 type : a bundle of 4 deformed bars
    MB-type : no central reinforcement
(4) central reinforcement anchor length in the flanged steel in the strong axis (H-50x50x5x7)
    CH5S-type : 5cm
    CH5M-type : 15cm
    CH5L-type : 30cm

There are a total of 24 specimens as listed in Table 1. Their dimensions and cross sections are shown in Figure 1. The central reinforcement anchor length is shown in Figure 2. The rebar strength is listed in Table 2.

The nomenclature of the test specimens is [type of central reinforcement]-[tensile steel ratio]-[tie ratio]-[axial load ratio].

The axial load ratio is defined as $\sigma = N/(Fc \times Ac + Fs \times As)$

Where Fc is compression strength of concrete, Ac is cross section of concrete, Fs compression strength of steel and As is total cross section of steel.

The test device is shown in Figure 3. The reinforced concrete columns are fixed to refrain from overturning at both ends and loaded by moving the loading device horizontally. The axial load is kept constant and the cyclic horizontal forces is applied by the jacks at right and left hand sides. The loading histories are shown in Figure 4. The displacement is measured using dial gage and the strain of rebars and reinforcing steel members is measured using wire strain gage.

Table 1 – List of Specimen

No.	Name of Specimen	Strength of Concrete(MPa)		Tie Ratio (%)	Tensile Steel Ratio (%)	Central Reinforcement ratio (%)
		Compression	Tension			
1	**MB-1.44-0.85-0.2**	22.1	2.2	0.85	1.44	-
2	**MB-1.00-0.85-0.2**	26.8	2.4	0.85	1.00	-
3	**MB-1.44-1.28-0.2**	36.1	3.1	1.28	1.44	-
4	**MB-1.00-0.85-0.35**	37.2	3.3	0.85	1.00	-
5	**MB-1.00-0.85-0.4**	23.1	2.2	0.85	1.00	-
6	**MB-1.44-0.85-0.4**	19.9	2.2	0.85	1.44	-
7	**MB-1.44-1.28-0.4**	34.1	3.1	1.28	1.44	-
8	**CB-1.00-1.28-0.2**	36.1	3.1	1.28	1.00	0.97
9	**CB-1.00-1.28-0.4**	34.1	3.1	1.28	1.00	0.97
10	**CB4-1.00-0.85-0.35**	37.2	3.3	0.85	1.00	2.01
11	**CP-1.00-1.28-0.2**	36.1	3.1	1.28	1.00	1.08
12	**CP-1.00-1.28-0.4**	34.1	3.1	1.28	1.00	1.08
13	**CPG-1.00-1.28-0.4**	25.0	2.6	1.28	1.00	1.08
14	**CH3-1.00-0.85-0.2**	26.8	2.4	0.85	1.00	1.39
15	**CH3-1.00-0.85-0.4**	23.1	2.2	0.85	1.00	1.39
16	**CH5S-1.00-0.85-0.2**	26.8	2.4	0.85	1.00	2.34
17	**CH5S-1.00-0.85-0.35**	37.2	3.3	0.85	1.00	2.34
18	**CH5S-1.00-0.85-0.4**	23.1	2.2	0.85	1.00	2.34
19	**CH5S-1.00-1.28-0.4**	25.0	2.6	1.28	1.00	2.34
20	**CH5M-1.00-0.85-0.4**	22.0	2.2	0.85	1.00	2.34
21	**CH5L-1.00-0.85-0.2**	25.3	2.4	0.85	1.00	2.34
22	**CH5L-1.00-0.85-0.4**	25.3	2.4	0.85	1.00	2.34
23	**CI5-1.00-0.85-0.2**	22.1	2.2	0.85	1.00	2.34
24	**CI5-1.00-0.85-0.4**	19.9	2.2	0.85	1.00	2.34

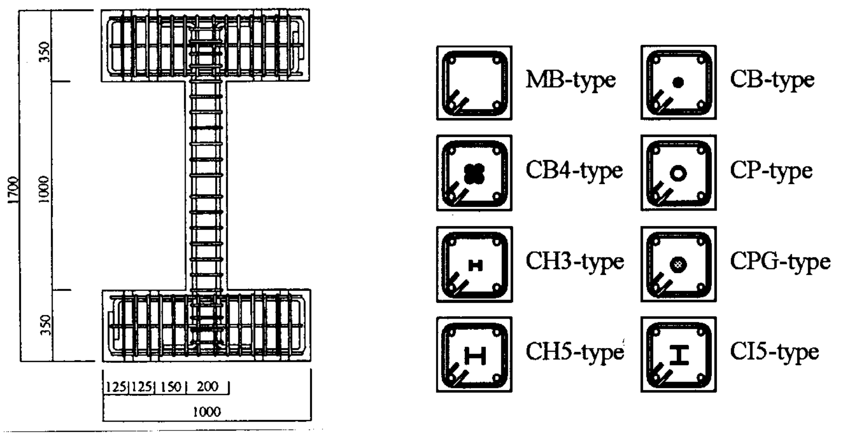


Figure 1 – Test Specimen

Table 2 – Reinforcement Properties

	Yield Strength (kgf/cm^2)	Max. Strength (kgf/cm^2)
D16	3896	5670
D19	3789	5737
D22	3948	5919
9ϕ	3262	4469

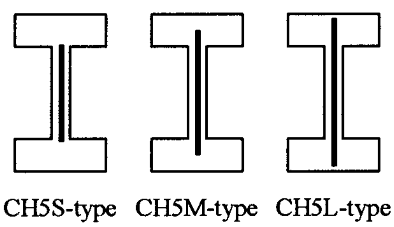


Figure 2 – Central Reinforcement Anchor Scheme

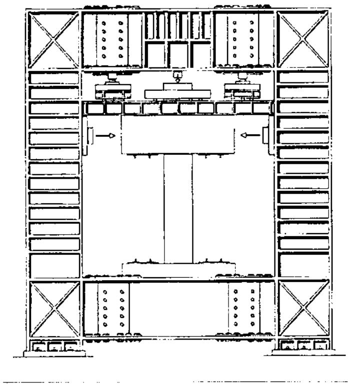


Figure 3 – Test device

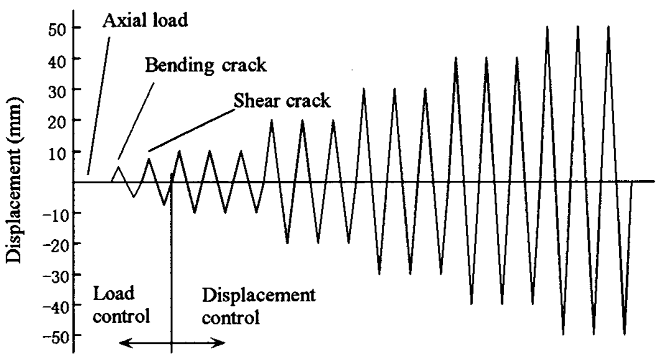


Figure 4 – Loading history

3. EXPERIMENTAL RESULTS

3.1 Effect due to axial force

First we have investigated the effect on the failure development due to the axial force. The conventional reinforced concrete columns without central reinforcement include specimen MB-1.00-0.85-0.2 and MB-1.00-0.85-0.4 and the specimens with larger axial force ratio have larger ultimate capacity. Also, the reinforced concrete columns with larger axial force ratio has smaller deformation at the ultimate capacity than the columns with smaller axial force ratio, as shown in Table 1. This is due to the fact that in the MB-1.00-0.85-0.2 columns, the tensile strength of the rebars determine the ultimate capacity whereas in the MB-1.00-0.85-0.4 columns, the concrete compression failure decides the ultimate capacity. In addition, the degradation of the capacity after the ultimate capacity point is more significant and more sudden in the reinforced concrete columns with larger axial force ratio than in the ones with smaller axial force ratio.

By examining the cracks shown in the figures for MB-1.00-0.85-0.2 and MB-1.00-0.85-0.4, we have observed that bending cracks, shear cracks, and compression cracks occurred in the member ends of both specimens at the respective ultimate capacities. The difference between the two specimens is that the specimen of MB-1.00-0.85-0.4 has more compression cracks than MB-1.00-0.85-0.2 specimen. The bonding slippage failure after the ultimate capacity point due to the cyclic shear loading is more significant in the MB-1.00-0.85-0.2 specimen with axial force ratio of 0.2 than the other. In addition, in the MB-1.00-0.85-0.4 specimen, the tendency of bending cracks and shear cracks after the ultimate capacity are observed in the middle section of the member.

The reinforced concrete with central reinforcement includes specimens CH5S-1.00-0.85-0.2 and CH5S-1.00-0.85-0.4 and the effect due to the axial force ratio are shown in Figures 2 and 3. It is observed from the figure that the similar conclusions can be drawn regarding the ultimate capacities as well as the crack development for the central reinforced RC columns as for the conventional RC columns without the central reinforcement.

3.2 Effect due to tie ratios

Next we have examined the effect on the failure development due to different tie ratios. It is observed that tie ratios do not cause any significant difference in the ultimate capacity and the drift at the ultimate capacity.

Regarding the behavior after the ultimate capacity, the following remarks can be stated. In the specimen with tie ratio of 0.85% the cracks develop to the middle section of the member whereas in the specimen with tie ratio of 1.28% the cracks tend to concentrate and do not propagate to the member middle section. Based upon the shear load/max shear load-cycle relationship shown in Figure 7, it is observed that at the tie ratio of 0.85%, the specimen with central reinforcement (CH5S-1.00-0.85-0.4) has shown significantly less capacity degradation even comparing with MB-1.00-0.85-0.4 and exhibited even more ductile behavior than the specimens with tie ratios of 1.28%. This clearly indicates that the central reinforcement improves the column ductility.

3.3 Effect due to central reinforcement

Thirdly, we have investigated the effect due to the central reinforcement shapes. The central reinforcement element includes deformed rebars (CB-1.00-1.28-0.4), steel tube(CP-1.00-1.28-0.4), steel tube with mortar infill (CPG-1.00-1.28-0.4) and flanged (H-shaped) steel(CH5S-1.00-1.28-0.4).

In the CB-1.00-1.28-0.4 specimen the cracks concentrates at the member ends and do not develop to the member middle section whereas in the remaining three specimens the cracks develop to the member middle section.

As far as the ultimate capacity, all the specimens have shown the same capacity and no noticeable difference is observed due to the different central reinforcement.

Regarding the capacity degradation post the ultimate capacity due to the cyclic horizontal shear loading, the specimens CPG-1.00-1.28-0.4 and CH5S-1.00-1.28-0.4 have demonstrated ductile behavior with respect to the other two specimens CB-1.00-1.28-0.4 and CP-1.00-1.28-0.4. CB-1.00-1.28-0.4 and CP-1.00-1.28-0.4 have similar capacity degradation without noticeable difference observed.

As the result, the behavior of the columns after reaching its ultimate capacity does not depend on the central reinforcement shapes but only on the cross sectional area of the central reinforcement.

To compare the effect due to the orientation and the cross sectional area of the flanged (H-shaped) steel, the following four specimens were tested. (a) Specimen MB-1.00-0.85-0.4 without the central reinforcement, (b) Specimen CH3-1.00-0.85-0.4 with the central reinforcement of (30mm x 30mm x 5mm x 7mm) flanged steel in the strong axis direction with central reinforcement ratio of 1.39%, (c) Specimen CH5S-1.00-0.85-0.4 with the central reinforcement of (50mm x 50mm x 5mm x 7mm) flanged steel in the strong axis direction with central reinforcement ratio of 2.34%, and (d) Specimen CI5-1.00-0.85-0.4 which is the same as Specimen CH5S-1.00-0.85-0.5 except that the central reinforcement is in the weak axis direction rather than in the strong axes direction.

In the MB-1.00-0.85-0.4 and CH3-1.00-0.85-0.4 specimens the cracks concentrates at the member ends whereas in the other two specimens (CH5S-1.00-0.85-0.4 and CI5-1.00-0.85-0.4) the cracks develop to the member middle section.

Regarding the cracks after the ultimate capacity, the following remarks can be stated. The specimen MB-1.00-0.85-0.4 without the central reinforcement has shown crack concentration at the member ends and non-ductile behavior. In the two specimens CH3-1.00-0.85 and CH5S-1.00-0.85-0.4 the cracks first developed at the member ends and then propagated to the middle section, and the specimen CI5-1.00-0.85-0.4 has no crack concentration and bigger cracks at the middle section with respect to the fore-mentioned two specimens.

Regarding the maximum capacity, there is no noticeable difference in the three specimens MB-1.00-0.85-0.4, CH3-1.00-0.85-0.4 and CH5S-1.00-0.85-0.4. The specimen CI5-1.00-0.85-0.4 with the central reinforcement in the weak direction has larger ultimate capacity and deformation at the ultimate capacity comparing to the remaining three specimens.

As far as the capacity degradation due to the cyclic horizontal shear loading after the ultimate capacity, MB-1.00-0.85-0.4 without central reinforcement and CH3-1.00-0.85-0.4 with 1.39% central reinforcement have shown similarly severe degradation, whereas CH5S-1-1.00-0.85-0.4 with 2.34% central reinforcement has shown ductile behavior with a small amount of degradation.

Among the four specimens, CI5-1.00-0.85-0.4 has the highest capacity but the capacity degradation after the ultimate capacity is also the most significant.

3.4 Effect due to the central reinforcement length

Specimen CH5L-1.00-0.85-0.4 has central reinforcement anchor length up to the beam ends (30cm long) and its ultimate capacity is higher than specimen CH5S-1.00-0.85-0.4 and CH5M-1.00-0.85-0.4 which have shorter central reinforcement anchor length. This is due to the fact that the central reinforcing flanged steel resists the shear force and therefore the bending moment in the case of CH5L-1.00-0.85-0.4.

Also, the capacity degradation due to cyclic loading after its ultimate capacity is most significant in the case of the specimen CH5L-1.00-0.85-0.4 which has the longest central reinforcement anchor length. It is believed that the yielding of the central reinforcement contributes towards this most significant capacity degradation.

There is no noticeable difference in the capacity degradation post ultimate capacity between specimens CH5S-1.00-0.85-0.4 and CH5M-1.00-0.85-0.4 where CH5S-1.00-0.85-0.4 has almost no anchor length and CH5M-1.00-0.85-0.4 has anchor length up to the beam middle section.

Finally, it is observed that there is almost no effect on the crack development due to different anchor length.

4. ANALYTICAL STUDY

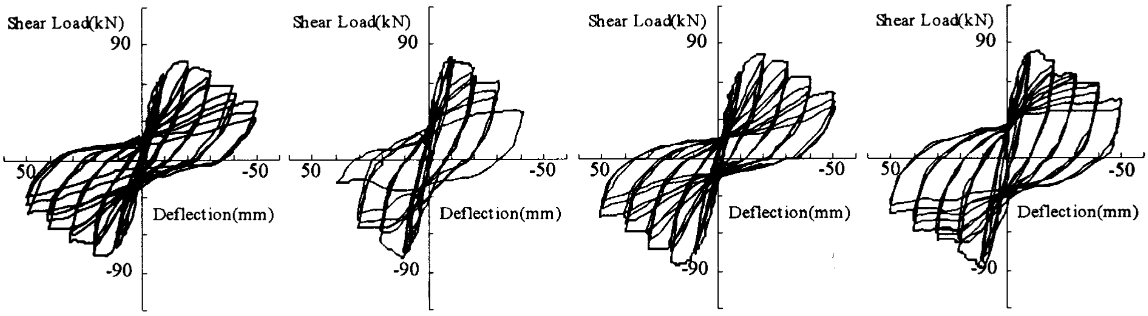
4.1 Analysis Summary

We have chosen 15 specimens out of the 24 specimens tested and performed theoretical analysis. After examining the experiment, we have concluded that the RC columns with shear span ratio of 2.5 have developed their failures according to the following steps as shown schematically in Figure 8 (a)—(e).

(a) First, the bending cracks develop at its farmost ends.
(b) Then the bending cracks continue to grow to the ends.
(c) The bending cracks further develop causing the bending and shear cracks and the bonding cracks.
(d) The column reaches its ultimate capacity when the shear and bonding cracks occur at the ends.
(e) The columns with central reinforcement will maintain its ultimate capacity while the shear cracks develop at the column center.

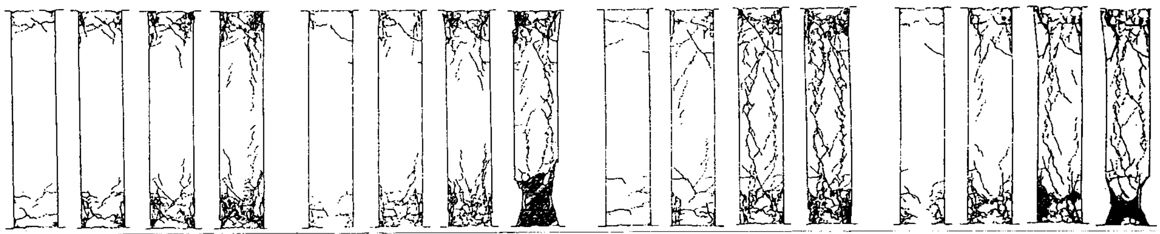
Table 3 – Experimental result

No.	Name of Specimen	Strength of Concrete (MPa)		Shear Load(kN)										
				Max.	Deflection / Length									
		Compression	Tension		1/100		2/100		3/100		4/100		5/100	
1	MB-1.44-0.85-0.2	22.1	2.2	83.6	64.7	-65.7	83.6	-82.3	70.6	-62.7	43.1	-43.1	30.7	-29.4
2	MB-1.00-0.85-0.2	26.8	2.4	76.4	66.6	-68.6	75.5	-72.5	67.6	-61.7	57.8	-51.9	44.1	-42.1
3	MB-1.44-1.28-0.2	36.1	3.1	102.9	95.1	-90.2	102.9	-100.9	87.2	-91.1	59.8	-60.8	33.3	-33.3
4	MB-1.00-0.85-0.35	37.2	3.3	122.5	122.5	-109.8	87.2	-89.2	50.0	-27.4	-	-	-	-
5	MB-1.00-0.85-0.4	23.1	2.2	78.4	78.4	-73.5	69.9	-65.7	58.8	-53.9	36.3	-17.6	-	-
6	MB-1.44-0.85-0.4	19.9	2.2	90.8	82.3	-86.2	88.9	-87.2	71.2	-61.7	36.6	-26.5	13.7	-11.8
7	MB-1.44-1.28-0.4	34.1	3.1	100.0	94.1	-97.0	75.5	-89.2	64.7	-75.5	58.8	-32.3	-	-
8	CB-1.00-1.28-0.2	36.1	3.1	96.0	86.2	-88.2	80.4	-86.2	66.6	-75.5	56.8	-57.8	53.9	-55.9
9	CB-1.00-1.28-0.4	34.1	3.1	93.1	89.2	-86.2	71.5	-65.7	60.8	-61.7	55.9	-47.0	-	-
10	CB4-1.00-0.85-0.35	37.2	3.3	125.4	125.4	-113.7	89.2	-96.0	55.9	-61.7	-	-	-	-
11	CP-1.00-1.28-0.2	36.1	3.1	102.9	93.1	-86.2	95.1	-92.1	75.5	-78.4	64.7	-66.6	57.8	-58.8
12	CP-1.00-1.28-0.4	34.1	3.1	98.0	97.0	-87.2	78.4	-65.7	61.7	-56.8	62.7	-57.8	-	-
13	CPG-1.00-1.28-0.4	25.0	2.6	96.0	96.0	-88.2	96.0	-87.2	72.5	-57.8	64.7	-52.9	57.8	-46.1
14	CH3-1.00-0.85-0.2	26.8	2.4	77.4	61.7	-74.5	76.4	-73.5	65.7	-63.7	52.9	-50.0	42.1	-39.2
15	CH3-1.00-0.85-0.4	23.1	2.2	79.4	77.4	-75.5	68.6	-74.5	56.8	-55.9	44.1	-43.1	32.3	-26.5
16	CH5S-1.00-0.85-0.2	26.8	2.4	80.4	65.7	-74.5	80.4	-80.4	73.5	-69.6	60.8	-57.8	50.0	-45.1
17	CH5S-1.00-0.85-0.35	37.2	3.3	118.6	118.6	-95.1	82.3	-86.2	44.1	-58.8	-	-	-	-
18	CH5S-1.00-0.85-0.4	23.1	2.2	82.3	79.4	-79.4	73.5	-73.5	63.7	-63.7	55.9	-52.3	49.0	-44.1
19	CH5S-1.00-1.28-0.4	25.0	2.6	101.9	100.9	-93.1	101.9	-84.3	74.5	-55.9	71.5	-46.1	-	-
20	CH5M-1.00-0.85-0.4	22.0	2.2	89.5	84.9	-93.1	83.6	-82.3	72.5	-67.6	60.8	-57.8	45.1	-43.1
21	CH5L-1.00-0.85-0.2	25.3	2.4	92.1	76.4	-83.3	89.2	-82.3	75.5	-71.5	63.7	-58.8	59.8	-53.9
22	CH5L-1.00-0.85-0.4	25.3	2.4	96.0	94.1	-89.2	76.4	-77.4	60.8	-64.7	57.8	-54.9	43.1	-43.1
23	CI5-1.00-0.85-0.2	22.1	2.2	77.7	52.9	-58.8	77.7	-75.5	71.9	-66.6	62.1	-55.9	52.3	-42.1
24	CI5-1.00-0.85-0.4	19.9	2.2	84.3	73.8	-81.3	81.0	-85.3	70.6	-71.5	61.4	-58.8	40.5	-35.3



MB-1.00-0.85-0.2 MB-1.00-0.85-0.4 CH5S-1.00-0.85-0.2 CH5S-1.00-0.85-0.4

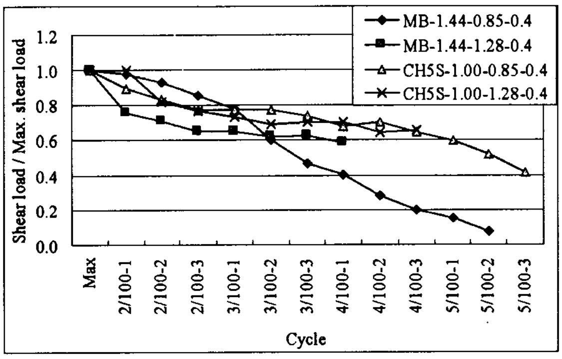
Figure 5-Shear load-Deflection relationship



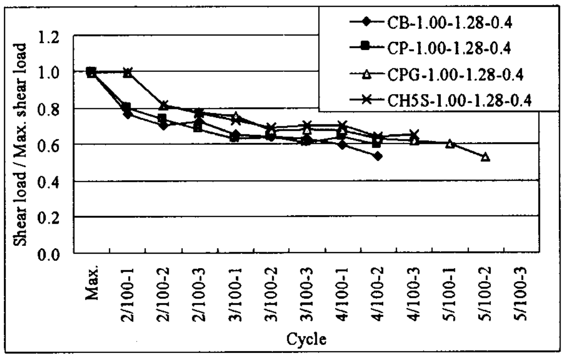
1/100 2/100 3/100 4/100 1/100 2/100 3/100 4/100 1/100 2/100 3/100 4/100 1/100 2/100 3/100 4/100

MB-1.00-0.85-0.2 MB-1.00-0.85-0.4 CH5S-1.00-0.85-0.2 CH5S-1.00-0.85-0.4

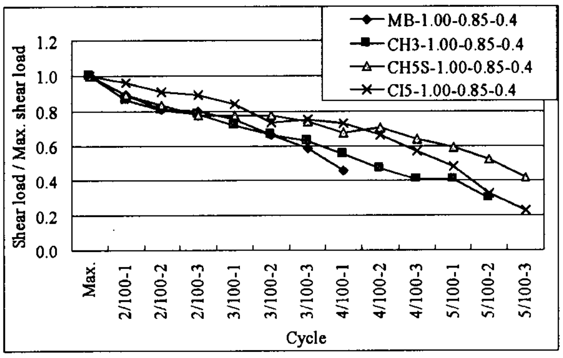
Figure 6-Crack patterns



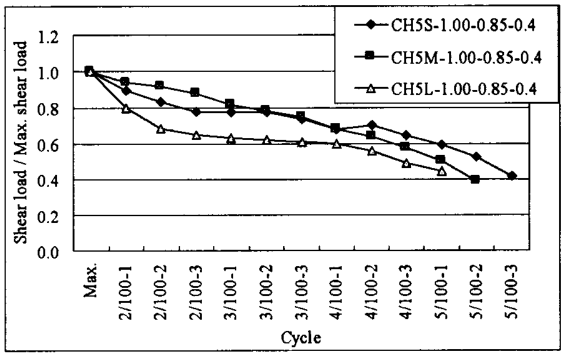
Effect due to tie ratios



Effect due to central reinforcement



Effect due to tie ratios



Effect due to the central reinforcement length

Figure 7-Shear load/Max shear load-Cycle relationship

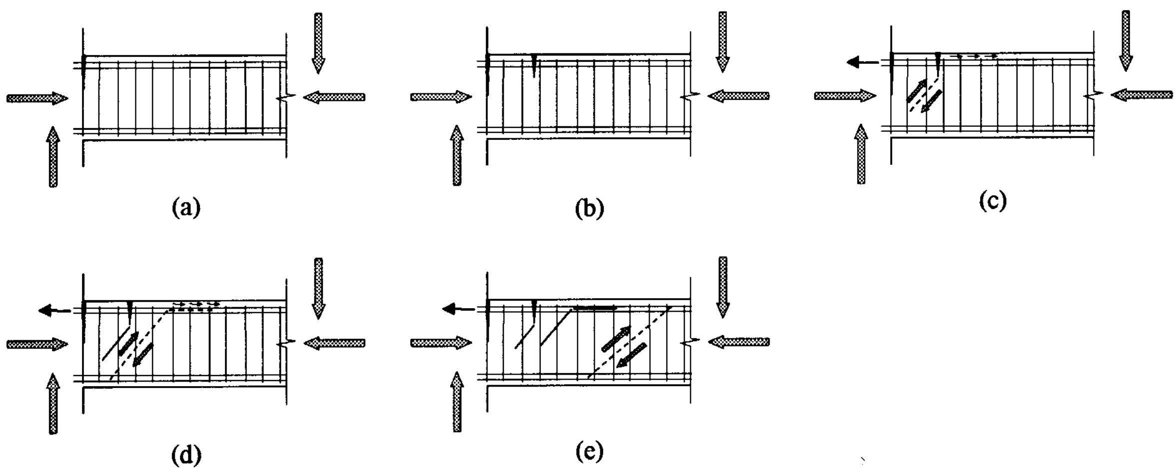


Figure 8- Process of failure

When acted upon by a large axial force, the RC columns with a large amount of ties will develop the bending cracks in Step 2 towards its ends whereas the columns with central reinforcement will not develop Step 4 shear cracks beyond the column neutral section.

Based on the above mentioned crack development scheme, we have developed a macro model. This model makes use of the truss theory as shown in the figure and consider the equilibrium. To the concrete failure model we have applied the shear transfer theory by Mattock and performed the analysis.

4.2 Analysis Results

Table 4 shows the analytical results and it can be observed that the analysis has shown a fair agreement with the experimental test results.

Table 4 – Analytical result

No.	Name of Specimen	Strength of Concrete (MPa)		Axial Load ratio	Tie Ratio (%)	Tensile Steel Ratio (%)	Central Reinforcement ratio (%)	Max. Shear Load(kN)	
		Compression	Tension					Experiment	Analisys
1	MB-1.44-0.85-0.2	22.1	2.2	0.2	0.85	1.44	-	83.6	73.5
2	MB-1.00-0.85-0.2	26.8	2.4	0.2	0.85	1	-	76.4	73.5
3	MB-1.44-1.28-0.2	36.1	3.1	0.2	1.28	1.44	-	102.9	101.92
4	MB-1.00-0.85-0.35	37.2	3.3	0.35	0.85	1	-	122.5	109.76
5	MB-1.00-0.85-0.4	23.1	2.2	0.4	0.85	1	-	78.4	77.42
6	MB-1.44-0.85-0.4	19.9	2.2	0.4	0.85	1.44	-	90.8	72.52
7	MB-1.44-1.28-0.4	34.1	3.1	0.4	1.28	1.44	-	100.0	106.82
14	CH3-1.00-0.85-0.2	26.8	2.4	0.2	0.85	1	1.39	77.4	82.32
15	CH3-1.00-0.85-0.4	23.1	2.2	0.4	0.85	1	1.39	79.4	76.44
16	CH5S-1.00-0.85-0.2	26.8	2.4	0.2	0.85	1	2.34	80.4	77.42
17	CH5S-1.00-0.85-0.35	37.2	3.3	0.35	0.85	1	2.34	118.6	135.24
18	CH5S-1.00-0.85-0.4	23.1	2.2	0.4	0.85	1	2.34	82.3	82.32
19	CH5S-1.00-1.28-0.4	25.0	2.6	0.4	1.28	1	2.34	101.9	100.94
23	CI5-1.00-0.85-0.2	22.1	2.2	0.2	0.85	1	2.34	77.7	75.46
24	CI5-1.00-0.85-0.4	19.9	2.2	0.4	0.85	1	2.34	84.3	85.26

5. CONCLUSIONS

In this paper, we have studied a seismic upgrade methodology for reinforced concrete columns through experiment and theoretical analysis. Based upon the results of this study, the following conclusions can be drawn.

(1) Concerning the ultimate capacity, there has been no noticeable difference between the RC columns strengthened with the central reinforcement and the conventional RC columns without central reinforcement. However, the RC columns with central reinforcement has shown superior ductility after reaching the ultimate capacity comparing the conventional RC columns.
(2) Flanged (H shaped) steel or steel tube with mortar infill better opt for the central reinforcing element.
(3) Regarding the effect of the anchor length, anchoring the central reinforcement up to the columns ends is more effective than anchoring the central reinforcement up to the beam ends.
(4) RC columns with central reinforcement can maintain up to 70% of its ultimate capacity at 4% story drift whereas conventional RC columns can only hold its 70% ultimate capacity at 3% story drift deformation.

As stated previously, the reinforced concrete columns in RC structures with little walls such as piloti are susceptible to damages during a strong earthquake even after increasing the column ties. Based on the results summarized above, it is believed that these columns can be strengthened by central reinforcement and with the central reinforcement, they will not collapse at the strong earthquakes since they can maintain their capacities even at 5% deformation.

6. REFERENCES

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