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Seismic Assessment of Non-Seismically Designed Reinforced Concrete Frames with Core Walls

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Abstract: This study examines the seismic performance of four non-seismically designed reinforced concrete structures with core walls and proposes proper rehabilitation methods. The structures were selected based on the height of the structures, occupancy levels and floor plans. The seismic assessment results were compared based on the performance of the structures before rehabilitation and after rehabilitation. The results indicate that most of the structures are unsatisfactory in terms of their failure to reach a level of *life safety*. Three types of rehabilitation methods, i.e., reinforcement walls, braces and columns, were applied to assess the seismic performance of the four case studies. The two-phase seismic assessments of the rehabilitated structures were conducted according to current provisions used in Korea.

Keywords: Core Wall, Reinforced Concrete (RC) Frame, Non-Seismically Designed, Seismic Assessment

Introduction

This study examines four cases of Reinforced Concrete (RC) frames that have core walls and were designed before 1988 without seismic considerations. The seismic performance of the selected structures was assessed according to existing design codes used in Korea. Recommendations regarding proper rehabilitation methods are offered based on the seismic assessment results. The study set selected for this study is a total of 320 building configurations. The configurations were chosen based on the height, occupancy or use and floor plan of the buildings.

RC frames with core walls were used in several of the structural components, including stair sections and elevator sections. Even if the RC frame was designed originally as a non-seismically designed structure, these core walls would still enhance the resistance to lateral loads and would affect the seismic performance of the structures. However, there is uncertainty with respect to the optimal location and size of the core wall. In order to evaluate these non-seismically designed structures in terms of their seismic performance, two structures with a core wall in the middle and two structures with a core wall on the side have been selected for this study. The results are compared before and after reinforcing the structures.

The literature provides several examples of seismic assessment research. Kang (2011) conducted a seismic assessment of a complex house with a bearing wall by using pushover analysis according to seismic design guidelines. Kim (2011) conducted nonlinear analysis of existing reinforced concrete structures and proposed an appropriate rehabilitation method using seismic design strength after evaluating the desired seismic capacity of the structures. Lee (2013) investigated seismic assessment and rehabilitation methods for mid-rise RC structures with core walls. The structures were rehabilitated using walls, columns and braces. Adebar and White (2002; Adebar, 2005; 2008) structures with core walls using response spectrum analysis.

Also, several countries, including Korea and the United States, have proposed seismic assessment guidelines for existing RC structures. In Korea (the focus of this study's case structures), two provisions are available for the seismic performance assessment of existing structures: The Korea Infrastructure Safety and Technology Corporation (KISTEC) (KISC, 2011) guidelines and the National Fire Management Agency (NEMA) (NEMA, 2012) guidelines. These Korean guidelines were developed based on the Federal Emergency Management Agency (FEMA) (FEMA,

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1997; 2000) and American Society of Civil Engineers (ASCE) guidelines in the United States (ASCE, 2003; 2006). The selected seismic assessment method used for this study is the KISTEC guidelines, which include time-intensive analyses compared to the other guidelines However, despite the time factor, the structural analysis and structural performance of individual members inherent of the (KISC, 2011) guidelines lead to precise results and, therefore, these guidelines have been selected for this study.

Seismic assessments were conducted for the selected structures before and after rehabilitation. The first evaluation is a preliminary seismic assessment and the second evaluation is a detailed seismic assessment. The seismic capacity of a structure might be computed conservatively in the first assessment based on the properties and proposed equations in the existing provisions. If the results from the first assessment do not meet the required seismic capacity, then the second seismic assessment might be recommended. For the detailed second seismic assessment, the selected structures are evaluated based on fundamental material information and detailed structural analysis results. If the performance does not meet the desired seismic performance in the second assessment, the implication is that the deficient member (s) in the structures require rehabilitation or further analysis, such as nonlinear analysis.

Loads are composed of a gravity load and an earthquake load. The structural behavior is categorized as either deformation-controlled action or force-controlled action. In this study, different load combinations were considered for each controlled action. The expected strength for deformation-controlled action, Q_{CE}, is defined as the mean value of the resistance of a component at the deformation level that is anticipated for a population of similar components. This parameter includes consideration of the variability in material strength as well as strain-hardening and plastic section development. The design strengths for force-controlled actions, Q_{CL}, are taken as the lower-bound strength, considering all the coexisting actions in the element (KISC, 2011).

Unless other procedures are specified in the standard, the procedures outlined in KBC 2009 to calculate nominal strength and mean strength were used in this study. For all cases, the strength reduction factor was taken as unity in the assessment of the existing structures. For the deformation-controlled action, the strength of the RC members was computed using the equations found in KBC 2009. To consider the difference between nominal strength and mean strength, the yield strength values of the reinforcement and concrete compressive strength were determined by multiplying each strength value by 1.2 (KISC, 2011).

Table 1-3 present the expected material properties, the shear stress values for columns with various failure modes and shear walls with respect to the location of the columns, respectively. The presented information was used to determine the strength values for each member of the RC structures.

The final assessment determines the Demand and Capacity Ratio (DCR) of the structures. Demand is the shear stress that is due to the seismic load. Capacity is computed as the summation of the shear resistance. Equation 1 shows the DCR values for a RC structure. Table 4 presents the performance level with respect to the DCR values:

$$DCR_i = \frac{S_{Ds} - W -_{\gamma i}}{C_i} \tag{1}$$

Where:
S_{DS} = The spectral response acceleration parameter at short periods
W = The total weight of the structure and
γ_i = The shear distribution coefficient.

Table 1. Expected or lower bound material properties (KISC, 2011)

Year strength	Before 1970 Lower bound	Before 1970 Average	1971-1988 Lower bound	1971-1988 Average
Concrete strength (f_{ck}, MPa)	13	15	15	18
Yield strength of reinforcement (f_y, MPa)	240	300	240	300
Year strength	1988-2000	After 2001		
Concrete strength (f_{ck}, MPa)	18	21	21	24
Yield strength of reinforcement (f_y, MPa)	300	375	300	375

Table 2. Average shear stress for columns (KISC, 2011)

Year classification of column		Construction year (MPa) Before 1970	1971-1987	1988-2000	After 2001
Shear failure v_{sc}	Short column h_o/D<2.0	1.17	1.23	1.34	1.41
	Intermediate column 2.0<h_o/D<6.0	0.86	0.9	0.98	1.03
Flexural failure v_{fc}	Long column $h_o/D \geq$ 6.0	0.46	0.47	0.52	0.53

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Table 3. Shear stress of shear wall with respect to condition of column (KISC, 2011)

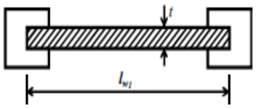
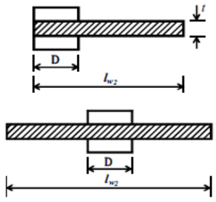
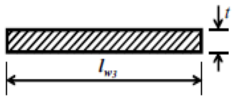
Section type	Cross-section	Cross-section, shear stress
Shear wall with columns on both sides		$A_{sw} = t \cdot l_{w1}$, $\upsilon_{sw} = 3.0$ MPa
Shear wall with a column on one side		$A_{sw} = t{\cdot}l_{w2}$, $\upsilon_{sw} = 2.0$ MPa
Shear wall without column		$A_{sw} = t{\cdot}l_{w3}$, $\upsilon_{sw} = 1.0$ MPa

Table 4. DCR values for performance level of rc structure (KISC, 2011)

DCR value	Performance level
DCR ≤ 0.5	Immediate Occupancy (IO)
0.5<DCR<0.75	Life Safety (LS)
0.75<DRC≤1.0	Collapse Prevention (CP)
1.0<DCR	Collapse

Seismic Assessment before Rehabilitation

The selected structures were built in 1986 and 1987. Table 5 presents basic information about these four structures, MC-1, MC-2, CC-1 and CC-2. The structures are identified according to the location of the core and the ratio of the core area to the total area. For example, MC-1 indicates that the core is located in the middle of the structure and is below 10% of the ratio of the core area to the total area. CC similarly indicates that the core is in the corner of the structure.

Figure 1 presents the floor plans for each of the structures. The structures were made with RC with core walls around the stairs and elevators.

Seismic Assessments of Selected Structures

First Seismic Assessment

In the first assessment of the four types of RC structures, the DCRs were used to determine seismic performance, as shown in Equation 2:

$$DCR_i = \frac{S_{Ds} - W - \gamma_i}{\sum V_i} \quad (2)$$

MC-1 is a 7-storey building constructed with a RC frame structure with a shear wall in the middle. This structure falls in the site classification of S_C and seismic Zone 1. Therefore, the spectral response acceleration parameter at short periods, S_{DS}, is 0.43. The weight of each storey is 2343.6 kN, which is the floor area multiplied by 10 kN/m^2. The demand values were computed using the seismic shear coefficient (γ_i) and spectral response acceleration parameter at short periods (S_{DS}). Table 6 shows the demand values for the MC-1 structure determined from Equation 2.

The shear stress values vary with respect to the aspect ratio of the columns and construction year. Most columns were classified as an intermediate column, except C1, C5, C13 and C16 in the MC-1 structure. MC-1 was constructed around 1987. The average shear strength values for the short, intermediate and long columns in MC-1 are 1.23 MPa, 0.74 and 0.47 MPa, respectively. The average shear stress values are 3MPa, 2MPa and 1 MPa for a shear wall with two columns, a shear wall with a column on one side and a shear wall without a column, respectively.

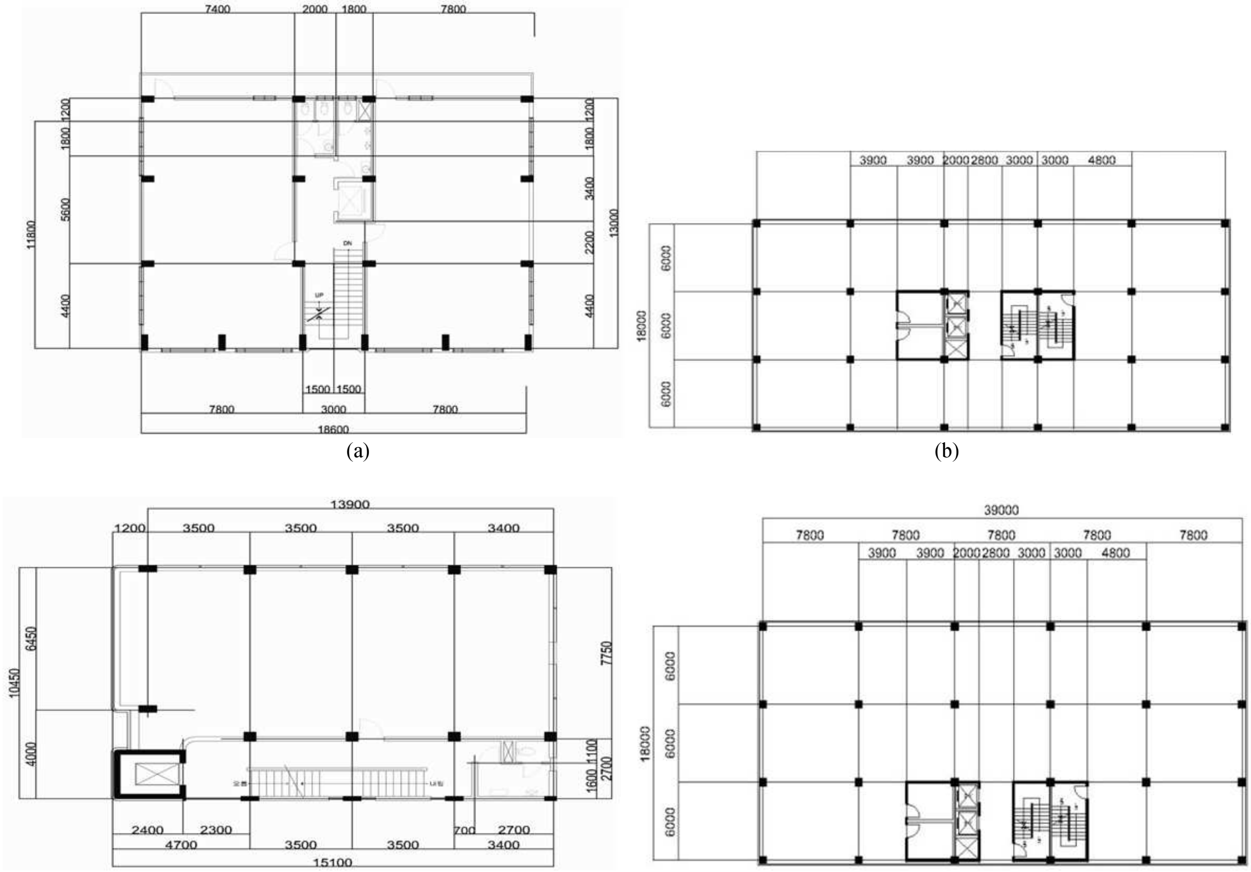
Table 7 presents first seismic assessment results in terms of the performance levels based on the DCRs for the MC-1 structure.

The highest DCR values for MC-1 are 0.98 for the y-direction and 0.9 for the x-direction for the first floor. These values indicate that the structure is at a level of *collapse prevention*.

In the same manner, the remaining structures (MC-2, CC-1 and CC-2) were evaluated for the first seismic assessment. Figure 2 shows the seismic performance levels for all four structures.

Figure 2 shows that the DCR value for CC-1 is 0.88 for both the x- and y-direction sat the level of *collapse prevention*. The DCR values for MC-2 and CC-2 are 0.55 for the x-direction at the level of *life safety*, but 0.81 for the y-direction at the level of *collapse prevention*. The DCR value for MC-1 with a core area of 2.4% is 0.9 for the x-direction at the level of *collapse prevention* and 0.98 for the y-direction. The DCR value for CC-1 with a core area of 5.6% is 0.88 for both the x-direction and y-direction at the level of *collapse prevention*.

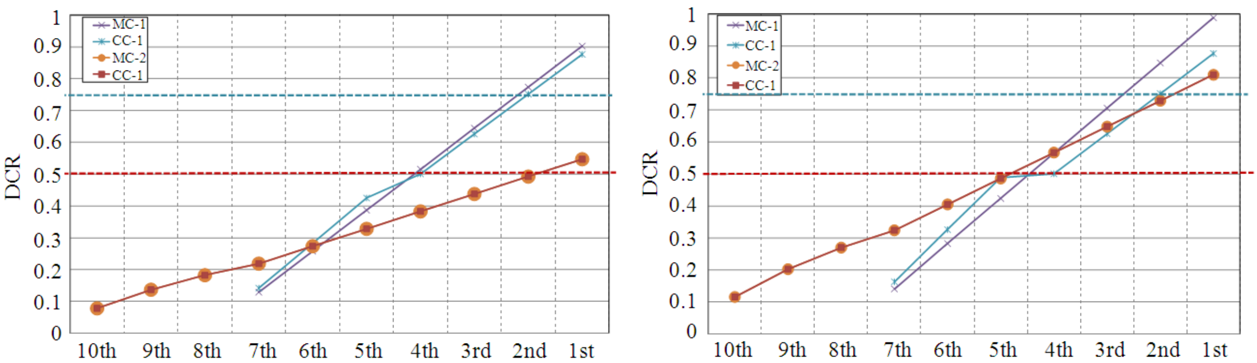
MC-2 and CC-2 have over 10% core area compared to the total areas of these two structures.



(a) (b)

(c) (d)

Fig. 1. Floor plans of selected structures, (a) Floor plan for MC-1, (b) Floor plan for MC-2, (c) Floor plan for CC-1, (d) Floor plan for CC-2



(a) (b)

Fig. 2. First seismic assessment results for each structure

Table 5. Selected structures

Structure ID	MC-1	MC-2	CC-1	CC-2
Gross floor area (m^2)	1640.52	7020	1024.1	7020
Number of storeys	7-storey	10-storey	7-storey	10-storey
Maximum height (m)	23.1	33.2	25.2	33.2
Occupancy or use	Facility	Office	Hospital	Office
Location of core	Middle	Middle	Corner	Corner
Ratio of core area to total area	2.40%	10%	5.60%	10%

Table 6. Demand values for MC-1

Storey	Height h_i, mm	Floor area (m^2)	Weight (w_i), (kN)	w_i*h_i	cumulated w_i*h_i	γ_i	Demand (kN)
7th	3300	234.36	2343.6	7733880	7733880	0.142	475.081
6th	3300	234.36	2343.6	7733880	15467760	0.285	950.162
5th	3300	234.36	2343.6	7733880	23201640	0.428	1425.24
4th	3300	234.36	2343.6	7733880	30935520	0.571	1900.32
3rd	3300	234.36	2343.6	7733880	38669400	0.714	2375.40
2nd	3300	234.36	2343.6	7733880	46403280	0.857	2850.48
1st	3300	234.36	2343.6	7733880	54137160	1.000	3325.56

Table 7. First seismic assessment results for MC-1

DCR for each floor	Direction	Demand, (*kN*)	Capacity, (*kN*)	DCR	Performance level
7th	X	475.0812	3686.49	0.12	IO
	Y	475.0812	3366.49	0.14	IO
6th	X	950.1624	3686.49	0.25	IO
	Y	950.1624	3366.49	0.28	IO
5th	X	1425.244	3686.49	0.38	IO
	Y	1425.244	3366.49	0.42	IO
4th	X	1900.325	3686.49	0.51	IO
	Y	1900.325	3366.49	0.56	IO
3rd	X	2375.406	3686.49	0.64	LS
	Y	2375.406	3366.49	0.70	LS
2nd	X	2850.487	3686.49	0.77	LS
	Y	2850.487	3366.49	0.84	LS
1st	X	3325.568	3686.49	0.90	CP
	Y	3325.568	3366.49	0.98	CP

Note: IO is immediate occupancy; LS is life safety and CP is collapse prevention

The DCR values for those structures are 0.55 for the x-direction at the level of *life safety*, but 0.81 for the y-direction at the level of *collapse prevention*. These results are due to the non-seismically designed structure and the location of the cores.

The results of the first seismic assessment indicate that the larger the core area, the better the seismic performance with an in crease in DCR values. However, when the core area is less than 10% of the total area, the effects are negligible. Most of the structures built between 1970 and into the 1980 s have a small core area. Therefore, rehabilitation is required for those structures.

If the structure is large enough to need an elevator, the core areas, such as the E/V and emergency stairs, are better protected and the core wall plays a role in the structure's internal seismic resistance. The performance levels of MC-2 and CC-2 with 10% core areas likewise depend on the locations of the cores. However, according to the existing provision specifications, the walls in these two structures do not meet the desired performance level (*life safety*) due to the skewed location of the walls. The first seismic assessment, as a preliminary assessment, disqualified the desired performance level (i.e., level of *life safety*), so the first assessment was followed by a second assessment. In fact, none of the selected structures met the minimum desired performance (*life safety*). In addition, the core location in the structure was not considered in the first assessment. Therefore further assessment was required.

Second Seismic Assessment

The second assessment was conducted based on the results from the structural analysis for each member. The DCR values for each member were used to determine the performance level. Ductility coefficients (*m*) were used to determine the structural performance level. Table 8 shows the DCR values for the vertical members in the MC-1 structure.

Table 9 shows the DCR values with respect to the desired performance level and the axially distributed load ratios.

In the second assessment, when each member satisfies the performance level based on the DCR values that correspond to each performance level and when each member carries over 80% axial load, the structure is considered to qualify for the desired performance level. In the same manner, the remaining structures were assessed and the results are presented in Fig. 3.

MC-1 and CC-1 are 7-storey buildings with core areas of 2.4 and 5.6%, respectively. The second seismic performance assessment results for each of these structures indicate that the performance levels are *collapse prevention* and *near collapse*, respectively. These results are the same as or worse. In particular, CC-1 has a core located on one side and the large core area of CC-1 was adversely affected in terms of its seismic performance. The minimum performance level, *life safety*, was not attained for these two structures, even in the second assessment.

Table 8. Demand and capacity for MC-1

Type	ID	Axial load P (*kN*)	Demand M_{uy}(*kN-m*)	M_{uz}(*kN-m*)	V_u (*kN*)	Capacity M_{ey}(*kN-m*)	M_{ez}(*kN-m*)	V_e (*kN*)
Column	C31	1543.900	96.94	30.76	29.38	43.07	36.496	245.46
	C32	1927.000	79.53	24.53	24.20	51.69	43.795	391.30
	C33	2773.800	131.10	55.87	40.85	38.03	15.663	244.91
	C34	601.4500	8.21	8.81	2.49	51.97	94.037	280.99
	C35	854.680	76.35	110.68	-23.14	86.24	205.420	378.63
	C36	1335.300	44.66	110.42	-13.53	53.12	206.970	378.30
	C37	1322.700	14.58	91.41	4.42	15.12	165.190	378.55
	C38	1343.400	40.89	91.67	12.39	78.25	154.300	375.77
	C39	1514.100	18.98	22.44	-5.75	122.10	98.856	412.08
	C40	1673.600	62.74	19.24	-19.01	134.30	108.740	398.84
	C41	689.750	106.76	38.78	32.35	205.70	92.495	381.94
	C42	1470.800	66.44	6.48	20.13	155.20	22.405	373.68
	C43	41.519	61.64	61.56	-18.68	74.75	120.970	309.32
	C44	439.740	13.39	58.48	-4.06	121.50	93.898	320.64
	C45	644.170	8.62	73.39	2.61	88.63	125.960	325.59
	C46	891.100	-56.53	46.78	17.13	114.80	83.999	337.70
Wall	W1	2402.200	732.25	0.00	314.43	300.30	0.000	223.84
	W2	2311.800	4680.30	0.00	2116.50	491.90	0.000	318.00
	W3	2829.600	1313.90	0.00	361.83	289.90	0.000	223.84

Table 9. Desired performance assessment for MC-1

ID	DCR level IO	LS	CP	NC	Desired performance LS	CP	NC	Axial load P, (kN)	Axially distributed load ratio
C31	1.77	1.77	1.18	0.11	NG	NG	OK	-1811.29	LS: 70%
C32	0.72	0.72	0.48	0.07	OK	OK	OK	-2083.48	(NG)
C33	1.80	1.80	1.20	0.02	NG	NG	OK	-1154.94	
C34	1.05	1.05	0.70	0.63	NG	OK	OK	-1668.06	CP: 77%
C35	0.64	0.64	0.43	0.23	OK	OK	OK	-1599.57	(NG)
C36	0.70	0.70	0.47	0.23	OK	OK	OK	-1599.96	
C37	0.62	0.62	0.41	0.14	OK	OK	OK	-1560.09	NP: 89%
C38	0.66	0.66	0.44	0.14	OK	OK	OK	-1582.66	(OK)
C39	0.59	0.59	0.39	0.38	OK	OK	OK	-1336.83	
C40	0.58	0.58	0.39	0.37	OK	OK	OK	-1287.14	
C41	0.65	0.65	0.43	0.05	OK	OK	OK	-1365.54	
C42	0.64	0.64	0.42	0.02	OK	OK	OK	-1351.20	
C43	0.59	0.59	0.40	0.33	OK	OK	OK	-933.65	
C44	0.52	0.52	0.35	0.24	OK	OK	OK	-927.45	
C45	0.68	0.68	0.46	0.15	OK	OK	OK	-937.78	
C46	0.65	0.65	0.43	0.10	OK	OK	OK	-930.34	
W1	5.73	4.77	3.58	3.01	NG	NG	NG	-859.03	
W2	4.94	4.12	3.09	3.10	NG	NG	NG	-970.75	
W3	6.45	5.38	4.03	3.00	NG	NG	NG	-861.83	

Note: OK indicates okay/acceptable performance level; NG indicates not good.
Note: IO is immediate occupancy; LS is life safety, CP is collapse prevention and NC is near collapse

Table 10. Reinforcing location

Rehabilitation method	Wall	Brace	Column
MC-1	Left bottom	Left bottom	1st floor
CC-1	Right top	Right top	1st floor
MC-2	All of exterior	All of exterior	1st floor
CC-2	Top exterior	Middle	1st floor

Therefore, further analysis, i.e., nonlinear analysis as a third assessment, may be required according to the specifications. However, this study does not include a third assessment.

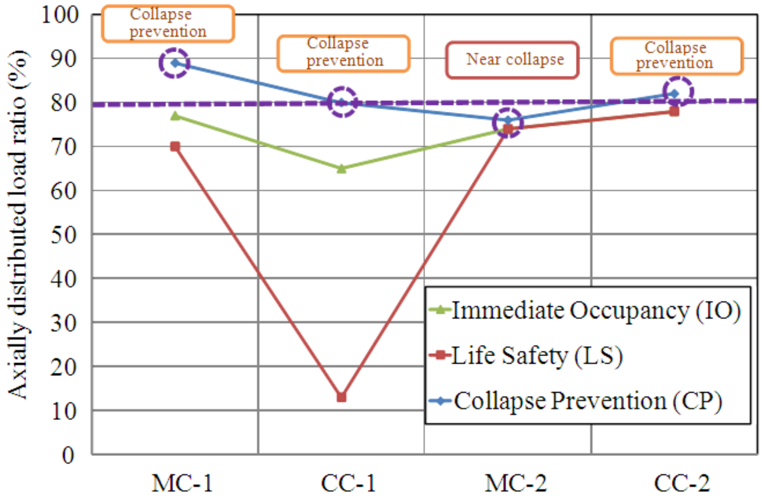


Fig. 3. Second seismic assessment results for each structure

Rehabilitation Methods

For the MC-1 structure, Fig. 4 shows three possible reinforcing locations for walls. The shear strength at the bottom of the structure and the displacement for each storey were used to determine the reinforcing location. The results of the first and second assessments indicate that the x-directional capacity is considerably less than the y-direction capacity, so a RC shear wall with a thickness of 200 mm was installed at the bottom left corner, as seen in Fig. 4.

As a second possible reinforcing method, steel braces (H-250×250×9×14) were installed at the same locations with a shear wall. Figure 5 shows three reinforcing locations for this second reinforcing method.

As a third possible reinforcing method, Fig. 6 shows three different reinforced locations for columns, including exterior columns, highly compressed columns and columns in the first floor. The cross-sections of the selected columns are increased by 20%. Only the last case, i.e., where all the columns in the first floor are reinforced, attains the desired performance level.

For the MC-2 and CC-2 structures, the same methods were used to determine the reinforcing locations for the three types of rehabilitation. The final reinforcing methods for all four structures are summarized in Table 10.

Seismic Assessment after Rehabilitation

First Seismic Assessment

Figure 7 presents comparisons of the first seismic assessment results before rehabilitation and after reinforcement in the x-and y-directions using walls, braces and columns. The first assessment was conducted without reinforcing braces, so the first assessment results for MC-2 and CC-2 in terms of reinforcing braces are not included. For MC-1, the performance in the x-direction is seen to be not as good as in the y-direction. However, both reinforcing methods are shown to attain the desired performance level of *life safety*. For the x-direction, the performance was improved to the *life safety* level using the reinforcing column method and to the *immediate occupancy* level using the reinforcing wall method. Both reinforcing methods provided sufficient strengthening. For the case of CC-1, the seismic performance in both the x- and y-directions is similar. With the use of a reinforcing column and wall, the performance was improved to the level of *life safety*. The performance of MC-2 and CC-2 improved from *life safety* to *immediate occupancy* for the x-direction.

The reinforcing column method shows better results than the reinforcing wall method for the y-direction.

Second Seismic Assessment

The goal of seismic rehabilitation is to attain the level of *life safety* in the second assessment. The second assessment results for each structures are presented with respect to each reinforcing method. Table 11 to 13 shows the second assessment results for MC-1 with respect to each reinforcing method, i.e., wall, brace and column, respectively.

The performance levels reached the level of *immediate occupancy* with the rehabilitation column method and the level of *life safety* with the rehabilitation brace and wall. Table 13 shows that the DCR values decreased for members that have an axially distributed load. If the current rehabilitation method is not satisfactory for the desired performance level, the column with an increase of cross-section reduces the DCR values of the walls, which decreases the axially distributed load. Overall, the most effective rehabilitation method is column rehabilitation.

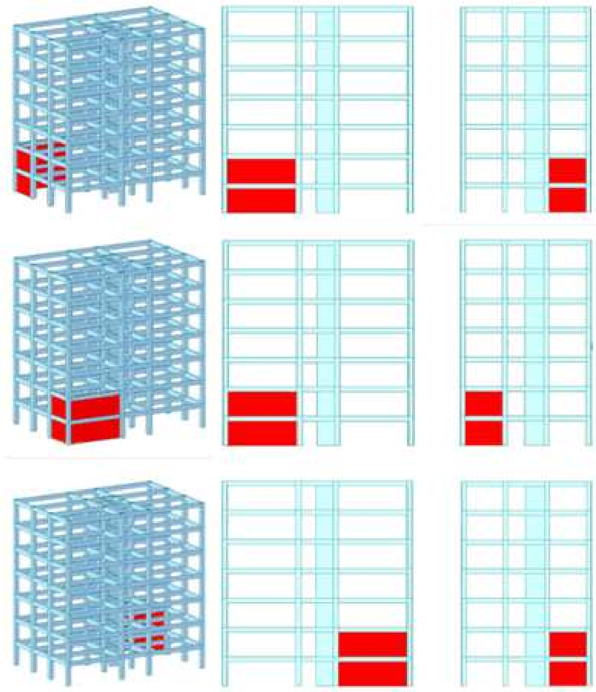


Fig. 4. MC-1 with respect to location of rehabilitation wall

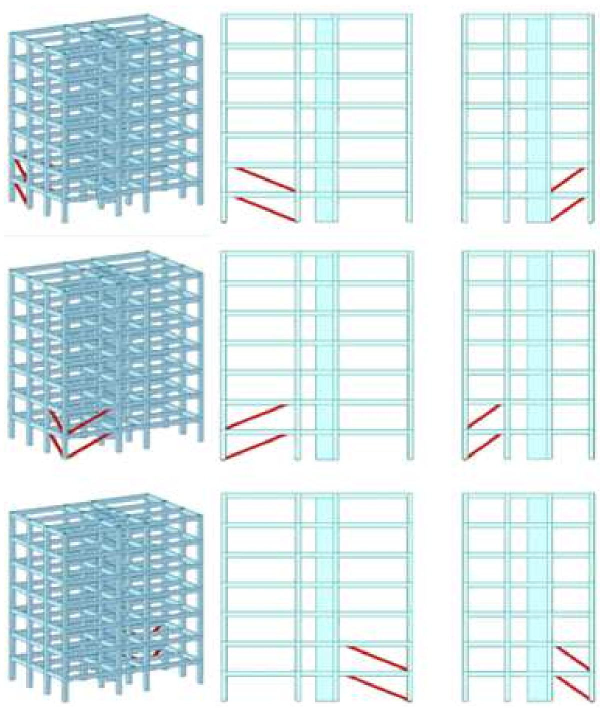


Fig. 5. MC-1 with respect to location of rehabilitation braces

Figure 8 presents comparisons of the desired performance levels for each rehabilitation method. Figure 8a shows that MC-1 is at a level of *collapse prevention*. All three reinforcing methods reach the desired performance level of *life safety*.

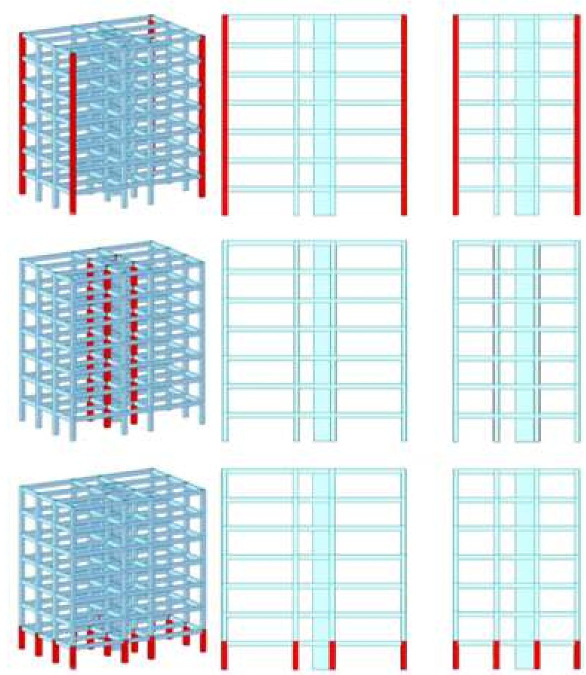


Fig. 6. MC-1 with respect to the location of rehabilitation columns

Figure 8b indicates that the seismic assessment level for MC-2 is *near collapse* before rehabilitation, although the desired performance level (*life safety*) is obtained regardless of the rehabilitation method. The seismic performance with the rehabilitation wall was at the levels of *immediate occupancy* and *life safety* and the seismic performance with the rehabilitation column and brace was at levels of *life safety*. The seismic performance improved in the order of rehabilitation wall>rehabilitation column> rehabilitation brace, because performance improved with an increase in the axially distributed load ratio. The use of the brace and column for rehabilitation reduced the axial load carried by the existing column member. Specifically, the rehabilitation wall show edsuperior performance by significantly reducing the axial load. For the case of a structure with a relatively large core wall, the wall carries more axial load than the column, which improves the seismic performance level significantly. In the case of MC-2, the core wall is located in the center of the structure so that there is no concerns, unless the newly installed wall is located in corner. The rehabilitation wall is the most effective method for this structure.

Figure 8c shows that CC-1 is at a level of *collapse prevention* prior to rehabilitation. The performance after rehabilitation meets the desired level of *life safety* regardless of the rehabilitation method. The axial load distributions are similar for the three rehabilitation methods. However, the DCR values in Table 11 to 13 show a radical decrease. Specifically, the performance of the wall improved with the reduction of the axial load distribution after rehabilitation wall reinforcement.

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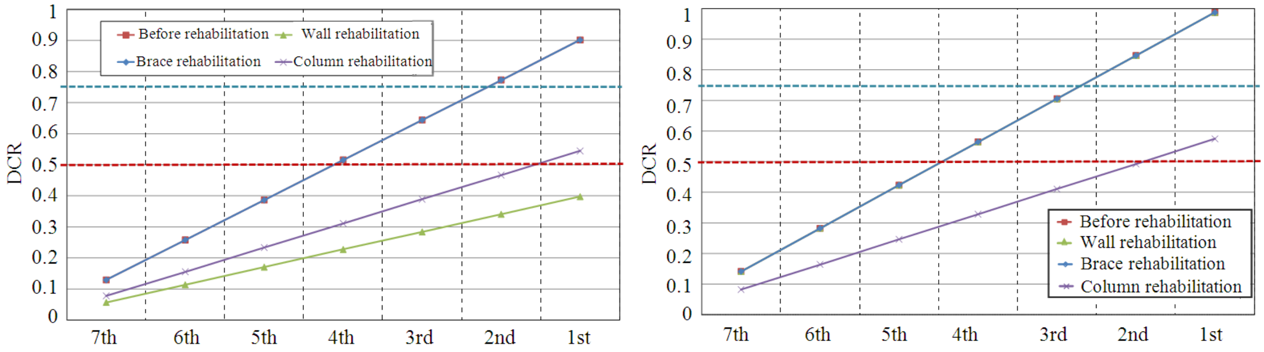

Fig. 7. Comparison of seismic assessment results before and after reinforcing

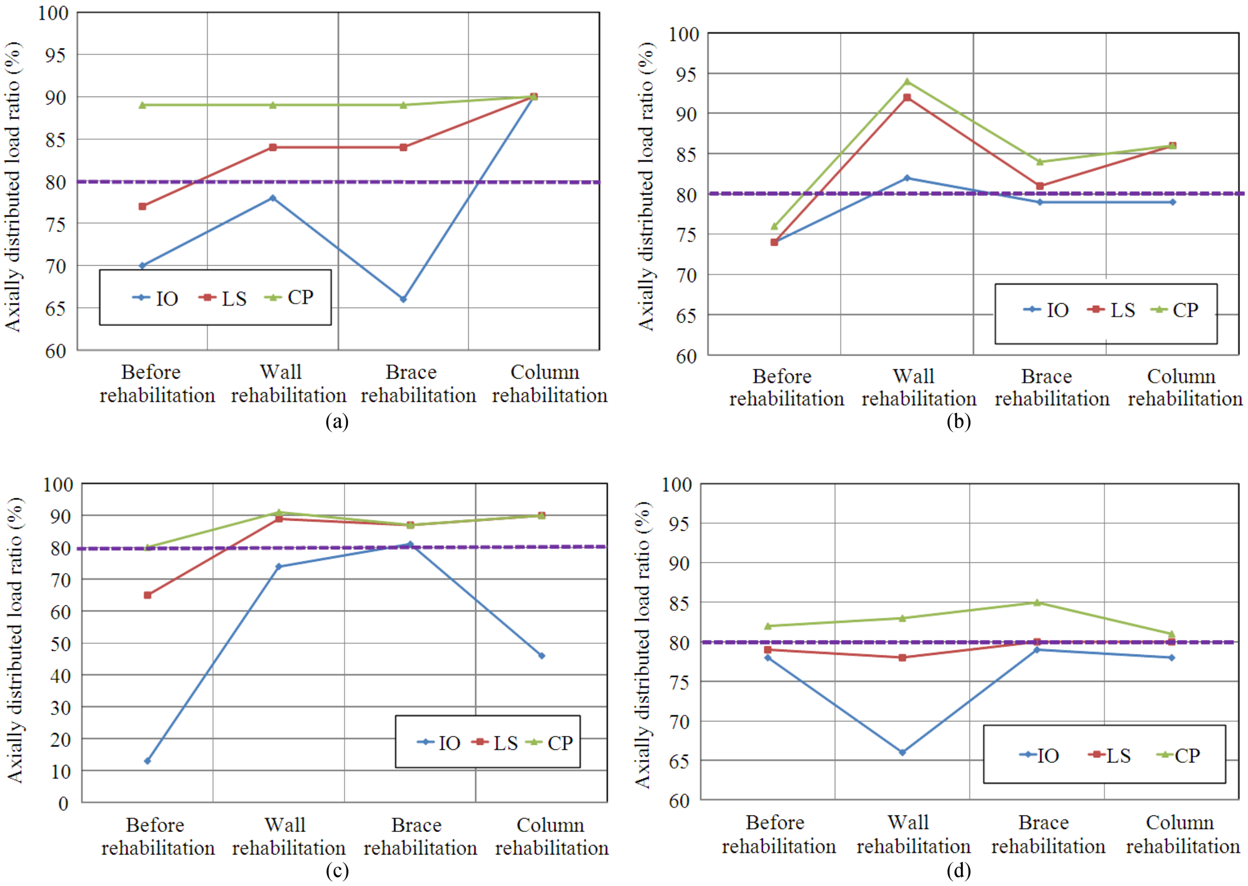

Fig. 8. Performance level for each structure, (a) MC-1, (b) MC-2, (c) CC-1, (d) CC-2

Overall, the rehabilitation brace is the most effective form of reinforcement because the installed braces take over the axial loads of the adjacent column. In short, consideration of the DCR of the column might be effective when installing braces.

Figure 8d shows that CC-2 was at a level of *collapse prevention* prior to rehabilitation. However, the rehabilitation brace and column methods improved the performance level to *life safety*. Similar to MC-2, the core area of CC-2 is 10% of the total area and is located on the lower part on the floor plan. To make the section symmetrical, the rehabilitation area was located in the upper part of the floor plan. Therefore, the axial load distribution changed.

The distribution factor of the axial load of the existing wall decreased and the performance decreased considerably to the *immediate occupancy* level due to the increase of the distribution factor on the existing column.

Table 11. Second seismic assessment of MC-1 after reinforcement with rehabilitation wall

ID	DCR values IO	LS	CP	NC	Desired performance level LS	CP	NC	Axial load P, (kN)	Axially distributed load ratio
C31	0.73	0.73	0.49	0.10	OK	OK	OK	-1790	LS: 78%
C32	0.60	0.60	0.40	0.38	OK	OK	OK	-2025	(NG)
C33	1.81	1.81	1.20	0.56	NG	NG	OK	-1191	
C34	1.09	1.09	0.73	0.63	NG	OK	OK	-1656	CP: 84%
C35	0.58	0.58	0.38	0.27	OK	OK	OK	-1598	(OK)
C36	0.54	0.54	0.36	0.39	OK	OK	OK	-1551	
C37	0.55	0.55	0.37	0.37	OK	OK	OK	-1565	V: 89%
C38	0.54	0.54	0.36	0.37	OK	OK	OK	-1586	(OK)
C39	0.56	0.56	0.38	0.36	OK	OK	OK	-1337	
C40	0.59	0.59	0.39	0.36	OK	OK	OK	-1301	
C41	0.61	0.61	0.41	0.08	OK	OK	OK	-627	
C42	0.55	0.55	0.37	0.33	OK	OK	OK	-1323	
C43	0.55	0.55	0.37	0.30	OK	OK	OK	-934	
C44	0.54	0.54	0.36	0.28	OK	OK	OK	-937	
C45	0.92	0.92	0.62	0.20	OK	OK	OK	-394	
C46	0.54	0.54	0.36	0.27	OK	OK	OK	-928	
W1	3.69	3.07	2.30	1.87	NG	NG	NG	-907	
W2	4.91	4.09	3.07	3.04	NG	NG	NG	-1025	
W3	4.79	3.99	3.00	1.17	NG	NG	NG	-875	

Table 12. Second seismic assessment of MC-1 after reinforcement with rehabilitation brace

ID	DCR values IO	LS	CP	NC	Desired performance level LS	CP	NC	Axial load P, (kN)	Axially distributed load ratio
C31	1.08	1.08	0.72	0.02	NG	OK	OK	-1791	LS: 66%
C32	0.65	0.65	0.43	0.39	OK	OK	OK	-2075	(NG)
C33	1.8	1.8	1.2	0.56	NG	NG	OK	-1181	
C34	1.06	1.06	0.71	0.63	NG	OK	OK	-1675	CP: 84%
C35	0.59	0.59	0.39	0.44	OK	OK	OK	-1597	(OK)
C36	0.58	0.58	0.39	0.44	OK	OK	OK	-1601	
C37	0.53	0.53	0.35	0.36	OK	OK	OK	-1563	V: 89%
C38	0.53	0.53	0.35	0.34	OK	OK	OK	-1586	(OK)
C39	0.58	0.58	0.38	0.37	OK	OK	OK	-1332	
C40	0.58	0.58	0.39	0.36	OK	OK	OK	-1290	
C41	0.85	0.85	0.57	0.03	OK	OK	OK	-1325	
C42	0.56	0.56	0.37	0.33	OK	OK	OK	-1355	
C43	0.56	0.56	0.37	0.34	OK	OK	OK	-925	
C44	0.55	0.55	0.37	0.27	OK	OK	OK	-935	
C45	1.32	1.32	0.88	0.03	NG	OK	OK	-1005	
C46	0.53	0.53	0.35	0.25	OK	OK	OK	-928	
W1	4.46	3.72	2.79	2.32	NG	NG	NG	-853	
W2	4.91	4.09	3.07	3.05	NG	NG	NG	-995	
W3	4.75	3.96	2.97	1.65	NG	NG	NG	-837	

Table 13. Second seismic assessment of MC-1 after reinforcement with rehabilitation column

ID	DCR values IO	LS	CP	NC	Desired performance level LS	CP	NC	Axial load P, (kN)	Axially distributed load ratio
C31	0.66	0.66	0.44	0.22	OK	OK	OK	-1867	LS: 90%
C32	0.43	0.43	0.29	0.09	OK	OK	OK	-2109	(OK)
C33	0.69	0.69	0.46	0.04	OK	OK	OK	-1363	
C34	0.52	0.52	0.35	0.08	OK	OK	OK	-1730	CP: 90%
C35	0.54	0.54	0.36	0.37	OK	OK	OK	-1615	(OK)
C36	0.57	0.57	0.38	0.34	OK	OK	OK	-1615	
C37	0.51	0.51	0.34	0.20	OK	OK	OK	-1579	V: 90%
C38	0.55	0.55	0.37	0.21	OK	OK	OK	-1600	(OK)
C39	0.46	0.46	0.30	0.79	OK	OK	OK	-1347	
C40	0.46	0.46	0.31	0.78	OK	OK	OK	-1303	
C41	0.56	0.56	0.37	0.14	OK	OK	OK	-1373	
C42	0.57	0.57	0.38	0.01	OK	OK	OK	-1361	
C43	0.44	0.44	0.30	0.32	OK	OK	OK	-931	
C44	0.42	0.42	0.28	0.17	OK	OK	OK	-925	
C45	0.52	0.52	0.35	0.23	OK	OK	OK	-936	
C46	0.50	0.50	0.33	0.13	OK	OK	OK	-930	
W1	5.30	4.42	3.31	2.68	NG	NG	NG	-733	
W2	4.32	3.60	2.70	2.75	NG	NG	NG	-838	
W3	5.34	4.45	3.34	1.87	NG	NG	NG	-810	

Table 14. Summary of second seismic assessment

Before rehabilitation			Brace (%)		Column (%)		Wall (%)	
MC-1	CP	LS 70	LS	LS 66	IO	LS 90	LS	LS 78
		CP 77		CP 84		CP 90		CP 84
		NC 89		NC 89		NC 90		NC 89
MC-2	NC	LS 74	LS	LS 79	LS	LS 79	IO	LS 82
		CP 74		CP 81		CP 86		CP 92
		NC 76		NC 84		NC 86		NC 94
CC-1	CP	LS 13	IO	LS 81	LS	LS 46	LS	LS 74
		CP 65		CP 87		CP 90		CP 89
		NC 80		NC 87		NC 90		NC 91
CC-2	CP	LS 78	LS	LS 79	LS	LS 78	CP	LS 66
		CP 79		CP 80		CP 80		CP 78
		NC 82		NC 85		NC 81		NC 83

Table 14 presents the overall results of the second seismic assessment after rehabilitation. If the desired capacity of mid-sized structures with RC frames with core walls is the level of *life safety*, then the three proposed rehabilitation methods (i.e., braces, columns, walls) all are effective.

However, the axial load distribution for the reinforcing wall could change in the case of a relatively large core located on the side of the structure. If the desired performance level is *immediate occupancy*, then the reinforcing column for MC-1, reinforcing brace for CC-1 and reinforcing wall for CC-2 are appropriate reinforcements, except for CC-2, which is not suitable for the proposed reinforcing methods.

Conclusion

This study examined the seismic assessment of non-seismically designed RC structures in terms of the deterioration of the structure with age. After rehabilitation reinforcements using bearing walls, braces and columns, seismic assessments of the RC structures were conducted. The following conclusions are drawn based on the results of this study.

Most non-seismically designed RC frame structures with core walls in Korea fail the *life safety* level in terms of seismic performance. Most structures perform poorly and are at the *collapse prevention* and/or *collapse* level.

To enhance the seismic performance of these structures, reinforcing methods, such as installing braces, reinforcing walls and adding columns could be effective in increasing the seismic capacity of the structure. The effectiveness of each reinforcing method depends on the size of the structure, its floor plan, constructability and cost evaluation.

In the case of a core wall in the middle of a structure, the reinforced column is a good rehabilitation method for a small area of the core wall, whereas the reinforced wall is more efficient for a large area of core wall.

In the case of a core wall located at the side of a structure, the reinforced brace is the most efficient rehabilitation method to enhance overall seismic performance.

With regard to the reinforcement column as a rehabilitation method, the livability of the structure should be considered to determine the size of the cross-section and then reinforcement in the lower elevation of the structure is more economical than in the higher elevation.

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Author's Contributions

All authors equally contributed in this work.

Ethics

This article is original and contains unpublished materials. The corresponding author confirms that all of the other authors have read and approved the manuscript and no ethical issues involved.

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