Seismic Performance Of Flat Plate System With Shear Reinforcements

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SUMMARY
In general, the flat plate system is designed as a building frame system or dual system with a shear wall, which has not yet been verified. In this study, the results of experimental study about three isolated interior flat slab-column connections were applied to input data of slab-column connections for non-linear pushover analysis to investigate the system level seismic capacity for 45 shear-reinforced flat plate systems. And the overstrength factor and a response modification factor are used as major parameters to define the seismic capacity of the system, both of which are design factors of the seismic resistance system in the IBC 2012(ICC, 2012) as an index. Analysis results showed that the flat plate system reinforced with shear band showed the efficiency of an RC intermediate moment resistance frame except for the 5-story case.

Keywords: flat plate system, effective response modification factor, shear reinforcement

1. GENERAL INSTRUCTIONS

As reinforced concrete flat plate structures do not contain beams and are simple in their construction, they are an effective way of reducing story height, costs and framework construction times compared with general frame systems. They also have a flexible unit plan. So the use of flat plate systems is increasing. However, the mechanism of plastic hinge propagation isn't as well developed due to its own limitations. Therefore, the system connections are particularly vulnerable to punching shear. And the punching failure of slab-column connection can lead to the collapse of the entire structure.

For this reason, slab-column connections must be reinforced in order to prevent the punching failure. And Stirrups and studs are widely used to reinforce vulnerable connections.

If shear reinforcement measures are applied, not only does the punching shear resistance increase, but the unbalanced moment resistance and energy absorbing capacity in the connections also increase. Through an experimental study focusing on the slab-column connections when three types of shear reinforcement measures are applied; stirrups, shear studs and shear bands, we found that flexural failure occurred due to an unbalanced moment, when the connections reached the maximum unbalanced moment strength before reached maximum shear strength. Based on these findings, a study on how shear reinforcement can increase the lateral displacement control capacity needed to be done at a systematic level.

In this study, the seismic capacity was investigated on a systematic level for 45 shear-reinforced flat plate systems, the slab-column connections of which were modeled based up on the results from the previous experiment. The overstrength factor and the response modification factor were used as major parameters to define the seismic capacity of the system, both of which are design factors of the seismic resistance system in the IBC 2012(ICC, 2012), as an index.

2. BACKGROUND

2.1. Eccentric shear transfer model

According to the “Eccentric shear transfer model” in ACI 318-08(ACI Committee, 2008), the shear force on connections rises as the acting unbalance moment increases, demonstrating the interrelationship between them which are delivery loads of the structural system. On the other hand, it is assumed that the unbalanced moment strength does not affect the punching shear strength in terms of resistance capacity, since the punching shearing strength that resists shear force and the unbalanced moment strength that resists the unbalanced moment are designed independently in the design criteria. However, the test results under the transverse or gravity load show that the punching shear and unbalanced moment are interrelated in terms of the resistance strength of the member that resists them as well as the acting load.

In Table 2.1, the results of experiments carried out on the transverse load of two-way slabs that have interior connections are examined. Row 19 of Table 2.1 shows the shear strength ratio at the final failure. The maximum shearing force is significantly less than the maximum punching shear strength required in most design criteria, with the exception of a case where the gravity load ratio on punching shear strength is high (See Row (5) in Table 2.1), and where there is no shear reinforcement. This result suggests that flexural failure might occur due to an unbalanced moment before the connections reach maximum shear strength, even if the correct shear reinforcements are installed.

Table 2.1. Published Tests Of Interior Slab-column Connections Subjected To V And M

(1)	(2)	(3)	(4)	(5)	(6)	(7)[*1]	(8)[*2]	(9)	(10)[*3]
Source	Label	Shear Reinforce-ment	f_{ck} MPa	$\frac{V_g}{V_c}$	M_{test}^{SR} (M_{test}) kN.m	V_{test}^{SR} (V_{test}) kN	V_n^{SR} (V_n) kN	$\frac{V_{test}^{SR}}{V_{test}}(\frac{V_{test}}{V_{test}})$	Final failure mode
Islam & Park (1976)	3C	None	29.7	0.23	(35.8)	(177)	(160)	(1.00)	P
	6CS	Stirrups	28.2	0.24	38.4	188	234	1.06	F
	7CS	Stirrups	29.7	0.24	41.7	202	240	1.14	F
	8CS	Stirrups	22.1	0.27	34.9	174	207	0.98	F
Robertson et al. (2002)	1C	None	35.4	0.21	(58.3)	(254)	(260)	(1.00)	F/P
	2CS	Stirrups	31.4	0.22	68.5	288	367	1.13	F
	3SL	Stirrups	43.4	0.17	71.0	291	432	1.15	F
	4HS	Studs	38.2	0.20	67.9	286	405	1.13	F
Elgabry & Ghali (1987)	1	None	35.0	0.45	(130)	(566)	(335)	(1.00)	P
	2	Studs	33.7	0.46	162	668	493	1.18	F
	3	Studs	39.0	0.85	142	754	530	1.33	F/P
	4	Studs	40.8	0.83	150	780	542	1.38	F/P
	5	Studs	45.6	1.18	105	786	573	1.39	F/P
Hawkins et al. (1989)	9.6AH	None	30.7	0.51	(97.9)	(459)	(369)	(1.00)	P
	9.6EH.34	Stirrups	25.5	0.59	99.8	477	505	1.04	F
	9.6EH.48	Stirrups	25.8	0.67	86.2	466	507	1.02	F
	9.6AL	None	28.9	0.72	(34.6)	(353)	(358)	(1.00)	F/P
	9.6EL.34	Stirrups	23.4	0.90	33.3	383	483	1.08	F
	9.6EL.56	Stirrups	28.5	0.97	43.1	465	533	1.32	F
	14AH	None	30.3	0.58	(100.2)	(486)	(354)	(1.00)	P
	14EH.49	Stirrups	25.1	0.70	117.9	557	483	1.15	F
	14AL	None	27.0	0.95	(43.4)	(441)	(334)	(1.00)	P
	14EL.49	Stirrups	26.9	1.06	44.4	479	500	1.08	F
Kang & Wallace (2008)	C0	None	38.6	0.30	(103)	(438)	(414)	(1.00)	P
	PS2.5	Thin plate	35.1	0.32	109	456	592	1.04	F

		stirrups							
	PS3.5	Thin plate stirrups	35.1	0.32	106	447	592	1.02	F
	HS2.5	Studs	35.1	0.32	104	441	592	1.01	F
Previous study (2012) (Song, J. K., 2012)	RC1	None	38.7	0.43	(81.1)	(390)	(361)	(1.00)	P
	SR1	Stirrups	38.7	0.43	101.4	449	542	1.15	F
	SR2	Studs	38.7	0.43	81.4	391	542	1.00	F
	SR3	Bands	38.7	0.43	99.2	443	542	1.14	F

*1 : $V_{test} = V_g + \frac{\gamma_v A_c c}{J_c} M_{test}$, $V_{test}^{SR} = V_g + \frac{\gamma_v A_c c}{J_c} M_{test}^{SR}$ *2 : $V_n = \frac{1}{3}\sqrt{f_{ck}} b_0 d$, $V_n^{SR} = \frac{1}{2}\sqrt{f_{ck}} b_0 d$

*3 : F - flexural failure, P – punching failure

2.2. Response modification factor

Since response modification factor was initially introduced in the ATC 3-06 report(ATC, 1978), much research has been carried out, and an evaluation equation for response modification factor was proposed in ATC-19(ATC, 1995), based on these research results. The factors of the equation can be used in quantifying the seismic performance of structures.

The flat plate structure is an undefined seismic resistance system in the design code. If it is demonstrated that the seismic performance of a flat plate structure is similar to that of the seismic force–resisting systems defined in the design code, through analytical and experimental study, the seismic performance factors(response modification factor (R); system overstrength factor(Ω_0); deflection amplification factor (C_d)) are applicable to design flat plate structure.

In Table 2.2, Seismic force–resisting systems and their values which can be applied for flat plate structure and is specified in Table 12.2-1 of ASCE/SEI 7-10(ASCE/SEI, 2010) are shown.

Table 2.2 Design Coefficients And Factors For Seismic Force-resisting Systems

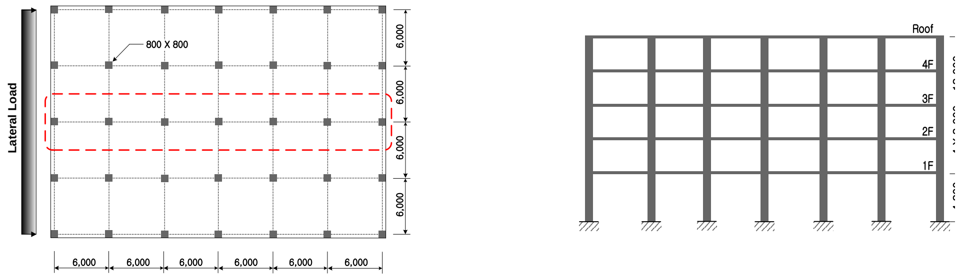
Seismic Force–Resisting System	Response Modification Coefficient, R	System Overstrength Factor, Ω_0	Deflection Amplification Factor, C_d	Structural System Limitations and Building Height (m) Limit Seismic Design Category		
				A or B	C	D
A. Bearing wall systems						
2. Ordinary reinforced concrete shear walls	4	2.5	4	-	-	60
B. Building frame systems						
6. Ordinary reinforced concrete shear walls	5	2.5	4.5	-	-	60
C. Moment-resisting frame systems						
6. Intermediate reinforced concrete moment frames	5	3	4.5	-	-	-
7. Ordinary reinforced concrete moment frames	3	3	2.5	-	-	NP
E. Dual systems with intermediate moment frames capable of resisting at least25% of prescribed seismic forces						
8. Ordinary reinforced concrete shear walls	5.5	2.5	4.5	-	-	60
F. Shear wall-frame interactive system with ordinary reinforced concrete moment frames and ordinary reinforced concrete shear walls	4.5	2.5	4	-	-	60

3. DEVELOPING ANALYTICAL MODELS FOR NONLINEAR STATIC ANALYSIS

3.1. Example structure

In this study, the results of the previous study on shear stud and shear band applications were applied to the input data of slab-column connections. They were then submitted to a non-linear pushover analysis because their ductility and energy absorption propertiesare better than others.(Song, J. K., 2012)

The model represented a multi-story residential building designed in accordance with ACI 318-08(ACI Committee, 2008). The compressive strength of concrete and the yield strength of the reinforcements were assumed to be 30MPa and 400 MPa, respectively. Dead and live loads were assumed to be 8.0kN/m^{2}and 1.0kN/m^2 for the roof floor, 9.0kN/m^2 and 2.0kN/m^2 for the exception of the roof floor, respectively. In the case of the seismic load, seismic zone factor, site classification, occupancy category and importance factors are 0.22, B, Ⅲ and 1.25, respectively. We conducted a 2-D analysis of the central portion of the structure as shown in Fig. 1. The story height of the ground floor and the remaining floors are 4.2m and 3m, respectively.



(a) Plan (b) Elevation

Figure 1. Example structure

45 shear-reinforced flat plate systems were developed. ①With reinforcing material; RC(with no reinforcement), SR(reinforced using Stud rails), SB(reinforced using Shear bands) ②by the number of stories ; 5, 10, 15 story structure ③by span ; 4m, 6m, 8m ④by number of bays ; 4bays, 6bays, 8bays and MIDAS Gen(MIDAS IT, 2012) was used to develop analytical models and to perform on linear static analysis. The effect of the higher mode was reflected by applying the lateral load distribution factor.(Kim, G. W., 2004)

3.2. Effective beam width

The effective beam width method is an analytical method used to analyze and design flat plate systems. By using this method, a flat plate system is analyzed as a frame system, and its slab-column connection is replaced with the effective beam-column connection.

In this study, 45 flat plate frame systems were modeled as 2-D plane frames by “equivalent effective beam width” in order to investigate the seismic performance of the flat plate structure.(Choi, J. W., 2001)

3.3. Development of analytical models for flat plate-column connections

To model the structure, the moment of inertia was modified and the plastic hinge properties were defined, corresponding to the existing test results. It was assumed that the moment of inertia of the exterior connection was 1.00 because no test was carried out for the exterior connections, thus there was no data. The plastic hinge properties of the connections were defined corresponding to the FEMA hinge type property.(FEMA, 2000)

The modified moments of inertia (effective moment of inertia (I_{eff})) and the plastic hinge properties are specified in Table 3.1 and Table 3.2. The values in bracket represent the upgraded values of the plastic hinge properties.

Table 3.1. Effective Moment Of Inertia (I_{eff}) For The Connections

Specimens	I-RC	E-RC	I-SR	E-SR	I-SB	E-SB
I_{eff}	$0.60I$	$1.00I$	$0.75I$	$1.00I$	$1.00I$	$1.00I$

Table 3.2. Plastic Hinge Properties Of The Connections

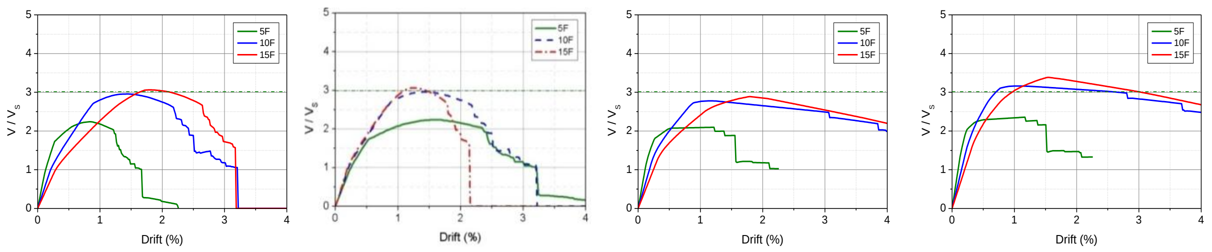
	RC (no reinforcement)		SR (Stud rail)		SB (Shear band)	
	D/D_y	M/M_y	D/D_y	M/M_y	D/D_y	M/M_y
A	0	0	0	0	0	0
B	1	0.75	1	0.85	1	1
C	3.2	0.98 (1.47)	8.8	1.03 (1.545)	9.2	1.28 (1.92)
D	5	0.75	22	0.77	36.5	1.02
E	5.2	0.2	22.5	0.2	36.8	0.2

4. EVALUATION OF THE EFFECTIVE RESPONSE MODIFICATION FACTORS ON SHEAR-REINFORCED FLAT PLATE SYSTEMS

4.1. Evaluation of the overstrength factor on flat plate systems

In this study, we estimated the overstrength factor (R_S) based upon the plastic hinge redistribution mechanism mentioned in FEMA 302(FEMA, 1997), FEMA 303(FEMA, 1997).

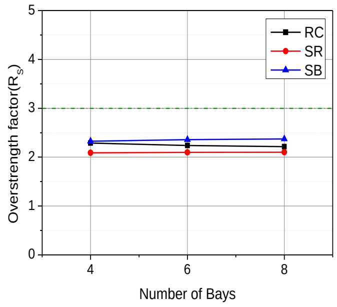
Fig. 2 illustrates the overstrength factor (R_S) of a structure with a 6m span and 6-bays. As you can see in this figure, the shear reinforcement did not affect the increase in the overstrength factor (R_S).The overstrength factor of structures over 10-storeys was about 3, which is the same value as the ordinary reinforced concrete moment frames and intermediate reinforced concrete moment frames. While the overstrength factor of 5-story structures did not meet the value of the design criteria, regardless of any shear reinforcement measures that may or may not have been applied.\



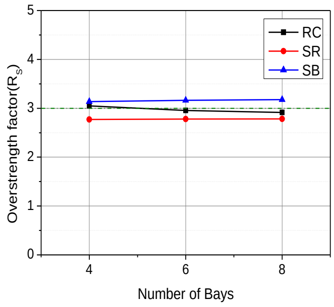
(a) RC (b) SR (c) SB

Figure 2. Overstrength factors of the example model (6m span, 6-bays)

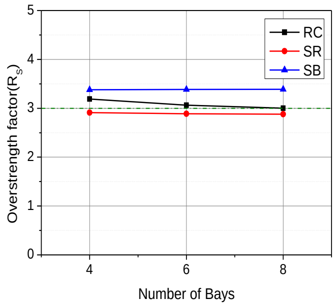
Fig. 3 illustrates the overstrength factors (R_S) of the structure by variables. The overstrength factor of structures over 10-storeys reinforced by shear band was more than 3.0. While the overstrength factors of structures with shear studs used as shear reinforcing measures, did not increase compared with those without reinforcement.



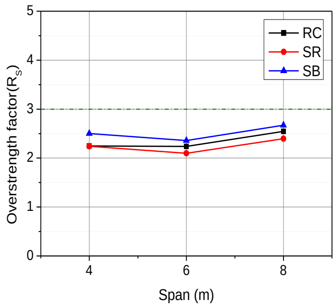
(a) Overstrength factor based upon the number of bays (5F)



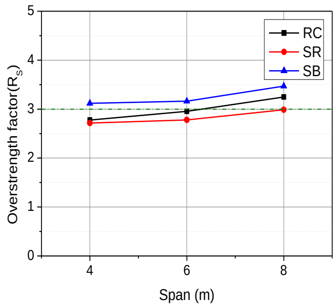
(b) Overstrength factor based on the number of bays(10F)



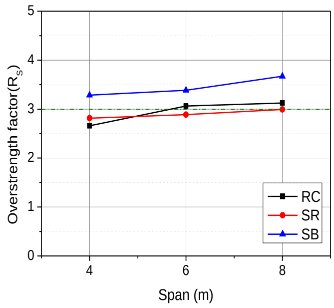
(c) Overstrength factor based upon the number of bays (15F)



(d) Overstrength factor based upon span (5F)



(e) Overstrength factor based upon span (10F)



(f) Overstrength factor based upon span (15F)

Figure 3. Overstrength factors of the example model based on the different variables

4.2. Evaluation of ductility factors and effective response modification factors on flat plate systems

Fig. 4 shows a nonlinear static pushover curve and definitions of Δ_{eff}, Δ_{max}, μ_{eff}, and μ_{max}. Δ_{eff} and Δ_{max}, which are defined based on the Inter-story Drift Index proposed by Bertero(Bertero, V. V., 1994). Δ_{eff} is taken as the displacement of the life safety level(1.5%), Δ_{max} is taken as the displacement of the collapse prevention level(2.0%).

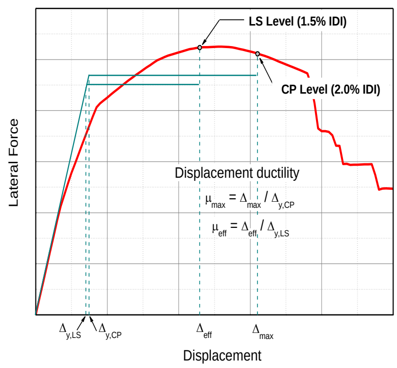


Figure 4. Parameters of the ductility factor

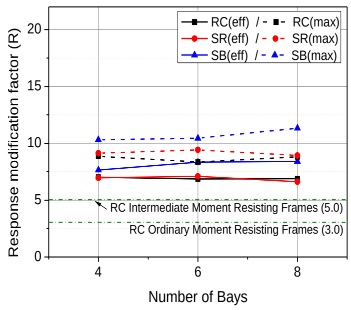
The structural system effective ductility,μ_{eff}, is defined as the ratio of Δ_{eff} and $\Delta_{y,LS}$, and the maximum ductility, μ_{max}, is defined as the ratio of Δ_{max} and $\Delta_{y,CP}$.The evaluation equation proposed by Fajfar is used to calculate the ductility factor, R_{μ}.(Fajfar, P., 2000) The Eqn. 4.1. is used for the calculation of the system response modification factor,R, where $R_{eff}$is denotes the effective response modification factor and R_{max} is denotes the maximum response modification factor.

$$R_{eff} = R_S \cdot R_{\mu,eff}$$
$$R_{max} = R_S \cdot R_{\mu,max} \tag{4.1}$$

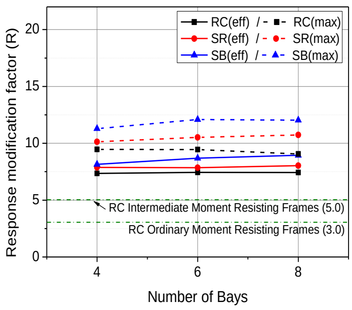
4.3. Evaluation of the response modification factor on flat plate systems

In this study, to compare the design criteria, the response modification factor, R was defined as the effective response modification factor,R_{eff}, and the maximum response modification factor, R_{max}, respectively. As mentioned in sec.4.2, the values of the system overstrength factor,R_S, did not meet the design criteria apart from the structure reinforced by shear band. While in the case of the effective response modification factor,R_{eff}, all of the structures met the standard.

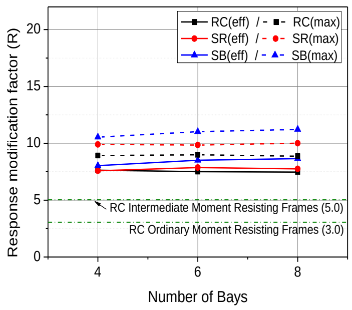
Fig. 5 shows the comparison between the effective response modification factor and the maximum response modification factor of the flat plate systems by different variables. Table 4.1 summarizes the results of the evaluation.



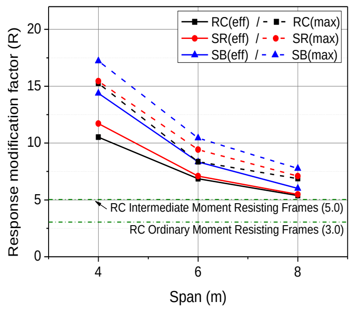
(a) Overstrength factor based upon the number of bays (5F)



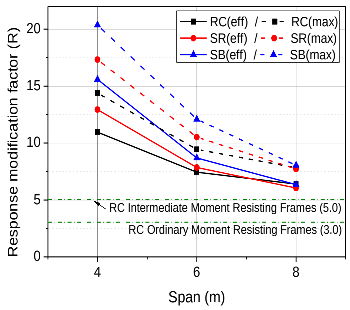
(b) Overstrength factor based upon the number of bays (10F)



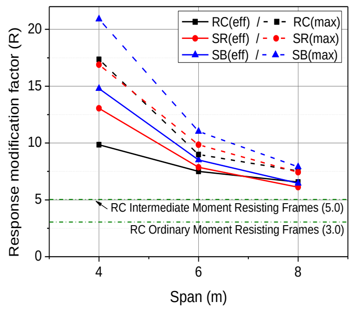
(c) Overstrength factor based upon the number of bays (15F)



(d) Overstrength factor upon span (5F)



(e) Overstrength factor based upon span (10F)



(f) Overstrength factor based upon span (15F)

Figure 5. Response modification factors of the example model by Variables

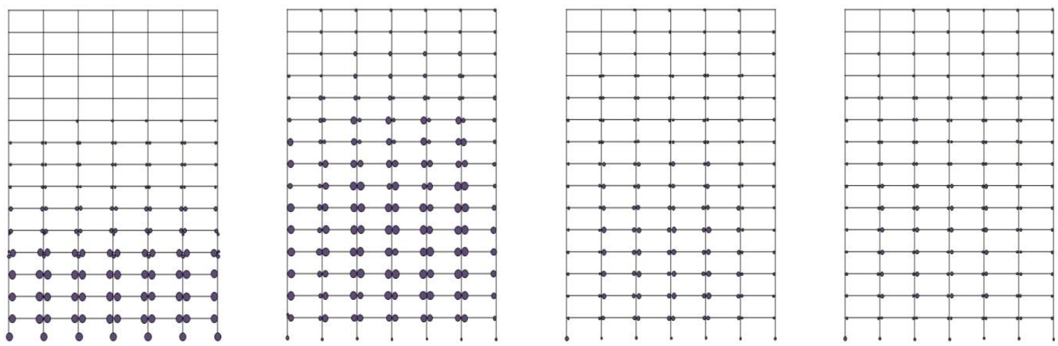
In the case of the effective response modification factor,R_{eff}, all of the example structures have values higher than that of the Intermediate reinforced concrete moment frames (5.0), regardless of shear reinforcement. When the shear stud is applied as a shear reinforcement measure and shear band is applied as a shear reinforcement measure, the value of the maximum response modification factor,R_{max}, increased by 5 ~ 10% and over 20%, respectively, which represents improved lateral resistance performance.

Table 4.1. Response Modification Factors Of The Example Model

Shear reinforcement	Story	R_S	$R_{\mu,eff}$	$R_{\mu,max}$	R_{eff}	R_{max}
None	5F	2.30	3.19	4.14	7.26	9.42
	10F	2.99	2.66	3.37	7.84	9.94
	15F	3.02	2.60	3.43	7.76	10.12
Stud rail	5F	2.18	3.47	4.58	7.50	9.91
	10F	2.81	3.03	4.01	8.44	11.16
	15F	2.90	2.91	3.70	8.38	10.66
Shear band	5F	2.44	3.66	4.65	8.86	11.25
	10F	3.21	2.96	3.98	9.41	12.65
	15F	3.42	2.71	3.58	9.16	12.11

4.4. Plastic hinge redistribution

To investigate plastic hinge propagation, a pushover analysis was done for the beam-column system modeled under the same conditions. The plastic hinge redistribution mechanism, when the roof drift ratio reached 2%, is illustrated in Fig. 6.



(a) Beam-column system (b) RC (c) SR (d) SB

Figure 6. Plastic hinge redistribution

Plastic hinge propagation is good in the beam-column system (Fig. 6(a)). Whereas in the RC example, the plastic hinges were placed at the bottom of the ground floor column, and the slab-column connections failed (Fig. 6(b)).The plastic hinge formation was less in SR (Fig. 6(c)) and SB (Fig. 6(d))due to the shear reinforcement.

5. CONCLUSION

In this study, the effective response modification factor was evaluated for flat plate structures without walls. Through a comparative analysis of the results, we defined the seismic force-resisting system applicable to flat plate systems. The results are as follows.

(1) In the case of low-rise buildings, the value of the overstrength factor was less than the value of the criteria, because redundancy is low for buildings with few members. However, structures over 10-storeys tall, which were reinforced by shear band met the value of the criteria.
(2) In the case of the effective response modification factor, R_{eff}, all of the example structures showed values over that of Intermediate reinforced concrete moment frames (5.0) regardless of shear reinforcement. When the shear stud is applied as a shear reinforcement measure, and shear band is applied as a shear reinforcement measure, the value of the maximum response modification

factor, R_{max}, increased by 5 ~ 10% and over 20%, respectively. It also shows that shear reinforcement of the connections increases the ductility of the entire structure.
(3) To apply building frame systems, dual systems with intermediate moment frames or shear wall-frame interactive systems within flat plate structures, the capacity of the flat plate frame system without walls, should be proportionate to that of ordinary reinforced concrete moment frames or intermediate reinforced concrete moment frames.

In the case of the low-rise buildings, it is impossible to define those systems because the system overstrength factor does not meet the standard. So the overstrength factor of the system needs to be increased. While except for the low-rise building, the flat plate frame system reinforced by shear band could be defined as an ordinary reinforced concrete moment frames or intermediate reinforced concrete moment frames.

ACKNOWLEDGMENTS

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