Influence of steel bars in monolithic floor slabs on the yielding-damage mechanism of frame structures

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Abstract

Aiming at the seismic design requirement of "strong column-weak beam" frame structure and combined with test data analysis and computer simulation, the influence of slab bar participation on inelastic response and the yielding-damage mechanism of frame structure have been investigated. Two design schemes that considering the slab bars or neglecting the slab bars were respectively applied to an example frame building to compare the damage situations, which were simulated by the nonlinear time history analysis program SARCF (Seismic Analysis of Reinforced Concrete Frames). The research indicated that slab bars in the tensile zone have a direct enhancing action on the capacity of the frame beam resisting negative moment, which will increase with the augmentation of the monolithic deformation of the beam-slab member. The effect of slab bar participation will develop adequately after the peak ground acceleration acting on the structure reaches the level of design basic acceleration of ground motion. The design method neglecting the slab bars will underestimate the actual flexural strength of the frame beam, which will result in a change of seismic behavior of the structure from "strong column-weak beam" to "strong beam-weak column", and the probability of the change will increase with the augmentation of earthquake action.

Keywords: frame structure, slab bars, strong column-weak beam, yielding mechanism, damage situation.

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1 Introduction

Reinforced concrete (RC) frames have been widely applied to multi-storey or high-rise buildings in seismic zones, because of their predominance in structural integrity and layout flexibility. Along with the enhancement of seismic fortification level, the RC frame structure has become a main structure form for public buildings, such as hospitals, schools and emporiums, and the seismic behaviour of the RC frame structure is attracting more attention from the whole society.

According to the design principle of ductile structure, the fame structure shall be designed to satisfy the requirement of "strong column-week beam", and to prevent the collapse of buildings resulting from the "column hinge mechanism" in strong earthquakes. Many examples of collapsed frame structures due to the "column hinge mechanism", however, have been reported continually from earthquake disasters in recent years [1–4]. In the Wen-Chuan earthquake of magnitude 8.0 on May 12th 2008, some frame structure buildings in high intensity regions were severely damage or collapsed due to the "column hinge mechanism" [5, 6] (Figs. 1–4), which have aroused further attention towards the design requirement of "strong column-week beam" in the Chinese civil engineering community.

Figure 1: Broken column tops in a frame hall.

Figure 2: Crushed column top on the ground floor.

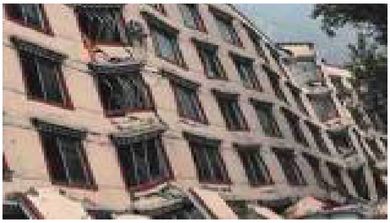

Figure 3: Failure frame of a teaching building.

Figure 4: Collapsed eight-storey frame building.

In current structural seismic design, making the flexural strength of the column stronger than that of the beam at the beam-column joint is one of the key methods to guarantee the "strong column-week beam". In the Code for Seismic

Design of Buildings (GB 50011-2001) [7], for the beam-column joints of earthquake resistant classification 1st, 2nd and 3rd frame structures, except the columns on top floor or the columns with an axial compression ratio of less than 0.15, the design composite moments of column ends should satisfy the following requirement

$$\sum M_c = \eta_c \sum M_b \tag{1}$$

Furthermore, classification 1st frame structures and frames in the nine degree seismic fortification zone should satisfy the requirement

$$\sum M_c = 1.2 \sum M_{bua} \tag{2}$$

where $\sum M_c$ is the sum of the design composite moments of the column ends framing into the beam-column joint, and the design composite moments of the column ends can be allotted by elasticity analysis; η_c is the moment augment coefficient for the column ends, η_c=1.4 for classification 1st, η_c=1.2 for classification 2nd, and η_c=1.1 for classification 3rd. $\sum M_b$ is the sum of the design composite moments of the beam ends framing into the beam-column joint; $\sum M_{bua}$ is the sum of the corresponded moments of the actual seismic bending capacity of the beam ends framing into the joint, and is to be calculated by the actual reinforcement area (including compressive reinforcement) and the standard material strength.

The co-action of slab and beam is one of the key concerns when checking the flexural strength of a monolithic beam-slab frame structure with the above formulas. In current concrete structure design codes [8, 9] for the monolithic beam-slab structure, only the bending capacity of the slabs in the compressive zone is to be considered as the flange plate in the T-beam when calculating the flexural strength of the beam. When the slab is in the tensile zone, the calculation of flexural strength of the beam is based on the rectangular cross section, since the tensile capacity of the concrete is not to be considered. In the majority of the current public building structures, the compression reinforcements paralleling with the beam in the monolithic slab usually have a diameter of $\Phi 10 - \Phi 14$, and are spaced between 100mm–200mm. The slab bar area within the effective compression flange range is about 10%–25% of the top reinforcement area of the beam end. In non-seismic structure design, neglecting the slab bars would evidently increase the safety factor for the frame beam to resist the negative moment. However, because of the requirement of “strong column-week beam” in the seismic structure design, the influence of the slab bars on the seismic behaviour of the frame structure should be further investigated.

Combined with the test data analysis and computer simulation, the action and influence of the slab bar participation on the yielding-damage mechanism of the monolithic frame structure has been analyzed and effective design methods to prevent “strong beam-week column” damage have been investigated.

2 The influence of slab bars on the bearing capacity of frame beams

Ma et al. [10] investigated the composite flexural strength of the flange plate and the web of the T-beam in researching the frame beam hysteretic characteristics [10]. For the convenience of comparison, one rectangular beam (R-3) and one T-beam (T-1) with the same web size and reinforcement were constructed (Figure 9). The $M-\phi$ hysteretic curves of the two beams were obtained with the same quasi-static loading scheme (Figure 10). The test result shows that the longitudinal bars in the flange slab could increase the capacity of the section resisting the negative moment effectively in both elastic and non-elastic cyclic phases. Therefore, the test report emphasized that the contribution of longitudinal slab bars should be considered when calculating the flexural strength of the beam end at the beam-column joint zone.

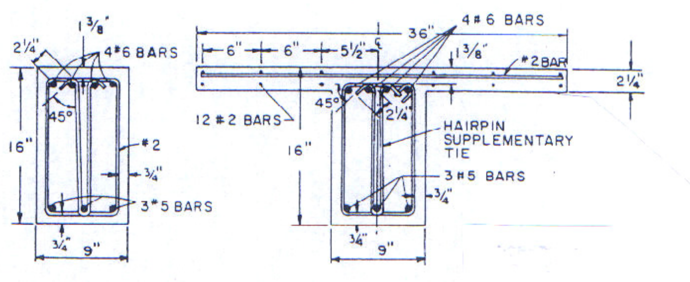

Figure 5: Beams R-3 (left) and T-1 (right) with the same web and reinforcement.

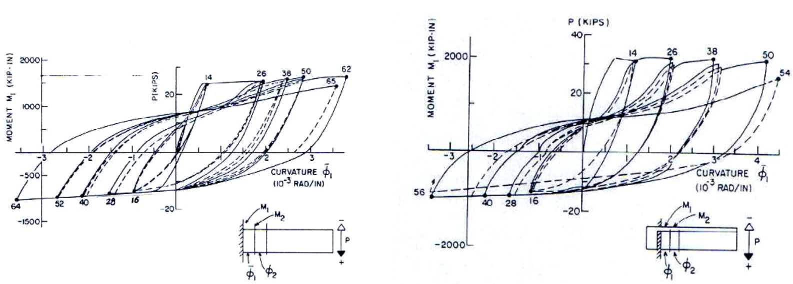

Figure 6: $M-\phi$ hysteretic curves of R-3 (left) and T-1 (right).

The effective tensile slab width of the beam-slab structure is an important factor influencing the flexural strength of the frame beam. Kiureghian [11], Zerbe and Durrani [12], and French and Boroojerdi [13] have tested and researched the bending resistance of the composite joints of the slab-beam-column and the slab in the T-beam. They found that the slab bars in the tensile zone have an obvious enhanced action to both the stiffness and strength of the beam. Based on the above test, Pantazopoulou set up a mathematical model for slab participation, simulated and analyzed the deformation characteristics and the effective width of the tensile slab in a theoretical way. The analysis and the test both show that the performance of slab participation depends on the cross section dimension and the deformation level of the beam-slab member [14]. The bending resistance of the tensile slab could be expressed as the function of effective slab width: the effective width should be approximately equal to 1.5 beam depths on each side of the web for pre-yielding response, and this width should be increased to 2.0 beam depths for moderate post-yielding response.

3 The influence of slab bars on the damage situation of the monolithic floor frame structure

Considering the design requirement of "strong column-week beam" and the strengthening effect of the slab bars on the beam, it is necessary to understand the influence of slab bar participation on the inelastic response and yielding-damage mechanism of the frame structure under earthquakes. Two different design schemes that consider the slab bar or neglect the slab bar were respectively applied to a frame building to compare the damage situations, and the computer alanysis was performed by the nonlinear time history analysis program SARCF [15] (Seismic Analysis of Reinforced Concrete Frames).

3.1 Design schemes of example building

The example frame building is a typical eight-storey office building with the monolithic RC structure, it is located in an eight degree seismic fortification zone, the type of site that is classification III; the earthquake resistant category of the structure is classification 2nd, and the design basic acceleration of the ground motion is 300 gal according to the seismic design code [7]. The frame dimension, member and joint numbers are shown in Figure 7.

To compare the influence of the slab bar participation, two different reinforcement schemes were designed as follows. **Scheme A**: according to the suggestion by Pantazopoulou and Moehle [14], we took the effective slab width b_t equal to 1.0 beam depth on each side of the web to participate in the beam-slab co-action, and the area of slab bars in the effective widths ($A_{sb} = 785mm^2$) should be included to the total reinforcement area of the beam top, then the column reinforcement is calculated based on Eq. (1). **Scheme B**: according to current seismic design methods, neglecting the slab bar participation, only the reinforcement in the web is calculated as the reinforcement area of the beam top, then the column reinforcement is calculated based on Eq. (1). Figure 8 shows the cross section of the beam-slab members and reinforcement of the two design schemes. Table 1 lists the cross section dimension and reinforcement area of the members, among which the strength grade of the reinforcement is HRB335, and the strength grade of the concrete for the columns in storeys 1-2 is C40, for the

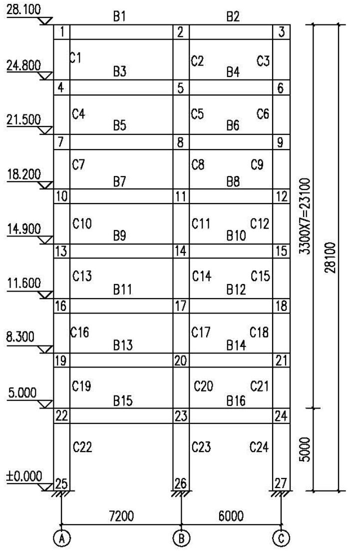

Figure 7: Frame dimensions (mm) and joint numbers.

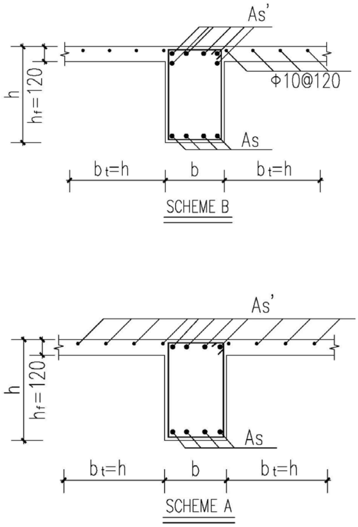

Figure 8: Section and reinforcement of beam-slab.

Table 1: Cross-section dimension and reinforcement of the members.

Type	Member number	Dimension (mm)		Reinforcement (mm^2)					
				scheme A		scheme B			
		b	h	A_s	A_s'	A_s	A_s'	A_{sb}	$A_s' + A_{sb}'$
Beam	B1-B4	300	650	1570	1570	1570	1570	785	2355
	B5-B14	300	650	1956	1956	1956	1956	785	2741
	B15-B16	300	650	2565	2565	2565	2565	785	3350
Column	C1, 3, 4, 6, 7, 9, 10, 12 C13, 15, 16, 18, 19, 21	450	450	$A_s = A_s' = 1964$					
	C2	550	550	$A_s = A_s' = 2281$					
	C5, 8, 11, 14, 17	550	550	$A_s = A_s' = 2829$					
	C20	550	550	$A_s = A_s' = 3079$					
	C22, 24	600	500	$A_s = A_s' = 3436$					
	C23	650	600	$A_s = A_s' = 4310$					

Table 2: Column-beam flexural strength ratio η_c .

Joint	1	2	3	4	5	6	7	8	9	10	11	12
scheme A	0.85	0.62	0.85	1.69	1.37	1.69	1.33	1.18	1.33	1.33	1.18	1.33
scheme B	0.56	0.49	0.56	1.52	1.09	1.52	0.98	1.00	0.98	0.98	1.00	0.98
Joint	13	14	15	16	17	18	19	20	21	22	23	24
scheme A	1.33	1.18	1.33	1.33	1.18	1.33	1.33	1.24	1.33	1.74	1.34	1.74
scheme B	0.98	1.00	0.98	0.98	1.00	0.98	0.98	1.05	0.98	1.34	1.16	1.34

other columns and all beams and slabs it is C30 [8]. Table 2 lists the actual column-beam flexural strength ratio η_c of each joint (that is, the moment augment coefficient of the column ends in Eq. (1)) in two schemes. It can be seen from tables 1 and 2 that the actual reinforcement of beams in scheme B is higher than that of scheme A, because of the participation of the tensile bars (A_{sb}) in the effective slab width. Therefore, with the same frame column reinforcement, the actual column-beam flexural strength ratio η_c of each joint in scheme B is lower than that in scheme A.

3.2 Analysis program and method

SARCF is a computer program for analyzing the nonlinear response of RC frames subjected to deterministic or randomly generated earthquake ground motion, and with the special function for simulating the damage situation of the RC members based on the energy dissipation theory [15, 16].

According to the damage definition of SARCF, for a RC member undergoing the cyclic loading, the damage can be observed after the section yielded, and, along with the increase of the non-elastic cyclic times, the bearing capacity deteriorates and the damage increases. The damage situation of the member is described by the damage index D_e : $D_e = 0$ means the member is not damaged at all; $D_e = 1$ means the member failed completely. To evaluate the total damage situation of a storey or the frame structure, SARCF proposed the storey damage index D_{sb} for beams and D_{sc} for columns, respectively, based on the energy dissipation proportion of the members on each storey. Besides, since the damage of the lower storeys is worse than that of the upper storeys, the structural damage index D_g is given according to the weighted average of the damage indices of all storeys in the structure.

The acceleration of ground motion input into the time history analysis program was the EL-Centro earthquake recorder with the total record duration of 19.9s and the main period of 0.5–0.55s (Fig. 9). To investigate the change of the

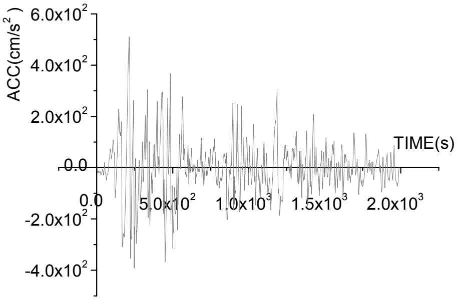

Figure 9: Ground acceleration histories (PGA = 510 gal).

damage situation of the columns and beams at each storey under different seismic intensity, a total of seven conditions were analyzed by adjusting the peak acceleration from 210 gal to 510 gal increasing by per grade of 50 gal.

3.3 Comparison of the damage situations

Table 3 lists the statistical damage indices of the beams and columns on each storey analyzed by the SARCF. The comparison of damage situations of the beams and columns on the second storey in the two schemes based on table 3 are shown in Figs. 10(a) and (b), respectively.

The statistical damage indices of the two schemes show that:

1) For the total damage situation of the members, under minor earthquake intensity, the member of the two schemes does not yield, the damage index of every member was 0, which indicates that the slab bar participation was not very evident. After the PGA exceeded the level of design basic acceleration of ground motion (300 gal), the damage indices of members increase with the augmentation of earthquake intensity, and the damage indices in the two schemes show greater differences, which indicate that the slab bar participation is increasing gradually.

2) For the yielding-damage situation of the beams, the damage indices of both schemes A and B have a similar development trend. Basically, the damage indices of the beams on each storey increase with the augmentation of the earthquake intensity, and the beam in scheme B yielded later than the beam in scheme A. For the lower storeys with more severe damage, the second storey for example, the damage index of the beams in scheme B is less than that in scheme A under each PGA condition (Fig. 10(a)).

3) For the yielding-damage situation of the columns, the columns on each storey in scheme B yielded earlier than those in scheme A, and the damage index of the columns in scheme B under each PGA condition are higher than those in scheme A, which is more evident on the ground storey and the second storey.

Figure 11 shows the damage distribution of each frame member under the time history of PGA = 460 gal in two schemes. Figure 12 shows the $M-\phi$ hysteretic curves of the beam end and column end at joint eight on sixth storey, which describes the influence of the slab bar participation on the yielding-damage mechanism.

Table 3: Damage indices under each PGA condition.

Damage type	Scheme	Storey	210gal	260gal	310gal	360gal	410gal	460gal	510gal
Beam D_{sb}	A	8	0	0	0	0	0	0	0
		7	0	0	0	0	**0.003**	**0.016**	**0.026**
		6	0	0	0	**0.003**	**0.021**	**0.036**	**0.052**
		5	0	0	**0.008**	**0.026**	**0.040**	**0.066**	**0.074**
		4	0	0	**0.017**	**0.037**	**0.046**	**0.072**	**0.100**
		3	0	0	**0.017**	**0.033**	**0.043**	**0.062**	**0.090**
		2	0	0	**0.010**	**0.023**	**0.035**	**0.046**	**0.072**
		1	0	0	0	0	0	**0.015**	**0.030**
	B	8	0	0	0	0	0	0	0
		7	0	0	0	0	0	**0.012**	**0.030**
		6	0	0	0	0	**0.015**	**0.030**	**0.047**
		5	0	0	0	**0.021**	**0.039**	**0.074**	**0.090**
		4	0	0	0	**0.024**	**0.035**	**0.064**	**0.088**
		3	0	0	**0.008**	**0.016**	**0.028**	**0.042**	**0.057**
		2	0	0	**0.005**	**0.015**	**0.027**	**0.039**	**0.045**
		1	0	0	0	0	0	**0.004**	**0.021**
Column D_{sc}	A	8	0	0	0	0	0	0	0
		7	0	0	0	0	0	0	0
		6	0	0	0	0	0	**0.001**	**0.012**
		5	0	0	0	0	0	**0.002**	**0.009**
		4	0	0	0	0	0	**0.005**	**0.027**
		3	0	0	0	0	0	**0.008**	**0.011**
		2	0	0	0	0	**0.014**	**0.024**	**0.045**
		1	0	0	**0.013**	**0.031**	**0.050**	**0.094**	**0.154**
	B	8	0	0	0	0	0	0	0
		7	0	0	0	0	0	0	0
		6	0	0	0	0	0	**0.014**	**0.045**
		5	0	0	0	0	0	**0.000**	**0.000**
		4	0	0	0	0	0	**0.022**	**0.045**
		3	0	0	0	0	0	**0.010**	**0.015**
		2	0	0	**0.002**	**0.027**	**0.033**	**0.042**	**0.076**
		1	0	0	**0.030**	**0.083**	**0.147**	**0.166**	**0.189**
Structure D_g	A		0	0	**0.046**	**0.114**	**0.166**	**0.251**	**0.356**
	B		0	0	**0.049**	**0.140**	**0.250**	**0.320**	**0.382**

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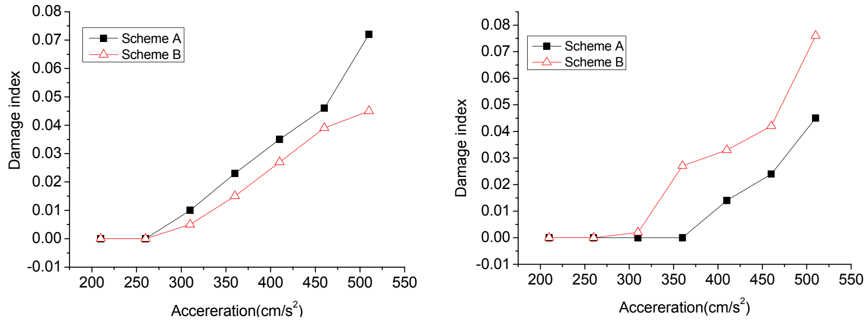

(a) Damage of beams (b) Damage of columns

Figure 10: Comparison of damage situations of the beams and columns on the second storey.

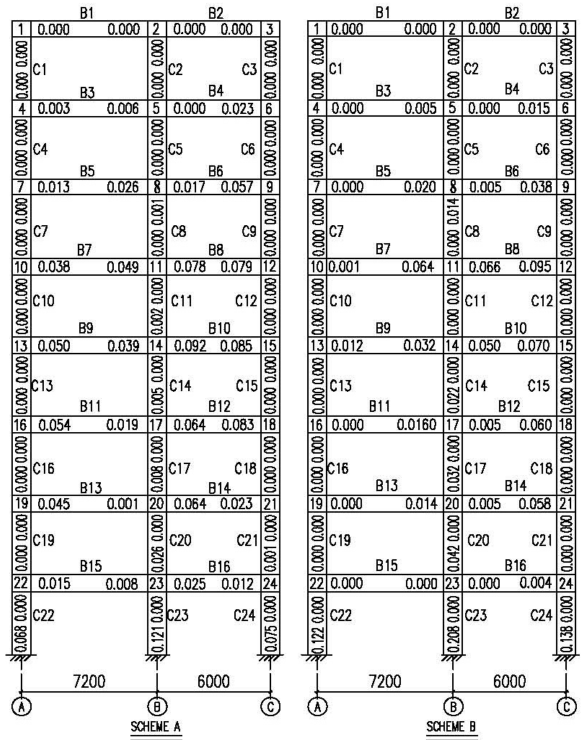

Figure 11: Frame member damage index D_e .

1) Scheme A considered the slab bar participation in the design, the actual flexural ability of the beams is coincident with the designed value. Under strong seismic force, the top reinforcement in the beam yielded, but the conjoint column did not, which made the damage situation of the joint a "strong column-week beam" type.

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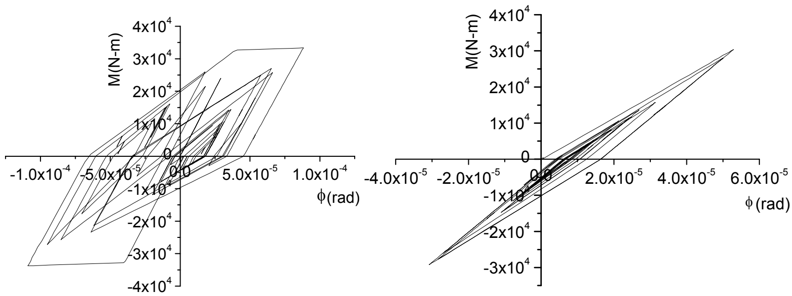
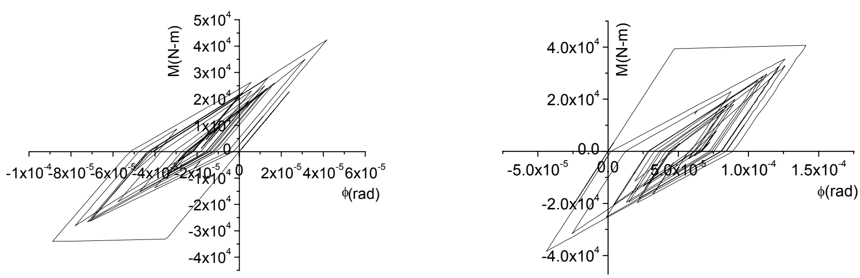


(a) $M-\phi$ curve of beam (b) $M-\phi$ curve of column

Scheme A



(a) $M-\phi$ curve of beam (b) $M-\phi$ curve of column

Scheme B

Figure 12: $M-\phi$ curves of the column end and the beam end at joint eight.

2) Scheme B neglected the slab bar participation in the design, however, the excess reinforcement (A_{sb}) increased the actual flexural strength of the beams. At the column-beam joint, the column yielded before the yield of the top reinforcement in the beam, which made the damage situation of the joint a “strong beam-week column” type.

4 Conclusion

Based on the results of this investigation the following conclusion and suggestion are drawn.

1) Slab bars in the tensile zone have a direct enhancing action on the capacity of the frame beam resisting negative moment, which will increase with the augmentation of the monolithic deformation of the beam-slab member. The effect of the slab bar participation will develop adequately after the peak ground acceleration acting on the structure reaches the level of the design basic acceleration of ground motion.

2) The design method neglecting the slab bars will underestimate the actual flexural strength of the frame beam, which will result in the change of seismic behaviour of the structure from "strong column-weak beam" to "strong beam-weak column", and the probability of the change will increase with the augmentation of earthquake action.

3) In seismic design, considering the participation of the tensile slab bars will accord with the economical and reasonable principle, which can not only reduce the total reinforcement amount, but also reflect the actual bending capacity of the member. The amount of slab bars in the tensile zone can be decided by the effective width b_t on each side of the web. According to the current seismic design standard [7], for 7, 8, and 9 degree seismic fortification zone the effective width b_t could be taken as 1.0–1.5 beam depth, and the larger value is applicable for the frame structures in the high seismic fortification zone. Furthermore, to satisfy the safety requirement of the static design and dynamic design, as well as the construction requirement of longitudinal reinforcement [7, 8], the area of slab bar participation should not be taken larger than 30% of the total area of longitudinal reinforcement at the top of the beam end.

In this investigation, the presupposition is the RC member with adequate shear capability when simulating and analyzing the damage situation of the member. For the actual engineering, the design that neglected the slab bars may induce the problem of "strong bending capacity-weak shear capacity" of the beam, which will need further study. In addition, for structural ductility and economical design under strong seismic forces, the plastic joints could occur at the lower end of the ground columns as well as at the end of columns on other storeys. Optimal design is needed to delay the occurrence of the plastic joints to the ground columns and lessen the damage of the columns on each storey. The slab bar participation should be also brought into the optimal design.

References

[1] Ergin Atimtay, P.E., and Recep Kanit, Learning Seismic Design from the Earthquake Itself, Practice Periodical on Structural Design and Construction, Vol. 11, No. 3, August 1, 2006:149–160

[2] M.H. Arslan and H.H. Korkmaz, What is to be learned from damage and failure of RCstructures during recent earthquakes in Turkey?, Engineering Failure Analysis 14 (2007) 1–22

[3] Adem Dogangun, Performance of RC buildings during the May 1, 2003 Bingol Earthquake in Turkey，Engineering Structures 26 (2004) 841–856

[4] Ahmed Ghobarah, Murat Saatcioglu and Ioan Nistor, The impact of the 26 December 2004 earthquake and tsunami on structures and infrastructure, Engineering Structures 28 (2006):312–326

[5] Zhao Xi'an, Some matters in structure seismic deisgn from Wen-Chuan earthquake disaster, building sturcture, 2008 (7):17–20

[6] Sun Jingjiang, Ma Qiang, Shi Hongbin, Sun Zhongxian, Introduction to the building damage in Wen-Chuan city under high seismic intensity, seismic engineering and project vibration, 2008 (6):7–15

[7] GB 50011 – 2001, Code for Seismic Design of Buildings, China Construction Industry Press, Beijing, 2001

[8] GB 50010 – 2002, Design code Concrete structure, China Construction Industry Press, Beijing, 2002

[9] ACI Committee 318 ,Building Code Requirements for Structural Concrete (ACI 318299) , and Commentary (ACI 318R299).

[10] Ma, S-Y. M., Bertero, V. V., and Popov, E. P., "Experimental and Analytical Studies on the Hysteretic Behavior of RC Rectangular and T-Beams", Report No. EERC-76-2, University of California, Berkeley, May, 1976

[11] Kiureghian, S., Hysteretic behavior of exterior R/C beam-column-slab subassembly, CE299 report (1983), Dept. of Civ. Engrg., Univ. of California, Berkeley, Calif.

[12] Zerbe, H. E. and Durrani, A. J., Effect of a slab on the behavior of exterior beam to column connections, Report No. 30 (1985), Dept. of Civ. Engrg., Rice Univ., Houston, Tex.

[13] French, C. W. and Boroojerdi, A, T-beam effect in structures subjected to lateral loading, Proc, 3d U.S. Int. Conf. on Earthquake Engrg., Vol. II, Charleston, S.C., 1986：1191-1202

[14] S. J. Pantazopoulou and J. P. Moehle, Idetification of Effect of Slabs on Flexural Behavior of Beams, Journal of Engineering Mechanics, Vol. 116, No. 1, January, 1990：91－106

[15] Jianli Yuan, C. Meyer, Y. S. Chung, Automatic Design for Uniform Damage Distribution by Program SARCF, Journal of Southeast University, 1995, 11(1A): 303-310

[16] Y. S. Chung, C. Meyer, and M. Shinozuka, Modeling of Concrete Damage, ACI Structural Journal, 1989, 86, No (3): 259-271