Effect of Column to Beam Strength Ratio on Performance of Reinforced Concrete Frames

I Ketut Sudarsana*, Ida Ayu Made Budiwati*, Putu Wiyta Aditya*

*Department of Civil Engineering
Udayana University, Badung, Bali, Indonesia
civil2001ca@yahoo.com; ksudarsana@civil.unud.ac.id

***Abstract* — A ductile reinforced concrete structure shall be designed to ensure that plastic hinges occur as many as possible before collapse. This paper investigates the effect of column to beam strength ratio on performance of ductile reinforced concrete buildings. Fourteen interior frame models of two building categories which are five and ten stories buildings were modeled and analyzed. The main parameter among those models is column to beam strength ratio of 1.0 to 2.0 which are 1.0, 1.2, 1.4, 1.6, 1.8, and 2.0. The values are the ratio between column nominal strength ($\sum M_{nc}$) and beam nominal strength ($\sum M_{nb}$). In this study, the ratio between column strength to beam probable strength ($\sum M_{prb}$) of 1.2 is also investigated. A static nonlinear pushover analysis was used to evaluate the performances of all models. Analysis results show that all models have a *life safety* performance level. A collapse mechanism of *beam sway mechanism* was achieved for strength ratios of 1.4 to 2.0 for five story frame models and 1.6 to 2.0 for ten story frames. The increase in the strength ratio up to 1.4 can increase ductility factor significantly, however, beyond that the strength ratio does not affect the ductility both for five and ten story frame models. Considering the ratio between column strength to beam probable strength of 1.2, the ductility factor increases by 16% and 25% respectively for five and ten stories, however, both frames still have performance level of *life safety* and collapse mechanism of *column sway mechanism*.**

Key words – Performance level, colum-to-beam strength ratio, reinforced concrete, ductility.

I. INTRODUCTION

A. Background

Many researches have been done to investigate the behavior of reinforced concrete structure due to earthquake load. Ref. [1] in clause 21.6.2.2 suggests that for ductile moment frames, the sum of nominal flexural strength of the column at beam-column joint shall be at least 1.2 times the sum of nominal flexural strength of beams framing into the joint. This clause is intended to guarantee the behavior of strong column-weak beam of reinforced concrete frames. However, a study by [2] on the frames of five and ten stories buildings shows that the strength ratio of 1.2 cannot guarantee a strong column-weak beam criteria occurred especially in the buildings located at high seismic regions. Code of other countries such as New Zealand and Mexico suggests a higher ratio which is in a range of 1.5 to 2.0 [3].

A study by [3] on two buildings (3 and 6 stories) in which the strength ratio were varied from 0.8 to 2.4 shows that the increase in the strength ratio results in the improvement on protection life safety to building occupants by avoiding a story mechanism occurred in a building. In term of seismic performance, the strength ratio obtained by changing the column reinforcement was more effective rather than changing the member dimension. Ref. [3] calculated the story drift by a technique to push the story in question, while the story below was restrained from lateral displacement. This technique was questioned by [4] being inconsistent with typical behaviors of frame subjected to earthquake excitation.

An in-elastic behavior of beams to response the seismic design loads is expected in order to distribute seismic energy. During this behavior, the reinforcement in beams may have reached strain hardening conditions so that the moment capacity of the beams may also increase from a nominal flexural moment (M_{nb}) to a probable flexural capacity (M_{prb}). This condition may affect the sum of nominal column moment at the joint ($\sum M_{nc}$), therefore, a study of this effect is needed.

B. Research Objective

This research investigates the effect of column to beam strength ratios of 1.0 to 2.0 which are 1.0, 1.2, 1.4, 1.6, 1.8, and 2.0 on the performance of reinforced concrete frames using a static nonlinear pushover analysis. Two model structures are studied which are five and ten stories buildings. This study also investigates the effect of considering probable flexural strength (M_{prb}) of beams in calculating the nominal flexural strength of the column at the joint, instead of using the nominal flexural strength (M_{nb}) of beams.

II. RESEARCH METHOD

A. Study Building Descriptions

Two building types was studied namely five and ten story buildings which represent low and medium rise buildings, respectively, as shown in Figure 1 and 2. The building is considered as a hotel. The concrete strength, rebar tensile strength for longitudinal reinforcement and rebar tensile strength for transversal reinforcement are 30 MPa, 400 MPa and 240 MPa, respectively.

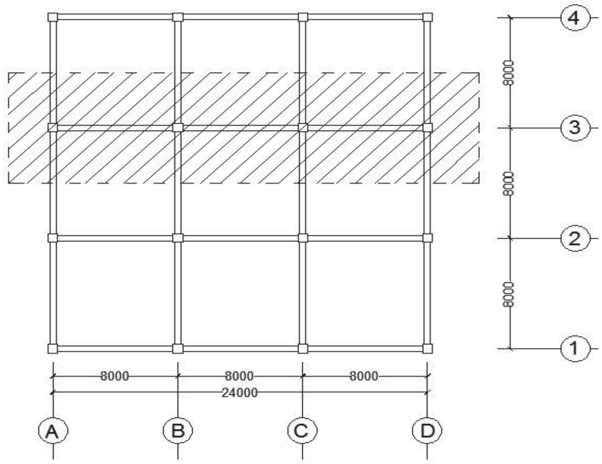
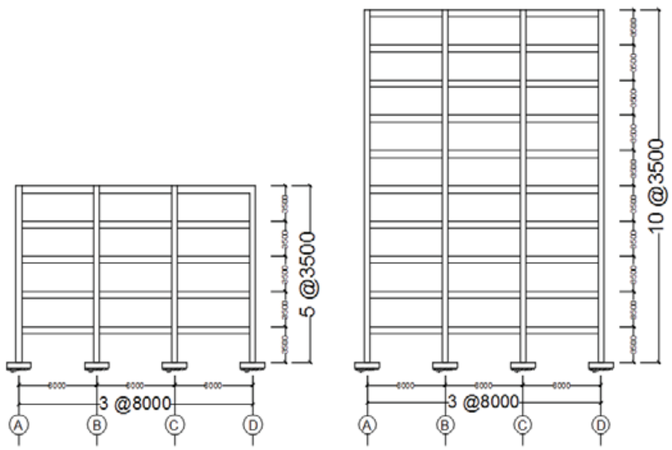


Fig.1 Plan of studied buildings



(a) (b)

Fig. 2 Interior frames of five (a) and ten (b) study buildings

Only the interior frames of each studied buildings as shown in Figure 2, were modeled and analyzed in accordance to [1] by varying the strength ratio of 1.0, 1.2, 1.4, 1.6, 1.8 and 2.0. The interior frame was considered due to supporting the largest gravity loads. In addition to those frame models, another model was also created for each story building in order to study the effect of using probable flexural strength (M_{prb}) instead of nominal flexural strength (M_{nb}) of beams to calculate the nominal flexural strength of column at the joint (ΣM_{nc}). Therefore, total frames to be studied in this research were fourteen frames as shown in Table I.

The frames were analyzed and designed to obtain beam dimension and reinforcement conforming [1] for a special moment frame. Once the beam reinforcement was obtained, then the nominal moment of the columns were calculated by considering the variation of column to beam strength ratio. Table II shows dimensions of all structural elements that satisfying all column to beam strength ratios to be studied. The slab thickness of 200 mm was applied for all models.

TABLE I
TOTAL STUDY MODELS

No	Model	No. of Story	Strength Ratio ∑Mnc/∑Mnb	Strength Ratio ∑Mnc/∑Mpr,b
1	M11	5	1,0	-
2	M12	5	1,2	-
3	M13	5	1,4	-
4	M14	5	1,6	-
5	M15	5	1,8	-
6	M16	5	2,0	-
7	M21	10	1,0	-
8	M22	10	1,2	-
9	M23	10	1,4	-
10	M24	10	1,6	-
11	M25	10	1,8	-
12	M26	10	2,0	-
13	M3	5	-	1,2
14	M4	10	-	1,2

B. Numerical Modelling and Analysis

Evaluation of the structural performance of all frames was carried out using static nonlinear pushover analysis provided by software SAP2000 v.15 after design results of all members were obtained. Plastic hinges behavior of the beams and columns were defined according to [5]. The flexural stiffness of columns and beams was reduced by considering crack inertia moment of 0.7 and 0.5 of the gross inertia moment. The structural performance was evaluated based on displacement target (δ_T) according to [5] as follows:

$$\delta_T = C_0 C_1 C_2 C_3 S_a (T_e/2\pi)^2 g \qquad (1)$$

where:

δ_T = Displacement target.

T_e = Effective fundamental period of the building in the direction under consideration, sec

C_0 = Modification factor to relate spectral displacement of an equivalent SDOF system to the roof displacement of the building MDOF system

C_1 = Modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response.

C_2 = Modification factor to represent the effect of pinched hysteretic shape, stiffness degradation and strength deterioration on maximum displacement response

C_3 = Modification factor to represent increased displacements due to dynamic P-D effects.

S_a = Response spectrum acceleration, at the effective fundamental period and damping ratio of the building in the direction under consideration

g = Acceleration of gravity 9,81 m/s²

Structural performance levels can be categorized according to [6] namely *Operational* (O), *Immediate Occupancy* (IO), *Life Safety* (LS), and *Collapse Prevention* (CP). The performance level of the models is determined base on the condition of which the ultimate plastic hinge occurred.

TABLE II
DIMENSION OF STRUCTURAL ELEMENTS

Story	Five story			Ten Story		
	Beams (mm)	Columns grid A,D (mm)	Columns grid B,C (mm)	Beams (mm)	Columns grid A,D (mm)	Columns grid B,C (mm)
1	400 x 700	450 x 450	550 x 550	400 x 800	600 x 600	675 x 675
2	400 x 700	450 x 450	550 x 550	400 x 800	600 x 600	675 x 675
3	400 x 700	450 x 450	550 x 550	400 x 800	600 x 600	675 x 675
4	400 x 700	450 x 450	550 x 550	400 x 800	600 x 600	675 x 675
5	350 x 600	450 x 450	550 x 550	400 x 800	600 x 600	675 x 675
6	-	-	-	400 x 800	525 x 525	600 x 600
7	-	-	-	400 x 800	525 x 525	600 x 600
8	-	-	-	400 x 800	525 x 525	600 x 600
9	-	-	-	400 x 800	525 x 525	600 x 600
10	-	-	-	350 x 800	525 x 525	600 x 600

III. Results and Discussion

A. Capacity

The capacity curves obtained from static nonlinear pushover analysis for five and ten story frame are shown in Fig. 3 and 4, respectively. It shows that the increase in column-to-beam strength ratio, resulted in the increase of the maximum lateral load and roof lateral deformation. The maximum base shear and lateral roof displacement is obtained for strength ratio of 2.0. Fig. 3 and 4 also show that the capacity curves are the same for all models in each category before yielding occurs. The curves start to deviate after yielding of reinforcement occurs in the structural members.

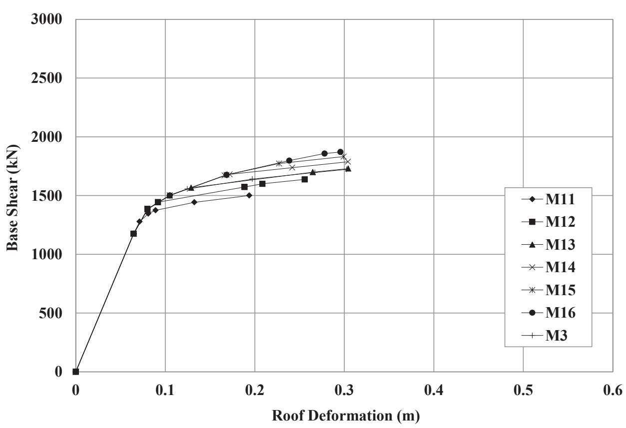


Fig. 3 Capacity curves of 5 story frames

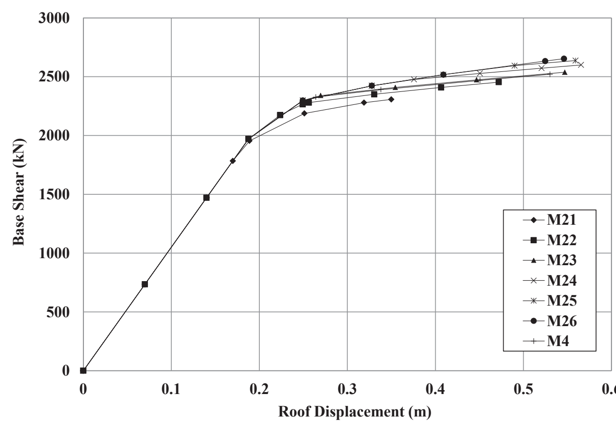


Fig. 4 Capacity curves of 10 story frames

Considering the probable flexural strength (M_{prb}) instead of the nominal flexural strength (M_{nb}) of the beams to obtain the nominal flexural column at joint (M_{nc}) as in M3 and M4 for five and ten story frames, respectively, both models increase their base shear and lateral roof displacement compare to Model M12 and M22, respectively.

Fig. 5 and 6 show lateral story displacement for each model at condition where the target displacement was reached. The story drifts as shown in Fig. 7 and 8 for each model were calculated from the data presented in Fig. 5 and 6. Comparing to drift limitation given in [7] where the maximum story drift calculated using Δ_α = 0,02 h_{sx} = 0.02 x 3.5 m = 0,070 m, all frame models are below the limits. Maximum story drift for 5 and 10 story building is 0.061m which is reached by Model M11 and M21.

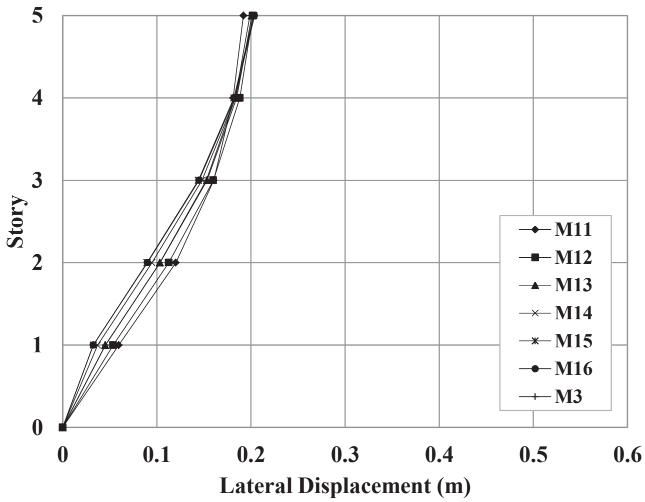


Fig. 5 Lateral displacement of the 5 story frames

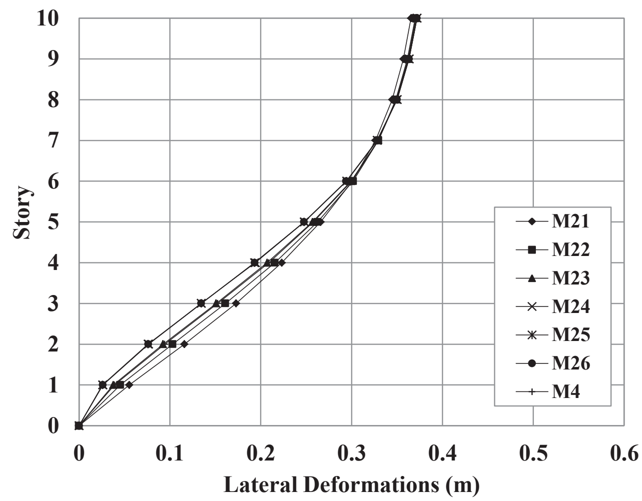


Fig.6 Lateral displacement of the 10 story frames

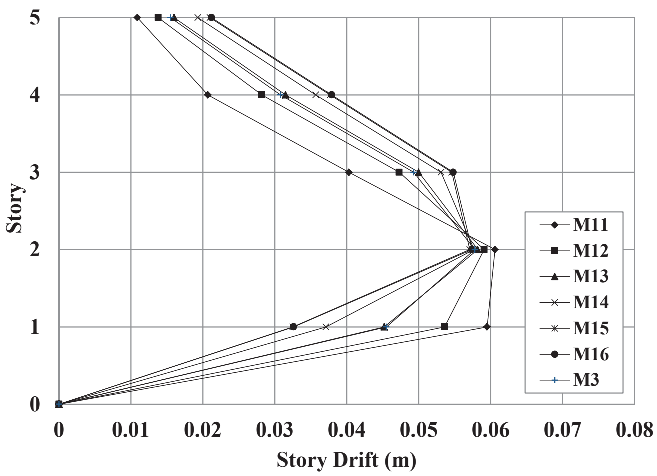


Fig. 7 Story drifts of the 5 story frames

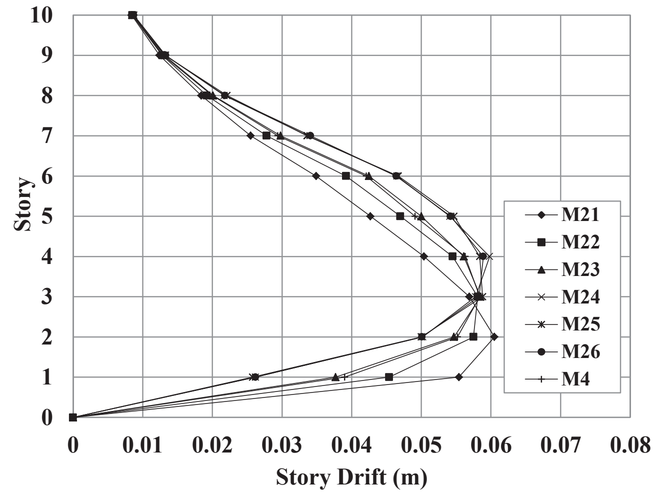


Fig.8 Story drifts of the 10 story frames

B. *Collapse Mechanism*

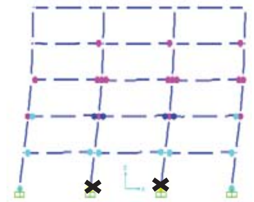
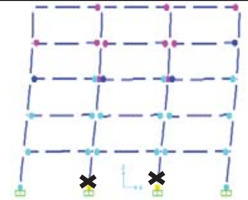
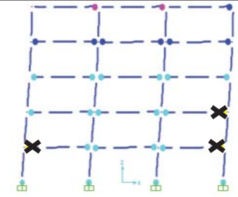
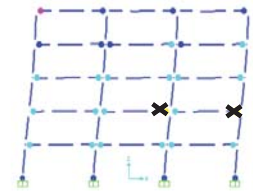
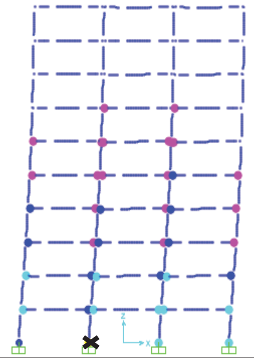
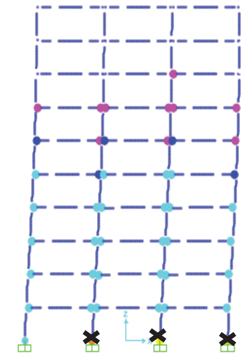
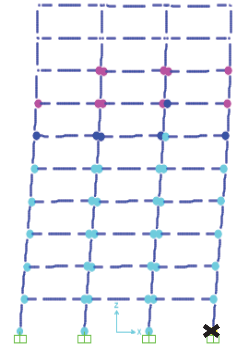
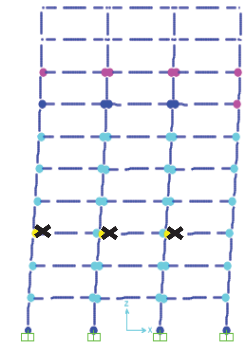
The five story frames M11 and M12 with strength ratio of 1.0 and 1.2, respectively, reach collapse or ultimate plastic hinges at the column base. However, the frame Model M11 reaches first yielding at column before all beams yield. This indicates a *column sway mechanism* (CSM) occurred. However, for frames with strength ratio of 1.4 to 2.0 and $1.2M_{prb}$, the ultimate plastic hinges occur in beams first, therefore, a *beam sway mechanism* (CSM) prevailed. In strength ratio range from 1.4 to 2.0 and 1.2 M_{prb}, the strong-column weak-beam design concept can be achieved.

Collapse mechanism behaviour of 10 story frame is the same as that of 5 story frames, except for frame Model M4 in which the strength ratio was calculated using $1.2M_{prb}$. This frame shows the collapse plastic hinge occurred at column end. Figures in Table III and IV show collapse mechanism of all studied models. Ultimate hinges or collapse hinges in the models are indicated by "x" sign.

Collapse hinges at column base occur in Model M11 and M12 for 5 story frame and in Model M21, M22 and M4 for 10 story frames. This indicates that the strength ratio up to 1.2 for 5 story frame and up to 1.4 for 10 story frame cannot make collapse at beams.

TABLE III

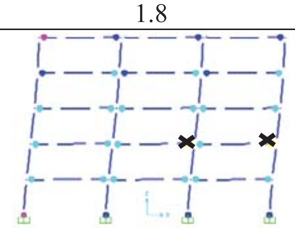
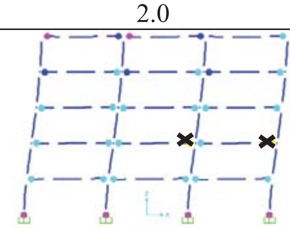
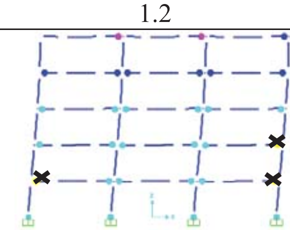
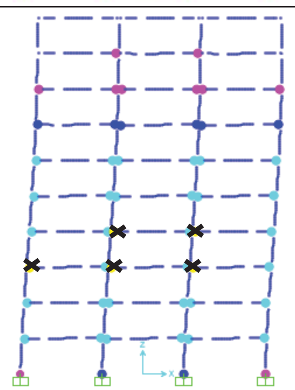
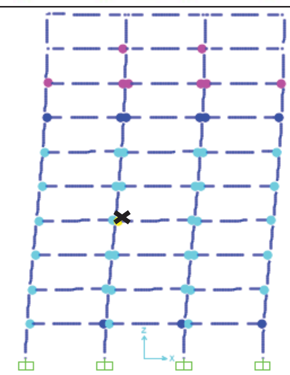
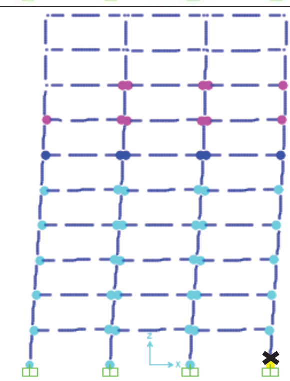
Collapse Mechanism for strength ratio ($\sum M_{nc}/\sum M_{nb}$) = 1.0, 1.2, 1.4, 1.6

Story	Strength Ratio ($\sum M_{nc}/\sum M_{nb}$)			
	1.0	1.2	1.4	1.6
5				
10				

✖ : Plastic hinges reach *ultimate point* or *collapse*

TABLE IV

Collapse Mechanism for strength ratio ($\sum M_{nc}/\sum M_{nb}$) =1.8, 2.0 and strength ratio ($\sum M_{nc}/\sum M_{prb}$) = 1.2

Story	Strength Ratio ($\sum Mnc/\sum Mnb$)		Strength Ratio ($\sum Mnc/\sum Mprb$)
	1.8	2.0	1.2
5			
10			

✖ : Plastic hinges reach *ultimate point* or *collapse*

C. *Performance Evaluation*

Evaluation of the building performance follows [5] and also [8] as given in (1). Summary of the calculation is given in Table V and VI. Both five and ten story frames for column-to-beam strength ratio of 1.2 to 2.0 have performance level of *life safety*. Only Model M21 of 10 story frame with a strength ratio of 1.0 show performance level of *collapse prevention*.

The strength ratio of 1.2 as consider in [1] shows *column sway mechanisms* (CSM) for both 5 and 10 story frames. This result was also found in [2] in which strong column-weak beam concept in frames cannot be achieved.

The largest base shear at reinforcement yields occurs in Model M13 and M23 for 5 and 10 story frames, respectively. The lowest one is in Model M11 and M12. Increasing the strength ratio is not automatically increase the yield base shear. This is due to the first yield hinges occurred as indication of yielding; do not happen in the same location. The base shear at frame reaching displacement target increases with the increase in the strength ratio, however, the values is slightly lower than the ultimate base shear.

The total number of plastic hinges at ultimate condition for 5 and 10 story frames are shown in Table V and VI, respectively. Distribution of hinges to the beams at collapse condition is much better as the strength ratio increase. Comparing the performances of Model M3 with M12 and Model M4 with M22, it shows that by considering probable flexural beams strength to calculate the strength ratio at joint gives better performances.

The changes in the strength ratio are also affects the displacement ductility factor of the frames. The ductility factor is obtained by dividing ultimate displacement with displacement at yielding as shown in Table V and VI. The ductility factor increases significantly for strength ratio increase from 1.0 to 1.4 for both five and ten story frames. The strength ratio increase from 1.0 to 1.2 and from 1.2 to 1.4, the ductility increase about 28% and 14.5%. After these ranges, the strength ratio does not affect the ductility factor significantly. The highest value of ductility factor is still lower than that of expected value for ductile frame which is 5

Comparing the effect of using 1.2 times beam probable moments to calculate column nominal moments as in the Model M3 and M4, results in the ductility factor of Model M3 and M4 increases by 16% and 25% from ductility of M12 and M22, respectively.

TABLE V
SUMMARY OF PERFORMANCE EVALUATION FOR 5 STORY FRAMES

Parameters	**Model**						
	M11	**M12**	**M13**	**M14**	**M15**	**M16**	**M3**
Strength Ratio	1.0	1.2	1.4	1.6	1.8	2.0	1.2 (Mpr,b)
δ_Y (*m*)	0.075	0.078	0.081	0.081	0.079	0.079	0.080
V_Y (kN)	1362.719	1411.913	1466.547	1462.539	1440.118	1437.746	1459.260
δ_T (*m*)	0.192	0.202	0.201	0.203	0.204	0.204	0.199
V_T (kN)	1499.078	1590.947	1637.544	1706.063	1734.704	1737.515	1642.077
δ_U (*m*)	0.194	0.256	0.304	0.304	0.299	0.296	0.303
V_U (kN)	1500.728	1637.679	1730.059	1787.833	1831.271	1870.633	1723.369
μ	2.58	3.29	3.77	3.78	3.78	3.74	3.77
Plastic hinges at δ_T reached	16 B-IO 4 IO-LS 12 LS-CP	6 B-IO 11 IO-LS 11 LS-CP	9 B-IO 11 IO-LS 11 LS-CP	12 B-IO 7 IO-LS 12 LS-CP	7 B-IO 7 IO-LS 13 LS-CP	7 B-IO 7 IO-LS 13 LS-CP	9 B-IO 11 IO-LS 11 LS-CP
Performance Level	LS	LS	LS	LS	LS	LS	LS
Plastic hinges location at δ_U	11 columns 21 beams	4 columns 27 beams	4 columns 27 beams	4 columns 28 beams	4 columns 28 beams	4 columns 28 beams	4 columns 27 beams
Collapse Mechanism	CSM	CSM	BSM	BSM	BSM	BSM	BSM

TABLE VI
SUMMARY OF PERFORMANCE EVALUATION FOR 10 STORY FRAMES

Parameters	**Model**						
	M21	**M22**	**M23**	**M24**	**M25**	**M26**	**M4**
Strength Ratio	1.0	1.2	1.4	1.6	1.8	2.0	1.2 (Mpr,b)
δ_Y (*m*)	0.199	0.209	0.211	0.209	0.209	0.209	0.211
V_Y (kN)	2084.149	2192.541	2219.169	2196.586	2195.170	2195.170	2216.761
δ_T (*m*)	0.370	0.370	0.371	0.372	0.372	0.372	0.371
V_T (kN)	2317.240	2380.493	2420.348	2472.034	2473.987	2473.987	2412.791
δ_U (*m*)	0.350	0.472	0.547	0.566	0.559	0.546	0.530
V_U (kN)	2306.855	2453.920	2537.755	2600.101	2637.213	2653.249	2520.486
μ	1.76	2.26	2.59	2.70	2.68	2.61	2.51
Plastic hinges at δ_T reached	19 B-IO 12 IO-LS 10 LS-CP 1 C-D	15 B-IO 19 IO-LS 9 LS-CP	17 B-IO 21 IO-LS 8 LS-CP	15 B-IO 21 IO-LS 9 LS-CP	12 B-IO 21 IO-LS 9 LS-CP	12 B-IO 21 IO-LS 9 LS-CP	15 B-IO 24 IO-LS 6 LS-CP
Performance Level	C	LS	LS	LS	LS	LS	LS
Plastic hinges location at δ_U	7 columns 36 beams	5 columns 43 beams	4 columns 47 beams	4 columns 48 beams	4 columns 50 beams	0 column 50 beams	4 columns 47 beams
Collapse Mechanism	CSM	CSM	CSM	BSM	BSM	BSM	CSM

IV. CONCLUSIONS

Based on analysis and discussions have been done, the following conclusions can be drawn:

a. The performance level of all studied frames is *life safety* except that the ten story frame with the strength ratio of 1.0 has performance level of *collapse*.
b. A collapse mechanism of *beam sway mechanism* was achieved for strength ratio of 1.4 to 2.0 for 5 story frame Model and 1.6 to 2.0 for ten story frame.
c. The increase in the strength ratio up to 1.4 can increase ductility factor significantly, however, beyond that the strength ratio does not affect the frame ductility both for five and ten story frame models.
d. Considering probable flexural strength (M_{prb}) instead of nominal flexural strength (M_{nb}) of beam to calculate the nominal column strength for strength ratio of 1.2 results in better frame performances. The ductility factor increase by 16% and 25% respectively for five and ten story structure.

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