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AN INVESTIGATION INTO DUCTILITY DEFINITIONS FOR REINFORCED CONCRETE MEMBERS AND FRAMES

H. Daneshvar[1]

ABSTRACT

In this paper, aspects of ductile behavior of reinforced concrete frame structures and individual members are investigated that deemed to be so important in performance-based design philosophy. Different types of analysis, including stress-strain analysis, moment-curvature analysis and pushover analysis were performed and corresponding ductility factors have been obtained. Comparisons are discussed regarding stress-strain curves, moment-curvature and moment-rotation diagrams of members and capacity diagrams of structures. It is shown that there are distinguishing differences in response of ductile members and structures in comparison with non-ductile ones. Various intriguing results are observed and behaviors are compared and contrasted. For example, based on the ACI code design categories for moment-resisting frames, ordinary and intermediate frames' columns behave similarly in terms of stress-strain diagrams while being distinctly different from special frames, a phenomenon that can be attributed to confinement properties, while there is a considerable difference in the moment-curvature diagrams of special frame beams compared to the other two ductility levels.

Introduction

Ductility is indicative of the capability of a structure or its members to deform inelastically without developing an unacceptable decrease in either strength or stiffness. Ductility can be defined quantitatively in several different ways. In general, four different ductility categories are used: material ductility (strain ductility), curvature ductility (cross-section ductility), rotation ductility (member ductility), and displacement ductility (structure ductility, kinematic ductility). The first three definitions are generally related to local ductility of an individual member, and the last is much more related to the global ductility of a system (Wakabayashi, 1986). Reinforced concrete (RC) moment resisting frames (MRF) are used as force-resisting systems in order to resist seismic forces. Different codes define different type of reinforced concrete moment frames as lateral force-resisting systems in order to resist seismic motions. For example ACI 318-05 introduce three type of MRF: special MRF (SMRF), intermediate MRF (IMRF) and Ordinary type (OMRF). The main differences among characteristics of various type of MRF depend on the concept of ductility (ACI318, 2005). Based on definition of ductility, more ductile frames have this capability to resist strong earthquake shaking without significant loss of strength or stiffness. Code's tool to achieve certain level of ductility is special proportioning and detailing requirements (Moehle, Hooper, & Lubke, 2008). For instance ACI code says that because of additional requirements and enhancement, special

[1] PhD Student, Dept. of Civil and Environmental Engineering, University of Alberta, Edmonton, Canada

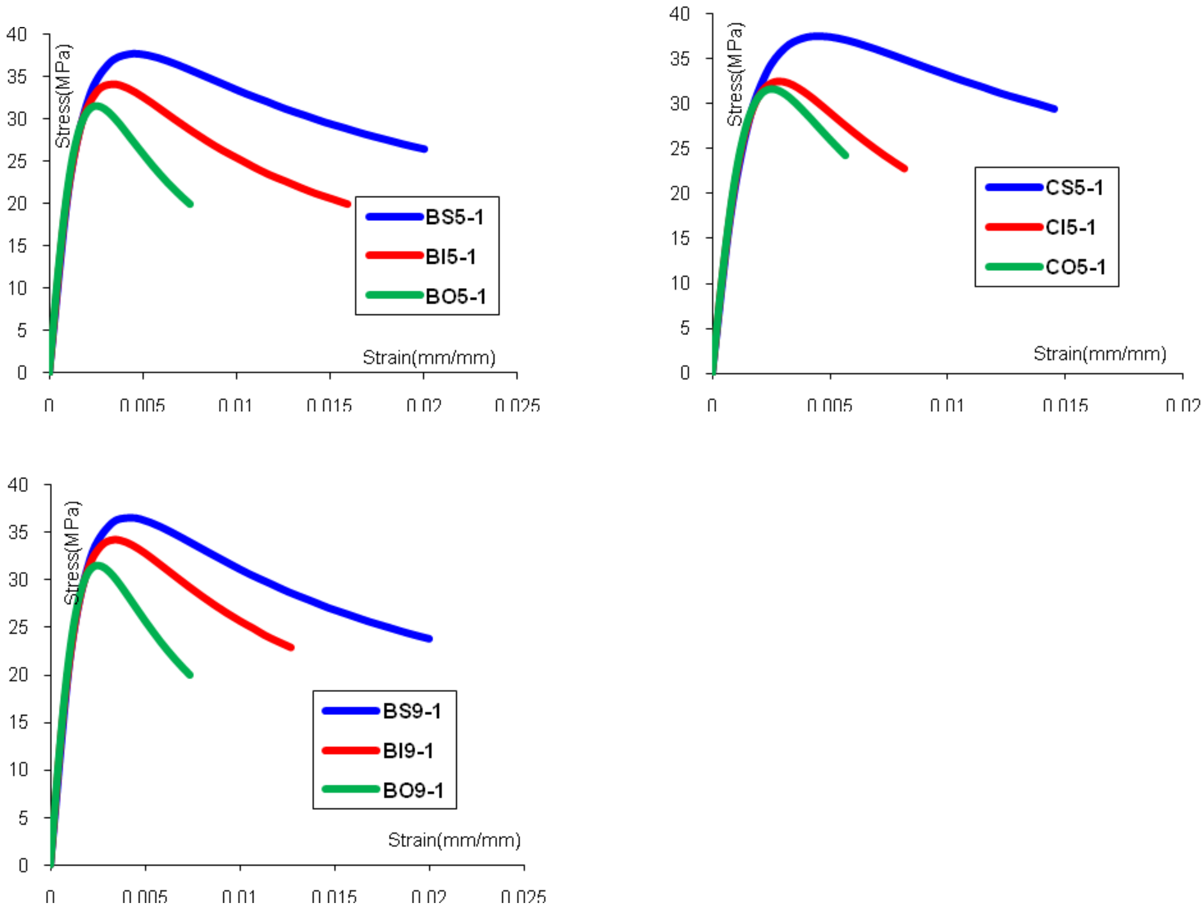
moment resisting frame behave in more desirable manner rather than less detailed frames. However, ACI has not quantified the differences between mentioned types of structures. In other words, a result of such a special requirement in real behavior of structure is under question. In current research, ACI has been selected as the reference. The R factor (seismic force reduction factor) differs among these systems. Basically, R represents the capability of the structure to dissipate seismic energy by going through inelastic behavior and the reliable reserve strength in the structure. The factor is influenced by many parameters such as fundamental period, damping, redundancy as well as ductility and is selected typically by consensus within code-writing bodies. The R factors, 8 for SMRF; 5 for IMRF and 4 for OMRF, are used in the denominator of the equation used to calculate the seismic base shear. The resulting decrease in the amount of seismic load for more ductile MRFs should be accompanied by several enhancements in structural behavior in order to dissipate energy through nonlinear behavior. Based on this fact, ACI318 has distinguished among the different types of MRFs with various parameters. In this research, it is assumed that the main source of ductile behavior in RC members comes from confinement due to transverse reinforcement. There are some restrictions on hoops spacing that lead to ductile behavior of reinforced concrete member which seems to be very important in term of ductility. For instance, maximum hoop spacing for beam in intermediate moment frame shall not exceed the smallest of (a) d/4; (b) Eight times the diameter of the smallest longitudinal bar enclosed; (c) 24 times the diameter of the hoop bar; (d) 12 in. All these clauses have been considered precisely in design and performance evaluation. In addition, other differences that code discussed has been taken in to consideration too. They are summarized in table 1.A1.

Introduction of Case Studies

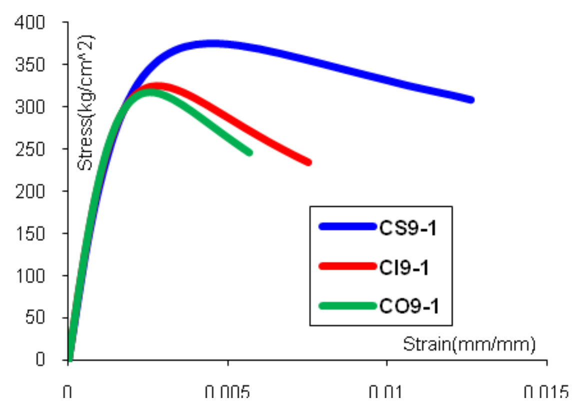
Nine RC-MRFs have been designed based on standard ACI 308-05, in three height levels (five, nine and fifteen storey frames) and three ductility levels (SMRF, IMRF and OMRF) with special attention to chapter 21 which deals with seismic design of MRF buildings as discussed earlier. In general, all design aspects of the buildings are the same except the fact that their design (nominal) ductilities are different. Elevation views of the SMRF, as an example, and the cross sections of all 9 frames are presented in Figure 1.A2 in appendix 2. Material properties that were assumed for concrete and reinforcement are f_c= 30 MPa and f_y= 400 MPa respectively. There are several models in order to simulate confinement in reinforced concrete component. In this research, Mander model has been adopted (Mander, Priestley, & Park, 1988). Investigate in code ductility definition includes stress-strain and moment-curvature analyses of members and push-over analysis of structures. Section analyses were performed by Section Designer Add-on Software of Sap2000 and frame analysis were conducted by Sap2000.

Material ductility or strain ductility

Strain ductility is defined as $\mu_\varepsilon = \varepsilon_u / \varepsilon_y$ in which ε_u and ε_y are ultimate strain and yield strain respectively. Unconfined concrete is able to hold small amount of strain; however, in confined concrete, total strain which concrete can stand before failure increase considerably. Since this type of ductility focuses on each member individually, there is no considerable difference among stress-strain curves of frame’s member for various height levels. Each member in SMRF has been compared by its corresponding member in other two types of frames. In figure 1, the results of one beam and one column for each height level have been brought. As it is observed , as we go higher in ductility based on code definition, the area under the stress-strain curve that somehow define cross section capability to absorb energy increase considerably.



(a)



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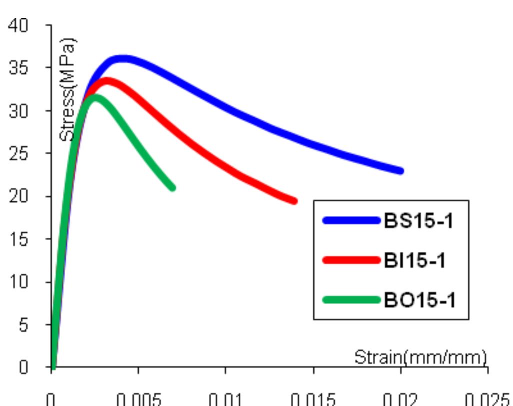
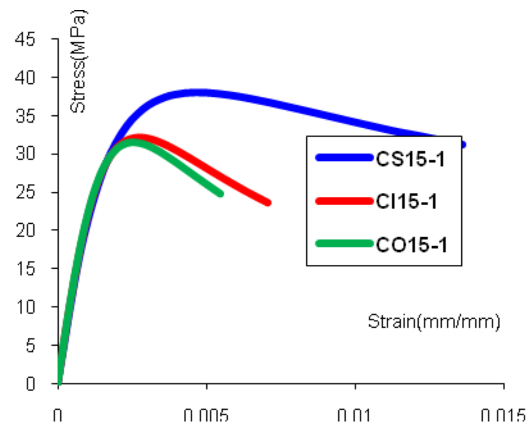




(c)

Figure 1. Stress-strain curves of one beam and one column of (a) five storey (b) nine storey (c) fifteen storey frames

There is an increase in strength that is approximately 30 percent for special moment resisting frames, 15 percent for intermediate and 5 percent for ordinary cross sections due to better confinement. This characteristic is another proof for Mander model capability for modeling confinement in reinforced concrete. Ultimate strain and corresponding strength before failure of cross section increase in special and intermediate members. Although it is obvious that stress-strain curve of intermediately designed members are always among the lower and upper bond of special and ordinary member behavior, aforementioned curve is intended to special member behavior in beams and much more closer to ordinary member performance in columns. Study all the stress-strain curves for all members of case study buildings result in table 1.

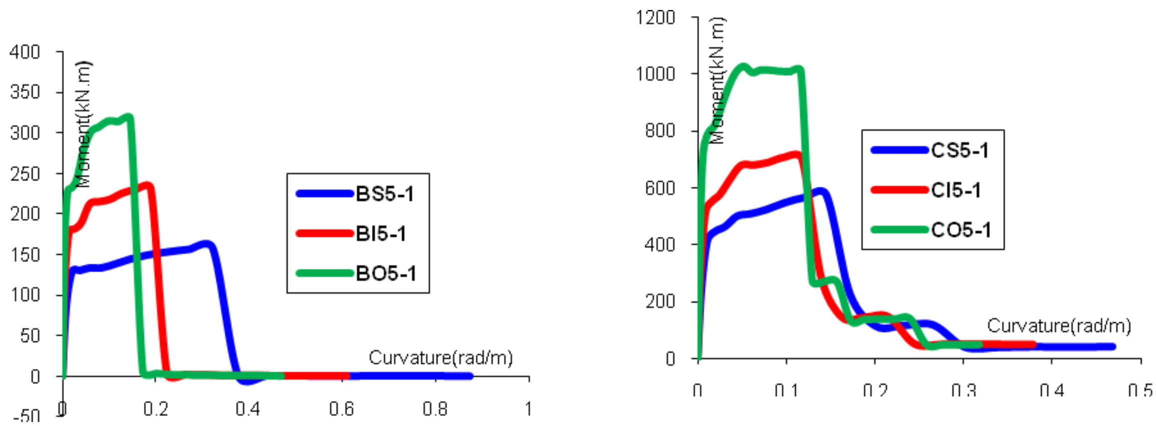
Table 1. Strain Ductility Summary

MRF Type	Member Type	Strain Ductility Factor
SMRF	Beams	8.5-9
	Columns	6-7.5
IMRF	Beams	5.5-7.5
	Columns	3.5-4
OMRF	Beams	3
	Columns	2.5

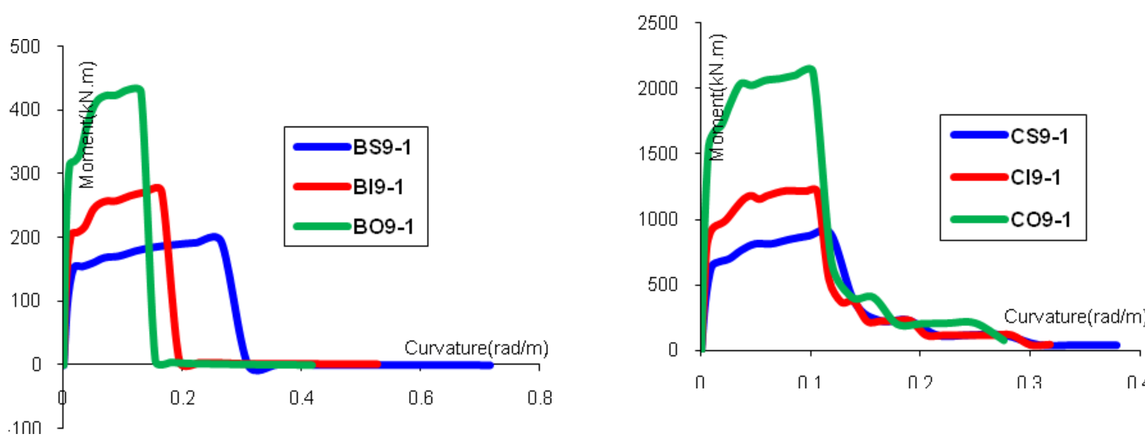
Curvature ductility and cross section ductility

Curvature ductility is one of the most important sources for studying inelastic displacements in structure. It is defined as $\mu_\Phi = \Phi_u / \Phi_y$ in which Φ_u and Φ_y are ultimate curvature and yield curvature respectively. Similar to strain ductility, there is no considerable difference among moment-curvature curves of frame's member for various height levels. The main goals of moment-curvature analysis of reinforced concrete in this research are first, determination of moment-curvature curves as well as curvature ductility factors for sections which are supposed to behave in different ductile manner; second, use these results as input for nonlinear analysis of frames in order to approach the real behavior of structure. Each member has been compared with its corresponding members in two other categories. Figure 2 shows the results of one beam and one column for each height level. It is observed that OMRF members behave in ductile manner similar to SMRF and IMRF members due to the fact that all members first designed to avoid brittle failure happened. SMRF beams have better performance compare to the other two categories. Since platic hinge definition attains from these graphs, it is another clue for moving through higher ductile behavior. In columns, although difference is distinguishable among three types of MRF's member, it is not significant, contrary to beams' behavior. Study all the stress-strain curves for all members of case study buildings result in table 2.

As mentioned earlier, in this research plastic hinge definitions are based on results of moment curvature analysis with assuming the plastic hinge length of h/2, half of section's depth (ATC, 1996). Real moment-rotation behaviors of plastic hinges have been assigned to probable formation location for determination of whole frame performance. As a general rule, vertical loads cause formation of plastic hinges along span of members. In this research, we consider low value of gravity load to force plastic hinges to form at member ends in order to probe into structural performance upon seismic loading.



(a)



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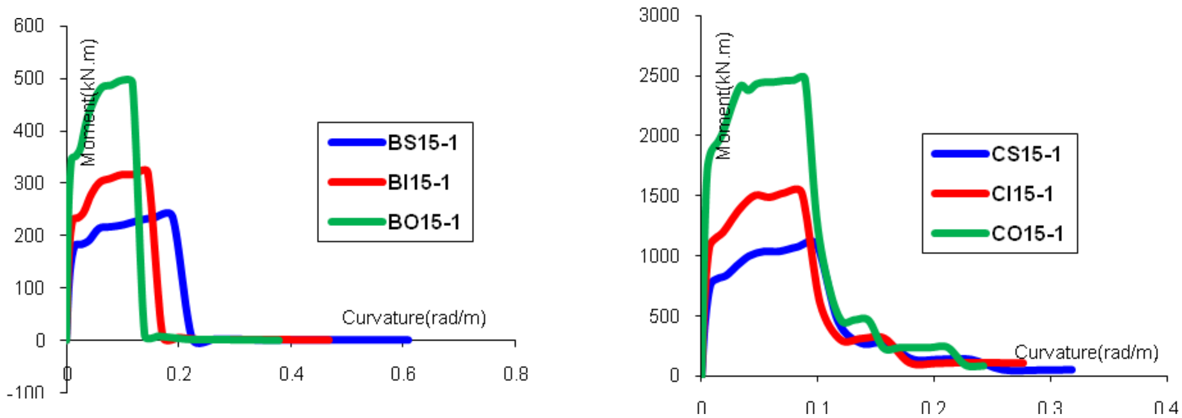

(c)

Figure 2. Moment-curvature curves of one beam and one column of (a) five storey (b) nine storey (c) fifteen storey frames

Table 2. Strain Ductility Summary

MRF Type	Curvature Ductility Factor - Beams
SMRF	20
IMRF	15
OMRF	14

Displacement ductility or structure ductility

It is defined as $\mu_\Delta = \Delta_u / \Delta_y$ which Δ_u and Δ_y are ultimate displacement and yield displacement respectively. $\Delta_u = \Delta_y + \Delta_p$. Δ_y is roof drift corresponding to yielding of structure at base and Δ_p is roof displacement according to plastic behavior. Formation of plastic hinge at the ends of beams and column hugely influences the displacement ductility factor. Displacement ductility factors are determined based on the results of nonlinear static analysis of the structures. Pushover analysis has been performed according to FEMA 356 guidelines (FEMA, 2000). Three gravity load cases, which have been considered as an initial condition for pushover analysis, are:
GR1: 0.9D (Lower bound), GR2: 1.1D+1.1L (Upper bound) and GR3: D+0.2L (Intermediate)
The structures were pushed using three lateral load patterns including triangular-shaped, first real mode and uniform. As observed in all capacity curves in figure 3, decrease in ductility of structure result in increase in base shear as well as lower displacement before failure. There is good coincidence between triangular pattern result and first mode shape ones although just the results of first mode pattern have been shown in figure3. In addition, the maximum gravity load case is deemed to be the critical gravity load case. Table 3 shows the values for the structure displacement ductility factors for the case study buildings.

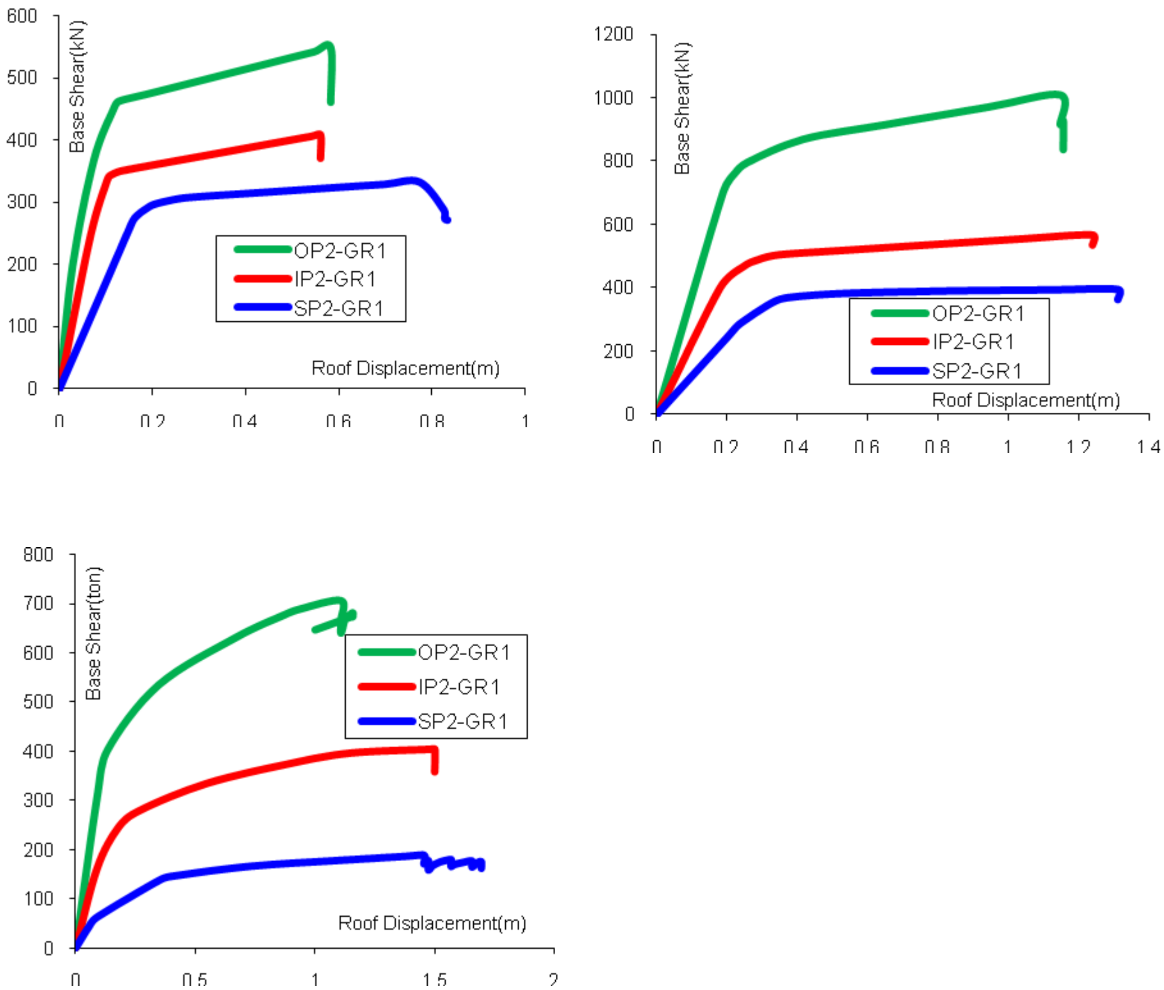

Figure 3. Capacity curves of (a) five storey (b) nine storey (c) fifteen storey frames

Table 3. Strain Ductility Summary

MRF Type	Frame Type	Displacement Ductility Factor
Low & Medium Rise	SMRF	7.5
	IMRF	5.5
	OMRF	5
High Rise	SMRF	17
	IMRF	15
	OMRF	5

Conclusions

Based on the ACI code design categories for moment-resisting frames, special and intermediate frames' members behave similarly in terms of stress-strain diagrams while being distinctly different from ordinary frames, a phenomenon that can be attributed to confinement properties, while there is a considerable difference in the moment-curvature diagrams of special frame members compared to the other two ductility levels. The lower the ductility, the higher the base shear that the structure can withstand, and the smaller the displacements it can stands before mechanism. In performance-based design philosophy, approaching the real behavior of the structure is desirable. There is a necessity to go through modeling the real behavior of the structural components rather than stick to code's definitions in order to investigate on the topic of ductility.

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Appendix 1

Table 1.A1. Differences among various types of MRFs according to ACI318-05 in American units (CSI, 2008)

Type of Check/Design	Ordinary MRF	Intermediate MRF	Special MRF
Column Design (interaction)	1 %<ρ< 8%	1 %<ρ< 8%	1 %<ρ< 6% and α=1.0
Column Shears	Specified Combination	Modified Combination (earthquake loads doubled)	Specified Combination
Beam Design Flexure	----	Column shear Capacity Φ=1.0 and α=1.0	Column shear Capacity Φ=1.0 ,α=1.0 and Vc = 0
	$\rho \leq 0.04$	$\rho \leq 0.04$	$\rho \leq 0.025$
Beam Min. Moment Override Check	$\rho \geq 3\sqrt{f'c} / f_y$, $\rho \geq 200/ f_y$	$\rho \geq 3\sqrt{f'c} / f_y$, $\rho \geq 200/ f_y$	$\rho \geq 3\sqrt{f'c} / f_y$, $\rho \geq 200/ f_y$
	$\rho \geq 3\sqrt{f'c} / f_y$, $\rho \geq 200/ f_y$	$\rho \geq 3\sqrt{f'c} / f_y$, $\rho \geq 200/ f_y$	$\rho \geq 3\sqrt{f'c} / f_y$, $\rho \geq 200/ f_y$

	No Requirement	$M^{+}_{uend} \geq ⅓M^{-}_{uend}$	$M^{+}_{uend} \geq ½M^{-}_{uend}$
Beam Design Shear	Specified Combination	$M^{+}_{uspan} \geq ⅕\max\{M^{+}_{u}, M^{-}_{u}\}$	$M^{+}_{uspan} \geq ¼\max\{M^{+}_{u}, M^{-}_{u}\}$
		$M^{-}_{uspan} \geq ⅕\max\{M^{+}_{u}, M^{-}_{u}\}$	$M^{+}_{uspan} \geq ¼\max\{M^{+}_{u}, M^{-}_{u}\}$
		Modified Specified Combination (earthquake loads doubled)	Specified Combination
Joint Design	----	Beam Capacity Shear with Φ=1.0 and α=1.0 plus VD+L	Column shear Capacity Φ=1.0 ,α=1.25 plus VD+L and Vc = 0

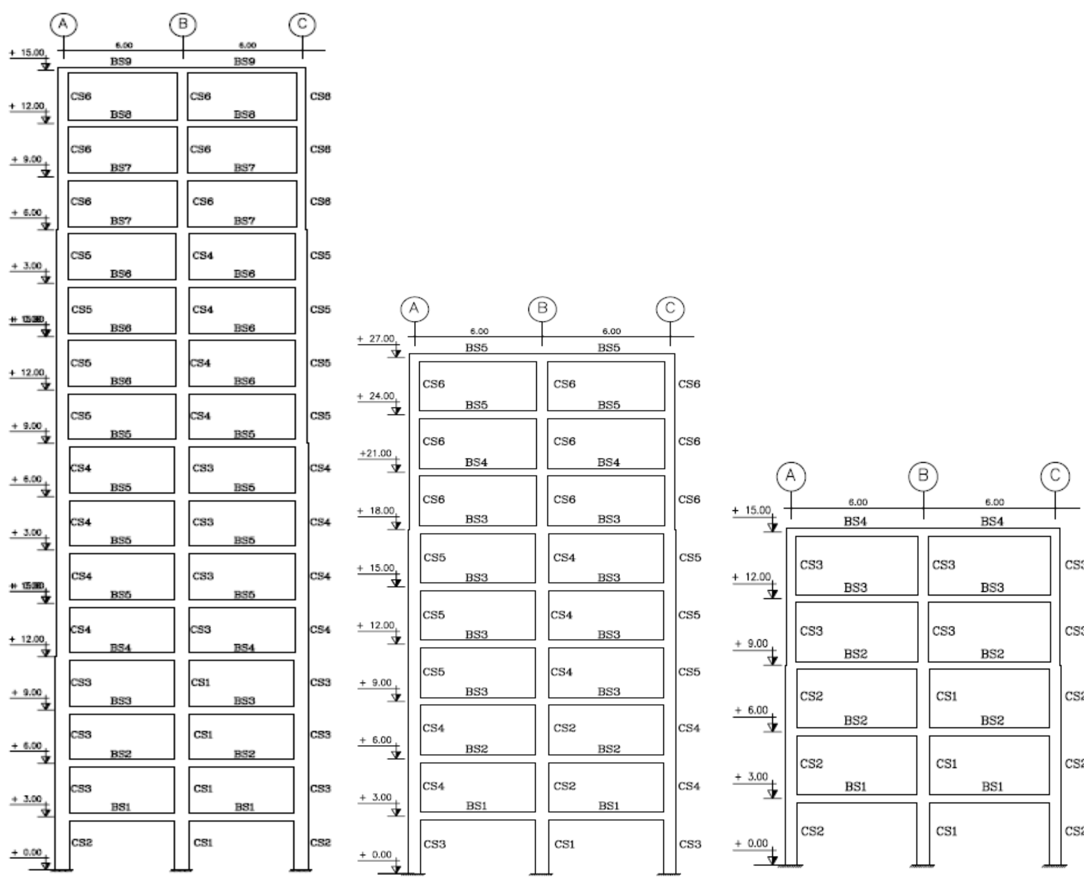

Figure 1.A2 Elevation-MRFs

Table 1.A2. MRF Cross sections for (a) five storey (b) nine storey (c) fifteen storey frames according to ACI318-05

Label	Dimension	No. Top Reinf.	No. Bottom Reinf.	Hoop spacing	Label	Dimension	No. Total Reinf.	Hoop spacing
BS5-1	350x300	4	2	75	CS5-1	550X550	16	125
BS5-2	350x300	5	3	75	CS5-2	500x500	12	125

BS5-3	350x250	4	2	75	CS5-3	450x450	8	113
BS5-4	350x250	4	2	75	CI5-1	650X650	16	160
BI5-1	450X350	4	2	100	CI5-2	600X600	12	160
BI5-2	400X300	4	2	87.5	CI5-3	550X550	12	160
BO5-1	550X400	4	3	-	CO5-1	750X750	20	-
BO5-2	550X400	5	3	-	CO5-2	700X700	20	-
BO5-3	450X350	4	2	-	CO5-3	650X650	16	-

(a)

Label	**Dimension**	**No. Top Reinf.**	**No. Bottom Reinf.**	**Hoop spacing**	**Label**	**Dimension**	**No. Total Reinf.**	**Hoop spacing**
BS9-1	400x300	4	2	87.5	CS9-1	650X650	20	125
BS9-2	400x300	5	3	87.5	CS9-2	650X650	16	125
BS9-3	400x300	6	3	87.5	CS9-3	600X600	20	125
BS9-4	400x250	5	3	87.5	CS9-4	600X600	16	125
BS9-5	400x250	4	2	87.5	CS9-5	550X550	12	125
BI9-1	500x350	4	2	113	CS9-6	500X500	8	125
BI9-2	500x350	5	3	113	CI9-1	750X750	24	160
BI9-3	500x350	6	4	113	CI9-2	750X750	20	160
BI9-4	450x350	6	3	100	CI9-3	700X700	24	160
BI9-5	450x300	5	2	100	CI9-4	700X700	20	160
BI9-6	450x300	4	2	100	CI9-5	650X650	16	160
BO9-1	600x400	5	3	-	CI9-6	600X600	12	160
BO9-2	600x400	7	5	-	CO9-1	850X850	24(Φ25)	-
BO9-3	600x400	8	6	-	CO9-2	850X850	24	-
BO9-4	550x400	7	5	-	CO9-3	800X800	24(Φ25)	-
BO9-5	500x350	6	4	-	CO9-4	800X800	24	-
BO9-6	500x350	5	3	-	CO9-5	750X750	20	-
					CO9-6	700X700	16	-

(b)

Label	**Dimension**	**No. Top Reinf.**	**No. Bottom Reinf.**	**Hoop spacing**	**Label**	**Dimension**	**No. Total Reinf.**	**Hoop spacing**
BS15-1	450X350	4	2	100	CS15-1	750X750	20	125
BS15-2	450X350	5	3	100	CS15-2	700X700	24	125
BS15-3	450X350	6	3	100	CS15-3	700X700	20	125
BS15-4	450X350	6	4	100	CS15-4	650X650	16	125
BS15-5	450X300	6	4	100	CS15-5	600X600	16	125
BS15-6	400X300	6	3	87.5	CS15-6	500X500	12	125
BS15-7	400X300	5	3	87.5	CI15-1	850X850	16(Φ25)	160
BS15-8	350X250	5	2	87.5	CI15-2	800X800	20(Φ25)	160
BS15-9	350X250	4	2	87.5	CI15-3	800X800	16(Φ25)	160
BI15-1	550X400	4	3	125	CI15-4	750X750	12(Φ25)	160
BI15-2	550X400	5	3	125	CI15-5	700X700	12(Φ25)	160
BI15-3	550X400	6	4	125	CI15-6	650X650	12(Φ25)	160
BI15-4	550X400	6	5	125	CO15-1	950X950	24(Φ25)	-
BI15-5	550X350	6	4	125	CO15-2	900X900	28(Φ25)	-
BI15-6	500X350	6	4	113	CO15-3	900X900	24(Φ25)	-
BI15-7	500X350	5	3	113	CO15-4	850X850	16(Φ25)	-

BI15-8	450X350	4	2	113	CO15-5	800X800	16(Φ25)	-
BO15-1	650X450	5	4	-	CO15-6	750X750	12(Φ25)	-
BO15-2	650X450	7	5	-				
BO15-3	650X450	8	6	-				
BO15-4	650X450	8	7	-				
BO15-5	650X400	8	7	-				
BO15-6	600X400	7	5	-				
BO15-7	600X400	6	4	-				
BO15-8	550X400	5	3	-				
BO15-9	550X400	4	3	-				

* : Table All dimensions are in mm.
** : Transverse and longitudinal rebars have nominal diameter of 10mm and 20mm respectively.

(c)