RATIONAL SELECTION OF STRUCTURAL DUCTILITY CAPACITY AND REINFORCEMENT DETAILS FOR SEISMIC DESIGN OF REINFORCED CONCRETE SHEAR WALL–FRAME STRUCTURE

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Abstract

This paper discussed the rational selection of structural ductility capacity and reinforcement details in reducing reinforcement demand for seismic resistance design of reinforced concrete building. In seismic resistance design, ductility capacity and inherent overstrength of lateral-force-resisting system is represented in seismic reduction factor R. For a reinforced concrete shearwall–frame structure, a range of ductility capacity or seismic reduction factor is available. To select high structural ductility capacity will result low seismic design force but requires special reinforcement details. However for low structural ductility capacity, ordinary reinforcement details may be sufficient but high seismic design force are expected. High seismic design force as well as special reinforcement details increase reinforcement quantity. In this study, a 12–storey of reinforced concrete shear wall–frame structure at the region of moderate seismic risk is analyzed as dual system (shear wall–frame system) or single system (shear wall system). The structure is designed in accordance with the Indonesian Building Code for Seismic Resistance Design. The study has been focussed on limited or full ductility structure with intermediate or special reinforcement details respectively. Results shown that single or dual lateral-force-resisting system with limited ductility and intermediate reinforcement details (IMRF) only required minimum reinforcement quantity.

Keywords: structural ductility capacity, seismic reduction factor, reinforced concrete shear wall – frame structure, dual system, Special Moment Resisting Frame (SMRF), Intermediate Moment Resisting Frame (IMRF).

Abstrak

Makalah ini membicarakan mengenai pemilihan secara rasional dari daktilitas struktur dan detail perkuatan dalam mereduksi keperluan perkuatan pada design struktur tahan gempa dari bangunan beton bertulang. Dalam perencanaan bangunan tahan gempa, daktilitas struktur dipresentasikan dalam faktor reduksi R. Untuk struktur dinding geser beton bertulang, interval nilai factor reduksi dapat ditemukan. Struktur dengan daktilitas yang tinggi menghasilkan gaya gempa yang kecil tetapi membutuhkan detail perkuatan khusus. Sebaliknya, untuk kapasitas struktur dengan daktilitas kecil, perkuatan klasik akan cukup tetapi menggunakan gaya gempa yang besar. Pada kasus ini, struktur dinding geser beton bertulang 12 lantai pada lokasi gempa yang moderat dianalisa sebagai dual system (sistem dinding geser – portal sistem) atau single system (system dinding geser). Struktur ini direncanakan berdasarkan Peraturan Gempa untuk Indonesia. Studi ini memfokuskan pada daktilitas terbatas ataupun daktilitas penuh dari struktur dengan *intermediate* atau perkuatan khusus. Hasil analisa menunjukkan bahwa sistem tunggal ataupun dual sistem dengan daktilitas terbatas dan sistem portal penahan momen menengah membutuhkan perkuatan minimal.

Kata kunci: daktilitas struktur, factor reduksi gempa, sistem dinding geser – portal beton bertulang, dual sistem, Sistem Rangka Pemikul Momen Khusus (SMRF), Sistem rangka pemikul momen menengah (IMRF)

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1. INTRODUCTION

Reinforcement provision for seismic resistance design of reinforced concrete structure is mostly attributed to the seismic design force and reinforcement details. Both factors are mainly governed by seismicity of the site and ductility capacity of the structure (Paulay, 1991). In high seismic region, lateral force induced by earthquake motions as well as seismic risk increases. At that region, all structural reinforced concrete members shall satisfy special reinforcement details. However, in regions of low and moderate seismic risks, seismic force is relatively at low or moderate levels. Ordinary or intermediate reinforcement details may be sufficient.

Seismic design force could be reduced by increasing structural ductility capacity which is (together with overstrength factor) expressed in seismic reduction factor (R). For a reinforced concrete structure consisting of shear wall and frame systems, its ductility capacity is determined by interaction between two subsystems as well as by capacity of the frame system as independent single system to resist design base shear. If lateral force in the frame system and its capacity in resisting seismic design force is relatively low, the structural system can be considered as a single system (shear wall system) and the ductility capacity of the whole structure is governed by ductility capacity of the shear wall. However, if its capacity is increased, the structure behaves as dual system (shear wall-frame system) and its ductility capacity also increases (Paulay, 1992).

Ductility capacity depends on the development of inelastic deformations throughout the structure. If all storeys of a multi-storey building can be fully mobilized into the inelastic action, its ductility capacity increases. In reinforced concrete structures, this can be achieved only if vertical members (columns and walls) remain essentially elastic in all storeys, with the exception of the base of the bottom storey. However all beams are allowed to undergo flexural inelastic deformations and significant rotation may develop and be followed by plastic hinging. Overdesign of columns and shear walls is typically achieved by application of the capacity designing rule. And special reinforcement details must be provided to ensure the formation of plastic hinges in the structure (CEB, 2000).

Reinforcement details of reinforced concrete members for the regions of low and moderate seismic risks can be designed in accordance with the ordinary, intermediate or special requirements respectively. For those regions, reinforcement provision for seismic design could be significantly reduced by providing adequate ductility to the structure and by selecting appropriate reinforcement details. Therefore, rational selection of both parameters could result an optimal seismic design.

To investigate influence of the selection of those parameters in reducing the quantity of reinforcement, comparative study of reinforced concrete shear wall-frame structure of 12-storey (at the region of moderate seismic risk, zone 3) will be carried out. This structure will be designed as dual or single system in accordance with the Indonesian Building Code for Seismic Resistance Design (SNI 03-1726-2002, 2002; SNI 03-2847-2002, 2002). As dual system, various combinations of structural ductility capacity and associated reinforcement details, i.e.: full ductility (R = 8.5) with Special Moment Resisting Frame (SMRF), weighted ductility (weighted R) with SMRF and limited ductility (R = 6.5) with Intermediate Moment Resisting Frame (IMRF) will be studied and compared. The structure is also designed as a single system (shear wall system) with limited ductility (R = 5.5) and IMRF.

2. SHEAR WALL–FRAME STRUCTURE FOR SEISMIC RESISTANCE

The most commonly used reinforced concrete structural systems for multistory buildings consist of structural frame system, structural wall system or dual system interacting structural frame system and structural wall system. Structural frames with an essentially complete space frames carry significant gravity loads while provide adequate resistance to lateral loads primarily by flexural action of members. When lateral loads are far higher than gravity loads, structural wall system is more suitable as lateral–load–resisting system. That structural system without a complete vertical load-carrying space frame provides support for all or most gravity loads and resistance to lateral loads by shear walls.

In some buildings the entire earthquake force resistance will be provided by both frames and walls (core walls). These are defined as dual or hybrid system. Shear walls and frames are designed to resist lateral forces in proportion to their relative rigidities, considering interaction between shear walls and frames on all levels. Their interaction is governed by rigidity of floor acting as diaphragm. When very rigid floor is connecting both subsystems, compatible storey deflections are assured especially at the lower half of the structure where shear wall system resists significant lateral forces. However, that condition is more difficult to achieve at the upper half of the structure where both subsystems deform independently. At those levels, frames tend to support lateral forces larger than the total lateral design forces. Consequently, walls near the top of the building are subjected to negative forces.

If base shear subjected to the frame in interaction with shear wall is less than 10 % of the total design base shear or without complete space frame, that structural system can be considered as shear wall system. However, shear walls and frames systems shall be considered as dual system if they satisfy the following conditions (UBC, 1997; SNI 03-1726-2002, 2002):

- An essentially complete space frame that provides support for gravity loads.
- Resistance to lateral load is provided by shear walls and structural frames. The latter must be designed to independently resist at least 25 percent of the design base shear.
- The two subsystems shall be designed to resist the total design base shear in proportion to their relative stiffness considering the interaction of the dual system at all levels.

For dual system, its structural ductility must be determined according to the structural ductility of frames and shear walls. When coupled shear walls in combination with frames are used, high structural ductility capacity is obtained because it allows formation of most plastic hinges in link beams and frames. For single system (shear wall system), structural ductility capacity must be limited because plastic hinges is only allowed at the base of the walls and at the link beams for coupled shear walls (Paulay, 1992).

3. DYNAMIC RESPONSE OF BUILDINGS

Differential equation governing the response of an MDF system to earthquake–induced ground motion can be written as (Chopra, 2000)

$$M\ddot{U} + C\dot{U} + KU = -M\ell\ddot{U}_g \tag{1}$$

where M, C and K are mass, damping and stiffness matrices respectively. $\ddot{U}$, $\dot{U}$ and U are acceleration, velocity and displacement vectors. $\ddot{U}_g$ and ℓ represent ground acceleration and its direction.

The most common and effective approach for linear structural system seismic analysis is the mode superposition method. This approach assumes the dynamic response of the system can be expressed as superposition of modal contributions

$$U = \sum_{n=1}^{N} \phi_n q_{n(t)} \tag{2}$$

where ϕ_n and q_n are dynamic modes and modal responses. In modal space, equation (1) can be rewritten as

$$\ddot{q}_n + 2\xi_n\omega_n\dot{q}_n + \omega_n^2 q_n = -\phi_n^t M\ell\ddot{U}_g / M_n \qquad n = 1, ..., N \qquad (3)$$

where ξ_n and ω_n represent modal damping ratios and natural frequencies and $M_n = \phi_n^t M\phi_n$

The maximum modal response and modal displacement can be calculated by using response spectra method

$$q_n^{max} = \phi_n^t M\ell S_a / M_n \text{ and } U_n^{max} = \phi_n q_n^{max} \qquad n = 1, ..., N \qquad (4)$$

S_a represents acceleration response spectrum. To obtain total displacements or total structural responses of all considered modes, modal combination method such as SRSS method or CQC method can be applied.

Indonesian Building Code for Seismic Resistance Design requires that at least 90 percent of the participating mass is included in the calculation of response for each principal horizontal direction and rotation in vertical direction. Also, the fundamental vibration mode shall be in translation (SNI 03-1726-2002, 2002). The uncertainty of seismic direction applied to the structure can be anticipated by considering seismic forces in two orthogonal directions. Orthogonal effects must be defined in accordance with 100/30 rule where 100 percent of prescribed seismic forces in one principal direction plus 30 percent of prescribed seismic forces in the perpendicular direction. The alternative method, which uses the SRSS combination of two 100 percent of prescribed seismic forces in any two orthogonal directions has been found as the most realistic combination (Wilson, 1996).

Acceleration response spectrum can be determined from

$$S_a = \frac{CI}{R} g \qquad (5)$$

where C = seismic response spectrum, I = importance factor, g = gravity acceleration and R = seismic reduction factor. For dynamic analysis, Indonesian Building Code for Seismic Resistance Design (SNI 03-1726-2002, 2002) requires that the dynamic design base shear must be not less than 80 percent of design base shear calculated from first modes V_1

$$V_1 = \frac{C_1 I}{R} W_t \qquad (6)$$

where C_1 = value of seismic response spectrum for first modes and W_t = gravity loads (dead loads plus reduced live loads).

4. SEISMIC REDUCTION FACTOR

One of most important parameters to define the total design base shear is seismic reduction factor R. It represents inherent overstrength factor f_1 = 1.6 and avalaible ductility μ of the different structural systems.

$$R = f_1\mu \qquad (7)$$

As structural ductility varies from μ = 1 (full elastic structure) to μ = 5.3 (full ductile structure), seismic reduction factor has a range from R = 1.6 to R = 8.5. Table 1 shows a list of ductility and corresponding seismic reduction factor (Wangsadinata, 2002).

As shown in Figure 1, nominal design base shear V_n is determined by dividing (scaling down) the elastic base shear V_e by appropriate seismic reduction factor. And associated structural ductility μ is defined as ratio between maximum displacement Δ_m and displacement when 1st plastic hinge Δ_y occurred in the structure.

$$V_n = \frac{V_e}{R} = \frac{CI}{R} W_t \qquad \mu = \frac{\Delta_m}{\Delta_y} \tag{8}$$

Table 1. Structural Ductility μ and Seismic Reduction Factor R

Categories of Structural Response	μ	R
Full Elastic	1.0	1.6
Limited Ductile	1.5	2.4
	2.0	3.2
	2.5	4.0
	3.0	4.8
	3.5	5.6
	4.0	6.4
	4.5	7.2
	5.0	8.0
Full Ductile	5.5	8.5

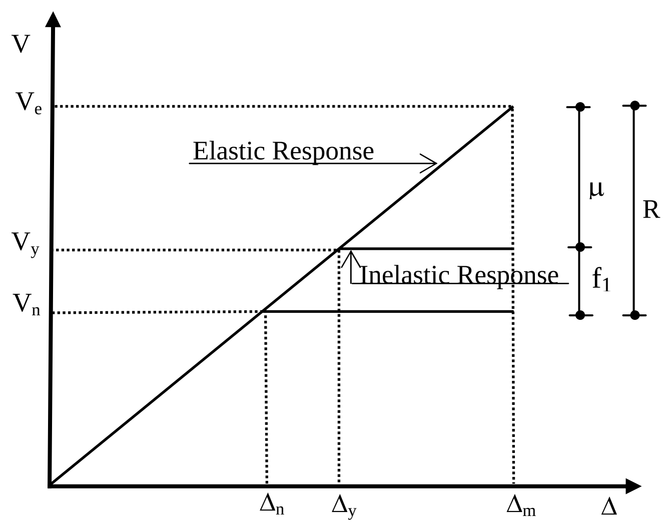


Figure 1. Typical Curve of Base Shear V vs Displacement Δ

For basic structural systems such as bearing wall system, building frame system, moment–resisting frame system or dual system seismic reduction factor R can be found in the code. However, if structural system at any principal direction consists of some subsystems with different structural ductilities, seismic reduction factor in that direction can be calculated using the weighting method (SNI 03-1726-2002, 2002; Wangsadinata, 2002).

$$R_w = \frac{\sum_s V_{ws}}{\sum_s V_{ws} / R_{ws}} \qquad w \in (x,y) \tag{9}$$

where V_{ws} and R_{ws} are base shear and seismic reduction factor of subsystem s in w direction.

5. DETAILING OF REINFORCEMENTS

Reference is made to SNI 03-2847-2002 (2002). Similar to ACI 318-89 (ACI Manual, 1997), no explicit capacity design requirements are imposed on frames according to the target seismic performance level. Capacity design is implicit in prescribing that certain performance categories dictate the use of certain types of frame. There are three types of frame:

- Ordinary Moment Resisting Frames (OMRF) – frames designed and detailed with no seismic provision.
- Intermediate Moment Resisting Frames (IMRF) – frames designed and detailed for structures in areas of moderate seismic hazard, in addition to all requirements of ordinary moment frames
- Special Moment Resisting Frames (SMRF) – frames designed and detailed to the full special seismic provision in areas of high seismic hazard.

There is a strong relationship between strength and ductility. Increase in strength or ductility means increase in seismic safety. Full ductile response is the aim for structures in high seismic risk areas while strength design is the objective for structures in moderate risk areas. In terms of seismic reduction factor and seismic zone, seismic design requirements for reinforced concrete structures in accordance with the code may be determined as a function of both parameters. Therefore, detailing of reinforcement could be proposed in Table 2.

Table 2. Seismic Design Requirements

Seismic Reduction Factor	Zone 1 & 2	Zone 3 & 4	Zone 5 & 6
$1.6 \leq R < 2.4$	OMRF	IMRF	SMRF
$2.4 \leq R < 6.4$	IMRF	IMRF	SMRF
$6.4 \leq R \leq 8.5$	SMRF	SMRF	SMRF

There are five aspects in the code that may be classified strictly under capacity design (SNI 03-2847-2002, 2002; CEB, 2000).

- The probable flexural strength, used to evaluate design shear forces is evaluated using steel yield stress of 1.25 times the specified yield stress and no strength reduction factor.
- For structures in high seismic risk areas, the design shear force is evaluated not from applied loads but from the probable design of the member under consideration. In areas of moderate risk, the design shear force is the largest of (a) that corresponding to nominal flexural strength and (b) from analysis under factored loads. Walls and diaphragms are exempt even in high seismic risk areas.
- When the axial load on frame member is less than fc’Ag/20, the contribution of axial load to concrete shear resisting mechanism is ignored
- The actual yield strength of tensile reinforcement should not be more than 120 MPa higher than the specified value. A further 20 MPa violation is permitted under retesting.
- The sum of column design flexural strength at a joint should be equal or greater than 1.2 times the sum of moments corresponding to flexural strength of the beams. This may be violated if any positive effect of the column is neglected, all negative effects are catered for and additional confinement reinforcement is placed up the full column height.

Other guidelines in the code are intended to develop inelastic action in members and ensure a level of ductility. Those requirements are listed below:

- For ductility of beams, they consist of clear span, hoops zone and spacing, concrete shear resistance, etc.
- For ductility of columns, they consist of volumetric ratio of spiral hoops, hoop spacing, stirrups for columns supporting discontinuous stiff members, etc.

– For design and detailing of joints, they consist of overstrength factor used in estimating actions on joints, hoops provided to not confine joints, joint width, etc.

6. NUMERICAL EXAMPLE

In order to illustrate the influence of seismic reduction factor and associated reinforcement details on seismic resistance design, a twelve-storey reinforced concrete building (at zone 3, soft soil) was studied and designed in accordance with Indonesian Codes (SNI 03-1726-2002, 2002; SNI 03-2847-2002, 2002). As shown in Figure 2, that office building has 18 m x 36 m plan and 4 m story height. Beam sizes are 400 mm x 700 mm and 350 mm x 700 mm for exterior and interior beams respectively and 300 mm x 500 mm and 200 mm x 500 mm for beams connecting shear walls respectively in X and Y directions. Other structural data are 400 mm x 900 mm column size, 120 mm slab thickness, 300 mm shear wall thickness, 33 MPa concrete grade and 400 MPa steel grade.

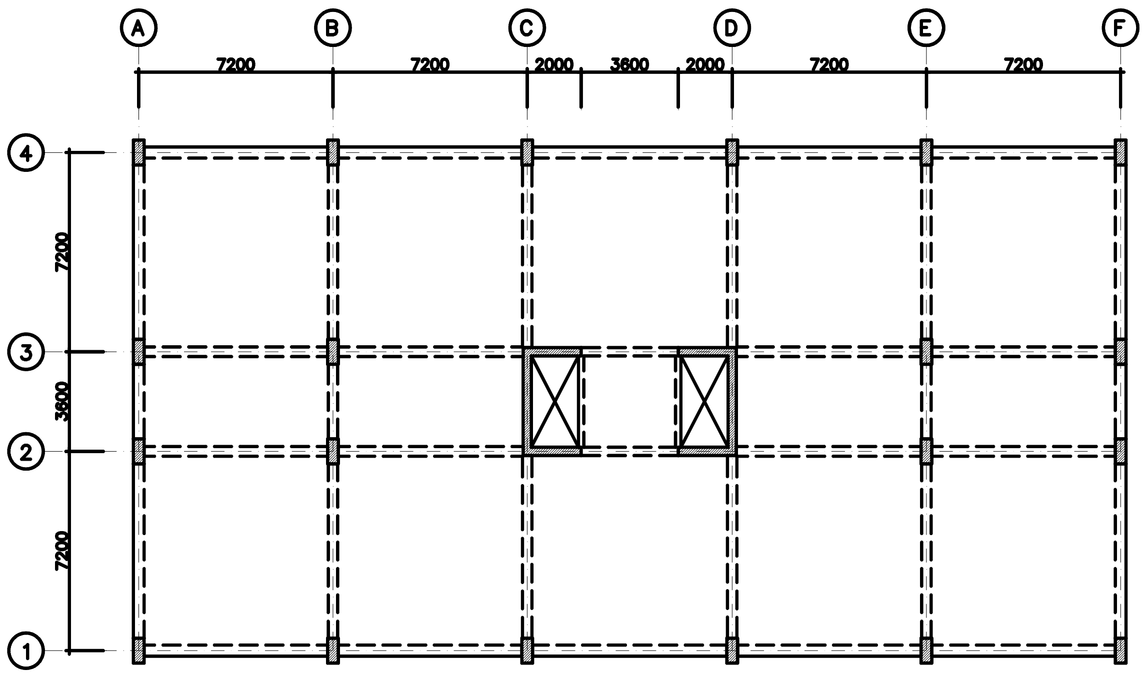


Figure 2. Building Layout

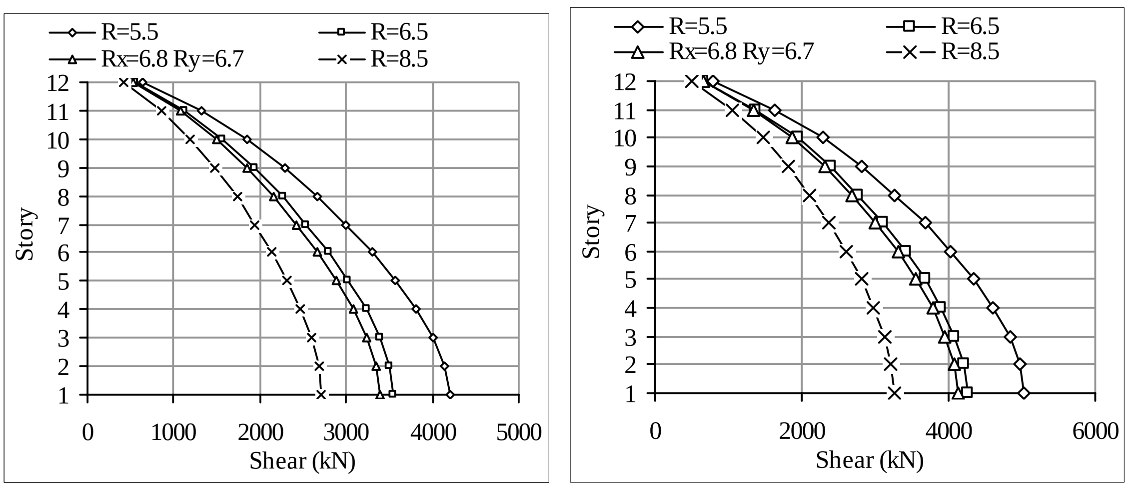
Fundamental natural period of the building is 2 seconds with associated dynamic mode in translation at X direction. Twelve dynamic modes have been included in the analysis for achieving at least 90 percent of the participating mass. CQC method was used as modal combination method and SRSS method for orthogonal seismic direction. Frame base shears are 19 percent and 13 percent of the design base shear in X and Y directions respectively.

Lateral structural system used in this building consists of complete space frames and core walls. That building can be designed as single structural system (shear wall/core wall system) or dual structural system (frame-shear wall system). For the former, the size of structural components has been revised in order to ensure that frame base shear not greater than 10 percent of design base shear. For the latter, frames have been designed to independently resist at least 25 percent of design base shear. Numerical simulations have been carried out for following cases:

– Case 1 : single system (shear wall system)
  - ◦ Limited ductility R = 5.5 with IMRF design.
– Case 2 : dual system (frame-shear wall system)
  - ◦ Limited ductility R = 6.5 with IMRF design.
  - ◦ Weighted ductility $R_x = 6.8$ & $R_y = 6.7$ with SMRF design.
  - ◦ Full ductility R = 8.5 with SMRF design.

7. RESULTS

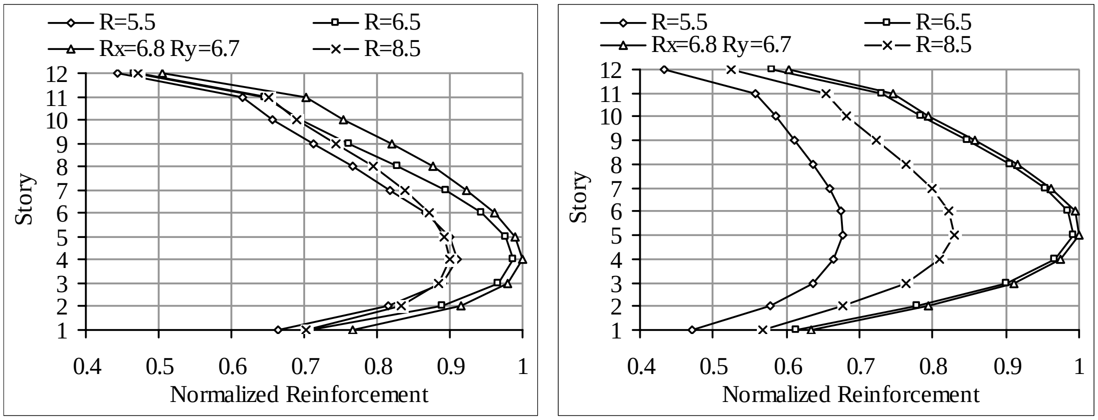
Story shears in X and Y directions are shown in Figure 3. Story shears in Y direction are greater than that in X direction because the structure is stiffer in that direction so it attracts more seismic forces. Story shears increase at lower stories and achieve their maximum values at the base of the structure. As seismic reduction factor R increases, story shears decrease. Smallest story shear is obtained for R = 8.5 and greatest story shears for R = 5.5.



a. X Direction b. Y Direction

Figure 3. Story Shears

Figure 4 presents longitudinal reinforcements for beams. Beams at medium stories (3rd to 8th story) need higher reinforcements than the one at lower or upper levels. Beams in X direction have relatively the same reinforcement quantity for different values of R (only 9 percent of difference). However, at Y direction, the difference between highest (for $R_x = 6.8$ & $R_y = 6.7$) and lowest (for $R_x = 6.8$ & $R_y = 6.7$) reinforcements is significant, more than 30 percent.

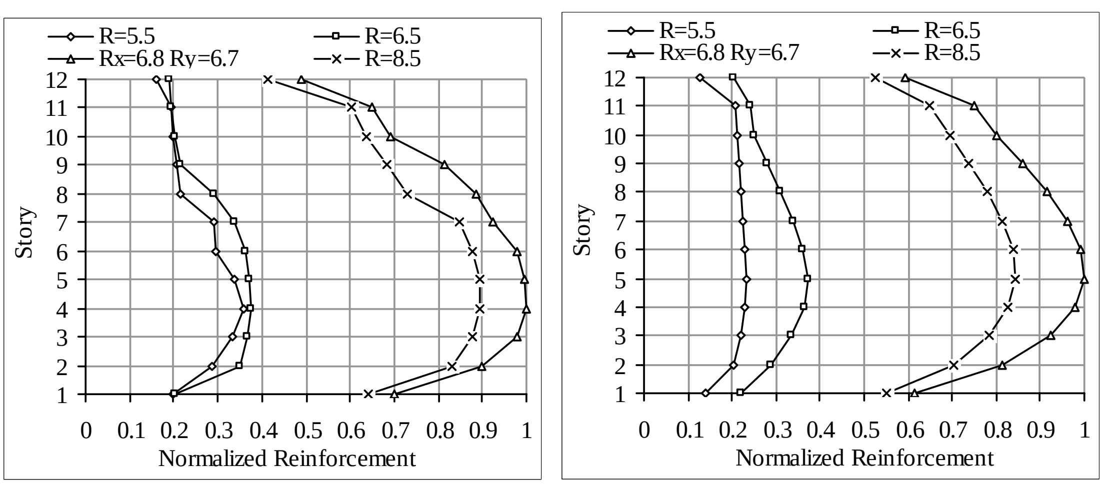


a. X Direction b. Y Direction

Figure 4. Longitudinal Reinforcement for Beams

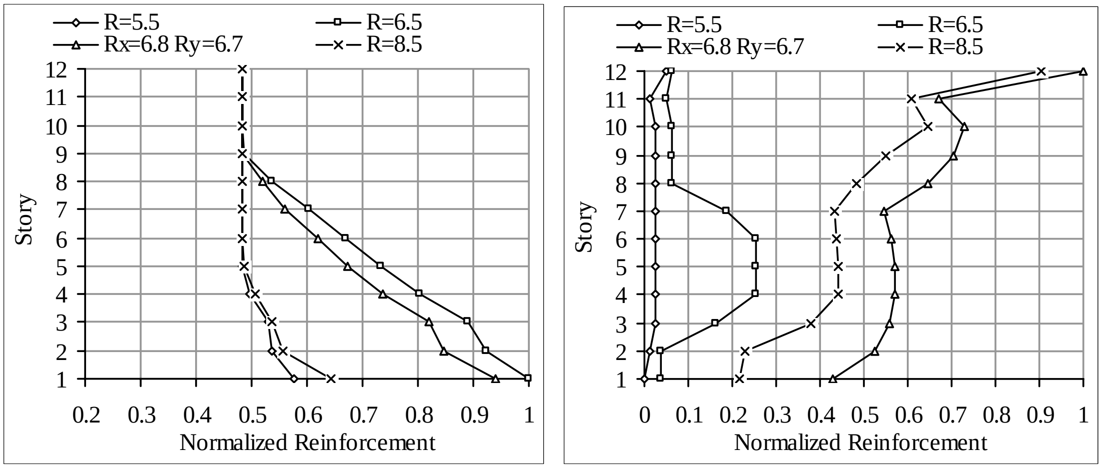
Compared to the IMRF design, SMRF design results higher shear reinforcement. As shown in Figure 5, shear reinforcement for beams designed in accordance with weighted R and SMRF is three times (in X direction) and four times (in Y direction) of that designed with R = 5.5 and IMRF. Similar to the longitudinal reinforcement, shear reinforcement at medium levels is higher than that at other levels.

Longitudinal and shear reinforcements for columns are presented in Figure 6. Minimum longitudinal reinforcements are obtained at levels 5 to 12 (for R = 5.5 and R = 8.5) and levels 9 to 12 (for other R). Below those levels, longitudinal reinforcements increase. As design base shear for frames designed with R = 5.5 or R = 8.5 is low, its longitudinal reinforcement is also minimum. Similar to the shear reinforcements for beams, shear reinforcements for columns with SMRF design are also higher. Only minimum shear reinforcement is required for IMRF design of R = 5.5. Unlike beams, at lower levels columns need less shear reinforcements because higher compressive axial force at lower columns contributes also to the shear resistance of the concrete.



a. X Direction b. Y Direction

Figure 5. Shear Reinforcement for Beams

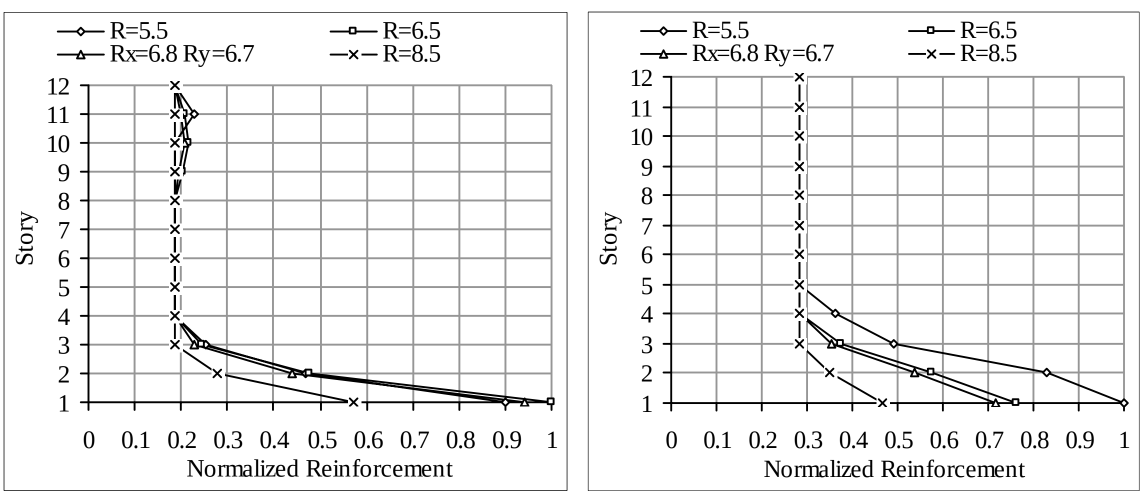


a. Longitudinal Reinforcement b. Shear Reinforcement

Figure 6. Column Reinforcement

As shown in Figure 7, longitudinal and shear reinforcements for shear walls are minimum from 3rd to 12th level. Below those levels, both reinforcements increase. Reinforcements for shear walls designed with R = 8.5 is the lowest quantity than that designed with other R. Its longitudinal reinforcement is only 57 percent of maximum values (with R = 6.5) and its shear reinforcement is 47 percent of maximum shear reinforcement (with R = 5.5).

Distribution of total design reinforcement is given in Figure 8. Minimum reinforcement is obtained for R = 5.5 and maximum reinforcement for weighted R. Higher design reinforcement is required at lower levels. At 3rd level, the difference between minimum and maximum reinforcement is approximately 35 percent. For R = 8.5, total reinforcement from 1st to 5th story is less than that of R = 6.5. However those above levels, the latter decreases.



a. Longitudinal Reinforcement b. Shear Reinforcement

Figure 7. Shear Wall Reinforcement

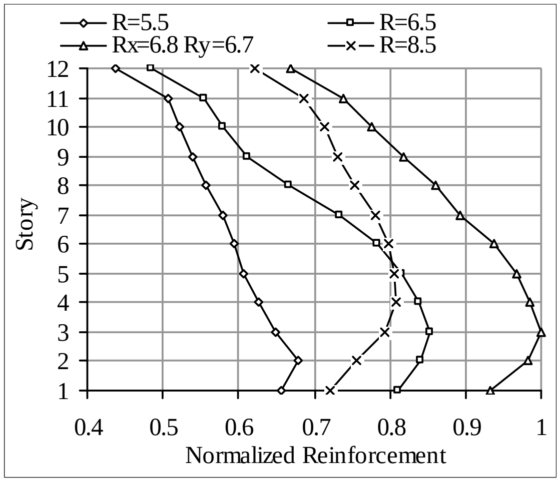


Figure 8. Distribution of Total Reinforcement

8. CONCLUSION

Results of this study can be summarized as following:

- Building with frame and shear wall systems can be designed either as single system or dual system. Selection of structural system is dictated by based shear received by frame system and its capacity provided to resist seismic forces.
- Selection of reinforcement design should be determined by seismic risk areas and structural ductility capacity. For high seismic risk area and high structural ductility capacity, SMRF design shall be mandatory. However in low seismic risk areas with low structural ductility capacity, OMRF design or IMRF design should be sufficient.
- SMRF design requires higher shear reinforcement than IMRF design. Average total shear reinforcement is approximately 21 percent (IMRF design) and 35 percent (SMRF design) of the total reinforcement.
- Average total reinforcement for shear walls is only 11 percent (IMRF design) and 8 percent (SMRF design) of the total reinforcement.
- As shown in Table 3, minimum reinforcement is achieved for single system with R = 5.5 and IMRF design or dual system with R = 6.5 and IMRF design.

Table 3. Normalized Total Reinforcement

Structural System	R	Reinf. Design	Normalized Reinforcement				
			Based on Structure		Based on Action		Total
			Frame	Shear Wall	Shear	Flexural	
Single System	5.5	IMRF	0.57	0.09	0.15	0.51	0.66
Dual System	6.5		0.74	0.07	0.16	0.65	0.81
	R_x = 6.8 & R_y = 6.7	SMRF	0.93	0.07	0.35	0.65	1.00
	8.5		0.78	0.07	0.30	0.55	0.85

9. REFERENCES

ACI Manual (1997) *Building Code Requirements for Structural Concrete and Commentary.* American Concrete Institute, Farmington Hills, MI, 369 pp.

CEB (2000) *Seismic Design of Reinforced Concrete Structures for Controlled Inelastic Response* Comite Euro-International du Beton, Thomas Telford, 174 pp.

Chopra, A. K. (2000) *Dynamics of Structures : Theory and Application to Earthquake Engineering*. Prentice-Hall, Inc., Englewood Cliffs, New Jersey.

Paulay, T., & M. J. N. Priestley (1992) *Seismic Design of Reinforced Concrete and Masonry Buildings*. John Wiley & Sons, Inc., USA.

Paulay, T. (1991) Simplicity and Confidence in Seismic Design", The Fourth Mallet-Milne Lecture, John Wiley & Sons, Inc., USA.

SNI 03-1726-2002 (2002) *Tata Cara Perencanaan Ketahanan Gempa untuk Bangunan Gedung*, Badan Standardisasi Nasional, Jakarta, 63 pp.

SNI 03-2847-2002 (2002). *Tata Cara Perencanaan Struktur Beton untuk Bangunan Gedung*, Badan Standardisasi Nasional, Jakarta, 278 pp.

UBC (1997). *Volume 2: Structural Engineering Design Provisions.* Uniform Building Code, USA, 492 pp.

Wangsadinata, W. (2002). *Standar Perencanaan Ketahanan Gempa untuk Struktur Gedung SNI – 03 – 1726 – 2002,* Proceeding of HAKI Conference on Profesionalisme dalam Dunia Konstruksi Indonesia, Jakarta, August 20 & 21, pp. A1 – A16.

Wilson, E. L. (1996). *Three Dimensional Dynamic Analysis of Structures*. Computers and Structures, Inc., California.