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Multi performance option in direct displacement based design

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Abstract. Compare to traditional method, direct displacement based design (DDBD) offers the more rational design choice due to its compatibility with performance based design which is controlled by the targeted displacement in design. The objectives of this study are: 1) to explore the performance of DDBD for design Level-1, -2 and -3; 2) to determine the most appropriate design level based on material efficiency and damage risk; and 3) to verify the chosen design in order to check its performance under small-, moderate- and severe earthquake. As case study, it uses regular concrete frame structures consists of four- and eight-story with typical plan, located in low- and high-risk seismicity area. The study shows that design Level-2 (repairable damage) is the most appropriate choice. Nonlinear time history analysis is run for each case study in order to verify their performance based on parameter: story drift, damage indices, and plastic mechanism. It can be concluded that DDBD performed very well in predicting seismic demand of the observed structures. Design Level-2 can be chosen as the most appropriate design level. Structures are in safe plastic mechanism under all level of seismicity although some plastic hinges formed at some unexpected locations.

1 Introduction

In traditional seismic structural design, i.e. force based design (FBD), force becomes the most important parameter. FBD using the estimated initial stiffness to determine structural period and the distribution of forces among members. Force is directly related to structural stiffness and displacement during elastic stage. However, for structures in plastic condition, the relationship is complex. Engineers usually reduce the calculated elastic force level due to ductility possessed by the structures. Ductility enabling structures to deform inelastically to the required deformation without loss of strength. If the resistance of the structure is less than the applied force, or if their deformations exceed the limitation, then redesign should be taken.

The interdependency between strength and stiffness as well as between strength and ductility cause FBD hardly to be compatible with performance based design where structural performance is measured by specific displacement for a certain earthquake level. On the other hand, direct displacement based design (DDBD) has been developed over the past ten years to overcome the problems of FBD [1-2].

Unlike FBD, DDBD uses displacement as the designed target. Structures should be designed so that it could reach the targeted design displacement rather than to be bounded by a certain displacement limitation. Previous studies have indicated that DDBD performed well in many kind of structures [3-5]. However, they only observed the performance of structures under design level-2 only, repairable damage condition.

DDBD actually offers three level of design options, i.e. level-1 no damage, level-2 repairable damage, and level-3 no collapse for three design intensity levels as shown in Table 1 [6].

Table 1. Design intensity and performance criteria [6].

Importance Class	Earthquake Design Intensity		
	Level-1 No Damage	Level-2 Repairable Damage	Level-3 No Collapse
I	Not required	50% in 50yrs	10% in 50yrs
II	50% in 50yrs	10% in 50yrs	2% in 50yrs
III	20% in 50yrs	4% in 50yrs	1% in 50yrs
IV	10% in 50yrs	2% in 50yrs	1% in 50yrs
Drift Limit	0.01	0.025	No Limit

The design intensity level is defined as probability of exceedance for a given intensity level depends on the structural occupancy usage and damage consequences. Table 1 also shows the story drift limits for each design intensity under condition where buildings with non-structural elements detailed to sustain building displacement. For structures in moderate to high

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seismicity, performance criteria for levels 1, 2 and 3 intensities shall be met. While for structures in low seismicity, performance criteria for level 3 intensity shall be met [6].

Based on those conditions, this study is aimed to: 1. explore the design procedure of DDBD on regular concrete frame structures for level 1, 2 and 3; 2. determine the most appropriate design level based on material efficiency and damage risk; and 3. verify the controlled design in order to check the structural performance under small-, moderate- and severe earthquake.

2 Case study

The study uses regular concrete frame structure consist of 4 and 8 story with equal span 6 and 8 m (denote as variant A and B), equal story height of 4 m (Fig.1). Thus, there are eight case studies will be involved in this study. Assumption made for the design including: concrete strength of 30 MPa; reinforcement yield stress of 400 MPa; structures are located in Surabaya and Jayapura city representing low- and high-seismicity area; importance class is II for office building.

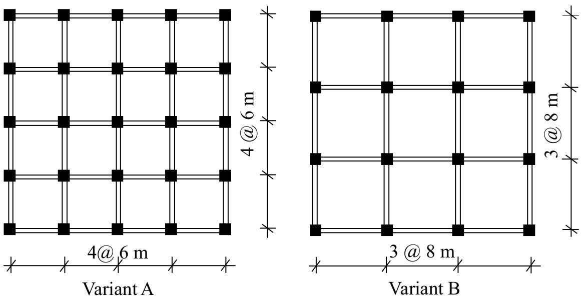

Fig. 1. Structure plans and variants.

3 Design and verification

3.1 DDBD Methodology

The eight observed structures are designed based on DDBD procedure [2] for three level of design, i.e. levels 1, 2 and 3 with probability of exceedance as much as 50%, 10% and 2% in 50 years (or equal to earthquake with 100, 500 and 2500 years return period respectively). The design drifts for Levels-1 and -2 are 0.010 and 0.025 [6]. Although there is no drift limitation for Level-3, but the study determines 0.040 as the design drift.

The elastic design response spectrum is constructed based on [7] for site class SE (soft soil) in Surabaya and Jayapura city. The study uses peak ground acceleration (PGA) calculation served by [8] to calculate the PGA of various return period, proportionally to the PGA of the maximum considered earthquake (MCE), i.e. 2500 years earthquake return period. The complete response spectrum for design Levels-1, -2 and -3 for Surabaya and Jayapura are shown in Fig.2.

The DDBD methodology [2] is illustrated in Fig.3. Starting by representing the building with a single degree of freedom (SDOF) model in Fig.3(a) and making a bi-linear relationship between lateral force – displacement in Fig.3(b). DDBD uses secant stiffness K_e to represent structural stiffness during inelastic at the design displacement Δ_d. In Fig.3(c) for a given level of ductility demand, the equivalent viscous damping ξ will be found.

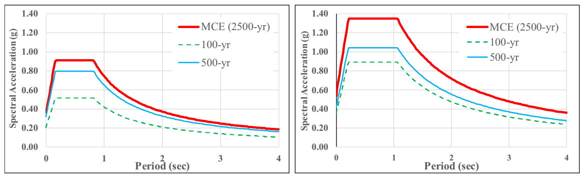

Fig. 2. Design response spectrum.

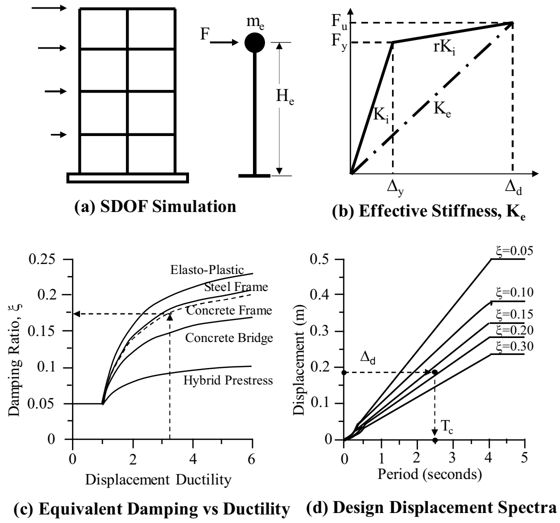

Fig. 3. Fundamental of DDBD [2].

The calculation of design displacement Δ_d is found from a normalized inelastic mode shape δ_i, where i = 1 to n are the stories, to the targeted displacement Δ_c (e.g. 0.010, 0.025 and 0.04) at the critical story following:

$$\Delta_i = \delta_i\,(\Delta_c / \delta_c) \tag{1}$$

where the mode shape depends on the floor height, H_i, and roof height H_n according to:

$$\text{for } n \leq 4:\ \ \delta_i = H_i / H_n \tag{2a}$$

$$\text{for } n > 4:\ \ \delta_i = (4/3)\,(H_i / H_n)\,\{1 - H_i / (4H_n)\} \tag{2b}$$

The equivalent design displacement is related to the story displacements by the relationship:

$$\Delta_d = \Sigma(m_i \Delta_i^2) / \Sigma(m_i \Delta_i) \tag{3}$$

where m_i is the mass at height H_i related with displacement Δ_i. The equivalent mass m_e and the effective height H_e of the SDOF are given by:

$$H_e = \Sigma(m_i \Delta_i H_i) / \Sigma(m_i \Delta_i) \tag{4}$$

The design displacement ductility factor can be related to the equivalent yield displacement Δ_y through:

$$\mu = \Delta_d / \Delta_y \tag{5}$$

The yield drift of a story θ_y, in a frame structure depended on geometry, and is independent of strength [2]. And for reinforced concrete frame it is defined as:

$$\theta_y = 0.5\ \varepsilon_y L_b / h_b \tag{6}$$

where L_b and h_b are the beam span between column centerline, and beam depth respectively, ε_y is the yield strain of the reinforcement steel.

The ductility demand can be estimated by assuming a linear yield displacement profile, and hence the yield displacement is given by:

$$\Delta_y = \theta_y / H_e \tag{7}$$

The equivalent viscous damping of the SDOF for reinforced concrete frame can be related to the design displacement ductility demand as follows:

$$\xi_{eq} = 0.05 + 0.565\ (\mu - 1)/(\mu\pi) \tag{8}$$

The effective period T_e of the SDOF from the peak displacement response can be found from the displacement spectrum, by relating the design displacement Δ_d to the corresponding equivalent viscous damping ξ_{eq} as illustrated in Fig.3(d).

The effective stiffness K_e of the SDOF at maximum displacement response Δ_d is given by:

$$K_e = F/\Delta_d = 4\pi^2 m_e/T_e^2 \tag{9}$$

The design base shear for the multi degree of freedom (MDOF) structure is found from the SDOF structure as:

$$F = V_{base} = K_e\Delta_d \tag{10}$$

The base shear force is then distributed to the floor level in proportion to the product of mass and displacement as:

$$F_i = V_{base}(m_i\Delta_i)/\Sigma(m_i\Delta_i) \tag{11}$$

The structure is then analyzed under the simplified force vector referring to [2] to determine the required flexural strength at the beam plastic hinge locations. The method is based on simple equilibrium condition among members. It is not unnecessarily to add the full gravity-load moments to these seismic moments so that structures may develop its ductility capacity in achieving the design displacement. And finally, DDBD also apply capacity design procedure in order to assure the condition of "strong column weak beam" so that structures are within safe mechanism.

3.2 Design Verification

After choosing the most appropriate design level by considering material efficiency and damage level, each case study is modelled using [9]. Nonlinear time history analysis (NLTHA) is run for each case study using the acceleration record of El-Centro 1940 N-S, which has been modified in order to match with the targeted response spectrum for Surabaya dan Jayapura city. Since structures have symmetrically in plan, then the NLTHA is only run in one direction. The force – deformation relationship is modelled using [10]. The acceptance performance is measured based on parameter: story drift, plastic hinge rotation (represent level of damage) and structural plastic mechanism.

4 Results

4.1 Material consumption

From eight study cases, the average consumption of material relative to condition of design Level-2 are shown in Table 2. Material consumption are calculated by adding the volume of bending and shear reinforcement in beam and column, as well as the volume of beam and column for concrete volume. It can be said that design Level-1 is the most expensive design, 1.65- and 2.03-times of reinforcement and concrete consumption of design Level-2. Conversely, design Level-3 is the most efficient design, but not so much different from material consumption of design Level-2.

Table 2. Average consumption of material – relative to design Level-2.

Material	Surabaya			Jayapura		
	Lev-1	Lev-2	Lev-3	Lev-1	Lev-2	Lev-3
Steel reinf.	1.65	1.00	0.95	2.05	1.00	0.95
Concrete	2.03	1.00	0.90	2.65	1.00	1.00

From efficiency point of view, absolutely design Level-1 is too expensive to be applied. Considering damage risk, although design Level-2 and -3 are almost the same, but design Level-2 can be expected to perform better. Therefore, the study choses design Level-2 as the controlled design, and performs nonlinear time history analysis in order to check its structural performance due to small-, moderate- and severe-earthquake. The structural performance measured by parameter: story drift, damage indices and plastic mechanism are discussed in subsequent sections as well as the detailed results.

4.2 Story drift

The story drift from the eight case studies resulting from nonlinear time history analysis are shown in Fig.4. Each structure is codified according to its number of story, variant of geometry plan and its location, e.g. 4A SBY means four story with variant A in plan, located in Surabaya.

The values shown in Fig.4 are taken from the maximum story drift during the considered excitation. Each of them are compared to the drift limit of each design level, i.e. 0.010, 0.025 and 0.040 due to small-, moderate- and severe-earthquake. It can be shown that all variants show excellent performance under story drift consideration. They are lower than the drift limit determined for each level of design. Thus, it can be said that the design Level-2 of DDBD successfully maintain the expected structural performance.

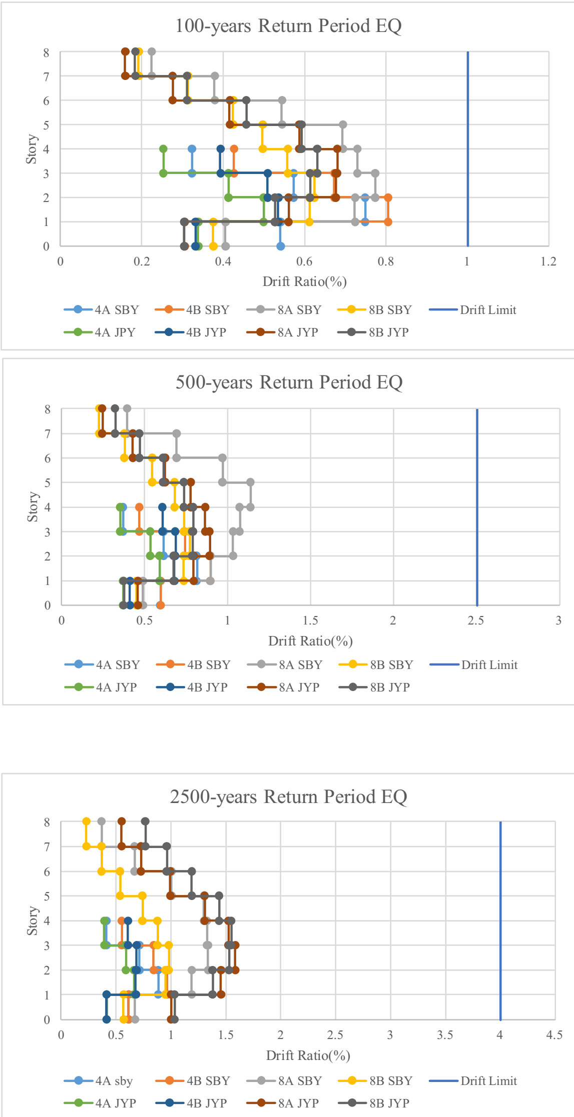

Fig. 4. Story drift.

4.3 Damage indices

Damage index is a parameter to measure the level of damage of the structure, defined as the plastic rotation of the member (beams and columns). Reference [6] determines the range of plastic rotation for each performance level. For beams, it is set as 0.004-0.008 rad, 0.008-0.01 rad and 0.01 rad – no limits for design Levels-1, -2 and -3 respectively. While for columns, it is set to 0.003-0.005 rad, 0.005-0.006 rad and 0.006 rad – no limits.

Table 3 and 4 show the structural performance based on the maximum plastic rotation during time history analysis on beams and columns. It can be shown that all beams have performed well under any earthquake level. Black blocked area in the table indicates unacceptable condition where the member rotation exceeding the limit.

Unfortunately, some columns experience excessive rotation due to small- to moderate-earthquake, e.g. structure 8A in Surabaya, 4A-, 8A- and 8B-in Jayapura, because columns experience load combination of axial tension and moment in a very short time. Despite these condition, all structures performed very well under severe-earthquake.

Table 3. Performance based on beam plastic rotation

Location	Return Period (Years)	First Yield	Lev-1 No Damage	Lev-2 Repairable Damage	Lev-3 No Collapse
Sura-baya	100	8A, 8B	4A, 4B		
	500		All		
	2500		4A, 4B, 8B	8A	
Jaya-pura	100	4A, 4B, 8B	8A		
	500	4A	4B, 8A, 8B		
	2500	4A	4B		8A, 8B
	Unacceptable performance				

Table 4. Performance based on column plastic rotation

Location	Return Period (Years)	First Yield	Lev-1 No Damage	Lev-2 Repairable Damage	Lev-3 No Collapse
Sura-baya	100	4A, 4B,8B			8A
	500	4A, 4B,8B			8A
	2500	4A, 4B,8B			8A
Jaya-pura	100	4B, 8B			4A, 8A
	500	4B, 8A			4A, 8B
	2500				4A, 4B, 8A, 8B
	Unacceptable performance				

4.4 Plastic mechanism

For the sake of safety, structures resulting from DDBD procedure are expected to experience "beam side sway mechanism" where the occurrence of plastic hinges are kept only at the beam, top floor columns and base columns as shown in Fig.5. To achieve this condition, designers apply capacity design in order to ensure condition of "strong column weak beam".

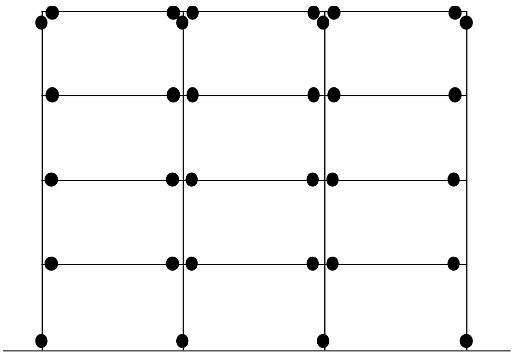

Fig. 5. Beam side sway mechanism.

Due to space limitation, this paper only presents the plastic mechanism of structure 8A SBY (Fig.6-8) [11]. The rotation performance of plastic hinge is indicated with color for each design level, i.e. yellow for first yield, green for Level-1 (no damage), light blue for Level-2 (repairable damage), and red for Level-3 (no collapse). The figures also show the condition of first yield and unacceptable condition (beyond Level-3 limitation), indicated with dark blue-color. The maximum rotation is indicated with square mark, while the plastic hinges in unexpected location are indicated with circle mark.

For low-earthquake excitation (Fig.6), the maximum plastic rotation in beam and column are in condition of first yield, some columns have already in no collapse criteria. For moderate-earthquake (Fig.7), the plastic rotation is worse than condition due to low-earthquake. Beam is in no damage condition while column is in no collapse condition. Finally, for severe-earthquake (Fig.8), structure experiences the worst plastic rotation. Beam and column are in repairable damage and no collapse respectively. These conditions are match with the plastic rotation performance as presented in Table 3 and 4. Capacity design successfully maintain the condition of 'strong column weak beam'. Yielding always initiate at the beams before at the columns.

Unfortunately, some unexpected yielding occurred at columns in story 1, 5 and 6 for low-earthquake, and in almost all stories for moderate- to severe-earthquake. It indicates that the beam side sway mechanism cannot be achieved perfectly. Nevertheless, the yielding of the column is still in first yield condition, and does not worsen the performance of the structures. The study does not identify any tendency of soft story mechanism. Thus, it can be concluded that structures are in safe mechanism. The complete plastic hinge formation of this study can be found in [11].

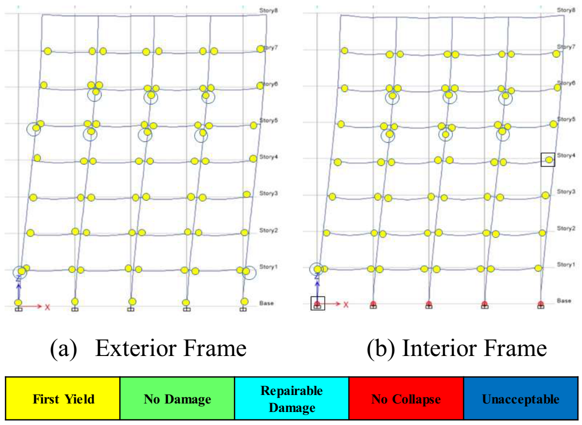

Fig.6. Plastic mechanism of 8A SBY, 100 yrs [11].

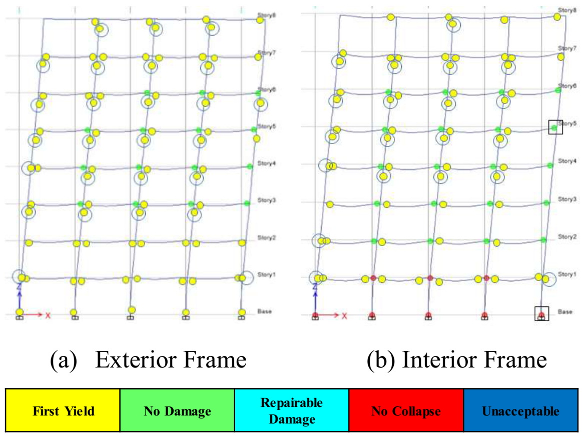

Fig.7. Plastic mechanism of 8A SBY, 500 yrs [11].

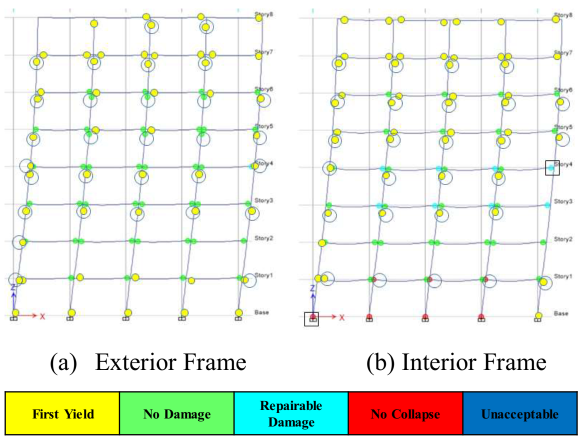

Fig.8. Plastic mechanism of 8A SBY, 2500 yrs [11].

5 Conclusion

It can be concluded that direct displacement based design (DDBD) performed very well in predicting seismic demand of regular concrete frame structures both in low- and highly risk seismicity area. Design

Level-2 (repairable damage), can be chosen as the most appropriate design level based on consideration of material efficiency and damage risk. Structures are in safe plastic mechanism under all level of seismicity although some plastic hinges formed at some unexpected locations.

References

1. M.J.N. Priestley. Bul. NZSEE, **26(3)**, 329-341 (1993).
2. M.J.N. Priestley, G.M. Calvi, M.J. Kowalsky. *Displacement-based seismic design of structures*. (IUSS Press, Pavia, Italy, 2007).
3. I. Muljati, F. Asisi, K. Willyanto. *Procedia Engineering*, **125**, 1050-1056 (2015).
4. I. Muljati, A. Kusuma, F. Hindarto. *Procedia Engineering*, 125 (2015) 1057-1064
5. I. Muljati, B. Lumantarna, R.P. Intan, A. Valentino. *Procedia Engineering* (to be published)
6. T.J. Sullivan, M.J.N. Priestley, G.M. Calvi. *A model code for the displacement-based seismic design of structures*. (IUSS Press, Pavia, Italy, 2012).
7. BSN, SNI-03-1726-2012. *Tata cara perencanaan ketahanan gempa untuk struktur bangunan gedung dan non gedung*. (Departemen Pekerjaan Umum, Jakarta 2012).
8. Indonesian Public Work Department, url: http://www.pusair-pu.go.id/pgacal (2011).
9. Computer and Structures, Inc. *CSI analysis reference manual for Sap2000, ETABS, SAFE, and CSI Bridge*. (Berkeley, California, USA, 2015).
10. L.A. Montejo. CUMBIA computer software. (North Carolina State University, (2007).
11. Yonatan, A. Hartono. Thesis No. **11012090** (Petra Christian University, Surabaya, Indonesia, 2016).

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