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Performance of direct displacement based design on regular concrete building against Indonesian response spectrum

Ima Muljati[a,*], Benjamin Lumantarna[a], Reynaldo P. Intan[a], Arygianny Valentino[a]

[a] *Civil Engineering Department, Petra Christian University, Surabaya, Indonesia*

Abstract

The renewal of Indonesian seismic code from SNI 1726-2002 into SNI 1726-2012 brings significant change in the design spectrum. Focused on several regular plan concrete building which have been design using displacement based design method, the aim of this study is to verify their performance using nonlinear time history analysis based on parameters: drift, damage indices, and plastic mechanism determined by FEMA 356. The excitation is spectrum consistent accelerogram based on El-Centro 1940 N-S, to match with the new Indonesian response spectrum for soft soil in low- and high intensity area. It is found that the code-designed buildings are not suitable for the targeted design of level-2 with maximum drift of 2.5% due to major. This is caused by improper selection of SNI spectrum as the design major earthquake. In fact, it is only equivalent to small earthquake. Although buildings survive up to a very rare earthquake without collapse but they suffer excessive damage and rotation due to small- to major-earthquake. The capacity design procedure is able to maintain ductile mechanism, but some columns experience yielding at prohibited locations.

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Keywords: Direct displacement based design; non-linier time history analysis; performance based design; regular plan building

1. Introduction

The latest Indonesian seismic code SNI 1726-2012 [1] brings significant changes in the design spectrum. In line with ASCE 7-10 [2] as the main reference of this code, SNI 1726-2012 uses the maximum considered earthquake (MCE) based on 2500-year return-period ground motion (with 2% in 50-year probability of exceedance). Code-

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* Corresponding author. Tel.: +62 816 504 496; fax: +62 31 298 3392.
E-mail address: imuljati@petra.ac.id

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designed buildings are expected to resist collapse for up to the MCE ground motions. However, the design level ground motion is determined from the MCE ground motion values divided by 1.5, known as the seismic margin, to convert the collapse level defined by the MCE maps. Therefore, the spectral acceleration formula for period of 0.2 sec and 1.0 sec (S_s and S_1) should be multiply by 2/3. The result of multiplication means that each cities will have different probability of exceedance, e.g. different return period [3,4].

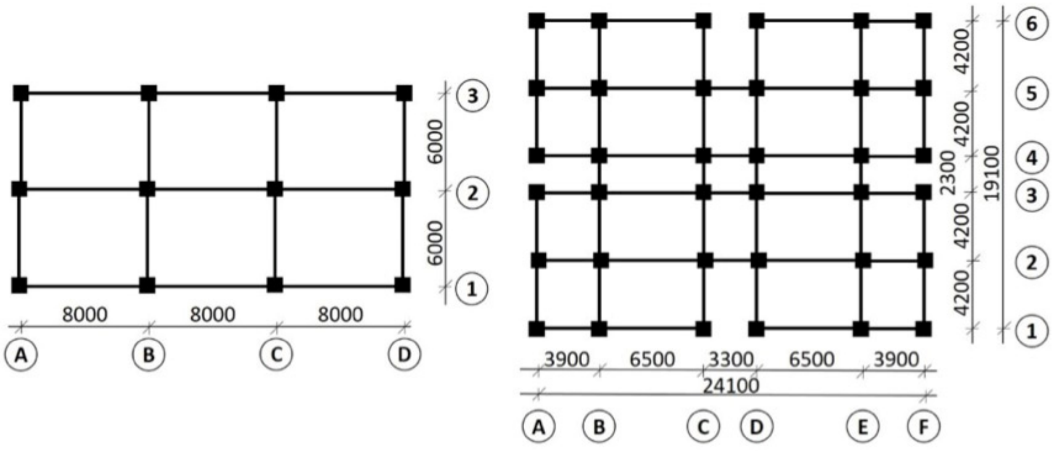
On the other hand, the former Indonesian seismic code, SNI 1726-2002 [5], uses the design level ground motion which is based on 500-year return period ground motion (10% in 50-year probability). Therefore, buildings that have been designed using this code should be re-examined its performance against the new response spectrum used in the new code. This is the main objectives of this study. It specifically chose several regular plan concrete buildings which has been designed using direct displacement based design (DDBD) methodology as case study.

DDBD itself is an alternative seismic design methodology to overcome the drawback of the previous and the most popular method, force based design (FBD) [6]. In DDBD the required strength at the designated plastic hinge locations is determined so that the structure may be able to achieve the targeted design displacement. It must then be combined with capacity design procedures to ensure that plastic hinges occur only where intended, and that non-ductile modes of inelastic deformation do not develop. Instead of following the standard design procedure, these capacity design procedures must be calibrated to the DDBD approach based on proposal of reference [6].

2. Case study

Previous studies reported that DDBD has successfully overcome the drawback of FBD [7-8]. Case study in reference [7] has been done based on SNI 1726-2002, while the latter reference [8] is based on SNI 1726-2012. All buildings are designed for the targeted displacement of level-2, with 2.5% maximum drift at the design level ground motion. The detailed design procedure of DDBD method including structural geometry and reinforcement data can be found in references [7-8]. This study only emphasizing and observing their performance against several seismic intensity level determined by the new code.

Their structural performance is examined using nonlinear time history analysis where the moment-curvature relationship of each member is calculated by CUMBIA [9]. The excitation uses spectrum consistence accelerogram based on El-Centro 1940 N-S as the ground acceleration, and has been modified in order to match with the targeted response spectrum defined by the new code. The modification process is done by RESMAT software program [10]. The structural performance level is measured using FEMA 356 [11] based on parameter: drift, plastic hinge rotation (damage indices), and their plastic mechanism. The observed buildings including their number of stories are shown in Fig.1.



a. Case 1, 6- and 10-story [7]

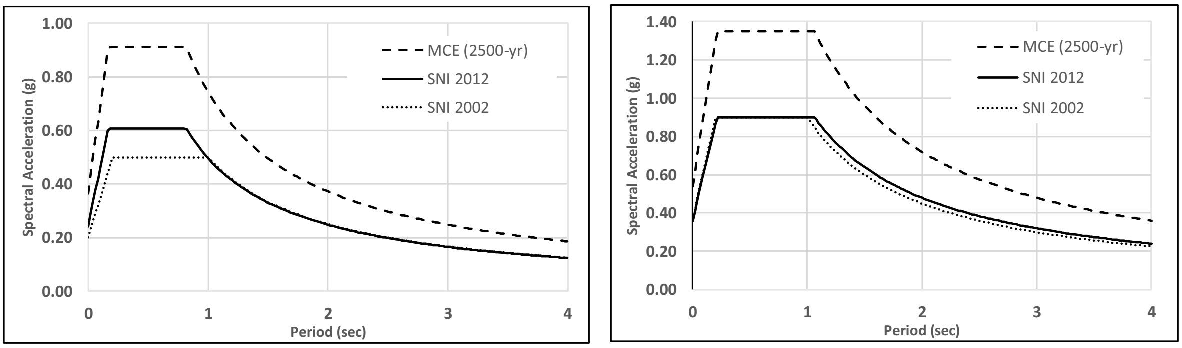
b. Case 2, 6-story [8]

Fig. 1. The observed regular plan building.

3. Response spectrum

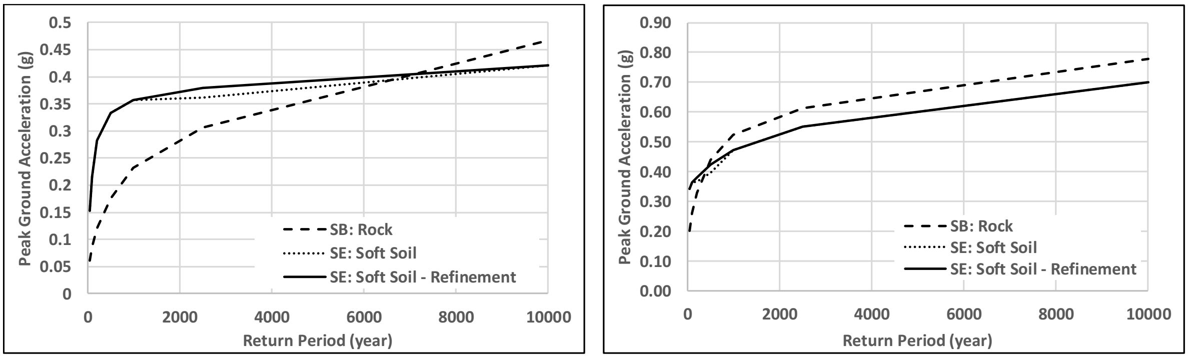
Response spectrum is a plot of peak acceleration induced by earthquake in an elastic single-degree-freedom system of period T and specified damping. The response spectra are usually shown for 5% of critical damping, but the spectral acceleration (SA) can be easily adjusted for damping values other than 5%. Fig. 2 shows the design response spectrum for Surabaya and Jayapura city determined by the previous and the later Indonesian seismic code, identified as dotted- and solid line respectively. Both cities are chosen to represent low- and high-seismicity area in Indonesia. The maximum considered earthquake (MCE) response spectrum determined by the new code, e.g. 1.5 times of the design response spectrum, identified as dashed line. It turns out there is a difference between the design response spectrum in SNI 2002 and 2012. For Surabaya, SNI 2002 provides a lower response spectrum than SNI 2012 while for Jayapura there is only slightly difference.

Indonesian Public Work Department through its website with its link in reference [12] serves computational facility to calculate the peak ground acceleration (PGA) for various return periods including the site coefficient (FPGA). PGA calculation for any site class are done based on PGA map for 2500-year return period (2% in 50-year probability of exceedance) on soil type rock (SB) according to SNI 1726-2012, marked as dashed-line in Fig. 3. And the origin relationship resulted from PGA calculation for site class SE is drawn as dotted-line in Fig.3. Some error happened, and this study conducting refinement in order to find the more realistic and smoother curve (marked as dashed-line). And the final results of PGA for Surabaya and Jayapura are drawn as solid-line in Fig. 3.



a. Surabaya b. Jayapura

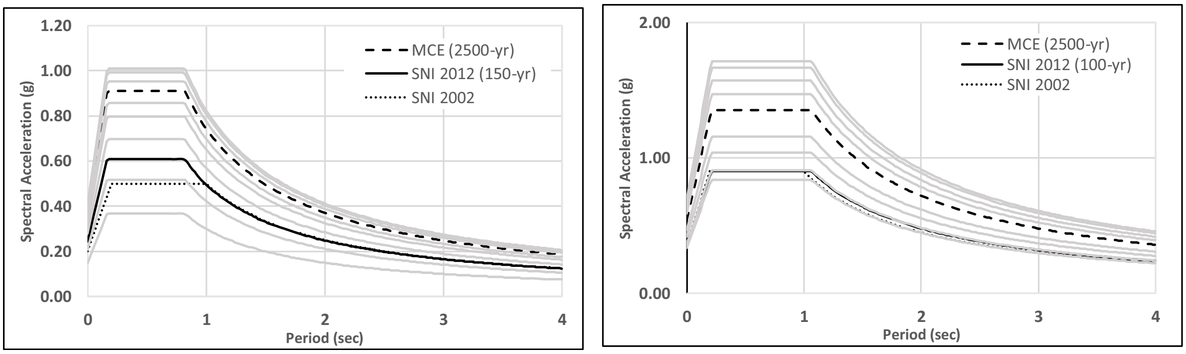
Fig. 2. Response spectrum.



a. Surabaya b. Jayapura

Fig. 3. Peak ground acceleration for various return period.

By using this relationship, the study can find the value of PGA for any return period. Thus, the response spectrum for each return period can be built, and performance level of each building being observed can be checked. This study uses ten levels of earthquake return period (i.e. 50-, 100-, 250-, 500-, 1000-, 2500-, 5000-, 7000-, 9000-, 10000-year) in order to evaluate the final performance of the structure. The plot of them all together with the MCE- and the design-response spectrum are shown as a set of response spectra in Fig. 4. It is found that the design response spectrum for Surabaya and Jayapura in SNI 2012 are equal to 150- and 100-year return period respectively.



a. Surabaya

b. Jayapura

Fig. 4. Response spectra for various return period (50 – 10000-year).

4. Design verification

Each case study is modeled in SAP2000 [13] and is run nonlinear time history analysis using the modified ground acceleration, adjusted with the targeted response spectrum. The ground acceleration is applied in x- and y-direction. Case study 1 and 2 will be considered, three different number of story, two seismic sites, two direction-considerations. Thus, there are totally twelve sets of analysis for each return period will be reported in the subsequent section.

5. Results and discussion

The following design verification resulting from nonlinear time history analysis are plotted in the performance matrices according to FEMA 356 [11] for parameter: drift and damage indices as well as information about their plastic mechanism. The circled numbers denote number of structures within the certain limit state, while white- and black circle represent Surabaya and Jayapura. The expected performance occurs when buildings perform properly in the determined limit state which is in *"operational level"* due to small earthquake (up to 72-year return period), in *"immediate occupancy"* due to moderate earthquake (up to 250-year return period), in *"life safety"* due to major earthquake (up to 500-year return period), and in *"collapse prevention"* due to a very rare earthquake (up to 2500-year return period) as listed in Table 1.

5.1. Drift

Table 1 shows the performance of structure measured by the maximum drift experienced during the excitation. It can be seen that all buildings as expected survive up to very rare earthquake without collapse. However, there is a significant difference between building performance in Surabaya dan Jayapura. For Surabaya, buildings suffering excessive drift due to small to moderate earthquake but they perform well in life safety limit state up to very rare earthquake with 10000-year return. While for Jayapura, excessive drift occurs during small to major earthquake, but they survive up to very rare earthquake with return period of 5000-year.

This study has shown clearly that the design response spectrum for Surabaya and Jayapura only equal to earthquake with return period of 150-year and 100-year, which categorized as small earthquake by FEMA 356 [11]. Therefore, the target design displacement of 2.5% is incorrect for this earthquake level. The true design displacement should be in level-1 with maximum displacement of 1.0%. Thus the strength demand in [7-8] is less than it should be. Not surprisingly if building suffer excessive drift due to small to major earthquake. Fortunately, they survive up to rare earthquake with return period of 10000-year and 5000-year for Surabaya and Jayapura respectively.

Table 1. Building performance based on drift.

Return period	Earthquake level	Operational level	Immediate occupancy	Life safety	Collapse prevention	Unacceptable
50	Small		⑥❷	❹		
100			⑥❷	❹		
250	Moderate		④❷	②	❹	
500	Major		④❷	②	❹	
1000			③❷	③	❹	
2500	Rare		①❷	⑤	❹	
5000			❷	⑥	❹	
7000			❷	⑥		❹
9000			❷	⑥		❹
10000			❷	⑥		❹
Max. drift (%)		0	0 – 1%	1 – 2%	2 – 4%	More than 4%
▨	Undesirable	○● Surabaya/Jayapura		Circled number: number of buildings in certain area		

5.2. Damage indices and plastic mechanism

Damage index is a parameter to measure the level of damage of structural member. It is defined as the rotation demand divided by the available rotation capacity which can be calculated using:

$$DI = \frac{\theta_n - \theta_y}{\theta_u - \theta_y} \tag{1}$$

where θ_n is the rotation occur at the member, θ_y and θ_u are the yield- and ultimate-rotation of the member. Table 2 shows the damage indices of beams and columns of the observed structure. The meaning of symbols and notation are similar to those already used in previous drift consideration.

For Surabaya, damage indices for beams are consistent with drift consideration. For return period of 500-year and larger all beams perform well in *life safety*. In the contrary, for Jayapura only two buildings are in similar condition, while the other four are in *"collapse prevention"* limit state. Interestingly, all beams successfully survive up to earthquake with return period of 10000-year.

On the other hand, damage indices for columns show different results. For Surabaya, the condition of *"life safety"* is only assured up to return period of 500-year. For return period of 1000-year and larger, there are two buildings have already in *"collapse prevention"*. While for Jayapura, columns able to survive in condition *"life safety"* up to return period of 2500-year. Again, all columns successfully survive up to earthquake with return period of 10000-year.

All buildings show ductile mechanism, yielding is started in beams and followed by columns. It can be said that the effort of capacity design in DDBD procedure effectively guarantee the condition of *strong column weak beam*. Unfortunately, there are still yielding in some columns at prohibited locations instead of at the top end of roof column and at the base column. Although all buildings suffer considerable damage but no one of them collapse up to a very rare earthquake with return period of 10000-year

Table 2. Building performance based on damage indices in beam and column.

		Beam				Column			
Return period	Earthquake level	Operational level	Immediate occupancy	Life safety	Collapse prevention	Operational level	Immediate occupancy	Life safety	Collapse prevention
50	Small		⑥❷		❹	②❹	④	❷	
100			④❷	②	❹	④❷	①❷	①❷	
250	Moderate		③❷	③	❹	③❶	①❷	②❸	
500	Major		①❷	⑤	❹	①	③❷	②❹	
1000			❷	⑥	❹	①	③❷	❹	②
2500	Rare		❷	⑥	❹		④❷	❹	②
5000			❷	⑥	❹		③❷	①❸	②❶
7000			❷	⑥	❹		③❷	②❷	①❷
9000			❷	⑥	❹		②❷	②	②❹
10000			❷	⑥	❹		②❷	②	②❹

▨ Undesirable ○● Surabaya/Jayapura Circled number: number of buildings in certain area

6. Conclusion and recommendation

It is concluded that the code-designed buildings using direct displacement based design (DDBD) in previous studies is not suitable for the targeted design of level-2, i.e. maximum drift of 2.5% due to major earthquake. This is caused by improper selection of SNI spectrum as the design major earthquake. In fact, it is only equivalent to small earthquake. Although buildings survive up to a very rare earthquake without collapse but they suffer excessive damage and rotation due to small- to major-earthquake. The capacity design procedure is able to maintain ductile mechanism, but some columns experience yielding at prohibited locations.

In order to increase the building performance, this study recommends to conduct further study of DDBD using two different target design drift, i.e. 1.0% and 2.5% for small- and major- earthquake.

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