Paper1

by Pamuda Pudjisuryadi

Performance of an existing reinforced concrete building designed in accordance to older indonesian codes (pptgiug 1983 and sksni t-15-1991-03): case study for a hotel in balikpapan

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Abstract. As a high seismic hazard country, Indonesia periodically updates its seismic and structural concrete codes. The current seismic and structural concrete codes of Indonesia are the SNI 1726-2012 and the SNI 2847-2013, respectively. Since every update usually demands higher requirement, existing buildings that were designed using older codes should be evaluated. This study investigates 9-storey hotel building in Balikpapan, Indonesia, which was designed using the PPTGIUG 1983 code and will be evaluated according to the current code. Non-linear direct integration time history analysis was conducted to analyse the building performance. The seismic load used was a spectrum consistent ground acceleration generated from El Centro 18 May 1940 North-South component in accordance to the current code. The result show that the existing building has good performance. The drift ratio of the building does not exceed 0.5% which is very satisfactory according to performance level set by FEMA 356. Maximum individual damage index in beam element was recorded as high as 0.0426 which is well below the serviceability limit state according to ACMC.

1 Introduction

Indonesian first seismic hazard map was the PMI 1970, which divided Indonesia into only three seismic zones [1]. The seismic map has undergone several updates, including PPTGIUG 1983 [2], SNI 1726-2002, and the current SNI 1726-2012 [3]. The updates were made in view of the occurrence of larger earthquake events than previously estimated and the development of new analytical methods that could result in better seismic mapping [4].

Changes in the seismic hazard map might result in increase of PGA (Peak Ground Acceleration) which consequently increases the earthquake design load. As consequence of this earthquake design load increase, existing buildings designed with older seismic codes are in need of evaluation. The performance of those buildings subjected by higher load demand should be investigated to determine if they need any strengthening.

In this research, a 9-storey hotel building in Balikpapan, Indonesia which was designed using older Indonesian seismic code (PPTGIUG 1983) and older Indonesian concrete code (SKSNI T-15-1991-03 [5] was chosen to be investigated. The building performance in resisting maximum considered earthquake according to SNI 1726-2012 was assessed. Story drifts and member damage indices were used to determine the building performance levels based on FEMA 356 [6] and ACMC [7], respectively.

2 PPTGIUG 1983 and SNI 1726-2012

The design seismic load in PPTGIUG 1983 was based on earthquake with 200-year return period, while SNI 1726-2012 was based on 2/3 of Maximum Considered Earthquake (MCE), which is earthquake with 2500-year return period. The factor of 2/3 is taken as a margin, when at the time the structure was subjected to design earthquake (2/3 of earthquake with 2500-year return period), there would be no major damages and could be reused with a number of necessary improvements. Meanwhile, when the structure was subjected to maximum considered earthquake (earthquake with 2500-year return period), major damages were permitted, but the structure should not collapse [6].

Comparison of elastic design response spectra for Balikpapan city, with soft soil site class based on PPTGIUG 1983 and SNI 1726-2012 can be seen in Figure 1. It can be seen that the elastic response spectrum based on SNI 1726-2012 is larger than that of the PPTGIUG 1983. Further, to take into account the ductility and over

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strength factor in special moment resisting frame (SMRF), those elastic design response spectra are reduced to nominal design response spectra which can be seen in Figure 2.

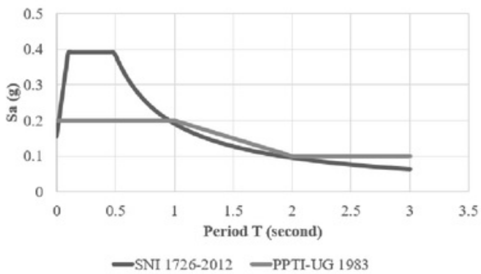

Fig. 1. Elastic Design Response Spectra of Balikpapan City (Soft Soil) According to PPTGIUG 1983 and SNI 1726-2012

It can be seen that the nominal design response spectra based on SNI 1726-2012 is smaller than that of PPTGIUG 1983. This is due to the different ductility and over strength factors used in both codes to reduce the elastic design response spectra. The seismic reduction factors of 7 and 4 are used in SNI 1726-2012 and PPTGIUG 1983, respectively. Differences of these factors related to relevant structural concrete codes. The structural concrete codes that correspond to PPTGIUG 1983 and SNI 1726-2012 are the SKSNI T-15-1991-03 and SNI 2847-2013 [8], respectively.

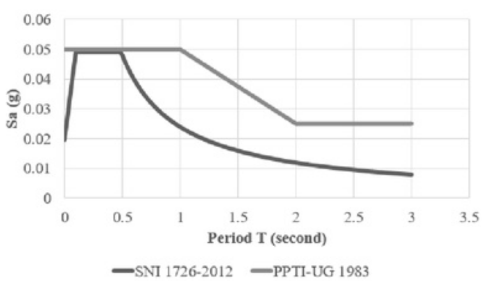

Fig. 2. Nominal Design Response Spectra (SMRF) of Balikpapan City (Soft Soil) According to PPTGIUG 1983 and SNI 1726-2012

3 SKSNI T-15-1991-03 and SNI 2847-2013

There are some differences between Indonesian structural concrete codes SKSNI T-15-1991-03 and SNI 2847-2013, in term of the Capacity Design provisions in SMRF Design. One significant difference between the two is the minimum requirement of the ratio of total nominal strength in columns with respect to that in beams adjoining to a connection. In SNI 2847-2013 [8], this ratio is equal to 1.2 as shown in Equation 1. In SKSNI T-15-1991-03 [5], this ratio can be determined from Equation 2. By converting the ultimate moment of the columns to nominal moment and substitute appropriate value of dynamic magnification factor and over strength factor, it was found that the ratio was about 1.7 [9].

$$\sum M_{n,k} = 1.2 \ x \sum M_{n,b} \tag{1}$$

where:

$M_{n,k}$ = Nominal Flexural Strength of Colums
$M_{n,b}$ = Nominal Flexural Strength of Beams

$$\sum M_{u,k} = 0.7 \ x \ \omega_d \ x \sum M_{kap,b} \tag{2}$$

where:

$M_{u,k}$ = Ultimate Flexural Strength of Colums
ω_d = Dynamic Magnification Factor (1.3)
$M_{kap,b}$ = Flexure Capacity of Beams

$$M_{kap,b} = \emptyset_0 \ x \ M_{nak,b} \tag{3}$$

where:

$\emptyset_0$ = Over Strength Factor
$M_{nak,b}$ = Nominal Flexure Strength of Beams

However, the SNI 2847-2013 specifies much stringent provision for column stirrups. Minimum stirrups A_{sh} (Equations 4 and 5) should be provided to ensure adequate curvature capacity in yielding regions. This is intended to maintain the axial load strength of columns if concrete cover spalls. There is also a difference in the requirements of the stirrups for beams, where SNI 2847-2013 also provides slightly more stringent requirements than SKSNI T-15-1991-03.

$$A_{sh} = 0.3 \ x \ (s \ x \ b_c \ x \ f_c'/f_{yt}) \ x \ [(A_g/A_{ch}) - 1] \tag{4}$$

$$A_{sh} = 0.09 \ x \ (s \ x \ b_c \ x \ f_c'/f_{yt}) \tag{5}$$

where:

s = Centre to Centre Stirrups Spacing
b_c = Cross Sectional Dimension of Column Core
f_{yt} = Specified Yield Strength of Stirrups
A_g = Gross Area of Column Section
A_{ch} = Core Area of Column Section

With those updates in both seismic and structural concrete codes, assessment of any buildings designed based on older codes should be conducted. In this study, a hotel in Balikpapan City – Indonesia which was designed by using the older codes (PPTGIUG 1983 and SKSNI T-15-1991-03) is chosen to be investigated.

4 Considered Building

The structure of the 9-storey hotel consists of special moment resisting frame and shear wall systems. The typical structural plan view of the building can be seen in Figure 3. The shear wall positions are marked in Figure 3. SAP2000 structural analysis software was used to model and analyse the structure. The 3D model of the considered building can be seen in Figure 4. Non-linear direct integration time history analysis was conducted to analyse the building performance. The seismic load used was a spectrum consistent ground acceleration generated from

El Centro 18 May 1940 North-South component in accordance to the new code. The original ground acceleration is shown in Figure 5, while the modified ground acceleration (which response spectra was matched to that of SNI 1726-2012) is shown in Figure 6.

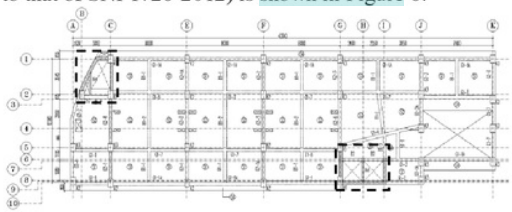

Fig. 3. Typical Structural Plan View of the Hotel

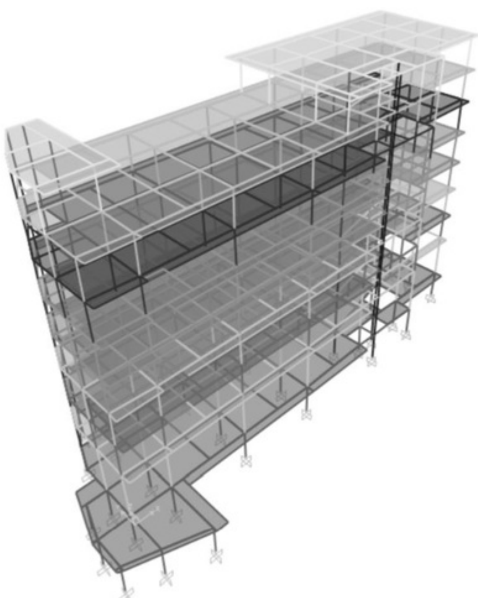

Fig. 4. 3D Model of the Considered Building

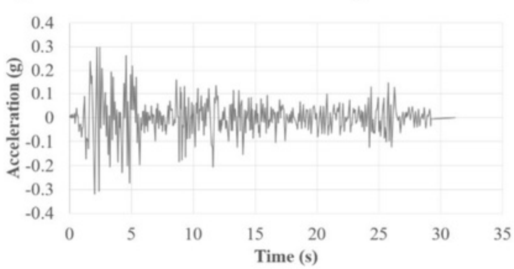

Fig. 5. Original Ground Acceleration

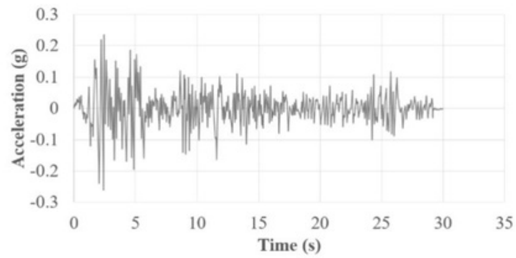

Fig. 6. Modified Ground Acceleration

In both ends of beam elements, non-linearity is modelled for bending and shear of their major axis. While in both ends of column and shear wall elements, non-linearity is modelled for shear in both axis as well as biaxial bending interaction. The moment-curvature and shear force-displacement for the beams and columns were obtained by using CUMBIA [10].

5 Analysis and Result

The displacement of the building subjected to previously mentioned ground accelerations (matched to response spectrum of elastic design earthquake and MCE of SNI 1726-2012) in both directions can be seen in Figures 7 and 8. It should be noted that the displacement profiles seen in the figures are not the real deformed shape, since the presented values are the maximum displacement during 30 seconds of full time history analysis that may not occur in the same time on each story.

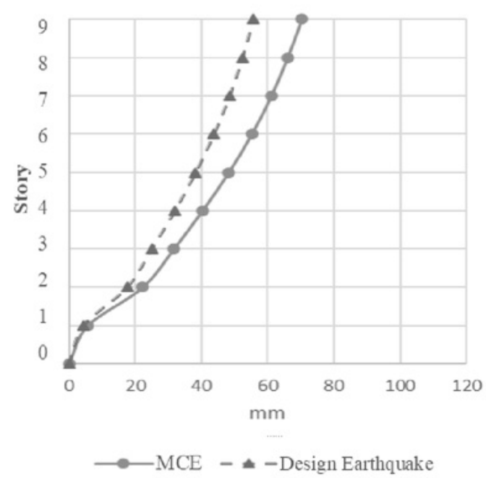

Fig. 7. Displacement in X Direction

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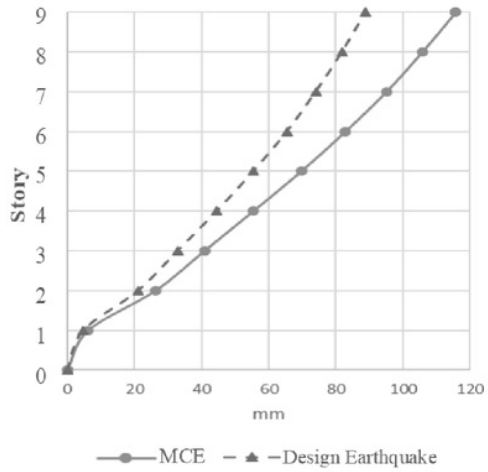

Fig. 8. Displacement in Y Direction

One of the more common parameters to assess the performance of the structure is to use the drift ratio, which in this study uses performance level limits based on FEMA 356. The largest drift ratio during time history analysis of each floor due to elastic design earthquake and maximum considered earthquake in x and y directions can be seen in Figures 9 and 10.

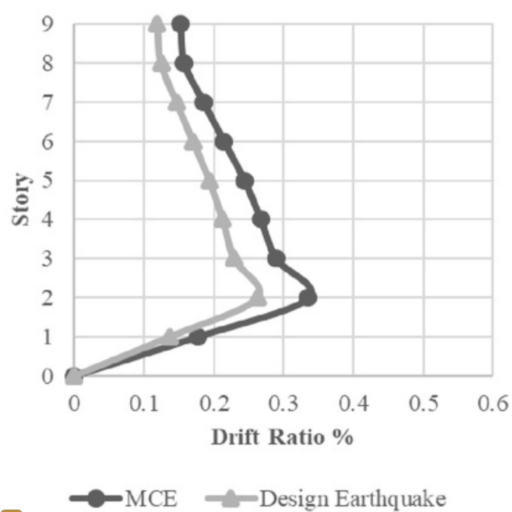

Fig. 9. Drift Ratio in X Direction

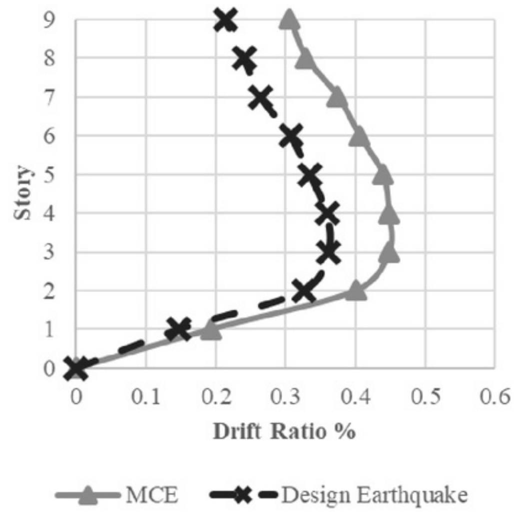

Fig. 10. Drift Ratio in Y Direction

The performance level of the building according to FEMA 356 can be seen in Table 1. The performance levels "OL", "IO", "LS", and "CP" in Table 1 refer to Operational Level, Immediate Occupancy, Life Safety, and Collapse Prevention, respectively.

Table 1. Performance Level of the Building According to FEMA 356.

Earthquake Level		**Performance level**			
		OL	**IO**	**LS**	**CP**
Design Earthquake (2/3 MCE)	X-dir.	-	0.263	-	-
	Y-dir.	-	0.362	-	-
Max. Considered Earthquake (MCE)	X-dir.	-	0.335	-	-
	Y-dir.	-	0.448	-	-
Max. Drift Ratio (%)		0.0	0.0-0.5	0.5-1.0	1.0-2.0

It can be seen that the performance of the structure is very good. Based on the maximum drift ratio, the structure is still at Immediate Occupancy Level while Collapse Prevention Performance Level is still considered satisfactory for building subjected to earthquake with 2500-year return period.

Beside of drift ratio, other parameter that can be used to determine the performance of buildings is the member damage index. In this study, the damage index is determined by Equation 6 [11].

$$DI = (\mu_m-1)/(\mu_u-1) \qquad (6)$$

where:

DI = Damage Index
μ_m = Maximum Ductility
μ_u = Ultimate Ductility

It is observed that the beam members experienced more severe damage than the column elements and shear wall elements. The maximum member damage indices of the building are presented in Table 2.

Table 2. Performance Level of the Building According to ACMC.

Earthquake Level		Performance level			
		O	**SLS**	**DCLS**	**S**
Design Earthquake (2/3 MCE)	X-dir.	0.022	-	-	-
	Y-dir.	0.027	-	-	-
Max. Considered Earthquake (MCE)	X-dir.	0.043	-	-	-
	Y-dir.	0.041	-	-	-
Max. Damage Index (%)		<0.1	0.1-0.25	0.25-0.40	0.40-1.00

Classification of the performance from the damage indices occurred is based on the Asian Concrete Model Code [7]. The performance levels “O”, “SLS”, “DCLS”, and “S” in Table 2 refer to Operational, Serviceability Limit State, Damage Control Limit State, and Safety, respectively. It can be seen that the performance of the building is very good, that it is still at the operational level for both elastic design earthquake (2/3 MCE) and maximum considered earthquake (2500-year return period).

Figures 11 and 12 show typical damages (plastic hinges location) of the building due to elastic design earthquake in X and Y directions, respectively. While Figures 13 and 14 show the plastic damages of the building due to maximum considered earthquake (MCE) in X and Y directions, respectively.

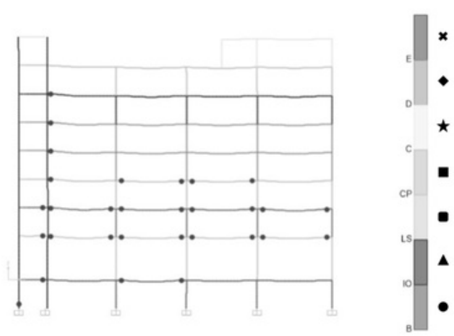

Fig. 11. Plastic Hinges Location of Frame 2 due to Design Earthquake in X Direction

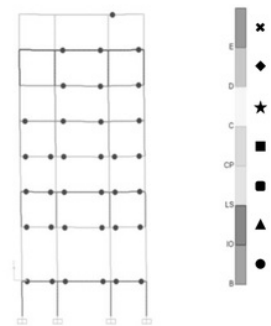

Fig. 12. Plastic Hinges Location of Frame F due to Design Earthquake in Y Direction

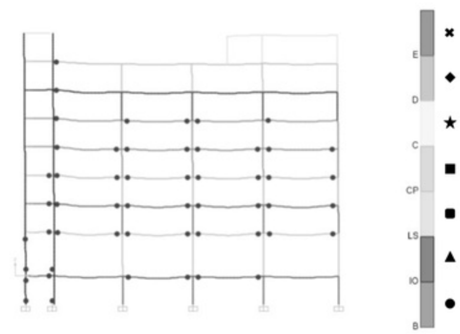

Fig. 13. Plastic Hinges Location of Frame 2 due to MCE in X Direction

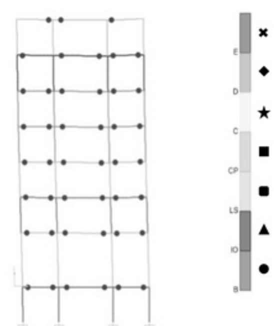

Fig. 14. Plastic Hinges Location of Frame F due to MCE in Y Direction

6 Conclusions

Based on the analysis that has been done one the existing building structure of a 9-storey hotel in Balikpapan, some conclusions can be obtained as follows:

1. The hotel building which was designed as SMRF according to older seismic and structural concrete codes (PPTGIUG 1983 and SKSNI T-15-1991-03) has shown very good performance against higher load demand specified by the newer seismic code (SNI 1726-2012).

2. Due to the design earthquake (2/3 MCE) and maximum considered earthquake (MCE, 2500-year return period) according to SNI 1726-2012, the hotel building shows maximum drift ratio below 0.5% and maximum damage index below 0.1. The performance is classified as Immediate Occupancy Level (according to FEMA 356) and Operational Level (according to ACMC 2001) which is very satisfactory.

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