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# COMPARISON OF DESIGNING HIGH-RISE BUILDINGS IN JAKARTA BASED ON PRESCRIPTIVE CODE AND ALTERNATIVES DUE TO SPECIFIC EARTHQUAKE MECHANISM INDONESIA

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## *Abstract*

High-rise buildings have recently become popular in Indonesia. Particularly, Jakarta, the capital city of Indonesia, has limited spaces left for livings and citizen's activities. The vertical space development is a solution for the rapid development of Jakarta. Most developers catch the opportunity and race in constructing taller buildings. One of the tall buildings, that is under construction, is a 72-floor-building with the height spans to 326.35 meter above the ground. The lateral restraint system of the buildings consists of shear walls, outriggers and belt trusses, which make this study more interesting.

The building is designed based on the prescriptive code. However, nowadays, alternatives are introduced worldwide to design tall buildings with a more reliable solution, written with intention of design for tall buildings. In this paper, we study the design by alternatives to provide more reliable solution for the tall building. The code specifies the seismic design based on two third of the Maximum Considered Earthquake (MCE), but the alternatives, such as LATBSDC 2014 and TBI 2010, specifies the analysis using the MCE as an objective to analyze the performance of the structures. To present a realistic analysis, we consider specific potential earthquake in Indonesia with earthquake mechanism such as megathrust, benioff, shallow crustal, and shallow background. This research for answering the challenge of probability MCE level occurs in Indonesia.

*keywords: high-rise buildings, prescriptive code, LATBSDC 2014, TBI 2010, Megathrust, Benioff, Shallow Crustal, and Shallow Background.*

## 1. Introduction

Jakarta as the capital city of Indonesia has population increases rapidly. The predicate is the second largest urban area population according to Demoghrapia World Urban in 2015. Buildings that rise vertically are appropriate to resolve the condition of Jakarta's population.

The one of great challenge of design tall building in Jakarta is earthquake. Jakarta is surrounded with many sources of earthquake mechanism, such as Megathrust, Benioff, Shallow Crustal, and Shallow Background.

Sunda Megathrust is a fault between the overriding Eurasian Plate and the subducting Indo-Australian plate that lies on from Myanmar to Bali. The distance from Jakarta to Sunda Megathrust is 170 km. The source of Sunda Megathrust mechanism has potency 9 magnitude earthquake. Sumatra Megathrust has occurred giant earthquake in 9 magnitude in 2004. Java Benioff zone is one of the active zone with 50 km - 500 km depth. Shallow Crustal is the source of earthquake mechanism more than 50 km depth while Shallow Background is less than 50 km. Figure 1 describes the sources of earthquake mechanisms in Jakarta Surrounding.

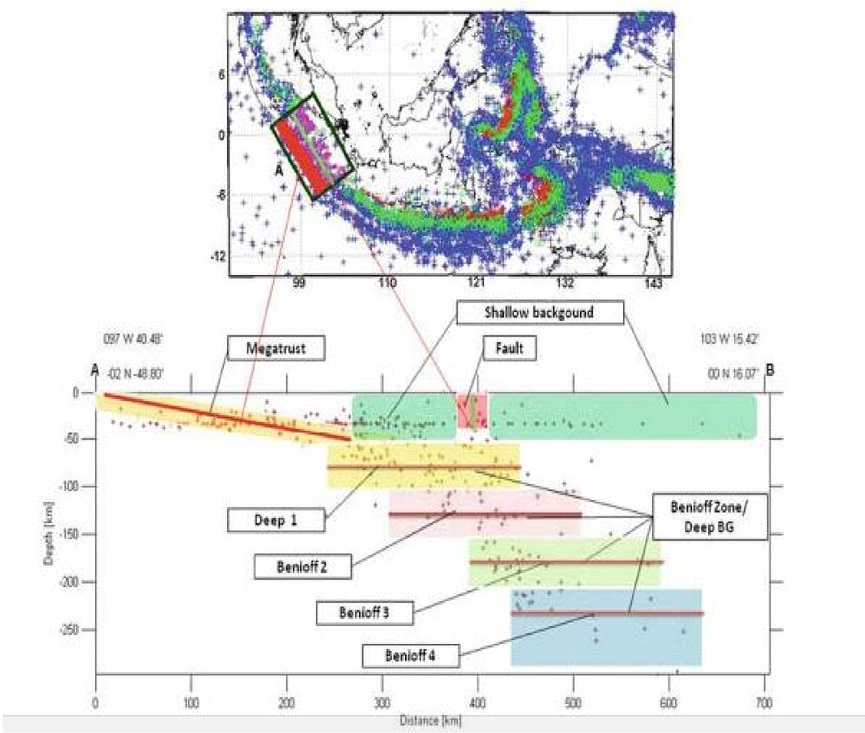



**Figure1.** Earthquake Mechanism (Irsyam et al, 2011)

To address the challenges of the earthquake mechanism in Jakarta surrounding, this paper studies the impact of the earthquake mechanism to find the appropriate design for building in Jakarta. This research focuses on comparison of building design based on prescriptive code (SNI 1726-2012) and performance-based design (PBD).

The prescriptive code which is used in Indonesia is based on ASCE 7-10. On the other hand, PBD guidelines, such as Los Angeles Tall Buildings Structural Design Council (LATBSDC 2014) and Tall building Initiative (TBI 2010), are most popular to design tall Building. The difference procedure of the seismic design between Code and PBD is the prescriptive code requires minimum strength Cs under design level and PBD requires building analysis under service level and Maximum Considered Earthquake level.

SNI 1726-2012 has some assumptions to design, such as Cs minimum, redundancy factor, response modification factor, overstrength factor, and drift amplification factor, whereas PBD requires reliable design to avoid using the assumption factors in code. The impact of the assumption factors in code makes the structure size is larger than the reality.

This research analyzes Tower 1 with 72 stories, which is one of the tallest buildings in Indonesia. Tower 1 has height 326,35 m above ground and the lateral restraint systems are shear wall, outriggers, and belt truss. The optimization of Tower 1 is conducted based on concept capacity design in PBD. The aim of the research is to find the appropriate building design under mechanism earthquake in Indonesia.

## 2. Theoretical Background

### 2.1 Earthquake Analysis

Earthquake analysis is a part of structure analysis to calculate the behavior due to seismic. The analysis is conducted to get the appropriate design, which resists to seismic. The Earthquake analysis consists of response history analysis and response spectrum analysis. Response history analysis requires structure must satisfy during an earthquake. The analysis determines the response structures to earthquake ground motion $\ddot{u}_g(t)$.

$$p - f_s - f_D = m\ddot{u} \text{ or } \quad m\ddot{u} + f_s + f_D = p(t) \tag{1}$$

Where $p(t) = - m\, \ddot{u}_g(t)$, m is mass, $f_s$ is elastic or inelastic resisting force, and $f_D$ is damping force.

Linier dynamic analysis uses response spectrum as input ground motion earthquake using a plot of peak or steady-state response (acceleration, velocity, and displacement) for analyzing structure performance. Each plot uses principle single degree of freedom system (SDOF) having a fixed damping ratio ζ. The peak response is plotted against natural period $T_n$ because natural period of vibration is more familiar concept than natural frequency. The peak response as followed (Chopra, 1995):

$$u_0(T_n, \zeta) = \max \left| u(t, T_n, \zeta) \right| \tag{2}$$

$$v_0(T_n, \zeta) = \max \left| v(t, T_n, \zeta) \right| \tag{3}$$

$$a_0(T_n, \zeta) = \max \left| a(t, T_n, \zeta) \right| \tag{4}$$

The inelastic analysis is simplified with the pushover analysis. The pushover analysis uses assumption which is controlled by first few mode of vibration through elastic and inelastic response until an ultimate condition. Figure2 shows the pushover concept.

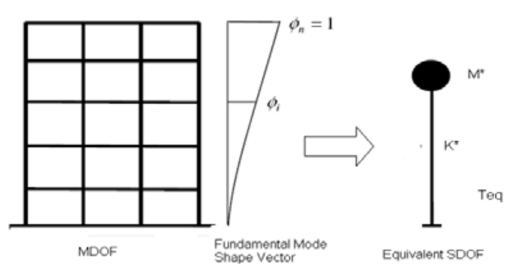



**Figure 2**. The Pushover Concept

In which M* is mass, mass can be represented as equivalent to the weight of the building, and K* is stiffness of building. The initial period $T_{eq}$ of the equivalent SDOF system is formulated as:

$$Teq = 2\pi\sqrt{\frac{M^*}{K^*}} \tag{5}$$

## 2.2 Selecting and scaling Ground Motion

Each earthquake has unique characteristics that depend on the earthquake magnitude, distance, source, attenuation, and local site condition. Ground motions are selected and scaled to be used as input earthquake for structural design. PBD guidelines provide the rules for selecting and scaling ground motions. A minimum of seven appropriate ground motions must be used for analysis. Each accelerogram selected must consist of at least two horizontal components. The selected ground motions are generally compatible with the earthquake magnitude and site source distance found from deaggregation. Scaling ground motion methods which are permitted PBD guidelines are spectral matching with modifying the frequency content of the ground motion, code scaling in time domain, and conditional mean spectrum. The advantages are to reduce the variability of records substantially and possible to enhance some of the frequencies.

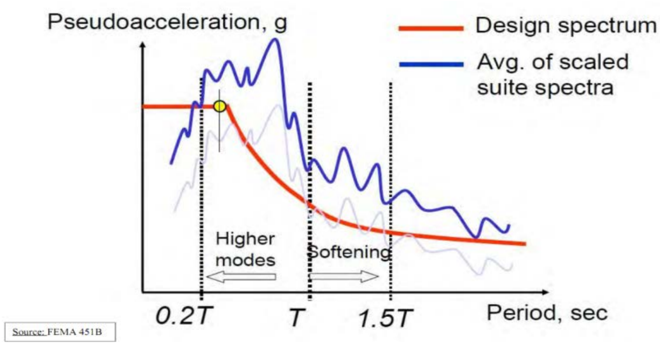



**Figure 3.** Code Scaling Method in Time Domain (Naeim F, 2015)

For scaling method, the period range is 0.2T to 1.5T, where T is first mode period. Figure 3 describes scaling method. The period range of scaling method is 0.2T to 1.5T, where T is first mode period. The scaling methods which are permitted PBD guidelines are spectral matching with modifying the frequency content of the ground motion, direct scaling in time domain, and conditional mean spectrum. The method of spectral matching is to reduce the variability of records substantially and possible to enhance some of the frequencies. In contrast, the method of direct scaling is to determine a constant scale factor with the amplitude of accelerogram.

## 2.2 Comparison of SNI 1726-2012 and PBD Guidelines

The appropriate design is supposed to establish structure design, feature architect, and cost. The guideline mostly used in Indonesia is SNI 1726-2012. But, many research propose alternative guidelines (such as Tall Building Initiative 2010 and Los Angels Tall Building Structure Design 2014) to create an appropriate design. Prescriptive code SNI 1726-2012 and PBD guidelines (TBI 2010 and LATBSDC 2014) have different method to design tall building. Table 1 shows the general comparison design between SNI 1726-2012 and PBD guidelines. The PBD guidelines require design in service level (43 years) with linear or nonlinear dynamic procedure and MCE level (2475 years) with nonlinear time history analysis to get reliable structure.

**Table 1.** Comparison of SNI1726-2012 and PBD Guidelines

| Topic | SNI 1726-2012 | TBI 2010 | | LATSBDC 2014 | |
|---|---|---|---|---|---|
| | | Service Level | MCE level | Service Level | MCE level |
| Ground Motion (number of years) | 2475 years | 43 years | 2475 years | 43 years | 2475 years |
| Type of Analysis | LDP | LDP or NDP | NDP | LDP or NDP | NDP |
| Minimum Strength | Vbase = 0.85Cs W | 43 years, 2.5% Damping | 2475 years, 2.5% Damping | 43 years, 2.5% Damping | 2475 years, refers to ATC-72 |

Prescriptive code SNI 1726-2012 and PBD guidelines (TBI 2010 and LATBSDC 2014) have different methods to design tall building. SNI 1726-2012 uses strength based concept, which considers minimum strength 0.85CsW. The aim of strength based is to minimize the weakness and maximize the strength. SNI 1726-2012 has required that building be built to a minimum level of safety. Specifically, the codes are expected to resist minor level of earthquake without damage, moderate level with some nonstructural damage, and major level of earthquake without collapse. To achieve satisfactory level, SNI 1726-2012 requires the building analysis in design level. The building structure has structure response coefficients, such as response modification factor R, deflection amplification factor $C_d$ , over-strength factor $\Omega_{0.}$ The definition of Response modification factor R is an important parameter that accounts on structural capacity to dissipate energy through inelastic behavior, Deflection amplification factor $C_d$ is a parameter to predict maximum deflection from deflection under design seismic force, and Over-strength factor $\Omega_0$ are amplified factor to estimate capacity design. These coefficients are needed to design with linear analysis method.

On the contrary, Performance-based guidelines, such as TBI 2010 and LATBSDC 2014, implement a capacity concept to design. Capacity concept is used to control elements structure in force control and deformation control, which depend on element capacity. Force control design actions for element characterized sharp loss strength and deformation control for elements, which have capable large deformation without significant loss strength.

Table 1 shows the comparisons of stiffness modifier for SNI 1726-2012 in design level and TBI guidelines in service level and MCE level. Table 3 shows the comparisons of stiffness modifier for SNI 1726-2012 in design level and TBI guidelines in service level and MCE level.

**Table 3**. Reinforced Concrete Stiffness Properties

| Element | SNI 1726-2012 | TBI Guidelines | |
|---|---|---|---|
| | | **Service Level** | **MCE Level** |
| Structure Wall | Flexural-0.7 I g<br>Shear-1.0 A g | Flexural-0.75 I g<br>Shear-1.0 A g | Flexural-1.0 Ec*<br>Shear-0.5 A g |
| Basement Wall | Flexural-0.7 I g<br>Shear-0.7 A g | Flexural-1 I g<br>Shear-1.0 A g | Flexural-0.8 I g<br>Shear-0.5 A g |
| Coupling Beam | Flexural-0.35 I g<br>Shear-0.35 A g | Flexural-0.3 I g<br>Shear-1.0 A g | Flexural-0.2 I g<br>Shear-1.0 A g |
| Diaphragm | Flexural-0.25 I g<br>Shear-0.25A g | Flexural-0.5 I g<br>Shear-0.8 A g | Flexural-0.25 I g<br>Shear-0.25A g |
| Moment Frames Beams | Flexural- 0.35 I g<br>Shear-1.0 A g | Flexural-0.7 I g<br>Shear-1.0 A g | Flexural- 0.35 I g<br>Shear-1.0 A g |

| Moment Frames Columns | Flexural- 0.7 I g<br>Shear-1.0 A g | Flexural-0.9 I g<br>Shear-1.0 A g | Flexural- 0.7 I g<br>Shear-1.0 A g |
|---|---|---|---|

***Ec** = 57000 $(fc')^{0.5}$ for $fc' \leq 6000$ psi ; Ec = 40000 $(fc')^{0.5}$ + $1\text{x}10^6$ for $fc'$> 6000 psi

## 3. Methodology

The selected ground motions are considered with earthquake mechanisms in radius 500 km from Jakarta. The ground motions use Probabilistic Seismic Hazard Analysis (PSHA) and Site Specific Response Analysis (SSRA). Then, the scaling of ground motion is conducted with three-dimensional scaling methods.

The Preliminary of Tower 1 is designed based on prescriptive code SNI 1726-2012. The code requires earthquake design two third of MCE. On the other hand, the research is conducting optimization based on PBD. Tower 1 based on PBD is designed under service level defined by 50% probability earthquake in 30 years and MCE level defined by 2% probability earthquake in 50 years. Nonlinear Dynamic Analysis is used for designing Tower 1 in service level and maximum considered earthquake.

## 4. Model Description

### 4.1 Ground Motion

The Ground motions have considered the earthquake mechanism in Jakarta Surrounding. Jakarta surrounding has potential earthquake mechanism, such as Megathrust, Benioff, Shallow Crustal, and Shallow Background. Singara, IW (2015) conducted Probabilistic Seismic Hazard Analysis (PSHA) and Site Specific Response Analysis (SSRA) to find 7 pairs of ground motions which represent potential specific earthquake for Jakarta. According to Singara (2015), the matched ground motions are Chi-chi earthquake, Tohoku earthquake, Padang earthquake, Whittier Narrows-01 earthquake, and Landres earthquake. The selected ground motions are shown in table 4 below.

**Table 4.** The Selected Ground Motions for Jakarta ( Singara, IW., 2015)

| Period (s) | Mechanism | Catalog | Earthquake | Magnitude |
|---|---|---|---|---|
| 5 | Megathrust | ILA051-N[(16)] | Chi-Chi Earthquake 20-Sept-1999 | 7.62 |
| | | MYG013110311146EW.at2[(17)] | Tohoku Earthquake 11-March-2011 | 9.00 |
| 10 | Megathrust | TAP075-N[(16)] | Chi-Chi Earthquake 20-Sept-1999 | 7.62 |
| | | MYG12110311146EW[(17)] | Tohoku Earthquake 11-March-2011 | 9.00 |
| | Benioff | Padang 30-11-2009[(18)] | Padang Earthquake 30-Sept-2009 | 7.60 |
| | Shallow Background | A-ORR000[(16)] | Whittier Narrows-01 Earthquake 10-Jan-1987 | 5.99 |
| | Shallow Crustal | SER270[(16)] | Landres Earthquake 29-June-1992 | 7.28 |

### 4.2 Building Structure

Tower 1 is one of the tallest buildings under construction in Indonesia. The tower has height of 326.35 meter, 72 stories, above 6 basements. This building uses reinforced concrete frames and the strength from Fc' 35 Mpa to 65 Mpa with rebar of 400 Mpa and steel profile 345 Mpa. This research analyzes Tower 1 with SNI 1726-2012 as preliminary design Tower 1A and optimizes Tower 1 with PBD into Tower 1B. The research uses Etabs v.13 for Tower 1 A and SAP 2000 for Tower 1B. SNI 1726-2012 requires pushover for system structure using belt truss and outrigger and PBD requires Nonlinear Dynamic Analysis. Table 5 describes dimension structure element and figure 4 shows structure model Tower 1A and Tower 1B.

**Table 5.** Dimension of Structure Element

| Structure Element | Tower 1A | Tower 1B |
|---|---|---|
| **Beam** | B 500x850 | B 500x850 |
| | B 700x1000 | B 700x1000 |
| | B 800x1000 | B 800x1000 |
| | B 900x1000 | B 900x1000 |
| | B 1600x2200 | B 1600x2200 |
| **Column** | K1 600x1200 – 2200x2200 | K1 600x1200 – 2200x2200 |
| | K1A 600x1200 – 2500x2500 | K1A 600x1200 – 2200x2200 |
| | K2 700x1500 – 1950x1950 | K2 700x1500 – 1950x1950 |
| | K2A 700x1750 – 2050x2050 | K2A 700x1750 – 2050x2050 |
| **Shear Wall** | W1 = 500 – 900 | W1 = 500 – 900 |
| | W2 = 500 – 900 | W2 = 500 – 900 |
| | W3 = 500 – 900 | W3 = 500 – 900 |
| | W4 = 500 – 900 | W4 = 500 – 900 |
| | W5 = 500 – 900 | W5 = 500 – 900 |
| | W6 = 500 – 900 | W6 = 500 – 900 |
| | Typical Shearwall LGF-L57 | Typical Shearwall LGF-L72 |
| | Typical Shearwall L57-L72 | |
| **Outrigger and Belt-truss** | Used | Not Used |

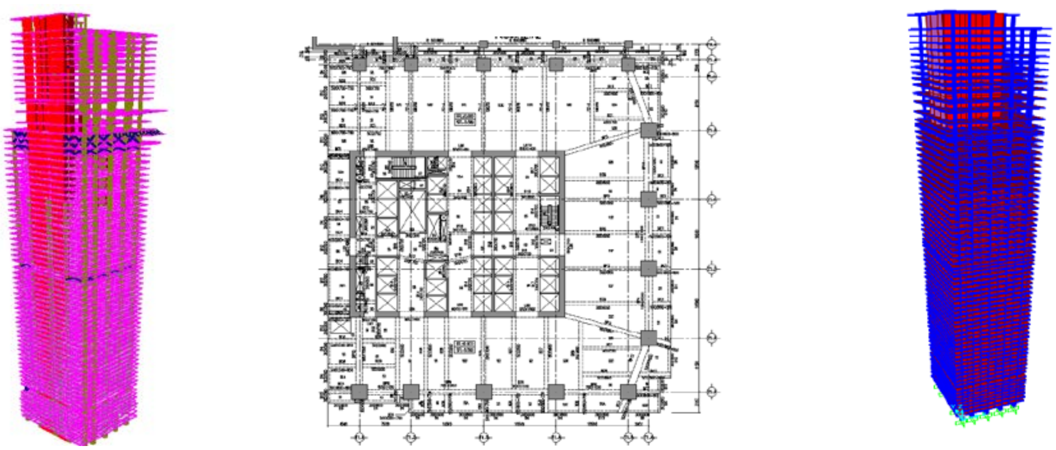


Figure 4. Structure Model Tower 1A (left), floor plan and Tower 1B (right)

## 5. Analysis and Result

5.1 Grounds Motion

The ground motions are scaled in time domain 0.2T to 1.5T (shown in figure 5). Figure 6 shows the ground motions in Jakarta under MCE level. The spectra averages of selected motions are not less than the target response spectra for the period between 0.2T and 1.5T.

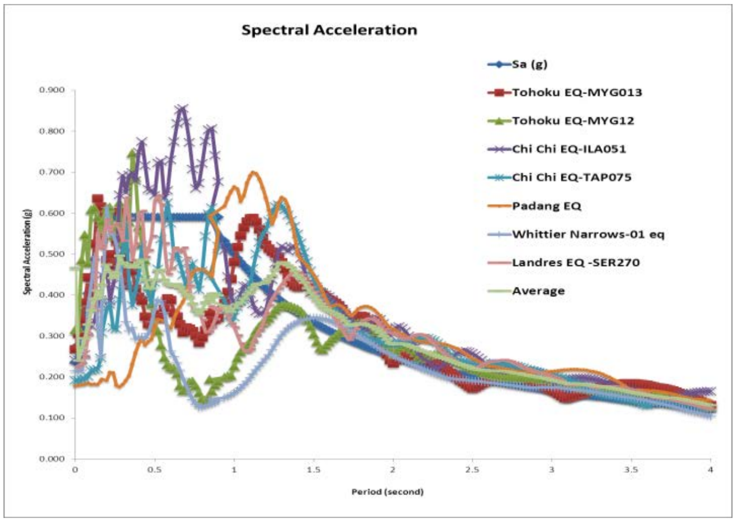
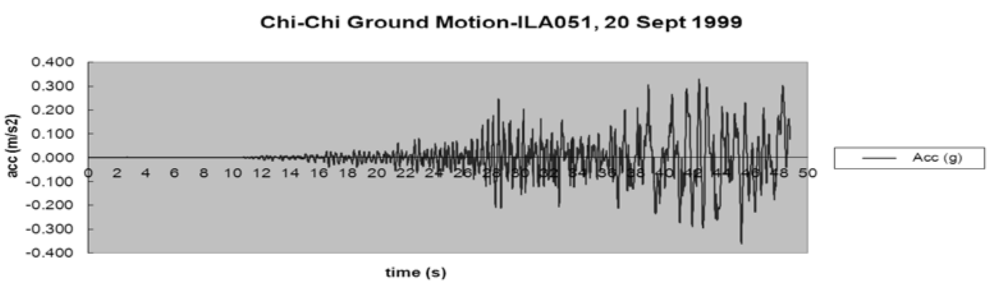
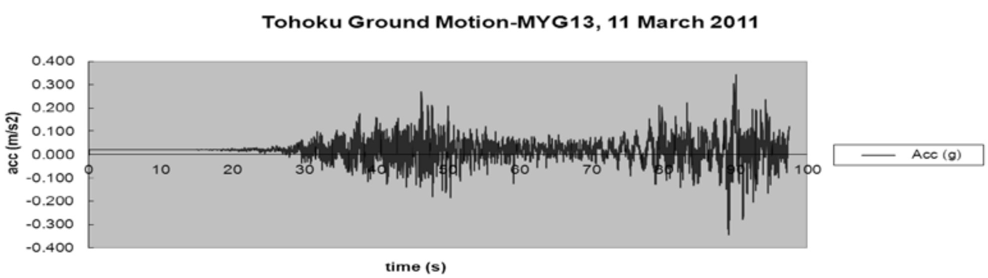
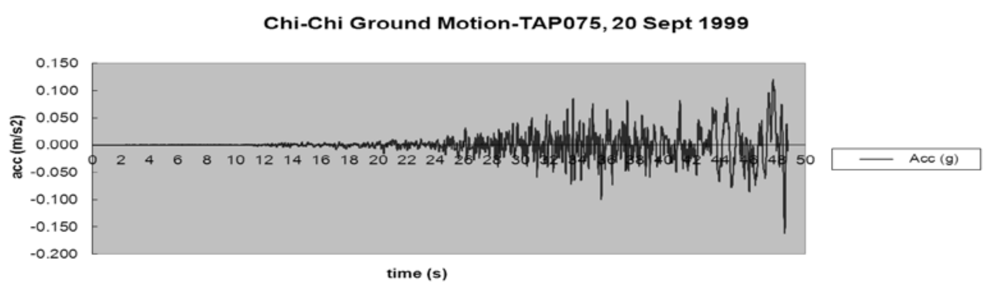



**Figure 5.** Response Spectra Scaled



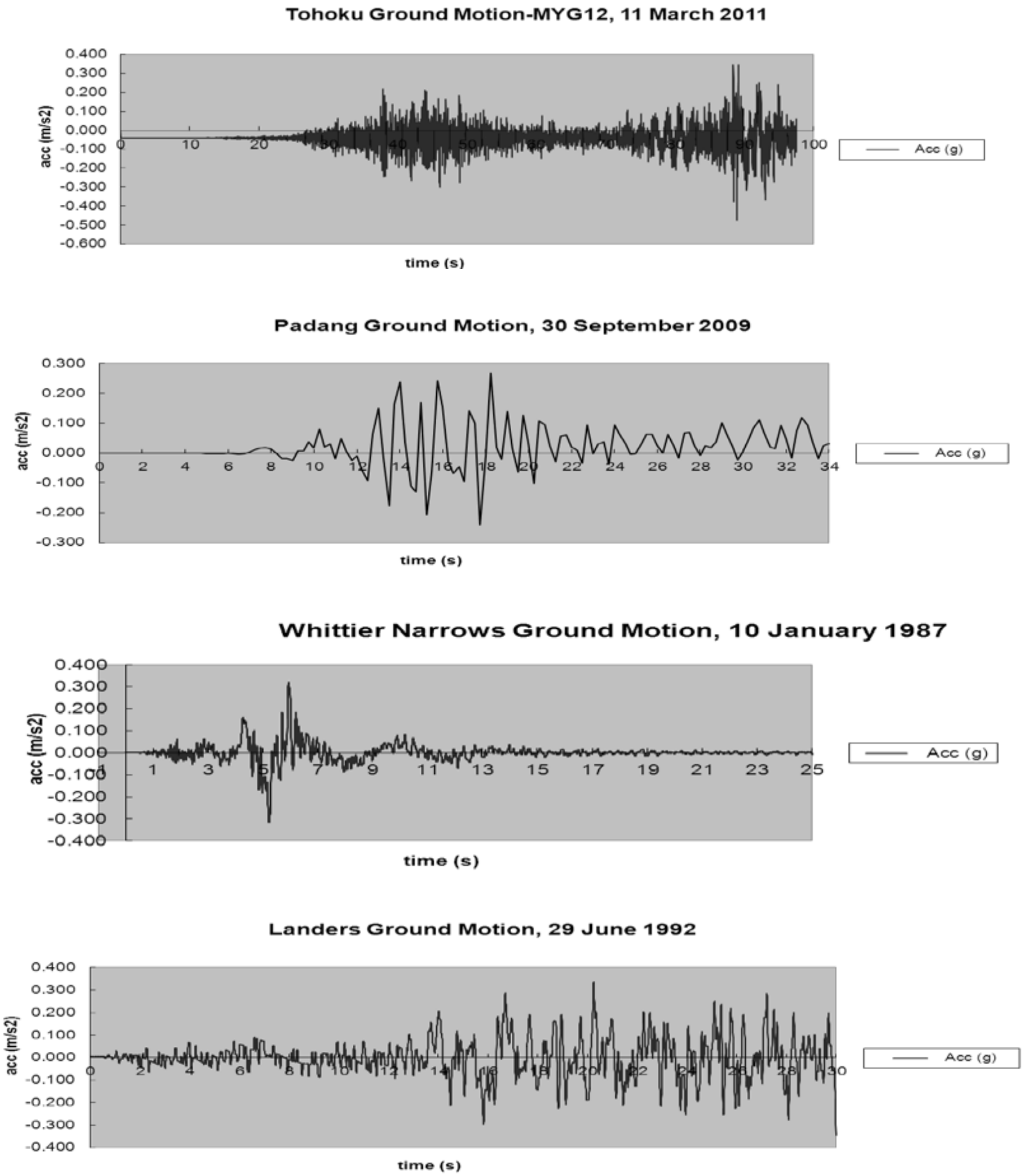



**Figure 6.** Ground Motion Scaled in 0.2T to 1.5T

## 5.2 Building Behavior

### 5.2.1 Modal Analysis

The period of Tower 1A is 7.13 s and 6.92 s and the natural period of the Tower 1B is 8.44 s and 7.9 s. The Tower 1A has modal participating ratio in principal direction x-axis 0.63 and y-axis 0.62 and Tower 1B in x-axis 0.62 and y-axis 0.62.

### 5.2.2 Story Drift

Table 6 shows roof drift of Tower 1A is 0.58% in design level. Tower 1A is designed by minimum strength requirement 85% Cs W. The drift ratio of Tower 1A has been multiplied by $C_d$ factor. Tower 1B is analyzed with 7 selected ground motions through non linear time history analysis. The máximum roof drift percentage under Maximum Considered Earthquake is 0.67% and the average is 0.48%.

**Table 6.** Roof Drift

| Tower 1A –CBD | Roof Drift |
|---|---|
| Drift inelastic | 0.58% |
| | |
| **Tower 1B -PBD** | **Roof Drift (NLTHA)** |
| Chi-chi EQ-ILA051 | 0.67% |
| Tohoku EQ-MYG13 | 0.59% |
| Chi-chi EQ-TAP075 | 0.56% |
| Tohoku EQ-MYG12 | 0.37% |
| Padang EQ | 0.37% |
| Whittier EQ | 0.29% |
| Landers EQ | 0.49% |
| Drift Average | 0.48% |

5.2.3 Structure Parameter

Structure parameters or structure response coefficients (R, $\Omega_0$, and $C_d$) are the important parameters that account on building capacity. Structure parameters are calculated with ASCE 7-10. Table 7 describes displacement and base shear. Table 8 shows structure parameters of Tower 1A with Pushover Analysis and Tower 1B with nonlinear Time History Analysis.

**Table 7.** Base Shear and Displacement of Tower 1A and Tower 1B

| Tower 1A Design Level | Tower 1A-Pushover Analysis Design Level | | Tower 1B-NLTHA MCE Level | |
|---|---|---|---|---|
| | Displacement (m) | Base Shear (KN) | Displacement (m) | Base shear (KN) |
| Minimum Strength | 0.664 | 63183 | 0.28* | 96720* |
| Yield | 0.765 | 72797 | 0.50 | 124300 |
| Ultimate | 1.846 | 173632 | 2.18 | 293000 |

*Minimum Strength of Tower 1B remains elastic under Service Earthquake Level

**Table 8.** Structure Parameters of Tower

| Structure Response Coefficients of Tower | Tower 1A-CBD | | |
|---|---|---|---|
| | Design | Pushover (Actual) | Ratio (actual/design) |
| Ω0 | 2.5 | 2.4 | 0.95 |
| R | 7 | Sufficient* | 1.03* |
| Cd | 5.5 | 2.4 | 0.44 |

| Tower 1B-PBD | |
|---|---|
| NLTHA(Actual) | |
| Vu/Vy | 2.36 |
| Vy > Vminimum strength | Sufficient* |
| Uu/Uy | 4.36 |

*Vyield/Vdesign > 1

5.2.4 Building Performance

The performance can show the level damage of building. The performances are immediate occupancy (no damage in structure), life safety (minor damage in structure), and collapse prevention. Building performance under MCE level is shown in figure 7. Structure elements of Tower 1A achieve immediate occupancy 4.76% and elastic 95.24%. Tower 1B remains elastic under Service earthquake level and achieves Life Safety under MCE level. The performance of column Tower 1B is 88.3% elastic, 11.40% IO, and 0.3% LS.

(a) Performance Elements of Tower 1A achieve IO (b)Performance Elements of Tower 1B achieve LS

**Figure 7.** Building Elements Performance

5.2.5 Comparison of Reinforcement

Tower 1A using Code Based Design has reinforcement ratio which is larger than Tower 1B. Performance-based Design method has reduced ±23% reinforcement of column Tower 1. Table 9 shows the comparison of reinforcement between Tower 1A and 1B.

**Table 9.** Reinforcement Percentage of Column on Ground Floor Level

| Column Type | Number of Column | Maximum Reinforcement Percentage | | % reduction |
|---|---|---|---|---|
| | | Tower 1A | Tower 1B | |
| K1 2200x2200 | 4 | 4.12% | 3.00% | 27.18% |
| K1A 2500x2500 | 5 | 1.96% | 1.70% | 13.27% |
| K2 1950x1950 | 2 | 5.58% | 3.36% | 39.78% |
| K2A 2050x2050 | 3 | 4.06% | 3.00% | 26.11% |
| | | | Average of % reduction | 23.78% |

## 6. Conclusion

The response spectrum method (dynamic analysis) is required method by SNI 126-2012 (ASCE 7-10). In contrary, PBD guidelines permit response spectrum method to analyze in service level (frequent earthquake, probability 30% in 50 years) and nonlinear dynamic (time history) method to analyze building in maximum considered earthquake ( rare earthquake, probability 2% in 50 years).

The tower designed with SNI 1726-2012 and Performance-based guidelines (such as TBI 2010 and LATBSDC 2014) have shown the differences results. The preliminary design of Tower 1A based on SNI 1726-2012 shows conservative result because the tall building has the assumption of structure response coefficients. SNI 1726-2012 requires performance-based check with pushover analysis to determine response modification factors for structure using outrigger and belt-truss. The result of SNI 1726-2012 is the building achieves performance in IO. Structure elements of Tower 1A achieve immediate occupancy 4.76% and remain elastic 95.24%. Furthermore, to get the result approach the reality, the optimization is conducted with PBD guidelines (TBI 2010 and LATBSDC 2014). PBD guidelines have optimized percentage of reinforcement for Tower 1B without outrigger and belt-truss. The elements of column Tower 1B under Maximum Considered Earthquake achieve 88.3% elastic, 11.40% IO, and 0.3% LS. PBD method has reduced reinforcement percentage of column ± 23% for Tower 1B.

## 7. Acknowledgements

This research is supported by PT Wiratman Consultant.

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