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PERFORMANCE BASED SEISMIC DESIGN OF REINFORCED CONCRETE TALL BUILDINGS IN INDONESIA

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Abstract: Rapid development of tall building construction has taken place over the last decade in Indonesia, especially in its capital, Jakarta. Reinforced concrete has been the preferred material of choice used for these buildings because it is economical and is easily handled by local contractors. Along with this rapid development, the Indonesian codes for structural design practices have experienced major changes, following the latest development of USA building design codes and performance-based design guidelines, especially those related to seismic design. This paper describes the latest seismic code in Indonesia and presents the state-of-the-practice for the design of tall buildings there. It also discusses the use of performance-based seismic design as an alternative method of design, considering the risk targeted maximum and service earthquakes, in the structural design of a 55 story residential tower in Jakarta.

1 INTRODUCTION

Indonesia is a heavily populated country and archipelago slightly larger than Québec. Many islands in the country are dotted at the west end of the Pacific Ring of Fire. In the past twenty years, the economy in Indonesia has been expanding at a rate slightly above 5%. This expansion has resulted in a boom in construction of all types of buildings, including the construction of a number of tall buildings up to 300 m high. In the past fifteen years, Indonesia has also experienced a number of large earthquakes which have resulted in the loss of thousands of lives and in large economic losses. The catastrophic 2004 M_w 9.1 Aceh, 2005 M_w 8.6 Nias, M_w 6.4 Yogyakarta and M_w 7.6 Padang earthquakes prompted authorities in Indonesia to appoint a commission to develop a modern seismic design code for the country. The deliberations of the commission resulted in the publication in 2012 of the Indonesia Seismic Code SNI 1726:2012 (SNI, 2012). This code contains a number of prescriptive requirements drawn from ASCE 7-10 (ASCE, 2010) and IBC-2009 and is mandatory for the design of low and medium rise buildings. The seismic designs for tall buildings in Indonesia are being carried out using Performance Based Seismic

Design (PBSD) methodologies and rely largely on the guidelines published by the Pacific Earthquake Engineering Research Center (PEER) [PEER, 2010] and by the Los Angeles Tall Buildings Structural Design Council (LATBSDC) [LATBSDC, 2015].

In this paper, the authors summarize the current state-of-the-practice seismic design approach in Indonesia, including the design approach for tall buildings. For this purpose, the paper describes the design and verification procedure of a 55-story reinforced concrete residential tower. The design was carried out using linear analysis under the legal requirements of SNI 1726:2012. The verification of the performance was carried out using a nonlinear analysis of the structure. A detailed nonlinear finite element model was developed for the building and was subjected to an ensemble of two-component horizontal ground motions conditioned at two hazard levels for the site, i.e., service level and risk-targeted maximum considered earthquake. Key response parameters were verified against response limits contained in the PEER and LATBSDC PBSD guidelines.

2 PRECRIPTIVE SEISMIC CODES AND BUILDING AUTHORITY CONSENSUS

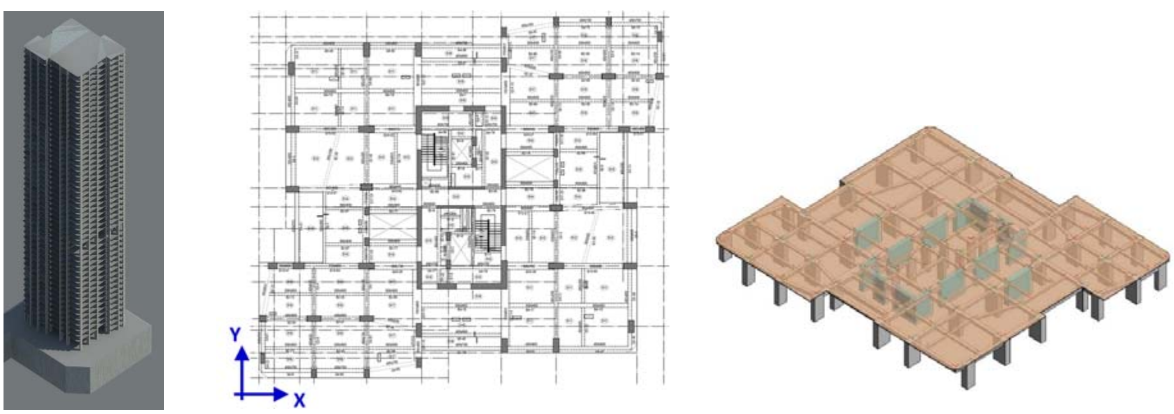
Structural engineers in Jakarta must submit the building structural design reports to the Jakarta Building Authority Committee (JBAC) for review and as a pre-requisite to obtain the building permit for buildings or eight or more stories. They also need to take into account the latest SNI 1726:2012 building code of Indonesia and other internationally recognized standards, codes or guidelines. The JBAC members are appointed by the Governor of Jakarta and their main task is to enforce the adoption of latest building regulations in Indonesia to ensure buildings meet satisfactory criteria for safety.

Until recently, force based design with Modal Response Spectrum Analysis (MRSA) was the most popular method used by structural engineers responsible for the design of multi-story buildings in Indonesia. The risk-targeted maximum considered earthquake seismic design criteria of 1% probability of building collapse have been adopted in SNI 1726:2012 and are already considered in the new seismic map. The MRSA in SNI 1726:2012 can be used to support the design of a building of any height. The building elements shall be detailed in accordance with the prescriptive provisions of the building standard of Indonesia SNI 2847:2013 (2013), which is based on ACI 318-11.

Structural engineers designing buildings 40 stories or higher may use state-of-the-practice seismic design methods, such as performance-based seismic design, not covered under SNI 1726:2012, as long as the design follow internationally recognized guidelines. Typically, the verification of the performance objectives chosen for the project is carried out by developing a state-of-the-practice nonlinear structural model of the building and subjecting it to an ensemble of two-component horizontal ground motions. Using a conditional mean spectrum approach (NIST, 2011), ensembles of at least seven ground motion pairs are developed per hazard level for two different hazard levels: (i) Service-Level Earthquake (SLE) with a return period of 43 years (50% probability of exceedance in 30 years) and the (ii) Risk-targeted Maximum Considered Earthquake (MCE_R) with a return period of 2,745 years (2% probability of exceedance in 50 years). This contrasts with the common US practice of checking the building performance at SLE using a linear model, for which a number of assumptions are made for the effective member stiffnesses that account for cracking.

3 CASE STUDY: 55-STORY RESIDENTIAL TOWER IN JAKARTA

A 55-story residential tower located in Jakarta is the focus of the case study in this paper. The 234 m tall tower has 55 stories over five levels of parking below grade, see Figure 1 (a). This building is rather regular. The typical floor plan is 46.15 m by 41.20 m, see Figure 1 (b). The built area is approximately 1,510 m^2 and the interstory height is 4.2 m. A dual system provides lateral load resistance to the tower. The dual system is composed of an oblong reinforced concrete central core wall with the longitudinal axis predominantly oriented in the Y-direction, and a number of three-dimensional moment-frames, with oblong columns, see Figures 1 (b) and (c). Walls making up the core of the building are connected via three diagonally reinforced coupling beams in the longitudinal direction and, in the transverse direction, by one conventionally reinforced coupling beam and one diagonally reinforced coupling beam. All the diagonally reinforced coupling beams are 1.10 m deep. The thickness of the core walls at the ground level is 700 mm and decreases to 400 mm at the upper floors in order to maximize the usable areas. The dimensions of the rectangular columns vary from a maximum width of 1000 mm at the ground level to 500 mm at the upper floors. The dimensions of the main beams and secondary beams vary in height with a maximum height of 700 mm and 500 mm, respectively. The thicknesses of the one- and two-way slabs in this building are 120, 140, 150, and 160 mm. They are all supported on beams. Flat slabs are used at the parking basement area below the ground level. The tower mat is supported on a 3.0 m thick foundation slab in order to be able to distribute vertical loads from the columns and core to the bored piles. The drilled shaft pile foundation under the tower-mat is 1.2 m in diameter, with an effective length of 50 m to transfer the axial load of the tower to the hard silt and very dense sand. The bored piles develop their load carrying capacity through both skin friction along their perimeter and end bearing at their toe.



(a) 3-D view

(b) Typical floor-plan

(c) 3-D view of typical floor plan

Figure 1: The 3-D overview and Residential-Tower typical floor plan

1.1 Structural Material and Loading

Concrete strength: f_c'= 35, 45, and 55 MPa were specified for the structural walls and columns, while f_c'= 40 MPa was used for beams and slabs. Steel reinforcement had a specified yield strength f_y = 400 MPa. Live loads at the lobby ground floor and typical apartment areas were 5 kPa and 2 kPa, respectively. The load combinations were used for carrying out LFRD in accordance with SNI 1726:2012 and SNI 1727:2013, which are similar to the ASCE 7-10 requirements.

1.2 Structural System, Building Category and Seismic Design Parameters

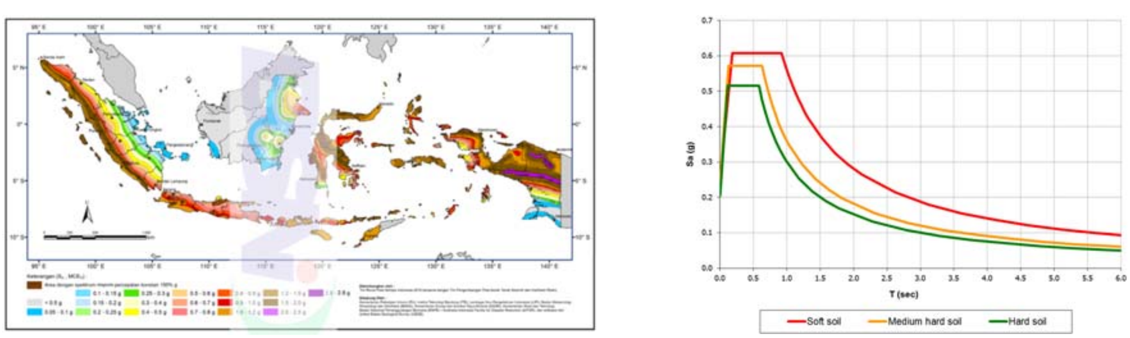
To meet the SNI 1726:2012 code prescriptive requirements, the parameters listed in Table 1 were calculated. The definition of these parameters is the same as those in ASCE 7-10 (2010).

Table 1: Seismic Design Parameters

Item	Description	Design Parameter	Reference
1	Risk Category	Residential = II	SNI 1726 ; Table 1
2	Importance Factor	I_e = 1.0	SNI 1726 ; Table 2
3	Site Class	Class D, Medium Soil	SNI 1726 ; Table 3
4	SDC Based on Short Period	D	SNI 1726 ; Table 6
5	SDC Based on 1s Period	D	SNI 1726 ; Table 7
6	DSSC at short period	S_{DS} = 0.572	
7	DSSC at 1s period	S_{D1} = 0.36	
8	Seismic Response Modification Factor	R = 7	SNI 1726 ; Table 9
9	Over strength Factor	Ω_0 = 2.5	SNI 1726 ; Table 9
10	Deflection Amplification Factor	C_d = 5.5	SNI 1726 ; Table 9

Note: SDC = Seismic Design Category
DSSC = Design Seismic Spectral Coefficient

The seismic spectral coefficient was taken from the MCE_R Indonesian seismic map at the location of the building as shown in Figure 2 (a). The design response spectrum for medium soil was used for this residential building as shown in Figure 2 (b).



(a) (b)

Figure 2: (a) MCE_R Indonesia seismic map for Ss - site class B and (b) design response spectrum at the Jakarta site

4 PERFORMANCE BASED SEISMIC DESIGN VERIFICATION

4.1. Performance Objectives

The intent of the study was to assess the adequacy of the code-compliant design for performance objectives corresponding to two hazard levels: service hazard level corresponding to a 50% probability of being exceeded in 30 years (i.e., return period of 43 years) and collapse prevention corresponding to the risk-targeted maximum considered earthquake with 2% of probability of being exceeded in 50 years (i.e., return period of 2475 years). According to the LATBSDC (2015), the objective of the verification of the building response at SLE is to ensure that the building structural and nonstructural components retain their general functionality during and after this frequent but low-intensity earthquake. Under such an earthquake scenario, damage must be minor. Repairs, if necessary, are expected to be performed without substantially affecting the normal use and functionality of the building. LATBSDC allows the analysis to be performed using either a three-dimensional linear or a nonlinear model and subject it to ground motions derived for the site and corresponding hazard level and for this building; the nonlinear analysis option was the preferred option. In Indonesia, it is very common for buildings to incorporate brick masonry partitions. ASCE 41-13 (2013) states that repairable damage to this type of partitions occurs at an interstory drift ratio ranging between 0.2% and 1%. The performance objective for this building at SLE was to keep the interstory drift ratios below 0.33% and, with such small ratios, control the damage to the brick masonry partition walls. For walls and coupling beams, the immediate occupancy acceptance criteria limits prescribed by ASCE 41 Table 10-19 and for columns, the immediate occupancy acceptance criteria limits prescribed by ASCE 41 Table 10-8 were adopted as structural response limits for the SLE.

LATBSDC (2015) recommends for the risk-targeted maximum considered earthquake, which has a probability of exceedance of 2% in 50 years or a return period of 2,475 years, that a building should respond well in its nonlinear range but have a low probability of collapse. The possibility of some structural as well as nonstructural damage is accepted. The mean of the maximum interstory drifts over all ground motions considered (Mean-Max) must be less than 3.0% at each level. Moreover, not a single ground motion shall result in an intrerstory drift ratio greater than 4.5%. For walls and coupling beams, the collapse prevention acceptance criteria limits prescribed by ASCE 41-13 Table 10-19 were adopted as damage limit states in this building. For columns, the collapse prevention acceptance criteria limits prescribed by ASCE 41 Table 10-8 were also adopted. In addition, LATBSDC recommends that the axial compression in a wall or column be kept less than $0.4A_g f'_{c;}$ this was also adopted as a performance limit.

4.2. Site-Specific Response Spectra and Input Ground Motions

Sengara et al. (2015) developed ensembles of horizontal two-component input ground motions to perform the nonlinear dynamic time-history analysis of the building. Seven SLE two-component records were used in the serviceability analysis. Eleven MCE_R ground motions pairs using the conditional mean spectrum method at target periods of 0.2, 1.0, 5.0 and 10 seconds were selected to perform the collapse prevention objective analysis. Some strong-motion database was used for spectral-matching of MCE_R ground motions and included the 1992 Landers earthquake, 1999 Chi-Chi earthquake, etc. Figure 3 shows the Site-Specific Response Spectra (SSRS) of the pseudo-acceleration and displacement response spectra of the ground motions, computed for 5% damping ratio.

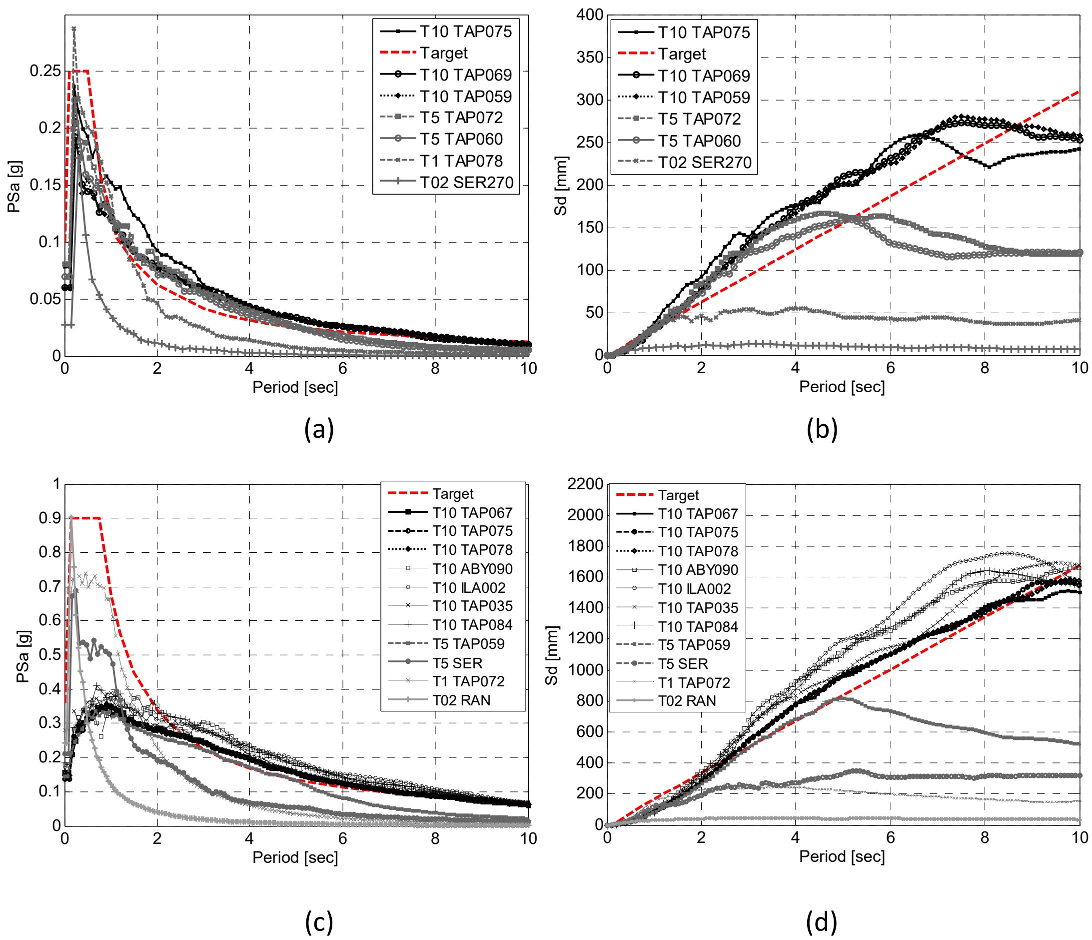


Figure 3: SSRS of the spectral ordinates: (a) Pseudo-acceleration for SLE; (b) Displacement for SLE; (c) Pseudo-acceleration for MCE$_R$; (d) Displacement for MCE$_R$

4.3. Model Description

To perform the performance-based analysis of the building for the targeted objectives, a detailed 3D nonlinear structural finite element model was developed in ETABS 2015, Version 15.2.2. The nonlinear modeling techniques used in developing the nonlinear model of the building were calibrated and validated using cyclic test data from experiments on reinforced concrete components and subassemblies (Cheung et al., 1989; Holden et al., 2003; Naish, 2010). These calibration and validation processes provided a reliable use of the nonlinear characteristics and capabilities of the finite elements used for the model.

The building core walls were modeled using two equal-length frame fiber hinges over the deformable part of the wall at each story. The length of the deformable part of the wall at every story was equal to the distance between the top of the bottom slab and the bottom of the top slab minus the height of the girder (if any) framing into the wall below the top slab. At any story, the portion of the wall intersecting with a girder framing into it was considered not deformable and modeled using a rigid end zone. Each of the two wall fiber hinges at every story was represented by a single integration point, one at the bottom and one at the top of the deformable part of the wall. In defining the wall fiber hinges, the cross-section of each flat wall was discretized into 8 steel fibers representing the longitudinal steel reinforcement and 2 rows of 8 concrete fibers representing the concrete for a total of 24 fibers per integration point. From the base of the building to the 41st floor, each column was modeled using the same approach as for the walls. Above the 41st floor, the columns were modeled as linear elastic Timoshenko beam-column elements to limit the computational cost of the FE model, since in the higher part of the building, the columns remain linear elastic under the seismic hazard levels (SLE and MCE_R) considered.

All beams defined to be part of the Lateral Force Resisting System (LFRS) were modeled as linear elastic Timoshenko beam-column elements with zero-length inelastic moment hinges at both ends of the deformable part of the girder. The inelastic moment hinges were defined with respect to bending of the girder in its vertical plane. The girder moment hinges are defined in terms of moment – rotation relations. All secondary beams supported on the beams described above were modeled as linear elastic Timoshenko beam-column elements. Diagonally reinforced coupling beams were modeled using a combination of two 3D truss elements in a diagonal configuration between rigid offsets, with the cross section of each truss element consisting of a steel fiber representing the diagonal reinforcing steel and a concrete fiber representing the diagonal concrete strut. These concrete and steel fibers were assigned with uniaxial nonlinear hysteretic material constitutive models representing the uniaxial stress-strain behavior of the diagonal reinforcing steel and diagonal concrete strut.

The analyses were conducted based on expected material properties. For concrete fibers, in wall and column fiber hinges, the uniaxial stress-strain behavior of concrete was modeled using the general pivot hysteretic model of Dowell et al. (1998) available in ETABS 2015. For steel fibers in wall and column fiber hinges, the uniaxial stress-strain behavior of the reinforcing steel is modeled using the general pivot hysteretic model. Pivot hysteretic model parameters for steel and concrete were calibrated and validated using experimental data.

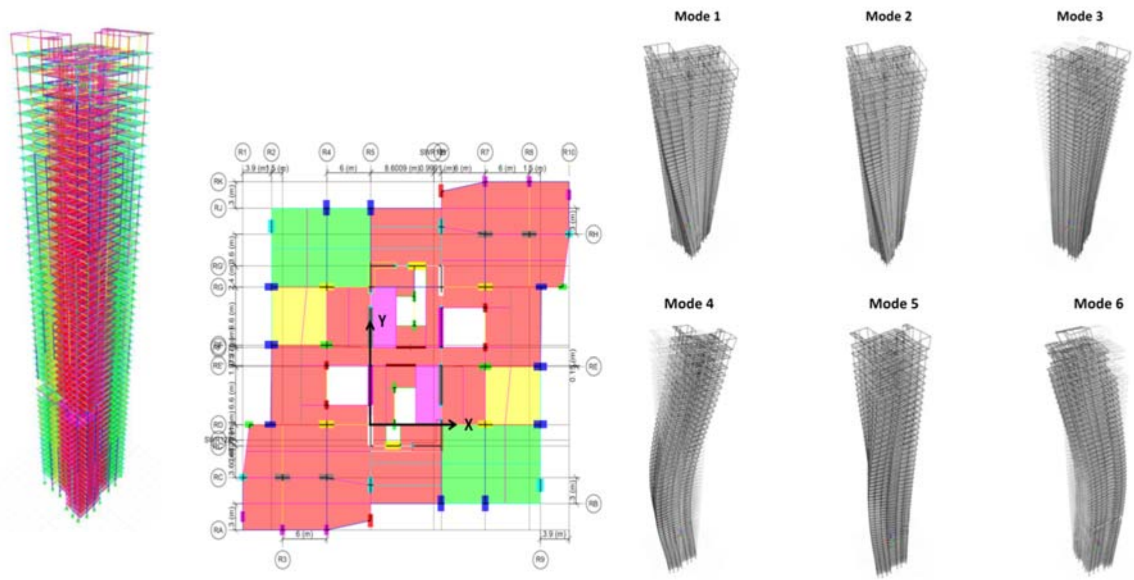
Table 2: Modal Properties

Mode	Undamped Natural Period [sec]	Mode shape	Modal participating mass ratio in X-dir	Cumulative modal participating mass ratios in X-dir	Modal participating mass ratio in Y-dir	Cumulative modal participating mass ratios in Y-dir
Mode 1	10.33	First mode in X-dir.	0.66	0.66	0.01	0.01
Mode 2	7.37	First torsional mode	0.00	0.66	0.02	0.03
Mode 3	7.31	First mode in Y-dir.	0.01	0.67	0.61	0.64
Mode 4	3.21	Second mode in X-dir.	0.15	0.82	0.00	0.64
Mode 5	2.70	Second torsional mode	0.00	0.82	0.00	0.64
Mode 6	2.09	Second mode in Y-dir.	0.00	0.82	0.19	0.83
Mode 7	1.70	Third mode in X-dir.	0.06	0.88	0.00	0.83
Mode 8	1.59	Third torsional mode	0.00	0.88	0.00	0.83
Mode 9	1.14	Fourth mode in X-dir.	0.02	0.90	0.00	0.83
Mode 10	1.13	Fourth torsional mode	0.01	0.91	0.00	0.83
Mode 11	1.06	Third mode in Y-dir.	0.00	0.91	0.06	0.89

For the linear elastic part of the models of the shear walls (shear and torsional behavior), columns, girders, beams and coupling beams (conventionally reinforced), the section stiffness modifiers set forth in LATBSDC were applied for the assessed performance objectives. These section stiffness modifiers accounted for the cracking of concrete in the various structural elements.

Before carrying the nonlinear time-history analysis, a modal analysis of the building was carried out for quality assurance purposes and to determine the periods at which to assign the modal damping ratios. The mass sources considered for the modal and nonlinear time-history analyses are: self-weight (100%), superimposed dead load (100%) and live load (25%). Table 2 summarizes the results of the modal analysis. A modal damping ratio of 2% percent was assigned to the first mode of the building in the X-direction (with period T_1) and to a period equal to $0.2T_1$.

Figure 4 shows the FE model of the tower and graphical representation of first 6 vibration modes.



(a) (b) (c)

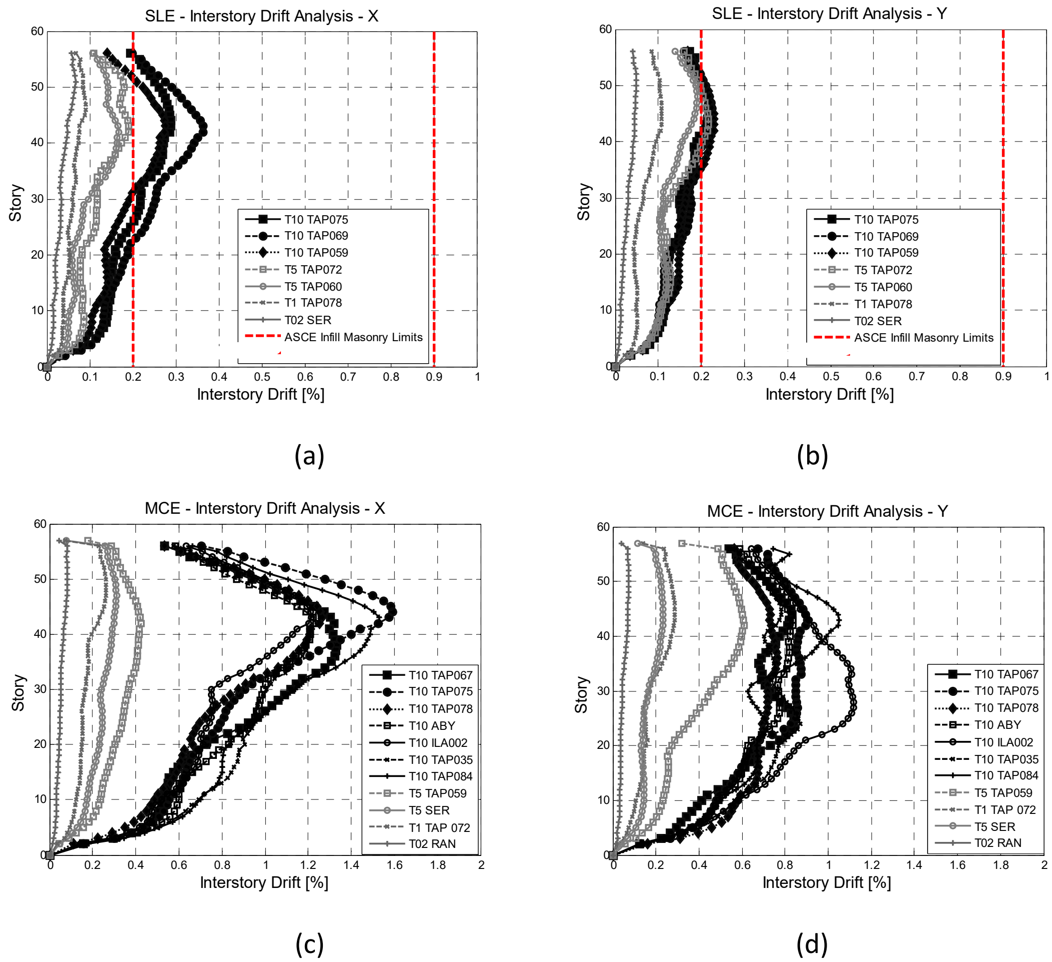
Figure 4: FE model of the Tower: (a) 3D-view; (b) Plan view of Level 07; (c) Graphical representation of first 6 modes of vibration

The finite element model of this 55 story building had 11,602 nodes, 31,480 frame elements, 4,615 membrane elements and 39,666 degrees of freedom.

4.4. Key Results

The drifts, beam and column plastic hinge rotations and coupling beam chord rotations recorded in each of the structural elements of the building responding to each of the SE and MCE_R ground motion pairs were read and analyzed statistically using an in-house written software. Figure 5 shows the interstory drift ratios for SLE and MCE_R ground motions. The T10 ground motions, which had the largest spectral displacements of the ensemble at around 10 s, induced the highest interstory drifts in the tower. Interstory drift ratios were larger in the X-direction than in the Y-direction and are generally lower than 1.6%. The response of the building to the SLE was found to be within the lower and upper bounds criteria limits set forth by ASCE 41-13 for infill masonry, given the type of partitions used in the building. For the MCE_R ground motions, the Mean-Max interstory drift ratios in the X- and Y directions are well-below the acceptance criterion limit of 3.0% over the entire building elevation. The maximum drift usage ratio is 1.34%/3% = 0.45 and is observed in the X-direction at level 43.

The Mean-Max shear distortion of the diagonally reinforced coupling beams over the entire building elevation is plotted in Figure 6 for seven T10 MCE_R ground motions. In all cases the shear distortion is low and well-below the collapse prevention accepted criterion limit of 0.05 radians. The maximum Mean-Max shear distortion usage ratio of 0.014/0.05 = 0.28 is observed in beam CB-2 at level 24.



(a) (b) (c) (d)

Figure 5: Drift ratios computed for: (a) SLE X-direction; (b) SLE Y-direction; (c) MCE X-direction; (d) MCE Y-direction

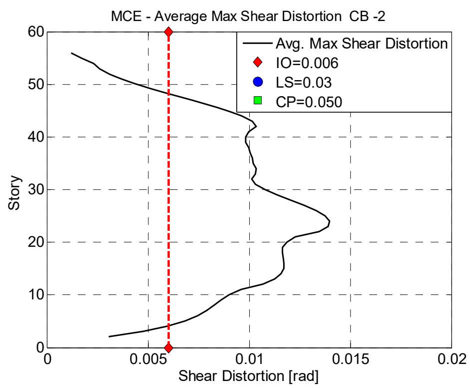


Figure 6: Shear distortion along building height for diagonally reinforced coupling beams. Mean-Max for the seven T10 MCE_R ground motions.

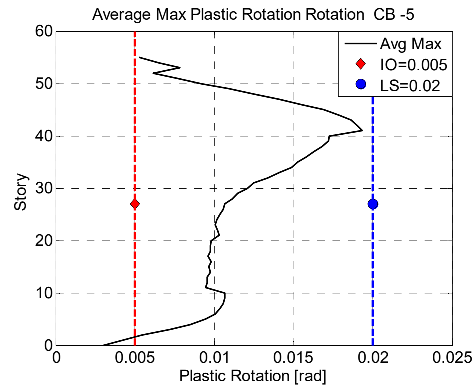


Figure 7: Beam end-rotation along building height for conventionally reinforced coupling beams. Mean-Max for the seven T10 MCE_R ground motions.

For conventionally reinforced beams, it was found that the most demanded are those coupled to the shear walls within the core of the building. Figure 7 plots the end beam rotation concentrated at the beam ends due to the development of plastic hinges. Such rotation differs from that calculated in the diagonally reinforced beams that is smeared over the entire beam span.

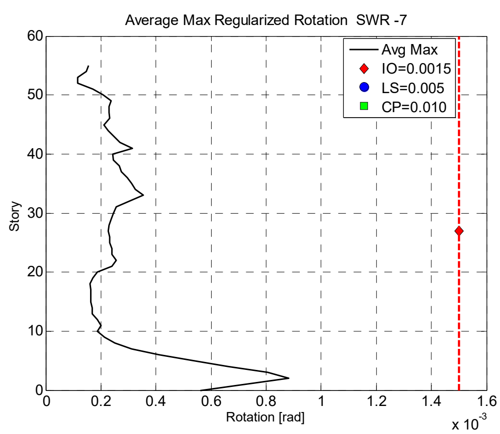


Figure 8: SWR-7 wall rotation demand. Mean-Max for the seven T10 MCE_R ground motions.

Chord rotations in conventional coupling beams tend to be greater than in the diagonally reinforced coupling beams. For conventional reinforced beams, the maximum Mean-Max rotation demand is 0.019 radians and occurs at level 41. The chord rotation usage ratio for this beam is still low at 0.019/0.04 = 0.48. Walls remain elastic at the MCE_R. The Mean-Max wall rotation demands are well within the acceptable criterion limit at collapse prevention of 0.01 radians. The maximum Mean-Max wall rotation usage ratio is observed at level 2 in wall SWR-7 and is 0.9×10^{-3} rad, see Figure 8.

5 DISCUSSION

There are several specific points which need to be observed in carrying out the Performance Based Design for Reinforced Concrete Buildings in Indonesia in order to proof that the building seismic performance will comply to the PEER Report No. 2010/5 and LATBSDC-2015:

- In the absence of real local ground motion records, the selection, scaling, and simulation of input ground motions must be carried out by geotechnical engineering experts familiar with probabilistic seismic hazard analysis. This is related to the selection of seven pairs of real earthquakes, which are going to be adopted from overseas ground motions and are suitable with the Indonesia seismological conditions.

- The adopted values of building material properties used for structural analysis purposes must be based on the local practice and test results from several real projects.
- There must be a calibration and validation of the finite element modeling techniques to be used in the ETABS computer program. Due to the limited local laboratory tests, available test results from foreign countries, which show some similarity with local practices can be used.
- The limitation of non-structural element performance shall be analyzed, especially for the local common practice of concrete frame infill walls made of clay-bricks or concrete aerated blocks.
- Reviews by peers who are experts in this field are needed to conclude that the building structural behavior meets international standards.

6 CONCLUSION

The response of a 55-story model building to the SLE was found to be within the lower and upper bounds criteria limits set forth by ASCE 41-13 for infill masonry, given the type of partitions to be used in the building. For the MCE_R ground motions, the mean-max interstory drift ratios in the X- and Y-directions were well-below the acceptance criterion limit of 3.0% over the entire building elevation. It should be expected that the non-linear analyses carried out for Performance Based Design, which take into account the local practice data and validated or calibrated hysteretic models, provide more accurate prediction of the real building performance.

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