16th World Conference on Earthquake, 16WCEE 2017
Santiago Chile, January 9th to 13th 2017
Paper N° 421
Registration Code: S-D1463138443

BEST PRACTICES IN SEISMIC DESIGN OF TALL TO MEGA TALL BUILDING STRUCTURES IN INDONESIA

Sugeng Wijanto(1), I Gde Widiadnyana Merati(2), I Wayan Sengara(3) and Takim Andriono(4)

(1) *Principal Director of PT. Gistama Intisemesta, Structural Consulting Firm – Jakarta, Indonesia*
Senior Lecturer Trisakti University – Jakarta and Editorial Board Member of WHE – EERI
s.wijanto1@gistama.com ; s.wijanto1@xtra.co.nz

(2) *Professor at Institute of Technology Bandung – Indonesia*
Chairman of Jakarta Building Authority Committee - Jakarta, Indonesia
widimerati@itb.ac.id

(3) *Associate Professor at Institute of Technology Bandung – Indonesia*
Member of Jakarta Building Authority Committee - Jakarta, Indonesia
wayansengara@yahoo.com

(4) *Director of PT. Gistama Intisemesta, Structural Consulting Firm – Jakarta, Indonesia*
andriono2509@gmail.com

Abstract

Currently more than 80 million people of Indonesia or almost one third of its total population lives in the major cities. This spurs the rapid development of high-rise buildings in its large cities, especially Jakarta. Along with this rapid development, the codes for structural design practices have experienced major changes, especially those which are related to earthquakes which normally governs the structural design of tall to mega tall buildings, since most of Indonesia regions are seismic prone.

This paper discusses the development of Indonesia building design standards and codes. Special highlights are given for the most current seismic standards and codes. Their implementation on advanced structural analyses and design methods for designing the foundation and upper-structure of mega-tall buildings such as the proposed Signature Tower in Jakarta is presented. This 111 floor tower with 7 basement layers could be the tallest building which is located in one of the most active seismic zone in the world. Since the building also stands on very deep soft soil layers of alluvial deposit, it has a complex seismic behavior. In addition the paper highlights how the Performance-Based Design (PBD) considering the Risk-targeted Maximum Considered Earthquake, (MCE_R) levels of seismic hazard was carried out.

Keywords: seismic zone; MCE_R level; mega-tall building; performance based design

1. Introduction

The number of high-rise buildings in Indonesia, especially in Jakarta, the capital city, has been growing significantly in this last two decades. An advisory committee, which consists of a group of senior structural engineers, has been set up since the year of 1974 by the Governor of Jakarta to assist the Building Authority in reviewing the submitted structural designs, as one of building design enforcement process. The recommendation given by this advisory committee is a pre-requisite in obtaining the building permit. The Jakarta Building Authority Committee (JBAC) shall enforce the adoption of latest building regulations in Indonesia to ensure the building has met the safety criteria. Structural engineers, who adopt the cutting edge technology in their designs, must also consider the latest international recognized standards of design practice and codes. The advisory committee might also recommend a series of experiments to be carried out to predict the structural behavior and performance of the particular building structure under severe load conditions. This paper discusses the current Indonesian Codes and Design Standards as well as the application of advance structural analyses and design methods on mega tall buildings such as the Signature Tower, one of the world's tallest proposed buildings. Located in Jakarta, one of the most active seismic regions, this 111-story tower with 7-level basement is designed standing on soft soil layers of alluvial deposit.

2. Current Indonesian Codes and Design Standards

2.1 Gravity loading

The currently implemented gravity load standard in Indonesia is SNI 1727:2013 [19]. This standard replaced the previous SNI 03-1727-1989, which was used for more than two decades. The new standard was adopted from ASCE 7-10 [4] with some local modifications excluding the seismic standards. Types of dead loads and live loads are much more completely listed in this current standard compared to the similar lists found in the previous one. Various reduction factors of live loads are applied depending on the tributary area.

2.2 Seismic loading

Most major cities in Indonesia are located in seismic prone areas. The country implemented its first Seismic Code in 1970 (Indonesian Loading Guidelines – N.I. 18). Based on this code, static analyses may be applied for buildings less than 40 meters. The seismic inertia forces are determined by multiplying the structural mass at particular floor levels to the earthquake acceleration determined by the code. Indonesia was divided into 3 seismic zones with acceleration standards of 25, 50 and 100 gal. Dynamic analyses shall be carried out for buildings taller than 40 m, and done by structural engineering specialists. No further guideline regarding the dynamic analysis was available. The code itself was very brief.

In 1976, the New Zealand Public Works and the Indonesian counterparts were commissioned to undertake a national earthquake engineering study in Indonesia in order to prepare Earthquake Resistance Design Guidelines for Houses and Buildings. The first and second drafts were issued in 1981 and 1983 respectively. The official design guidelines were finally published in 1987. The Indonesian region was divided into 6 seismic zones (Zone 1 is regions with the highest seismic risk while zone 6 is regions with almost zero risk). Two types of soil condition, i.e. soft soil and hard soil were taken into considerations. Three types of analysis were introduced, i.e. static load equivalent analysis, spectral modal analysis and numerical integration response analysis. Static analysis was applicable for buildings less than 40 meters. The total base shear forces were determined by multiplying the total building mass to the seismic coefficient determined from the code, the importance factor and the structural type factor. The total base shear was then distributed into each level in accordance to the displacement shape of the building.

The 1987 Earthquake Resistant Design Guideline was then replaced by The 2002 Earthquake Resistant Design Standard (SNI 1726:2002). The Indonesian region was divided into 6 seismic zones. Based on this standard, zone 1 represents regions with the lowest seismic risks while zone 6 represents the highest. Three types of soil condition, i.e. soft soil, medium soil and hard soil, were taken into account.

Recently, after several major earthquakes struck Indonesia (such as Aceh 2004, Nias 2005, Yogyakarta 2006, Padang 2009 earthquakes), a new committee was appointed and the current earthquake resistant design standard referring to ASCE 7-10 and IBC-2009 was published in Year 2012, i.e. SNI 1726:2012 [18]. Due to these recent large earthquakes in Indonesia, stricter code-based seismic design criteria with options of conducting a more detailed seismic hazard analysis shall be applied for tall buildings.

These design criteria are based on a new concept that considers both seismic hazard of the site and integrating it with fragility of the building to derive the so-called Risk-targeted Maximum Considered Earthquake (MCE_R). This MCE_R is defined as a 1% probability of the building collapse in 50 years. The MCE_R is developed by calculating the risk-integral, consisting of hazard curves of Maximum Considered Earthquake (MCE). It is defined as the 2% probability of exceedance (PE) in 50 years resulting from a probabilistic seismic hazard analysis (PSHA) of the site and fragility function of the building. For tall building seismic design, seismic criteria are also made for Service Level Earthquake (SLE), defined as 50% PE in 30 years. The seismic design criteria are also selected in conformance with the PEER-TBI Guidelines 2010 [9], LATBSDC 2014 [8] and ASCE 41-13 [3].

2.3 Wind loading

When compared to the previous standard, SNI 03-1727-1989, major changes was found in the wind loading chapter as stated in the current standard, SNI 1727:2013. There are two options in applying wind loading, i.e. based on the prescriptive current code or conducting a series of wind tunnel tests. For buildings of more than 50 floors (200 m height), the JBAC requires a series of wind tunnel tests to be carried out. A minimum 1:500 scale model, including its surroundings within a 600 m diameter should be adopted and measured using a very sensitive High-Frequency Force Balance (HFFB) test. The wind climate model needs to be scaled so that the magnitude of the wind velocity for the 100-year return period corresponding to a mean hourly wind speed of 40 m/s at the gradient height in an open terrain. The summary of predicted peak overall structural wind loads from the wind tunnel test should be presented in the report. A return period of 700 years shall be applied for risk category II structures and a return period of 1700 years for risk category III-IV structures

The predicted peak accelerations for the 1-year return period and the 10-year accelerations with 1% critical damping ratio are preferable to check the acceptable for human comfort in high-rise buildings. These values should below the 1-year return period of the International Organization for Standardization (ISO) limit value and the 10-year acceleration limit under the Rowan Williams Davies & Irwin Inc (RWDI) criteria for specific building.

2.4 Loading combination

Several condition of loading combination should be checked such as ultimate limit state and serviceability limit state. The loading combinations used for carrying out design strength and allowable strength in accordance with SNI 1726:2012 and SNI 1727:2013 are as follows:

Design strength:

1. $1.4\,D$
2. $1.2\,D + 1.6\,L + 0.5\,(L_r \text{ or } R)$
3. $1.2\,D + 1.6\,(L_r \text{ or } R) + (L \text{ or } 0.5\,W)$
4. $1.2\,D + 1.0\,W + L + 0.5\,(L_r \text{ or } R)$
5. $1.2\,D + 1.0\,E + L$
6. $0.9\,D + 1.0\,W$
7. $0.9\,D + 1.0\,E$

Allowable strength:

1. D
2. $D + L$
3. $D + (L_r \text{ or } R)$
4. $D + 0.75\,L + 0.75\,(L_r \text{ or } R)$
5. $D + (0.6\,W \text{ or } 0.7\,E)$

6a. $D + 0.75\,L + 0.75(0.6\,W) + 0.75\,(L_r \text{ or } R)$

6b. $D + 0.75\,L + 0.75(0.7\,E)$

7. $0.6\,D + 0.6\,W$
8. $0.6\,D + 0.7\,E$

Note: S (snow load) was not included since most of Indonesia regions have never experienced this load.

Some additional considerations are added such water levels in normal condition and flooding conditions. Flooding data of certain locations can be checked from the relevant building authorities.

3. Seismic Design Spectra Criteria for Tall and Mega Tall Buildings

Seismic design criteria need to be formulated for tall and mega tall buildings in compliance to current Indonesian seismic building code (SNI 1726:2012). These criteria can be simply derived from the code by identifying the site class and determining the maximum (S_M) and design (S_D) spectra. For code-based spectra, the maximum ground surface spectra shall be based on mapped S_s (short period) and S_1 (1-second period) base-rock of the site. After identifying the site-classification, then the amplification factors F_a and F_v need to be selected from the code to amplify S_B motions to ground-surface in order to define the ground-surface maximum spectra S_M and design spectra S_D=2/3S_M.

For tall and mega tall buildings, a site-specific seismic hazard analysis needs to be conducted. The analysis consists of PSHA and site-specific response analysis (SSRA) to recommend ground surface maximum risk-targeted and design response spectra. For tall building structures that require Performance Based Design (PBD) analysis, then seven pairs of seismic input ground motions need to be developed considering the PSHA and SSRA. Another essential aspect of the seismic design is seismic load to basement wall and to the foundations. The following sections describe in more specific the seismic design criteria analysis.

3.1 Probabilistic Seismic Hazard Analyses and Site Specific Response Analyses

Current practices of PSHA and SSRA in Indonesia, for Site Class F and mega tall buildings, such as the proposed Signature Tower, are conducted to meet all the requirements of SNI 1726:2012 in which most of the requirements are referred from ASCE 7-10. In addition, PEER-TBI Guidelines 2010 and LATBSDC 2014 are also adopted. The essential aspects of the PSHA are seismic source zoning and identification of potential seismic sources within 500km radius of the site such as from subduction and shallow crustal faults, as shown in Fig.1. Ground motions predictive equation (GMPE) should be adopted appropriately considering the types of seismic sources. S_B MCE uniform hazard spectra (UHS) of 2% PE in 50 years need to be computed and MCE_R need to be derived through integration with fragility curve adopting log-normal standard deviation (β) of 0.65 as required by the codes.

For SSRA, two main components of the analysis need to be available, that is S_B motions and shear wave velocity (V_s) profile. The S_B motions should be derived from de-aggregation analysis as part of the PSHA, to identify controlling earthquake magnitude and its distance. The de-aggregation analysis needs to be conducted for several oscillator periods corresponding to periods of interest of the structure. Seven seismic motions scaled to UHS at various periods of interest need to be developed. Shear wave velocity profile is commonly obtained from seismic downhole test. For greater depth to identify base rock (SB) level, micro-tremor survey could be done. Fig.2 shows an example of V_s profile and recommended ground surface S_M spectra of a project in Jakarta. More detail procedures on PSHA and SSRA, for a site in Jakarta is elaborated in Sengara et al. 2012 [13].

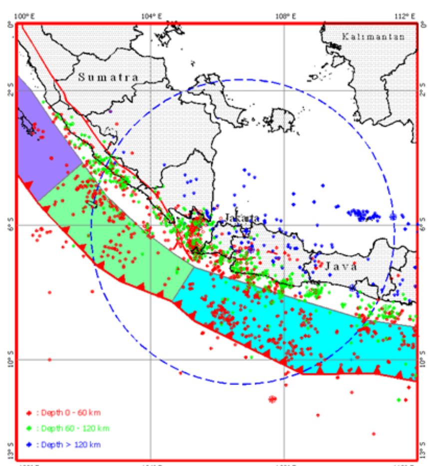


Fig. 1 - Seismic source zoning of PSHA for Jakarta

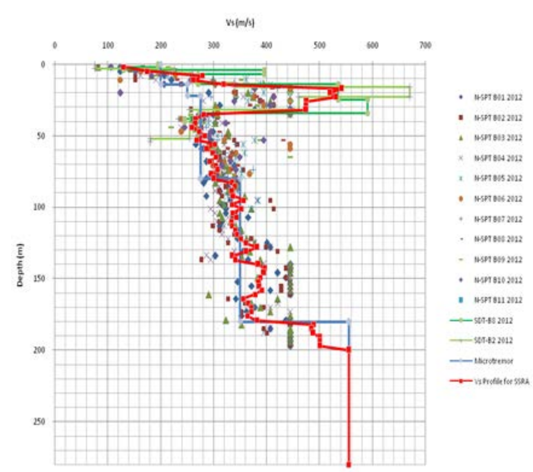


Fig. 2 - Shear wave velocity profile of a site in Jakarta

3.2 Seismic Input Ground Motions

Non-linear time-history structural analysis requires a set of seismic time-history input-motions at reference ground-surface or base of the structure. The time-history input motions should be developed in correspondence to the site and seismic source characteristics. Therefore, the development of time-history ground motions should be integrated from PSHA and SSRA. For this purpose, the current practice in Indonesia adopts ASCE 7-10, PEER-TBI Guidelines, 2010 and LATBSDC 2014.

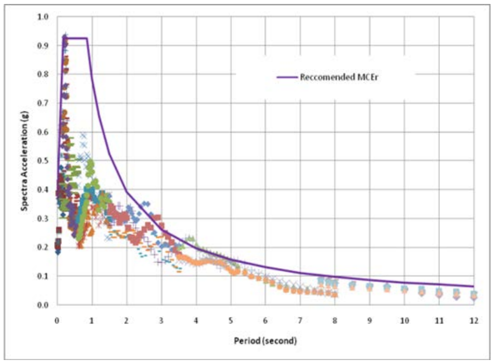


Fig. 3 - Recommended site-specific ground-surface MCE_R spectra of a site in Jakarta

Seven pairs of ground motions scaled to recommend ground surface spectra at various periods of interest are generated adopting spectral-matching techniques such as that proposed by Abrahamson, 1992 [1] and that built-in EZ-FRISK computer program (Risk Engineering, 2011 [11]). Each generated pair of ground motions should correspond to controlling magnitude and distance earthquakes to be scaled at period of interest. The generated ground motions also should be scaled at various periods of interest, as also follow the procedures of base S_B motions. Sengara et al. 2015 [16] presents in more detail an example of seismic time-history development for a site in Jakarta. Fig.3 show the recommended site-specific ground surface MCE_R spectra of a site in Jakarta.

3.3 Seismic Loads on Basement and Foundation

Seismic load to basement wall should correspond to the specified S_B mapped Maximum Considered Earthquake Geometric Mean (MCE_G) specified in the codes with seismic amplification F_{PGA} correspond to the site class. MCE_G corresponds to 2% PE in 50 years. Several methods of seismic pressure distribution on basement wall are adopted such as Seed-Whitman (1970) and Wood (1973), and Sitar et al., 2012 [17]. Recently, a method proposed by Izumi and Miura, 2004 [7] for seismic load to the foundation is also considered.

4. Structural Analysis and Design with the Signature Tower as a Case Study

The JBAC has issued a consensus in carrying out the seismic force analyses for towers with basement. A two-stage analysis procedure needs to be carried out in accordance with SNI 1726:2012 Chapter 7.2.3.2, as follows:

1. Create an upper structure model with base fixity at the ground level or floor level near the ground surface (Model-1 as shown in Fig.4). A 3-D dynamic analysis is conducted in accordance with the structural system, seismic design category and other parameters. The first- and second-mode should be dominated in translation. A seismic coefficient value was determined based on the biggest value of the ETABS's building period, approximate value of building period and C_s minimum. Base shear value at least 85% static equivalent base shear.

2. Create another upper structure model which includes the whole basement (Model-2 as shown in Fig.4) and run 3-D dynamic analyses using this model. Check the story shear value at the ground level and it should be at least 85% of the static equivalent base shear from step 1. Multiply the seismic inertia force, applied to the basement, by 1.5 to ensure the basement structure elements capacity such as beam, column, shear-wall, basement-wall and pile-cap or mat foundation are stronger than the upper structure elements. Apply the inertia and kinematic forces at the basement slurry walls, by taking into account the area spring constant provided by the geotechnical engineer.

3. Use Model-3 as shown in Fig. 4 to apply the seismic forces from the upper part of the structure, as well as all inertia and kinematic forces at basement levels to the foundation. However, since the foundation must be designed using the Allowable Stress Design (ASD) method based on service load combinations while the upper structure is designed based on factored loads using the Load and Resistance Factor Design (LFRD)

method, then all seismic forces from the upper structure to be applied on the foundation shall be multiplied by Ω_0 (over strength system factor).

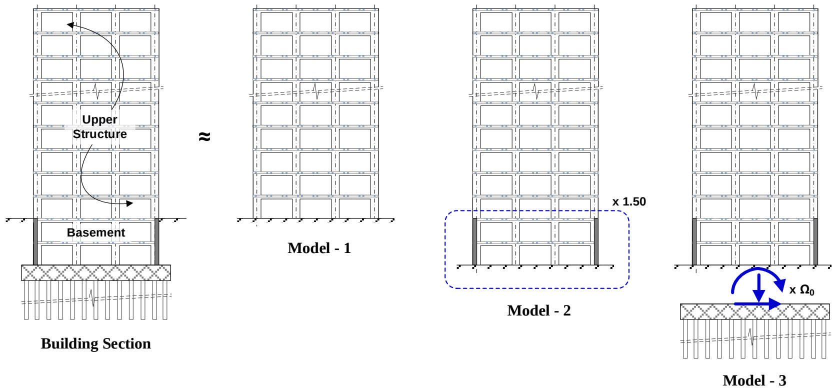


Fig. 4 – Structural Analysis for Tower with basement (in accordance with JBAC)

With the growing up of Indonesia economy recently, tall buildings with sophisticated architect designs become so popular among the building developers. PBD analysis becomes preferable among the professional structural consultants to achieve economical design and fulfill the building performance as the client desires. To demonstrate acceptable behavior of the lateral system, at least seven time history records scaled to site specific 2475 year event to suit 5% damped response spectrum values are used in carrying out nonlinear response history analyses. The PBD Concept offers a new approach to the design of seismic structures, namely by setting multiple objective performance levels for the building structure which is expected to be achieved when the building is hit by an earthquake with a certain intensity. SEAOC Vision 2000 [12] for example, proposes four levels of design earthquake (frequent, occasional, rare, and very rare) with estimated return periods and probability of occurrence during the effective age of the building as presented in Table 1. Furthermore SEAOC Vision 2000 recommends four levels of structural performance (fully operational, operational, life safe, near collapse) for different levels of earthquake intensity plan as presented in Fig.5.

Table 1 - Earthquake design level for both design and verification [12]

Earthquake Design Level	Mean Return Period	Probability of Exceedance
Frequent	43 years	50 % in 30 years
Occasional	72 years	50 % in 50 years
Rare	474 years	10% in 50 years
Very Rare	2475 years	2 % in 50 years

Note: These mean return periods are typically rounded to 50, 75, 500 and 2.500 years, respectively

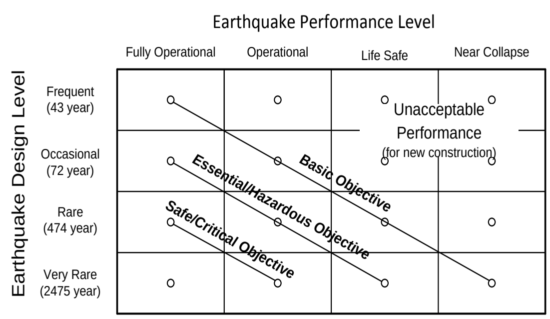


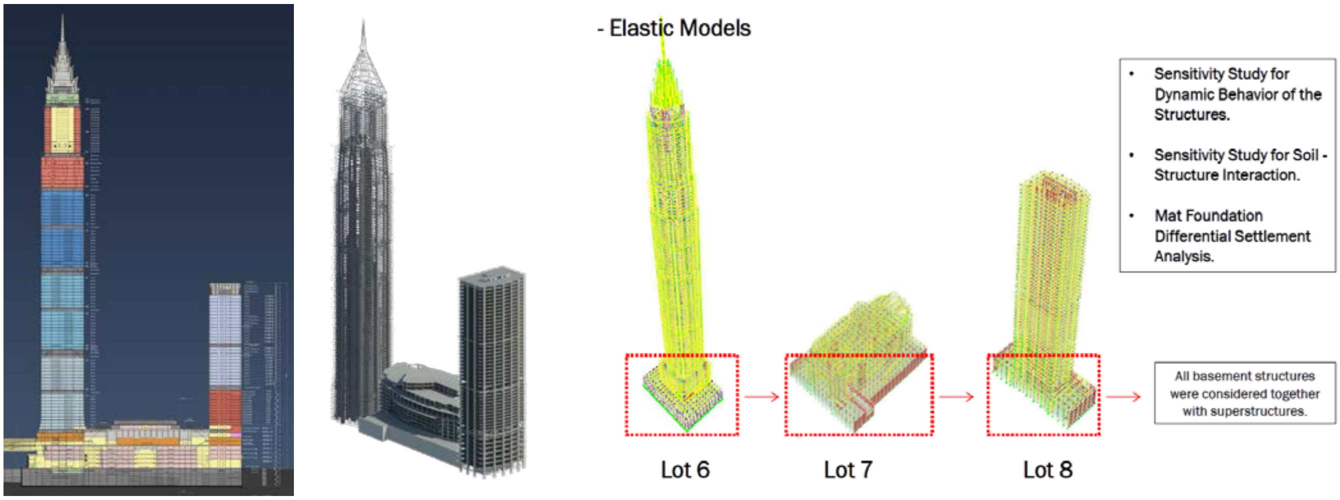
Fig. 5 - Recommended Earthquake Performance Objectives for Buildings [12]

JBAC requires more stringent performance goal for tall-mega tall buildings to be set to meet life safety (LS) limits for MCE level EQ and immediate occupancy (IO) limits for design level EQ (2/3 MCE) per ASCE 41-13 which is equivalent to Risk Category IV requirements as stated in ASCE 7-10.

4.1 Signature Tower: A Case Study

The 111-story Signature Tower is located in Jakarta, Indonesia one of the most active seismic zones in the world; a prudent selection of seismic resisting system is very crucial. Classified as Seismic Design Category D, the project presents great challenges to engineers because of its high seismic risk and extremely soft soil conditions. The capital city Jakarta is located in the Jakarta basin. It is known that this basin is mostly dominated by alluvial deposits which consist of relatively compressible clay, sand and gravels associated with young, highly weathered volcanic rock of tuffacceous clay and sand.

As shown in Fig.6, the 111-story Signature Tower is connected to the 13-story retail podium and also the 52-story office tower at the other edge. The project has a total floor area of approximately 778,000 m^2, including 7 floors of basement.



(a) Building section (b) 3-D Model [21] (c) ETABS Elastic Models [21]

Fig. 6 - Building section, 3-D model and elastic models of Signature Tower

4.1.1 Foundation and Basement Wall

Drilled shaft bored piles with a mat foundation system was planned to be used for this project to support the weight of the building and resist overturning moments from wind and seismic loads. The tower mat is relatively thick 9.0 m to distribute vertical loads from columns and core to the bored piles. The drilled shaft pile foundation under the tower-mat is 1.5 m in diameter, with an effective length of 90 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 the perimeter and end bearing of the toe. The piles, with 2.5-D c.t.c. spacing, cover the 90x100m-octagonal mat to resist the axial load of the tower's weight. Slurry wall construction of 1.2 m thick is used for the entire perimeter basement wall. The working condition for the slurry wall is separated in two stages: during the construction stage to retain the soil and top-down construction implications and at the service stage to resist the soil at-rest pressure, the groundwater pressure and dynamic seismic lateral earth pressure.

4.1.2 Upper Structure

The tower utilizes a lateral load resisting called as the “Core-Outriggers-Mega Frame” system. It includes a hybrid concrete core, outrigger trusses, hybrid super columns and an exterior mega frame, which consists of super columns and belt trusses. The hybrid concrete core is linked by steel outrigger trusses at three levels to the eight super composite perimeter columns. These structural components are intended to be the primary lateral load resisting system of the tower. The secondary system consists of a mega frame with its super columns and belt trusses, which are placed at six levels (see Fig.7). The reason for selecting the above lateral load resisting system can be described as follows. The square concrete core with 31m wide faces efficiently encloses numerous elevators and stairs needed to service tower occupants, but it is not sufficient by itself to resist extreme overturning moments generated by lateral wind and seismic loads, as well as to control deflections and drifts to the required comfort level. The most economically feasible approach to resist overturning and improve stiffness is to engage outer columns to benefit from a longer moment arm. Hence, three sets of two-story outrigger trusses aligned with flange core walls are used to tie the core and exterior super columns together.

The two-story outrigger trusses are located between levels 33~35, 58~60, and 91~93. A one-story head truss is located at levels 109~110 to control the drift at the tower's top. In addition, the tower is wrapped by a mega frame consisting of nine belt trusses linking the super columns together and transferring secondary column gravity loads to super columns. The one-story belt trusses are located between levels 10~11, 22~23, 47~48, 72~73, 83~84 and 109~110. Two-story belt trusses are located between levels 33~35, 58~60, and 91~93.

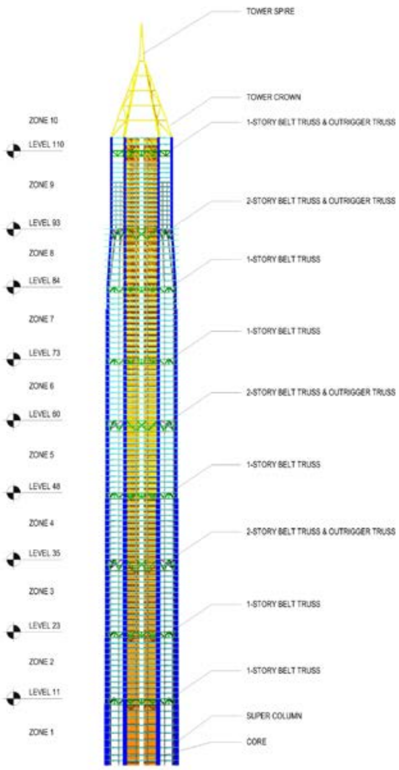


Fig. 7 - Tower elevation view showing outriggers and belt trusses [22]

The thickness of the core walls at the ground level is 1.1 m and decreases to 0.6 m at the upper floors in order to maximum the usable areas. The dimensions of the super columns are 3.5 m x 5.0 m at the ground level and decrease to 1.5 m x 1.5 m at the upper floors. These columns are straight at the lower floors and slope gradually at the upper floors to fit the tower's profile.

As part of the gravity system, the floor consists of composite floor decks with steel beams and girders. The tower's gravity load on every floor is supported by the core, super columns and much smaller gravity columns. The gravity forces in the gravity columns are collected by belt trusses and transferred to the super columns. This load path not only reduces the accumulated gravity forces taken by the gravity columns, but also helps to reduce tensile forces in the super columns due to lateral loads. At the top of each zone, gravity columns are connected to the bottom of belt trusses with vertically slotted connections to release potential stresses due to creep, shrinkage and differential movements.

The PBD incorporates a series of time history analyses with seven sets of acceleration records in order to evaluate the building's performance under different levels of seismic hazard, in addition to the JBAC requirement. By carrying out this design method it is hoped that an efficient and safe structure can be achieved.

The retail podium structure was designed to resist the lateral forces by special structural wall and special concrete moment frames. The podium’s gravity framing system is mainly a one-way concrete slab system with concrete girders and filler beams on typical bays. Concrete slab thickness varies from 125 mm to 180 mm, depending upon the floor's usage. Thus all structural wall and moment frames on this structure have been designed to satisfy special moment frame requirement stated in SNI 2847:2013 [20]. A steel truss system was selected for supporting the roof structure, which is accessible and public space with a garden. Steel trusses on the roof make a spacious area of 56 m x 88 m, without any columns at the floor. The steel trusses are combined with steel sections embedded in concrete columns and some horizontal bracing on the roof level. These help diaphragm the forces spreading out to other frames.

The 52-story office building was designed to resist the lateral forces by mainly employing reinforced concrete dual-system. A core wall with special moment resisting frame was selected as a lateral resisting system above the ground floor. Thus all moment frames on this structure have been designed to satisfy dual systems requirement stated in SNI 2847:2013.

The 111-story Tower, the retail podium and the 52 story office building have different structural behaviors, so expansion joints are an effective way to separate the structures to avoid the structural interference between dissimilar structures. Expansion joints, double columns, were placed between the buildings at above ground level.

4.2 Performance Based Design Analysis

Dynamic behavior of tall buildings is quite different from the assumptions made in the development of code provisions and requires alternative procedures such as PBD analysis for appropriate modeling, analysis, design and acceptance criteria. Perform-3D program was used for PBD-analysis of Signature Tower to fulfil the JBAC consensus. General geometric and numerical modeling considerations for the non-linear Perform-3D model were defined in the accordance with the ASCE 41-13. The design model of the standalone tower and rigid at mat foundation was carried out for nonlinear time history. The super column, belt truss and outrigger diagonals sections were modeled using 4 equivalent fibers simulating I_2, I_3, A which emulate concrete with a steel embedded section. For inelastic modeling of the shear wall, 3 equivalent fibers were specified for both concrete and steel with 2 monitor fibers located at the outermost structural fibers. This model permitted out-of-plane bending of the section during analysis. When the flexural failure governed the design of nonstandard steel "FEMA" beams, the maximum plastic moment M_p and yield curvature θ_y of these beams were determined based on ASCE 41-13. To model nonlinear flexural behavior, the link beams were modeled as a compound flexural section composed of two FEMA Beams with 60% of their original elasticity. Table 2 below provides a summary of the material used in the inelastic modeling in Perform-3D. The expected strengths of structural steel, reinforcing steel, and concrete, as well as the equations to calculate the concrete modulus of elasticity as suggested by the LATBSDC 2014 were also taken into account in this elastic modeling.

Table 2 - Model material properties [21]

Inelastic Non-Buckling Steel Material	Inelastic 1D Concrete Material
F_y = 420 MPa • Elastic-Perfectly-Plastic • Symmetric • No strength loss or cyclic degradation • Strain capacity = 0.0024	C60 Concrete • Elastic-Perfectly-Plastic • No tension strength • Strength loss considered • No cyclic degradation • Tension strain capacity= 0.01 • Compression strain capacity= 0.002
	C60 Confined Concrete • Elastic-Perfectly-Plastic • No tension strength • Strength loss considered • Tension strain capacity= 0.01 • Compression strain capacity= 0.002
	C80 Confined Concrete • Elastic Perfectly Plastic • Symmetric • No strength loss or cyclic degradation • Tension strain capacity= 0.01 • Compression strain capacity= 0.002

Gravity loads were defined as line loads on structural/core wall and super column elements. The translational and rotational mass at each level was assigned as a lumped mass at each story. All nodes at each level were connected with a rigid diaphragm. P-delta effects were considered in the nonlinear analysis. Gravity load analysis was conducted before the nonlinear time history analysis.

As a damping assumption for 43-year seismic serviceability, 2.5% (or less) damping is reflected in response spectrum. While for MCE_R, 1% to 2% modal damping plus Rayleigh damping of higher modes based on PEER/ATC 72-1 Section 2.4 [6], plus hysteretic behavior were taken into account. Hysteretic Behavior were defined as follows: nonlinear material and member properties established using Perform-3D definition; initial NLRH analysis properties assume strains and rotations remain below levels initiating strength degradation, results to be compared to assumption; material and member property definitions and acceptance criteria based on values in ASCE 41-13 unless otherwise noted; and concrete coupling beam acceptance rotation limits between LS and CP values for diagonally reinforced beams based on Naish, 2010 [5] flexural hinge parameters for conforming transverse reinforcement.

Boundary conditions: effect of foundation stiffness on tower behavior will be studied using spring supports. If the effect is found to be minor (changing demands by 15% or less), subsequent models will cause rigid supports at the tower mat level. Classification of tower structural members and connections stated as follows:

a) Deformation-controlled members and connections: diagonally-reinforced coupling (link) beams at the tower core, parallel-reinforced coupling beams at the tower core, for flexure (design shear for capacity), composite link beams at the tower core for shear / flexure, core wall piers with boundary zone confinement (subject to compressive stress check), columns in tension with complete joint penetration (CJP) splices for full steel section tensile strength

b) Force-controlled members and connections: belt truss chords and diagonals, core wall piers with compressive stress beyond balance point, core wall piers acting in shear, outrigger chords and diagonals, outrigger connections, columns in compression, columns in tension not with CJP splices, diaphragm in shear, diaphragm collectors (drags) and chords, foundation mat shear based on concrete plus reinforcement, foundation mat flexure

4.3 Performance Based Design - Results

The building fundamental periods obtained from Perform-3D for the first three modes were 11.368 second, 11.52 second and 7.61 second respectively. While from the ETABS linear analysis the fundamental periods of these first three modes were 11.475 second, 11.421 second and 6.854 second. The differences are insignificant. The inter-story drift in the X and Y direction for all of the time history earthquakes are 0.4%-0.8% and 0.5%-0.9% respectively. It is below relative to the Life Safety limit state of 1.5%.

The greatest contributors to inelastic energy dissipation for all earthquake time histories were the steel link beams modeled with either a shear hinge or plastic, flexural hinge. Preliminary code based design was used to determine area of shear and flexural reinforcement to generate shear hinge properties for Link beams used in non-linear performance based analysis model. The performance levels exhibited by link beams indicate that the initial code based design lead to minimal yielding and hence minimal energy dissipation under MCE_R level event. The outrigger and belt truss diagonals by the percentage fall under each demand-capacity ratio limit state. It is observed that no members exceeded the IO limit state.

Fig.8 below plots the non-linear result of story shear, overturning moment, story drift and displacement for both X- and Y- direction.

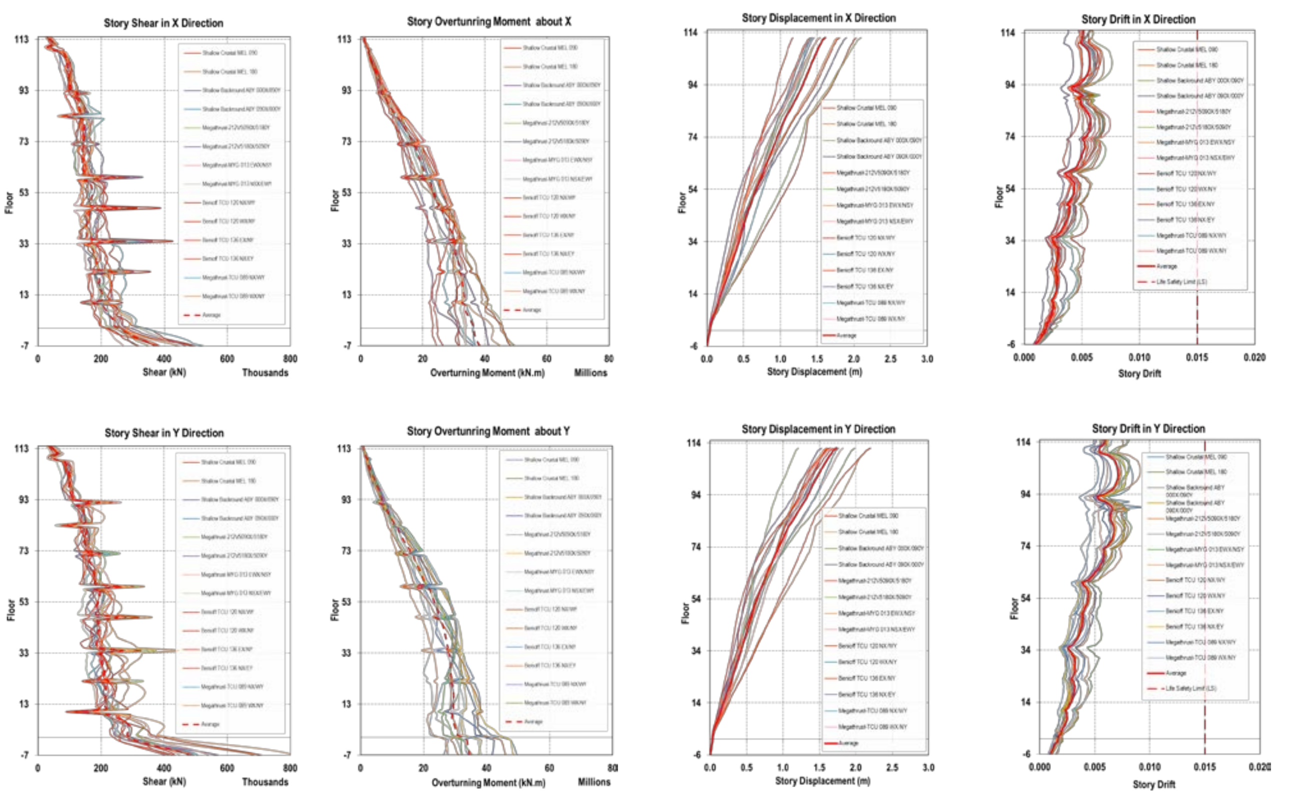


Fig. 8 – Story shear, story overturning moment, displacements and story drift for all time history earthquakes in the X- and Y- direction [21]

The PBD-result of Signature Tower has shown fulfill the minimum requirement from ASCE 41-13 and also the JBAC consensus of Jakarta. Nonlinear response history analysis shows that this mega-tall structure provide an acceptable level of performance under severe MCE_R event. The average lateral displacement or story drifts for all seven sets of ground input motions are within the code limit. Link beams exhibit considerable yielding, but plastic rotations are within the IO level. Outrigger and belt truss framing member demands are still in the elastic range.

5. Conclusion

1. Due the recent occurrence of major earthquakes and the rapid development of tall buildings in Indonesia, the importance of implementing the latest seismic codes and design standards into new building structures is without undue delay.
2. The role of the local professional engineers in observing and auditing the interpretation and implementation of these latest codes and design standards is essential, in order to guarantee life safety and seismic performance of building structures which in turn would enhance their competitive advantages in this era of globalization.
3. Risk-targeted maximum considered earthquake seismic design criteria of 1% probability of building collapse have been adopted in the new 2012 Indonesian building code. For conducting a Performance based Design using Non-linear Analysis, pairs of seismic ground motions need to be generated from site-specific probabilistic hazard and response analysis identifying the controlling earthquakes under various periods with particular ground motions at natural period of the structure, for both maximum considered and service level earthquakes. At this stage the Performance Based Design (PBD) using Non-linear analysis is being considered by the Building Authorities in Jakarta as an alternative way to prove that the building performance is within the limit stated by current Indonesian Standard.
4. As a case study, the Performance based Design (PBD) using Non-linear Analysis was carried out for the Signature Tower in Jakarta. The PBD-results fulfilled the minimum requirements of ASCE 41-13 and also the JBAC (Jakarta Building Authority Committee) consensus.

6. Acknowledgements

We acknowledge valuable discussions with Jakarta Building Authority Committee members who contributed significantly to the preparation of this paper. Our sincerely thanks to the Signature Tower project developer team for allowing their project to be discussed as a case study in this paper. We would also express our thanks to Mr. Dennis Poon from Thornton Tomasetti and his engineering team for letting us to use their design report as a reference.

7. Copyrights

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