16th World Conference on Earthquake Engineering, 16WCEE 2017

Santiago Chile, January 9th to 13th 2017

Paper N° 1861

Registration Code: S-A1463140591

PERFORMANCE BASED SEISMIC DESIGN OF REINFORCED CONCRETE TALL BUILDINGS

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Abstract

The need for tall buildings are steadily increasing all over the world, in parallel to the need for new living spaces in large cities. Improvements in technological equipment, material science and analysis methods have opened opportunities to construct new life areas by rising along the vertical direction instead of horizontal. Unlike regular buildings, tall buildings are peculiar due to their specific architectural properties and building configurations. On the other hand, most of these buildings are located in the regions of high seismicity. The behavior of tall buildings under seismic effects is crucial since the contribution of higher mode effects is significant on their dynamic behavior. Seismic response of tall buildings under the effect of seismic loading is one of the most sophisticated problems in earthquake engineering. High strength materials and innovative structural systems are generally employed to resist the unique challenges introduced by these structures in the regions of high seismicity. In this study, the behavior of tall buildings under seismic loading is summarized first by utilizing the performance based seismic design (PBSD) approach since current prescriptive seismic codes are too restrictive and inadequate to understand the anticipated behavior of tall buildings and pursue a reasonable design. Considering all these facts, several institutions and building officials have recently proposed and published alternative consensus guidelines, which are based on performance based design concepts. In this study, all of these issues are addressed. PBSD is commonly conducted by performing nonlinear dynamic analysis (NDA) for tall buildings. PBSD approach is quite sophisticated and a time consuming process, from creating nonlinear modelling to the interpretation of results. However, there are also variety of uncertainties from modelling of the component to the selection of ground motions, and to the definition of performance target levels. This study especially focusses on nonlinear modeling of reinforced concrete shear walls. Finally, a reinforced concrete unsymmetrical-plan tall building with 34 stories is designed in Istanbul according to the Turkish Seismic Code. For both service and maximum earthquake levels, nonlinear time history analysis is employed by using a suite of ground motions for checking the results in compliance with the determined target performance levels. The results have indicated that satisfactory seismic performance can be obtained through the use of performance based seismic design procedures.

Keywords: Reinforced concrete tall buildings; Performance based seismic design; Verification of nonlinear modeling shear wall; nonlinear dynamic analysis

1. Introduction

Performance based seismic design by employing nonlinear dynamic analysis (NDA) has been increasingly used to understand the behavior of tall buildings. It has led to a new vision for the seismic design of tall buildings, pioneering to a smart shift in analysis and assessment methods from the prescriptive force-based design methods based on linear elastic analysis under reduced seismic loads and capacity design principles, to non-prescriptive displacement-based design methods based on nonlinear analysis and performance evaluations with respect to the expected demand parameters [1,2,3]. In general, tall buildings comprise of a reinforced concrete core shear wall along with a peripheral frame structure having a low redundancy. Unlike conventional regular buildings, shear walls are designed to withstand not only nearly all of the lateral forces (seismic and wind) but also a considerable amount of gravity forces due to having less redundant structural systems. In other words, the safety factor for preventing collapse of a tall building is low and it may collapse if a major component of the primary structural system sustains heavy damage or collapse. PBSD of tall buildings are generally based on an approach which consists of a preliminary design stage based on the capacity design principles under design earthquakes with a return period of 475 years (moderate earthquakes), followed by two performance evaluation stages, service level and collapse prevention checks, respectively. Service level evaluation stage is to check structure under the high probability of occurrence (frequent) earthquakes with return periods of 43~72 years (small to moderate earthquakes). Collapse prevention evaluation stage checks the structure under rare earthquakes with a return period of 2475 years (severe earthquakes). In the first evaluation stage, it is generally desirable that the tall buildings remain essentially elastic. Linear response spectrum analysis is generally utilized for this stage since permanent damage is not appreciated; however, nonlinear response history analysis may also be utilized. In the second evaluation stage, the performance target is maintaining stability under expected strong earthquakes, namely collapse of the structures must be prevented. Limited damage in specified locations is permitted for reasonable designs [2,3,4,5]. Conventional seismic design codes are force-based methods that account for inelastic response indirectly by reducing the results of linear elastic response with an inelastic response factor R. The value of R depends on the structural system and the level of ductility as explained in seismic design codes. Nonlinear behavior is considered indirectly by this approach. After all, as observed from Fig. 1, which describes design base shears from the Turkish Seismic Code (2007) for a typical site, the effective base shear force is minimized from the value specified in seismic code for that structural system to a smaller value depending on building period, seismic zone and other factors. The required strength is usually controlled by minimum base-shear requirements for a tall building [6]. In addition, current prescriptive seismic codes are too restrictive on structural height, period and minimum base shear requirement [7]. Accordingly, several institutions (task groups) or building officials have proposed improved building codes and published non-prescriptive seismic design guidelines for tall buildings based on displacement-based methods developed in the last decade since the need has grown. The Los Angeles Tall Buildings Seismic Design Council (LATBSDC) published the first guideline in 2005. It is updated several times in the light of developments in performance-based design (2005, 2008, 2011, 2013, and 2014, 2015). It is a consensus document between structural engineers and certified authorities which is ‘‘an alternative procedure for seismic analysis and design of tall buildings located in the Los Angeles Region’’ [3]. Another guideline for the seismic design of high-rise buildings was published by the Council on Tall Buildings and Urban Habitat (CTBUH, 2008), which is a non-profit organization and an international group in the area of tall buildings and sustainable urban design. This guideline is also a consensus document. One of the other guidelines ‘‘Administrative Bulletin NO: AB-083’’ was prepared for the San Francisco Department of Building Inspection (SFDBI) in 2007 by Structural Engineers Association of Northern California (SEONAC). Then it has been updated in 2014. The objective of this administrative bulletin was to show requirements and recommendations for the seismic design of new tall buildings located in the San Francisco region [4]. The most comprehensive and favorable guideline, ‘‘Tall Buildings Initiative (TBI)’’ has been prepared by the Pacific Earthquake Engineering Research (PEER) Center working group in 2010. It consists of twelve specific tasks and five reports on the developing consensus from performance objectives to modeling and acceptance criteria for seismic design and analysis to the instrumentation of tall buildings. It is a pioneering guideline and aims to offer an alternative non-prescriptive procedure for the next generation seismic codes for tall buildings [2]. Kandilli Observatory and Earthquake Research Institute has prepared a seismic design guideline for Istanbul Metropolitan Municipality (IMM) in 2008. ‘‘Istanbul seismic design code for tall buildings’’ was not published officially, but has been used in practice [5]. A dedicated chapter for the seismic design of tall buildings has recently been added to the Turkish seismic design code (2017).

Unlike conventional linear elastic procedures, NDA considers nonlinear behavior explicitly and produces quite reasonable results under design and maximum considered earthquake shaking if it is utilized properly [8]. However, this method is more sophisticated and time consuming compared to linear elastic methods. Where NDA is used, there are three important steps; modeling, analysis and assessment, respectively. The first step is modeling where selection of correct inelastic component types for each structural member is carried out. Inelastic component types are mainly categorized in three groups, continuum finite element models, fiber models and lumped plasticity models. Each of these models has some advantages and shortcomings but fiber models can be generally used for shear wall elements and lumped plasticity models for frame elements in practice since current analytical modeling and computer analysis software and their capabilities are mostly based on these models. In addition, unlike linear elastic analyses, the results of nonlinear analyses are influenced and depend on the gravity load effects directly; therefore, the selection of appropriate expected gravity load is important. It is generally taken as dead load [G] and some portion of the design live load [0.2~0.3Q] [9]. The second step of analysis is where a suitable suite of representative ground motion sets and suitable damping values are chosen. Tall buildings are long period structures so it might be troublesome to detect appropriate ground motion records to obtain accurate response from these structures. Either spectrum matching or scaling method based on the target linear response spectrum shape is used to choose and manipulate ground motions for expected performance levels [2]. If uniform hazard spectrum is utilized, then spectrum matching may be preferred for tall buildings [8]. On the other hand, tall buildings are special structures thus scaling procedure with conditional mean spectrum from site-specific seismic hazard analysis can be carried out by considering all of the properties of site conditions and the fundamental periods of the structure. This method is being widely employing for tall buildings [10]. One of the most important parameter is the viscous damping ratio. It is generally taken as 2~5 % for concrete structures and 2~3 % for steel structures with respect to the target performance level. In addition, P-Delta effects must be considered not only at the design stage but also at the performance evaluation stages [9]. The last part is assessment stage where the interpretation of the results and checking the building behavior in compliance with the determined target performance criteria are performed.

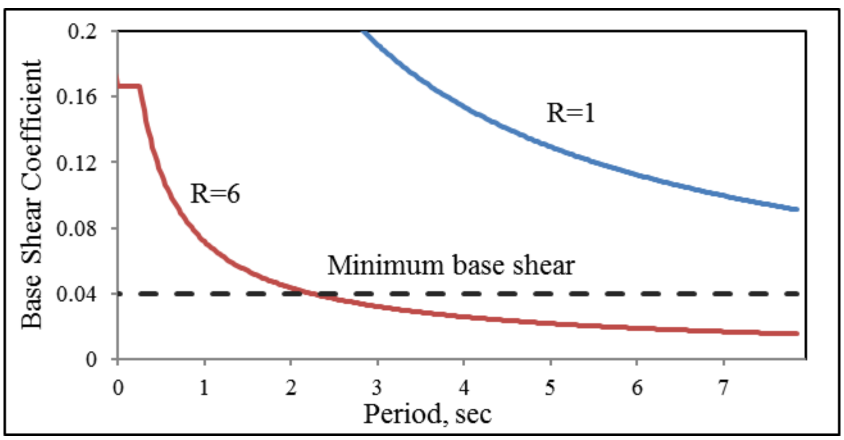


Fig. 1 – The reduced and minimum base shear forces according to TEC 2007.

2. Types of Nonlinear Models

The fundamental nonlinearity sources in a structure are material nonlinearity and geometric nonlinearity. For a realistic analytical simulation of the structure, both geometric and material nonlinearity should be taken into consideration but the first question comes to mind is how an appropriate model should be generated to forecast structural response. There are variety of parameters that need to be considered during the selection of a favorable inelastic model. These are the type of structural system, type of members which comprise the structural system, materials, expected overall response of the members or components, governing and controlling type of actions desired to be captured during analysis, unknowns and uncertainties which comes from inherent nonlinear behavior, the analysis objectives and necessary demand parameters, design and construction (stage construction), time and effort, computer analysis software and its capabilities [9].

Several inelastic structural component types are available in practice but they can be mainly categorized into three groups, based on the degree of idealization in the model. The term ‘‘degree of idealization’’ refers to where and how inelastic action is modeled in a member such as integrated inelastic behavior of a member idealized at a point (lumped plasticity model) or a zone (fiber model) or distributed by a specific characteristic length over the entire length, finite element model. Basically in fiber and continuum finite element models, expected nonlinear behavior of the component is captured explicitly by the nonlinear behavior of the material that constitutes the component. Whereas finite element models are based on more complex material constitutive relationships, fiber models are based on simpler basic uniaxial material properties to capture the overall response of the structure in practice. Unlike continuum finite element model, cross section of a member is divided into steel and concrete fibers according to steel or concrete included in the fiber model shown in Fig 2. The last type of nonlinear model is the concentrated hinge model based on the overall response of prismatic components. It is characterized by force and deformation relationships, which is the backbone curve of component that identifies the capacity of the component under monotonic loading. This action is changeable from one component type to the other and depends on the expected behavior of member under the expected loading. The main objective of the backbone model for a component is to capture the basic features of the component behavior, namely the initial stiffness, strain hardening, ultimate strength, strength loss and related deformation capacity [9].

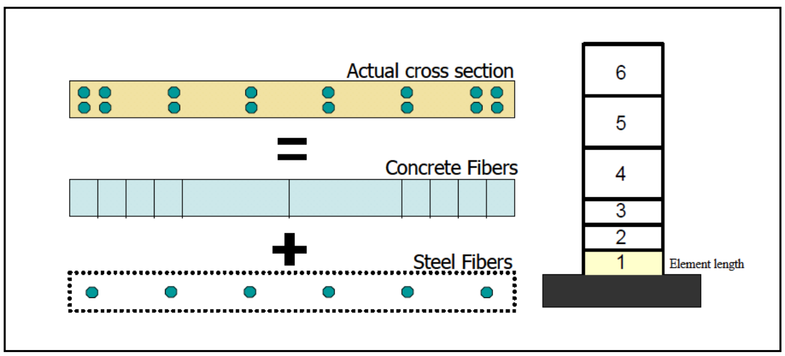


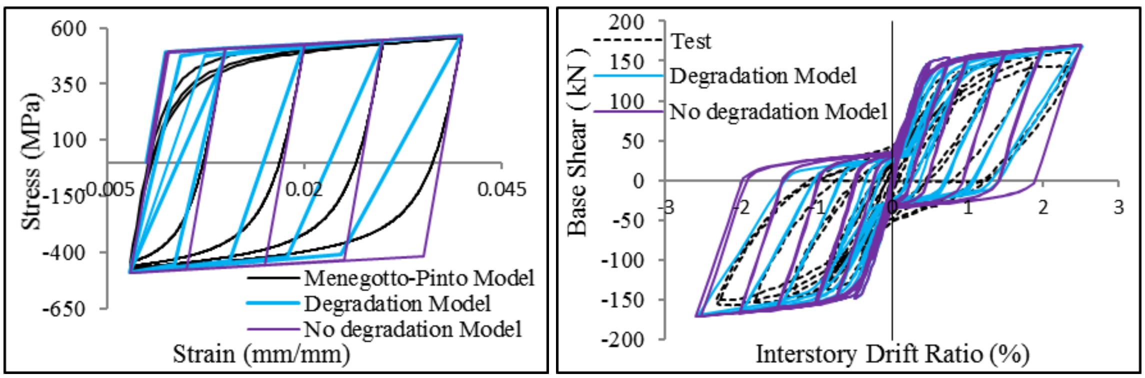
Fig. 2 – Idealized cross section for fiber model and elevation of shear wall reduced and minimum base shear forces according to TEC 2007.

In practice of performance based seismic design of tall buildings, use of continuum finite element models is neither practical nor available with current analysis software. Instead, concentrated hinge model is more practical generally for frame member types such as columns, beams etc. There are two reasons of this. First, it is not practical to use fiber models in the modeling of frame members, as it requires so much time during analysis with the existing computer analysis programs. Second, current analytical models and acceptance criteria specified in codes for frame type member are based on lumped plasticity (concentrated hinge) models. On the other hand, fiber models can represent the behavior of shear walls more accurately than the others since it may not be realistic to model complex core shear walls by simple concentrated hinge models that define the inelastic behavior of a member at a point. In addition, use of more concentrated hinges for a complex shear wall is also not a simple and practical task in practice [9].

2.1 Nonlinear Modeling of Reinforced Concrete Shear Walls

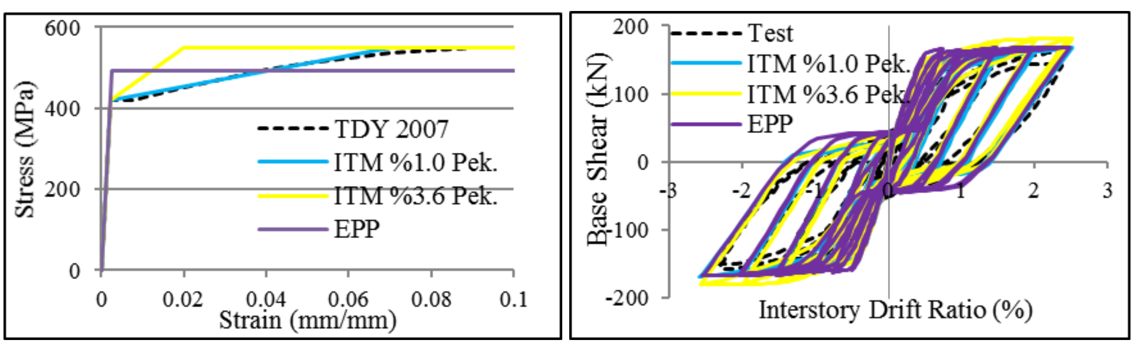
Fiber models generally represent shear wall elements if NDA is utilized in practice. In fiber model, a shear wall member consists of a number of wall elements as illustrated in Fig 2. Each of the wall elements is comprised of a number of steel and concrete fibers, which are also illustrated in Fig 2. Most significant parameters in fiber modeling are using correct material stress-strain relationship, choosing a convenient number of wall elements, reinforcing steel and concrete fiber size and plastic hinge (element) length for a realistic nonlinear model. For this purpose, these parameters are calibrated and selected with respect to test results, sensitivity and/or parametric studies [9].

A moderate strain hardening (1 %) with a simplified trilinear model for considering cyclic loading effects for steel, and a simplified confined concrete models for wall boundaries and unconfined concrete models for wall web is sufficient to predict building behavior under cycling loading. Model predictions are compared with test results in Figs. 3, 4 and 5. The results also reveal that that using more number of wall elements has resulted in less accurate results since inelastic deformations concentrate in a single element especially when drift ratio has increased. Instead of using too many elements, using an equal plastic hinge length with moderate wall elements produce results that are more rational. However, as the drift ratio increases, the discrepancies between the results of wall region, shown in Fig. 5-b, have grown [1]. According to Salas [11], these differences might have occurred because of shear-flexural interaction.



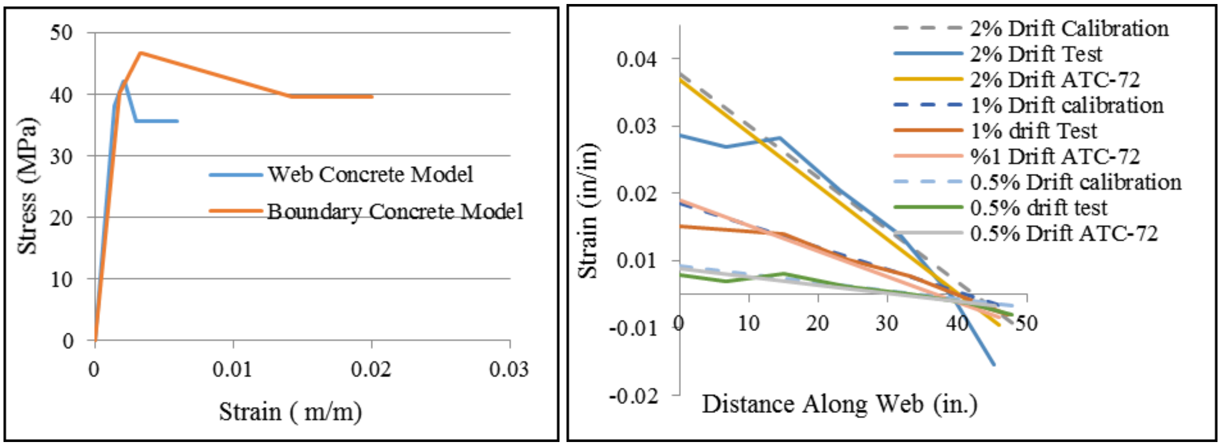
(a) (b)

Fig. 3 – Comparison of (a) Menegotto-Pinto steel model with two different PERFORM 3D models and (b) analytical models and Wallace RW2 specimen test results for rectangular wall section



(a) (b)

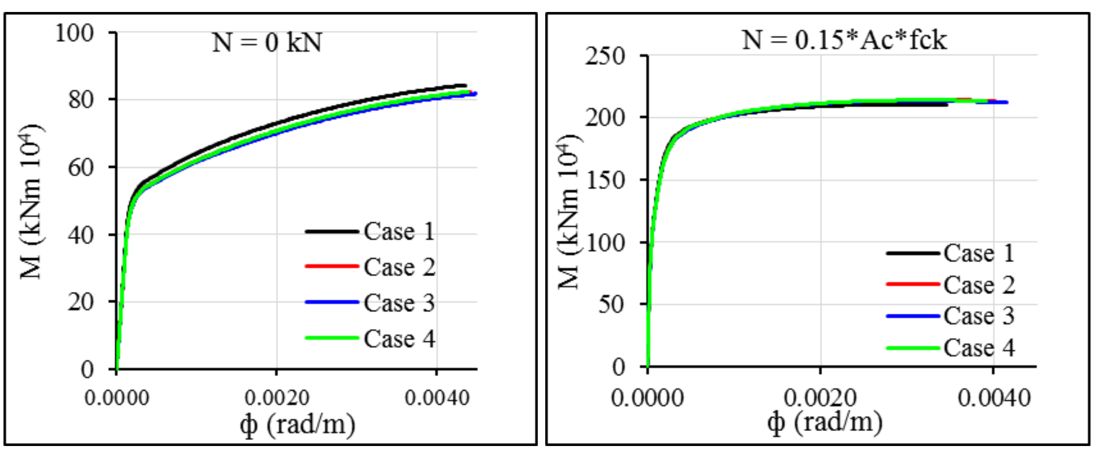
Fig. 4 – (a) Comparison of different idealized steel models under monotonic loading and (b) the results obtained by using these models, compared to Wallace RW2 specimen test results for rectangular wall section.



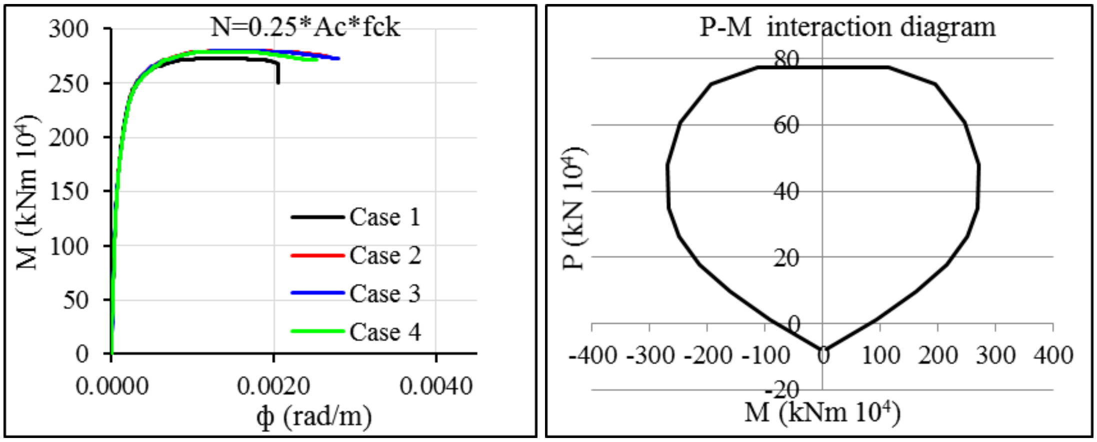
(a) (b)

Fig. 5 – (a) Comparison of web concrete model with boundary concrete model, (b) Strain distribution at the base of the wall for 6 elements

Clearly, a tall building has much more components and connections that makes the analysis and modeling stage more time consuming. Using many fibers can require much longer time during analysis and modeling of tall buildings. In addition, using too many fibers is generally not feasible in the commercially available software, such as Perform3D V5 [12]. In other words, number of fibers is limited in the analysis programs. Accordingly, how the number of fibers can affect the analysis results should be examined by comparing the results of detailed commercially available section analysis programs such as XRACT with Perform 3D to obtain the optimum number of fibers in modeling. For this purpose, parametric studies should be carried out. For example, four case studies with detailed properties are given in Budak [1], with three different axial load levels (0, $0.15f_cA_c$ and $0.25f_cA_c$). The results presented in Fig 6 show that instead of using too many fibers with more wall elements, using relatively more fibers in the wall boundaries with relatively less fibers in the wall web give more realistic results (comparison of case 4 with case 1 (more detailed case)). In addition, using confined concrete models for wall boundaries and unconfined concrete models for wall web is sufficient for predicting building behavior rationally under cycling loading [1,9].



(a) (b)



(c) (d)

Fig. 6 – (a-d) The effect of fiber size of the wall element for different axial load levels and interaction diagram

3. Performance Levels and Acceptance Criteria for Tall Buildings

Non-prescriptive seismic guidelines for tall buildings define two performance levels for tall buildings. These are service level and collapse prevention level, respectively. Service level evaluation stage is to check the structure under high probability of occurrence (frequent) earthquakes with return periods of 43 years, (50% probability of exceedance in 30 years). At this stage, it is generally desirable that the tall buildings remain essentially elastic. A small post-yield deformation is allowed for ductile members but a permanent damage is not appreciated. Collapse prevention evaluation stage is to check the structure under the low probability of occurrence earthquakes with return periods of 2475 years (50% probability of exceedance in 2 years). It is desired to maintain stability under expected strong earthquakes, namely collapse of the structures is undesirable. Instead of these, limited damage in specified locations and a specific stress value is permitted for reasonable designs [2, 3, 4]. On the other hand, IMM defined three performance levels (IO, LS and CP). Expected minimum performance regions for different earthquake levels are given in Table 1 [5, 6].

Table 1 – Expected minimum performance regions for different earthquake levels

The usage purpose	Probability of exceeding		
	50 % in 50 years	10 % in 50 years	2 % in 50 years
Ordinary tall buildings (residences, offices, hotels)	Minimum damage (IO)	Significant damage (LS)	Severe damage (CP)
Special tall buildings (Schools, Hospitals, health facilities)	-	Minimum damage (IO)	Significant damage (LS)

Alternative non-prescriptive seismic guidelines for tall buildings propose acceptance criteria in ASCE 41-13 and/or TEC-2007 in order to evaluate seismic performance of members but these acceptance criteria are inadequate for tall buildings since their seismic demands are different from conventional regular buildings [2, 3, 5]. Since seismic performance of tall buildings is based on expected material properties, limit states of some components (force-controlled members) should be revised. Further, some additional acceptance criteria, especially on the overall building behavior, have been described for tall buildings in these documents. According to PEER-TBI and LATBSDC, overall building acceptance criteria for tall buildings involve maximum transient drift, residual drift and loss of story strength. Although transient drift (δ_{max} and/or δ_{ave}) is employed both for service level evaluation and collapse prevention levels, residual drift ($\delta_{r,max}$ and/or $\delta_{r,ave}$) and loss of story strength is only employed for the collapse prevention level. The following limit states for overall building performance are proposed in order to check the performance of structure with respect to determined performance levels [2,3].

For service level evaluation:

- If less than seven pairs of ground motion are used, $\delta_{max} \leq 0.5\%$
- If seven or more pairs of ground motion are used, $\delta_{ave} \leq 0.5\%$:

For collapse prevention evaluation:

- If seven or more pairs of ground motion are used, $\delta_{ave} \leq 3.0\%$
- If seven or more than seven pairs of ground motion are used, $\delta_{max} \leq 4.5\%$
- If seven or more pairs of ground motion are used, $\delta_{r,\,ave} \leq 1.0\%$
- If seven or more pairs of ground motion are used, $\delta_{r,\,max} \leq 1.5\%$
- Final total story strength ≥ 0.8*(initial total story strength)

4. An Application of Performance Based Seismic design of Tall Buildings

The case study building is 34 stories tall, with 2 basement levels and 32 upper ground levels, which is 115 meters in height. Typical floor plan, which is 19 m by 48.25 m, and 3D view of the building, is shown in Fig.7. Due to the lack of an official seismic design code for tall buildings, the building is designed according to the Turkish Seismic Code based on the capacity design principles under design earthquake with a return period of 475 years in Istanbul, located in a severe seismic zone, i.e. [6]. Selected materials are C45 and S420 for concrete and steel respectively. All of the selected design parameters and section properties are given in Budak [1].

Preliminary design is conducted by considering R=6 as the response reduction factor, but the earthquake load is modified in order satisfy minimum base shear requirement and the design is performed. Both service (SLE) and collapse prevention level (MCE), NDA are employed by using a set of seven ground motions selected according to the principle of spectrum matching method as shown in Fig.8. Some of the selected results are shown below to check their compliance with the determined target performance levels selected from alternative non-prescriptive seismic guidelines for tall buildings.

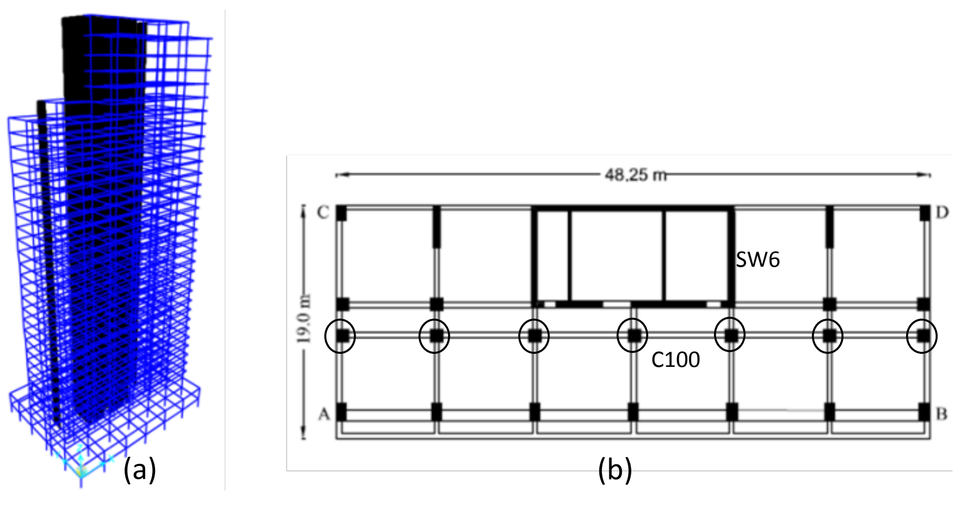
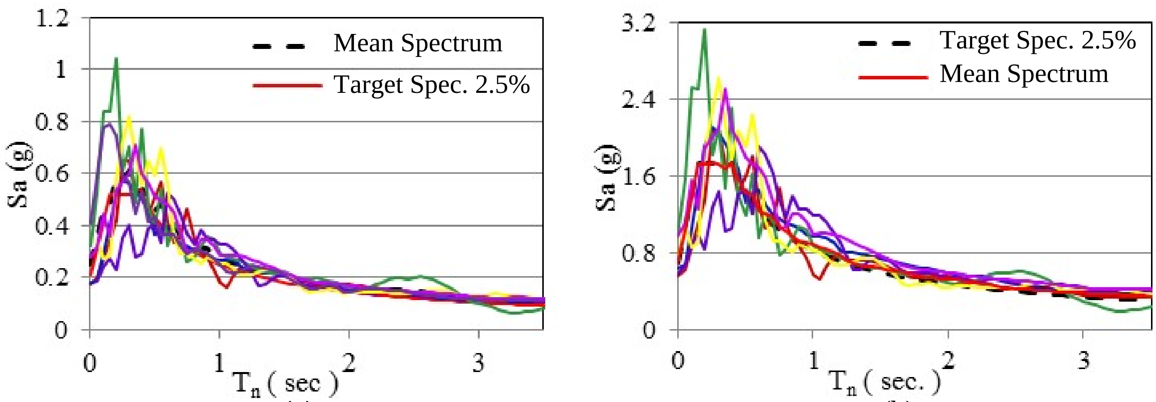


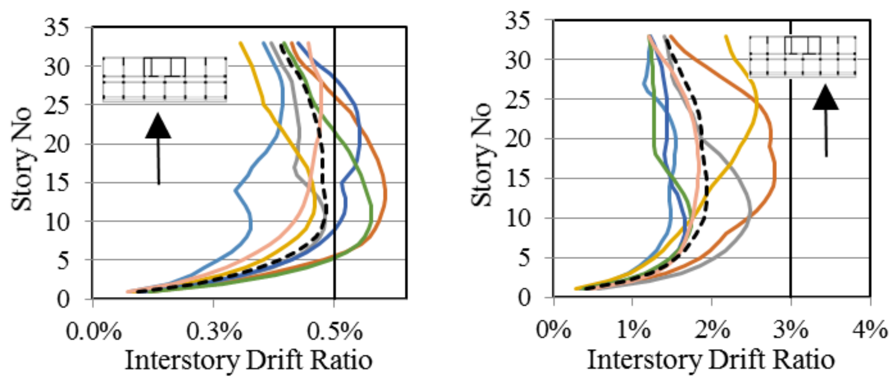
Fig. 7 – (a) 3D view (b) typical floor plan of the building



(a) (b)

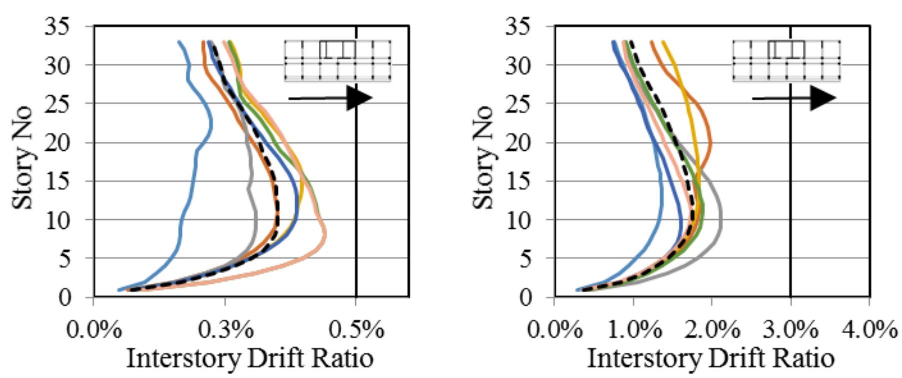
Fig. 8 – (a) Selected ground motions with acceleration response spectra for service and (b) MCE level.

Performance limits for maximum average transient interstory drift ratios are taken as 0.5% and 3% for service level and collapse prevention level evaluations, respectively. In addition, the limit state of the maximum drift ratio in each story for each ground motion is taken as 4.5% for collapse prevention level. As observed in Fig. 9 and Fig.10, the obtained interstory drift results in X and Y direction of the building satisfy target performance levels. The results also show that the contribution of higher modes on interstory drift ratio is significant.



(a) (b)

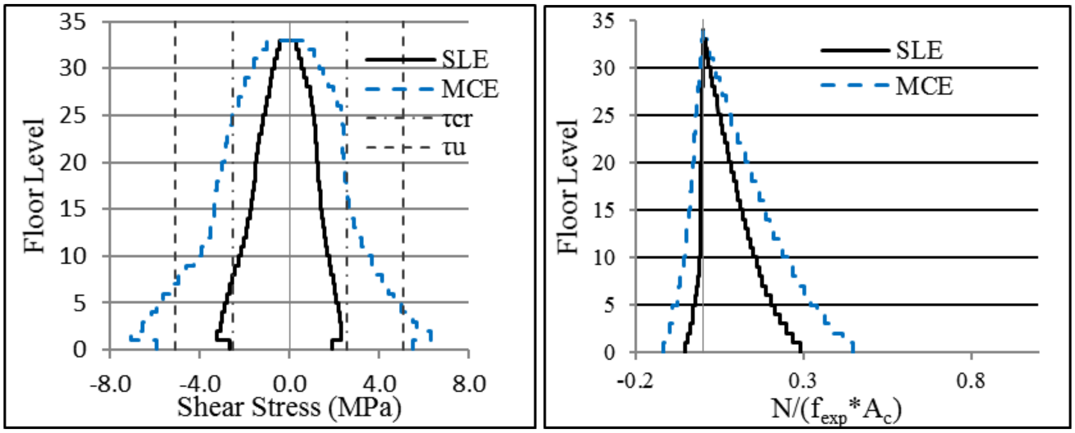
Fig. 9 – (a) Maximum transient interstory drift ratio in Y direction for service and (b) MCE level



(a) (b)

Fig. 10 – (a) Maximum transient interstory drift ratio in Y direction for service and (b) MCE level

The results in Fig.11-a show that the contribution of higher modes to total shear stress is significant under MCE ground shakings. The shear stresses in walls decrease rapidly from the second story to the 10th story, but change in the shear stress is not significant, as evidenced by the asymptotical distribution in Figure 11, from 10th story to 25th story. The results also show that shear stress is significantly over the limit in the first six stories. Shear stresses reach their maximum value at the second story due to the presence of a podium level. High shear forces are transferred between the podium level and the tower. Moreover, SW6 shear wall also exceeds the limit of shear cracking stress under SLE shaking. According to these results, revision of the preliminary design is necessary by increasing the dimension of shear walls. Fig.11-b shows the axial load level in shear wall SW6.



(a) (b)

Fig. 11 – Comparison of average (a) shear stress and (b) axial load level of shear wall SW6 under SLE and MCE shaking

One of the performance parameters controlling the damage of columns under an earthquake excitation is axial load because of reasons related to ductility. Fig.12 shows that according to the LATBDSC 2015, the results from selected columns do not satisfy the required performance level (40%) so that the preliminary design of columns should also be revised by increasing section dimensions or characteristic strength of concrete.

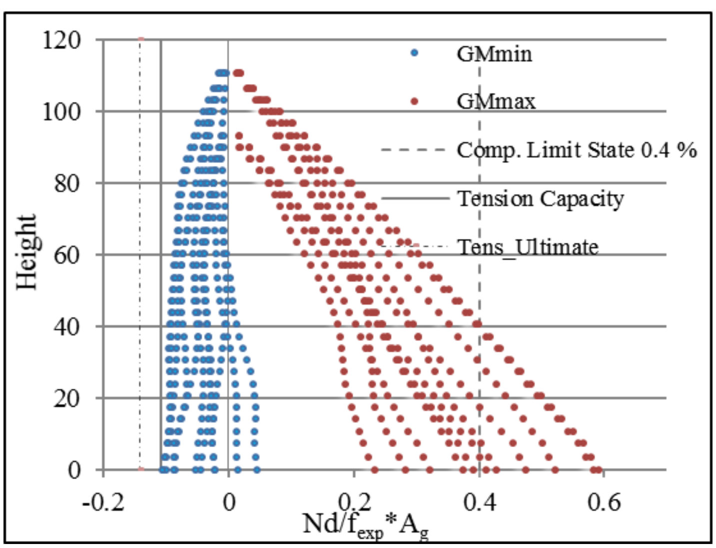


Fig. 12 – Average axial load level on selected columns (C100 under MCE shaking

One of the performance parameters to anticipate the damage of shear walls under an earthquake excitation is axial strain. According to TEC-2007, ASCE41-13 [13] and other non-prescriptive guidelines for tall buildings, the amount of strain in compression and tension at each story level, shown in Fig.13, is acceptable. The results also show that a plastic zone, which is nearly equal to 15 % of building height, is developed at the base of each shear wall under the MCE excitation.

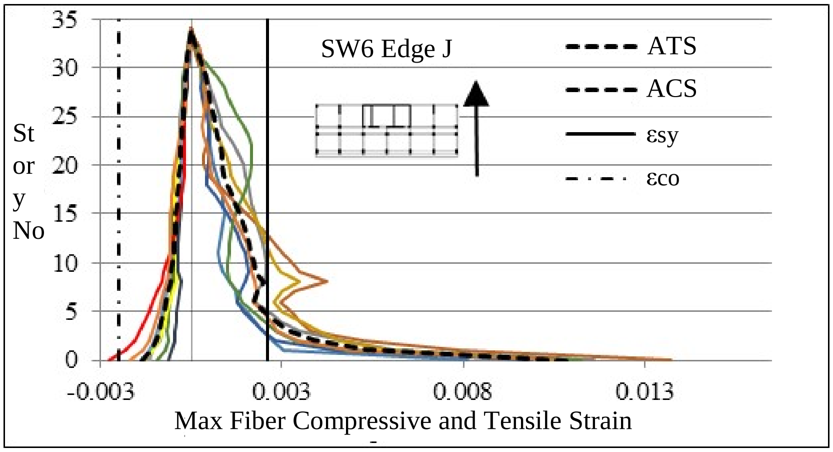


Fig. 13 – Axial strain at edge J (external edge) of shear wall SW6 under MCE shaking

5. Conclusion

Performance based seismic design of tall buildings by conducting NDA as an analysis tool is being increasingly used, which is summarized in this study. In addition, the key parameters of nonlinear modeling of reinforced concrete shear walls are examined. The results of a case study show that although the case study building is designed according to the provisions of TEC-2007 for regular buildings, it does not provide adequate resistance under expected earthquake levels. The primary reason is the inadequacy of TEC-2007 for tall buildings.

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