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PERFORMANCE BASED DESIGN OF A HIGH RISE BUILDING BASED ON ISTANBUL TALL BUILDING SEISMIC CODE

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High-rise construction has increased rapidly in Istanbul especially in last five years. As the current Turkish Seismic Code does not cover the design criteria for high-rise buildings, no specific Code is being used about the seismic design of type of buildings. In order establish a set of criteria a draft code named as "Istanbul Seismic Design Code for Tall Buildings (ISDCTB)" has been prepared by Department of Earthquake Engineering of Kandilli Observatory and Earthquake Research Institute in 2008. The proposed document follows the performance based design approach which has been introduced to earthquake engineering society in the beginning of 2000s. Recently a group of researchers and practicing engineers in California, USA released a document named as "Tall Building Initiative" in 2010. Additionally some supporting documents have also prepared by the same group for analysis and design aspects of tall buildings.

Although there is no compulsory regulation of using the relevant Code in Istanbul, both the designers and investors are asking to use the relevant Code for more reliable and economic design. As such the so-called Renaissance Tower, an 43 storey office type building located in Asian side of Istanbul with 195 m total height is analysed and designed based on the criteria defined in Istanbul Seismic Design Code for Tall Buildings.

The structural system has been divided in to two as "lateral load resisting system" and "gravity system" for seismic and gravitational actions. The lateral load resisting system is composed of reinforced concrete core and buckling restrained braces (BRB) and the gravity load system is composed of peripheral columns and flat slabs. The foundation system is composed of mat with 2.5 m thinness and piles with 1.2 m diameter.

A site specific seismic hazard analysis has been performed; relevant parameters and ground motion time history data sets are prepared in two orthogonal directions to be used in nonlinear time history analysis in order to determine the deformation demands. The BRB element are located at a specific location in height so that the lateral displacement demands due to seismic actions are control by the BRB system connected to the core in one direction. The core parts are connected to each other with link beams made of structural steel. A series of nonlinear time history analysis has been performed with 14 simulated ground motions acting simultaneously on the lateral load resisting system. As the result of the analyses deformation demands in terms of concrete and steel strains are compared with the limits defined in ISDCTB. As for the wind effects a 3D wind tunnel test has been performed in order to determine the reliable wind effects acting on the structure.

In addition to the detailed evaluation in the analysis and design phase special emphasis has been applied on the construction phase. Especially during the foundation construction special procedures are defined and precautions are taken for the pouring of mass concrete, construction joint, horizontal shear effect, temperature difference, creep and shrinkage. As for the link beams and BRB elements special production and testing procedures have been defined before production and installation process. As an item defined in ISCTB, structural health monitoring system will be established in the

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structure for monitoring the response of the structural system and act as an early warning system that is lined to the vital systems of the building like natural gas system.

As the result of whole work performed, it can be deduced that the Renaissance Tower has been analysed, designed and constructed through state of the art engineering methodologies and quality controlled material and workmanship. Through the performance based design approach the building will suffer minimum damage under design basis earthquake and continue to be functional after a minimum time of recovery. As the number of high-rise buildings is increasing all around Turkey a special High Rise Code for whole Turkey is under preparation process. Independently from an obligatory point of view, such building with complex response and high cost of investment, a special emphasis should be paid by the owner as in the case of Renaissance Tower. Considering the efforts allocated for the construction the relevant structure it can be represented as one of the good examples in high rise design and construction.

INTRODUCTION

Parallel to the rapid urbanization in major cities in Turkey high-rise building construction has increased significantly. The majority of the high-rise construction consists of cast in place reinforced concrete and is located in Istanbul. Considering the level of seismic hazard in Istanbul, the design and construction process of these buildings have become a big challenge. On the other hand, the current Turkish Seismic Code does not cover the analysis and design criteria for this special type of buildings. Unfortunately seismic design of the buildings is performed without any official regulation. Currently some of the high-rise buildings are designed based on international Codes such as “Seismic Design Guidelines for Tall Buildings”, “International Building Code” and unofficial “Istanbul Seismic Design Code for Tall Buildings”. All of these codes follow the performance based design approach which has been introduced to earthquake engineering society in last decade. The design process of a 43 story high-rise RC building named as Renaissance Tower which has a lateral load resisting system made of RC core supported with BRB elements as outrigger system is summarized in this study.

SEISMIC DESIGN GUIDELINES FOR TALL BUILDINGS IN ISTANBUL

As it has been mentioned above, “Istanbul Seismic Design Code for Tall Buildings-2008” which is based on PEER Performance Based Design approach philosophy has been introduced to Turkish engineering society. The analysis and design criteria is defined in terms of deformations for ductile, and strength for brittle response modes.

The basic idea that constitutes the objective of performance based design is to capture the nonlinear response of the structures under different level of seismic input through a set of acceptance criteria. As such, similar approach have been applied to the Renaissance Tower aiming at better understanding the behavior and performance of the building structure when subjected to Maximum Considered Earthquake (MCE) level ground motions.

The basic principle of Istanbul Seismic Design Code for Tall Buildings-2008 (ISDCTB-2008) is based on performance-based design under earthquake action. In this approach, the damage to occur in the elements of structural system under given levels of earthquake ground motion is quantitatively estimated and checked in each element whether it exceeds the acceptable damage limits. The acceptable damage limits are specified under various earthquake levels in conformity with the performance objectives identified for the structure. Since the earthquake damage to be estimated at element level is generally represented by the nonlinear deformations to occur beyond the elastic strain limits, performance-based design approach is directly related to nonlinear analysis methods and the deformation-based design concept. Nevertheless, linear analysis methods are permitted in the Code as well in the framework of strength-based design approach for performance objectives where limited damage is expected.

All the design and analysis process is performed based on PEER Report 2010/05, ASCE/SEI 41/06 together with Istanbul Seismic Design Code for Tall Buildings (ISDCTB, 2008).

All the acceptance criteria in terms of deformation and force demands on individual structural components are defined in ISDCTB -2008 and ASCE41 document. Global demand parameters such as story drifts are also used as an important indicator of possible damage to nonstructural components and overall building performance. In the case of the Renaissance Tower it is assumed that lateral columns perform in linear range under axial loads and shear walls will remain elastic under shear demands.

Nonlinear response is accepted in limited range in the shear walls for flexure, link beams and outriggers. Axial strain in core walls, outriggers and rotation of link beams are assumed to be deformation controlled actions whereas shear in the core walls and axial force on the lateral columns are assumed to be force controlled actions.

As for the Istanbul Seismic Design Code for Tall Buildings ISDCTB 2008 document indicates the performance objective as Collapse Prevention Performance Levels for the structure when subjected to the Maximum Considered Earthquake ground motion intensity. On the other hand ASCE41 defines Collapse Prevention Limit as; ensuring a small risk of partial or complete building collapse by limiting structural deformations and forces to the onset of significant strength and stiffness degradation

Performance Levels and Objectives

In ISDCTB-2008 document the performance levels of tall buildings are defined below with respect estimated damages to occur in three different earthquake levels. (E1,E2 and E3). The acceptable damage limits for those performance levels shall be quantitatively defined separately for each structural type or element. The performance levels are defined as;

Minimum Damage (Uninterrupted Occupancy) Performance Level describes a performance condition such that no structural or nonstructural damage would occur in tall buildings and in their elements under the effect of an earthquake or, if any, the damage would be very limited.

Controlled Damage (Life Safety) Performance Level describes a performance condition where limited and repairable structural and nonstructural damage is permitted in tall buildings and in their elements under the effect of an earthquake..

Extensive Damage (No-collapse Safety) Performance Level describes a performance condition where extensive damage may occur in tall buildings and in their elements under the effect of an earthquake prior to the collapse of the building.

The regions in between the above-defined performance levels are identified as performance ranges as describes in Figure 1 below. The region below (MD – UO) Performance Level is defined as Minimum Damage / Uninterrupted Occupancy Performance Range, the region in between (MD – UO) Performance Level and (CD – LS) Performance Level is defined as Controlled Damage / Life Safety Performance Range, the region in between (CD – LS) Performance Level and (ED – NC) Performance Level is defined as Extensive Damage / o-collapse Safety Performance Range and the region above the (ED – NC) Performance Level is defined as Collapse Range. Additionally minimum performance objectives identified for tall buildings are given below in Table 1.

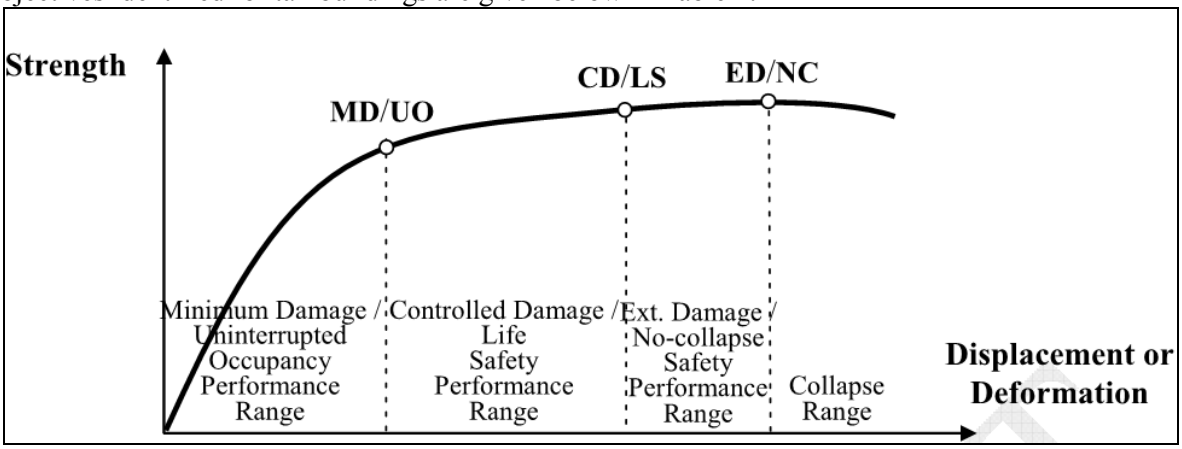

Figure 1. Performance levels and regions defined in ISDCTB -2008

Table 1. Building class minimum performance objective relationship

Building Occupancy Class	**(E1) Earthquake Level**	**(E2) Earthquake Level**	**(E3) Earthquake Level**
Normal occupancy class (residence, hotel, office building, etc.)	MD/UO	CD/LS	ED/NC
Special occupancy class (health, education, public admin. Building, etc)	-	MD/OU	CD/

Performance Based Seismic Design Stages of Tall Buildings

In ISDCTB-2008 document a four staged design process is defined. The relevant design stages are; *Design Stage (I – A):* Preliminary Design with Linear Analysis for Controlled Damage/Life Safety Performance Objective under (E2) Level Earthquake, *Design Stage (I – B):* Design with Nonlinear Analysis for Controlled Damage/Life Safety Performance Objective under (E2) Level Earthquake. *Design Stage (II):* Design Verification with Linear Analysis for Minimum Damage/ Uninterrupted Occupancy Performance Objective under (E1) Level Earthquake. *Design Stage (III):* Design Verification with Nonlinear Analysis for Extensive Damage/No-Collapse Safety Performance Objective under (E3) Level Earthquake. The above mentioned analysis and design process is summarized in Table 2 below.

Table 2. Analysis and design procedure defined in ISDCTB 2008

Design Stage	***Design Stage I-A***	***Design Stage I-B***	***Design Stage II***	***Design Stage III***
Design Type	Preliminary Design	Design	Verification	Verification
Earthquake Level	Normal class buildings (D2- earthquake)	Normal class buildings (D2- earthquake)	Normal class buildings (D2- earthquake)	Normal class buildings (D3- earthquake
	Special class buildings (D3- earthquake)	Special class buildings (D3- earthquake)	Special class buildings (D2- earthquake)	
Performance Objective	Life Safety	Life Safety	Immediate Occupancy	Collapse Prevention
Analysis Type	3D linear response spectrum	3D nonlinear time history	3D linear response spectrum	3D nonlinear time history
Structural System Behavior Coef.	R≤7	-	R=1.5	-
Story drift ratio limit	%2	%2.5	%1	%3.5
Section stiffness in RC frame members	Effective stiffness (from TSC2007)	Effective stiffness (moment curvature analysis)	Effective stiffness (moment curvature analysis)	Effective stiffness (moment curvature analysis)
Material Strength	Characteristic strength	Expected Strength	Expected Strength	Expected Strength
Acceptance Criteria	Strength & story drift ratio	Strain & story drift ratio	Strength & story drift ratio	Strain & story drift ratio

EARTHQUAKE RESISTANT DESIGN OF RENAISSANCE TOWER

The Renaissance Tower is owned by Renaissance Real estate Investment Inc. and the relevant building is a 43 story office high-rise building located in the financial district of Asian side of Istanbul. The structural lateral load resisting system is composed of RC core supported with buckling restrained braces. The building has a 2.5m thick mat foundation supported by 1.2m diameter piles and flat slab system is used with 30 cm thickness.

The architectural design is performed by FX Fowle located in New York and seismic analysis and design is performed by De Simone Consulting Engineers located also in New York. Wind tunnel test are also performed by CPP engineering. The architectural sections and wind tunnel test are given in Figure 1.

Figure 2. Architectural sections in two perpendicular directions

Seismic Input

The seismic input involves the site specific response spectrum for two levels as MCE level with 2% exceedance in 50 year; return period of 2475 years and a the ground motion inputs used in nonlinear response history analysis includes seven ground motions (MCE level with 2% exceedance in 50 year; return period of 2475 years) in pairs of X and Y directions. The ground motions are scaled so as to match the site specific response spectrum that has been obtained based on the spectral parameters for DBE and MCE level earthquake. The site specific response spectra are given in Figure 3. Additionally the basic characteristics of the ground motions are given in the Table 3 below.

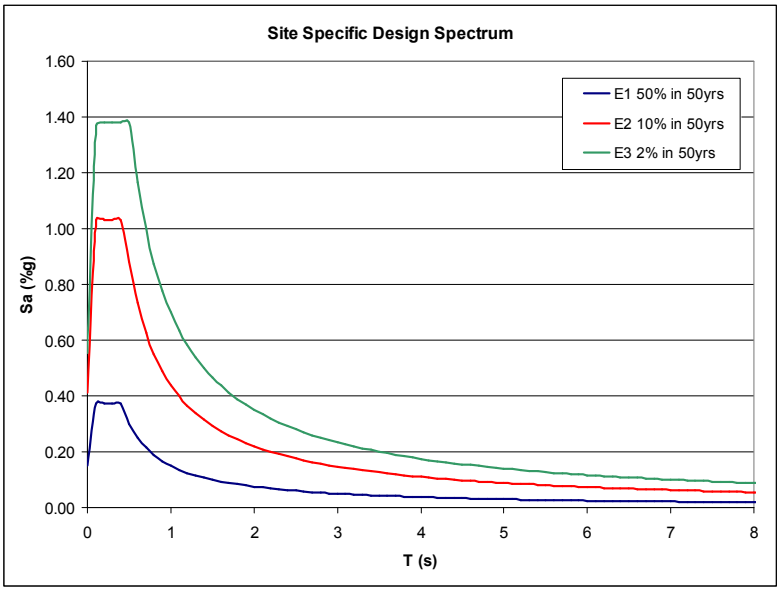

Figure 3. Site specific acceleration for DBE and MCE

Table 3. Ground motion parameters used in the analysis

EQ ID	Earthquake	Fault Mechanism	Station	Components	Distance (km)
1	1999, M=7.14 Duzce, Turkiye	Strike Slip	Lamont 362 Lamont	362 E 362 N	23.42
2	1999, M=7.14 Landers, USA	Strike Slip	CDMG 23559 Barstow	BRS000 BRS090	34.86
3		Strike Slip	USGS 5070 North Palm	NPS000 NPS090	26.84
4		Strike Slip	CDMG 1249 Desert Hot	DSP000 DSP090	21.78
5		Strike Slip	USGS 5071 Morongo	MVH000 MVH090	17.32
6	1999, M=7.4 Kocaeli, Turkiye	Strike Slip	ERD 99999 Arcelik	ARC000 ARC090	13.49
7	1979, M=6.53 Imperial Valley,	Strike Slip	UNAMUCSD 6604 Cerro	H-CPE147 H-CPE237	15.19

As it is indicated in both ASCE42 and ISDCTB 2008, the acceleration records in two perpendicular directions are applied simultaneously along the principal axes of the structure. The acceleration records are then rotated by 90° and the structure is reanalyzed.

Structural System and Modeling

The Performance Based Design and Analysis of the Renaissance Tower is done by using Perform 3D software by Computers & Structures, Inc. to generate a mathematical model capable of capturing the nonlinear behavior of the structure.

Basically the structural system is considered to be composed of two major subsystems such as "lateral load resisting system" and "gravity system". The lateral load system is composed of the RC core and Buckling Restrained Braces (BRBs) used as outrigger system. The gravity system is composed of flat slab system supported by the peripheral columns. The illustrations of both systems are given in Figure 4 and Figure 5.

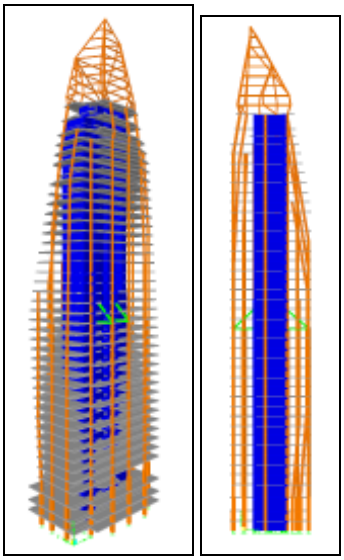

Figure 4. Mathematical model of the lateral load resisting and gravity system

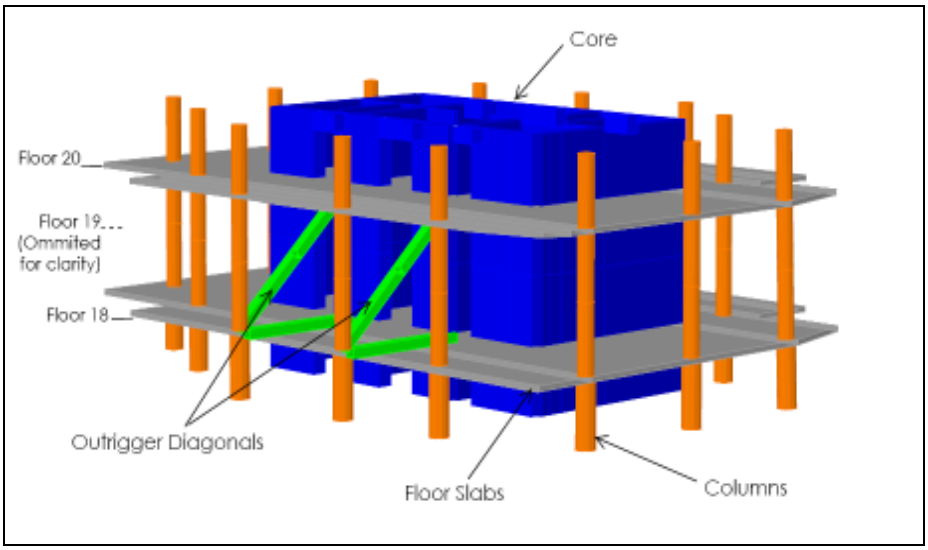

Figure 5. Detail of model of the lateral load resisting and gravity system model

The Geometric and numerical modeling considerations for this nonlinear model are in accordance with PEER Report 2010 and ASCE 41 as;

- Shear walls have been modeled as wall (shell) elements that can exhibit both flexural and shear deformation. The wall element is a 4-node rectangular finite element having both in-plane and out of plane bending stiffness as well as axial and shear stiffness.
- Wall elements have been assigned as fiber (distributed inelasticity) elements, which involves subdividing the wall section into concrete and steel fibers. This represents the stiffness, strength and deformation capacity of the element with reinforcements for nonlinear time

history analysis. Therefore, both nonlinear behaviors (i.e. cracking, yielding) in flexure and shear could be observed as realistically as possible.

- Strain gages are used in order to observe the amount of axial tension and compression strains in the shear walls.
- Buckling Restrained Braces are modeled as outriggers. Force-Deformation (Backbone) curves of these elements are obtained directly from the manufacturers and are assigned into the model.
- All gravity loads are applied as point loads on the walls and lateral columns.
- Floor mass is assigned as a lumped mass as well as a torsional mass on each floor diaphragm.
- Inelastic material properties (stress-strain curves) based on the Mander Concrete Model for both confined and unconfined concrete have been assigned. Rebar steel properties are based on the ASTM standards.
- Fixed boundary conditions are assigned at the foundation level of the structure.

Both material and geometric nonlinearities (P-Delta effects) are considered. Modeling parameters for the nonlinear behavior of link beams and shear walls are based on ASCE41. Seismic weight is calculated per ASCE41 by considering dead load and 25% of the live load.

In the mathematical model structural elements that contribute to the lateral load resisting system are considered. As such shear walls, link beams, outriggers, and lateral columns are modeled in Perform 3D model. Shear walls are modeled as shell elements whereas link beams, outriggers and lateral columns are modeled as frame elements

Acceptance Criteria for Performance Based Design

The main target of the performance based design is to obtain yielding in components that are reliably capable of a ductile response. Desirable modes of inelastic response of Renaissance Tower include the following;

- Flexural and shear yielding in structural steel link beams
- Flexural yielding in shear walls and conventionally reinforced concrete link beams with relatively slender proportions
- Yielding of outrigger elements (Buckling Restrained Braces)

Collapse Prevention performance limit is targeted for deformation controlled actions in Renaissance Tower under MCE level ground shaking according to PEER Guidelines and ISTBC 2008.

Shear walls in flexure, link beams and buckling restrained braces are assumed to respond with nonlinear behavior, allowing these elements to dissipate seismic energy. All other force controlled members are assumed to remain elastic. Force and deformation capacities are based on PEER Report, ASCE 41 and ISTBC 2008.

Global Acceptance Criteria

According to PEER Report, the mean of the absolute values of the peak transient drift ratios from the suite of analyses at each story level shall not exceed 3% with a maximum of 4.5% under MCE level evaluation. ISDCTB 2008 limits this value to 3.5%. Residual drift is also limited by PEER Report. The mean of the absolute values of residual drift ratios from the suite of analyses at each story shall not exceed 1 % and a maximum of 1.5%. This residual story drift ratio is intended to protect against excessive post-earthquake deformations that will likely result in excessive building down time as a result of needed repairs or even condemnation.

Acceptance Criteria at the Component Level

The performance of each component is determined based on deformation control actions as link bema rotations, shear wall strains and rotations. Both ISDCTB 2008 and PEER guidelines refer to ASCE 41 for plastic deformation limits for steel link beams. In order to determine the strain demand s strain gages are modeled at the edges of the wall elements for tension and compression in the walls. PEER Report and ASCE 41 recommends maximum concrete compressive strain in the unconfined concrete to be limited to 0.2% and confined concrete to be limited to 1.5%. Longitudinal reinforcement strain is also to be limited to 5% in tension and 2% in compression in order to suppress rebar buckling and fracture. Additionally in order to measure wall rotations, 4-node rotational gages are defined at the foundation level and just above the outriggers.

ANALYSIS PROCEDURE

In the "Istanbul Seismic Design Code for Tall Buildings" a 4 stage analysis procedure is proposed. Basically in the first stage a linear response spectrum is performed in order to determine the structural system in terms of stiffness and strength. In the next stage nonlinear time history analysis is performed based on nonlinear properties of the members determined in the first stage. The second stage acts as a verification of the preliminary design based on the deformation based acceptance criteria.

For the case of Renaissance Tower a series of nonlinear time history analysis are performed using the seven earthquake ground motion pairs provided in the seismic hazard report for MCE level evaluation with 2% probability of exceedance in 50 years. The seismic input is applied on the structural system in the two perpendicular directions are applied simultaneously along the principal axes of the structure. Subsequently directions of acceleration records are rotated by 90° and the analysis is repeated. Therefore seismic demands are calculated as the average of the results obtained from these fourteen analyses. The following load combinations are used in the analysis considering P-Delta effects. The combination used in performance evaluation of the system is defined as (DL + 0.25LL) +(1.0Ex + 1.0Ey) where Ex and Ey are two pair of the same earthquake time history data applied at 0° and 90° rotation.

ANALYSIS RESULTS

Based on the analysis procedure performed for all ground motions deformation and internal force demands are determined.

As the first demand parameters the Table 4 summarizes maximum displacement, base shear and overturning moments generated in the structure as a result of the 14 analyses under MCE level ground motions. The results consist of 7 pairs for which the maximum is given comparing the 0° and 90° rotation resultants.

Table 4. Displacement, base shear and over turning demands obtained from NLRHA

	Maximum Displacement (m)		Maximum Base Shear (kN), V/W (Ratio)		Maximum Overturning Moment (kN.m)	
Earthquake Pair (Max. of 0° & 90°)	**X**	**Y**	**VX**	**VY**	**MY**	**MX**
ARC000 ARC090	**1.81**	1.68	120,280 (13.2%)	130,180 (14.3%)	5,660,100	4,416,700
BRS000 BRS090	1.63	**1.61**	103,080 (11.3%)	130,280 (14.3%)	5,910,400	4,590,900
H-CPE147 H-CPE237	1.5	1.38	94,961 (10.4%)	140,445 (15.5)	5,864,600	4,324,400
DSP000 DSP090	1.4	1.32	88,504 (9.7%)	123,990 (13.7)	5,549,500	4,517,500
362 E 362 N	1.78	**1.49**	100,210 (11%)	154,190 (17%)	5,510,300	4,670,500
MVH000 MVH090	1.43	1.36	117,640 (12.9%)	148,760 (16.4)	5,856,500	4,420,900
NPS000 NPS090	1.45	1.7	98,800 (10.9%)	121,120 (13.3%)	5,911,500	4,417,000

The results of NLRHA is given in the following plots that represents story displacements, cumulative story shear, and drifts throughout the building. Additionally strains and of the structural walls and BRB elements are also represented between Figure 6 to Figure 10.

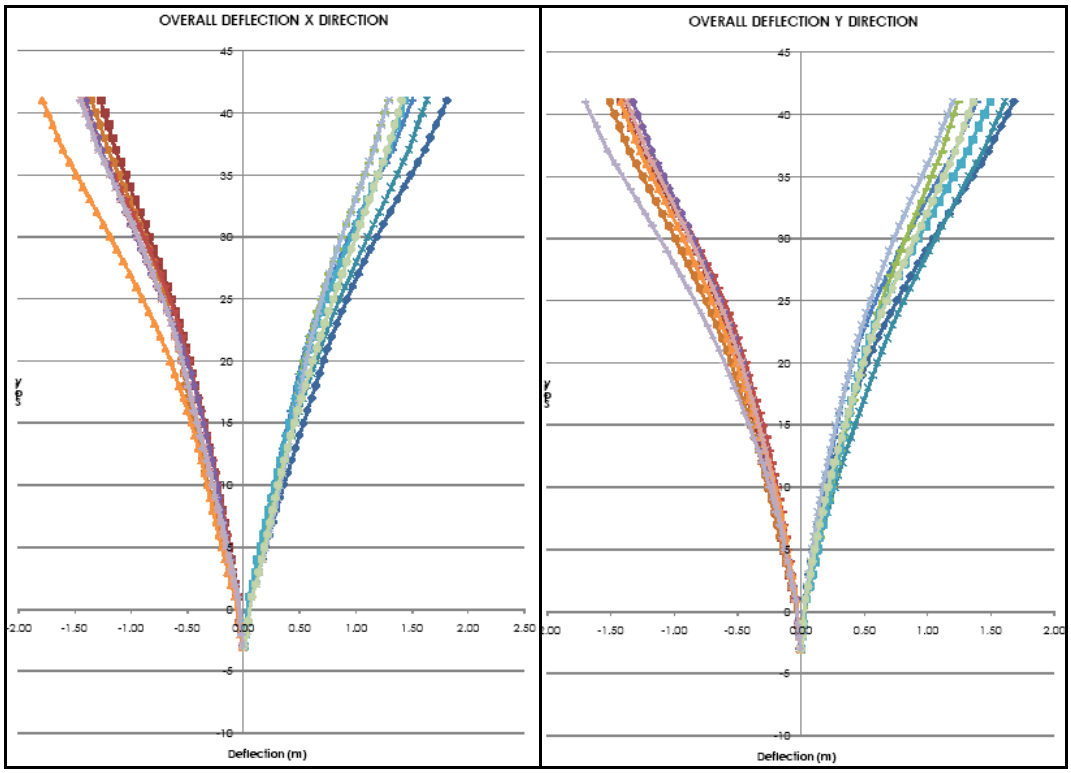

Figure 6. Overall Deflection in x&y-Direction under MCE Level Ground Motions

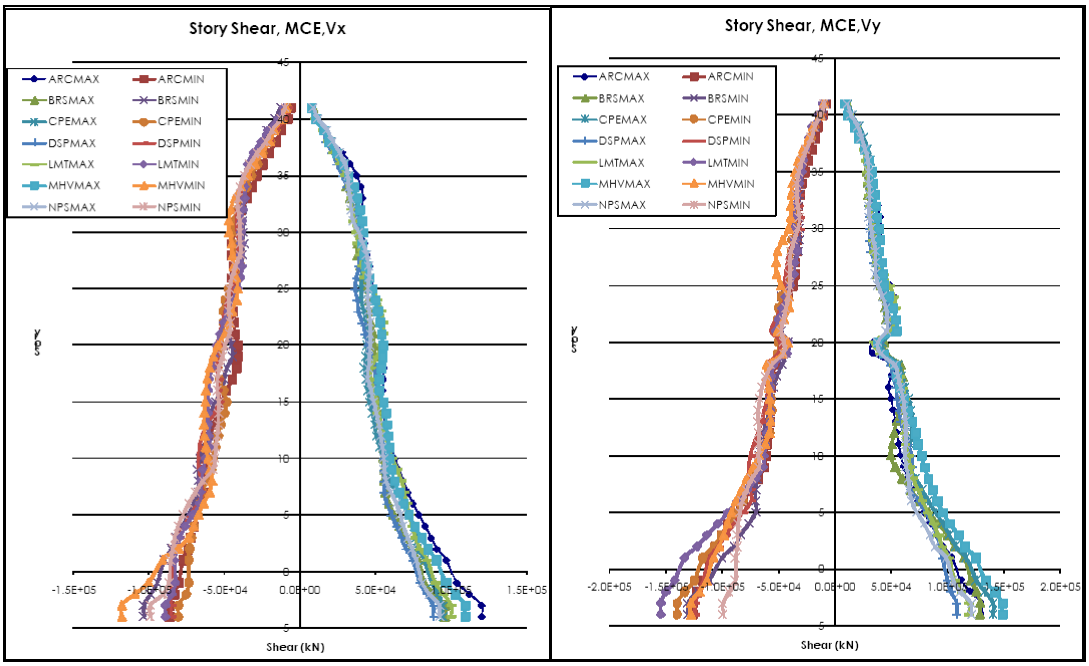

Figure 7. Story Shear Forces in x&y-Direction under MCE Level Ground Motions

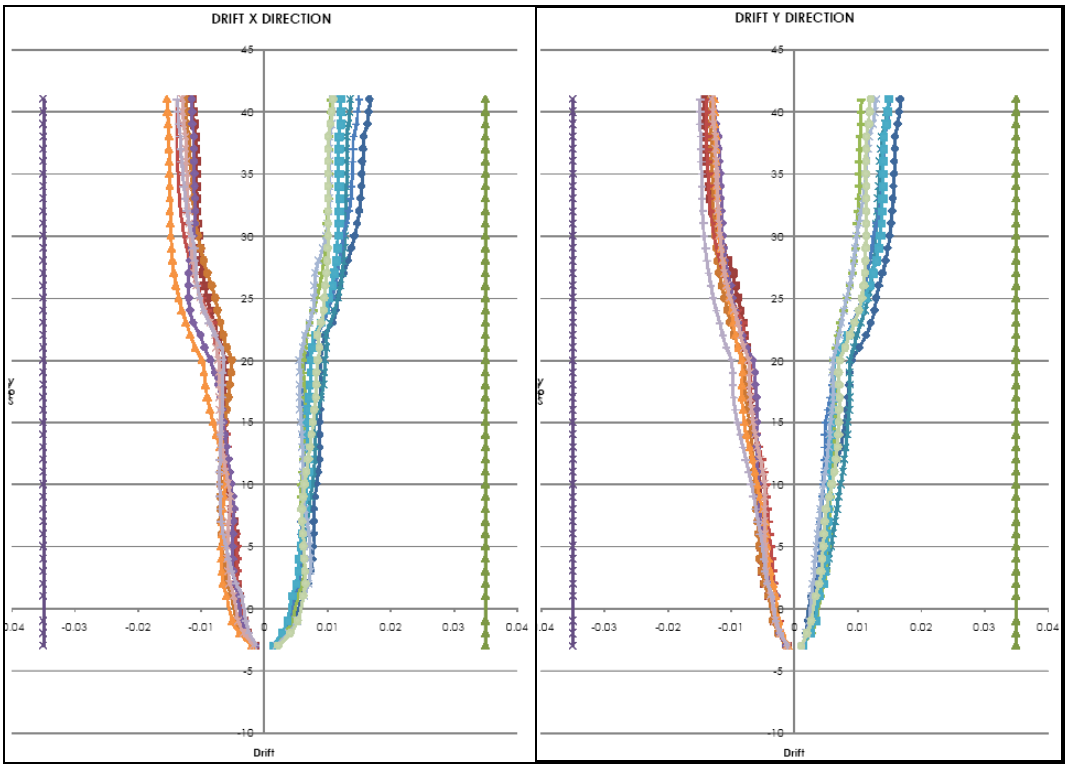

Figure 8. Drifts in x&y-Direction under MCE Level Ground Motions

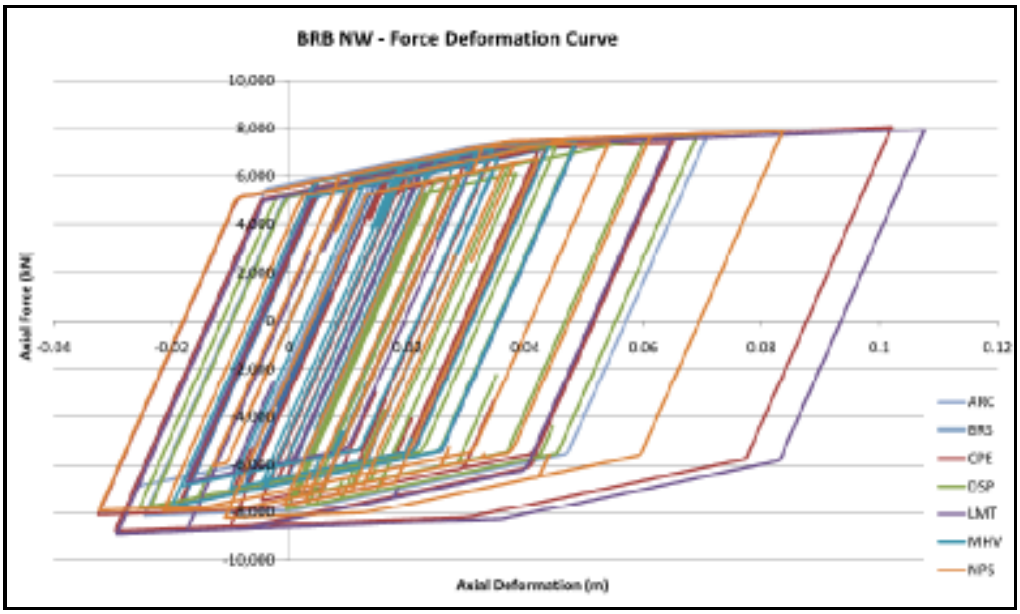

Figure 9. The Hysteresis Response of BRB - NW Element under MCE level ground motions

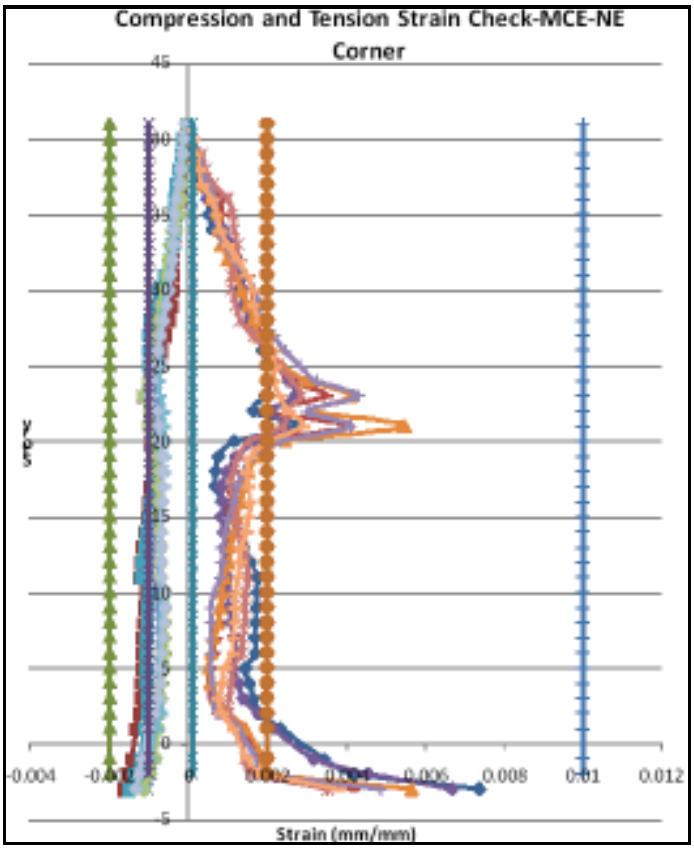

Figure 10. The compression and tension strain demands at NE corner under MCE Level Ground Motions

PERFORMANCE CONTROL

As the key issue of the Performance Based Design, in order to determine the overall performance level of the structural system, structural system and elements are checked by each performance criteria defined in PEER Report, 2010/05, ASCE/SEI 41/06 together with Istanbul Seismic Design Code for Tall Buildings (ISDCTB, 2008). The performance control is based on "global" and "component" level. In global level "building peak transient drift" and "residual drift"

demands are checked. In the component level “link beam rotations”, “shear wall strains”, “Buckling Restrained Braces axial deformations” and “shear wall rotations” for deformation controlled actions are checked. For force controlled actions “shear force in shear walls” and “axial force in lateral columns” are checked based on the acceptance criteria defined in relevant documents.

Global Acceptance Criteria

The evaluation of peak transient drifts is done by considering the maximum absolute vector value of the drift in each story from each of the analyses in the suite, rather than the mean of the maximum drift in the positive direction and the maximum drift in the negative direction taken separately, and rather than the drift along defined axes without consideration of drift in the orthogonal direction.

The mean absolute values of the peak transient drift ratio obtained from the nonlinear analysis under MCE level ground motions are well below the limits of 3% outlined in the PEER Report and 3.5% in the ISBCTB 2008. The mean absolute value of residual drift of 1 % as limited by the PEER Report is also met.

Table 5. Drift and Residual Drift demands obtained from NLRHA

Earthquake Pair (Max. of 0° & 90°)	**Peak Transient Drift (h%)**		**Residual Drift (h%)**	
	X	**Y**	**X**	**Y**
ARC000 ARC090	1.67%	1.67%	0.19%	0.18%
BRS000 BRS090	1.36%	1.47%	0.22%	0.17%
H-CPE147 H-CPE237	1.50%	1.50%	0.016%	0.086%
DSP000 DSP090	1.15%	1.27%	0.10%	0.19%
362 E 362 N	1.54%	1.48%	0.40%	0.20%
MVH000 MVH090	1.30%	1.30%	0.13%	0.058%
NPS000 NPS090	1.37%	1.48%	0.39%	0.25%

CONCLUSION

The most reliable and advance type of analysis methodology nonlinear response history analysis has been performed for the seismic design of Renaissance Tower. The most recent and state of the art methodology and performance evaluation has been applied.

All the design and analysis process is performed based on Istanbul Seismic Design Code for Tall Buildings (ISDCTB, 2008) and PEER Report, 2010/05, ASCE/SEI 41/06. Performance Based Design and analysis of the Renaissance Tower developed using performance based capacity design procedures has verified that the structure's performance will be acceptable when subjected to MCE level ground shaking.

The performance of the Renaissance Tower is evaluated in terms of local and global parameters are summarized as follows:

- The story drift limits has been a very effective indicator of the seismic response and it is used to result in efficient designs process. There is general consensus that up to threshold level of story drift, structures with proper yielding mechanisms and good detailing will perform without significant loss of strength and that properly attached nonstructural components will not pose a major life safety hazard.

- The residual drift ratio is intended to protect against excessive post-earthquake deformations that likely will cause condemnation or excessive repair downtime. This criterion adds enhanced performance for Renaissance Tower.
- The link beams are also very significant structural elements that have the effect on the response of the core system. Yielding of link beams in coupled wall systems is a major energy dissipating mechanism during severe seismic events. The analysis was repeated several times in order to optimize the link beam sizes so that they yield close to their rotational limits and therefore provide maximum energy dissipation.
- The outrigger system is used to control the lateral deformations in the core system. This system has provided significant amount ductility in the short direction of the building. The structure is balanced in terms of energy dissipation with the link beams in the long direction and with the BRB's in the short direction.
- The walls that make up the core system are of great importance on the lateral load resisting system. Since plastic hinges are expected to form in the shear walls, it is essential to observe the amount of rotation at these hinging locations. As the result of the limited rotations in the wall elements both the concrete and the reinforcing steel deforms close to their ultimate strain limits, one shall expect to see a small amount of rotation in the wall elements
- Shear failure in shear walls is considered a brittle failure type and must be avoided. Force controlled actions are divided into two types by PEER, critical and non critical actions.
- While the capacity assumptions are somewhat conservative, shear capacity are only exceeded in very limited areas. Although there is no thickness increase needed, some additional amount of shear reinforcement shall be added to the shear reinforcement presented on the drawings previously.
- In ISDCTB 2008 shear capacities of reinforced concrete elements are calculated using expected strengths without strength reduction factors. These calculated capacities are compared against demand without amplification, similar to non-critical actions as defined in the PEER Report.
- For Renaissance Tower, the given ground motions had been amplified by 15% as stated in the Seismic Hazard report. As such the possible uncertainty in the mean value response quantities were compensated for by Prof. Erdik et.al in the development of these ground motions. The designers indicated that preventing shear failure is critical and as such, a strength reduction factor Ø=0.75 has been used in our capacity calculations. This is unlike the requirements of ISDCTB 2008.
- Additionally, as significant tension is developed in the walls, shear capacities are calculated assuming strength contribution by the reinforcing steel only and calculated the maximum shear strength according to ACI 318.
- The lateral columns that support the outrigger system has subject to high level of axial loads. As such axial compression failure in reinforced concrete columns is a critical force controlled action so it has been avoided in the design process.

As in the case of all structural design process of high-rise buildings in high seismic areas the analysis and design process should be based on the “Performance Based Design” approach the involves the most recent advancers and methodologies in earthquake engineering field.

In Turkey a committee has been formed consisting of experts from academia and practicing engineers in order to finalize Turkish Seismic Code for “Seismic Design of Tall Buildings” which basically follows the PEER approach for Performance Based Design.

As the last point it should be emphasized that in addition to the state of the art design process performed in the Performance Design of tall buildings, such as Renaissance Tower, a set of regulations should also be created and followed for construction process.

REFERENCES

Assessment of Site Specific Earthquake Hazard for Renaissance Tower, Istanbul, (2011), M. Erdik, K. Şeşetyan, M.B. Demircioğlu, C.Tüzün, C.Yenidoğan

Structural Design Report of Renaissance Tower (2011), De Simone Consulting Engineers

Mander, J.B., Priestley, M.J.N., and Park, R. (1988b) "Theoretical stress-strain model of confined concrete." J. Struct. Eng.,114(8), 1804-1826.

Mander, J.B., Priestley, M.J., and Park, R. (1988a) "Observed stress-strain behavior of confined concrete." J. Struct. Eng., 114(8), 1827-1849.

Istanbul Seismic Design Code for Tall Buildings,(2008), Kandilli Observatory and Earthquake Research Institute Department of Earthquake Engineering,

American Concrete Institute, Building Code Requirements for Reinforced Concrete, 2008, ACI 318-08 (ACI 318)

Pacific Earthquake Engineering Research Center, Seismic Design Guidelines For Tall Buildings, 2010, (PEER) Relevant Portions