Performance-Based Design of 111 Main in Salt Lake City

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Courtesy of City Creek Reserve, Inc.

111 Main's reinforced concrete core wall system provides vertical and lateral support for an innovative 25-story office tower suspended over adjacent performing arts center.

ocated in a region of high seismicity in close proximity to the active Salt Lake Segment of the Wasatch Fault Zone, the new 111 Main office tower in Salt Lake City, Utah, comprises 501,455 square feet of Class A office space. The 25-story building rises 387 feet above grade and contains a penthouse roof-level steel hat-truss system with all perimeter columns suspended to allow for air-rights overhang at adjacent performing arts center. The overall project design challenges and solutions were described in STRUCTURE magazine, June 2016. This article focuses on the two stage performance-based seismic design methods undertaken by the design team during the project design development, independent peer review, and approval process. Designed to meet the minimum requirements of the 2012 International Building Code (IBC) and ASCE 7-10 provisions, the building superstructure construction incorporates a ductile reinforced concrete core wall system that exceeds the height limit of 160 feet per ASCE 7-10 Table 12.2.1. Thus, as a non-prescriptive alternate design method permitted by IBC Section 104.11, performance-based seismic design procedures were adopted following the guidelines of the Pacific Earthquake Engineering Research Center (PEER) Tall Building Initiative Guidelines (2010). The PEER TBI guidelines require that code equivalent or better performance is demonstrated at peak Maximum Considered Earthquake (MCE_R, 2% in 50yr, 2475 ARP) demands. Under construction, 111 Main is scheduled to be completed in August 2016.

Two Stage Performance-Based Design

In addition to the ambitious design challenge of hanging all 18 perimeter steel columns from penthouse roof trusses to allow an airrights overhang at the new 4-story performing arts center directly to the south of the tower, the project was driven by a fast-paced design and construction schedule to achieve project deadlines and commitments. During the Stage 1 procedure, final proportioning of reinforced concrete core wall design including thicknesses, openings, boundary zones, and link beams was achieved using simplified tri-directional linear response spectrum analysis and design. During the Stage 2 procedure, which included rigorous oversight by the Seismic Design Review Panel (SDRP), the team conducted tri-directional nonlinear response history analysis (NLRHA) to demonstrate that the structure design, determined in Stage 1, satisfied the performance-based inelastic design criteria of the PEER TBI guidelines.

Site-Specific Response Spectra and Ground Motions

Site-specific response spectra and ground motions for the Maximum Considered Earthquake (MCE_R) were developed per ASCE 7-10 requirements by the geotechnical engineer, URS, and the SDRP. The MCE_R spectrum was defined as the lesser of the deterministic and probabilistic MCE_R ground motions. To address the response of the hat-truss supported structure, vertical spectra was developed based on the median V/H ratios of Gölerce and Abrahamson, defining two sets



Figure 1. Structural systems description.

of conditioned horizontal and vertical response spectra. For the Stage 2 NLRHA procedure, two suites of seven sets of three-component ground motion time histories, spectrally scaled to the MCE_R spectra, were selected. Each of the seven ground motions were randomly rotated as recommended by SDRP.

Structural Systems Description

The superstructure typical framed levels consist of WF steel composite deck and slab construction with perimeter W14 columns as shown in Figure 1. The ductile reinforced concrete shear wall core construction includes 30-inch-thick walls extending from the top of the basementlevel pile-cap foundations to the underside of the trussed penthouse at Level 25. The core walls are configured on two grid lines in the east-west direction (62 feet 6 inches) and three grid lines in a northsouth direction (42 feet 9 inches) utilizing specified self-consolidating concrete with a strength of 8,000 psi except at Level 24, below the hat-trusses, where 10,000 psi is needed. Perimeter column loads representing approximately 40% of the building gravity dead and live loads are transferred from the roof hat-trusses to the top of the core walls via six articulated spherical structural steel bearings. The core wall loads are transferred to a deep foundation system consisting of driven steel HP-piles extending to depths of 100 feet and greater below grade, with a total of 373 HP14 piles.

Stage 1: 3D Linear Analysis and Design

The design team was challenged to use linear dynamic modal response spectrum analysis (MRSA) during a 10-week design development phase to finalize the proportions, design, and quantities of the ductile reinforced concrete bearing wall system. These details were needed prior to building a 3D-nonlinear analysis model to demonstrate compliance with the performance-based design PEER TBI MCE_R level acceptance criteria. The two-stage strategy included a Stage 1 linear MRSA and a Stage 2 nonlinear analysis verification of the Stage 1 results which would occur during the final construction document design phase.

A 3-dimensional ETABS by Computers and Structures, Inc. (CSI, v2013) linear analysis model was developed for the MRSA with 5% modal damping. The structure seismic force resisting system (SFRS) was modeled and included the complete gravity and lateral structural system load path from foundations (pinned at basement level), basement walls, Level 1 diaphragm, core walls, hat-trusses, floor framing, hanging perimeter columns, and structural bearings at top of core walls below Level 25 trusses. Stiffness property modifiers were used as recommended in PEER/ATC 72-1 (2010) for the concrete lateral and gravity elements to account for cracked section



Figure 2. New and modified core wall openings to reduce stress concentration and increase ductility.

Element	EI	GA
Basement Wall	0.20	0.12
Core Shear Wall	0.50	1.00
Link Beam	0.15	1.00
Level 1 Diaphragm	0.20	0.12
Tower Diaphragm	0.50	0.50

Table 2. R-Factors at MCE_R level used in Stage 1.

Element	R _x and R _y	R _z
Link Beam	3.5	1.0
Flexural in Shear Wall	3.5	1.0
Shear in Shear Wall	2.0	1.0
Steel Roof Truss	2.0	1.0
Steel Hanging Column	2.0	1.0
Foundation System	2.5	1.0

Table 3. Expected material properties (ϕ =1.0).

Material	Expected Strength
Reinforcing Steel in Concrete	1.17 f _y
Concrete	1.3 f' _c
ASTM A992/A572 Steel	1.1 F _y

properties during a seismic event as summarized in *Table 1* with additional conservative assumptions made for the shear stiffness of core wall and link beam elements, as well as the lower bound stiffness for the ground level diaphragm.

To estimate demands at MCE_R level using Stage 1 MRSA, more conservative values of ductility based system response modification R-factors were utilized for the bearing wall system than prescribed by code minimum (ASCE 7-10) requirements (R=5 at DE, $\frac{2}{3}$ MCE_R). These values, as summarized in *Table 2*, were based on recent research summarized in NEES webinar, Performance, Analysis and Design of Flexural Concrete Walls by Lehman and Lowes (2013). The research included a ¹/₃ scale testing program with wall specimens detailed with varying parameters per ACI 318-11 requirements, correlated with FEMA P695 probability based collapse prediction modeling. The research concluded that R-factors for various high rise concrete wall systems to achieve 20% probability of failure at MCE_R was about 3.5, which is significantly lower than the ASCE 7-10 value and the shear demand in core wall should be increased by a flexural overstrength factor and dynamic amplification factor. A simplified approach was utilized for 111 Main using R=3.5 for deformation controlled actions and R=2 for force controlled actions. At MCE_R level demands, expected material properties and capacity reduction factor, $\phi = 1.0$, were assumed as summarized in Table 3.

Since IBC 2012 and ASCE 7-10 do not explicitly address the directional combinations for vertical seismic load, the ASCE 4-98 (2000) Section 3.2-26 standard for the *Seismic Analysis of Safety-Related Nuclear Structures* was referenced for defining the combined horizontal and vertical seismic load combinations. Combining with PEER TBI

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(2010) Section 7.6.1, the tri-directional seismic load combinations used in Stage 1 analysis at MCE_R MRSA were as follows, where L_{exp} is the expected live load (25% of the unreduced live load):

- 1) $1.0D + L_{exp} \pm 1.0E_x \pm 0.4E_y \pm 0.4E_z$
- 2) $1.0D + L_{exp} \pm 0.4E_x \pm 1.0E_y \pm 0.4E_z$
- 3) $1.0D + L_{exp} \pm 0.4E_x \pm 0.4E_y \pm 1.0E_z$

Following coordination of core wall openings with the Architecture/MEP design team, and preliminary analyses indicating high wall and link beam shear stress concentrations, the structural design team introduced additional new and modified core wall openings to reduce stress concentrations at story stiffness and strength transitions as shown in *Figure 2*. The additional link beams introduced by these openings significantly increased the ductility of the structure while balancing the stiffness along parallel core wall grid lines.

Stage 2: 3D Nonlinear Analysis and Design

With the development and design of the SFRS determined from Stage 1 linear elastic procedures, a Stage 2 nonlinear analysis procedure was used to evaluate the design using the PEER TBI procedures and acceptance criteria during the project construction document phase. Establishing the project specific procedures and criteria included the collaborative recommendations of the SDRP in the final verification, design, and detailing of the structure. The SDRP input included additional performance checks consistent

with the intent of the PEER TBI guidelines. The guidelines provide an alternative to the prescriptive procedures for seismic design in ASCE 7 with the intent to permit the design of buildings using non-prescriptive systems of equivalent performance that are capable of achieving the seismic performance objectives of Occupancy Category II buildings. The guidelines consider the inelastic response of the structure's global behavior and element components using NLRHA at the MCE_R level collapse prevention performance objective. The 111 Main acceptance criteria included consideration of bounded basement level backstay effects, inter-story and residual drift limits, as well as consideration of force and deformation controlled element component performance in the SFRS at the MCE_R. Serviceability at the 43-year earthquake event was also considered, but spectral demands were significantly less than those from the MCE spectrum reduced by the Stage 1 response modification factors.



Figure 3. PERFORM-3D (CSI) nonlinear analysis model.

Therefore, the structure remains essentially elastic and satisfies the service-level immediate occupancy performance objective.

Stage 2 Nonlinear Analysis Modeling

The 3-D nonlinear analysis model using PERFORM-3D (CSI, v2011) is shown in Figure 3. The model included the core walls, all hanger columns and hat truss elements comprising the SFRS and load path in resisting both horizontal, and vertical seismic loads. At typical steel framing levels, masses were lumped at perimeter columns and core wall locations, with rigid diaphragms at Levels 2 to 24. The model was pinned at a basement foundation level with no soil support springs modeled at basement walls. Concrete core walls were modeled using the PERFORM-3D Inelastic Shear Wall element using two vertical steel and concrete fibers per panel zone element and four fibers per each boundary zone element. Coupling beams were modeled using five different models, depending on reinforcement arrangement and aspect ratio, utilizing both conventional and diagonally reinforced beams. Model material properties were based on most recent recommended research and testing results.

Performance Studies

Some significant performance studies were conducted in assessing the structure and model before commencing with the NLRHA to address questions posed by the SDRP. These studies included hat-truss gravity load effects on the concrete core ductility and strength,

cracking vs. yield moment core capacity, displacement-based design concepts assessing the distribution of vertical reinforcement ratio, and coupling beam strength vs. core flexural strength checks over the height of the core wall structure. *Figure 4* illustrates nonlinear core wall behavior using a core cross section fiber model (XTRACT) in consideration of global axial-moment and moment-curvature effects demonstrating a substantial increase in moment strength due to additional compressive loads from the hat-truss. Cracking vs. yield moment capacity at upper levels with low flexural reinforcement ratio was also considered.

Stage 1 & Stage 2 Summary Results

Representative summary analysis results show good correlation in comparison of Stage 1 (MRSA) and Stage 2 (NLRHA) procedures.



MAX INTERSTORY DRIFT RATIO



Figure 5. Summary Stage 1 & Stage 2 interstory drift demands.

While predicted MCE_R inelastic inter-story drift ratio demands (Figure 5) are similar, the nonlinear analysis of Stage 2 illustrates a more uniform redistribution of peak pier wall shear stresses. These global summaries, as well as additional Stage 2 results including limits on wall strains at confined and unconfined concrete, coupling beam rotations and residual drift demands, demonstrate full conformance with the project design criteria and with the performance intent of ASCE 7-10 for Occupancy Category II buildings.

Conclusions

Faced with an accelerated design and construction project schedule, the SOM structural design team developed a two-stage performancebased methodology which required close collaboration with the SDRP to achieve performance objectives and meet project milestone deadlines. While utilizing a simplified Stage 1 linear MRSA procedure, the two-stage methodology allowed for early coordination of the seismic force-resisting system with architecture and MEP design teams without compromising the quality of structural solution delivered in the final design. The simplified Stage 1 approach can be used in concept and schematic design of tall core wall buildings in high seismic regions to assess design feasibility quickly without the need for complex nonlinear analysis. The reader is referred to a more detailed summary of this twostage performance-based design approach in SEAOC 2015 Convention paper, by the authors, titled, Performance-Based Design of 111 Main.

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