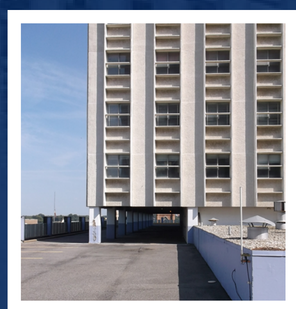
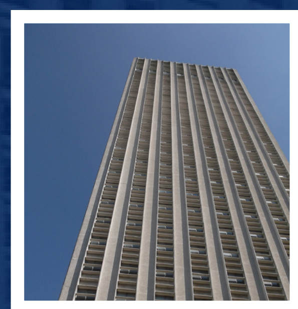
ASCE 31 SEISMIC ASSESSMENT
100 N MAIN STREET
MEMPHIS, TENNESSEE

October 2, 2013

2650 Thousand Oaks Blvd, Suite 3200
Memphis, TN 38118

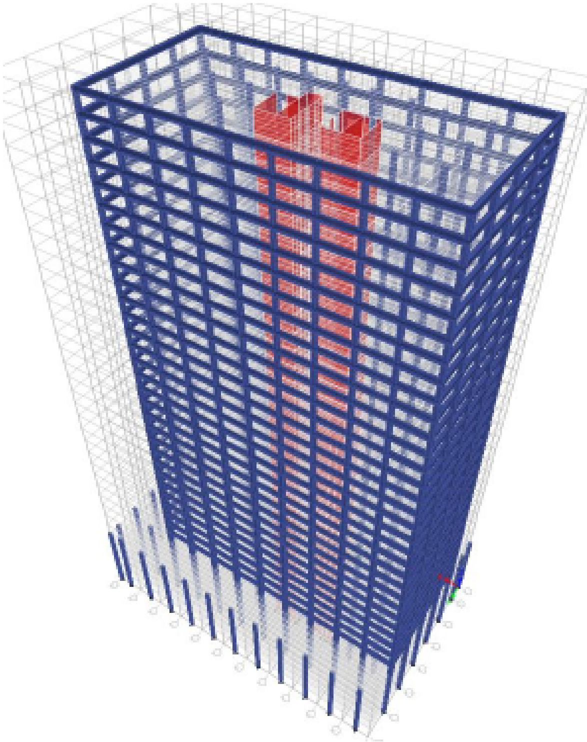
SSR Smith Seckman Reid, Inc.

Smith Seckman Reid, Inc.
2650 Thousand Oaks Blvd
Suite 3200
Memphis TN 38118
www.ssr-inc.com

TABLE OF CONTENTS

ASCE 31 SEISMIC ASSESSMENT
100 NORTH MAIN STREET
MEMPHIS, TN

100 N Main Exterior



3D Model of Podium & Tower

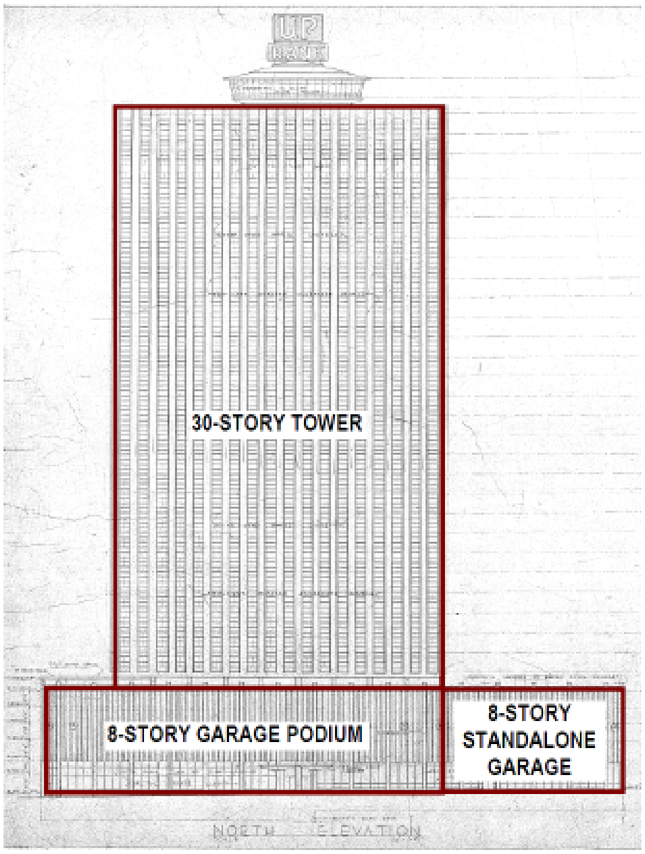
1. EXECUTIVE SUMMARY

Smith Seckman Reid (SSR) was retained by One Hundred North Main, LLC to conduct a structural evaluation for determining the expected seismic response of the 38 story office tower at 100 N. Main Street in Memphis, TN. This evaluation was performed in accordance with *ASCE 31-03: Seismic Evaluation of Existing Buildings*. A Tier 1 evaluation for an expected Life Safety performance level was conducted. Our evaluation included basic field observations to note the general status of the structure, it's compliance with the construction documents, and to identify any seismic deficiencies not shown on the plans.

The results of the Tier 1 ASCE 31 study presented in this report identify numerous items that either require retrofitting or more detailed investigation to determine the magnitude of the deficiency. SSR recommends that the owner proceed with a more detailed seismic study and development of the conceptual seismic retrofit scheme. This scheme would involve the addition of concrete shear walls to the garage structure, base isolation of the tower columns, and perimeter shear walls to tower. A more detailed investigation and design would provide the owner with an order of magnitude cost estimate to aid in the decision making process.

2. BUILDING DESCRIPTION

100 North Main Street is a 38 story reinforced concrete structure constructed in 1963. The building consists of a 30 story tower on top of an 8 story parking garage / mixed use podium. The 8 story structure consists of two structurally independent buildings separated at the western most column of the tower with double columns and a 1" expansion joint. The garage only 8-story structure has plan dimensions of 110'x140'. The 8-story podium has plan dimensions of 220'x140', and the towers dimensions are approximately 182'x87'. On the primary roof level (floor 38, 400 feet above ground level), there is a circular, steel framed roof restaurant and 25' tall 4-sided sign structure with antennae and telecommunications equipment mounted to it. Typical story heights are 9 feet for the garage and 11 feet for the tower. The total square footage of the building is approximately 840,000 ft^2.



Building Naming Convention

CONSTRUCTION DOCUMENTS

We reviewed the architectural & structural drawings for the building's construction. The construction drawing set was complete. Specifically, we reviewed the following drawings:

> Robert Lee Hall & Associates Architects, Ellers & Reaves Structural Engineers, titled "100 NORTH MAIN BUILDING"; sheets A1-A38, S1-S17, dated 07/23/1963.

The building was designed under the Memphis Municipal Code. As stated on the structural sheet S2, the building was designed for a wind pressure of 20 psf at the base increasing to 30 psf at the roof. This results in a total lateral design load of 2050 kips in the N-S direction and 970 kips in the E-W direction.

GRAVITY LOAD SUPPORT SYSTEM

The gravity system for the building is a 8" thick concrete flat slab framing to concrete columns that range from 20" square to 30" square on a 20' grid. 12" to 18" thick concrete shear walls create the elevator and stair shafts interrupting the columns grid. The perimeter of the tower has 16"x33" beams framing between exterior columns that provide anchorage points for the precast concrete curtain wall. The garage levels have 9" thickened slabs or drop capitals around the columns.

LATERAL LOAD RESISTING SYSTEM

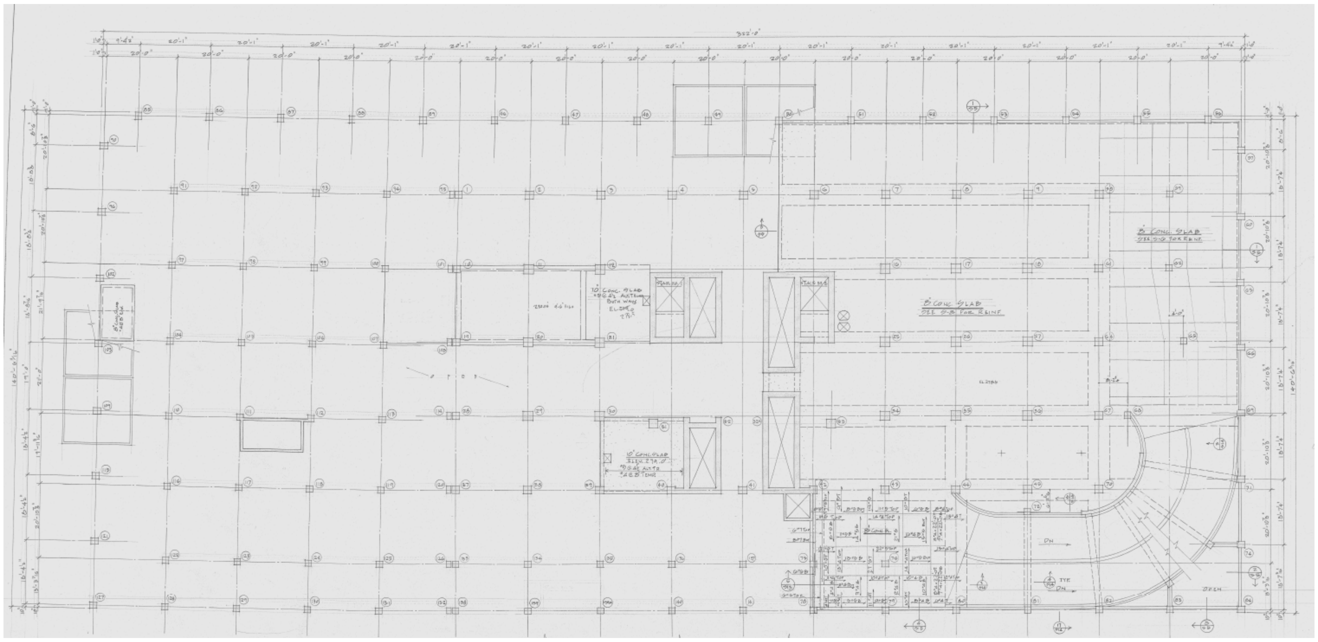
As stated on the general notes on page S2 of the original drawings, the "structural frame of the main building is designed to share lateral loads with elevator / stair shafts". This creates a dual lateral force resisting system combining the flat slab & column moment frame with shear walls near the center of the tower. Neither the concrete moment frame nor the concrete shear walls are detailed for the ductile performance required to accommodate expected building drifts from the current design magnitude earthquake.

FOUNDATION

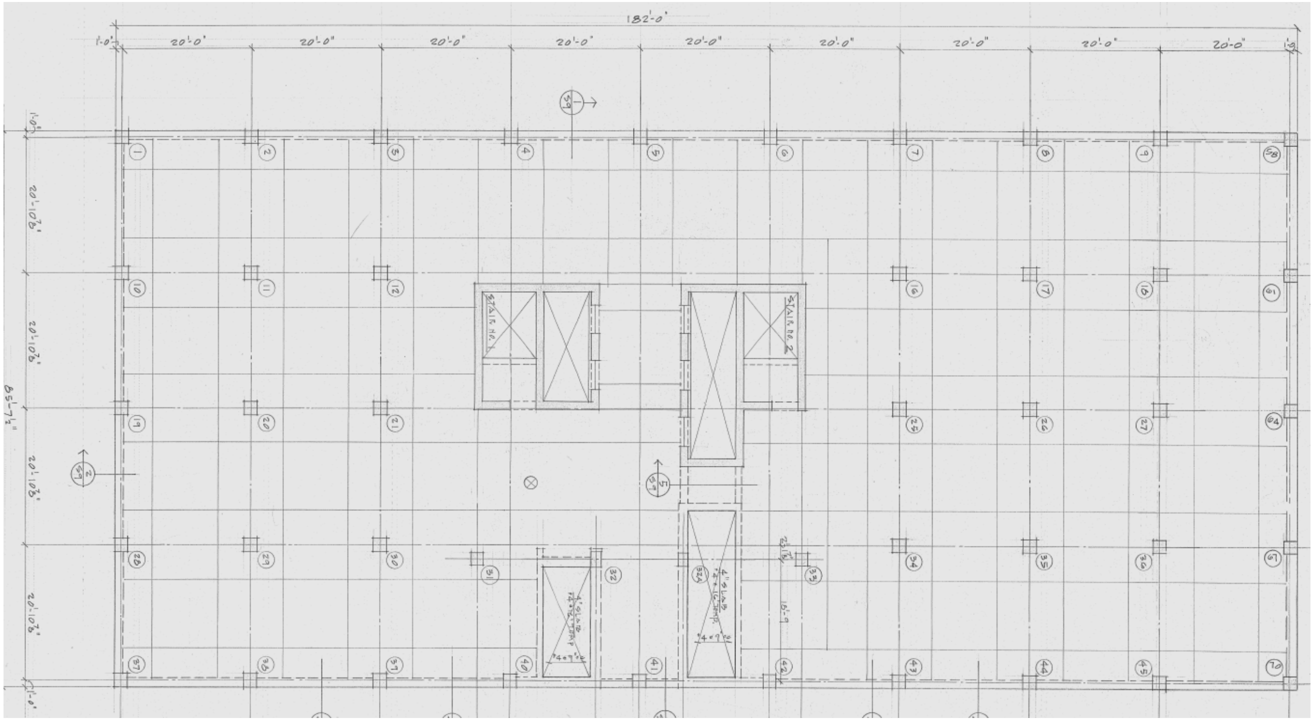
The foundation consists of driven 14" square pre-stressed concrete piles driven 35' to 45' deep for an allowable capacity of 140 tons. Pile caps range from 3'-6" to 6'-4" thick, with wall caps or caps for multiple piles containing top reinforcing steel in addition to bottom steel.

CLADDING

The exterior of the tower is precast panels with exposed marble aggregate. They are attached via wedge inserts to the perimeter beams. The garage level precast panels span multiple floors and attach to bolts embedded in the flat slab.



Garage Level Framing Plan



Typical Tower Framing Plan

3. FIELD OBSERVATIONS

Andy Kizzee of SSR conducted a visual inspection of the building on September 17, 2013. The visual survey confirmed that the building construction generally conforms to the information provided on the construction documents. The tower portion of the structure is generally in good condition and no signs of structural deterioration, abnormal concrete cracking, or foundation settlement were visible. The garage portion of the structure, being exposed to the elements and cars continuously for nearly 60 years, shows significant signs of deterioration. Concrete has delaminated or spalled off exposing rusting reinforcing at walls, columns, slabs, and beams on every level. Various repair methods are visible at many of these locations, with varying degrees of success. Please refer to Appendix D for a more detailed description of the garage deterioration. We recommend that the more detailed analysis and general repair suggestions outlined for the garage in Appendix D should be undertaken whether or not the building is upgraded for seismic.

Base of Tower at 8th Level Roof

1" Expansion Joint

Typical Parking Level

Typical Flat Slab at Interior Column

4. ASCE 31: TIER 1 SCREENING PHASE

ASCE 31-03: Seismic Evaluation of Existing Buildings is the nationally recognized standard which uses a three-tier process to assess the seismic hazards for existing buildings. The goal is to identify any vulnerabilities in a buildings lateral force resisting system that could lead to significant failure or collapse of a building. The Tier 1 procedure (performed for this study) is a preliminary screening to identify potential structural and non-structural seismic deficiencies. The Tier 1 procedure uses a series of checklists addressing structural, non-structural, and site hazards. Each item is evaluated and assigned as "C – Compliant" or "NC – Not Compliant". Compliant items are acceptable, while Not Compliant items require further investigation and possibly retrofit.

LIFE SAFETY PERFORMANCE

The evaluation of 100 N. Main is based on a Life Safety (LS) performance level defined as

"After a design earthquake significant damage to the structure but some margin against partial or total structural collapse remains. Some structural elements and components are severely damaged but this has not resulted in large falling debris hazards. Injuries may occur, however the overall risk of life threatening injury as a result of structural damage is expected to be low. It should be possible to repair the structure, however for economic reasons this may not be practical."

In summary, Life Safety performance is intended to prevent loss of life during the design level earthquake. The Tier 1 procedure for 100 N. Main identified numerous items that are Non-Compliant for Life Safety Performance.

DESIGN ACCELERATIONS

The design level earthquake considered in the ASCE 31 procedures is 2/3 of the ground acceleration expected by an earthquake with a recurrence interval of 2,500 years. Probable ground accelerations are based on data from the 2008 United States Geological Survey (USGS) hazard maps and are adjusted by site soil parameters. The 100 N. Main site was classified as site class D based on previous geotechnical investigations at nearby buildings. The results are a short period acceleration (S_{DS}) of 0.737g and a long period acceleration (S_{D1}) of 0.397g (See Appendix C). Per ASCE 31 guidelines, the site is classified as having a High level of seismicity.

STRUCTURAL ANALYSIS

Due to the height of the tower, its flat slab moment frame system, combined shear wall and moment frame lateral system, and large vertical irregularity, SSR constructed basic 3D computer models of the standalone garage and tower. The factors listed above are generally not accounted for in the rough calculations normally performed for an ASCE 31 Tier 1 analysis. These factors created inaccurate estimations of the natural frequency (period) and expected lateral deflection of each structure. The period of the structure directly correlates to the amount of lateral load to be resisted by the building. The amount of lateral drift determines the level of damage the building sustains. The results from the 3D structural models are below.

Structure	Period (sec)	Lateral Drift	Drift Ratio
8 Story Garage	2.16	2'-8"	4%
Tower	4.68	4'-10"	1.20%

The flat slab moment frame of the 8 Story standalone garage causes a much greater period than the ASCE 31 equations, lowering the design forces. But it also leads to a drift ratio of more than double the recommended amount. The concrete is not detailed to remain elastic at this drift limit and will experience punching shear failures.

The combined moment frame and shear walls system of the tower leads to a period that is 50% smaller than predicted by the ASCE 31 equation, increasing the lateral loads to be resisted. The drift is an acceptable range, but the stresses in the shear walls and concrete columns are greater than allowed (see checklists in Appendix A and calculations in Appendix B).

BUILDING TYPE & CHECKLISTS

The building is primarily classified as Building Type C1: Concrete Moment Frame. Due to the combined lateral force resisting system, the checklists from Building Type C2: Concrete Shear Wall were also used. A Tier 1 Evaluation of this building involved completing the following checklists:

3.7.9	Basic Structural Checklist
3.7.9S	Supplemental Structural Checklist
3.8	Geologic Site Hazards and Foundations Checklist
3.9.1	Basic Nonstructural Component Checklist
3.9.2	Intermediate Nonstructural Component Checklist

See Appendix A for the completed checklists. The comments next to each item provide a brief explanation of the building features that make it Compliant or Not-Compliant. In some cases, how the proposed retrofit will address or correct this deficiency is identified.

STRUCTURAL DEFICIENCIES

The structural deficiencies identified in this study are as follows:

- Adjacent Buildings: The 1” expansion joint between the 8-story garage and the tower is not large enough to accommodate expected deflections.
- Geometry: The tower is set back on 3 sides from the edge of the garage podium, resulting in a large vertical irregularity.
- Torsion: In the garage podium beneath the tower, and in the tower itself the distance between the story center of mass and the story center of rigidity is greater than 20% of the building width due to the un-symmetric layout of the elevator and stair walls. This results in a torsional irregularity causing the building to twist under lateral loading.
- Deterioration of Concrete: The garage sections display excessive concrete deterioration.
- Shear Stress (Shear Walls): The shear stresses in the concrete shear walls, calculated using the Quick Check procedure, exceeds allowable stresses at the 24th level where the number of walls is reduced.
- Axial Stress Check (Columns): The axial stresses in the tower columns, calculated using the Quick Check procedure exceeds allowable stresses at the tower base
- Drift: The calculated drift calculated using the Quick Check procedure is 6% for the 8 story garage and 11% for the tower. The recommended maximum is 2%
- Flat Slab Frames: 8” flat slab framing into concrete columns is prone to punching shear failure.
- Column Bar Splices: The slices of the smaller column reinforcing bars are too short.
- Uplift at Pile Caps: Pile caps for single columns do not have top reinforcing.

GEOLOGIC SITE HAZARDS AND FOUNDATION DEFICIENCIES

- Ties Between Foundation Elements: There are no ties between pile caps, which could result in differential settlement.

NONSTRUCTURAL DEFICIENCIES

The existing building has numerous non-structural deficiencies because it was constructed before seismic design was performed in the Memphis area. Because the building is going to be completed renovated, including mechanical and electrical equipment, ceilings, windows, etc. we have not indicated deficiencies related to those items that will be replaced. However, it is intent of the project architect to leave the precast panels on the tower in place and only replace the windows. Below is a list of deficiencies related to the precast curtain wall.

- Cladding Isolation & Multistory Panels – The precast cladding is attached to the concrete frame using wedge inserts. These relatively rigid concrete panels and their connections will not be capable of accommodating the recommended 1% drift limit (1'-3" for the 11'-0" story height) for panel connections.

5. CONCLUSIONS

Based on our Tier 1 evaluation, we have determined that the 100 North Main Tower & Garage Building does not meet the assigned performance goal of Life Safety. According to ASCE 31 Standards, we would normally recommend that the building go through a more detailed analysis to further explore the deficiencies identified in this ASCE 31 Tier 1 study. This more detailed analysis would provide numerical results identifying how overstressed the deficient elements of the building are. However, considering planned scope and current state of the project, we propose that the detailed structural analysis revealing the specific overstress ratios of structural elements be substituted by going directly to the design of a conceptual seismic retrofit design.

A quick estimate of how overstressed the lateral load resisting system is can be obtained by comparing the approximate seismic base shear from the Vertical Distribution of Seismic Forces calculation on the first page of Appendix B. This sheet tallies a total base shear of 9,134,000 lbs. Comparing this to the design wind load of 2,050,000 lbs in the N-S direction, and 970,000 lbs in the E-W direction, we see that the seismic design loads required to be resisted by the building are five to ten times larger than the design wind load.

Based on recent high profile renovation projects in downtown Memphis, a seismic retrofit may possibly be required by insurers or lenders prior to renovation. We think that the owner would benefit by knowing the approximate cost for seismically retrofitting the building. This cost estimate of the retrofit scheme will provide an approximate dollar amount that can be weighed against the inherent risk associated with the building in its current un-retrofitted state. It will also enable the owner to make better informed decisions during this early stage of project development.

Per SSR's original proposal, in coordination with Simpson Gumpertz and Heger from San Francisco, we would design a conceptual seismic retrofit. Primary elements of this proposed retrofit scheme include:

- Addition of shear walls to the standalone garage and podium. This will greatly stiffen the structure bringing the drifts down to an acceptable level.

- Base isolation of each of the 45 tower columns at the 8th level (roof of the garage). These base isolators will reduce the seismic load by approximately half.

- To further reduce deflection and add strength to the tower, punched window shear walls would be added to reduce the deflections down to an acceptable level. These walls would be installed between the exterior columns.