DEPT OF BLDGS 121187143 Job Number

ES589099645 Scan Code

# STRUCTURAL PEER REVIEW STATEMENT

This structural peer review and report, dated 20 March 2015, is complete for the superstructure and the foundation submission.

**Structural Peer Reviewer Name:** William J. Faschan
Leslie E. Robertson Associates

**Structural Peer Reviewer Address:** 40 Wall Street, FL 23
New York, NY 10005

**Project Address:** 401 Ninth Avenue, New York, Block #729, Lot 60

**Department Application Number for Structural Work:** #121187143

**Structural Peer Reviewer Statement:**

I, William J. Faschan, am a qualified and independent NYS licensed and registered engineer in accordance with BC Section 1627.4, and I have reviewed the structural plans, specifications, and supplemental reports for 401 Ninth Ave, Block #729, Lot 60, Application #121187143 and found that the structural design shown on the plans and specifications generally conforms to the superstructure and foundation and structural requirements of Title 28 of the Administrative Code and the 2008 NYC Construction Codes. The Structural Peer Review Report is attached.

**New York State Registered Design Professional**
(for Structural Peer Reviewer)

Name William J. Faschan

Signature

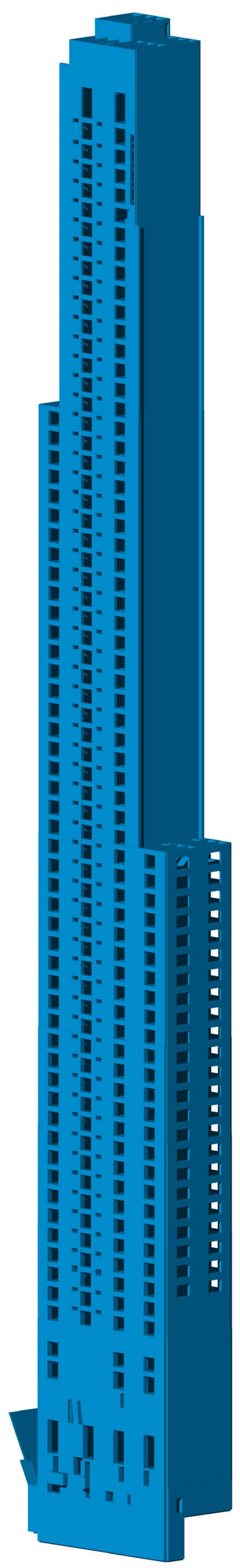
Date 03/20/15

Cc: Project Owner: John Durschinger
Project Registered Design Professional: Charles Besjak

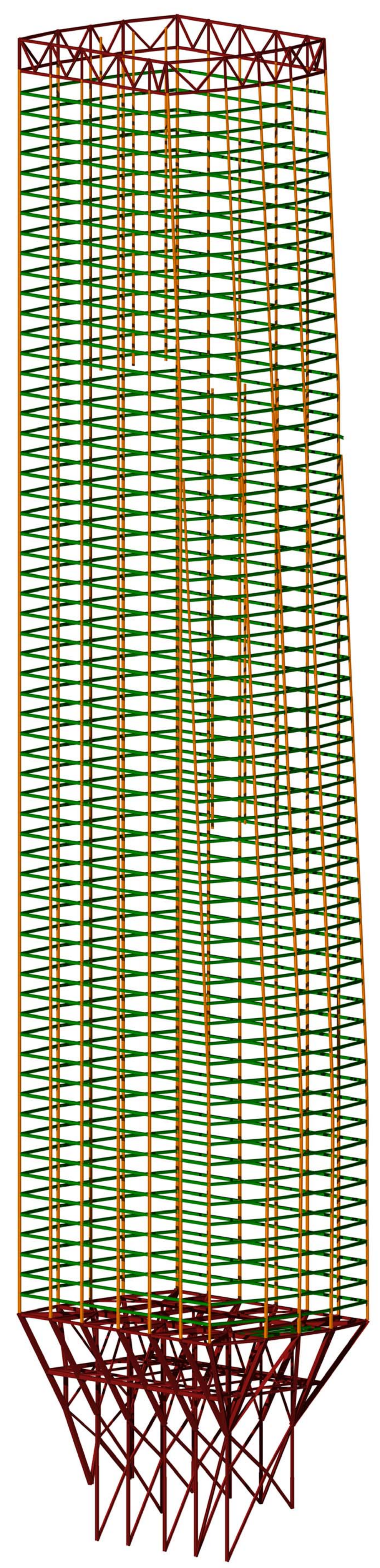
# APPENDIX A

## MANHATTAN WEST: NORTH TOWER PEER REVIEW

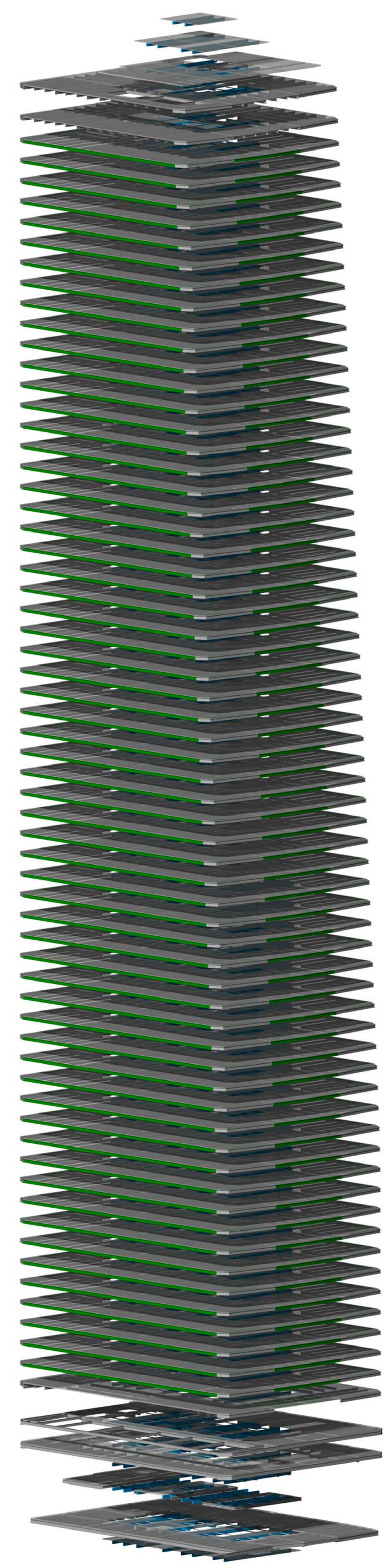
### STRUCTURAL DRAWING LIST



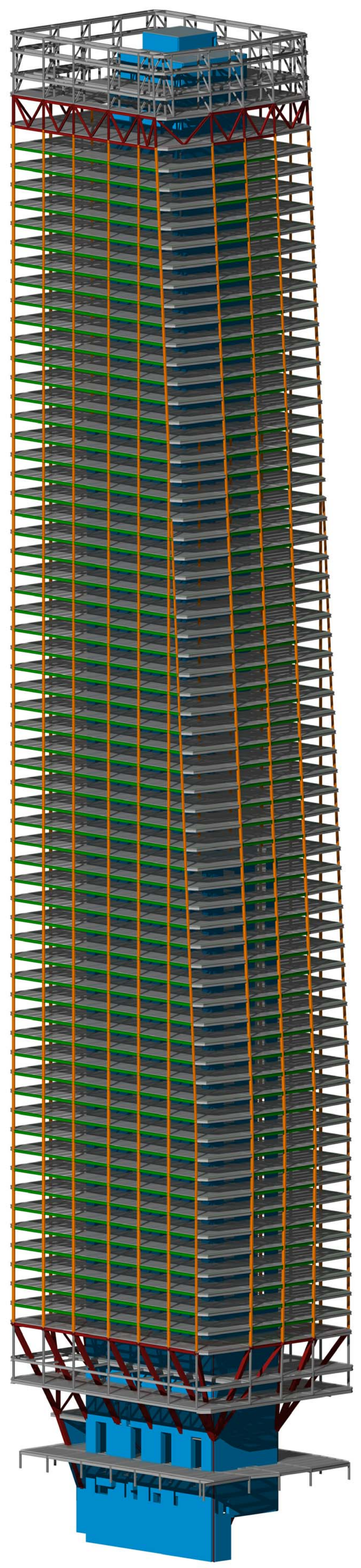
REINFORCED CONCRETE CORE WALLS



STEEL COLUMNS, "KICKERS" & BELT TRUSSES



COMPOSITE DECK & STEEL FLOOR FRAMING



OVERALL STRUCTURAL SYSTEM

| STRUCTURAL DRAWINGS | | |
|---|---|---|
| DRAWING NO. | DRAWING NAME | SCALE |
| S-001 | STRUCTURAL SYSTEMS DRAWING AND SHEET LIST | NTS |
| S-002 | TYPICAL STRUCTURAL SYMBOLS AND ABBREVIATIONS | NTS |
| S-003 | EXCAVATION AND FOUNDATION NOTES | NTS |
| S-004 | STRUCTURAL CONCRETE NOTES | NTS |
| S-005 | STRUCTURAL STEEL AND METAL DECK NOTES | NTS |
| S-010 | LOADING DIAGRAMS | NTS |
| S-011 | LOADING DIAGRAMS | NTS |
| S-020 | CONSTRUCTION SEQUENCES | NTS |
| S-050 | OVERALL LEVEL B PLAN | AS NOTED |
| S-051 | OVERALL GROUND FLOOR PLAN | AS NOTED |
| S-052 | OVERALL 2ND FLOOR PLAN | AS NOTED |
| S-053 | OVERALL 3RD FLOOR PLAN | AS NOTED |
| S-054 | OVERALL 4TH FLOOR PLAN | AS NOTED |
| S-FND | FOUNDATION LOWER PLAN (TRACK LEVEL) | 1/8"=1'-0" |
| S-097 | FOUNDATION UPPER PLAN | 1/8"=1'-0" |
| S-098 | CELLAR B1 PITS FLOOR FRAMING PLAN | 1/8"=1'-0" |
| S-099B1-A | CELLAR B1 FLOOR FRAMING PLAN - PART A | 1/8"=1'-0" |
| S-099B1-B | CELLAR B1 FLOOR FRAMING PLAN - PART B | 1/8"=1'-0" |
| S-100B-A | CELLAR B FLOOR FRAMING PLAN - PART A | 1/8"=1'-0" |
| S-100B-B | CELLAR B FLOOR FRAMING PLAN - PART B | 1/8"=1'-0" |
| S-101-A | GROUND FLOOR FRAMING PLAN - LOBBY - PART A | 1/8"=1'-0" |
| S-101-B | GROUND FLOOR FRAMING PLAN - LOBBY - PART B | 1/8"=1'-0" |
| S-102 | 2ND FLOOR FRAMING PLAN | 1/8"=1'-0" |
| S-103 | 3RD FLOOR FRAMING PLAN | 1/8"=1'-0" |
| S-104 | 4TH FLOOR FRAMING PLAN - MECHANICAL | 1/8"=1'-0" |
| S-105 | 5TH FLOOR FRAMING PLAN - MECHANICAL MEZZANINE | 1/8"=1'-0" |
| S-106 | 6TH FLOOR FRAMING PLAN- LOW-RISE | 1/8"=1'-0" |
| S-116 | 16TH FLOOR FRAMI NG PLAN - TYPICAL LOW-RISE | 1/8"=1'-0" |
| S-131 | 31ST FLOOR FRAMING PLAN - MID-RISE | 1/8"=1'-0" |
| S-137 | 37TH FLOOR FRAMING PLAN - TYPICAL MID-RISE | 1/8"=1'-0" |
| S-138 | 38TH FLOOR FRAMING PLAN | 1/8"=1'-0" |
| S-153 | 53RD FLOOR FRAMING PLAN - HIGH-RISE | 1/8"=1'-0" |
| S-159 | 59TH FLOOR FRAMING PLAN - TYPICAL HIGH-RISE | 1/8"=1'-0" |
| S-168 | 68TH FLOOR FRAMING PLAN - MECHANICAL | 1/8"=1'-0" |
| S-169 | 69TH FLOOR FRAMING PLAN - MECHANICAL MEZZANINE | 1/8"=1'-0" |
| S-170 | ROOF FRAMING PLAN - MECHANICAL PENTHOUSE | 1/8"=1'-0" |
| S-171 | ROOF FRAMING PLAN | 1/8"=1'-0" |
| S-201 | TOWER ELEVATIONS | AS NOTED |
| S-202 | TOWER ELEVATIONS | AS NOTED |
| S-203 | TOWER SECTIONS | AS NOTED |
| S-204 | TOWER SECTIONS | AS NOTED |
| S-301 | CORE WALL CONT. FOOTING SECTIONS & DETAILS | AS NOTED |
| S-302 | CORE WALL CONT. FOOTINGS SECTIONS & DETAILS | AS NOTED |
| S-307 | SPREAD FOOTING SCHEDULE, SECTIONS & DETAILS | AS NOTED |
| S-310 | WEST PERIMETER COLUMN FOUNDATION SECTIONS & DETAILS | AS NOTED |
| S-311 | WEST PERIMETER COLUMN FOUNDATION SECTIONS & DETAILS | AS NOTED |
| S-315 | FOUNDATION WALL ELEVATIONS | AS NOTED |
| S-316 | FOUNDATION WALL SECTIONS & DETAILS | AS NOTED |
| S-317 | FOUNDATION WALL SECTIONS & DETAILS | AS NOTED |
| S-319 | FOUNDATION WALL SECTIONS & DETAILS | AS NOTED |
| S-325 | SLAB-ON-GRADE SECTIONS & DETAILS | AS NOTED |
| S-340 | REINFORCED CONCRETE CORE WALL SCHEDULE, SECTIONS & DETAILS | AS NOTED |
| S-341 | TYPICAL REINFORCED CONCRETE WALL DETAILS | AS NOTED |
| S-342 | CORE WALL REINFORCEMENT LAYOUT | AS NOTED |
| S-343 | CORE WALL REINFORCEMENT LAYOUT | AS NOTED |
| S-344 | CORE WALL REINFORCEMENT LAYOUT | AS NOTED |
| S-345 | CORE WALL REINFORCEMENT LAYOUT | AS NOTED |
| S-346 | CORE WALL REINFORCEMENT LAYOUT | AS NOTED |
| S-347 | CORE WALL REINFORCEMENT LAYOUT | AS NOTED |
| S-348 | CORE WALL REINFORCEMENT LAYOUT | AS NOTED |
| S-349 | CORE WALL REINFORCEMENT LAYOUT | AS NOTED |
| S-350 | CORE WALL REINFORCEMENT LAYOUT | AS NOTED |
| S-351 | CORE WALL REINFORCEMENT LAYOUT | AS NOTED |
| S-352 | CORE WALL REINFORCEMENT LAYOUT | AS NOTED |
| S-353 | CORE WALL REINFORCEMENT LAYOUT | AS NOTED |
| S-354 | CORE WALL REINFORCEMENT LAYOUT | AS NOTED |
| S-355 | LINK BEAM SCHEDULES, SECTIONS & DETAILS | AS NOTED |
| S-356 | LINK BEAM SECTION AND DETAILS | AS NOTED |
| S-360 | CORE WALL ELEVATIONS ALONG LINE 3 | AS NOTED |
| S-361 | CORE WALL ELEVATIONS ALONG LINE 5 | AS NOTED |
| S-362 | CORE WALL ELEVATIONS ALONG LINE C | AS NOTED |
| S-363 | CORE WALL ELEVATIONS ALONG LINE D | AS NOTED |
| S-364 | CORE WALL ELEVATIONS ALONG LINE E | AS NOTED |
| S-365 | CORE WALL ELEVATIONS ALONG LINE F | AS NOTED |
| S-366 | CORE WALL ELEVATIONS ALONG LINE G | AS NOTED |
| S-367 | CORE WALL ELEVATIONS ALONG LINE 4 | AS NOTED |
| S-368 | CORE WALL ELEVATIONS WITHIN SERVICE CORE BETWEEN LINES D & E | AS NOTED |
| S-370 | REINFORCED CONCRETE BEAM SCHEDULE, SECTIONS & DETAILS | AS NOTED |
| S-375 | REINFORCED CONCRETE SLAB SCHEDULE, SECTIONS & DETAIL | AS NOTED |
| S-401 | STRUCTURAL STEEL COLUMN SCHEDULE, SECTIONS & DETAILS | AS NOTED |
| S-402 | STRUCTURAL STEEL COLUMN SCHEDULE, SECTIONS & DETAILS | AS NOTED |
| S-405 | KICKER COLUMNS AT CORE LINES C TO F - SCHEDULES, ELEVATIONS, SECTIONS & DETAILS | AS NOTED |
| S-406 | KICKER COLUMNS AT CORE LINE G - SCHEDULES, ELEVATIONS, SECTIONS & DETAILS | AS NOTED |
| S-407 | KICKER COLUMNS AT CORE LINE 3 & 5 SCHEDULES, ELEVATIONS, SECTIONS & DETAILS | AS NOTED |
| S-408 | KICKER COLUMN AT LINE B - SCHEDULE, ELEVATIONS, SECTIONS & DETAILS | AS NOTED |
| S-409 | KICKER COLUMN SECTIONS & DETAILS | AS NOTED |
| S-410 | KICKER COLUMN SECTIONS & DETAILS | AS NOTED |
| S-411 | COLUMNS & DIAGONALS EAST OF LINE H BELOW LEVEL 6 - SECTIONS & DETAILS | AS NOTED |
| S-415 | LOWER BELT TRUSS SCHEDULES, SECTIONS & DETAILS | AS NOTED |
| S-416 | UPPER BELT TRUSS SCHEDULES, SECTIONS & DETAILS | AS NOTED |
| S-417 | BELT TRUSS SECTIONS & DETAILS | AS NOTED |
| S-420 | TYPICAL STRUCTURAL STEEL SECTIONS & DETAILS | AS NOTED |
| S-421 | TYPICAL STRUCTURAL STEEL SECTIONS & DETAILS | AS NOTED |
| S-422 | RETAIL SECTIONS | AS NOTED |
| S-501 | METAL DECK SLAB SCHEDULES, SECTIONS & DETAILS | AS NOTED |

STRUCTURAL SYSTEM DESCRIPTION

A. FOUNDATIONS

1. Foundation System

The primary foundation system for the North Tower is a series of continuous reinforced concrete footings on rock supporting the Tower's central reinforced concrete core walls. These footings are excavated in the underlying rock beginning near the Track Level of the site. The nominal rock capacity at Track Level is 40 to 60TSF. The footings under the core are embedded into the sound rock to enhance the allowable strength increases per the NYC Building Code. A significant number of rock anchor tie-downs are required, primarily under the north and south continuous footings of the core, to resist the overturning forces from the governing lateral loads. In addition to the continuous footings under the core, a mega column foundation is constructed on the west face of the Tower to take the load from the perimeter columns which "kick" in from the north, south and west sides towards the middle of each face. These foundations are constructed near the B Level and occur just behind the existing retaining wall and Platform support structure.

B. SUPERSTRUCTURE

1. Gravity System

The North Tower consists of seventy-one stories above grade and one below-grade level. Height above Plaza Level is currently anticipated to be approximately 995'. Typical floor-to-floor heights in the office floors is 13'-6". The eastern face of the building slopes back to the west over the height of the Tower leading to a smaller floor plan at the top.

The gravity system utilizes concrete slab on composite metal deck supported by structural steel floor beams and girders outside of the core wall system and one-way reinforced concrete beams and slabs within the core area. The floor beams and girders frame to either perimeter columns, spandrel beams or the concrete core at the center of the building. The spandrel beams are moment connect to the perimeter columns at each story, creating a perimeter moment frame over the height of the tower. The column elements are rolled shapes where possible. Built-up structural steel column shapes are required in the lower portions of the building where loads are the highest. The majority of exterior columns along grid line 1 and 7 will "kick back" to the concrete core from level 6 to level 2. The mechanical floors have in-plane bracing to take out the tension forces. Additional bracing is utilized below level 6 along grid line A and B, to take the shear force due to unbalanced live load cases. With all the gravity loads transferred back to the core, the uplift due to wind or seismic is also reduced. At the upper mechanical levels, a structural steel belt truss is used which aids in equalizing the columns loads, as well as minimizing overall building drift. The belt trusses will also provide system redundancy for progressive collapse mitigation.

2/20/2015 8:16:46 PM

MANHATTAN WEST: NORTH TOWER

375 Ninth Avenue, New York, NY 10001

Client

Brookfield

Brookfield Place
250 Vesey Street, 15th Floor, New York, NY 10281

Architecture/Structural Engineering

SOM

Skidmore, Owings & Merrill LLP
14 Wall Street, New York, NY 10005

Civil Engineering

Philip Habib & Associates
102 Madison Avenue #11, New York, NY 10016

MEP Engineering

Jaros Baum & Bolles
80 Pine Street, New York, NY 10005

Vertical Transportation

Edgett Williams Consulting Group, Inc.
102 East Blithedale Ave. Suite 1, Mill Valley, California 94941

Sustainable Design

Viridian Energy & Environmental
50 Washington Street, Norwalk, CT 06854

Geo-Technical Engineering

Mueser Rutledge Consulting Engineers
14 Penn Plaza, 225 W. 34th Street #610, New York, NY 10122

Landscape Consultant

Field Operations
475 10th Avenue, New York, NY 10018

Security Consultant

Ducibella, Ventor & Santore
250 State Street #F1, North Haven, CT 06473

Blast Consultant

Weidlinger Associates, Inc.
40 Wall Street, New York, NY 10005

Acoustical Consultant

Cerami & Associates
404 Fifth Avenue #8, New York, NY 10018

Vibration Consultant

Wilson, Uhrig & Associates, Inc.
65 Broadway, Suite 401, New York, NY 10006

Code Consultant

Code Consultants Professional Engineers PC
215 West 40th Street, 15th Floor, New York, NY 10018

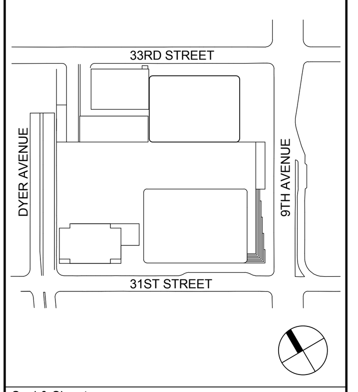
Facade Maintenance Consultant

Entek Engineering LLC
166 Ames Street, Hackensack, NJ 07601

Wind Tunnel Consultant

Rowan Williams Davies & Irwin Inc.
650 Woodlawn Road West, Guelph
Ontario, Canada N1K 1B8

Key Plan:




Seal & Signature:

PRELIMINARY
NOT FOR CONSTRUCTION

| No. | Date | Description |
|---|---|---|
| 8 | 20 FEB 2015 | ISSUED FOR REVIEW |
| 7 | 19 DEC 2014 | ISSUED FOR DD CONFIRMATION |
| 6 | 20 JUN 2014 | ISSUED FOR FOUNDATION BID |
| 5 | 20 JUN 2014 | ISSUED FOR FOUNDATION PERMIT |
| 4 | 20 DEC 2013 | ISSUED FOR FOUNDATION PERMIT |
| 3 | 15 NOV 2013 | ISSUED FOR DD PROGRESS PRICING |
| 2 | 12 JUL 2013 | ISSUED FOR 50% DESIGN DEVELOPMENT |
| 1 | 15 MAR 2013 | ISSUED FOR RECONCILIATION |

Sheet Name:

STRUCTURAL SYSTEMS DRAWING AND SHEET LIST

Project No.: 211157

Date: 20 FEB 2015

Scale:

File No.: S-001

Sheet No.: S-001

B-SCAN Sheet No.: S-001.00

# APPENDIX B

## MANHATTAN WEST: NORTH TOWER

Foundation Design Recommendations Report

**FOUNDATION DESIGN RECOMMENDATIONS**
**MANHATTAN WEST**
**NEW YORK, NEW YORK**

**Brookfield Properties**
**Three World Financial Center**
**200 Vesey Street, 11th Floor**
**New York, NY  10281-1021**

**Mueser Rutledge Consulting Engineers**
**14 Penn Plaza – 225 West 34th Street**
**New York, NY  10122**

**June 10, 2008**
***(Revised) February 18, 2015***

Mueser Rutledge
Consulting Engineers
14 Penn Plaza · 225 West 34th Street · New York, NY 10122
Tel: (917) 339-9300 · Fax: (917) 339-9400
www.mrce.com



June 10, 2008 ***(Revised February 18, 2015)***

Brookfield Properties
Three World Financial Center
200 Vesey Street, 11th Floor
New York, NY 10281-1021

Attention: Mr. James White

Re: Manhattan West
Foundation Design Recommendations Report
New York, NY
MRCE File 9560

Gentlemen:

This report transmits our recommendations for foundation design for the Ninth Avenue development. Prior to this report, design recommendations have been provided to the design team by Mueser Rutledge Consulting Engineers (MRCE) on an ongoing basis. Drilled caissons and/or spread footings have been selected for the foundations of the high rise buildings and for the low rise podium structures.

This report is intended to accompany our report titled "Subsurface Investigation Data Report, 9th Avenue Development" dated ***December 17, 2014***. That report summarized findings of the subsurface investigations. The revisions for rock subgrade reaction modulus are shown in "***Bold Italic***" in page 7.

**EXHIBITS**

| | |
|---|---|
| Plate No. 1 | Site Plan |
| Sheet 2 of 5 | Site Survey Plan |
| Figure 1 | Recommended Lateral Wall Surcharge Values |
| Figure 2 | Typical Retaining Wall Drainage Concept |
| Figure S-1 | 2008 NYCBC Seismic Design Spectrum |
| Appendix A | Rock Properties used for Foundation Design |
| Appendix B | Caisson Capacity Information |

## PROJECT DESCRIPTION

The Ninth Avenue site is between 31st and 33rd streets, immediately west of Ninth Avenue and the Post Office Building as shown on Plate No 1. Site grades and boundaries are shown on Sheet 2 of 5, the site survey plan. The site is bounded by 31st Street to the south, Ninth Avenue to the east, 33rd Street to the north, and the elevated portion of Dyer Avenue to the west.

The site includes the open-cut rail corridor operated by Amtrak and the Long Island Rail Road (LIRR), referred to as track level. The rail corridor is in a rock cut excavation made for the Pennsylvania Railroad and the Long Island Rail Road in about 1908. That cut is approximately 45 to 60 feet below street grade. The site also includes the two at-grade parking lots at 31st and 33rd streets, referred to as "Terra Firma". The at-grade portion of the site accessed from 33rd street surrounds the "Loft Building", a 12-story structure, which will remain. The site has a total area of approximately 207,000 square feet broken down as follows: North parking lot area 50,600 square feet, track corridor area 124,200 square feet, and south parking lot area 32,200 square feet.

Our understanding of the proposed project is based on structural drawings provided by Skidmore, Owings and Merrill (SOM). The proposed scheme consists of two high-rise structures in the eastern portion of the site and a plaza area covering the remainder of the site. Foundations in the western portion of the site are designed to accommodate two additional future structures. The two high-rise buildings are located over the rail corridor necessitating load transfer structures and foundations restricted to limited spaces between the tracks.

## FOUNDATION DESIGN RECOMMENDATIONS

Our recommendations and geotechnical design parameters for design of the project foundations are as follows:

### Influence Lines for Footing Bearing Surfaces

For this report, an "Influence Line" will refer to the line drawn from the bottom of a structure not to receive any significant new load, upwards from that structure on a specified incline. For the bedrock on this project, we recommend the incline of that influence line should be taken as 1 Vertical to 1 Horizontal (1:1), or 45 degrees.

That recommended influence incline is slightly flatter than for influence lines in bedrock based on empirical elastic stress distribution. However, empirical distributions can permit as much as 10% of the applied load to act above that influence line, which in the case of this project may be large. That is to say influence lines are typically not lines of zero excess stress (stress above the geostatic condition), but only reduced excess stress. Also, analyses of joint orientation and inclination of the project bedrock show that unfavorable "wedges" of bedrock may exist in almost any direction. Those wedges, or blocks of rock, may push against other blocks of rock in an unfavorable orientation, when loaded either vertically or laterally.

On a case-by-case basis it may be possible to have a footing above the design influence line. The use of rock bolting can secure the rock mass below footings, allowing a steeper influence line.

**Friction and Bond Values**

For caissons, the (Pre-2008) NYC Building Code allows up to 200 pounds per square inch (psi) bond for sound rock which has been cleaned. That value was selected based on testing in Manhattan Schist which showed that at twice that value, failure did not occur. However, because of the very high column loads, design of the caissons using a 200 psi bond value had led to an unfavorably high aspect ratio of length to diameter. The long caisson lengths based on a 200 psi bond value, raised the concern that the distribution of shear along the caisson would be excessively high near the top of the caisson. High concentrations of shear could cause progressive bond failure, transferring load undiminished further down the caisson length. Moreover, published literature suggests that a ratio of length-to-diameter for caissons greater than about eight would result in essentially no load transfer to the bottom of the caisson. To address that issue and develop a caisson design that more closely approaches the actual shear distribution along the caisson, we have recommended using a bond value of 300 psi. Use of any value in excess of 200 psi may require a variance from New York City DOB (Note: The 2008 Building Code does not require a variance for caisson bond values). A procedure for performing a pullout test has been developed by MRCE as part of the effort to support the variance allowing a bond value of 300 psi to be used in rock socket design for the caissons. That pullout test has yet to be performed.

The 2008 NYC Building Code (“New” Code) does not include limitations on the allowable bond stress for caisson rock sockets. The 2008 Code requires that the rock socket be sufficient to develop the full load-bearing capacity of the caisson pile with a minimum safety factor of two. The results of the pullout tests shall be evaluated on the basis of the limiting safety factor.

The design calls for the caisson caps to be fully embedded in sound bedrock. Therefore, the caps must be cast directly against clean, sound bedrock.

For friction of concrete footings on sound bedrock, we recommend an angle of internal friction of 35 degrees, which translates to a frictional coefficient of 0.7.

For allowable bond values of tie beams and caisson caps against bedrock, we have recommended using 25 psi bond. That reduced value will account for a relatively shallow cut.

**Caisson Bearing Values**

We have recommended that caissons be designed for the zero end bearing condition. However we have also recommended that the caisson subgrades be cleaned to a high standard. That approach allows the caissons (significantly shortened by designing for 300 psi bond) to carry some percentage of the applied vertical load in end bearing as a redundancy to side shear. The recommendation to not design for end bearing but provide for that possibility is in agreement with the recommendation for providing a prismatic core steel section. We recommend that the caisson rock surface be inspected with a video camera prior to concrete placement.

**Caisson Core Steel**

To account for possible transfer of applied load undiminished (or only partially diminished) to the lower portion of the caissons, we have recommended maintaining the core steel sections prismatic for the full depth of the caisson, rather than reducing the section with depth. While this recommendation adds to the cost per foot of the core steel, the reduction in total caisson length of a third (length based on 300 psi vs 200 psi bond), should compensate. The significant savings in time to drill and clean the shorter caisson lengths provides cost savings.

Core steel should be designed using an allowable stress of 0.5 $F_y$. The Pre-2008 NYC Code does not allow that yield stress to exceed 36 ksi. However, we have been successful in the past in getting a variance for steel with a yield stress of up to 50 ksi, allowing a design working stress of 25 ksi for core steel. Note that in accordance with the 2008 New York City Building Code, the allowable compressive stresses for caisson piles shall not exceed: concrete 0.33 $f'_c$, structural steel core 0.50 $F_y$, the former yield stress limit of 36 ksi has been eliminated.

To provide additional lateral capacity to resist lateral loads, a vertical reinforcement cage consisting of bars and horizontal ties can be used. The reinforcing cage will be efficient in resisting bending moments at the top of caissons because the steel will be concentrated around the perimeter. The cage should be extended to a sufficient depth to accommodate the bending moments (Figure 7B, Appendix B). For frictionless caissons, high bending moments occur at the top of the caisson and at the bottom of the sleeve (Figure 8B, Appendix B). These caissons will require long reinforcing cages.

**Retaining Walls**

For design of retaining walls we recommend the following earth pressure parameters:

For granular soil (including fill) use:

- Maximum Friction angle = 32 degrees,
- Unit weight = 120 pcf
- Active pressure coefficient = 0.31
- At-rest pressure coefficient = 0.47

To calculate the lateral pressure of a stable rock face, use an equivalent fluid pressure of 20 psf per foot of depth.

For permanent design of walls retaining soil or backfill, use the at-rest earth pressure condition. Apply to the calculated earth and bedrock pressure distribution, either a sidewalk surcharge or a seismic surcharge, whichever is greater. For the surcharge computations, see Figure 1.

All loads from footings within the influence line should be included in the calculation of lateral rock pressures on the new retaining walls for the Northeast Tower core. Those loads may be calculated using elastic attenuation.

A permanent drainage system to preclude buildup of water behind retaining walls must be included or a design water pressure accounted for.

**Subsurface and Wall Drainage**

To prevent water pressure from building up behind retaining walls, positive drainage of seepage water to the back of the wall must be provided. We attach Figure 2, providing an example of a wall drainage system that may be used. The intercepted groundwater must be conveyed to and collected at the base of the retaining walls (toe drain). The collected water should be discharged to either the Amtrak track drainage system, or to a project sump. Drainage by exfiltration should typically not be permitted. Drainage system design may be subject to Amtrak approval.

For drainage below slabs-on-grade which may be at risk of upward seepage, provide an under slab drainage system. Because the prevailing (non-perched) water table is anticipated to be lower than project slabs, the under slab drainage system is not a true underdrain system which flows continually or regularly (and which NYC typically does not permit). We typically refer to such a "dry" system as an emergency relief system. For recommended slab-on-grade preparation, see the subheading "Slab-on-Grade" under "Construction Recommendations".

The relief drain stone placed above the prepared subgrade should be washed ASTM No. 57 stone, installed to a minimum thickness of 9-inches. Within the stone layer, embed a network of drainage pipes comprised of 4-inch diameter, perforated drainage pipe. Install the pipes level (not pitched) on an interconnecting grid, not exceeding 40 feet in either grid dimension. Use semi-rigid perforated pipe on the perimeter of the drainage pipe grid, with cleanouts installed on each line of the perimeter (the sump qualifies as a cleanout for the lines discharging into it). Semi-rigid pipe is recommended as it remains straight and enhances the ability to clean those pipes out, if needed. On the interior of the drainage grid, corrugated perforated pipe is typically used. Corrugated pipe is flexible and can easily be maneuvered to avoid footings and other obstructions during placement. Provide collection / discharge sumps as needed. Use cast iron outfall pipe through the sump walls.

**Seismic Design Parameters**

The proposed development will be designed in accordance with the seismic criteria of the 2008 New York City Model Code (the "Code"). The site would be classified as Class B that has the following parameters:

| | | |
|---|---|---|
| $S_S = 0.365$ | $F_a = 1.0$ | $S_{DS} = (2/3) \times S_S \times F_a = 0.243$ |
| $S_1 = 0.071$ | $F_v = 1.0$ | $S_{D1} = (2/3) \times S_1 \times F_v = 0.047$ |

Figure S-1 shows the corresponding design response spectrum for a structural damping of five percent. Tabulated values of spectral accelerations are provided for selected structural periods. The spectral accelerations have already been multiplied by two thirds, as per the Code's guidelines.

We understand that an Occupancy Category and a Seismic Use Group classification for the proposed building has been selected by the Architect.

**Vertical Rock Anchors**

Rock anchors, used as permanent tie downs must be double corrosion protected. The design load in the tie down will be based on the allowable design for the anchor bar selected. To calculate the length of tie down required, compute the required length by two methods: 1) based on a bond value of 150 psi (corrugation to grout) or 50 psi (grout to bedrock), whichever is less, and 2) the length required for dead weight restraint by a cone of bedrock. For the dead weight cone, calculate the volume of bedrock in a cone 30 degrees from vertical, with its apex at the base of the anchor. Use 100 pounds per cubic foot for the unit weight of submerged bedrock to derive the total dead-weight restraint. The design length of the anchor will be the longer of the lengths calculated by bond value or by dead weight cone. The factor of safety applied to the length for both bond and the dead weight shall be 1.0 or greater.

**Tie Beams**
Tie Beams are needed to transmit lateral loads between foundation elements. The requirement for tie beams is set by the NYC Seismic Code, the need to restrain caissons that contain "frictionless" sleeves, and the need to laterally brace footings which may be at or above an influence line.

## FOUNDATION ANALYSIS

Below is a discussion of parameters used for foundation analysis. These parameters were provided to SOM over the past few months so that they could proceed with design of the foundations and caissons.

**Elastic Constants**
Jointed rock masses comprise interlocking angular particles or blocks of hard brittle material separated by discontinuity surfaces which may or may not be coated with weaker materials. The strength and elastic response of such rock masses depends on the strength of the intact material and on their freedom of movement, which, in turn, depends on the number, orientation, spacing and shear strength of the discontinuities. The Hook et al. 2002 Failure Criterion was used to estimate the strength of jointed rock masses for this project.

Selected rock mass parameters compiled for the project, are provided in Appendix A, and properties are also provided in the data report dated May 19, 2008.

**Lateral Subgrade Reaction Modulus**
Laterally loaded caissons socketed in rock represent a classical soil/rock-structure interaction problem. The soil/rock reaction depends on the caisson lateral displacement, while lateral displacement is dependent on the soil/rock response and flexural stiffness of the caissons. The caisson can be modeled as an elastic beam or elastic beam column, while rock mass response can be modeled as discrete springs using subgrade reaction procedure or as continuum using finite element method. For this project subgrade reaction procedure was used to model the lateral rock resistance to lateral loads imposed by the superstructure on foundation elements.

For caisson caps, side friction (bond) and passive resistance of the cap and caissons contribute to lateral capacity, and base friction (bond) is ignored. Analytical methods based on subgrade reaction modulus treat the caissons as a beam on elastic foundation problem. For footings, lateral resistance is developed by base friction.

The Rock Mass Modulus (RMM) is used to estimate lateral subgrade reaction moduli for footings, caisson caps, and caissons. Those lateral subgrade reaction moduli are then used to evaluate lateral spring stiffness for each foundation element in the overall structural model. The rock mass modulus for this project was obtained after careful review of all the collected rock core data, including strength testing.

**Rock Lateral Spring Stiffness**

To evaluate lateral resistance of footings, caissons, and caisson caps embedded in rock, lateral springs which represent the load-deformation behavior of the rock mass were used. The following lateral spring stiffness values were assigned by MRCE for use by the structural engineer's model using the structural analysis and design software "ETABS".

To estimate bond and passive resistance versus displacement, we provided elastic (rock) springs for ETABS analysis. Those springs representing resistance of caisson caps are modeled at the bottom of the caisson cap with the spring value calculated using the actual cap side shear and passive resistance areas. Where multiple columns are supported by a single cap, it is required that displacement compatibility is maintained. Caisson cap stiffness may be modeled stiffer than actually calculated by transformed section, if needed to maintain displacement compatibility for very small rock displacements.

The recommended spring constants are:

Footing Resistance to Lateral Movement:
- Below Footings Friction, $k_{h\text{-footing\_friction}} = 1.72 \text{x} 10^4$ (kips/ft$^3$)

Springs Applied to Caisson Caps:
- Side Friction, $k_{h\text{-cap\_friction}} = 1.12 \text{x} 10^4$ (kips/ft$^3$),
- Passive Resistance, $k_{h\text{-cap\_passive}} = 0.80 \text{x} 10^4$ (kips/ft$^3$)
- Base Friction – Assume zero

Caissons:
- Passive Resistance, $k_{h\text{-caisson\_passive}} = 1.30 \text{x} 10^5$ (kips/ft$^3$)

Elastic (rock) springs representing passive rock resistance to caisson movement are modeled along the caisson depth below the bottom of the caisson cap based on the actual horizontally projected area of the caisson in passive resistance.

Spring reactions should be evaluated not to exceed allowable rock bearing pressures. Also, spring reactions represent displacement dependent resistance, and the sum of the reactions at each cap should not exceed the applied loads.

Footings Vertical Bearing:
***Based on Televiewer records and rock face excavation observed in the Mega Column and Amtrak Stairway excavations.***
***- Below Footings Bearing***
***Foundation Level at Elev. -10 or Lower, $k_{v\text{-footing\_bearing}} = 4.0 \text{x} 10^4$ (kips/ft$^3$)***
***Foundation Level higher than Elev. -10, $k_{v\text{-footing\_bearing}} = 2.4 \text{x} 10^4$ (kips/ft$^3$)***

The below footings bearing spring stiffness was determined based on an allowable rock bearing pressure 40 tons per square foot (tsf).

**RISA Analysis**

The Structural Analysis software RISA was used to develop moment and shear vs. displacement plots for single caissons. RISA analysis did not account for the cap side friction and passive resistance and only considered the caisson passive resistance. The purpose of these plots is to check moments and shears at the top of the caissons derived from the overall superstructure and foundation ETABS analysis. In producing these plots, we used the caisson core steel per Caisson Schedule drawing no. S-301A by SOM, revision 6 dated 5/16/08. The lateral spring stiffness for passive resistance of the caissons used in the RISA analysis was kh-caisson = 1.3x105 (kips/ft3).

Sample results from RISA calculations for caissons designated CC8 “Compression Controlled Caisson, at the track level”, and FCC9 “Frictionless Compression Controlled caisson, at B2 level” are provided in Appendix B with evaluation criteria and relevant figures. We understand that the caisson sections issued by SOM on May 16, 2008 did not include reinforcement cages that will contribute to bending and shear resistance. Therefore the plots provided in Appendix B will need to be revised to reflect the additional caisson reinforcement when it is available.

**Group Effect – PLAXIS Analysis:**

The Finite Elements software *PLAXIS* was used to evaluate the group action effect on lateral passive caisson resistance for caps supported by multiple caissons. A 2-D plain strain model was used to provide insight on the group effect of lateral caisson capacity. A cap with eight caissons laterally loaded by wind force in the east direction is shown in Appendix B. A 3-D model was not used because the current version of the Plaxis program significantly over-estimates lateral circular pile capacity.

The group effect on lateral caisson capacity estimated by the 2-D *PLAXIS* model is shown on Figure 10B in Appendix B. The 2-D model indicates that the trailing caisson capacity is 55% of the forward caisson. The model also indicates that the average caisson capacity is 75% of the forward caisson capacity. The reduced capacity for trailing caissons is referred to as “shadowing effect”, or “group effect”.

## CONSTRUCTION RECOMMENDATIONS

The following are our construction related recommendations:

**Caisson Construction**

Caisson installation will first require excavation of the caisson caps, within a tightly braced support of excavation, carried through ballast, broken rock (blast rock) and sound bedrock. The design of that SOE near potentially active tracks will need to consider the design train load and must be approved by the railroads. Excavation of the bedrock will require pre-drilling by either line drilling or closely spaced holes to preclude over-break of bedrock beyond the cap footprint due to the tight tolerances and abutting trackage. The caisson caps must be cast directly against the bedrock. The caisson caps should be cleaned of all debris, loose rock and powdered rock.

The high capacity caissons will need to be located and installed to tight tolerances. Overhead power and support wires will severely restrict caisson drill rig maneuverability at track level. Production rates for drilling into the hard “Hartland” bedrock, which exists at the site, will be slow. The portions of the bedrock mass identified as "Pegmatite" in our May 2008 Data Report contain quartzite and will be particularly difficult to drill through.

The railroads will likely require complete elimination of "flyrock" potential caused by the drilling and even fugitive dust.

**Footing Excavations**
All footing subgrades will be on bedrock. Footing excavations near structures, or carried more than a few feet into bedrock, will require pre-drilling. Because of that requirement, the quality of the bedrock (NYC Classification) should be determined during the excavation process, so that the required size of the footing can be assessed as early in the excavation process as possible. Typically, the footing subgrades are covered with a mudmat after acceptance. Both the mudmat and the footing concrete must be cast directly against the bedrock (not formed). We have estimated that allowable bearing intensities for footings on bedrock at Terra Firma level will be either 20 tsf or 40 tsf. For footings at track level, the values are anticipated to be 40 tsf, with the possibility of some footings at 60 tsf. Footing bearing pressures may be increased for embedment at the rate of 10% per foot ***(after the first foot)***, up to twice the nominal bearing value, if Code requirements are met.

**Slab-on-Grade**
Preparation of a rock subgrade for slab-on-grade can be one of two means. Either the excavated rock surface is cleaned of all rock debris, or a limited thickness of compacted rock debris is left in place and covered by a layered system.

The rock subgrade must be cleaned thoroughly to eliminate the need for the layered covering. If that is not practical to achieve, up to 3-inches of compacted rock debris may be left in place, provided that debris layer is covered by a minimum thickness of 2-inches of concrete sand, followed by a non-woven, perforated separation geo-textile. Any drainage layer is constructed above the separation geotextile.

Over-excavation is usually made up by an increased thickness of crushed stone above the covering layer.

**General Excavation and Support of Excavation**
Lowering existing grades on terra firma will require a highly coordinated, railroad approved excavation plan due to the proximity of the trackage below. Apart from the track safety issues, it will be crucial not to clog the existing drainage system behind the existing retaining walls which are to be lowered, or removed. That permanent drainage system contains sizable open channels that must be sealed in advance of the general excavation to preclude accumulation of debris in the bottom of the drainage chases. The existing system is described under the subheading, "Retaining Wall Drainage". Construction of new retaining walls will also require providing permanent wall drainage.

Excavation of the soil materials will be performed in advance of retaining wall lowering. Protection (and possible underpinning) of the Loft building will be required. As of this writing, test pits at the Loft Building had not yet been performed.

Excavation of bedrock will require line drilling on the perimeter (except possibly for the retaining wall side). Line drilling or closely spaced pre-drill rocks will be needed. The exposed rock face should be secured by rock bolts, as the excavation proceeds.

Excavation at track level will include large amounts of rock debris. That debris will contain significant quantities of fines. It will be important not to allow those materials to mix with ballast and disrupt the ability of the ballast to drain.

**Retaining Wall Drainage**

Providing adequate drainage of retaining walls has two components; maintaining adequate drainage of the existing retaining walls, and providing similar full drainage of new retaining walls.

The existing retaining walls have a robust system of drainage chases that were built into the wall at the time of construction, dated approximately 1908. Those chases connect to a bottom pipeline which in turn connects to the Amtrak drainage system by a series of pipes cast in as a weep holes. It is critical, to maintain the viability of that drainage system. Therefore, construction practices which allow debris to enter the drainage chases should not be permitted.

For the proposed retaining walls, we recommend a similar approach. However, rather than providing open drainage chases, we are recommending the use of geo-synthetics. The recommended geo synthetic “wick drains” will collect water and channel it by gravity to a lower perforated drainage pipe. That collection pipe will then in turn discharge to either a sump built for the project, or to the existing Amtrak track drainage system. The perforated drainage pipe and connecting weep holes should be protected from concrete intrusion by a horizontal strip of heavy drainage composite, secured by batten strip to the rock face about a foot above the drainage pipe, and extend below the drainage pipe by 12 to 18 inches.

**Rock Bolts**

Temporary rock bolts will be needed to stabilize the rock face as rock is removed. Typically, rock bolts are not double corrosion protected, therefore conventional reinforcing bar may be used. The rock bolts should be drilled on a slight downward incline (typically 5 to 30 degrees depending on joint orientation) to a length of about half the height of the rock bolt above final subgrade level, or track level ( whichever applies). A minimum bolt length of eight feet is recommended. The rock bolt hole should be flushed clean and the rock bolts inserted and fully grouted. Rock bolt grout will be a highly fluid cement grout with a minimum 28-day compressive strength of 4000 psi. The cover of grout over the length of the rock bolt should be a minimum of one inch. Add a minimum of 1/8-inch to the rock bolt thickness to account for potential corrosion. Steel plates are torqued to the rock face over the bolts. Rock bolts typically have no free-length and are not tensioned. However, if conditions warrant, providing for a free-length and tensioning the bolts may be required.

Rock bolts must be installed as bedrock is excavated in benches. Pattern bolting is recommended, with tighter patterns beneath buildings or when future footings are expected to be within the 1:1 influence line. Typical bolting patterns for cut rock faces not below structures, are a 5 to 20 feet center-to-center horizontal spacing and 5 to 8 feet vertical center-to-center spacing. Bolts should be staggered between horizontal rows. The tighter bolt spacings are required where rock is highly fractured, of poor quality or where the jointing pattern dips into the site. Rock masses that are less fractured, horizontally fractured or have joints that dip away from the site, may have more widely spaced rock bolts. Beneath structures, particularly beneath footings, the rock bolt pattern should likely not exceed five feet in any direction. Selection of the rock bolt pattern and inclination should be by the Contractor in consultation with the Engineer.

Where rock bolt heads are to protrude into the retaining wall for additional lateral restraint of that wall, rock bolt details may be stipulated by the structural engineer.

**Dewatering**

Dewatering of the existing soils (terra firma) on site, are anticipated to require only sump dewatering, as groundwater will follow the top of bedrock. As the bedrock is cut, some seepage through the joints in the rock can be anticipated. The greatest volume of water anticipated (other than from intense storms) will be from the caissons, which are drilled deeply into rock, well below the prevailing water table. As water pumped from the caissons is likely to contain significant quantities of crushed rock silt and sand sized particles, proper sedimentation will be required prior to discharging that water.

**Waterproofing**

Waterproofing of all the below ground spaces should be performed. Waterproofing can be either by membrane waterproofing or by use of the concrete additive KIM manufactured by Kryton. These systems, while both effective, should not be mixed in any one installation. It is our experience that for waterproofing of blind-sided applications, such as pouring the retaining walls directly against rock, the KIM system is more effective than the membrane system because of the irregularities of the cut rock face and protrusions of rock bolts. Where membranes are adhered to a formed concrete wall, it is our experience that the membrane system is highly effective. In all cases, the manufacturer's recommendations should be followed.

For slab-on-grades, use of under slab drainage will negate the need to waterproof those slabs. However, a high-quality vapor barrier should be provided beneath the slab.

Very truly yours,

**MUESER RUTLEDGE CONSULTING ENGINEERS**

By: ______________________________

David R. Good

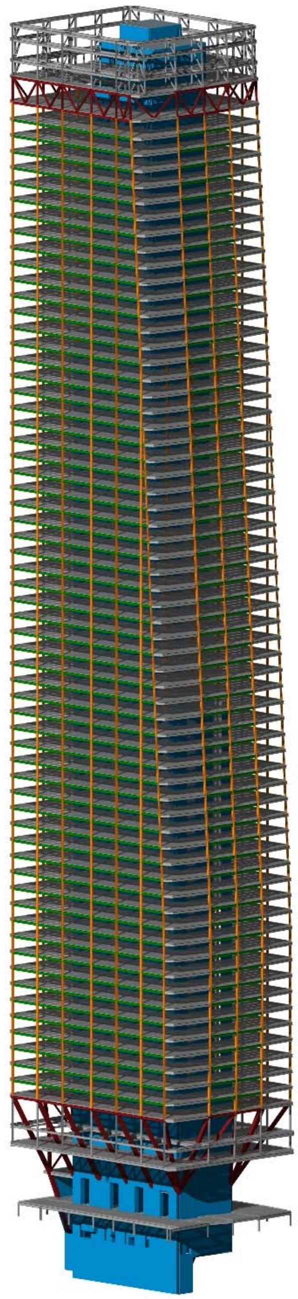
MW:FNE:DRG:F:\95\9560\9560D - NE Tower\Engineering Report\9560 Engineering Report_Revised_2-18-2015.docx

# APPENDIX C

## MANHATTAN WEST: NORTH TOWER

Design Criteria

# MANHATTAN WEST: NORTH TOWER
# STRUCTURAL DESIGN CRITERIA



**MARCH 19, 2015**

Brookfield

SOM

# TABLE OF CONTENTS

# 1. STRUCTURAL DESIGN CRITERIA

## 1.1. Codes And Standards

a. New York City Building Code, 2008

b. Minimum Design Loads for Buildings and Other Structures, ASCE 7-02, American Society of Civil Engineers (ASCE).

c. Building Code Requirement for Reinforced Concrete, ACI 318-02, American Concrete Institute (ACI).

d. Specification for Structural Steel Buildings - Allowable Stress Design and Plastic Design, including Supplement N0.1, 2001, AISC 335-89s1 – American Institute of Steel Construction (AISC).

e. Load and Resistance Factor Design Specification for Structural Steel Buildings LRFD (1999)

## 1.2. Materials

### 1.2.1. Concrete

| | |
|---|---|
| Footings, Caps | f'c = 8,000 psi |
| Foundation Walls | f'c = 5,000 psi |
| | f'c = 8,000 psi |
| Shear Walls | f'c =8,000 psi (@ 56 days) |
| | f'c =10,000 psi (@ 56 days) |
| Slabs and Beams | f'c = 5,000 psi |
| Slab-on-Grade | f'c = 4,000 psi |
| Slabs on Metal Deck | f'c = 4,000 psi |
| Topping/Fill Slabs | f'c = 4,000 psi |
| Mud Slabs | f'c = 3,000 psi |

### 1.2.2. Reinforcing Steel

| | |
|---|---|
| Reinforcing Bars | ASTM A615 Gr. 60 |
| Welded Wire Fabric | ASTM A185 |

### 1.2.3. Structural Steel

| | |
|---|---|
| Rolled Shapes | ASTM A992 Gr. 50 |
| Plates for Built-Up Shapes | ASTM A572 Gr. 50 |
| Welds | E70xx Electrodes |
| | |
| Built-Up Steel Columns | ASTM A572 Gr. 50 |
| Floor Beams | ASTM A992 Gr. 50 |
| W14 Columns, Hangers, & Braced Frame Diagonals | ASTM A913 Gr. 65 |
| W12, W24, W30, W36 Columns | ASTM A992 Gr. 50 |
| Wt Diagonals | ASTM A992 Gr. 50 |
| Connections, Plates | ASTM A572 Gr. 50 |
| Angles | ASTM A36 |
| Bolts | ASTM A325, A490 |
| Welding | E70XX Electrodes |

### 1.2.4. Tie-Downs

| | |
|---|---|
| Threaded Bars (Multiple corrosion protection) | ASTM A702 Fu = 150ksi |
| Grout | f`c=5,000 psi |

## 1.3. Design Loads

### 1.3.1. Gravity Loads

The gravity loads applied to the tower structure are summarized in the following table, detailed loading diagrams are included on the drawing set.

Table 1: Gravity Loads

| Superimposed Gravity Loads | | | | | |
|---|---|---|---|---|---|
| Area Function | SDL (psf) | | | | LL (psf) |
| | Finishes/ Fills | Partitions | CMEP | Total | Total |
| Office | 10 | 20 | 5 | 35 | 50 |
| Retail | 30 | 20 | 10 | 60 | 100 |
| Corridor & Lobby | 30 | 10 | 10 | 50 | 100 |
| Ground Floor Lobby | 40 | 10 | 10 | 60 | 100 |
| Toilets | 30 | 10 | 10 | 50 | 50 |
| Stairs | 0 | 0 | 10 | 10 | 100 |
| Storage | 0 | 0 | 10 | 10 | 125 |
| Back of House | 15 | 10 | 10 | 35 | 50 |
| Loading Dock | 15 | 10 | 10 | 35 | 125 |
| Mechanical | 0 | 0 | 10 | 10 | 150 |
| Light Mechanical | 5 | 25 | 10 | 40 | 120 |
| Heavy Mechanical | 0 | 0 | 10 | 10 | 225 |
| Fuel Tank | 5 | 0 | 10 | 15 | 880 |
| Plaza | 230 | 10 | 10 | 250 | 250 |
| Tree Pit (excluding weight of tree) | 500 | 0 | 0 | 500 | 30 |
| Terrace | 40 | 10 | 10 | 60 | 100 |
| Roof (Non-Accessible) | 50 | 0 | 10 | 60 | 20 |
| Roof Mechanical | 0 | 0 | 10 | 10 | 250 |
| Heavy Construction | 10 | 0 | 0 | 0 | 600 |
| Typical Construction | 10 | 0 | 0 | 0 | 250 |

### 1.3.2. Cladding Loads

Loads shown are per square foot of vertical surface and must be multiplied by the height of the floor. Exterior wall is assumed to be hung from floor above.

Table 2: Cladding Loads

| Area | CLAD (psf) | Comments |
|---|---|---|
| Typical Glass Curtain Wall | 20 | |

### 1.3.3. Live Load Reduction

According to ASCE 7-05, Section 4.8, Live loads are reduced in accordance with the following equation in SI units:

$$L = L_0\left(0.25 + \frac{15}{\sqrt{K_{LL}A_T}}\right)$$

$L$ = Reduced design live load per square foot of area supported by the member
$L_0$ = Unreduced design live load per square foot of area supported by the member
$A_T$ = Tributary Area, in square feet

$K_{LL}$ = Live load element factor as follows:

<table>
<tr><td>Interior columns</td><td rowspan="2">4</td></tr>
<tr><td>Exterior columns without cantilever slabs</td></tr>
<tr><td>Exterior columns with cantilever slabs</td><td>3</td></tr>
<tr><td>Corner columns with cantilever slabs</td><td rowspan="3">2</td></tr>
<tr><td>Edge beams without cantilever slabs</td></tr>
<tr><td>Interior beams</td></tr>
<tr><td>All other members not identified above including:</td><td rowspan="5">1</td></tr>
<tr><td>Edge Beams with Cantilever slabs</td></tr>
<tr><td>Cantilever beams</td></tr>
<tr><td>Two-way slabs</td></tr>
<tr><td>Members without provisions for continuous shear transfer normal to their span</td></tr>
</table>

- $L$ shall not be less than $0.50L_0$ for members supporting one floor
- $L$ shall not be less than $0.40L_0$ for members supporting two or more floors.
- Live loads greater than 100 psf shall not be reduced. Exception is made for members supporting two or more floors for which it may be reduced by 20%.
- Roof live loads and Passenger Car Garage live loads shall not be reduced.

### 1.3.4. Wind Loads

**New York City Building Code, 2008**

Determined according to New York City 2008 Building Code and ASCE 7-05.

Table 3: Parameters for Code Wind Load Calculation

| PARAMETER | VALUE | COMMENT |
|---|---|---|
| Occupancy Category | III | NYC-2008, Table 1604.5 |
| Wind Importance Factor | $I_w = 1.15$ | NYC-2008, Table 1604.5 |
| Basic Wind Speed<br>(3 seconds gust speed at 33 feet above the ground, 50-year mean recurrence interval) | V = 98 mph | NYC-2008, Section 1609.3 |
| Wind Directionality Factor | $K_d = 0.85$ | NYC-2008, Section 1609.6.1.1 and ASCE 7-05 Table 6-4 |
| Exposure category<br>East-West Direction (X-Dir)<br>North-South Direction (Y-Dir) | <br>B<br>A | NYC-2008, Section 1609.4 |
| Topographic Factor | $K_{zt} = 1.0$ | ASCE 7-05, Section 6.5.7.2 |
| Enclosure Classification | Enclosed Building | ASCE 7-05,Section 1609.2 |
| Minimum wind load | 20 psf | NYC-2008, Section 1609.1.2 |
| Damping ratio | 2% | |

**Wind Tunnel Loads**

Wind tunnel loads were determined based on the wind tunnel report by RWDI dated November 21, 2014. Reference the report for wind loads and combinations for the three considered wind load scenarios. The wind tunnel loads provided include the effects of directionality in the local wind climate. These loads do not contain safety or load factors and are to be applied to the building's structural system in the same manner as would wind loads calculated by code analytical methods. The loads are the cumulative summation of the wind-induced loads at the structural level 'Level B' (i.e. grade) centered about the reference axis, exclusive of combination factors. A total damping ratio of 2.0% of critical was used for structural load calculations. The loads correspond to a 50-year return period basic wind speed (3-second gust) of 98 mph.

### 1.3.5. Seismic Loads

Seismic Loads were determined according to New York City 2008 Building Code and ASCE 7. The following table shows the parameters used to calculate the loads.

Table 4: Seismic Design Parameters

| PARAMETER | VALUE | COMMENT |
|---|---|---|
| Occupancy Category | III | NYC-2008, Table 1604.5 |
| Seismic Use Group | II | NYC-2008, Table 1604.5, Note (a) |
| Seismic Importance Factor | $I_E = 1.25$ | NYC-2008, Table 1604.5 |
| Spectral acceleration at short periods | $S_s = 0.365$ g | NYC-2008, Section 1615.1 |
| Spectral acceleration at 1-sec period | $S_1 = 0.071$ g | NYC-2008, Section 1615.1 |
| Site Class | "*B*" (Rock soil profile) | NYC-2008, Table 1615.1.1 |
| Site Coefficients | $F_a = 1.00$<br>$F_v = 1.00$ | NYC-2008, Table 1615.1.2(1) |
| Design spectral response accelerations | $S_{DS} = 0.243$g<br>$S_{D1} = 0.047$g | NYC-2008, Section 1615.1.3 |
| Seismic Design Category | *B* | NYC-2008, Tables 1616.3(1-2) |
| Structural System | Ordinary Reinforced Concrete Shear Wall | NYC-2008, Table 1617.6.2 |
| Height Limit | NL (No Limit) | NYC-2008, Table 1617.6.2 |
| Response Modification Factor | $R = 4$ | NYC-2008, Table 1617.6.2 |
| Deflection Amplification Factor | $C_d = 4$ | NYC-2008, Table 1617.6.2 |
| Overstrength Factor | $\Omega_0 = 2.5$ | NYC-2008, Table 1617.6.2 |

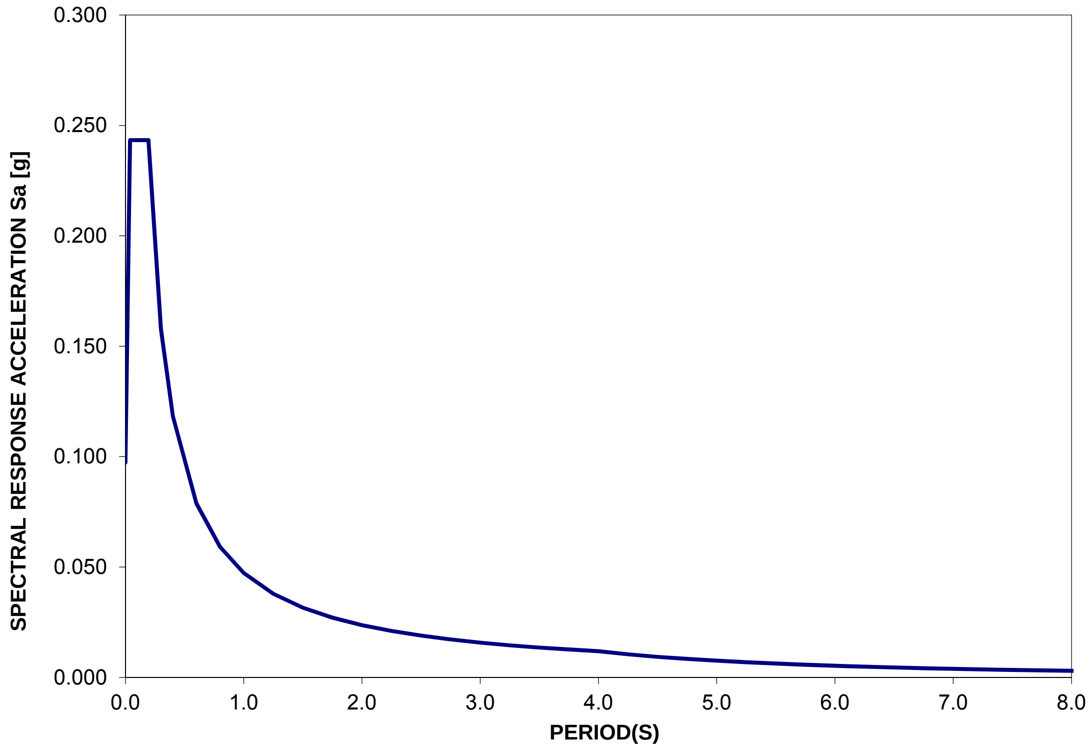



Figure 1: Seismic Design Response Spectrum

### 1.3.6. Load Combinations (ASCE 7-0)

The following loads and load combinations have been considered, including bidirectional effects of the lateral loads:

**Symbols and Notation**

$D$ = Dead Load

$E$ = Earthquake

$H$ = Earth Pressure and/or Ground water

$L$ = Live Load

$L_r$ = Roof Live Load

$R$ = Rain Load

$S$ = Snow Load

$T$ = Self-straining Force (Temperature)

$W$ = Wind Load

**Load Combinations**

**Strength Design Load Combinations**

1. $1.4D$
2. $1.2(D + T) + 1.6(L+H) + 0.5(L_r \text{ or } S \text{ or } R)$
3. $1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (L \text{ or } 0.8W)$
4. $1.2D + 1.6W + L + 0.5(L_r \text{ or } S \text{ or } R)$
5. $1.2D + 1.0E + L + 0.2S$
6. $0.9D + 1.6W + 1.6H$
7. $0.9D + 1.0E + 1.6H$

Notes:

a. Load factor on L in combinations 3, 4 and 5 is permitted to equal 0.5 for all occupancies, except for garages, areas occupied as places of public assemblies and all areas where L is greater than 100 psf.

b. Load Factor on H is zero in combinations 6 and 7 if it counteracts W or E.

c. Where wind load W has not been reduced by a directionality factor it shall be permitted to use 1.3W in place of 1.6W.

d. Where E, the load effects of earthquake, is based on service-level seismic forces, 1.4E shall be used in place of 1.0E.

**Service Load Combinations**

1. D
2. D + H + L + T
3. D + H + ($L_r$ or S or R)
4. D + H + 0.75(L + T) + 0.75($L_r$ or S or R)
5. D + H + (W or 0.7E)
6. D + H + 0.75(W or 0.7E) + 0.75L + 0.75($L_r$ or S or R)
7. 0.6D + W + H
8. 0.6D + 0.7E + H

## 1.4. System Criteria

### 1.4.1. Vertical Deflection

Table 5: Permissible lateral deflections

| PARAMETER | VALUE RELATIVE TO SPAN | ABSOLUTE VALUE | COMMENT |
|---|---|---|---|
| Dead + Live Loads on Floor Beams & Two Way Slabs | L/240 | 1-1/2" | NYC-2008, Table 1604.3 |
| Live Loads Only on Typical Framing Members | L/360 | 1" | NYC-2008, Table 1604.3 |
| Live Loads Only on Perimeter Framing Members | L/600 | 3/8" | |
| Dead + Live Loads on Elevator & Escalator Support Beams | L/1666 | | |

### 1.4.2. Lateral Deflection

Table 6: Permissible lateral deflections

| PARAMETER | VALUE | COMMENT |
|---|---|---|
| Wind overall structural deflection | H/400 | |
| Wind story drift | h/300 | |
| Seismic story drift | 0.015 | NYC-2008, Table 1617.31.1 |

where:

h = story height
H = height from the base of the structure to the roof

### 1.4.3. Vibration

Acceleration Limit 0.50%

# APPENDIX D

## MANHATTAN WEST: NORTH TOWER

Wind Tunnel Test Report

Tel: 519.823.1311
Fax: 519.823.1316

Rowan Williams Davies & Irwin Inc.
650 Woodlawn Road West
Guelph, Ontario, Canada
N1K 1B8

# Manhattan West Northeast Office Tower

New York, NY

# Final Report

## Wind-Induced Structural Responses

RWDI # 1300745
November 21, 2014

**SUBMITTED TO**

**Charles Besjak, PE SE AIA**
Director
Skidmore, Owings & Merrill LLP
14 Wall Street
New York, NY 10005
P: (212) 298-9431
F: (212) 298-9781
M: (917) 855-0321
charles.besjak@som.com

**SUBMITTED BY**

**Kathryn Tang, B.A.Sc., EIT**
Technical Coordinator
kathryn.tang@rwdi.com

**Matthew Browne, M.Eng., P.Eng.**
Wind Engineering Specialist / Associate
matthew.browne@rwdi.com

**Robert W. Summers, B.Sc.(Eng.)**
Senior Project Manager / Associate
bob.summers@rwdi.com

**Jon K. Galsworthy, Ph.D., P.Eng.**
Project Director / Principal
jon.galsworthy@rwdi.com

# TABLE OF CONTENTS



## Tables



## Figures



## Appendices

# 1. INTRODUCTION

Rowan Williams Davies & Irwin Inc. (RWDI) was retained by Skidmore, Owings & Merrill LLP to study the structural wind loading on the proposed Manhattan West Northeast Office Tower in New York, NY.

The objectives of this study were:

i. to provide wind loading information for the overall structural design; and,

ii. to determine the wind-induced accelerations at the uppermost occupied floors.

The following table summarizes relevant information about the design team, methods used, results of the study and the governing parameters:

| | | |
|---|---|---|
| **Project Details:** | | |
| | Architect and Structural Engineer | Skidmore, Owings & Merrill LLP of New York, NY |
| | Measurement Technique | High Frequency Force Balance (HFFB) |
| **Key Results and Recommendations:** | | |
| | Coordinate System for Structural Loading | Figure 4 |
| | Summary of Predicted Peak Overall Structural Wind Loads | Tables 2a to 2c |
| | Effective Static Floor-by-Floor Wind Loads | Tables 3a to 3c |
| | Recommended Wind Load Combinations | Tables 4a to 4c |
| | Predicted Peak Accelerations at Top Occupied Floor | Acceptable for an office occupancy – See Figures 6a to 6c |
| **Selected Analysis Parameters:** | | |
| | Basic Wind Speed per New York City Building Code 2008 | 98 mph (3-second gust) at 33 ft in open terrain |
| | Importance Factor on Wind Pressure | 1.0 |

The wind tunnel test procedures met or exceeded the requirements set out in Section 6.6 of the ASCE 7-05 Standard. The following sections outline the test methodology for the current study, and discuss the results and recommendations. Appendix A provides additional background information on the testing and analysis procedures for this type of study. For detailed explanations of the procedures and underlying theory, refer to RWDI's Technical Reference Document - Wind Tunnel Studies for Buildings (RD2-2000.1), which is available upon request.

# 2. WIND TUNNEL TESTS

## 2.1 Study Model and Surroundings

A 1:400 scale model of the proposed development was constructed using the architectural drawings listed in Table 1. The model was tested in the presence of all surroundings within a full-scale radius of 1600 ft, in RWDI's 8 ft x 6.5 ft boundary layer wind tunnel facility in Guelph, Ontario for the following test configurations:

**Configuration 1:** Proposed Manhattan West Northeast Office Tower with existing and under-construction surroundings.

**Configuration 2:** Proposed Northeast Office Tower with existing and under-construction surroundings as well as the Manhattan West Southwest Tower.

**Configuration 3:** Proposed Northeast Office Tower with existing and under-construction surroundings, with both the Manhattan West Southwest and Southeast Towers.

Photographs of the wind tunnel study model are shown in Figures 1a to 1c, corresponding to test configurations 1, 2, and 3, respectively. An orientation plan showing the location of the study site is given in Figure 2.

## 2.2 Upwind Profiles

Beyond the modelled area, the influence of the upwind terrain on the planetary boundary layer was simulated in the testing by appropriate roughness on the wind tunnel floor and flow conditioning spires at the upwind end of the working section for each wind direction. This simulation, and subsequent analysis of the data from the model, was targeted to represent the following upwind terrain conditions. Wind direction is defined as the direction from which the wind blows, measured clockwise from true north.

| Upwind Terrain | Wind Directions (Inclusive) |
|---|---|
| Open/Suburban –mid to high-rise buildings immediately upwind, followed by the Hudson River and various fetches of suburban and open terrain beyond | 10° to 20°, 210° to 360° |
| Suburban –mid to high-rise buildings immediately upwind, followed various fetches of suburban terrain, open terrain, and water beyond | 30° to 200° |

# 3. WIND CLIMATE

In order to predict the full-scale structural responses as a function of return period, the wind tunnel data was combined with a statistical model of the local wind climate. The wind climate model was based on local surface wind measurements taken at JFK, LaGuardia and Newark Airports and a computer simulation of hurricanes. The hurricane simulation was provided by Applied Research Associates, Raleigh, NC using the Monte Carlo Technique. Over 100,000 years of tropical storms were simulated to account for the variability of hurricane wind speed with direction.

Figure 3 shows a comparison of strength and directionality of the hurricane and extra-tropical (i.e., non-hurricane wind climates for New York. These plots are illustrative only and are not to be used directly for predictions of wind-induced responses. The upper two plots show the directionality of common winds on the left and extreme winds on the right. Since hurricanes are extreme events, they are only included on the right plot. It can be seen that for the extreme events, the winds from the northwest are the strongest, with a secondary lobe for winds from the east. The lower plot shows the wind speeds from each data set as a function of return period. It is clear from the plot that the common events (i.e., lower return periods) are dictated by the extra-tropical winds whereas at longer return periods, the hurricanes generate the most significant wind speeds for strength design.

The design wind speed for New York, as specified in the 2008 New York City Building Code, is a 3-second gust wind speed of 98 mph at a height of 33 ft in open terrain. This wind speed is also shown in Figure 3. For the wind loading predictions for strength design, the wind climate model was scaled to match the design wind speed at the 50-year return period. It is common practice to consider a more representative wind climate for serviceability considerations, including the prediction of accelerations. Therefore, the code-matched wind climate was not used for the acceleration predictions.

# 4. RESULTS AND RECOMMENDATIONS

## 4.1 Predicted Peak Shear Forces and Moments

The reference axis system used to define the forces and moments is illustrated in Figure 4. The overall wind-induced overturning moments, shear forces and torsional moments acting at Structural Level 'LEVEL 1-PLAZA' have been predicted for the design return period and are presented for all test configurations in Tables 2a to 2c.

The loads were determined using the fundamental building vibration periods listed in Tables 2a to 2c, and the corresponding mode shapes provided by the structural engineer on September 15, 2014. Appendix B contains a summary of the provided dynamic properties. The damping ratio was taken as 2% of critical which was considered representative for the building's structural system for strength design.

For illustrative purposes, the overall wind-induced loads for each wind direction are presented in Figures 5a to 5c for all test configurations. The loads in these figures are the values based on the design wind speed, assuming this wind speed applies equally to all directions. In other words, there is no allowance for the relative probability that the design wind speed will occur from different directions. This information simply illustrates the raw source data used in predicting the peak design loads.

Effective static wind loads that correspond to the predicted overall moments and shears are provided on a floor-by-floor basis in Tables 3a to 3c. To account for the simultaneous action of the x, y, and torsional components in Tables 3a to 3c, recommended wind load combination factors are provided in Tables 4a to 4c. There are 24 basic combinations in the table, representing each of eight possible sign sets (+++, ++-, +-+, etc.) with each of Fx, Fy and Mz reaching their individual maximum percentages for that sign set. As an example of applying the combination factors, let us consider Load Case 1 of Table 4a. This load case requires the application of +95% of the Fx, +45% of the Fy, and +50% of the Mz floor-by-floor loads from Table 3a. **It is recommended that all load cases be considered for overall structural design.**

**The wind loads provided in this report include the effects of directionality in the local wind climate. These loads do not contain safety or load factors and are to be applied to the building's structural system in the same manner as would wind loads calculated by code analytical methods.**

## 4.2 Deflections

Deflections have not been specifically evaluated in this study. Normally the structural engineer evaluates floor-to-floor and overall deflections by applying the wind load distributions derived from the wind tunnel tests to a structural computer model of the building. These deflections may then be reviewed by the structural engineer to assess the potential for excessive shearing in wall systems and partitions.

## 4.3 Accelerations

The predicted wind-induced accelerations at the top occupied floor, taken as Structural Level 'LEVEL68' (930.5 ft above Structural Level 'LEVEL 1-PLAZA'), are summarized in Figures 6a to 6c. In addition to the peak values shown in the plot, the peak X, Y and torsional components are also tabulated. The peak accelerations were determined as a function of return period for the provided building periods, and an overall damping ratio of 1.5% of critical which was considered representative for the building's structural system for serviceability considerations. The torsional component, which was included in the total acceleration predictions, was calculated at a representative distance of 68 ft, based on the radius of gyration, from the reference z-axis (given in Figure 4). Results were evaluated both with and without the influence of hurricanes in the wind climate model. As discussed in Appendix A, occupant comfort is assessed at shorter return periods when hurricanes are included in the analysis since, for stronger hurricanes, occupants who choose to remain should not expect normal conditions to prevail.

Figures 6a to 6c also present acceleration criteria from the International Organization for Standardization (ISO 10137:2007(E)), and RWDI's suggested criteria based on different occupancies. In all cases, 5- and 10-year criteria apply to non-hurricane winds only.

From Figures 6a to 6c, it can be seen that the predicted peak accelerations are within the ISO based office criteria for the 1- return period. The 10-year accelerations are also within the RWDI criteria for an office tower. Therefore, it is our opinion that the predicted accelerations are acceptable for human comfort in an office building. It should be noted that building accelerations are a serviceability issue and typically not a safety issue, provided the associated deflections are accounted for in the structural design and the cladding/glazing system design.

## 4.4 Torsional Velocities

Also of interest for occupant comfort are the peak torsional velocities. The Council on Tall Buildings and Urban Habitat (CTBUH) have suggested torsional velocity limits for the 1- and 10-year return periods. As with the accelerations, the 10-year predictions are only of practical concern during non-hurricane winds. **Note that these guidelines are tentative and based on limited research which is still ongoing.** The predicted torsional velocities at the top occupied floor are also shown along with the tentative criteria in Figures 6a to 6c. It can be seen that the predicted torsional velocities are within the criteria for the 1- and 10-year return period. Therefore, in our opinion the torsional velocities are acceptable for human comfort.

# 5. APPLICABILITY OF RESULTS

## 5.1 The Proximity Model

The structural wind loads and building motions determined by the wind tunnel tests and the associated analysis are applicable for the particular configurations of surrounding buildings modelled. City development over time can cause changes in the surroundings from those tested, resulting in loads and accelerations that could differ from those predicted in this report.

Changes in surroundings can be divided into two categories:

a) addition or demolition of buildings far upwind, having the effect of changing the roughness of the earth's surface and thereby changing the general wind exposure of the site; and

b) addition or demolition of buildings close to the site, which can cause changes in the local flow patterns about the study building.

Based on the past history of city developments it appears that, with respect to Category (a), development over time is far more likely to increase rather than reduce building density. This implies that the development over time would more likely diminish loads on the study building rather than increase them. With respect to Category (b), the wind tunnel tests were conducted to represent the current state of the development of the nearby surroundings, including known projects expected to be completed in the near future.

If, at a later date, additional buildings besides those considered in the tested configuration are constructed near the project site, then some load changes could occur. Unless, however, a building of unusual stature is constructed nearby, the normal use of safety or load factors can be expected to cover the potential increases in structural loads. The consequence of increased motion, should it occur, is that a greater percentage of the occupants would notice the motions or find them objectionable.

## 5.2 Study Model and Structural Properties Information

The results presented in this report pertain to 1) the structural properties, as shown in Appendix B; and, 2) the scale model of the proposed development, constructed using the architectural information listed in Table 1; and, 3) the phasing of the proposed development, as reflected in the test configurations. Should there be any design changes that deviate substantially from the above information; the results for the revised design may differ from those presented in this report. Therefore, if the design changes, RWDI should be contacted and requested to review the impact on the wind loads and building responses.

# TABLES

# TABLE 1: DRAWING LIST FOR MODEL CONSTRUCTION

The drawings and information listed below were received from SOM Architects and were used to construct the scale model of the proposed Manhattan West Northeast Office Tower. Should there be any design changes that deviate from this list of drawings, the results may change. Therefore, if changes in the design are made, it is recommended that RWDI be contacted and requested to review their potential effects on the wind conditions.

| File Name | File Type | Date Received (dd/mm/yyyy) |
|---|---|---|
| 20140731_Manhattan_West_Masterplan_North_Tower | Rhinoceros | 19/08/2014 |
| TOWERSECTION | AutoCAD Drawing File | 23/09/2014 |

**Table 2a:** **Summary of Predicted Peak Overall Structural Wind Loads**

**Configuration 1**

| Moments | | | Shears | |
|---|---|---|---|---|
| **My (lb-ft)** | **Mx (lb-ft)** | **Mz (lb-ft)** | **Fx (lb)** | **Fy (lb)** |
| 3.63E+09 | 4.19E+09 | 1.10E+08 | 5.25E+06 | 6.85E+06 |

## Notes:

1. The above loads are the cumulative summation of the wind-induced loads at the Structural Level 'LEVEL 1-PLAZA' (i.e. - at grade) centered about the reference axis shown in Figure 4, exclusive of combination factors.

2. Total damping ratios of 2.0% of critical were used for structural load calculations.

3. The above loads are based on the structural properties provided on September 15, 2014. The natural building periods were as follows:

   Mode 1: 6.989 s (Primarily Y-Sway);

   Mode 2: 5.733 s (Primarily X-Sway);

   Mode 3: 3.731 s (Primarily Torsion).

4. The above loads correspond to a 50-year return period basic wind speed (3-second gust) of 98 mph.

**Table 2b: Summary of Predicted Peak Overall Structural Wind Loads**

**Configuration 2**

| Moments | | | Shears | |
|---|---|---|---|---|
| **My (lb-ft)** | **Mx (lb-ft)** | **Mz (lb-ft)** | **Fx (lb)** | **Fy (lb)** |
| 3.11E+09 | 3.99E+09 | 1.07E+08 | 4.65E+06 | 6.38E+06 |

## Notes:

1. The above loads are the cumulative summation of the wind-induced loads at the Structural Level 'LEVEL 1-PLAZA' (i.e. - at grade) centered about the reference axis shown in Figure 4, exclusive of combination factors.

2. Total damping ratios of 2.0% of critical were used for structural load calculations.

3. The above loads are based on the structural properties provided on September 15, 2014. The natural building periods were as follows:

   Mode 1: 6.989 s (Primarily Y-Sway);

   Mode 2: 5.733 s (Primarily X-Sway);

   Mode 3: 3.731 s (Primarily Torsion).

4. The above loads correspond to a 50-year return period basic wind speed (3-second gust) of 98 mph.

**Table 2c:** **Summary of Predicted Peak Overall Structural Wind Loads**

**Configuration 3**

| Moments | | | Shears | |
|---|---|---|---|---|
| **My (lb-ft)** | **Mx (lb-ft)** | **Mz (lb-ft)** | **Fx (lb)** | **Fy (lb)** |
| 3.02E+09 | 4.45E+09 | 1.72E+08 | 4.65E+06 | 7.08E+06 |

## Notes:

1. The above loads are the cumulative summation of the wind-induced loads at the Structural Level 'LEVEL 1-PLAZA' (i.e. - at grade) centered about the reference axis shown in Figure 4, exclusive of combination factors.

2. Total damping ratios of 2.0% of critical were used for structural load calculations.

3. The above loads are based on the structural properties provided on September 15, 2014. The natural building periods were as follows:

   Mode 1: 6.989 s (Primarily Y-Sway);

   Mode 2: 5.733 s (Primarily X-Sway);

   Mode 3: 3.731 s (Primarily Torsion).

4. The above loads correspond to a 50-year return period basic wind speed (3-second gust) of 98 mph.

CONSULTING ENGINEERS
& SCIENTISTS



**Table 3a: Effective Static Floor-by-Floor Wind Loads**

**Configuration 1**

| Floor | Height Above 'LEVEL 1-PLAZA' (ft) | Fx (lb) | Fy (lb) | Mz (lb-ft) |
|---|---|---|---|---|
| LEVEL1-PLAZA | 0.0 | 7500 | 30900 | 402000 |
| LEVEL2 | 23.0 | 16700 | 67800 | 850000 |
| LEVEL4 | 51.5 | 13200 | 59600 | 723000 |
| LEVEL5 | 65.0 | 12200 | 59500 | 679000 |
| LEVEL6 | 92.0 | 12200 | 63000 | 711000 |
| LEVEL7 | 105.5 | 8100 | 41600 | 475000 |
| LEVEL8 | 119.0 | 8100 | 42600 | 485000 |
| LEVEL9 | 132.5 | 8100 | 43800 | 508000 |
| LEVEL10 | 146.0 | 8100 | 45200 | 530000 |
| LEVEL11 | 159.5 | 8600 | 46600 | 567000 |
| LEVEL12 | 173.0 | 10300 | 48100 | 645000 |
| LEVEL13 | 186.5 | 12400 | 49800 | 715000 |
| LEVEL14 | 200.0 | 14600 | 51500 | 753000 |
| LEVEL15 | 213.5 | 17000 | 53300 | 792000 |
| LEVEL16 | 227.0 | 18700 | 54500 | 823000 |
| LEVEL17 | 240.5 | 20300 | 55600 | 852000 |
| LEVEL18 | 254.0 | 22700 | 57300 | 895000 |
| LEVEL19 | 267.5 | 25500 | 59400 | 929000 |
| LEVEL20 | 281.0 | 28000 | 61400 | 970000 |
| LEVEL21 | 294.5 | 30600 | 63300 | 1010000 |
| LEVEL22 | 308.0 | 33200 | 65400 | 1051000 |
| LEVEL23 | 321.5 | 36000 | 67600 | 1093000 |
| LEVEL24 | 335.0 | 38700 | 69800 | 1143000 |
| LEVEL25 | 348.5 | 41500 | 72100 | 1181000 |
| LEVEL26 | 362.0 | 43400 | 73600 | 1212000 |
| LEVEL27 | 375.5 | 45200 | 74800 | 1250000 |
| LEVEL28 | 389.0 | 47800 | 77000 | 1294000 |
| LEVEL29 | 402.5 | 50700 | 79400 | 1330000 |
| LEVEL30 | 416.0 | 54200 | 82300 | 1384000 |
| LEVEL31 | 429.5 | 55100 | 82800 | 1392000 |
| LEVEL32 | 443.0 | 53600 | 81000 | 1376000 |
| LEVEL33 | 456.5 | 56600 | 83300 | 1423000 |
| LEVEL34 | 470.0 | 59700 | 85600 | 1469000 |
| LEVEL35 | 483.5 | 62800 | 87900 | 1514000 |
| LEVEL36 | 497.0 | 66000 | 90300 | 1559000 |
| LEVEL37 | 510.5 | 69300 | 92600 | 1603000 |
| LEVEL38 | 524.0 | 72500 | 95000 | 1655000 |
| LEVEL39 | 537.5 | 75800 | 97500 | 1696000 |
| LEVEL40 | 551.0 | 79100 | 99900 | 1738000 |
| LEVEL41 | 564.5 | 83500 | 103500 | 1790000 |
| LEVEL42 | 578.0 | 86800 | 106000 | 1839000 |
| LEVEL43 | 591.5 | 90000 | 108500 | 1879000 |
| LEVEL44 | 605.0 | 93400 | 111200 | 1918000 |
| LEVEL45 | 618.5 | 96700 | 113700 | 1955000 |

| Floor | Height Above 'LEVEL 1-PLAZA' (ft) | Fx (lb) | Fy (lb) | Mz (lb-ft) |
|---|---|---|---|---|
| LEVEL46 | 632.0 | 99900 | 116200 | 1994000 |
| LEVEL47 | 645.5 | 103100 | 118800 | 2032000 |
| LEVEL48 | 659.0 | 105900 | 120900 | 2065000 |
| LEVEL49 | 672.5 | 108900 | 123100 | 2100000 |
| LEVEL50 | 686.0 | 112000 | 125700 | 2136000 |
| LEVEL51 | 699.5 | 116800 | 129800 | 2187000 |
| LEVEL52 | 713.0 | 115000 | 127400 | 2157000 |
| LEVEL53 | 726.5 | 109400 | 121200 | 2098000 |
| LEVEL54 | 740.0 | 111500 | 122700 | 2130000 |
| LEVEL55 | 753.5 | 114100 | 124600 | 2157000 |
| LEVEL56 | 767.0 | 116900 | 126800 | 2187000 |
| LEVEL57 | 780.5 | 119700 | 128700 | 2216000 |
| LEVEL58 | 794.0 | 122400 | 130700 | 2238000 |
| LEVEL59 | 807.5 | 125000 | 132700 | 2259000 |
| LEVEL60 | 821.0 | 129600 | 136600 | 2289000 |
| LEVEL61 | 834.5 | 132000 | 138500 | 2303000 |
| LEVEL62 | 848.0 | 135200 | 141200 | 2329000 |
| LEVEL63 | 861.5 | 136700 | 142400 | 2331000 |
| LEVEL64 | 875.0 | 138800 | 144200 | 2338000 |
| LEVEL65 | 888.5 | 140700 | 146000 | 2351000 |
| LEVEL66 | 902.0 | 140800 | 145300 | 2351000 |
| LEVEL67 | 915.5 | 148600 | 152600 | 2351000 |
| LEVEL68 | 930.5 | 314400 | 318000 | 5817000 |
| LEVEL69 | 951.5 | 208300 | 216200 | 3642000 |
| LEVEL70 | 965.5 | 131300 | 135200 | 835000 |
| LEVEL71 | 983.0 | 121000 | 124000 | 914000 |
| **Total** | | **5.25E+06** | **6.85E+06** | **1.10E+08** |

## Notes:

1. The loads given in this table should be used with the load combination factors given in Table 4a.
2. The loads given in this table are centered about the reference axis shown in Figure 4.
3. The above loads correspond to a 50-year return period basic wind speed (3-second gust) of 98 mph.

RWDI
CONSULTING ENGINEERS
& SCIENTISTS

**Table 3b: Effective Static Floor-by-Floor Wind Loads**

**Configuration 2**

| Floor | Height Above 'LEVEL 1-PLAZA' (ft) | Fx (lb) | Fy (lb) | Mz (lb-ft) |
|---|---|---|---|---|
| LEVEL1-PLAZA | 0.0 | 7500 | 24100 | 392000 |
| LEVEL2 | 23.0 | 16700 | 52600 | 835000 |
| LEVEL4 | 51.5 | 13900 | 47300 | 721000 |
| LEVEL5 | 65.0 | 14000 | 47600 | 677000 |
| LEVEL6 | 92.0 | 15800 | 51200 | 707000 |
| LEVEL7 | 105.5 | 10500 | 33700 | 471000 |
| LEVEL8 | 119.0 | 11400 | 34700 | 482000 |
| LEVEL9 | 132.5 | 12600 | 35900 | 506000 |
| LEVEL10 | 146.0 | 13900 | 37300 | 530000 |
| LEVEL11 | 159.5 | 15300 | 38700 | 555000 |
| LEVEL12 | 173.0 | 17600 | 40200 | 631000 |
| LEVEL13 | 186.5 | 20000 | 41900 | 697000 |
| LEVEL14 | 200.0 | 21700 | 43700 | 735000 |
| LEVEL15 | 213.5 | 23400 | 45500 | 772000 |
| LEVEL16 | 227.0 | 24600 | 46600 | 802000 |
| LEVEL17 | 240.5 | 25700 | 47800 | 829000 |
| LEVEL18 | 254.0 | 27500 | 49600 | 872000 |
| LEVEL19 | 267.5 | 29600 | 51700 | 905000 |
| LEVEL20 | 281.0 | 31500 | 53800 | 945000 |
| LEVEL21 | 294.5 | 33500 | 55800 | 984000 |
| LEVEL22 | 308.0 | 35500 | 57900 | 1023000 |
| LEVEL23 | 321.5 | 37600 | 60200 | 1065000 |
| LEVEL24 | 335.0 | 39600 | 62400 | 1114000 |
| LEVEL25 | 348.5 | 41700 | 64800 | 1152000 |
| LEVEL26 | 362.0 | 43200 | 66300 | 1181000 |
| LEVEL27 | 375.5 | 44300 | 67700 | 1218000 |
| LEVEL28 | 389.0 | 46300 | 69900 | 1262000 |
| LEVEL29 | 402.5 | 48600 | 72400 | 1297000 |
| LEVEL30 | 416.0 | 51300 | 75400 | 1350000 |
| LEVEL31 | 429.5 | 51700 | 75900 | 1358000 |
| LEVEL32 | 443.0 | 50200 | 74100 | 1340000 |
| LEVEL33 | 456.5 | 52500 | 76500 | 1387000 |
| LEVEL34 | 470.0 | 54900 | 78800 | 1432000 |
| LEVEL35 | 483.5 | 57300 | 81200 | 1476000 |
| LEVEL36 | 497.0 | 59800 | 83700 | 1520000 |
| LEVEL37 | 510.5 | 62300 | 86100 | 1564000 |
| LEVEL38 | 524.0 | 64800 | 88600 | 1615000 |
| LEVEL39 | 537.5 | 67400 | 91100 | 1656000 |
| LEVEL40 | 551.0 | 69900 | 93600 | 1697000 |
| LEVEL41 | 564.5 | 73500 | 97300 | 1749000 |
| LEVEL42 | 578.0 | 76000 | 99800 | 1797000 |
| LEVEL43 | 591.5 | 78600 | 102400 | 1836000 |
| LEVEL44 | 605.0 | 81200 | 105100 | 1875000 |
| LEVEL45 | 618.5 | 83800 | 107800 | 1912000 |

| Floor | Height Above 'LEVEL 1-PLAZA' (ft) | Fx (lb) | Fy (lb) | Mz (lb-ft) |
|---|---|---|---|---|
| LEVEL46 | 632.0 | 86300 | 110400 | 1950000 |
| LEVEL47 | 645.5 | 88800 | 113000 | 1987000 |
| LEVEL48 | 659.0 | 90900 | 115100 | 2020000 |
| LEVEL49 | 672.5 | 93100 | 117500 | 2055000 |
| LEVEL50 | 686.0 | 95600 | 120100 | 2090000 |
| LEVEL51 | 699.5 | 99500 | 124400 | 2141000 |
| LEVEL52 | 713.0 | 97600 | 121900 | 2109000 |
| LEVEL53 | 726.5 | 92400 | 115700 | 2049000 |
| LEVEL54 | 740.0 | 93900 | 117200 | 2080000 |
| LEVEL55 | 753.5 | 96000 | 119200 | 2107000 |
| LEVEL56 | 767.0 | 98100 | 121300 | 2136000 |
| LEVEL57 | 780.5 | 100100 | 123500 | 2164000 |
| LEVEL58 | 794.0 | 102100 | 125500 | 2185000 |
| LEVEL59 | 807.5 | 104200 | 127600 | 2206000 |
| LEVEL60 | 821.0 | 107900 | 131500 | 2235000 |
| LEVEL61 | 834.5 | 109700 | 133600 | 2249000 |
| LEVEL62 | 848.0 | 112100 | 136300 | 2274000 |
| LEVEL63 | 861.5 | 113200 | 137500 | 2276000 |
| LEVEL64 | 875.0 | 114700 | 139400 | 2282000 |
| LEVEL65 | 888.5 | 116100 | 141200 | 2294000 |
| LEVEL66 | 902.0 | 116100 | 140600 | 2294000 |
| LEVEL67 | 915.5 | 122500 | 147700 | 2294000 |
| LEVEL68 | 930.5 | 264200 | 313900 | 5774000 |
| LEVEL69 | 951.5 | 171300 | 210700 | 3572000 |
| LEVEL70 | 965.5 | 107900 | 131800 | 738000 |
| LEVEL71 | 983.0 | 99500 | 120900 | 782000 |
| **Total** | | **4.65E+06** | **6.38E+06** | **1.07E+08** |

## Notes:

1. The loads given in this table should be used with the load combination factors given in Table 4b.
2. The loads given in this table are centered about the reference axis shown in Figure 4.
3. The above loads correspond to a 50-year return period basic wind speed (3-second gust) of 98 mph.

CONSULTING ENGINEERS & SCIENTISTS

**Table 3c: Effective Static Floor-by-Floor Wind Loads**

**Configuration 3**

| Floor | Height Above 'LEVEL 1-PLAZA' (ft) | Fx (lb) | Fy (lb) | Mz (lb-ft) |
|---|---|---|---|---|
| LEVEL1-PLAZA | 0.0 | 11800 | 24000 | 751000 |
| LEVEL2 | 23.0 | 25600 | 52400 | 1620000 |
| LEVEL4 | 51.5 | 22200 | 48000 | 1361000 |
| LEVEL5 | 65.0 | 21600 | 48700 | 1282000 |
| LEVEL6 | 92.0 | 23200 | 53100 | 1338000 |
| LEVEL7 | 105.5 | 15400 | 35100 | 896000 |
| LEVEL8 | 119.0 | 16300 | 36500 | 917000 |
| LEVEL9 | 132.5 | 17300 | 38000 | 952000 |
| LEVEL10 | 146.0 | 18500 | 39600 | 990000 |
| LEVEL11 | 159.5 | 19700 | 41300 | 1027000 |
| LEVEL12 | 173.0 | 22500 | 43100 | 1160000 |
| LEVEL13 | 186.5 | 25400 | 45200 | 1285000 |
| LEVEL14 | 200.0 | 26900 | 47200 | 1340000 |
| LEVEL15 | 213.5 | 28400 | 49300 | 1395000 |
| LEVEL16 | 227.0 | 29400 | 50800 | 1440000 |
| LEVEL17 | 240.5 | 30400 | 52200 | 1481000 |
| LEVEL18 | 254.0 | 32000 | 54300 | 1541000 |
| LEVEL19 | 267.5 | 33900 | 56800 | 1588000 |
| LEVEL20 | 281.0 | 35500 | 59100 | 1644000 |
| LEVEL21 | 294.5 | 37200 | 61400 | 1699000 |
| LEVEL22 | 308.0 | 39000 | 63800 | 1753000 |
| LEVEL23 | 321.5 | 40900 | 66500 | 1811000 |
| LEVEL24 | 335.0 | 42700 | 69000 | 1878000 |
| LEVEL25 | 348.5 | 44500 | 71700 | 1930000 |
| LEVEL26 | 362.0 | 45800 | 73500 | 1972000 |
| LEVEL27 | 375.5 | 46900 | 75100 | 2023000 |
| LEVEL28 | 389.0 | 48600 | 77600 | 2084000 |
| LEVEL29 | 402.5 | 50600 | 80400 | 2132000 |
| LEVEL30 | 416.0 | 53000 | 83800 | 2205000 |
| LEVEL31 | 429.5 | 53400 | 84500 | 2218000 |
| LEVEL32 | 443.0 | 52000 | 82800 | 2198000 |
| LEVEL33 | 456.5 | 54000 | 85500 | 2261000 |
| LEVEL34 | 470.0 | 56100 | 88200 | 2323000 |
| LEVEL35 | 483.5 | 58300 | 90800 | 2384000 |
| LEVEL36 | 497.0 | 60500 | 93600 | 2443000 |
| LEVEL37 | 510.5 | 62700 | 96300 | 2503000 |
| LEVEL38 | 524.0 | 65000 | 99200 | 2572000 |
| LEVEL39 | 537.5 | 67200 | 102000 | 2628000 |
| LEVEL40 | 551.0 | 69500 | 104700 | 2685000 |
| LEVEL41 | 564.5 | 72600 | 108800 | 2754000 |
| LEVEL42 | 578.0 | 74900 | 111600 | 2819000 |
| LEVEL43 | 591.5 | 77100 | 114600 | 2873000 |
| LEVEL44 | 605.0 | 79400 | 117600 | 2926000 |
| LEVEL45 | 618.5 | 81700 | 120500 | 2976000 |

| Floor | Height Above 'LEVEL 1-PLAZA' (ft) | Fx (lb) | Fy (lb) | Mz (lb-ft) |
|---|---|---|---|---|
| LEVEL46 | 632.0 | 83900 | 123400 | 3028000 |
| LEVEL47 | 645.5 | 86100 | 126400 | 3079000 |
| LEVEL48 | 659.0 | 88000 | 128800 | 3124000 |
| LEVEL49 | 672.5 | 90000 | 131400 | 3171000 |
| LEVEL50 | 686.0 | 92200 | 134300 | 3220000 |
| LEVEL51 | 699.5 | 95600 | 138800 | 3288000 |
| LEVEL52 | 713.0 | 94000 | 136500 | 3249000 |
| LEVEL53 | 726.5 | 89300 | 130100 | 3172000 |
| LEVEL54 | 740.0 | 90600 | 131800 | 3214000 |
| LEVEL55 | 753.5 | 92500 | 134100 | 3251000 |
| LEVEL56 | 767.0 | 94300 | 136500 | 3291000 |
| LEVEL57 | 780.5 | 96200 | 138800 | 3330000 |
| LEVEL58 | 794.0 | 98000 | 141200 | 3359000 |
| LEVEL59 | 807.5 | 99700 | 143500 | 3388000 |
| LEVEL60 | 821.0 | 103000 | 147800 | 3428000 |
| LEVEL61 | 834.5 | 104600 | 150100 | 3447000 |
| LEVEL62 | 848.0 | 106800 | 153100 | 3482000 |
| LEVEL63 | 861.5 | 107600 | 154400 | 3485000 |
| LEVEL64 | 875.0 | 109000 | 156600 | 3495000 |
| LEVEL65 | 888.5 | 110300 | 158700 | 3513000 |
| LEVEL66 | 902.0 | 110300 | 157900 | 3513000 |
| LEVEL67 | 915.5 | 116400 | 165900 | 3513000 |
| LEVEL68 | 930.5 | 243600 | 344200 | 8223000 |
| LEVEL69 | 951.5 | 161300 | 235500 | 5341000 |
| LEVEL70 | 965.5 | 101600 | 147300 | 1585000 |
| LEVEL71 | 983.0 | 93700 | 135000 | 1843000 |
| **Total** | | **4.65E+06** | **7.08E+06** | **1.72E+08** |

## Notes:

1. The loads given in this table should be used with the load combination factors given in Table 4c.
2. The loads given in this table are centered about the reference axis shown in Figure 4.
3. The above loads correspond to a 50-year return period basic wind speed (3-second gust) of 98 mph.

CONSULTING ENGINEERS
& SCIENTISTS

**Table 4a: Recommended Wind Load Combinations Factors**

**Configuration 1**

| Load Case | Recommended Wind Load Combination Factors for Simultaneous Application of Loads in Table 3a | | |
|---|---|---|---|
| | X Forces (Fx) | Y Forces (Fy) | Torsion (Mz) |
| 1 | +95% | +45% | +50% |
| 2 | +95% | +45% | -50% |
| 3 | +95% | -55% | +55% |
| 4 | +95% | -50% | -50% |
| 5 | -100% | +45% | +50% |
| 6 | -100% | +45% | -50% |
| 7 | -100% | -50% | +60% |
| 8 | -100% | -40% | -50% |
| 9 | +45% | +100% | +30% |
| 10 | +45% | +100% | -45% |
| 11 | +30% | -95% | +50% |
| 12 | +30% | -95% | -35% |
| 13 | -30% | +100% | +30% |
| 14 | -30% | +100% | -45% |
| 15 | -40% | -95% | +50% |
| 16 | -40% | -95% | -35% |
| 17 | +30% | +30% | +100% |
| 18 | +40% | +50% | -95% |
| 19 | +30% | -60% | +100% |
| 20 | +40% | -30% | -95% |
| 21 | -40% | +30% | +100% |
| 22 | -30% | +50% | -95% |
| 23 | -40% | -60% | +100% |
| 24 | -30% | -30% | -95% |

## Notes:

1. Load combination factors have been produced through consideration of the structure's response to various wind directions, modal coupling, correlation of wind gusts, and the directionality of strong winds in the local wind climate.

**Table 4b: Recommended Wind Load Combinations Factors**

**Configuration 2**

| Load Case | Recommended Wind Load Combination Factors for Simultaneous Application of Loads in Table 3b | | |
|---|---|---|---|
| | X Forces (Fx) | Y Forces (Fy) | Torsion (Mz) |
| 1 | +95% | +50% | +65% |
| 2 | +95% | +50% | -30% |
| 3 | +95% | -65% | +65% |
| 4 | +95% | -65% | -30% |
| 5 | -100% | +50% | +55% |
| 6 | -100% | +50% | -30% |
| 7 | -100% | -60% | +60% |
| 8 | -100% | -55% | -30% |
| 9 | +50% | +100% | +30% |
| 10 | +50% | +100% | -35% |
| 11 | +60% | -95% | +60% |
| 12 | +60% | -95% | -35% |
| 13 | -30% | +100% | +30% |
| 14 | -30% | +100% | -35% |
| 15 | -45% | -95% | +60% |
| 16 | -45% | -95% | -35% |
| 17 | +35% | +30% | +100% |
| 18 | +45% | +45% | -85% |
| 19 | +35% | -65% | +100% |
| 20 | +40% | -40% | -85% |
| 21 | -40% | +30% | +100% |
| 22 | -50% | +40% | -85% |
| 23 | -40% | -65% | +100% |
| 24 | -50% | -40% | -85% |

## Notes:

1. Load combination factors have been produced through consideration of the structure's response to various wind directions, modal coupling, correlation of wind gusts, and the directionality of strong winds in the local wind climate.

CONSULTING ENGINEERS
& SCIENTISTS

**Table 4c: Recommended Wind Load Combinations Factors**

**Configuration 3**

| Load Case | Recommended Wind Load Combination Factors for Simultaneous Application of Loads in Table 3c | | |
|---|---|---|---|
| | X Forces (Fx) | Y Forces (Fy) | Torsion (Mz) |
| 1 | +95% | +30% | +30% |
| 2 | +95% | +30% | -30% |
| 3 | +95% | -50% | +35% |
| 4 | +95% | -45% | -30% |
| 5 | -100% | +30% | +50% |
| 6 | -100% | +30% | -60% |
| 7 | -100% | -60% | +50% |
| 8 | -100% | -60% | -60% |
| 9 | +30% | +75% | +30% |
| 10 | +30% | +75% | -30% |
| 11 | +55% | -100% | +30% |
| 12 | +55% | -100% | -30% |
| 13 | -50% | +75% | +30% |
| 14 | -50% | +75% | -30% |
| 15 | -30% | -100% | +30% |
| 16 | -30% | -100% | -30% |
| 17 | +40% | +30% | +75% |
| 18 | +30% | +30% | -100% |
| 19 | +40% | -60% | +75% |
| 20 | +30% | -30% | -100% |
| 21 | -45% | +30% | +75% |
| 22 | -45% | +30% | -100% |
| 23 | -45% | -60% | +75% |
| 24 | -45% | -30% | -100% |

## Notes:

1. Load combination factors have been produced through consideration of the structure's response to various wind directions, modal coupling, correlation of wind gusts, and the directionality of strong winds in the local wind climate.