LERA

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17 November 2014
File: P933

Mr. Peter Donohoe
Vice President - Construction
Rockrose Development Corp.
666 Fifth Avenue, 5th Floor
New York, NY 10103

Via e-mail: peter.donohoe@rockrose.com and mail

43-22 Queens Street
Superstructure Permit Application
Structural Peer Review

Dear Mr. Donohoe:

At the request of Rockrose Development Corp., Leslie E. Robertson Associates, R.L.L.P. has conducted a Structural Peer Review of the structural design of 43-22 Queens Street as required by New York City Building Code Section 1627. This report summarizes the extent and findings of our review.

We have reviewed the following:

- Plans listed in Appendix A.
- *Report Geotechnical Investigation, Eagle Warehouse Site, 43-22 Queens Street, Long Island City, NY,* dated *Revised May 30, 2014*, by RA Consultants LLC. Pages 1 to 10 are attached to this report as Appendix B.
- Structural Design Criteria shown in Drawing FO-001.01 dated XX-XX-14. A copy is attached as Appendix C.
- Preliminary Results, Wind Induced Structural Responses, Eagle Warehouse, New York City, NY, dated 25 April 2014 by Rowan Williams Davies & Irwin, Inc. Refer to Appendix D.

Through our review, we have confirmed the following aspects of the structural design, as required by Section 1627.6.1:

- the design loads conform to the Building Code;
- the design criteria and design assumptions conform to the Building Code;
- the design properly incorporates the recommendations of the geotechnical engineer;
- the design properly incorporates the preliminary recommendations of the wind tunnel laboratory;

- the structure has a complete load path;
- based on our independent calculations of representative structural components, we find that the design of the components have adequate strength;
- the structural plans are in general conformance with the architectural plans regarding loads and other conditions that affect the structural design; and
- the structural plans are generally complete.

Accordingly, we find the design of the structure to be in general conformance with the structural design provisions of the Building Code.

In addition to new building components not required to be reviewed by Section BC1627 of the code, the following aspects of the design have not been reviewed:

- The effect of the new foundation loads and construction on adjacent buildings.
- The design of underpinning of adjacent buildings.

The opinions expressed in this letter represent our professional view, based on the information made available to us. In developing these opinions, we have exercised a degree of care and skill commensurate with that exercised by professional engineers licensed in the State of New York for similar types of projects. No other warranty, expressed or implied, is made as to the professional advice included in this letter.

Regards,
LESLIE E. ROBERTSON ASSOCIATES, RLLP

William J. Faschan

WJF/[illegible]

cc: Mr. Matthew Burton, WSPCS via e-mail: matthew.burton@wspcs.com

STRUCTURAL PEER REVIEW STATEMENT

This structural peer review and report, dated 17 November 2014, is complete for 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: 43-22 Queens Street, Long Island City, Block #266, Lot 3

Department Application Number for Structural Work: #420651823

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 43-22 Queens Street, Block #266, Lot 3, Application #420651823 and found that the structural design shown on the plans and specifications generally conforms to the 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 Review only)

Name William J. Faschan

Signature

Date 11/17/14

STATE OF NEW YORK
WILLIAM J. FASCHAN
058672
LICENSED PROFESSIONAL ENGINEER

Cc: Project Owner: Peter Donohoe
Project Registered Design Professional: Matthew Burton

APPENDIX A

43-22 QUEENS STREET PEER REVIEW

STRUCTURAL DRAWING LIST

DRAWING NUMBER	DRAWING TITLE	REV	DATE
FO-001.00	General Notes, Legends and Abbreviations	3	XX-XX-2014
FO-100.00	Foundation (1st Floor) Framing Plan	2	08-29-2014
FO-200.00	Foundation Typical Details 1	3	08-29-2014
FO-201.00	Foundation Typical Details 2	3	08-29-2014
FO-202.00	Foundation Typical Details 3	3	08-29-2014
FO-300.00	Foundation Sections 1	2	08-29-2014
FO-301.00	Foundation Sections 2	2	08-29-2014
S-010.00	1st Floor Overall Framing Plan	3	08-29-2014
S-020.00	2nd Floor Overall Framing Plan	2	08-29-2014
S-021.00	2nd Floor Framing Part Plan	2	08-29-2014
S-030.00	3rd Floor Plan	2	08-29-2014
S-031.00	3rd to 6th Floor Framing Part Plan	2	08-29-2014
S-040.00	4th Floor Overall Framing Plan	2	08-29-2014
S-050.00	5th Floor Overall Framing Plan	2	08-29-2014
S-060.00	6th Floor Plan	2	08-29-2014
S-070.00	7th Floor Overall Framing Plan	2	XX-XX-2014
S-071.00	7th Floor Framing Plan Part 1	2	08-29-2014
S-072.00	7th Floor Framing Plan Part 2	2	08-29-2014

DRAWING NUMBER	DRAWING TITLE	REV	DATE
S-073.00	7th Floor Framing Plan Part 3	2	08-29-2014
S-080.00	8th Floor Framing Plan	2	08-29-2014
S-090.00	9th Floor Framing Plan	2	08-29-2014
S-110.00	10th-19th Floor Framing Plan	2	08-29-2014
S-200.00	20th Floor Framing Plan	2	08-29-2014
S-210.00	21st Floor Framing Plan	2	08-29-2014
S-220.00	22nd-34th Floor Framing Plan	2	08-29-2014
S-350.00	35th Floor Framing Plan	2	08-29-2014
S-360.00	36th Floor Framing Plan	2	08-29-2014
S-370.00	37th-54th Floor Framing Plan	2	08-29-2014
S-470.00	47th-54th Floor Framing Plan	1	XX-XX-2014
S-550.00	Main Roof and Bulkhead Framing Plans	1	08-29-2014
S-940.00	Shearwall Reinforcement Plan (Fnd-29th FL)	3	08-29-2014
S-941.00	Shearwall Reinforcement Plan (30th-Roof)	1	08-29-2014
S-945.00	Typical Shearwall Details	3	08-29-2014
S-950.00	Column Schedule	3	08-29-2014
S-951.00	Typical Column Details	3	08-29-2014
S-960.00	Typical Superstructure Details 1	3	08-29-2014
S-961.00	Typical Superstructure Details 2	3	08-29-2014
S-962.00	Typical Superstructure Details 3	3	08-29-2014
S-963.00	Typical Superstructure Details 4	2	08-29-2014
S-965.00	Typical Masonry Details	3	08-29-2014
S-970.00	Superstructure Sections	2	08-29-2014
S-975.00	Superstructure Sections 2	1	08-29-2014
S-980.00	Typical Stair Details	3	08-29-2014

APPENDIX B

43-22 QUEENS STREET

Geotechnical Investigation Report

RA Consultants LLC
Geotechnical Engineering

Walter J. Papp, Jr., Ph.D, P.E.
Senior Partner

Nidal M. AbiSaab, P.E.
Partner

Robert Alperstein, P.E.
Consultant

May 8, 2014
Revised May 30, 2014

13C1164

Rockrose Development Corporation
666 Fifth Ave, 5th Floor
New York, NY 10103

Attn: Allen Dzbanek

re: Report
Geotechnical Investigation
Eagle Warehouse Site
43-22 Queens Street
Long Island City, NY

Dear Mr. Dzbanek:

This report is submitted in general accordance with our agreement dated November 15, 2013. It covers a geotechnical investigation related to the proposed high-rise tower (approximately 50-stories), and renovations/additions to the existing buildings and/or areas of new low- to mid-rise construction at the referenced address.

The site consists of Lots 3, 16, 20 and 21, Block 266 in the Court Square area of Long Island City, Queens, NY. The irregular shaped site is bound between Queens Street to the east, Dutch Kills Street to the west, MTA/AMTRAK (Sunny Side Yards) to the south and Jackson Avenue (fronting lot 20 and 21) to the north. The total area of the site is approximately 76,000-sqft. Existing low rise buildings occupy the north property line along Jackson Avenue. The NYCT subway tunnel for the E, M and R lines lies below Jackson Street. We estimate part of the proposed development will be within 200-ft of MTA/AMTRAK (Sunny Side Yards) and the NYCT structure and will require their approval or letter of no impact for construction.

The warehouse slab level is el 14.3 North American Vertical Datum of 1988 (NAVD88). Sidewalk grades along Queens Street and Dutch Kills Street increase from south to north, ranging from el. 11.6- to 13.6 and el. 9.6- to 11.6, respectively.

Subsurface data in the area from nearby projects suggested that the site may be underlain by 10-ft of uncontrolled fill followed by glacial deposits with bedrock approximately 25-ft below sidewalk level. Data from several of our projects in the area suggested that the bedrock surface elevation could be highly variable. Groundwater was expected to be within 10-ft of the sidewalk level. Our investigation generally confirmed available data with slight variations as discussed below.

Eighteen borings were drilled within the building footprint and in the sidewalk adjacent to the site using DK50 Drilling Rigs and a Portable Electric Drilling Rig. Monitoring wells were installed in two completed borings.

PURPOSE AND SCOPE OF SERVICES

The purpose of the geotechnical investigation was to obtain subsurface data at the site and to provide recommendations for design and construction of foundations and related geotechnical aspects of the project based on the data obtained.

You engaged Warren George Inc. (WGI) to drill eighteen borings and Coffey Contracting to excavate 7 test pits.

We provided the following services:

1. Prepared a proposed boring location plan for submittal and approval by the NYCT Outside Projects Division (presented in Appendix B).
2. Observe the drilling operations to log samples in the field.
3. Observed and logged the test pits.
4. Evaluate the data and submitted this report.

INVESTIGATION

Borings

Eighteen borings were drilled for this investigation by WGI at the approximate locations shown in Figure 1. The borings were drilled between March 7th and April 7th, 2014. The borings were advanced using rotary drilling. Variable lengths of steel casing were used to stabilize the upper portions of the borings, as necessary. Samples were obtained generally at 5-ft depth intervals by the Standard Penetration Test (SPT) method (ASTM D 1586). A donut hammer was used for the SPT. Upon encountering N-values generally exceeding 100-blows/ft (or as indicated by the driller's "feel" of the drill tools) an NX-size diamond bit, double tube core barrel was used to retrieve rock core. Core recovery and Rock Quality Designation (RQD) as a percentage of the run were determined and recorded.

Monitoring wells were installed in completed borings B-1W and B-7W; and groundwater level measurements were taken immediately and one day after wells were installed. Each well consisted of 1-1/4-in diameter PVC riser pipe with the lower 10-ft section slotted. The annulus between the borehole and monitoring well was backfilled with silica sand.

The drilling operation was observed and boring samples were logged in the field by our Mr. John Lorenz. The boring logs are presented in Appendix A.

Percolation tests were performed in Borings Nos. B-17 and B-18 as requested during the investigation. Data from the test and estimate permeability of the soil is presented in Appendix D.

Several concrete cores were made through the first floor slab at locations selected by the structural engineer. The core locations and thickness of the concrete slab at those locations are presented in Appendix E.

<u>Test Pits</u>

Seven test pits were excavated for this investigation by Coffey Contracting at the approximate locations shown in Figure 1, between March 6th and March 19th, 2014. The test pits were excavated using a small excavator and hand tools. They were braced by timber lagging where necessary. The excavation was observed and documented by our Mr. John Lorenz. The test pit logs are presented in Appendix B.

TP-1a and TP-1b were excavated in the center of the building footprint approximately 60-ft west of Queens Street. The two test pits indicated that the adjacent column has a concrete footing extending approximately 9-ft below ground surface bearing on class 1b bedrock.

TP-2a and TP-2b were excavated in the building footprint approximately 40-ft east of TP-1a and TP-1b. The test pits indicated that the adjacent column has a concrete footing extending approximately 11-ft below ground surface and is bearing on 2- to 3-ft thick Fill layer (possibly Till), underlain by bedrock (class 1b per NYCBC).

TP-3 was excavated adjacent to a column along the east side wall of the building, approximately 20-ft east of TP-2a and TP-2b. The test pit indicates that the adjacent column has a footing extending approximately 11-ft below ground surface bearing on class 1b bedrock.

TP-4 was excavated adjacent to a column within the building footprint approximately 20-ft west of TP-1a and TP-1b. The test pits indicated that the adjacent column has a concrete footing extending approximately 10.5-ft below ground surface and is bearing on class 1b bedrock.

TP-5 was excavated adjacent to a column along the same line as the other six test pits, approximately 20-ft west of TP-4. The test pit indicates that the adjacent column has a concrete footing extending approximately 8-ft below ground surface and is bearing on Silt.

SUBSURFACE CONDITIONS

Subsurface strata as generalized from the boring data and increasing with depth below ground surface may be summarized as follows:

Fill: Fill, generally consisting of a mixture of gravel, sand, and construction debris extended to about 3.5- to 13-ft below ground surface (about el 10 to 1 NAVD88). The fill is uncontrolled and is classified as class 7 in accordance with the NYCBC. The N-values varied from 5- to 36-blows/ft with two samples requiring more than 100-blows/ft (refusal).

Silt: Silt, where encountered, was found below the Fill layer with thickness varying from 4.5- to 7.5-ft and N-values ranging from 8- to 54-blows/ft. It consisted of brown silt with sand (ML per USCS and class 5a, 5b, and 6 per NYCBC).

Clay: Clay appeared to be present in a local area along the west side of the site. However, it could be present elsewhere. Where present, it was found below Fill or Silt layers with its thickness varying from 10- to 40-ft and N-values ranging from 9- to 55-blows/ft with one sample recorded at 100-blows/ft. It consisted of brown to gray clay with varying percentages of silt and sand (CL and CL-ML per USCS and class 4a and 4b per NYCBC).

Varved Silt: Varved Silt appeared to be present in a local area in the southwest corner of the site. However, it could be present elsewhere. Where present, it was found below Fill or Clay layers with its thickness varying 5- to 16-ft and its N-values ranging from 9- to 21-blows/ft with one sample recorded at 61-blows/ft. It consisted of gray varved silt with clay (ML per USCS and class 6 per NYCBC).

Till: Glacial deposit was encountered below Fill, Silt, Clay, or Varved Silt layers with thickness varying from 5.5- to 16.5-ft and N-values ranging from 13- to 100-blows/ft. It consisted of brown sand with varying percentages of silt and gravel (SM and SW per USCS, class 3a and 3b per NYCBC).

Rock: Bedrock (class 1a and 1b per NYCBC) was encountered at variable depths and elevations across the site, ranging from el 7 to el -41.5 (NAVD88). It is predominantly schistose gneiss, varying mostly from hard sound rock to medium hard rock. The core recoveries and RQD's typically exceeded 80% and 70% respectively.

Groundwater: Groundwater measurements were made in monitoring wells B-1W and B-7W. The measurements are shown on the boring logs indicating stabilized groundwater level varying from about el 2.8 (NAVD88). Groundwater was encountered in the test pits at el. 3.3.

The groundwater levels should be expected to vary with seasonal precipitation as well as long term variations of the nearby East River and other unknown factors.

EVALUATION AND RECOMMENDATIONS

We understand that the existing warehouse building will be renovated except for the northeast quadrant where it will be demolished to make way for the proposed tower. We also understand that the existing basement space in the south about quarter of the building will remain and no new below grade space will be created.

Foundations

The foundations should bear on or in bedrock because of anticipated high column loads associated with a 50-story building. Top of bedrock within the footprint of the proposed tower varies between 10- to 28-ft depth (el. 3.5 and -14.5) below present slab level.

At the test pit locations, the warehouse columns are generally founded on deep piers to bedrock. The perimeter walls appear to bear on continuous shallow foundations in natural soils.

Shallow Foundations:

Shallow foundations would be appropriate where bedrock is within 15-ft of the existing basement slab. They should be designed for an allowable bearing capacity of 40-tsf. This will require excavating in tight timber sheeted pits and dewatering (discussed later). With bedrock encountered at a maximum of 28-ft depth (el. -14.5) and groundwater at about el 2.5, shallow foundations to rock may be impractical.

Settlements for foundations bearing on bedrock should be negligible.

Deep Foundations:

Driven Piles

Driven piles will cause vibrations that could densify the soils below the existing warehouse and nearby footings, leading to potential settlement. Further, considering the small tower footprint and limited access, in our opinion, driven piles driving would be inappropriate.

Drilled Caissons

Drilled caissons socketed into rock would be appropriate deep foundations that would minimize vibrations during installation. Experience indicates that these likely would be acceptable to NYC Transit, AMTRAK and MTA. They should be installed using internal flush duplex drilling with water as the drilling fluid and sealed into the rock. A down-the-hole-hammer may be used only to excavate the rock socket.

The caissons should be designed in accordance with NYC Building Code requirements and the rock socket may be designed using an allowable side shear of 200- lbs/in^2. End bearing should be neglected in the design for caissons having a diameter less than 24-in. Caissons smaller than 18-in in diameter sometimes are referred to as "mini-caissons" although the NYC Building Code makes no distinction regarding caisson diameter.

Typical allowable caisson designs are shown below:

Typical Caisson Capacities

Design Load [tons]	**Caisson Type**	**Caisson Dimensions**	**Rebar Size & Diameter**	**Rebar Quantity**	**Bond Length in Rock Socket [ft]**
500	Mini Caisson	13-3/8 in x 0.5"	#28 – 3-1/2 in	2	12
1,200+	Caisson	24-in x 0.5"	#28 – 3-1/2	6	15

Allowable lateral load capacities are estimated to be 10-tons and should be verified with a load test. Allowable uplift capacities are likely to be about half or possibly more than the allowable compressive capacities. This is dependent on the rock socket length and structural capacity of the reinforcing bars.

Pile Load Tests

Pile load tests of the mini-caissons or caissons are unnecessary if all of the rock sockets are video inspected by a qualified Geotechnical Engineer.

Floor Slabs

Generally, Fill was encountered below the existing slab. The observed fill was free of deleterious material and generally compact. In our opinion the existing slab, generally 6-in or greater could be reused, this should be confirmed by the structural engineer. New slabs may be designed using a coefficient of subgrade reaction of 75-$tons/ft^3$.

Compacted Fill

Imported materials for use as compacted structural fill should be a mixture of sand and gravel having a maximum particle size of 4-in and less than 12 per cent passing the No. 200 sieve. If the fill will support floor slabs it should be compacted in thin lifts with vibratory rollers, jumping jacks or vibratory plate compactors to a dry density of at least 95 per cent of the maximum dry density obtained in the laboratory with the modified Proctor compaction test (ASTM D 1557). The maximum lift thickness should be 12-in with the vibratory roller. If hand operated compaction equipment is used lift thicknesses should be no greater than 6-in.

Non-structural fill (*e.g.* for courtyard areas or if structural slabs are used in design) should consist of similar materials, but the amount passing the No. 200 sieve could be up to 18 per cent and the required compacted density should be at least 90 per cent of the modified Proctor maximum dry density.

Porous fill below floor slabs on grade should consist of gravel or crushed stone with a maximum particle size of 1-in and zero passing the No. 200 sieve. If the material will be used for long term drainage purposes only natural crushed stone may be used. It may be compacted with at least four passes of a vibratory roller as described earlier, with no field density testing required.

<u>Groundwater Control</u>

<u>During Construction</u>

Groundwater levels were measured at approximately el 1 but depending upon precipitation and severe storm events, water levels are expected to be higher. Excavations for shallow foundations extending to bedrock will require dewatering. We expect that sumps and pumps can handle the expected groundwater flow through the dense glacial soils. In all situations involving sumps, filters (*e.g.* non-woven geotextile liners) should be used to minimize movements of fine soil particles.

<u>After Construction</u>

The finished grade (approximately el 14.3) is above the general 100-yr flood plain level. We understand the existing basement on the south side of the building will be used. The existing grade levels outside the building on the south side are several ft lower than the 100-year flood level. Flood gates should be considered if building openings exist or are considered where the site grade is lower than el. 12.

We recommend a design groundwater level of approximately 3-ft above the measured levels, or approximately el 5.5. The top of the existing slab at basement level (el. 1.6) is at or below the measured groundwater levels. Higher levels could be experienced in the future due to major flooding or site flooding due to water main breaks.

If floor to ceiling heights permits, we recommend leaving the existing cellar slab and install a crushed stone layer above it followed by a wearing slab. Slight leakage through the existing slab would be captured by the crushed stone or gravel layer leading to a sump pit through 4-in diameter perforated pipes. We recommend a vapor barrier between the crushed stone and the new wearing slab.

The crushed stone or gravel should have a maximum particle size of 1-in and zero passing the No. 200 sieve.

The drainage pipes should be perforated 4-in diameter PVC wrapped in a non-woven geotextile spaced about 15-ft apart and pitched to drain to sumps equipped with self-activating pumps having a capacity of at least 10-gal/min. The potential for pipe clogging is minimal because the flow would be through fine cracks in the concrete and the geotextile wrap should prevent entrance of any minor fines into the pipes. Therefore, systematic cleanouts are unnecessary, in our opinion. However, providing at least two access points to the pipes would be prudent.

Elevator pits should be waterproofed and designed to resist uplift due to possible high groundwater levels. A small sump and pump should be provided inside the completed pit to collect and remove seepage.

Excavations and Lateral Support

Temporary open excavation side slopes should be no steeper than 1: 1½ (v:h). Below the water table the side slopes may have to be flattened to 1:3 (v:h) to maintain stability. We anticipate the contractor to use tight sheeted pits and possibly soldier piles and lagging to sheet and shore the local excavations. Where sheeting is used with a single level of bracing the bracing may be designed to resist active earth pressures using a total unit weight of 120-lbs/ft^3 and effective friction angle of 30^o. Where multiple bracing levels are used, a uniform earth pressure distribution should be used with the intensity calculated as 0.65 x the maximum active pressure.

Were bedrock removal is necessary for construction of footings or elevator pits, we recommend slot drilling the perimeter of the new structure prior to excavation.

Permanent basement and pit walls may be designed for the following two conditions:

1. Earth pressures at rest based on a triangular distribution with the earth pressure increasing at a rate of 63-lbs/ft^2/ft of depth above the groundwater level and 94-lbs/ft^2/ft below the groundwater level (includes hydrostatic pressures).

2. Active earth pressures plus seismic pressures. This may be based on a triangular pressure distribution with a seismic earth pressure coefficient of 0.4 and soil unit weight of 125-lbs/ft^3. Pit walls below the groundwater table should consider a soil buoyant unit weight of 63-lbs/ft^3 plus hydrostatic pressures based on the unit weight of water (62.4-lbs/ft^3). Lateral pressures for basement walls need not consider earthquake and major flood occurring simultaneously.

Underpinning

Underpinning of existing and adjacent structures will be necessary if the proposed excavation level is below adjacent foundations and a retaining system cannot be designed to prevent intolerable settlements or lateral movements of the adjacent structures. Typically, underpinning may be required if the excavation bottom lies below an influence line of approximately 1:1½ (v:h) drawn from the bottom of the adjacent foundation to the bottom of the proposed excavation. The contractor should verify the existing foundation elevations in the field before proceeding with mass excavation.

Underpinning should extend to competent materials and to at least about 6-in below the adjacent excavation and should be constructed in the dry. Groundwater control, if necessary, could be controlled with sumps and pumps. As discussed above we anticipate that groundwater should be below the proposed basement grade. Tight sheeting or lagging should be used in excavating the underpinning pits to minimize movement of fines from beneath adjacent footings or floor slabs. Excavation for each lagging board should extend no deeper than 6-in below the bottom of the lagging board.

Steel wedges or jacking should be used to transfer the foundation loads to the underpinning. Slight settlements of underpinned structures should be expected during the underpinning process. These movements can be minimized by use of jacking if the underpinning is supported by soil.

The underpinning should be designed to resist lateral earth pressures as well as the vertical foundation loads. Therefore lateral bracing or tiebacks will be required.

Potential Effects of Construction on Adjacent Building

Adjacent Buildings

The existing and adjacent buildings may experience slight vibrations during excavation due to normal movement of construction vehicles and due to construction of caissons.

As discussed above underpinning may be required for the existing and adjacent buildings abutting the site. Slight adjustment-type settlements of the structures may occur, possibly with resulting cosmetic damages. This is normal, but care should be taken to minimize potential settlements by utilization of appropriate underpinning design and construction techniques such as jacking, tight lagging, and minimal lift thicknesses as discussed above.

A precondition survey of adjacent buildings should be undertaken prior to construction. The adjacent buildings should be monitored for settlement and lateral movement during construction. The retaining structures supporting the excavation should also be monitored during construction. Visual observations should be taken daily for cracks in adjacent buildings, pavements, sidewalks, local settlements, etc.

These activities will help to protect against unjustified claims and to provide documented information for negotiating legitimate concerns.

Subway Tunnel

We understand that the building fronting Jackson Avenue will be renovated and new excavations will be unnecessary. We anticipate no effects on the subway from the proposed construction. The Transit Authority will review the support of excavation and foundation drawings prior to DOB approval. Due to the significant distance between the proposed tower and the tunnel we expect they will issue a letter of no impact. We recommend scheduling a meeting with the TA Outside Projects Group and MTA/AMTRAK to discuss the proposed building and obtain their feedback early during the design.

<u>Seismic Considerations</u>

The proposed tower will be partially supported on piers to bedrock and the remainder founded on high capacity drilled caisson and majority of the exiting foundations bearing on bedrock. The site may be classified as Class C “Very dense soil and soft rock profile” in accordance with NYCBC Table 1615.1.1 (Site Class Definition). No potentially liquefiable soils below the groundwater level were encountered and liquefaction need not be considered for design.

LIMITATIONS

The recommendations presented herein are based on our evaluation of the subsurface conditions as disclosed by the 18 borings drilled and seven test pits excavated for this investigation and our understanding of the project as described above. If subsurface conditions are found to differ from those described above or if project conditions change we should be notified and requested to modify our recommendations as necessary.

We appreciate this opportunity to be of service and look forward to working with you as the project proceeds.

Very truly yours,
RA CONSULTANTS LLC

STATE OF NEW YORK
WALTER JOSEPH PAPP JR.
No. 084812
LICENSED PROFESSIONAL ENGINEER

Walter J. Papp, Jr., P.E.

APPENDIX C

43-22 QUEENS STREET

Design Criteria

ABBREVIATIONS:

AB	ANCHOR BOLT
ABV	ABOVE
AC	AIR CONDITIONER
ACI	AMERICAN CONCRETE INSTITUTE
ADD'L	ADDITIONAL
ADJ	ADJACENT
AFF	ABOVE FINISHED FLOOR
AISC	AMERICAN INSTITUTE OF STEEL CONSTRUCTION
ALT	ALTERNATE ALUM ALUMINUM
ANCH	ANCHOR
ANG	ANGLE
APPD	APPROVED
APPROX	APPROXIMATE
ARCH	ARCHITECTURAL
ASTM	AMERICAN SOCIETY FOR TESTING AND MATERIALS
AVG	AVERAGE
AWS	AMERICAN WELDING SOCIETY
B	BASE
BETW	BETWEEN
BF	BRACE FRAME
BKT	BRACKET
BL	BUILDING LINE
BLDG	BUILDING
BM	BEAM
BOTT	BOTTOM
BRK	BRICK
B/STL	BOTTOM OF STEEL
BS	BOTH SIDES
CANT	CANTILEVER
CF	CUBIC FOOT
CG	CENTER OF GRAVITY
CIP	CAST IN PLACE
C.JT	CONCRETE JOINT
CL	CENTER LINE
CLG	CEILING
CLR	CLEAR
CM	CONSTRUCTION MANAGER
CMU	CONCRETE MASONRY UNITS
COL	COLUMN
CONC	CONCRETE
COND	CONDITIONS
CONN	CONNECTION
CONST	CONSTRUCTIONS
CONT	CONTINUOUS
CONTR	CONTRACTOR
COORD	COORDINATE
CORR	CORRUGATED
CY	CUBIC YARD
DEMO	DEMOLITION
DEPT	DEPARTMENT
DET	DETAIL
DIA	DIAMETER
DIM	DIMENSION
DIR	DIRECTION
DN	DOWN
DWL	DOWEL
DWG	DRAWING
E	EAST
EA	EACH
EF	EACH FACE
EL	ELEVATION
ELEC	ELECTRIC
ELEV	ELEVATOR
EMBD	EMBEDMENT
ENCL	ENCLOSURE
EOR	ENGINEER OF RECORD
EOS	EDGE OF SLAB
EP	EMBEDDED PLATE
EQ	EQUAL
EQUIP	EQUIPMENT
ETC	ETCETERA
EW	EACH WAY
E-W	EAST WEST
EXIST	EXISTING
EXP	EXPANSION
EXP JT	EXPANSION JOINT
EXT	EXTENSION
EXTR	EXTERIOR
FL	FLOOR
FND	FOUNDATION
FOB	FACE OF BUILDING
FP	FIRE PROOFING
FT	FOOT
FTG	FOOTING
GA	GAUGE
GALV	GALVANIZED
GC	GENERAL CONTRACTOR
GB	GRADE BEAM
GRTG	GRATING
GYP. BD	GYPSUM BOARD
H	HIGH
HDR	HEADER
HGT	HEIGHT
HORIZ	HORIZONTAL
HP	HIGH POINT
HR	HOUR
HS	HIGH STRENGTH
HVAC	HEAT, VENTILATION & AIR CONDITIONING
ID	INSIDE DIAMETER
IF	INTERIOR FACE
IN	INCH
INCL	INCLUDING
INFO	INFORMATION
INSUL	INSULATION
JT	JOINT
K	KIP (1000 POUNDS)
KPF	KIPS PER SQUARE FOOT
KSI	KIPS PER SQUARE INCH

L	LOW
LB	LINK BEAM
LBS	POUNDS
LB/FT	POUNDS PER FOOT
Ld	DEVELOPMENT LENGTH
LG	LONG
LL	LIVE LOAD
LLRS	LATERAL LOAD RESISTING SYSTEM
LP	LOW POINT
LRFD	LOAD RESISTANCE FACTOR DESIGN
LT	LIGHT
LW	LIGHT WEIGHT
MAX	MAXIMUM
MAS	MASONRY
MATER	MATERIAL
MAX	MAXIMUM
MC	MOMENT CONNECTION
MD	METAL DECK
MECH	MECHANICAL
MEP	MECHANICAL ELECTRICAL AND PLUMBING
MEZZ	MEZZANINE
MF	MOMENT FRAME
MFG	MANUFACTURER
MIN	MINIMUM
MISC	MISCELLANEOUS
N	NORTH
NA	NOT APPLICABLE
NIC	NOT IN CONTRACT
NO	NUMBER
N-S	NORTH-SOUTH
NTS	NOT TO SCALE
NW	NORMAL WEIGHT
O/C	ON CENTER
OPNG	OPENING
OPP	OPPOSITE
PC	PILE CAP
PCF	POUNDS PER CUBIC FEET
PL	PLATE
PLF	POUNDS PER LINEAR FOOT
PSF	POUNDS PER SQUARE FOOT
PSI	POUNDS PER SQUARE INCH
PT	POST TENSION
RC	REINFORCED CONCRETE
RD	ROOF DRAIN
REF	REFERENCE
REINF	REINFORCEMENT
REQ'D	REQUIRED
RFI	REQUEST FOR INFORMATION
S	SOUTH
SB	SPANDREL BEAM
SCHED	SCHEDULE
SECT	SECTION
SF	SQUARE FOOT
SHT	SHEET
SL	SLAB
SPA	SPACING
SPEC	SPECIFICATIONS
SQ	SQUARE
SR	STUDRAIL
STD	STANDARD
STIFF	STIFFENER
STL	STEEL
STRUCT	STRUCTURAL
SW	SHEARWALL
SIM	SIMILAR
T&B	TOP AND BOTTOM
THK	THICK
T/	TOP OF
TBD	TO BE DETERMINED
TEMP	TEMPORARY
TSF	TONS PER SQUARE FOOT
TYP	TYPICAL
UON	UNLESS OTHERWISE NOTED
U	UPTURNED BEAM
VERT	VERTICAL
VIF	VERIFY IN FIELD
W	WEST
W/	WITH
W/O	WITH OUT
WF	WIDE FLANGE
WP	WORKING POINT
WPG	WATER PROOFING
WS	WATER STOP
WT	WIND TRUSS
WWF	WELDED WIRE FABRIC
℄	CENTERLINE
⅊	PLATE
⊥	ANGLE
&	AND
ø	DIAMETER
@	AT

GENERAL NOTES:

1. ALL WORK TO BE PERFORMED IN COMPLIANCE WITH THE NEW YORK CITY BUILDING CODE, LATEST EDITION AND ALL SUPPLEMENTS.
2. CONTRACTOR SHALL VERIFY ALL CONDITIONS AND DIMENSIONS IN THE FIELD AND BE RESPONSIBLE FOR ACCURATE COORDINATION WHERE POSSIBLE. EXISTING FRAMING DIMENSIONS WERE TAKEN FROM EXISTING DWGS. AND SHALL BE VERIFIED ON SITE. DISCREPANCIES SHALL BE REPORTED TO ARCH. AND ENGINEER BEFORE PROCEEDING.
3. TEMPORARY SHORING IS REQUIRED AT ALL LOCATIONS WHERE PARTIAL REMOVAL OF BEAMS IS REQUIRED. CONTRACTOR IS RESPONSIBLE FOR ENGINEERING AND CONTROLLED INSPECTION OF TEMPORARY SYSTEMS.
4. THE CONTRACTOR SHALL USE THESE DRAWINGS IN CONJUNCTION WITH THE ARCHITECTURAL AND MECHANICAL DRAWINGS. IN THE EVENT OF CONFLICTS, THE CONTRACTOR SHALL NOTIFY THE ARCHITECT AND THE ENGINEER.
5. ALL UNDERPINNING, SHEETING, SHORING OR OTHER CONSTRUCTION REQUIRED FOR THE SUPPORT OF ADJACENT PROPERTIES, BUILDINGS, SIDEWALKS, UTILITIES, ETC., SHALL BE SUBJECT TO SPECIAL INSPECTION AS REQUIRED BY THE CODE. THE CONTRACTOR SHALL RETAIN A LICENSED PROFESSIONAL ENGINEER ACCEPTABLE TO THE ENGINEER OF RECORD TO PROVIDE THE NECESSARY DESIGN AND THE REQUIRED INSPECTION. THE CONTRACTOR'S PROFESSIONAL ENGINEER SHALL PREPARE AND FILE THE REQUIRED FORMS FOR THE WORK WITH THE BUILDING DEPARTMENT.

FOUNDATION NOTES:

1. THE STRUCTURE TO BE SUPPORTED ON THE DRILLED-IN CAISSON AS RECOMMENDED BY LATEST GEOTECHNICAL REPORT.
2. WHERE EXISTING FOOTING OR FOUNDATIONS OF ADJACENT PROPERTY IS LOWER THAN ELEVATIONS SHOWN, NEW FOUNDATIONS ARE TO BE LOWERED TO SAME ELEVATION. WHERE NEW FOUNDATION IS LOWER THAN EXISTING FOUNDATION CONTRACTOR IS TO UNDERPIN EXISTING FOUNDATION. CONTRACTOR IS TO ESTABLISH EXISTING CONDITIONS BEFORE COMMENCING WORK AND NOTIFY THE ENGINEER.
3. DESIGN IS PERFORMED IN ACCORDANCE WITH "RA CONSULTANTS" PRELIMINARY GEOTECHNICAL REPORT FINAL DRAWINGS WILL BE ISSUED ONLY AFTER THE FINAL GEOTECHNICAL REPORT IS ISSUED, ANALYZED AND IMPLEMENTED. NO CONSTRUCTION IS ALLOWED UNTIL THIS EXERCISE IS COMPLETED.
4. VERTICAL CONSTRUCTION JOINTS IN ALL FOUNDATION WALLS SHALL BE USED ONLY IF UNAVOIDABLE, OR UNLESS OTHERWISE NOTED, AND TO BE LOCATED AT LEAST 4'-0" FROM ANY SUPPORTING COLUMN OR WALL OPENING. DISTANCE BETWEEN JOINTS IN WALL SHALL BE ALLOWED AS PER SPECIFICATIONS. NO HORIZONTAL CONSTRUCTION JOINTS WILL BE ALLOWED IN GRADE BEAMS.
5. IN NO CASE SHALL TRUCKS, BULLDOZERS, OR OTHER HEAVY EQUIPMENT BE PERMITTED CLOSER THAN 8'-0" FROM ANY FOUNDATION WALL UNLESS APPROVED BY THE ENGINEER.
6. TEMPORARY BRACING SHALL BE PROVIDED FOR ALL BUTTRESSES. WHERE BUTTRESSES DO NOT EXIST OR SPACING BETWEEN BUTTRESSES EXCEED 25 FEET, AND WHERE THE DIFFERENCE IN LEVEL BETWEEN INSIDE AND OUTSIDE GRADE IS MORE THAN 4'-0", INTERMEDIATE BRACING SHALL BE PROVIDED. WHERE RAMPS OCCUR, THE GRADE ELEVATION OUTSIDE OF RAMP WALLS SHALL BE USED IN FIGURING THE DIFFERENCE IN LEVEL. CORNER BUTTRESSES NEED NOT BE BRACED. NO BACKFILLING IS TO BE DONE BEFORE ALL SLABS BRACING WALLS ARE IN PLACE UNLESS APPROVED BY THE ENGINEER. PROVIDE TEMPORARY BRACING FOR ALL PIERS AND SUMP PITS.
7. CONTRACTOR TO INSTALL ALL PIPE SLEEVES, BOXED OPENINGS, ANCHOR BOLTS, ETC., AS REQUIRED FOR THE VARIOUS TRADES. WALL POCKETS TO RECEIVE BEAMS AND SLABS SHALL BE PROVIDED AS REQUIRED FOR THE SUPERSTRUCTURE. SHOP DRAWINGS SHOWING THE POSITION OF OPENINGS SHALL BE SUBMITTED TO THE STRUCTURAL ENGINEER PRIOR TO PLACING CONCRETE.
8. MINIMUM COVER FOR REINFORCING STEEL SHALL BE 1½" FOR SLABS AND INTERIOR WALL SURFACES EXPOSED TO VEHICULAR/PEDESTRIAN TRAFFIC; AND COLUMNS (TIES, STIRRUPS OR PRIMARY REINFORCEMENT). FOR ALL CONCRETE EXPOSED TO WEATHER AND EARTH FILL, COVER SHALL BE 2". FOR CONCRETE PLACED AGAINST EARTH, MINIMUM COVER SHALL BE 3". TOP COVER FOR REINFORCEMENT IN RETAIL SLABS TO BE 3/4".
9. THE STRUCTURAL ENGINEER OR HIS FIELD QUALIFIED REPRESENTATIVE MUST CHECK AND APPROVE ALL STEEL REINFORCING PRIOR TO CONCRETE PLACEMENT.

CAISSON NOTES:

1. THE DESIGN AND INSTALLATION OF CAISSONS, CAISSONS CAPS, AND RELATED CONSTRUCTION IS TO CONFORM TO THE REQUIREMENTS SET FORTH IN THE NEW YORK CITY BUILDING CODE AND THE SPECIFICATIONS.
2. DRILLED CAISSONS SHALL HAVE THE FOLLOWING PARAMETERS AS PER RECOMMENDATION FROM GEOTECHNICAL ENGINEER:
   A. 13⅜" ø CAISSON
      - COMPRESSIVE LOAD CAPACITY = 500 T
      - UPLIFT LOAD CAPACITY = 150 TON
   B. UPLIFT AND LATERAL FIELD TEST ARE REQUIRED.
4. CAISSON INSTALLATION TO BE SUPERVISED BY A LICENSED PROFESSIONAL ENGINEER.
5. CAISSON OPERATIONS TO BE IN ACCORDANCE WITH THE NEW YORK CITY BUILDING CODE, AND ARE SUBJECT TO SPECIAL INSPECTION IN ACCORDANCE WITH NEW YORK CITY BUILDING CODE.
6. A PLAN SHOWING THE IDENTIFICATION OF ALL CAISSONS AND A CAISSONS NUMBERING PLAN IS TO BE SUBMITTED TO THE ENGINEER OF RECORD FOR FILING WITH THE BUILDING DEPARTMENT PRIOR TO COMMENCEMENT OF DRIVING OPERATIONS.
7. LOAD TESTS (IF NECESSARY) SHALL BE PERFORMED AS PER THE REQUIREMENTS OF THE NEW YORK CITY BUILDING CODE. LOCATION OF TEST CAISSONS TO BE APPROVED BY THE ENGINEER OF RECORD.
8. ALL CAISSON GROUPS AND CAISSON CAPS TO BE CONCENTRIC WITH COLUMNS AND WALLS ABOVE UNLESS OTHERWISE NOTED ON PLAN.
9. RECORDS OF PENETRATION OF EVERY CAISSON AND THE BEHAVIOR OF SAME DURING DRILLING ARE TO BE SUBMITTED TO THE ENGINEER OF RECORD.
10. AN "AS-DRIVEN" CAISSON LOCATION PLAN AND CAISSON LOGS ARE TO BE SUBMITTED TO THE ENGINEER OF RECORD FOR APPROVAL, NO CAISSON CAPS ARE TO BE PLACED BEFORE THIS IS DONE.
11. ESTIMATED AVERAGE CAISSON LENGTH IS PER GEOTECHNICAL CONSULTANT. CAISSON LENGTH COULD VARY DUE TO ACTUAL SOIL CONDITION.
12. FOR DETAILS REFER TO GEOTECHNICAL CONSULTANT REPORT AND SPECIFICATIONS.

CAISSON INSTALLATION PROCEDURE

1. MOBILIZE TO SITE.
2. SET UP RIG ON PROPER LOCATION AND PLUMB MAST.
3. DRILL IN CAISSON USING DUPLEX DRILLING METHODS. CLEAN WITH WATER ONLY. NOTE: OUTSIDE CASING TO BE ADVANCED 2-DIAMETERS OR 2-FOOT MINIMUM PRIOR TO CLEANING.
4. CASING IS DRILLED-IN TO BEDROCK AS INDICATED ON DRAWINGS. DRILL ROCK SOCKET AS SHOWN ON DRAWINGS.
5. FLUSH HOLE CLEAN OF SPOILS. IF PILE TIP IS BELOW GWT, FLUID LEVEL INSIDE CASING TO BE MAINTAINED AT TOP OF PILE DURING CLEAN OUT. A BUCKET OR AUGER MAY BE USED TO CLEAN HOLE. (AIR MAY BE USED IN COMPACTED TILL OR ROCK).
7. INTRODUCE REINFORCING THREADBAR WITH SPACERS AND PUSH TO THE BOTTOM OF THE PILE.
8. PLACE 3/4-INCH DIAMETER PVC GROUT TUBE TO THE BOTTOM OF ROCK SOCKET AND GROUT THE CAISSON FROM THE BOTTOM TO DISPLACE THE DRILLING FLUID. CONTINUE GROUTING UNTIL GOOD GROUT FLOWS OUT THE TOP OF THE PILE.
9. CUT THREADBAR TO PROPER ELEVATION AS SHOWN ON CONTRACT DRAWINGS.

CONCRETE NOTES

A. CONCRETE

1. ALL CONCRETE SHALL BE NORMAL WEIGHT CONTROLLED CONCRETE, U.O.N. , AND COMPLY WITH THE A.C.I. BUILDING CODE AND THE CURRENT NEW YORK CITY BUILDING CODE.
2. CONCRETE STRENGTH SHALL BE AS FOLLOWS, UNLESS OTHERWISE NOTED:

FOUNDATION, SLAB ON GRADE, PIERS, PILE CAPS & MAT	5000PSI	
SLABS/BEAMS	7200 PSI	1ST-16TH FL.
SLABS/BEAMS	6000 PSI	17TH-46TH
SLABS/BEAMS	5000 PSI	47TH-ROOF
SHEAR WALLS	SEE COLUMN SCHEDULE	
COLUMNS	SEE COLUMN SCHEDULE	

3. NO CONCRETE SHALL BE PLACED UNTIL THE CONTRACTOR HAS INSTALLED ALL THE INSERTS AND DOVETAILS NECESSARY TO PROVIDE SUPPORT FOR MULLIONS, APPLIED FINISHES, PARTITIONS, PIPES, DUCTS, EQUIPMENT, ETC., AS REQUIRED IN ARCHITECTURAL, H.V.A.C. AND STRUCTURAL DRAWINGS. WHERE BRICK VENEER EXCEEDS 18" IN HEIGHT, PROVIDE DOVETAIL TYPE MASONRY ANCHORS SPACED AT 24" O/C IN ALL BACK UP VERTICAL CONCRETE SURFACES.
4. CONTRACTOR SHALL VERIFY LOCATIONS AND DIMENSIONS OF ALL SLOTS, PIPE SLEEVES, DUCTS AND ANY OTHER CONCRETE PENETRATIONS AS REQUIRED FOR VARIOUS TRADES BEFORE CONCRETE IS PLACED.

SHOP DRAWINGS SHOWING COMPOSITE LAYOUT OF ALL PENETRATIONS MUST BE SUBMITTED TO THE ENGINEER FOR APPROVAL PRIOR TO CONSTRUCTION.

5. ALL PLUMBING AND ELECTRICAL SLOTS SHALL BE FILLED WITH CONCRETE TO THE SAME DEPTH AS FLOOR AFTER CONDUITS AND/OR PIPES ARE INSTALLED.
6. NO PIPES OR CONDUITS EXCEEDING 1/3 SLAB THICKNESS IN OUTSIDE DIAMETER NOR OVER NOMINAL 2" INSIDE DIAMETER SHALL BE EMBEDDED IN THE STRUCTURAL CONCRETE FLOOR OR SLAB. NO PIPES AND/OR CONDUITS SHOULD BE PLACED CLOSER THAN 3 DIAMETER ON CENTER NOR PASS WITHIN 24" OF COLUMN FACE, U.O.N. JUNCTION BOXES MAY BE PLACED IN STRUCTURAL CONCRETE SLAB BUT SHALL NOT EXCEED 4½" X 4½" X 3½" IN DEPTH AND SHALL BE SEPARATED FROM OTHER JUNCTION BOXES BY NOT LESS THAN 8" OF CONCRETE.
7. ALL MEMBERS IN THE FLOOR SYSTEM INCLUDING BEAMS, BRACKETS, COLUMN CAPITALS AND HAUNCHES SHALL BE PLACED MONOLITHICALLY. WHERE VERTICAL CONSTRUCTION JOINTS ARE NECESSARY, MAY BE MADE AT CENTER OF BEAM OR SLAB USING APPROVED BULKHEADS AND ADDITIONAL REINFORCING AS SHOWN ON DETAILS.
8. NO CONCRETE FLOOR SYSTEM IS TO BE INSTALLED UNTIL AT LEAST TWO HOURS HAVE PASSED AFTER THE SUPPORTING COLUMNS AND WALLS ARE PLACED.
9. WHEN PLACING CONCRETE AGAINST AN ADJACENT BUILDING OR AT EXPANSION JOINT, AT LEAST 1" (U.O.N.) OF HIGH DENSITY STYROFOAM SHALL BE PLACED AT THE INTERFACE BETWEEN THE EXISTING AND NEW CONCRETE. IN ADDITION, THE CONTRACTOR MUST TAKE ALL THE NECESSARY MEASURES SO AS NOT TO CREATE ANY DAMAGE TO THE EXISTING CONSTRUCTION WHILE PLACING THE NEW CONCRETE.
10. ALL WORK MARKED S.S. (SUPERSTRUCTURE) IN FOUNDATION DRAWINGS SHALL BE PART OF THE SUPERSTRUCTURE CONTRACT.
11. ALL SLABS ON GROUND ARE IN SUPERSTRUCTURE CONTRACT (U.O.N.). SLABS WITHIN THE BUILDING ARE FRAMED (REINFORCED) SLABS SUPPORTED BY PILE CAPS AND GRADE BEAMS. SEE GROUND FLOOR FRAMING PLANS.
12. TEMPORARY SHORING AND RESHORING SHALL REMAIN IN PLACE AT LEAST 28 DAYS AFTER PLACEMENT OF CONCRETE.
13. NO DEVIATION FROM THE STRUCTURAL PLANS SHALL BE PERMITTED WITHOUT THE EXPRESS WRITTEN CONSENT OF THE STRUCTURAL ENGINEER.

B. REINFORCEMENT

1. ALL STEEL REINFORCEMENT (STIRRUPS AND TIES INCLUSIVE) SHALL HAVE AN ULTIMATE TENSILE STRENGTH OF 90,000 PSI AS PER A.S.T.M. A615 GRADE 60. THE CONTRACTOR SHALL FURNISH AND INSTALL ALL THE CHAIRS, REBARS, TIES, SPACERS, ETC., TO SECURE AND SUPPORT THE REINFORCING WHILE PLACING THE CONCRETE.
2. THE CONTRACTOR MUST SUBMIT REINFORCING SHOP DRAWINGS TO THE STRUCTURAL ENGINEER FOR REVIEW. NO CONSTRUCTION IS TO BE STARTED UNTIL THE SHOP DRAWINGS ARE REVIEWED BY THE ENGINEER.
3. THE STRUCTURAL ENGINEER OR HIS FIELD QUALIFIED REPRESENTATIVE MUST CHECK AND APPROVE ALL STEEL REINFORCEMENT PRIOR TO CONCRETE PLACEMENT.
4. ALL REINFORCING BARS MARKED CONTINUOUS SHALL BE LAPPED AT SPLICES AND CORNERS IN CONFORMANCE WITH LAP SPLICE TABLES IN TYPICAL DETAILS UNLESS OTHERWISE NOTED. LAP CONTINUOUS TOP BARS AT CENTER BETWEEN SUPPORTS AS REQUIRED. TERMINATE CONTINUOUS BARS AT END SUPPORTS WITH STANDARD HOOKS, U.O.N.
5. MINIMUM COVER FOR REINFORCING STEEL SHALL BE ¾" FOR INTERIOR SLABS AND INTERIOR WALL SURFACES; 1½" FOR BEAMS, GIRDERS AND COLUMNS (TIES, STIRRUPS OR PRIMARY REINFORCEMENT). FOR ALL CONCRETE EXPOSED TO WEATHER AND EARTH FILL, COVER SHALL BE 2" (1½" FOR STIRRUPS). FOR CONCRETE PLACED AGAINST EARTH, MINIMUM COVER SHALL BE 3".

C. CODES AND TESTS

1. THIS STRUCTURE HAS BEEN DESIGNED UNDER THE PROVISIONS OF THE NEW YORK CITY BUILDING CODE AS AMENDED AND A.C.I. 318.
2. ALL CONTROLLED CONCRETE SHALL COMPLY WITH THE A.C.I. 318 BUILDING CODE. APPLICATION FOR CONTROLLED CONCRETE WITH CONCRETE TESTS AND CURVES OF TESTS FOR THE PRELIMINARY DESIGN MIX PREPARED BY AN APPROVED LABORATORY MUST BE SUBMITTED TO THE ENGINEER FOR FILING WITH THE BUILDING DEPARTMENT. NO CONCRETE SHALL BE PLACED WITHOUT THE DESIGN MIX BEING APPROVED BY THE BUILDING DEPARTMENT.
3. DESIGN AND CONSTRUCTION OF FORMWORK IS TO COMPLY WITH THE A.C.I. 318-89 BUILDING CODE AND NEW YORK CITY BUILDING CODE AS AMENDED.
4. THE DESIGN DETAILS AND NOTES INCLUDED HEREIN ARE IN COMPLIANCE WITH LOCAL LAW 17/95.

D. SEISMIC AND WIND CRITERIA

1. THE STRUCTURE HAS BEEN DESIGNED IN ACCORDANCE WITH THE LATEST NEW YORK CITY BUILDING CODE (NYCBC 2008).
2. WIND DESIGN DATA:
   - WIND LOAD BASED ON PROJECT SPECIFIC WIND TUNNEL TEST FROM RWDI (ROWAN WILLIAMS DAVIES & IRWIN INC.) DATED IN ACCORDANCE WITH NYCBC 2008.
3. EARTHQUAKE DESIGN DATA:
   - SEISMIC IMPORTANCE FACTOR = 1
   - S_{DS} = 0.365g, S_{D1} = 0.071g
   - SITE CLASS = B
   - SEISMIC FORCE RESISTING SYSTEM = ORDINARY REINFORCED CONCRETE SHEAR WALL
   - DESIGN BASE SHEAR (V) SERVICE: E/W = N/S =
   - SEISMIC RESPONSE COEFFICIENT (C): E/W = N/S =
   - RESPONSE MODIFICATION FACTORS: R = 4 PER NYCBC TABLE 1617.6.2
   - ANALYSIS PROCEDURE USED = EQUIVALENT LATERAL FORCE PROCEDURE

STRUCTURAL STEEL NOTES

1. ALL STRUCTURAL STEEL INCLUDING PLATES: A.S.T.M A-572 GR 50 U.O.N.
2. ALL BOLTS: A.S.T.M A-490X, 1"ø U.O.N.
3. ALL ELECTRODES: 120XXX OVER BASE MATERIAL.
4. ALL WELDING PER A.W.S. D1.1 LATEST SPECIFICATION.
5. ALL FULL AND PARTIAL PENETRATION WELDS TO BE TESTED WITH ULTRASONIC, MAGNETIC PARTICLE OR DYE PENETRANT METHODS.
6. ALL GIRDERS MARKED G1, TO BE TESTED FOR LAMELLAR TEARING BY ULTRASONIC METHOD.

MASONRY NOTES

1. SEE ARCHITECTURAL DRAWINGS AND SPECIFICATIONS FOR COMPLETE REQUIREMENTS FOR C.M.U. MASONRY CONSTRUCTION AND APPEARANCE. DETAILS AND NOTES SHOWN ON THE STRUCTURAL DRAWINGS ARE INTENDED TO SUPPLEMENT ARCHITECTURAL REQUIREMENTS AND TO DEFINE ELEMENTS WHICH PROVIDE STRUCTURAL STRENGTH AND STABILITY.
2. DETAILS, SECTIONS, SCHEDULES, ETC. AND THESE NOTES, REPRESENT THE MINIMUM REQUIREMENTS FOR STRUCTURAL ADEQUACY. WHERE ARCHITECTURAL REQUIREMENTS DIFFER FROM STRUCTURAL, THE MORE STRINGENT SHALL BE FOLLOWED.
3. CODE: MASONRY WALL CONSTRUCTION SHALL CONFORM TO THE NEW YORK CITY BUILDING CODE AND TO ACI 530/ASCE-5 AS REFERENCED BY THE NYC CODE.
4. MASONRY UNITS SHALL BE LIGHTWEIGHT HOLLOW LOAD BEARING CONCRETE MASONRY (CMU). COMPRESSIVE STRENGTH OF MASONRY F'M SHALL BE A MINIMUM OF 1,500 PSI.
5. MORTAR SHALL BE TYPE M OR S.
6. HORIZONTAL JOINT REINFORCEMENT SHALL BE TRUSS TYPE GALVANIZED COLD-DRAWN STEEL WIRE CONFORMING TO ASTM A 951.
7. PROVIDE HORIZONTAL JOINT REINFORCEMENT IN EVERY OTHER JOINT (16" O.C. VERTICALLY) UNLESS PLANS OR DETAILS CALL FOR CLOSER SPACING OR ADDITIONAL REINFORCEMENT.
8. BAR REINFORCEMENT: ASTM A 615 GRADE 60, PER SCHEDULE. FOR ADDITIONAL REINFORCEMENT SEE WALL REINFORCEMENT ELEVATION.
9. ALL CELLS WITH REINFORCEMENT SHALL BE GROUTED SOLID FOR THE FULL EXTENT OF BAR, VERTICAL AND HORIZONTAL.
10. GROUT SHALL HAVE A MINIMUM COMPRESSIVE STRENGTH OF 2,000 PSI. GROUT SHALL BE "FINE" AS DEFINED BY ASTM C 476.
11. STEEL ANGLES: ASTM A 36. STEEL IN AN EXTERIOR WALL OR EXPOSED TO THE EXTERIOR SHALL BE GALVANIZED. 12. CONTRACTOR SHALL COORDINATE ALL MASONRY WORK WITH WORK OF OTHER TRADES: ARCHITECTURAL, STRUCTURAL, MEP.

NON-STRUCTURAL ITEMS SHOWN ON THE STRUCTURAL/FOUNDATION DRAWINGS

1. THE FOLLOWING NON-STRUCTURAL ITEMS MAY BE SHOWN ON THE STRUCTURAL AND/OR FOUNDATION DRAWINGS FOR THE PURPOSE OF CLARITY IN INTERFACE WITH STRUCTURAL AND/OR FOUNDATION WORK. ITEMS BELOW MAY NOT BE FULLY DEFINED ON THE STRUCTURAL/FOUNDATION DRAWINGS. THE INFORMATION FOR NON-STRUCTURAL ELEMENTS IS FURNISHED BY OTHER CONSULTANTS AS LISTED BELOW. ALL RFI AND SHOP DRAWINGS RELATED TO THESE NON-STRUCTURAL ITEMS SHALL BE SUBMITTED TO THE CONSULTANTS LISTED BELOW FOR THEIR REVIEW AND APPROVAL.

GEOTECHNICAL ENGINEER:
- FOUNDATION/UNDERSLAB WATERPROOFING, DAMPPROOFING SYSTEMS
- WALL AND UNDERSLAB DRAINAGE SYSTEM, INCLUDING SUMP PITS, GRAVEL & PIPING, CLEANOUTS
- ROCK ANCHORS
- CAISSONS AND PILES, INCLUDING REINFORCMENT
- ROCK CONTOURS

ARCHITECT OF RECORD:
- SUMP PITS
- WATERPROOFING/DAMPPROOFING APPLIED TO EXPOSED SURFACES, ELEVATOR OR SUMP PIT INTERIOR SURFACES
- PAINT
- FIREPROOFING
- CONCRETE CURBS: HEIGHT, WIDTH, EXTENT, LOCATION
- BRICK, BLOCK, TILE MASONRY, METAL PANELS, PRECAST FACADE PANELS, CURTAIN WALLS AND ALL OTHER FACADE SYSTEMS
- ROOFING SYSTEMS, DRAIN LOCATIONS, SLOPES TO DRAINS FILLS, INSULATION, PAVERS OR GRAVEL
- FLOATING/SECONDARY SLABS

LOADING SCHEDULE

OCCUPANCY	PARTITION/FILL &/OR FINISHES (PSF)	CEIL. & MECH. (PSF)	LIVE LOAD
ROOF (ACCESS)	30	5	100
STAIRS	–	5	100
CORRIDORS	30	5	40
RESIDENTIAL	12	5	40
MECHANICAL ROOMS	40	5	150
15TH FLOOR TERRACE	15	5	60
RD FLOOR ROOF (NO ACCESS)	50	5	40
GROUND FLOOR RETAIL LOBBY	25	5	100
CELLAR FITNESS ROOM	25	5	100

U.O.N. ON PLAN.

LEGEND:

- a. [...] INDICATES THE BOTTOM OF FOUNDATION WALL ELEVATION
- b. <...> INDICATES THE TOP OF FOUNDATION WALL ELEVATION
- c. (...) INDICATES THE TOP OF PILECAP ELEVATION
- d. (XXxXX) INDICATES SIZE OF PIER IN INCHES, FIRST DIMENSION SHOWN IS IN THE EAST-WEST DIRECTION.
- e. INDICATES ADDITIONAL TOP REINFORCEMENT AT SUPPORTS
- f. INDICATES ADD'L BOTTOM REINFORCING AT SUPPORTS
- g. INDICATES ADDITIONAL TOP REINFORCEMENT CONTINUOUS BETWEEN SUPPORTS
- h. INDICATES ADDITIONAL BOTTOM REINFORCEMENT CONTINUOUS BETWEEN SUPPORTS
- i. INDICATES CHANGE IN ELEVATION
- j. INDICATES CONCRETE COLUMN/SHEARWALL/FOUNDATION WALL
- k. INDICATES CONCRETE COLUMN/FOUNDATION WALL/SHEARWALL BELOW
- l. INDICATES SLAB OPENING (FIRST DIMENSION IS IN EAST-WEST DIRECTION)
- m. INDICATES COLUMN/POST/HANGER ABOVE OR BELOW
- n. (XX) INDICATES COLUMN DESIGNATION
- o. (P) INDICATES POST DESIGNATION
- p. (H) INDICATES HANGER DESIGNATION
- q. (SW-X) INDICATES SHEARWALL DESIGNATION
- r. INDICATES STIRRUPS

POST SCHEDULE:

- (P1) INDICATES 14"ø CONCRETE POST W/5-#7 VERT & #3 @14 TIES
- (P2) INDICATES 12x12 CONCRETE POST W/4-#6 VERT & #3 @12 TIES
- (P3) INDICATES 16x16 CONCRETE POST W/4-#7 VERT & #4 @8 TIES
- (P4) INDICATES 16x8 CONCRETE POST W/4-#6 VERT.& #3 @8 TIES
- (P5) INDICATES 20x22 CONCRETE POST W/4-#8 VERT. & #3 @16 TIES EPOXY COATED BARS
- (P6) INDICATES 20x12 CONCRETE POST W/6-#8 VERT. & #4 @12 TIES

HANGER SCHEDULE:

- (H1) INDICATES 12x12 CONCRETE HANGER W/4-#11 THREADS BARS W/D6 LENTON TERMINATION AT EA. END. & REINF. TO BE MECH. SPLICE ONLY.

SPECIAL INSPECTIONS

(TERMINOLOGY PER CURRENT TR-1) SPECIAL INSPECTION	CURRENT CODE REFERENCES
CONCRETE - CAST IN PLACE	1704.4
CONCRETE TEST CYLINDERS* (TR2)	1905.6
CONCRETE DESIGN MIX* (TR3)	1905.3
SOILS - SITE PREPARATION	1704.7.1
SOILS - FILL PLACEMENT & IN-PLACE DENSITY	1704.7.2 1704.7.3
SOILS - INVESTIGATIONS (BORINGS/TEST PITS) (TR4)	1704.7.4
PILE FOUNDATIONS & DRILLED PIER INSTALLATION (TR5)	1704.8
UNDERPINNING	1704.9.1
EXCAVATION - SHEETING, SHORING AND BRACING	1704.19 & 3304.4.1
FIRESTOP, DRAFTSTOP AND FIREBLOCK SYSTEMS	1704.25

* THESE TEST MUST BE PERFORMED BY A LICENSED CONCRETE TESTING LAB.

NOTES:

1. REFER TO THE PROJECT SPECIFICATIONS FOR ADDITIONAL INFORMATION ON SCOPE AND DETAILED REQUIREMENTS FOR INSPECTIONS.
2. ALL SPECIAL INSPECTIONS SHALL BE PERFORMED UNDER THE SUPERVISION OF A PROFESSIONAL ENGINEER LICENSED IN THE STATE OF NEW YORK.
3. REPORTS OF RESULTS SHALL BE SUBMITTED TO THE OWNER AND ARCHITECT FOR REVIEW. SIGNED COPIES OF ALL TESTS AND INSPECTION REPORTS SHALL BE FILED WITH THE BUILDING DEPARTMENT (THROUGH THE APPLICANT).
4. REPORTS SHALL STATE WHETHER RESULTS COMPLY WITH CONTRACT REQUIREMENTS, SUMMARIZE THE TYPE OF TEST, THE LOCATION OR COMPONENT TESTED, AND RECOMMEND ANY REMEDIAL MEASURES REQUIRED. REPORT SHOULD NOTE ANY OTHER DEVIATIONS FROM THE CONTRACT DOCUMENTS.
5. FOR ITEMS OF WORK OF OTHER TRADES WHICH ARE SUBJECT TO SPECIAL INSPECTION, SEE THE CITY OF NEW YORK BUILDING CODE, AS WELL AS ARCHITECTURAL, MECHANICAL, ELECTRICAL, PLUMBING, ETC. DRAWINGS AND SPECIFICATIONS.
6. IN ADDITION TO THE ABOVE REQUIREMENTS, ALL COLUMN SPLICE, BEAM MOMENT CONNECTIONS AT BEAMS DESIGNATED AS "LLRS" AND BRACE FRAME OR WIND TRUSS CONNECTIONS (PER S-940 SERIES OF DWGS.) SHALL COMPLY WITH THE INSPECTION REQUIREMENTS OF AWS D1.8 "STRUCTURAL WELDING CODE-SEISMIC SUPPLEMENT", IF WELDING IS PRESENT IN CONNECTION.

PROJECT:

EAGLE WAREHOUSE

BLOCK 266 LOT 3, 16, 18, 20 & 21
43-22 QUEENS STREET, LONG ISLAND CITY

OWNER/ APPLICANT:
43-22 Queens Street L.L.C.
C/O Rockrose Development Corp.
666 Fifth Avenue
New York, NY 10103
TEL. : (212) 847-3700
FAX : (212) 757-1875

ARCHITECT:
SLCE*Architects*
1359 BROADWAY
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TEL.: (212) 979-8400
FAX.: (212) 979-8387

STRUCTURAL ENGINEER:
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15 West 44th Street, 2nd Floor
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ELEVATOR CONSULTANT:
Van Deusen & Associates
5 Regent Street, Suite 524
Livingston, NJ 07039
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West Seneca, NY 14224
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SLADE ARCHITECTURE
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New York, NY 10007
TEL. : (212) 677-6380

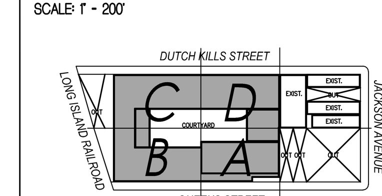
ZONING & CODE CONSULTANT
CCA
100 Church Street New
York, NY 10007
TEL. : (212) 385-1818

No:	Date:	Revision:
3	XX-XX-14	XXXXXX
2	08-29-14	DOB - NB FILING
1	06-30-14	DOB ZONING

NORTH ARROW

N

KEY PLAN
SCALE: 1" - 200'



DRAWING TITLE:

GENERAL NOTES, LEGEND & ABBREVIATIONS

SEAL & SIGNATURE:

DATE: AUG. 28 2012
PROJECT No: 1310870
DRAWN BY:
CHECK'D BY:
DWG. No:
FO-001.00
DRAWING : OF 168

APPENDIX D

43-22 QUEENS STREET

Wind Tunnel Report

Preliminary Results - Wind-Induced Structural Responses
Eagle Warehouse - New York City, New York, RWDI Project #1400955
April 25, 2014

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

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

Configuration	Frequency Case	My (lb-ft)	Mx (lb-ft)	Mz (lb-ft)	Fx (lb)	Fy (lb)
Existing	Case 1 (T)	5.26E+08	**1.06E+09**	**5.39E+07**	1.37E+06	**3.00E+06**
Future	Case 1 (T)	**5.29E+08**	1.06E+09	5.31E+07	**1.38E+06**	2.98E+06

Notes:

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

(2) A total damping ratio of 2.0% of critical was used for structural load calculations.

(3) The above loads are based on the structural properties as provided on April 7, 2014. The Case 1 (T) natural building frequencies were as follows:

Mode 1: 0.1887 Hz (primarily X coupled with torsion)
Mode 2: 0.2326 Hz (primarily Y)
Mode 3: 0.3030 Hz (primarily torsion coupled with X).

(4) The above loads correspond to a 50-year return period basic wind speed (3-second gust) of 9

Table 3a: Effective Static Floor-by-Floor Wind Loads
Worst Case Test Configuration

Floor	Height (ft) Above Grade	Fx (lb)	Fy (lb)	Mz (lb-ft)
STORY1	0	5600	21100	52000
STORY2	15.02	11100	42400	116000
STORY3	30.01	10400	40000	131000
STORY4	43	9600	37700	142000
STORY5	55.97	10200	38100	166000
STORY6	68.89	11400	38600	194000
STORY7	81.84	11200	35100	208000
STORY8	91.84	9500	31500	233000
STORY9	101.84	10200	31800	253000
STORY10	111.84	9000	32000	239000
STORY11	121.84	9600	32800	265000
STORY12	131.84	10300	33400	293000
STORY13	141.84	11100	34200	322000
STORY14	151.84	11700	35000	353000
STORY15	161.84	12400	35800	386000
STORY16	171.84	13200	36800	420000
STORY17	181.84	14000	37500	455000
STORY18	191.84	14800	38400	489000
STORY19	201.84	15300	39100	520000
STORY20	211.84	15900	40100	556000
STORY21	221.84	16800	40900	595000
STORY22	231.84	17500	41800	633000
STORY23	241.84	18200	42800	670000

STORY24		251.84	19100	43900	714000
STORY25		261.84	20100	45200	758000
STORY26		271.84	21000	46400	804000
STORY27		281.84	21900	47600	852000
STORY28		291.84	22500	48300	871000
STORY29		301.84	22900	48900	886000
STORY30		311.84	23800	50300	933000
STORY31		321.84	24700	51500	980000
STORY32		331.84	25600	52800	1029000
STORY33		341.84	26600	54000	1078000
STORY34		351.84	31500	65000	1245000
STORY35		366.84	37400	77500	1481000
STORY36		381.84	34600	69700	1428000
STORY37		391.84	30300	59400	1276000
STORY38		401.84	30800	60300	1302000
STORY39		411.84	31500	61500	1351000
STORY40		421.84	32400	62800	1401000
STORY41		431.84	33200	64100	1450000
STORY42		441.84	34100	65400	1500000
STORY43		451.84	34900	66800	1550000
STORY44		461.84	35700	68100	1600000
STORY45		471.84	36500	69400	1649000
STORY46		481.84	37300	70700	1699000
STORY47		491.84	38100	72000	1748000
STORY48		501.84	38800	73300	1797000
STORY49		511.84	39500	74600	1845000
STORY50		521.84	40300	75900	1893000
STORY51		531.84	41000	77000	1940000
STORY52		541.84	41600	78300	1986000
STORY53		551.84	42300	79600	2031000
STORY54		561.84	45200	85500	2168000
ROOF		573.84	64600	122000	2800000
BULKHEAD		603.84	41600	78500	116000
SUMS	-		1.38E+06	3.00E+06	5.39E+07

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.

'STORY1
ctors

98 mph

Table 4a: Recommended Wind Load Combination Factors

Load Case	Factor for Simultaneous Application of Loads in Table 3a X Forces (Fx)	Y Forces (Fy)	Torsion (Mz)
1	+85%	+60%	+40%
2	+85%	+60%	-30%
3	+85%	-30%	+40%
4	+85%	-30%	-30%
5	-100%	+30%	+45%
6	-100%	+30%	-45%
7	-100%	-30%	+45%
8	-100%	-30%	-45%
9	+30%	+100%	+30%
10	+30%	+100%	-30%
11	+30%	-90%	+30%
12	+30%	-90%	-30%
13	-30%	+100%	+30%
14	-30%	+100%	-30%
15	-30%	-90%	+30%
16	-30%	-90%	-30%
17	+30%	+30%	+100%
18	+30%	+30%	-100%
19	+30%	-30%	+100%
20	+30%	-30%	-100%
21	-60%	+30%	+100%
22	-60%	+30%	-100%

23	-60%	-30%	+100%
24	-60%	-30%	-100%

Note:

(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.

Preliminary Results - Wind-Induced Structural Responses
Eagle Warehouse - New York City, New York, RWDI Project #1400955
April 25, 2014

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.

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

Configuration	Frequency Case	My (lb-ft)	Mx (lb-ft)	Mz (lb-ft)	Fx (lb)	Fy (lb)
Existing	Case 1 (T)	5.75E+08	**1.13E+09**	**6.08E+07**	1.49E+06	**3.14E+06**
Future	Case 1 (T)	**5.78E+08**	1.13E+09	6.00E+07	**1.50E+06**	3.11E+06

Notes:

(1) The above loads are the cumulative summation of the wind-induced loads at Structur (i.e.: grade) centered about the reference axis shown in Figure 4, exclusive of combir

(2) A total damping ratio of 1.5% of critical was used for structural load calculations.

(3) The above loads are based on the structural properties as provided on April 7, 2014. The Case 1 (T) natural building frequencies were as follows:

Mode 1: 0.1887 Hz (primarily X coupled with torsion)
Mode 2: 0.2326 Hz (primarily Y)
Mode 3: 0.3030 Hz (primarily torsion coupled with X).

(4) The above loads correspond to a 50-year return period basic wind speed (3-second g

Table 3b: Effective Static Floor-by-Floor Wind Loads
Worst Case Test Configuration

Floor	Height (ft) Above Grade	Fx (lb)	Fy (lb)	Mz (lb-ft)
STORY1	0	5600	20200	48000
STORY2	15.02	11100	40600	109000
STORY3	30.01	10400	38500	127000
STORY4	43	9600	36300	140000
STORY5	55.97	10200	36800	167000
STORY6	68.89	11500	37400	200000
STORY7	81.84	11500	34200	218000
STORY8	91.84	9700	30800	245000
STORY9	101.84	10400	31200	268000
STORY10	111.84	9300	31400	256000
STORY11	121.84	9900	32300	285000
STORY12	131.84	10700	33000	317000
STORY13	141.84	11600	33900	351000
STORY14	151.84	12300	34800	386000
STORY15	161.84	13100	35700	424000
STORY16	171.84	14000	36900	463000
STORY17	181.84	14900	37800	504000
STORY18	191.84	15700	38800	543000
STORY19	201.84	16400	39600	579000

STORY20	211.84	17000	40700	620000
STORY21	221.84	18000	41700	665000
STORY22	231.84	18800	42700	708000
STORY23	241.84	19600	43900	751000
STORY24	251.84	20600	45200	801000
STORY25	261.84	21700	46700	852000
STORY26	271.84	22700	47900	905000
STORY27	281.84	23700	49500	961000
STORY28	291.84	24400	50200	982000
STORY29	301.84	24800	50900	999000
STORY30	311.84	25900	52500	1053000
STORY31	321.84	26800	53900	1107000
STORY32	331.84	27900	55400	1163000
STORY33	341.84	29000	56800	1220000
STORY34	351.84	34200	68100	1406000
STORY35	366.84	40600	81200	1673000
STORY36	381.84	37700	73500	1617000
STORY37	391.84	33100	63100	1447000
STORY38	401.84	33600	64000	1477000
STORY39	411.84	34500	65500	1534000
STORY40	421.84	35500	67000	1591000
STORY41	431.84	36400	68500	1647000
STORY42	441.84	37500	70000	1705000
STORY43	451.84	38400	71600	1762000
STORY44	461.84	39200	73100	1820000
STORY45	471.84	40100	74600	1877000
STORY46	481.84	41000	76100	1933000
STORY47	491.84	41900	77600	1990000
STORY48	501.84	42700	79100	2046000
STORY49	511.84	43400	80600	2102000
STORY50	521.84	44400	82100	2156000
STORY51	531.84	45100	83400	2211000
STORY52	541.84	45800	84900	2263000
STORY53	551.84	46600	86300	2316000
STORY54	561.84	49800	92600	2471000
ROOF	573.84	71000	132200	3189000
BULKHEAD	603.84	45600	85000	109000
SUMS	-	1.50E+06	3.14E+06	6.08E+07

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.

.

al Level 'STORY1'
ıation factors.

ɟust) of 98 mph.

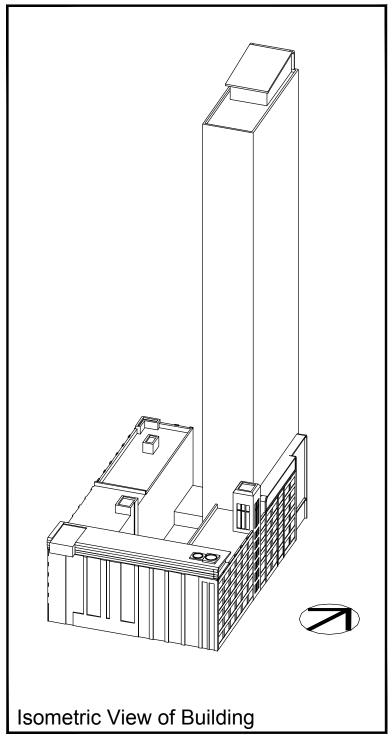
Table 4b: Recommended Wind Load Combination Factors

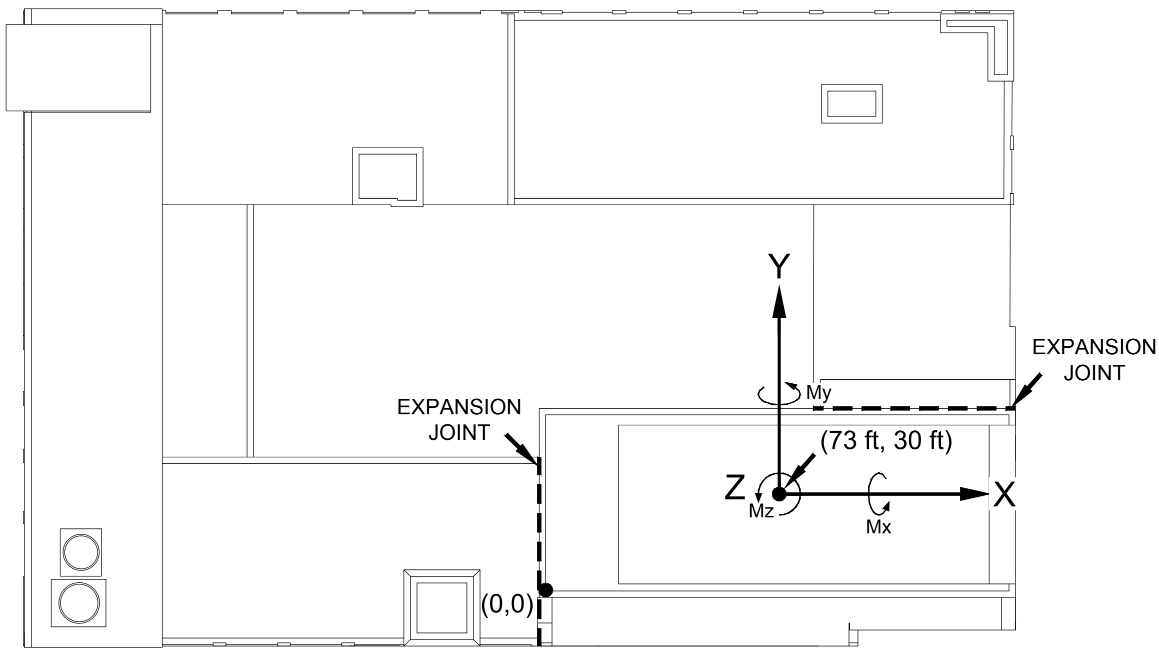
Load Case	Factor for Simultaneous Application of Loads in Table 3b X Forces (Fx)	Y Forces (Fy)	Torsion (Mz)
1	+85%	+60%	+35%
2	+85%	+60%	-30%
3	+85%	-30%	+35%
4	+85%	-30%	-30%
5	-100%	+30%	+45%
6	-100%	+30%	-45%
7	-100%	-35%	+45%
8	-100%	-35%	-45%
9	+35%	+100%	+30%
10	+35%	+100%	-30%
11	+30%	-90%	+30%
12	+30%	-90%	-30%
13	-35%	+100%	+30%
14	-35%	+100%	-30%
15	-30%	-90%	+30%
16	-30%	-90%	-30%
17	+30%	+35%	+100%
18	+30%	+35%	-100%

19	+30%	-35%	+100%
20	+30%	-30%	-100%
21	-60%	+35%	+100%
22	-55%	+35%	-100%
23	-60%	-35%	+100%
24	-55%	-30%	-100%

Note:

(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.



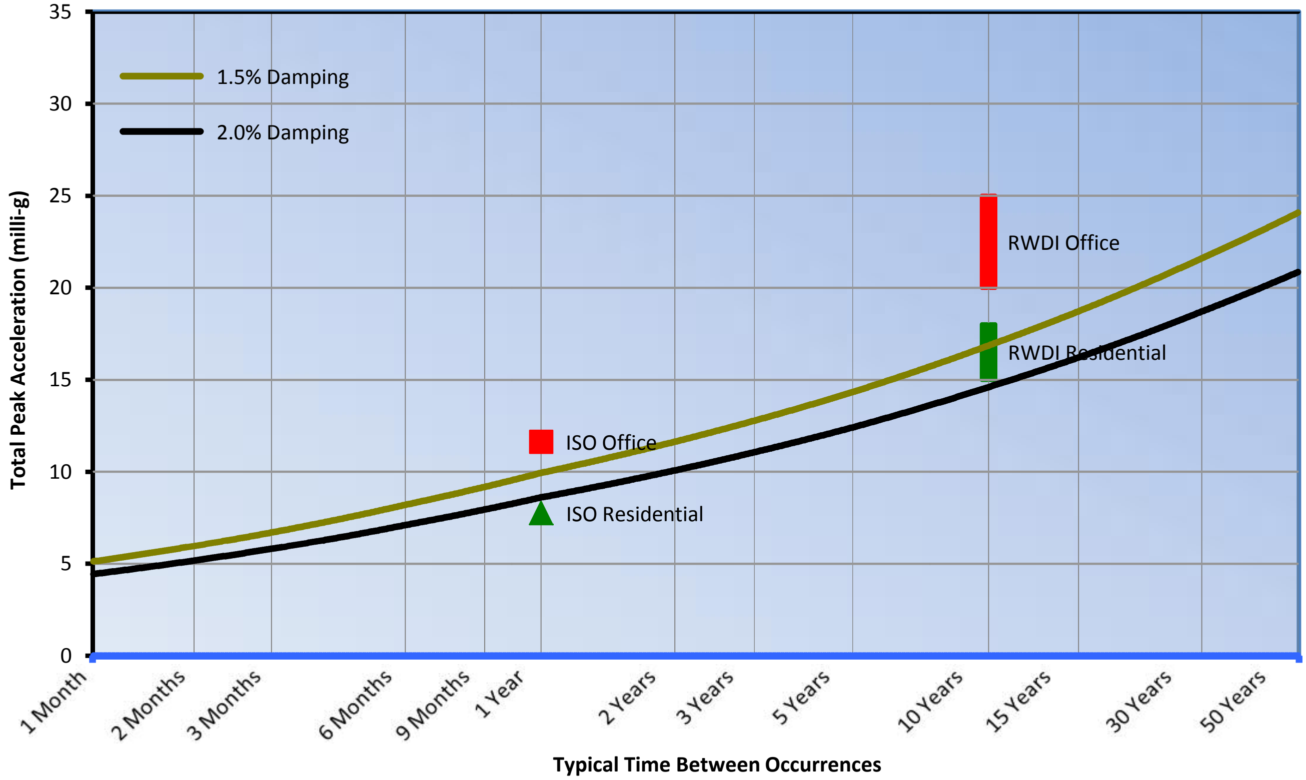


ROOF PLAN

Note:
Point (0,0) indicates co-ordinate origin provided by the structural engineer.

0 30 60ft

Co-ordinate System for Structural Loading	True North	Drawn by: DJM	Figure: 4
		Approx. Scale: 1"=60'	
Eagle Warehouse - New York City, NY	Project #1400955	Date Revised: April 25, 2014	



Return Period (Years)	Peak Accelerations(2) (milli-g) Total - [X, Y and torsional components]		Peak Torsional Velocities (milli-rads/sec)		
	1.5% Damping	2.0% Damping	1.5% Damping	2.0% Damping	CTBUH(5) Criteria
1	9.9 - [6.0, 9.3, 6.4]	8.6 - [5.2, 8.0, 5.5]	2.5	2.2	1.5
5	14 - [8.7, 13, 9.3]	12 - [7.5, 12, 8.0]	3.7	3.2	-
10	17 - [10, 16, 11]	15 - [8.7, 14, 9.4]	4.2	3.7	3

Notes:

(1) Frequencies of 0.1887, 0.2326, and 0.3030 Hz were used along with the indicated damping ratios.

(2) Accelerations are predicted at Structural Level 'STORY54' (561.84 ft above Structural Level 'STORY1') at a radial distance of 48 ft from the central axis of the tower (given in Figure 4).

(3) ISO is the International Organization for Standardization, and the current standard (ISO 10137:2007) provides acceleration criteria for buildings at the 1-year return period. The criteria plotted on the graph have been generated based on a response-weighted interpretation of the individual modal component of the ISO criteria.

(4) RWDI's criteria for residential and office buildings are based on research, experience and surveys of existing buildings, and is in agreement with general practice in North America.

(5) The Council on Tall Buildings and Urban Habitat (CTBUH) provides tentative torsional velocity criteria for the 1- and 10-year return periods.

(6) The above predictions do not include the influence of hurricanes, which is negligible in New York at the return periods of interest for occupant comfort.

Predicted Peak Accelerations and Torsional Velocities **Worst-Case Configuration**	Figure No. 6	RWDI
Eagle Warehouse - New York City, NY — Project #1400955	Date: Apr. 25, 2014	