LERA

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18 May 2016
File: P993

Ms. Danielle Axelrod
The Axelrod Group
New York, New York
Via e-mail: danielle.axelrod@gmail.com

281 Fifth Avenue
Structural Peer Review

Dear Danielle:

At the request of The Axelrod Group, Leslie E. Robertson Associates, R.L.L.P. has conducted a Structural Peer Review of the design of 281 Fifth Avenue as required by New York City Building Code Section 1617. This report summarizes the extent and findings of our review.

We have reviewed the following:

- Plans listed in Appendix A.
- *Geotechnical Investigation Report, Proposed 281 Fifth Avenue Development*, dated 18 May 2015, by Langan Engineering and Environmental Services. Pages 1 to 17 are attached to this report as Appendix B.
- Structural Design Criteria shown in Drawing FO-001.01 dated 04-08-16. See Appendix C.
- Study of Wind Effects for 281 Fifth Avenue, New York, NY, dated 25 November 2015 by the Boundary Layer Wind Tunnel Laboratory. Pages 1 to 19 are attached to this report as Appendix D.

Through our review, we have confirmed the following aspects of the structural design, as required by Section 1617.5.1:

- 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 recommendations of the wind tunnel laboratory;
- the structure has a complete load path;

- based on our independent calculations of representative foundations, columns, walls, beams and slabs, we find that the design of the structure has 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 and foundation design provisions of the Building Code.

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.

Very truly yours,

LESLIE E. ROBERTSON ASSOCIATES, R.L.L.P.

William J. Faschan

WJF/pi

cc: Ms. Susan Erdelyi Hamos, WSPCS
Via e-mail: Susan.ErdelyiHamos@wspcs.com

Mr. Jim Herr, Rafael Vinoly
Via e-mail: jherr@rvapc.com

STRUCTURAL PEER REVIEW STATEMENT

This structural peer review and report, dated 18 May 2016, is complete for the foundation and superstructure 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: 281 Fifth Avenue, Block #859, Lot #85

Department Application Number for Structural Work: #121193136

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 281 Fifth Avenue, Block #859, Lot #85, Application #121193136 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

STATE OF NEW YORK
WILLIAM JAMES FASCHAN
058872
LICENSED PROFESSIONAL ENGINEER

Signature ____________ Date 05/18/16

Cc: Project Owner: Ms. Danielle Axelrod
Project Registered Design Professional: Mr. Silvian Marcus

APPENDIX A

Plans Reviewed

~~Architectural Drawings, for DOB Submission, dated 4/8/2016;~~
~~Structural Drawings, for DOB Submission, dated 4/8/2016.~~

Refer to Appendix E List of Documents Reviewed, included in LERA report titled "281 Fifth Avenue Peer Review Structural Calculations" dated May 2016.

Comment added by W. Faschan, LERA 9/1/2016 Rev. 1

APPENDIX B

Geotechnical Investigation Report

18 May 2015

Mr. Ran Korolik
Victor Homes
3349 Highway 138
Bldg C, Suite C
Wall, New Jersey, 07753

RE: Geotechnical Investigation Report
Proposed 281 Fifth Avenue Development (the "Site")
New York, New York
Langan Project No.: 100464201

Dear Ran:

As requested, and in accordance with our 27 March 2015 proposal and subsequent authorization by your office, we have performed a geotechnical investigation at the Site consisting of six (6) drilled borings and five (5) exploratory test pits. This report summarizes our understanding of the Site conditions and the currently proposed development scheme, and presents the sub-surface investigation work performed to date, our investigation findings, and our foundation design recommendations for the proposed development. Our recommendations regarding other geotechnical aspects of construction, such as excavation, dewatering, temporary support of excavation and underpinning, and protection of adjacent structures, are also provided herein.

PROJECT DESCRIPTION

Site Conditions

The Site is located in the southeast corner of the intersection between Fifth Avenue and East 30th Street in Manhattan, and consists of Tax Lots 85, 86 and 87 on Tax Block 859; the total footprint area of the Site is about 7,300 square feet. The Site is currently occupied by three vacant 1-to-6-story buildings, each with one basement level, and is bordered by Fifth Avenue to the west, East 30th Street to the north, and 1-to-5-story buildings, each with one basement level, to the south and east. The 4-story 1 and 3 East 29th Street buildings located in close proximity to the Site are designated New York City (NYC) Landmarks; portions of these buildings are located within the NYC Department of Buildings (DOB) designated 90-foot influence zone originating from the Site.

We reviewed a 4 March 2015 topographic boundary and utility survey (the Site Survey) prepared by our office. Based on this survey, existing grades along the Fifth Avenue and East 30th Street sidewalks range from about el 40 (all elevations in this report are referenced to the North American Vertical Datum of 1988, NAVD88) near the northeast corner of the Site to about el 42 near the southwest corner of the Site. The Site Survey also indicates existing building vaults are located beneath the Fifth Avenue and East 30th Street sidewalks west and north of the Site, respectively.

Adjacent Buildings

We visited the NYCDOB Manhattan Records Room located at 2 Broadway to research available foundation-related information for the neighboring/bordering buildings. We were able to compile the following information regarding adjacent buildings from the NYCDOB records, the Site Survey, and our observations in the exploratory test pits:

- 275 Fifth Avenue Building (Tax Lot 88): This 5-story-building, with one basement level, borders the southern portion of the Site. NYCDOB OASIS website indicates this building was constructed in 1925. The Site Survey indicates the basement slab of this building is at el 32.5. Exploratory test pits TP-2 and TP-3 performed within this building indicate this building is supported on shallow foundations bearing on rock about 10.5 to 12.5 feet below the sidewalk level (i.e., at about el 30.5); the building foundation wall was observed to be composed of brick in the upper portion and stone blocks in the lower portion. No additional foundation-related information was available at the NYCDOB Records Room at the time of our visit.

- 2 East 30th Street Building (Tax Lot 84): This lot borders the eastern portion of the Site and is occupied by a 5-story-building, with one basement level, in the northern portion of the lot and a one-story extension, with no basement, in the southern portion of the lot. NYCDOB OASIS website indicates this building was constructed in 1925. The Site Survey indicates the basement slab of this building is at el 30.2 and the first floor level slab is t about el 40. Exploratory test pits TP-1, TP-4 and TP-5 performed within the Site and within this building indicate the 5-story building in the northern portion of this lot is supported on shallow foundations bearing on rock at least 10 feet below the sidewalk level (i.e., below about el 30), and the one-story extension in the southern portion of this lot is supported on shallow foundations bearing on soil about 7 to 9 feet below the sidewalk level (i.e., about el 33). The building foundation wall for the 5-story building was observed to be composed of brick and stone, and the wall footing for the one-story extension was observed to be composed of concrete. No additional foundation-related information was available at the NYCDOB Records Room at the time of our visit.

Proposed Development

We understand the proposed development plans are to demolish the on-site buildings and construct a 53-story residential tower. The upper 50 stories of the tower will have a footprint area of about 5,200 square feet, and the tower will extend above a 3-story podium occupying the entire site footprint; a single 14.5-foot-deep basement will be located beneath the podium covering the entire Site footprint. We also understand that after the existing building demolition, the existing building vaults beneath the East 30th Street and Fifth Avenue sidewalks will be completely backfilled. Based on the 15 April 2015 structural drawings prepared by WSP (Project Structural Engineer), we understand proposed column loads vary from about 950 kips to about 12,700 kips.

GEOTECHNICAL SUB-SURFACE INVESTIGATION AND FINDINGS

Sub-surface Investigation

The sub-surface investigation at the Site consisted of performing six (6) drilled borings and five (5) test pits; refer to Figure 2 for a location plan showing approximate locations of the borings and test pits.

All borings were performed by Warren George, Inc. (WGI) under the full-time special inspection of a field engineer from our office as required by the NYC Building Code (NYCBC). Prior to performing the borings, a private utility mark-out was performed around the proposed boring locations by NAEVA Geophysics, and final boring locations deemed clear of detectable sub-surface obstructions and utilities were marked out. Borings B1 through B3 were performed between 9 and 19 July 2014, and borings B4 through B6 were performed between 27 February and 6 March 2015. Borings B1 and B2 were performed along the adjacent sidewalks using a truck-mounted drill rig, and borings B3 through B6 were performed from within the on-site and neighboring/bordering building basements using a portable electric drill rig. The test pits were excavated by WGI using hand-excavation techniques and tools under the full-time observation of Langan. Test pits TP1 through TP4 were performed between 4 and 19 March 2015, and test pit TP5 was performed on 13 and 14 April 2015.

All borings were performed using conventional mud-rotary drilling and rock coring techniques. Soil samples were obtained in the borings using Standard Penetration Test (SPT) procedures in general accordance with provisions of ASTM D1586; a donut hammer was used to advance the SPT sampler. The borings were advanced to depths of about 14 to 27 feet below the existing Site and sidewalk grades. During soil drilling, near-continuous soil sampling was performed, where possible, to top of rock. After rock was encountered, the borings were advanced into rock using a 5-foot-long, NX-size, double-tube core barrel to obtain rock core samples. The rock cores in each boring were advanced at least 10 feet into NYCBC Class 1b or better rock.

Borings were completely backfilled upon completion and were patched on top with cement mortar, except for borings B1 and B4, in which temporary groundwater monitoring wells were installed. Well MW1 consisted of an about 10-foot-long, 2-inch-diameter slotted-screen PVC pipe and an about 8-foot-long, 2-inch-diameter solid PVC riser. Filter sand was placed around and extending about 2 feet above the screen pipe, an about 2-foot-thick bentonite seal was placed above the sand, and the remainder of the annular space around the PVC pipe was backfilled with drill cuttings and grout. Well MW2 consisted of an about 16.5-foot-long, 1.75-inch-diameter slotted screen PVC pipe. Filter sand was placed around the pipe extending to about 1.5 feet below top of basement slab; the remaining annular space around the pipe up to the top of the basement slab was filled with bentonite. The wells were flushed with clean water and then developed by bailing the water out. Our field engineer subsequently performed groundwater level measurements in the wells.

The soil and rock samples were classified in the field by our engineer using NYCBC soil and rock classifications; these classifications were later confirmed in our Elmwood Park, New Jersey laboratory.

SUB-SURFACE CONDITIONS

The sub-surface conditions encountered in the borings and test pits generally consisted of surficial fill overlying Mica Schist and Gneiss rock. Boring profiles are presented as Figures 3 and 4, and copies of the boring and test pit logs are included in Appendices A and B. The following paragraphs summarize the generalized soil, rock, and groundwater conditions:

Fill

The fill was observed to extend to about 9 to 17 feet below existing sidewalk grade (i.e., to about el 23 to 31.5), and generally consisted of fine to coarse sand with varying proportions of silt and gravel. Wood, cinder, brick, and concrete fragments were also observed within the fill. The fill was generally observed to be loose to very dense as evidenced by SPT N-values ranging from 7 to 73 blows/foot; higher SPT N-values are attributed to obstructions encountered within the fill. The fill is classified as NYCBC Class 7 material.

Rock

Mica Schist and Gneiss rock was encountered beneath the fill in all the borings. In borings B2 through B5, the top about 1 to 5 feet of rock was observed to be in a weathered condition, and is classified as NYCBC Class 1d material. NYCBC Class 1c or better rock was encountered below the weathered rock in borings B2 through B5 and below the fill in borings B1 and B6. The top of NYCBC Class 1b or better rock was encountered in the borings ranging between about el 23 and about el 29.5.

Groundwater

Static groundwater level was measured in the temporary monitoring wells MW1 and MW2 at about el 27.5 and about el 31, respectively. Our field engineer was unable to lower the water levels in these wells below the above mentioned static levels using a plastic bailer.

We researched and reviewed FEMA Preliminary Flood Insurance Rate Map, Community Panel Number 3604970201G, effective date of 5 December 2013. A portion of the map is reproduced as Figure 5. This map indicates the Site is located in an area identified as Zone X: "Areas determined to be outside the 0.2% annual chance floodplain".

FOUNDATION DESIGN RECOMMENDATIONS

The proposed development plans call for construction of a 14.5-foot-deep basement level with top of basement slab at about el 25.5. We therefore anticipate proposed building foundations will bear at or below about el 21.5. NYCBC Class 1b or better rock was encountered at this level in all the borings performed at the Site and is suitable for supporting the building on shallow foundations bearing on rock. The 2014 NYCBC would permit column and wall foundations bearing on Class 1b or better rock to be designed using an allowable bearing pressure of 40 tons/ft^2. Regardless of loading, individual column footings should be at least 3 feet by 3 feet in plan dimensions and wall footings should be at least 2 feet wide.

A coefficient of static friction of 0.5 can be preliminarily used for design of footings bearing on rock and subject to lateral loads; if additional lateral resistance is required, we can provide supplemental recommendations for shear keys or drilled-in and grouted steel dowels. Footings subject to uplift loads can be tied down using 1-$^7/_8$-inch-diameter, 150 ksi steel, double-corrosion-protected, rock anchors; these anchors can be designed for a maximum design uplift capacity of 246 kips. Higher uplift capacity of up to 616 kips per anchor can be provided by using a similar 3-inch-diameter anchor threadbar. The minimum anchor drill-hole diameter for the 246-kip anchor should be 5 inches; the minimum anchor drill-hole diameter for the 616-kip anchor should be 8 inches. The minimum anchor bond lengths for the 246-kip and 616-kip anchors should be 20 feet and 35 feet, respectively, into NYCBC Class 1b or better rock. The minimum anchor free length should be 15 feet; the actual free length requirements will need to be finalized based on final anchor layout and group uplift requirements. The above recommendations are based on each anchor being tested to 133% of its design load and locked-off at or above the design load.

We recommend any elevator cores and associated deep pits should be located as far away from the Site perimeter as possible.

Seismic Design

We reviewed the 2014 NYCBC seismic design requirements with respect to the boring data and the proposed depth of basement excavation. Our review indicates the NYCBC would allow the use of the following seismic design parameters:

2014 NYCBC / IBC Seismic Parameter	Value
Seismic Site Class	Class B
Mapped Maximum Considered Earthquake Spectral Response Accelerations	S_s = 0.281 (short periods) S_1 = 0.073 (1-second period)
Site Coefficients as a function of Site Class and Mapped Spectral Response Acceleration	F_a = 1.00 (short periods) F_v = 1.00 (1-second period)

Below-grade Slab and Wall Construction

The below-grade walls should be assumed to be fixed against rotation and designed to sustain soil, rock, hydrostatic, surcharge, and dynamic loading. The highest groundwater level was measured at about el 31 in the temporary groundwater monitoring wells installed at the site; this water level represents a perched water condition near the top of rock. Considering the potential for the groundwater to accumulate behind the foundation walls up to this level, we recommend the perimeter foundation walls be designed to resist a hydrostatic pressure arising from a water level at el 31. In addition, the foundation walls along streets should be checked against a temporary water level near street grade, should a water main break occur along these streets. Surcharge loading along streets and the associated sidewalks should also be considered in foundation wall design. Portions of the below-grade walls along the east and south sides of the Site should be designed to withstand lateral loading from adjacent building foundations, calculated as a surcharge. A schematic diagram showing how to apply the above loads is attached as Figure 6.

The foundation walls should be socketted below the bottom of the under-slab drainage system described below a minimum of 2 feet into NYCBC Class 1b or better rock. Rock excavation should be carefully performed such that no over-break or shattering of the rock to remain occurs. Once the walls are properly constructed and socketed as indicated above, we anticipate lateral groundwater and perched water flow beneath the proposed lowest basement slab from behind the foundation walls would be minimal. It is our understanding that NYCDEP regulations require the volume of groundwater to be discharged into the City sewers to be limited to 10,000 gallons / day for the entire site; we request this be confirmed by the project Mechanical Engineer. As the excavation progresses, we would need to observe groundwater conditions at the Site to determine if upward flow at the bottom of excavation would approach, or potentially exceed, the above-described NYCDEP discharge limitations. At this time however, considering the relatively good quality rock encountered in the borings, we anticipate such upward flow to be relatively low, and that the lowest level basement slab can be designed as a slab-on-grade bearing on a layer of compacted crushed drainage stone placed over compacted excavation subgrade. We recommend the slab-on-grade constructed as described above should be designed using a modulus of subgrade reaction of 150 pounds/inch^3. We recommend a pressure slab option be priced as an add-alternate, in case groundwater conditions in the mass excavation preclude the use of a slab-on-grade; the likelihood of encountering such a condition is considered relatively low.

The under-slab drainage system should consist of a minimum 9-inch thick layer of ¾-inch free-draining crushed stone, which should be placed over Mirafi 140N filter fabric. A network of 6-inch-diameter perforated SCH 40 drainage pipes should be placed within the stone to collect any water that may accumulate in the stone and discharge it into a sump pit for pumping and subsequent disposal. We recommend a dual chamber sump pit be provided and a 150 gpm sump pump (with a 150 gpm emergency pump) be installed in the sump pit. A Hydroduct 220 (or equivalent) filter drainage mat should be placed behind all foundation walls extending from the wall bottom or top of Class 1c or better rock, whichever is higher, to one foot below the sidewalk level. We also recommend the new below-grade walls and the lowest level slab be fully waterproofed using a positive-side membrane-type waterproofing system, and water-stops be placed at all below-grade joints.

OTHER GEOTECHNICAL CONSIDERATIONS

Demolition, Excavation, and Dewatering

Demolition of the on-site buildings should be performed with care so as not to cause damage or loss of support to the existing neighboring / bordering buildings. The existing vaults below adjacent sidewalks should be located by means of a survey to determine if they may impact the demolition or future construction work. Prior to demolition, pre-demolition conditions documentation should be performed at the Site as subsequently discussed herein to document existing conditions of the neighboring / bordering buildings. The project Environmental Engineer should advise if test pits should be excavated after building demolition to identify the locations, number, and conditions of any underground storage tanks (USTs), and presence of any potentially impacted soil and groundwater at the Site.

After demolition, the Site will be mass-excavated about 15 feet to the proposed basement slab subgrade level. Additional excavation will be required for foundation and pit construction. We anticipate excavation in the fill soils can be performed using typical excavation equipment. We anticipate about 8 to 10 feet of rock excavation and removal will be necessary for foundation construction in most of the areas. Rock excavation in the weathered rock can be performed using conventional excavation equipment. Although the quality of rock to be removed is relatively good below the top layer of weathered rock, we expect rock excavation can be accomplished using hydraulic hoe rams, splitters, and excavators fitted with special ripping teeth. Rock excavation should be controlled to prevent excessive rock over-break and vibrations that may adversely affect the existing neighboring / bordering structures. If the Contractor elects to use blasting to expedite the rock excavation process, the Contractor should strictly comply with all applicable monitoring requirements. A test and production blasting program should be prepared and submitted to the NYC Fire Department, and other applicable City agencies for review, approval, and permitting; the plan should first be provided to the Owner and the Geotechnical Engineer for review. As part of the test blasting program, rock blasting should be completed in a controlled area to demonstrate Contractor's means and methods will not result in excessive vibrations or other potential adverse impacts, and rock over-break beyond the limits of the blast area.

Rock excavation work will need to be performed very carefully to ensure vibration and movement threshold levels established for the project are not exceeded, and neighboring buildings and utilities are not adversely impacted or damaged. Vibration and rock over-break control methods should be used during excavation. These methods typically include line and channel drilling, and in case of blasting, pre-splitting and smooth wall blasting. The purpose of these methods is to: ensure an air cushion, establish a crack plane between the periphery holes, and thereby minimize the propagation of primary vibrations and strain cracking in the rock mass beyond the excavation perimeter. The method and procedure selected will depend upon the relative location of the excavation work with respect to the neighboring / bordering buildings. Line drilling should consist of 2 to 3-inch-diameter holes spaced center-to-center at no more than twice their diameter. Channel drilling should consist of 2 to 3-inch-diameter holes drilled adjacent to one another. Excavation vibration control can also be achieved by limiting the equipment impact energy, or in case of blasting, by reducing the charge per delay, to that value that would produce non-damaging levels of ground vibrations. The ground vibrations and adjacent building movements should be monitored and reviewed during the work.

The limiting peak particle velocity of vibration at the neighboring NYC Landmarked buildings is 0.5 inch/second per the NYCDOB Technical Policy and Procedure Notice (TPPN) 10/88 requirements; the vertical and lateral movement threshold at the NYC Landmarked buildings is 0.25 inch. At this time, we recommend a preliminary limiting resultant peak particle velocity of 1 inch/second measured at the other neighboring / bordering buildings; limiting vibration levels at neighboring utilities should be determined after discussions with the utility owners. These are tentative values and field conditions may require adjustments to lower threshold levels. We also recommend a preliminary limiting movement at the non-Landmarked neighboring / bordering buildings be determined by a Structural Engineer after observation of these structures. Neighboring Landmarked building monitoring should be performed per the applicable NYCDOB TPPN 10/88 requirements.

The Contractor's soil and rock excavation, removal, and associated monitoring plan should be prepared by Contractor's Professional Engineer licensed in the State of New York and experienced in similar controlled soil and rock excavation and removal activities. This plan should be submitted to the Owner and the Geotechnical Engineer for review.

We anticipate mass and foundation excavation can be performed without the need for dewatering, except for rainwater and perched water pumping and discharge. Once the excavation extends into rock, if individual rock seams yielding higher flow volumes of perched water are encountered, we anticipate they can be plugged using commercially available hydro-active cement grout products to control flow of groundwater into the excavation.

The demolition debris, remnants of former foundations, fill, and rock excavated during mass and foundation excavation, along with any encountered groundwater, will need to be disposed off-site. These materials and any existing USTs should be removed and disposed of, along with any petroleum contaminated fill, soil, rock, and groundwater, per the applicable NY State DEC and NY City DEP regulations. We recommend that removal, cleaning, and disposal of any USTs encountered on-site and removal of any associated contaminated fill, soil, rock, and groundwater be fully documented for Ownership's records. The dewatering effluent will need to be properly treated, if necessary, and discharged into the City sewer system in accordance with the applicable City and State environmental regulations. We anticipate a NYCDEP sewer discharge permit will be required to allow pumping and discharge of rain and perched water. The Contractor's calculations regarding the estimated dewatering effluent volume will be required to obtain this permit. In our experience, NYCDEP limits the amount of temporary dewatering effluent entering into the City sewer system. Note if the rate of dewatering effluent discharge from the site exceeds the initially approved and paid-for allowance, the City may require additional higher per gallon fees to be paid in order to discharge the additional effluent into the sewer system.

Temporary Excavation Support and Underpinning

Temporary soil and rock excavation support will be required along the East 30th Street and Fifth Avenue sidewalks bordering the northern and western portions of the Site, respectively. In addition, rock excavation support will be required along the 2 East 30th Street and 275 Fifth Avenue buildings; underpinning is anticipated to be required along the one-story extension of the 2 East 20th Street building to properly transfer loads from this building sufficiently below the proposed bottom of excavation level.

We anticipate a laterally braced drilled soldier pile and lagging temporary excavation support system can be used for soil retention along the adjacent sidewalks. Where it is possible to excavate the weathered / fractured rock using a conventional backhoe excavator, rock excavation support system will need to be designed and constructed similar to the soil excavation support / retention system. We anticipate excavation sidewalls in NYCBC Class 1c or better rock can be supported using conventional tie-back anchors and rock bolts. If necessary, shotcrete and / or wire mesh and steel dowels may also be required to provide supplemental support to the rock face. We anticipate a conventional laterally braced underpinning pier system can be used along the one-story extension to the 2 East 30th Street building. Excavation support and underpinning elements should be installed with excavation carefully performed in front of these systems such that no adverse impact, damage, or loss of ground from beneath the neighboring / bordering building foundations, streets, sidewalks, and below-grade utilities occurs.

Where underpinning and lateral bracing elements are expected to extend beyond the Site limits, prior permission should be obtained from the adjacent property owners to confirm they will allow such elements to extend beneath their property. In addition, locations and depths of street utilities including the existing building vaults located beneath the 30th Street and Fifth Avenue sidewalks should be verified and permission from the necessary City agencies should be obtained prior to installing tie-back anchors or similar bracing systems beyond the site property limits. Internal rakers, and diagonal and cross braces should be installed wherever possible to avoid installing tie-back anchors that can potentially cause adverse impacts to adjacent utilities.

The 2014 NYCBC Section 1814.1 requires site-specific plans and details to be prepared and submitted for underpinning and temporary excavation support systems. All temporary excavation support and underpinning systems should be designed by the Excavation Contractor's Professional Engineer, licensed in the State of New York. The imposed soil and rock loading, temporary hydrostatic pressure, and neighboring building foundation and other surcharge loading (including that for streets, sidewalks, yards, and temporary construction equipment and staging) should be accounted for in the design. Design drawings should be submitted, signed and sealed, for NYCDOB review and approval. A New-York-State-licensed Professional Engineer, independently engaged by Ownership, will need to provide Special Inspection of the temporary excavation support and underpinning work as required by the NYCDOB. During rock excavation, the excavated rock face should be inspected by the Contractor's Professional Engineer and the exposed rock fractures and joints should be carefully mapped, so they can make the necessary adjustments to their rock face bracing design.

Protection of Neighboring Structures

Neighboring / bordering NYC Landmarks and other buildings, and all utilities, sidewalks, and streets surrounding the Site should be protected against adverse impact during demolition, excavation, and subsequent construction. Special care should be taken during demolition, soil and rock excavation and removal, and excavation support and underpinning construction work to ensure adverse impacts, such as development of cracks, ground loss, instability, or loss of support, do not result at the neighboring / bordering structures.

We recommend pre-demolition conditions documentation should be performed to identify existing conditions of the neighboring / bordering structures prior to start of site activities. As a minimum, pre-demolition conditions documentation should consist of photographic and supplemental video documentation of select exposed accessible portions of exterior and interior neighboring building facades within close proximity of the on-site buildings to be demolished. After demolition is complete and prior to beginning excavation and foundation construction work at the Site, post-demolition conditions documentation should also be performed. As a minimum, post-demolition conditions documentation should consist of photographic and supplemental video documentation of exposed accessible portions of exterior and interior neighboring Landmarked building facades within 90 feet of the Site; for non-Landmarked buildings, documentation should be performed for façade areas within at least 25 feet of the Site. As part of each round of documentation, ambient vibrations at the Site and at select neighboring building basement locations should be measured. In addition, crack-monitoring gauges should be installed across select cracks observed in the facades during the documentation. Prior to beginning on-site work, elevation and lateral position control points should also be established at select locations along the neighboring / bordering building exterior facades near ground and roof levels, and the initial positions of these control points should be surveyed by a surveyor licensed in the State of New York. The results of these documentations should be incorporated in the Construction Protection Plan (CPP) to be prepared for the neighboring Landmarked buildings. If NYCDOB or NYC Landmarks Preservation Commission or the neighboring building owners require documentation of any additional areas of these or any other structures, such documentation should be completed prior to initiating demolition and / or excavation activities at the Site.

The neighboring / bordering buildings should be monitored during demolition, on-site excavation, excavation support and underpinning construction, and foundation construction activities using the above-mentioned crack monitoring gauges and elevation and lateral position control points. This is necessary, so the Contractor performing the work can determine if the neighboring / bordering buildings are at risk of being adversely impacted by their work, and so the Contractor can make any necessary changes to their means and methods to avoid such adverse impacts. Vibration levels at the neighboring / bordering buildings and at the neighboring Landmarked buildings within 90 feet of the Site should also be continuously monitored during on-site activities using seismograph vibration monitors placed at strategic locations at these structures. At this time, we anticipate movement and vibration threshold levels of 0.25 inch and 0.5 inch/second, respectively, can be established at the adjacent NYC Landmarked buildings per the NYCDOB TPPN 10/88 requirements. At this time we also anticipate movement and vibration threshold levels of 0.25 inch and 1.0 inch/second, respectively, can be established for the adjacent non-Landmarked buildings. These preliminary threshold levels are subject to modifications based on results of neighboring building monitoring during construction.

CONTRACTOR AND OWNER OBLIGATIONS

The Contractor is responsible for construction quality control, which includes satisfactorily constructing the foundation system and any associated temporary works to achieve the design intent while not adversely impacting or causing loss of support to neighboring buildings, other structures, or utilities. Construction activities that can alter the existing ground conditions, such as soil and rock excavation, excavation support and underpinning construction, tie-down anchor installation and stressing, and foundation construction can also potentially induce stresses, vibrations, and movements in nearby structures and utilities, and disturb nearby structure occupants. Contractors working at the Site must ensure that their activities will not adversely affect the performance of the structures, occupants, and utilities, and take all necessary measures to protect the existing structures during construction. By using this report, the Owner agrees that Langan will not be held responsible for any damage to adjacent structures.

This report's preparation and use is based on the condition that the project construction contract between the Owner and their Contractor(s) will include: 1) Langan being added to the Project Wrap and/or Contractor's General Liability Insurance as an additional insured, and 2) language specifically stating the Foundation Contractor will defend, indemnify, and hold harmless the Owner and Langan against all claims related to disturbance or damage to adjacent structures or properties.

ADDITIONAL SERVICES

At this time we recommend the following additional geotechnical services be performed for the proposed development:

1. Technical specifications should be prepared for the geotechnical aspects of proposed construction. We anticipate the specification sections would include:
   - Sheeting, Bracing and Underpinning
   - Excavation, Filling, and Grading
   - Tie-down Rock Anchors
   - Foundation Drainage
   - Foundation Waterproofing
2. Site-specific temporary excavation support and underpinning design drawings should be prepared and submitted to the NYCDOB for approval and permitting purposes.
3. Pre- and post-demolition conditions documentation of the neighboring / bordering buildings should be performed prior to commencing on-site demolition and excavation work, respectively.
4. During construction, foundation subgrade preparation should be inspected per the special inspection requirements of the NYCBC. In addition, quality assurance inspection of foundation drainage installation and tie-down anchor installation and testing should be performed. A qualified Special Inspection Agency engaged directly by Ownership will need to perform special inspection during the temporary excavation support and underpinning work.
5. Neighboring / bordering structures should be monitored during on-site activities using seismographs, crack gauges, and elevation and lateral position control points.

Langan has investigated and interpreted the Site subsurface conditions and developed the foundation design recommendations contained herein, and is therefore best suited to perform quality assurance observation and testing of building foundation construction (shallow foundation and slab subgrade preparation, and tie-down anchor installation and testing) work. Recognizing that construction is essentially the completion of design, Langan's quality assurance observation and testing during foundation construction is necessary to maintain our continuity of responsibility as it relates to the building foundation for this project.

CLOSURE

The conclusions and recommendations given in this report represent our best engineering judgment as to the sub-surface conditions at the Site and appropriate foundation systems for the proposed construction, based on our current understanding of the proposed development plans and results of our sub-surface investigations performed at the Site to date. Recommendations given are contingent upon one another and no recommendation should be followed independent of the others. Any changes in the location, elevation, and / or loading of the proposed structure should be brought to our attention and we should be provided with the building drawings once they are finalized, so we can review, confirm, or modify (if necessary) the recommendations provided herein.

This report has been prepared to assist the Owner's Architect and Structural Engineer in their design. The recommendations given in this report should be incorporated in the final design through inclusion in the Project Construction Drawings and foundation-related technical specifications. Our office should be provided with final foundation drawings and details prepared by the Project Structural Engineer and Architect, so we can confirm our recommendations are properly incorporated in the construction documents. Our office should also review foundation-related contractor submittals and construction procedures related to the geotechnical aspects of construction. Langan cannot assume responsibility for use of this report for any areas beyond the limits of this study or for any projects not specifically discussed herein.

Environmental concerns (such as potential presence of underground storage tanks and potentially contaminated soil and groundwater) may exist at the Site and have not been addressed in detail in this report. These concerns should be addressed by the Project Environmental Engineer.

We thank you for allowing us to assist you on this interesting project. If you have any questions regarding this report, please call.

Sincerely,
Langan Engineering, Environmental, Surveying and Landscape Architecture, D.P.C.

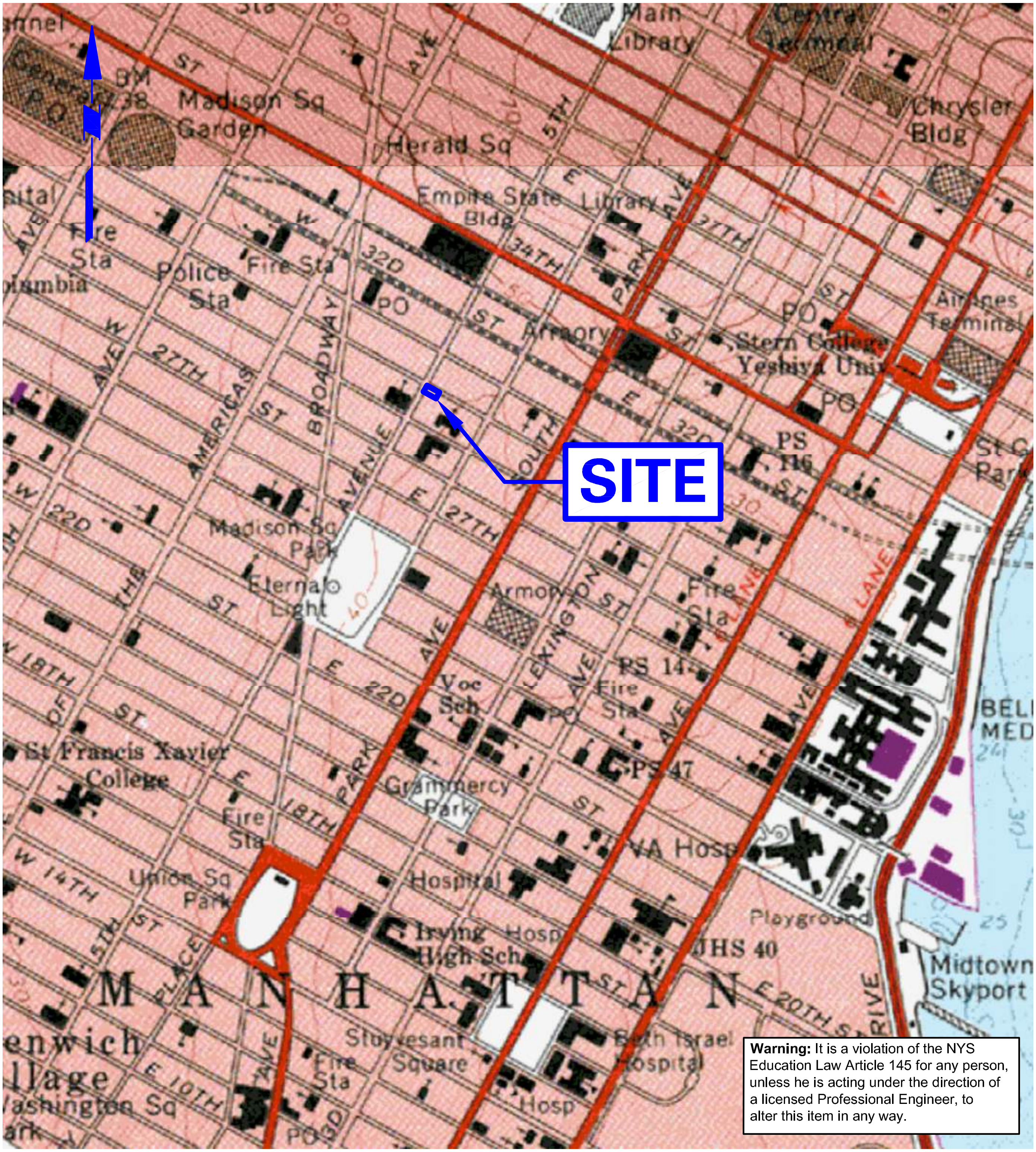
Vaidya

Satyajit A. Vaidya, P.E.
Senior Associate / Vice President

cc: Danielle Axelrod / Victor Homes
Rudy Frizzi / Langan

Attachments: Figures 1 through 6, and Appendices A and B

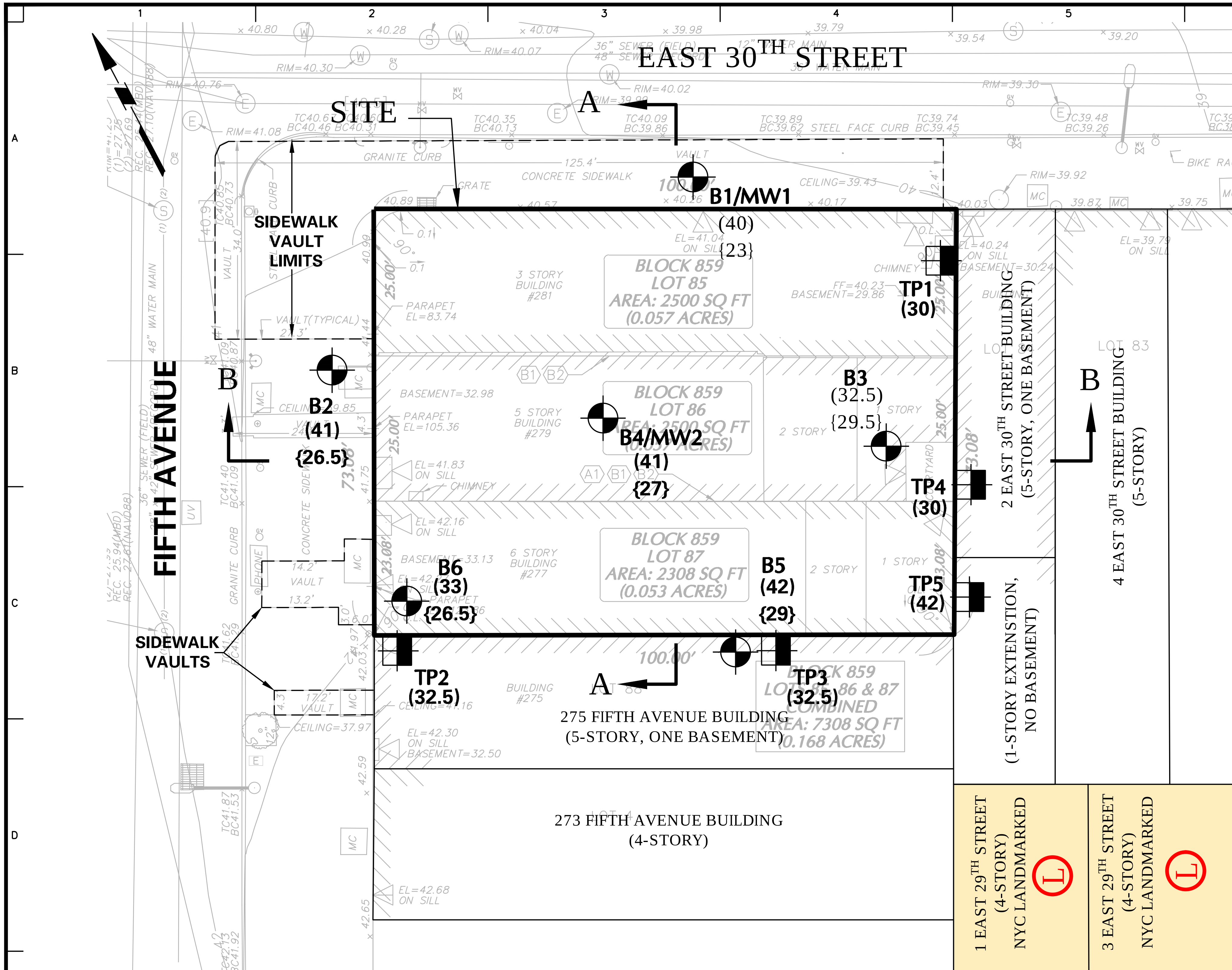
FIGURES



REFERENCE: USGS, BROOKLYN, QUADRANGLE, NEW YORK, 7.5-MINUTE SERIES (TOPOGRAPHIC), CREATED 1967, AND REVISED 1979.

LANGAN	Project	Drawing Title	Project No. 100464201	Drawing No.
River Drive Center 1, 619 River Drive Elmwood Park, NJ 07407 T: 201.794.6900 F: 201.794.0366 www.langan.com	PROPOSED 281 FIFTH AVENUE DEVELOPMENT	SITE LOCATION MAP	Date 5/18/2015	1
Langan Engineering, Environmental, Surveying and Landscape Architecture, D.P.C. Langan Engineering and Environmental Services, Inc. Langan CT, Inc. Langan International LLC Collectively known as Langan			Scale 1"=1000'	
			Drawn By QOO	
NJ CERTIFICATE OF AUTHORIZATION No. 24GA27996400	NEW YORK NEW YORK		Checked By SAV	

Filename: X:\EP\data2\100464201\Cadd Data - 100464201\Dwg\Geotechnical Report Figures\Figure 1 - USGS Site Location Map.dwg Date: 5/18/2015 Time: 10:32 User: crobles Style Table: Langan.stb Layout: A Size Sheet (Bottom)



LEGEND

B1/MW1 — APPROXIMATE LOCATION AND IDENTIFICATION NUMBER OF BORING / TEMPORARY MONITORING WELL

TP1 — APPROXIMATE LOCATION AND IDENTIFICATION NUMBER OF TEST PIT

(40) — APPROXIMATE GROUND SURFACE ELEVATION AT BORING AND TEST PIT LOCATION

{23} — APPROXIMATE ELEVATION OF TOP OF NYCBC CLASS 1b OR BETTER ROCK OBSERVED IN THE BORING

NOTES

1. THE BASE FIGURE IS REPRODUCED FROM A 4 MARCH 2015 TOPOGRAPHIC BOUNDARY AND UTILITY SURVEY PREPARED BY LANGAN.
2. ALL BORING LOCATIONS AND GROUND SURFACE ELEVATIONS ARE APPROXIMATE.
3. ELEVATIONS REFERENCED TO THE NORTH AMERICAN VERTICAL DATUM OF 1988 (NAVD88), WHICH IS 1.1 FEET ABOVE THE NATIONAL GEODETIC SURVEY VERTICAL DATUM OF 1929, NGVD29 (MEAN SEA LEVEL AT SANDY HOOK, NEW JERSEY)
4. ALL BORINGS WERE PERFORMED AND TEMPORARY MONITORING WELLS INSTALLED BY WARREN GEORGE, INC. UNDER THE FULL-TIME SPECIAL INSPECTION OF LANGAN. BORINGS B1 THROUGH B3 WERE PERFORMED AND TEMPORARY MONITORING WELL MW1 WAS INSTALLED BETWEEN 9 AND 19 JULY 2014. BORINGS B4 THROUGH B6 WERE PERFORMED AND TEMPORARY MONITORING WELL MW2 WAS INSTALLED BETWEEN 27 FEBRUARY AND 6 MARCH 2015.
5. ALL TEST PITS WERE PERFORMED BY WARREN GEORGE, INC. UNDER THE FULL-TIME OBSERVATION OF LANGAN. TEST PITS TP1 THROUGH TP4 WERE PERFORMED BETWEEN 4 AND 19 MARCH 2015. TEST PIT TP5 WAS PERFORMED ON 13 AND 14 APRIL 2015.

WARNING: IT IS A VIOLATION OF THE NYS EDUCATION LAW ARTICLE 145 FOR ANY PERSON, UNLESS HE IS ACTING UNDER THE DIRECTION OF A LICENSED PROFESSIONAL ENGINEER, TO ALTER THIS ITEM IN ANY WAY.

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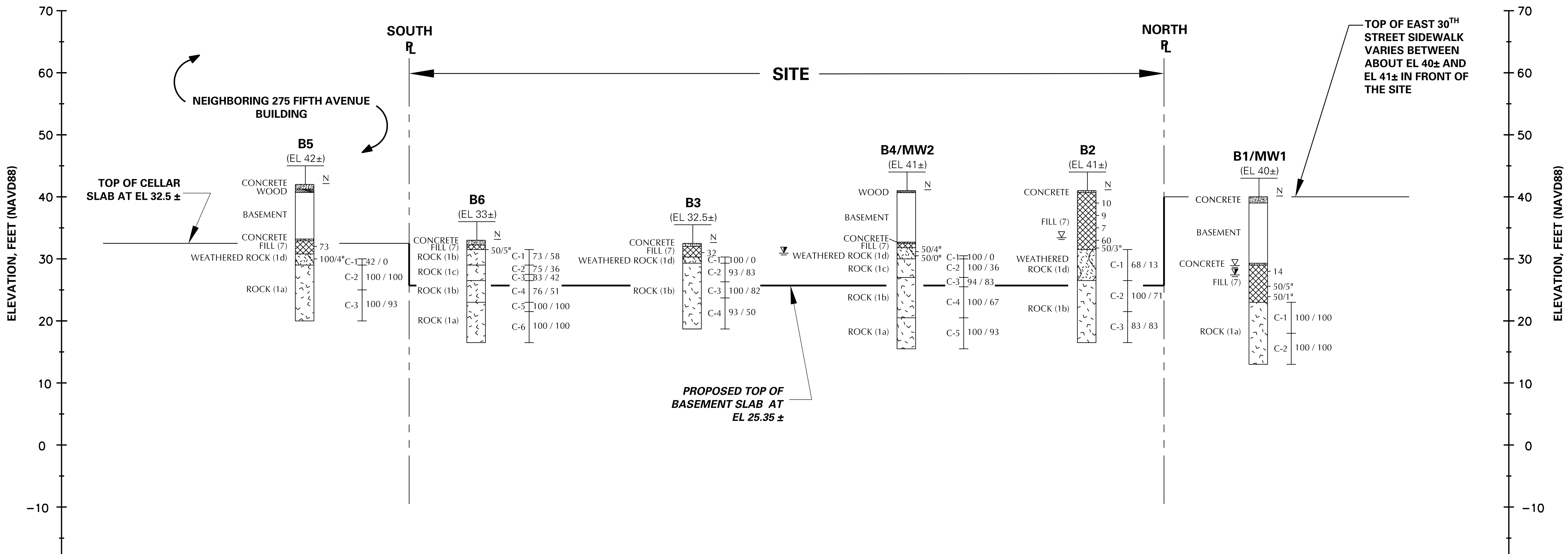
Project	Drawing Title	Project No.	Drawing No.
PROPOSED 281 FIFTH AVENUE DEVELOPMENT NEW YORK NEW YORK	INVESTIGATION LOCATION PLAN	100464201	1
		Date: 5/18/2015	
		Scale: 1"=20'	
		Drawn By: QOO	
		Checked By: SAV	

Filename: \\langan.com\data\EP\data2\100464201\Cadd Data - 100464201\Dwg\Geotechnical Report Figures\Figure 2 - Boring Locaction Plan_New Survey.dwg Date: 5/18/2015 Time: 09:34 User: qowokoniran Style Table: Langan.stb Layout: ANSIB-BL



SECTION A-A

VERTICAL SCALE: 1"=10'

HORIZONTAL SCALE: N.T.S.

BORING KEY:

BX/MWX
(EL XX±)

(XX) N

C-X REC / RQD

LEGEND:

BX	BORING IDENTIFICATION
MWX	TEMPORARY GROUNDWATER MONITORING WELL IDENTIFICATION
(EL XX)	APPROXIMATE GROUND SURFACE ELEVATION AT BORING LOCATION
N	STANDARD PENETRATION RESISTANCE; NUMBER OF BLOWS OF A 140 LB DONUT HAMMER FREE FALLING 30 INCHES TO DRIVE A 2-INCH-O.D. SPLIT SPOON SAMPLER 12 INCHES AFTER 6 INCHES OF INITIAL PENETRATION
(XX)	2014 NYC BUILDING CODE MATERIAL CLASSIFICATION
C-X	ROCK CORE RUN IDENTIFICATION AND LENGTH
REC	ROCK CORE RECOVERY %
RQD	ROCK QUALITY DESIGNATION %
▼	GROUNDWATER LEVEL OBSERVED IN THE TEMPORARY GROUNDWATER MONITORING WELL 24 HOURS AFTER DEVELOPING THE WELL
▽	GROUNDWATER LEVEL FIRST OBSERVED IN BORING

NOTES:

1. ALL BORINGS WERE PERFORMED AND TEMPORARY GROUND WATER MONITORING WELLS INSTALLED BY WARREN GEORGE, INC. UNDER THE FULL-TIME SPECIAL INSPECTION OF LANGAN. BORINGS B1 THROUGH B3 WERE PERFORMED AND TEMPORARY GROUND WATER MONITORING WELL MW1 WAS INSTALLED BETWEEN 9 AND 19 JULY 2014. BORINGS B4 THROUGH B6 WERE PERFORMED AND TEMPORARY GROUND WATER MONITORING WELL MW2 WAS INSTALLED BETWEEN 27 FEBRUARY AND 6 MARCH 2015.
2. THIS PROFILE SHOWS GENERALIZED SUBSURFACE CONDITIONS AT THE RESPECTIVE BORING LOCATIONS. VARIATIONS IN CONDITIONS SHOULD BE EXPECTED BETWEEN BORINGS. FOR A DETAILED DESCRIPTION OF CONDITIONS ENCOUNTERED, SEE BORING LOGS.
3. ALL BORING LOCATIONS ARE APPROXIMATE. GROUND SURFACE ELEVATIONS AT THE BORING LOCATIONS, FIFTH AVENUE AND EAST 30TH STREET SIDEWALK ELEVATIONS, AND NEIGHBORING BUILDING CELLAR ELEVATIONS ARE INFERRED FROM A 4 MARCH 2015 TOPOGRAPHIC BOUNDARY AND UTILITY SURVEY PREPARED BY LANGAN.
4. ELEVATIONS REFERENCED TO THE NORTH AMERICAN VERTICAL DATUM OF 1988, NAVD88, WHICH IS 1.1 FEET ABOVE THE NATIONAL GEODETIC SURVEY VERTICAL DATUM OF 1929, NGVD29 (MEAN SEA LEVEL AT SANDY HOOK, NEW JERSEY).

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NJ CERTIFICATE OF AUTHORIZATION No. 24GA27996400

Project

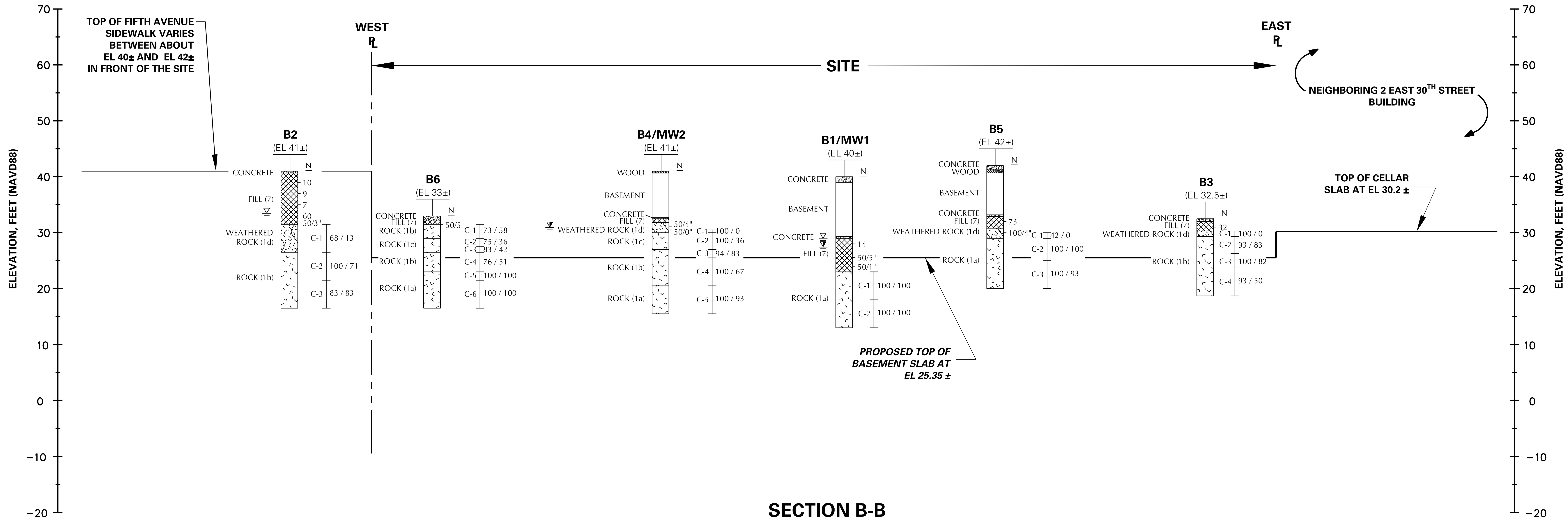
PROPOSED 281 FIFTH AVENUE DEVELOPMENT

NEW YORK NEW YORK

Drawing Title

BORING PROFILE A-A

Project No. 100464201	Drawing No. 3
Date 5/18/2015	
Scale AS SHOWN	
Drawn By QOO	
Checked By SAV	

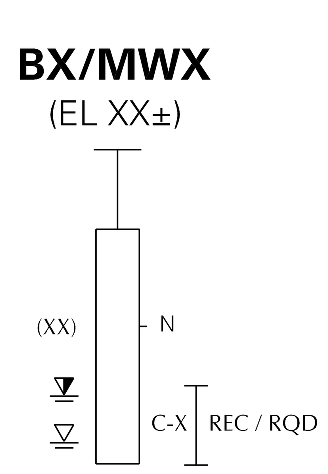


SECTION B-B

VERTICAL SCALE: 1"=10'

HORIZONTAL SCALE: N.T.S.

BORING KEY:



LEGEND:

BX	BORING IDENTIFICATION
MWX	TEMPORARY GROUNDWATER MONITORING WELL IDENTIFICATION
(EL XX)	APPROXIMATE GROUND SURFACE ELEVATION AT BORING LOCATION
N	STANDARD PENETRATION RESISTANCE; NUMBER OF BLOWS OF A 140 LB DONUT HAMMER FREE FALLING 30 INCHES TO DRIVE A 2-INCH-O.D. SPLIT SPOON SAMPLER 12 INCHES AFTER 6 INCHES OF INITIAL PENETRATION
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NOTES:

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ABU DHABI ATHENS DOHA DUBAI ISTANBUL PANAMA
NJ CERTIFICATE OF AUTHORIZATION No. 24GA27996400

Project

PROPOSED
281 FIFTH AVENUE
DEVELOPMENT

NEW YORK NEW YORK

Drawing Title

BORING PROFILE
B-B

Project No. 100464201	Drawing No.
Date 5/18/2015	
Scale AS SHOWN	4
Drawn By QOO	
Checked By SAV	

LEGEND:

ZONE X — AREAS DETERMINED TO BE OUTSIDE THE 0.2% ANNUAL CHANCE FLOODPLAIN.

Warning: It is a violation of the NYS Education Law Article 145 for any person, unless he is acting under the direction of a licensed Professional Engineer, to alter this item in any way.

REFERENCE: FEMA PRELIMINARY FLOOD INSURANCE RATE MAP, COMMUNITY PANEL NUMBER 3604970201G, EFFECTIVE DATE 5 DECEMBER 2013.

LANGAN River Drive Center 1, 619 River Drive Elmwood Park, NJ 07407 T: 201.794.6900 F: 201.794.0366 www.langan.com Langan Engineering, Environmental, Surveying and Landscape Architecture, D.P.C. Langan Engineering and Environmental Services, Inc. Langan CT, Inc. Langan International LLC Collectively known as Langan	Project **PROPOSED 281 FIFTH AVENUE DEVELOPMENT** NEW YORK NEW YORK	Drawing Title **FEMA PRELIMINARY FLOOD INSURANCE RATE MAP**	Project No. 100464201 Date 5/18/2015 Scale 1"=1000' Drawn By QOO Checked By SAV	Drawing No. **5**

Filename: X:\EP\data2\100464201\Cadd Data - 100464201\Dwg\Geotechnical Report Figures\Figure 5 - FIRM Map.dwg Date: 5/18/2015 Time: 10:35 User: crobles Style Table: Langan.stb Layout: A Size Sheet (Bottom)

APPENDIX C

Structural 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, BOTTOM
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
LLV	LONG LEG VERTICAL
LP	LOW POINT
LRFD	LOAD RESISTANCE FACTOR DESIGN
LT	LIGHT
LW	LIGHT WEIGHT
L.W.	LONG WAY
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
STD	STANDARD
STIFF	STIFFENER
STL	STEEL
STRUCT	STRUCTURAL
E.W.	EACH WAY
SW	SHEARWALL
S.W.	SHORT WAY
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
PL	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 WAS 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 DEMOLITION 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. THE CONTRACTOR'S PROFESSIONAL ENGINEER SHALL PREPARE AND FILE THE REQUIRED FORMS FOR THE WORK WITH THE BUILDING DEPARTMENT.

FOUNDATION NOTES:

A. EXCAVATION

1. ALL SPREAD FOOTINGS AND FOUNDATION WALLS SHALL BEAR ON 40 TSF ROCK NYCBC CLASS 1b OR BETTER, U.O.N ON PLAN
2. WHERE THE REQUIRED BEARING MATERIAL IS NOT FOUND AT THE ANTICIPATED ELEVATION SHOWN (ELEVATION BASED ON BORING INTERPOLATED DATA) THE FOOTINGS SHALL BE LOWERED TO A DEPTH AT WHICH THE REQUIRED BEARING CAPACITY IS FOUND.
3. 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 FOUNDATIONS CONTRACTOR IS TO UNDERPIN EXISTING FOUNDATION. CONTRACTOR IS TO ESTABLISH EXISTING CONDITIONS BEFORE COMMENCING WORK AND NOTIFY THE ENGINEER.
4. 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. THE CONTRACTOR'S PROFESSIONAL ENGINEER SHALL PREPARE AND FILE THE REQUIRED FORMS FOR THE WORK WITH THE BUILDING DEPARTMENT.

B. CONCRETE AND STEEL REINFORCEMENT

1. NO CONCRETE FOOTING OR FOUNDATION WALL SHALL BE POURED UNTIL SUBGRADE FOR SAME HAS BEEN APPROVED BY A LICENSED PROFESSIONAL ENGINEER.
2. ALL CONCRETE SHALL BE NORMAL WEIGHT CONTROLLED CONCRETE, U.O.N., AND COMPLY WITH A.C.I. BUILDING CODE AND THE CURRENT NEW YORK CITY BUILDING CODE.
3. CONCRETE STRENGTH SHALL BE AS FOLLOWS, UNLESS OTHERWISE NOTED: 8000 PSI.
   - -FOUNDATION FOR COLUMNS/WALLS 7000 PSI.
   - -BUTTRESSES, FOUNDATION WALLS 7000 PSI.
   - -FOOTING SEALERS/ MUD SLAB 3000 PSI.
   - -SLAB ON GROUND* 4000 PSI.

   *IF SLAB ON GROUND IS POURED BEFORE THE COLUMNS ABOVE AND THE COLUMN STRENGTH IS 5000 PSI OR GREATER, THE SLAB ON GROUND STRENGTH IS TO BE ACCORDING TO THE "DETAIL OF BEAM AND SLAB CONCRETE PLACEMENT AT HIGH STRENGTH COLUMN." IN ADDITION THE DOWELS EXTENDING ABOVE THE FOOTINGS, PIERS OR PILE CAPS ARE TO BE LENGTHENED A MIN. 12", BEYOND THAT SHOWN OR CALLED FOR IN DETAILS.
4. ALL STEEL REINFORCEMENT SHALL HAVE AN ULTIMATE TENSILE STRENGTH OF 90,000 PSI AS PER A.S.T.M. A615-83 GRADE 60. THE CONTRACTOR SHALL FURNISH AND INSTALL ALL THE NECESSARY CHAIRS, REBARS, TIES, SPACERS, ETC., TO SECURE AND SUPPORT THE REINFORCING WHILE PLACING THE CONCRETE. ALL #14 AND LARGER DIAMETER REBAR MUST TO BE GRADE 97 REINFORCEMENT. NOTES FOR GRADE 97 REINFORCEMENT (#14, #18, #20, & #24 BARS):
   a. ALL 97 ksi REINFORCING SHALL MEET THE NYC BUILDING DEPARTMENT BULLETION 2010-003 FOR MATERIAL AND SPECIAL INSPECTION REQUIREMENTS.
   b. THE YIELD STRENGTH OF 97 ksi REBAR SHALL BE TAKEN AS THE STRESS CORRESPONDING TO A STRAIN OF 0.35% AS EVALUATED BY ASTM A370.
   c. MINIMUM REINFORCEMENT ELONGATION SHALL NOT BE LESS THAN 6% PER ASTM A370.
   d. MECHANICAL COUPLERS SHALL BE PROVIDED THAT CONFORM TO SECTION 3.5 OF AC 237 AND CH. 12 OF ACI 318. ADDITIONALLY, MECHANICAL COUPLERS SHALL COMPLY WITH AC 133. SPLICES SHALL BE STAGGERED SUCH THAT NO MORE THAN HALF OF THE TOTAL REINFORCEMENTIS SPLICED WITHIN 36 INCHES. ALTERNATE COUPLERS LOCATION AT EVERY OTHER BAR PER REQUIREMENTS OF THIS NOTE IN SHEARWALLS AND CONCRETE COLUMNS.
5. ALL BARS MARKED CONTINUOUS, SHALL BE LAPPED ACCORDING TO TABLES ON FO-203 AT SPLICES AND CORNERS EXCEPT AS OTHERWISE SHOWN ON PLANS. LAP CONTINUOUS TOP BARS AT CENTER BETWEEN SUPPORTS AND BOTTOM BARS AT SUPPORTS. HOOK TOP BARS AT DISCONTINUOUS ENDS.
6. VERTICAL CONSTRUCTION JOINTS IN ALL 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 STRAP BEAMS.
7. IN NO CASE SHALL TRUCKS, BULLDOZERS, OR OTHER HEAVY EQUIPMENT BE PERMITTED CLOSER THAN 8'-0" FROM ANY FOUNDATION WALL UNLESS THE WALL IS BRACED BY THE CONTRACTOR.
8. 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.
9. 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.
10. 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".
11. FOR PIER SIZES SEE STRUCTURAL DRAWINGS. WHERE PIER IS REQUIRED BUT NOT SHOWN ON PLANS THE SIZE OF THE PIER SHOULD BE AS SHOWN IN TYP. DETAIL ON FO-201.
12. WHERE A PIER IS INDICATED ON THE FOUNDATION PLAN BUT ELIMINATED IN THE FIELD (GOOD MATERIAL HIGHER THAN ASSUMED) THE ENGINEER SHALL BE NOTIFIED AS DEPTH OF FOOTING MAY NEED TO BE INCREASED.
13. 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.
14. THE STRUCTURAL ENGINEER OR HIS FIELD QUALIFIED REPRESENTATIVE MUST CHECK AND APPROVE ALL STEEL REINFORCING PRIOR TO CONCRETE PLACEMENT.

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 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.

SUPERSTRUCTURE 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:
   SLABS AND BEAMS U.O.N. ON PLANS 8600 PSI FROM 1ST TO 19TH FL.
   SLABS AND BEAMS U.O.N. ON PLANS 7200 PSI FROM 20TH TO UP
   SHEAR WALLS & COLUMNS SEE COLUMN SCHEDULE
   LINK BEAMS SAME AS SHEARWALL STRUCTURE
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, PIPEDE 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 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"x4½"x3½" 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. VERTICAL CONSTRUCTION JOINTS 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. TEMPORARY SHORING AND RESHORING SHALL REMAIN IN PLACE AT LEAST 28 DAYS AFTER PLACEMENT OF CONCRETE.
11. 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. ALL #14 AND LARGER DIAMETER REBAR MUST TO BE GRADE 97 REINFORCEMENT. NOTES FOR GRADE 97 REINFORCEMENT (#14, #18, #20, & #24 BARS):
   a. ALL 97 ksi REINFORCING SHALL MEET THE NYC BUILDING DEPARTMENT BULLETION 2010-003 FOR MATERIAL AND SPECIAL INSPECTION REQUIREMENTS.
   b. THE YIELD STRENGTH OF 97 ksi REBAR SHALL BE TAKEN AS THE STRESS CORRESPONDING TO A STRAIN OF 0.35% AS EVALUATED BY ASTM A370.
   c. MINIMUM REINFORCEMENT ELONGATION SHALL NOT BE LESS THAN 6% PER ASTM A370.
   d. MECHANICAL COUPLERS SHALL BE PROVIDED THAT CONFORM TO SECTION 3.5 OF AC 237 AND CH. 12 OF ACI 318. ADDITIONALLY, MECHANICAL COUPLERS SHALL COMPLY WITH AC 133. SPLICES SHALL BE STAGGERED SUCH THAT NO MORE THAN HALF OF THE TOTAL REINFORCEMENTIS SPLICED WITHIN 36 INCHES. ALTERNATE COUPLERS LOCATION AT EVERY OTHER BAR PER REQUIREMENTS OF THIS NOTE IN SHEARWALLS AND CONCRETE COLUMNS.
3. 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.
4. THE STRUCTURAL ENGINEER OR HIS FIELD QUALIFIED REPRESENTATIVE MUST CHECK AND APPROVE ALL STEEL REINFORCEMENT PRIOR TO CONCRETE PLACEMENT.
5. 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.
6. 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 AND THE NEW YORK CITY BUILDING CODE. A SPECIAL AMENDMENT FORM 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 IS TO BE PLACED BEFORE SUCH AN AMENDMENT IS APPROVED BY THE BUILDING DEPARTMENT.
3. DESIGN AND CONSTRUCTION OF FORMWORK IS TO COMPLY WITH THE A.C.I. 318 BUILDING CODE AND THE NEW YORK CITY BUILDING CODE AS AMENDED.
4. TRANSPORTING, PLACING, CURING AND DEPOSITING OF CONCRETE SHALL COMPLY WITH THE A.C.I. BUILDING CODE.
5. ALL REINFORCING BARS SHALL BE DEFORMED BARS CONFORMING TO "SPECIFICATIONS FOR DEFORMED BILLET-STEEL BARS FOR CONCRETE REINFORCEMENT" A.S.T.M. A615 GRADE 60. THE STEEL SUPPLIER SHALL PROVIDE THE ENGINEER WITH AN AFFIDAVIT OF THE PRODUCER OF STEEL CERTIFYING THAT THE STEEL MEETS THE REQUIREMENTS OF THE A.S.T.M.
6. ALL STRUCTURAL STEEL (LINTELS, DUNNAGE BEAMS, ETC.) SHALL CONFORM TO A.S.T.M. A-36, U.O.N.

D. SEISMIC AND WIND CRITERIA

1. THE STRUCTURE HAS BEEN DESIGNED IN ACCORDANCE WITH THE LATEST NEW YORK CITY BUILDING CODE (NYCBC 2014).
2. WIND DESIGN DATA:
   PER WIND TUNNEL LAB REPORT BY BLWT DATED NOVEMBER 24, 2015
3. EARTHQUAKE DESIGN DATA:
   AS PER GEOTECHNICAL REPORT BY LANGAN DATED MARCH 6, 2015
   - SEISMIC IMPORTANCE FACTOR = 1
   - $S_S = 0.281$, $S_1 = 0.073$
   - SITE CLASS B
   - SEISMIC DESIGN CATEGORY = B
   - SEISMIC FORCE RESISTING SYSTEM = ORDINARY REINFORCED CONCRETE SHEARWALLS
   - RESPONSE MODIFICATION FACTORS: R = 5
   - ANALYSIS PROCEDURE USED = EQUIVALENT STATIC METHOD
   - DEFLECTION AMPLIFICATION FACTOR C_D = 4.5
   - DESIGN BASE SHEAR (V) : E/W = 875 kips; N/S = 875 kips
   - SEISMIC RESPONSE COEFFICIENT (C) : E/W = 0.0082; N/S = 0.0082
4. STRUCTURAL SEPARATIONS (NYCBC-1613.7.): ALL STRUCTURES SHALL BE SEPARATED FROM ADJACENT STRUCTURES. WHEN A STRUCTURE ADJOINS A PROPERTY LINE NOT COMMON TO A PUBLIC WAY (TYPICALLY SIDE OR REAR LOT LINES), THAT STRUCTURE SHALL ALSO BE SET BACK FROM THE PROPERTY LINE BY AT LEAST 1 INCH FOR EACH 50 FEET OF HEIGHT AND A MINIMUM OF 1 INCH FOR STRUCTURE WITH HEIGHTS LESS THAT 50 FEET.

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

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

CONTROLLED 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
MASONRY	1704.5
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
UNDERPINNING	1704.20.3
WALL PANELS, CURTAIN WALLS AND VENEERS (ATTACHMENT TO BUILDING)	1704.10
SPRAYED FIRE RESISTANT MATERIALS	1704.11
STRUCTURAL SAFETY - STRUCTURAL STABILITY	1704.20
EXCAVATION - SHEETING, SHORING AND BRACING	1704.20 & 3304.4.1
FIRESTOP, DRAFTSTOP AND FIREBLOCK SYSTEMS	1704.27
PROGRESS INSPECTION	
FOOTING AND FOUNDATION	109.3.1
FINAL	28-116.2.4.2 & 109.5 AND DIRECTIVE 14-(1975)

* 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.

LOADING SCHEDULE

OCCUPANCY	PARTITION/FILL &/OR FINISHES (PSF)	CEIL. & MECH. (PSF)	LIVE LOAD
ROOF	30	5	100
STAIRS	–	5	100
CORRIDORS	30	5	100
RESIDENTIAL	12	3	40
MECHANICAL ROOMS	20	5	75
PRIVATE TERRACE	30	5	60
LOBBIES/PUBLIC TERRACE/RETAIL	30	5	100

LEGEND:

1. * INDICATES ADDITIONAL WIND BARS
2. [...] INDICATES THE BOTTOM OF FOUNDATION WALL ELEVATION
3. <...> INDICATES THE TOP OF FOUNDATION WALL ELEVATION
4. (...) INDICATES BOTTOM OF PILE CAP ELEVATION
5. Fxxxx INDICATES FOOTING MARK
6. (XXxXX) INDICATES SIZE OF PIER IN INCHES, FIRST DIMENSION SHOWN IS IN THE EAST-WEST DIRECTION.
7. INDICATES ROCK ANCHOR.
8. INDICATES UNDERSLAB DRAINAGE PIPE
9. INDICATES CLEANOUT
10. INDICATES ADDITIONAL TOP REINFORCEMENT AT SUPPORTS
11. INDICATES ADD'L BOTTOM REINFORCING AT SUPPORTS
12. INDICATES ADDITIONAL TOP REINFORCEMENT CONTINUOUS BETWEEN SUPPORTS
13. INDICATES ADDITIONAL BOTTOM REINFORCEMENT CONTINUOUS BETWEEN SUPPORTS
14. INDICATES WALKING COLUMN MID-DEPTH SLAB REINFORCEMENT
15. INDICATES D6 LENTON TERMINATOR
16. INDICATES MOMENT CONNECTION
17. INDICATES CAMBERS
18. INDICATES STRUCTURAL STEEL BEAMS
19. 1ST & 4TH LAYERS; 2ND & 3RD LAYERS — INDICATES ORDER OF BAR PLACEMENT AS SHOWN ON PLAN.
20. INDICATES CHANGE IN ELEVATION
21. INDICATES CONCRETE COLUMN/SHEARWALL/FOUNDATION WALL
22. INDICATES CONCRETE COLUMN/SHEARWALL BELOW
23. INDICATES CONCRETE WALL
    INDICATES SLAB OPENING (FIRST DIMENSION IS IN EAST-WEST DIRECTION)
24. INDICATES COLUMN ABOVE OR BELOW
25. (XX) INDICATES COLUMN DESIGNATION
26. (P) INDICATES POST DESIGNATION
27. (SW-X) INDICATES SHEARWALL DESIGNATION

POST SCHEDULE:

- (P1) INDICATES HSS 10x8x½ STEEL TUBE POST.
- (P1A) INDICATES HSS 10x10x⅝ STEEL TUBE POST.
- (P2) INDICATES 12x14 CONC. POST REINF. W/4-#7 VERT. & #3 TIES @12" O.C.
- (P3) INDICATES 18x18 CONC. POST REINF. W/4-#7 VERT. & #4 TIES @12" O.C.

281 Fifth Avenue

NEW YORK, NY

OWNER:
VICTOR NOMAD LLC

CONSTRUCTION MANAGER:
LEND LEASE
200 PARK AVENUE, 9TH FLOOR
NEW YORK, NY 10166
TEL: 212 592 6700

ARCHITECT :
RAFAEL VIÑOLY ARCHITECTS PC
50 VANDAM STREET
NEW YORK, NY 10013
TEL: 212 924 5060 FAX: 212 924 5858

STRUCTURAL ENGINEER:
WSP GROUP
228 EAST 45TH STREET, 3RD FLOOR
NEW YORK, NY 10017
TEL: 212 687 9888

MEP/FP/IT ENGINEER:
MGE ENGINEERING
116 WEST 32ND STREET, 12TH FLOOR
NEW YORK, NY 10001
TEL: 212 643 9055

GEOTECH CIVIL ENGINEER:
LANGAN ENGINEERING
300 KIMBALL DRIVE, 4TH FLOOR
PARSIPPANY, NJ, 07054
TEL: 973 560 4900

VERTICAL TRANSPORT. CONSULTANT:
VAN DEUSEN & ASSOCIATES
120 EAGLE ROCK AVENUE, SUITE 310
EAST HANOVER, NJ, 07936
TEL: 973 994 9220

ENCLOSURE CONSULTANT:
VIDARIS, INC.
360 PARK AVENUE SOUTH, 15TH FLOOR
NEW YORK, NY, 10010
TEL: 212 689 5389

FACADE MAINTENANCE CONSULTANT:
ENTEK ENGINEERING LLC
166 AMES STREET
HACKENSACK, NJ, 07601
TEL: 201 820 2801

ACOUSTICAL CONSULTANT:
CERAMI & ASSOCIATES
404 FIFTH AVENUE
NEW YORK, NY, 10018
TEL: 212 616 4179

LIGHTING CONSULTANT:
ONE LUX STUDIO
158 WEST 29TH STREET, 10TH FLOOR
NEW YORK, NY, 10001
TEL: 212 201 5790

SPECIFICATIONS CONSULTANT:
ROBERT SCHWARTZ & ASSOCIATES
589 EIGHTH AVENUE, 17TH FLOOR
NEW YORK, NY, 10018
TEL: 212 691 3248

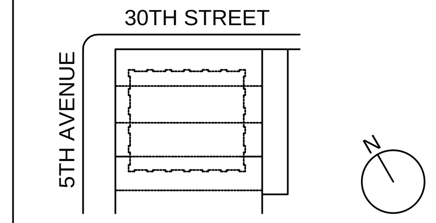
ENERGY MODELING CONSULTANT:
STEVEN WINTER ASSOCIATES, INC
307 SEVENTH AVENUE, SUITE 1701
NEW YORK, NY, 10001
TEL: 212 564 5800

INTERIOR DESIGN CONSULTANT:
JEFFREY BEERS INTERNATIONAL
156 5TH AVENUE, PENTHOUSE 2
NEW YORK, NY, 10010
TEL: 212 352 2020

DESIGN DEVELOPMENT
NYC DEPT. OF BUILDINGS SUBMITTAL

ARCHITECT'S SEAL

ISSUE NO.	ISSUE DATE	DESCRIPTION
	04/08/2016	DOB SUBMITTAL
	04/08/2016	SUPERSTRUCTURE EARLY BID
	03/25/2016	DOB FOUNDATION POST APPROVAL AMENDMENT
	01/27/2016	FOUNDATION BID ADD. #1
	01/18/2016	FOUNDATION BID
	12/23/2015	SCHEMATIC DESIGN
	04/15/2015	DOB SUBMITTAL
	02/28/2015	SCHEMATIC DESIGN



KEY PLAN AND NORTH SIGN

IF THIS DRAWING IS NOT 42" X 30" IT IS A REDUCED PRINT; REFER TO GRAPHIC SCALE

0 1 2

SCALE N.T.S

SHEET TITLE : GENERAL NOTES, LEGENDS & ABBREVIATIONS

SHEET NUMBER : FO-001.01

APPENDIX D

Wind Effects Study

The Boundary Layer Wind Tunnel Laboratory

A STUDY OF WIND EFFECTS FOR

281 Fifth Avenue
New York, NY

P. Case

General Information, Structural Loads – Interim Report (Properties of November 19, 2015)
BLWT-F062-IR3-2015- V2 / November 25, 2015

Western University, Boundary Layer Wind Tunnel Laboratory, Faculty of Engineering, BLWTL Building
1151 Richmond St., London, ON, Canada N6A 5B9 t. 519.661.3338, f. 519.661.3339 www.blwtl.uwo.ca

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SUMMARY AND MAIN FINDINGS

This report on the study of wind action on the proposed 281 Fifth Avenue building in New York City provides information on the overall wind loads suitable for use in the design of the structural system. These results are based on structural dynamic properties provided on November 19, 2015.

A force balance model test was carried out in the wind tunnel for the determination of overall structural wind loads and building accelerations.

Two configurations of the surroundings were tested. Configuration 1 included a full build out of the current surroundings. For Configuration 2 specific influential buildings were removed (reduced) from the surroundings.

Figure 1 provides elevation views of the as-tested model. Figure 4 shows close-up views of the 1:400 scale force balance model that includes the configurations (Fig. 4c) tested.

Winds in New York City are associated with two basic types of weather systems: hurricane and non-hurricane winds. For non-hurricane winds, a design probability distribution of upper-level (500 m) wind speed and direction had been previously developed for the area on the basis of full scale meteorological records from La Guardia International, JFK International and Newark International airports and a consideration of the New York building code. For hurricane winds, a simulation technique is used, involving thousands of simulated hurricanes matching the characteristics of actual recorded hurricanes that have been felt in New York.

Statistical predictions of extreme values of loads and responses were made for various return periods taking into consideration the combined effects of both hurricane and non-hurricane winds. For non-hurricane winds, an “up-crossing” method is used (Reference 1). For hurricane winds, a “storm passage” method is used, whereby the impact of each of the thousands of hurricanes is tracked at every step during its passage and the resulting loads and responses are determined.

The highlights and main findings of this study are as follows:

Wind Climate

- For strength considerations, the directional non-hurricane wind climate model, when combined with the hurricane climate, has been matched to an 80mph fastest mile wind speed, 98 mph 3-sec gust, at 33ft (10m) in open country terrain, consistent with the 2008 New York City Building Code. This is equivalent to an hourly-mean wind speed of about 112mph (50m/s) at 500m in standard open terrain.
- For the analysis of the wind tunnel data, the design wind speed at a height of 10 m is converted to an upper level wind speed. Predictions of mean-hourly wind speeds at the 500 m reference height for various return periods are shown in Figure 2.
- Directional characteristics of the wind events are indicated by the probability distribution of Appendix A and the relative importance factors, see Figure 3.

Overall Building Loads and Responses

- The overall structural loads and responses were determined using the force balance technique.
- Predictions were determined using total damping ratios 1.5% and 2% for both structural loads and acceleration responses. In addition to the base case (Case 1) building period set, four additional sets of periods were evaluated. These additional building periods (cases 2 through 5) were included at the request of the structural engineer to help understand the sensitivity of the building to changes in the building’s natural frequencies. For accelerations 3%, 4% and 5% damping ratios were also evaluated.

- The predicted peak accelerations for a 10-year return period are contained in Table 2a for Configuration 1 and the 1.5% and 2% damping ratios. Table 2b contains acceleration responses for higher damping ratios, which might be achieved through a supplementary damping system.
- For residential occupancy good building performance can be expected when predicted acceleration levels are below 18 milli-g. For the 281 Fifth Avenue building, it is anticipated that supplementary damping (total damping of about 4%) would be required to achieve the acceptable acceleration level. Note that the resultant accelerations in Table 2 are the worst that would be expected in the building since they are calculated at the maximum distance from the centre of coordinates at the top occupied floor. All accelerations decrease at lower elevations.
- Table 3a summarizes the predicted 50-year base moments for Configuration 1. Table 3b summarizes the predicted 50-year base moments for Configuration 2. Table 3c provides code estimated loads (ASCE 7 procedures). As the configuration 1 predictions are greater than 80% of the ASCE 7 loads, Configuration 2 results are provided for information.
- Effective floor-by-floor static force distributions, corresponding to the 50-year predicted base moments determined from the wind tunnel study, are given in Table 4a (1.5% damping ratio) and Table 4b (2% damping ratio). These are provided for the Configuration 1 results. The results shown are for the Base case (Case 1). The recommended load cases are given in Table 5.

Notes

- Predictions for an R-year return period (mean recurrence interval of R years) represent levels which are expected to occur *on average* once in R years. For reference, the risk of exceeding an R-year return period load in a design life of L years is $1-(1-1/R)^{L}$. Thus, for example, the risk of exceeding a 50 year load in a design lifetime of 50 years is about 64%, whereas the risk of exceeding a 1000 year load in a 50 year design life is about 5%.
- The predictions in this report are best estimates and **have not been factored** in any way. For instance, no load factors, such as those typically required by building codes, have been applied.

DETAILS OF THE STUDY

Project Name: 281 Fifth Avenue, New York

Project Location: On the northwest corner of Fifth Avenue and 30th Street intersection.

Project Description: The tower as analysed consists of 51 stories and is about 674' to the top. The tower has plan dimensions of about 69' x 79.5'. Figure 1 provides some elevation views of the 'as-tested' project. Figure 4 shows close-up views of the 1:400 scale pressure model.

Test Dates: Force Balance – November 2015

Report Scope and Format: The results presented in this report include the following components:

1. The full-scale wind climate in order to determine the strength and directionality of the wind;
2. experimental wind tunnel measurements to determine the aerodynamic data relevant to this project;
3. the calculation of the wind-induced dynamic loads and responses.

The combination of (1) to (3) provide statistical predictions of wind loads and/or responses for various return periods. These predictions are obtained by summing the contributions to the probability from all wind directions. The report is then organized as follows:

Section 1 – The wind climate for New York city
Section 2 – The modelling of the site and the wind
Section 3 – The determination of overall structural loads and responses

General Reference: Discussion and details of the general methodology used by the Alan G. Davenport Wind Engineering Group can be found in "Wind Tunnel Testing – A General Outline" (Reference 1).

1 THE WIND CLIMATE FOR NEW YORK

1.1 Introduction

- The statistical wind climate model for New York City comprises the combined effect of two complementary probability distributions of wind speed and direction as detailed below.
- The first represents the non-hurricane or extra-tropical winds and has been derived from available surface hourly wind speed and direction data recorded at La Guardia International, JFK International and Newark International airports. Readings associated with hurricanes have been excluded from these records. The methodology for its development is detailed in Reference 1.
- The second wind climate model represents the typhoon winds that affect the New York area. This model was obtained from a hurricane simulation study which employs an updated version of Applied Research Associates (ARA) HURSIM hurricane simulation code (References 2,3), which is the same model used to define the design wind speeds given in ASCE 7-10.
    - i. The wind field model used in the computer code has been extensively validated for surface level winds at both coastal and inland stations. The uncertainties associated with the prediction of hurricane wind speeds resulting from the use of the HURSIM model are discussed in Reference 4.
    - ii. Predictions of hurricane wind speed vs. return period are given for surface level winds and are based upon a simulation of 100,000 years of storms passing within 250 km of New York City. For storms within the 250 km limit, wind speeds and directions are computed every 10 minutes.
    - iii. Predicted wind speeds at the site have been derived using the conditional wind speed exceedence probabilities obtained by rank ordering the simulated maximum wind speeds resulting from the simulation of the 100,000 years of storms. An interpolation technique is then used to obtain wind speed exceedence probabilities.
    - iv. The storm passages approach is used to make statistical predictions of wind induced loads and responses during all hurricane wind events (Reference 1).

1.1 Results

- The wind climate analysis predicts an hourly-mean 50-year wind speed of about 107mph (48m/s) at 500m. This represents the wind speed determined from a direct analysis of the available wind records from La Guardia International, JFK International and Newark International airports. This represents the best-estimate or unadjusted wind climate, and is used in the evaluation of building responses related to serviceability.
- For the evaluation of overall structural loads the non-hurricane wind climate was adjusted so that the predicted 50-year fastest mile wind speed, when combined with the hurricane climate, matched an 80mph fastest mile, 98 mph 3-sec gust, wind speed at 33ft (10m) in open country terrain as per the 2008 New York City Building Code. This corresponds to an hourly-mean wind speed of about 112mph (50m/s) at 500m in standard open terrain. The adjusted non-hurricane climate is referred to as the ‘Code-adjusted non-hurricane’ wind climate. The combined non-hurricane and hurricane climate is referred to as the ‘Code-matched’ wind climate model.
- Extreme wind speeds for different return periods for the hurricane, non-hurricane and combined (Code-matched) wind climates at a reference height of 500m in open country terrain are given in Figure 2.
- Directional characteristics of the wind events are indicated by the probability distributions and the relative importance factors; see Figure 3.
- The annual design probability distribution of mean-hourly wind speed and wind direction at reference height (500m) is shown in Appendix A for the non-hurricane wind climate.

2 THE MODELLING OF THE SITE AND THE WIND

2.1 Overall Approach

- The basic tool used is the Laboratory's boundary layer wind tunnel. The tunnel is designed with a very long test section, which allows extended models of upwind terrain to be placed in front of the model of the building under test. The modelling is done in more detail close to the site. The wind flow then develops characteristics which are similar to the wind over the terrain approaching the actual site. This methodology has been highly developed (see References 5 and 6) and is detailed below.

2.2 Model Design

- Close-up views of the 1:400 scale force balance model are shown in Figure 4.
- Components:
    1. The force-balance model, built in detail from lightweight, high-density foam and mounted on a force balance at its base.
    2. A detailed proximity model of the surrounding city built in block outline from Styrofoam for a radius of approximately 1600'.
    3. Generic models of upstream terrain, see below.
- The building model and the proximity model are rotated to simulate different wind directions with the upstream terrain being changed as appropriate.
- The upstream terrain was modelled using generic roughness blocks and turbulence-generating spires to produce wind characteristics representative of those at the project site. Four different terrain models were used. These are shown in Figure 5 and the azimuth ranges over which they were used are shown in Figure 6.

2.3 Characteristics of the Modelled Wind

- Figure 7 presents vertical profiles of the mean speed and of the intensity of the longitudinal component of turbulence, measured just upstream of the centre of the turntable, for each upstream terrain exposure.
- The model profiles are good representations of the expected variation of full-scale wind speed and turbulence over the building height. The reference wind speed measured in the wind tunnel has been scaled such that the expected full-scale wind speeds at roof height are achieved.

3 THE DETERMINATION OF OVERALL STRUCTURAL LOADS AND RESPONSES

3.1 Overall Approach

- The overall structural loads and responses were obtained using the force-balance technique whereby a lightweight, stiff, geometrical representation of the tower was tested on an ultra-sensitive force balance.
- This technique allows the direct measurement of the time histories of wind forces of the building in the form of base shears and moments. Modal forces for the fundamental sway and torsional modes of vibration of the building are calculated using the measured base forces and the dynamic properties of the building.
- Statistics of the base forces and modal forces of the building were determined from the time histories. Estimates of the full-scale responses, including the resonant response of the tower, were subsequently made using random-vibration analysis methods.
- Predictions of wind loads and effects were determined by combining the aerodynamic data with the statistical wind climate model described in Section 1.
- Views of the model are shown in Figure 4.
- Two configurations of the surroundings were tested. Configuration 1 included a full build out of the current surroundings. For Configuration 2 and for the significant wind directions, specific influential buildings were removed (reduced) from the surroundings. Figure 4c shows views of the configurations tested.

3.2 Aerodynamic Data

- Mean and rms base bending moments and torsion were determined at 10° intervals for the full 360° azimuth range at the center of coordinates of the building at first floor level. The sign convention used is presented in Figure 8. These aerodynamic data are shown in coefficient form in Figure 9 and are tabulated in Appendix B.
- Spectra of the base bending moments and torsion were determined at 10^{o} intervals for the full 360^{o} azimuth range. Plots of these quantities are shown in Appendix C.

3.3 Statistical Predictions of Loads and Responses

3.3.1 General

- The dynamic properties of the tower were developed by WSP on November 19, 2015. The mode shapes are shown in Figure 10 and the mass distribution is given in Table 1. As specified by WSP, a structural damping ratio of 1.5% and 2% of critical, for all three fundamental modes of vibration, was used for the determination of the loads and accelerations. Additional damping ratios of 3%, 4% and 5% were evaluated for accelerations.
- In addition to the Base period set (Case 1), four additional sets of the building periods were evaluated. For Cases 2 through 5, only the building periods were changed while the mass, MMI and mode shapes remained unchanged from the Base case (Case 1) set of building properties. These additional cases were requested by the structural engineer to help understand the sensitivity of the building to changes in the building's natural frequencies.
- By combining the mean, rms and spectra of the modal forces and the dynamic properties of the building, accelerations, torsion velocities and moments have been calculated using the methodology outlined in Reference 1.
- The experimentally and analytically-obtained peak moment, acceleration and torsion velocity data were integrated with the wind climate model to provide predictions of moments and accelerations

for various return periods. The results are summarized in Tables 2 and 3 for accelerations and base overturning moments, respectively.

3.3.2 Accelerations

- Accelerations are calculated at a height corresponding to the top occupied floor of the building; in this case, the LV48, 592.5’ above ground. Torsional accelerations are expressed as linear accelerations at a distance of 42’ from the centre of coordinates of the building. The centroidal accelerations are the combination of the x and y accelerations, and the corner accelerations are the combination of the x, y and torsional accelerations.
- For residential occupancy good building performance can be expected when predicted acceleration levels are below 18 milli-g. For the 281 Fifth Avenue building, it is anticipated that supplementary damping (total damping of about 4%) would be required to achieve the acceptable acceleration level.
- Note that the resultant accelerations in Table 2 are the worst that would be expected in the building since they are calculated at the maximum distance from the centre of coordinates at the top occupied floor. All accelerations decrease at lower elevations. Furthermore, the torsion-induced acceleration reduces as the centre of the floor plate is approached at any floor.

3.3.3 Base Moments

- Table 3a summarizes the predicted 50-year base moments for Configuration 1. Table 3b summarizes the predicted 50-year base moments for Configuration 2.
- Base overturning moments were also determined following ASCE 7 procedures (Table 3c), using the Case 1 set of periods. Generally, ASCE procedures limit the base overturning moments determined from wind tunnel studies to 80% of that determined using ASCE 7 methodology, unless supplemental tests are carried out (i.e. Configuration 2) for the significant wind directions in which specific influential buildings were removed or reduced in size. In this case the limiting value of 80% may be reduced to 50%. As the configuration 1 predictions are greater than 80% of the ASCE 7 loads, Configuration 2 results are provided for information. The results for Configuration 2 tests do highlight the potential influence of nearby tall buildings. The structural engineer should confirm the magnitude of the base overturning moments calculated using ASCE 7.
- Note that based on its location and following the NYC Building Code guideline, an Exposure B has been used for the code calculations.

3.4 Effective Static-Force Distributions

- Representative effective static force distributions reflecting the combined static and dynamic loading of the building were evaluated for the x, y and torsional directions. The details of the procedure are included in Reference 1.
- The effective loads are provided as effective floor-by-floor loads. These are given in Table 4a for 50-year return period and 1.5% damping, and in Table 4b for 50-year return period and 2.0% damping. These loads are to be applied at the (report) centre of coordinates given in Figure 8 at every level. The effective loads correspond to the Case 1 set of periods and Configuration 1.
- It should be stressed that Configuration 2 studies did produce larger loads and responses than Configuration 1 results, thus highlighting the significant impact of specific nearby structures.
- For application of the effective static loads, companion-load cases have been derived based on considerations of overall load effects using the load data obtained in the current study. The recommended load cases are given in Table 5.
- It should be appreciated that these effective static-loading distributions are representative of the most likely severe wind loading conditions, and that the detailed loading may change somewhat for different wind directions, since both the details of the mean pressure distribution and the mix

between mean and dynamic responses will vary from angle to angle. These distributions will also change if the dynamic properties of the building change from those assumed, although their normalized shape varies slowly. These shapes can therefore be used in conjunction with base loads predicted for somewhat different building properties.

A SUMMARY OF THIS REPORT IS PRESENTED AT THE BEGINNING

REFERENCES

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2) Vickery, P.J. and D. Wadhera, “Statistical Models of Holland Pressure Profile Parameter and Radius To Maximum Winds of Hurricanes From Flight Level Pressure and H*Wind Data, J. Appl. Meteor., 47, 2008. 2497-2417

3) Vickery, P.J., D. Wadhera, M.D. Powell and Y. Chen, “A Hurricane Boundary Layer and Wind Field Model For Use in Engineering Applications”, J. Appl. Meteor., 48, 2009. 381–405.

4) Vickery, P.J.; D. Wadhera, L.A. Twisdale Jr. and F. M. Lavelle, “United States Hurricane Wind Speed Risk and Uncertainty”, J. Struct. Engrg.. 135, 2009. 301-320

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6) Surry, D. and Isyumov, N., "Model Studies of Wind Effects - A Perspective on the Problems of Experimental Technique and Instrumentation", Int. Congress on Instrumentation in Aerospace Simulation Facilities, 1975 Record, pp. 76-90.

7) Isyumov, N., “Motion Perception, Tolerance and Mitigation”, 5th World Congress of the Council on Tall Buildings and Urban Habitat, Amsterdam, The Netherlands, May 1995.

TABLES

TABLE 1 MASS AND MASS MOMENT OF INERTIA

MASS LOCATION				MASS	MMI
LEVEL	X(ft)	Y(ft)	HEIGHT (ft.)	(lb-sec2/ft)	(lb-ft-sec2)
T.O. TOWER	0.018	-0.049	674	32244	34294154
T.O. BH	-0.001	0.003	660.25	39644	41210428
LEV52-ROOF	0.02	-0.056	646.5	65132	67364926
LEV51-TANK	0.033	-0.072	630	74428	51173882
LEV50-MECH2	0.023	-0.026	619	83270	58939650
LEV49-MECH1	0.017	-0.03	608	60945	60134927
LEV48	-0.009	0.029	592.5	38795	39987446
LEV47	-0.009	0.029	577	38795	39987446
LEV46	-0.009	0.029	561.5	38795	39987446
LEV45	-0.009	0.029	546	38795	39987446
LEV44	-0.009	0.029	530.5	38795	39987446
LEV43	-0.009	0.029	515	38795	39987446
LEV42-DUPL	-0.009	0.029	499.5	38795	39987446
LEV41-DUPL	-0.004	0.014	484	36186	37676659
LEV40	0.001	-0.002	473	33764	35413188
LEV39	0.001	-0.002	462	33764	35413188
LEV38	0.001	-0.002	451	33764	35413188
LEV37	0.001	-0.002	440	33764	35413188
LEV36	0.001	-0.002	429	33764	35413188
LEV35	0.001	-0.002	418	33765	35262789
LEV34	0.001	-0.002	407	33765	35262789
LEV33	0.001	-0.002	396	33765	35262789
LEV32	0.001	-0.002	385	33765	35262789
LEV31	0.001	-0.002	374	33765	35262789
LEV30	0.001	-0.002	363	33765	35262789
LEV29	0.001	-0.002	352	33765	35262789
LEV28	0.001	-0.002	341	33765	35262789
LEV27	-0.002	0.425	330	55797	37866722
LEV26-MECH2	0.015	0.436	319	76243	56432838
LEV25-MECH1	0.014	0.243	308	56509	55374211
LEV24	-0.006	0.02	297	37148	37367849
LEV23	-0.006	0.02	286	37148	37367849
LEV22	-0.006	0.02	275	37148	37367849
LEV21	-0.006	0.02	264	37148	37367849
LEV20	-0.006	0.02	253	37148	37367849
LEV19	-0.006	0.02	242	37148	37367849

LEV18	-0.006	0.02	231	37147	37518247
LEV17	-0.006	0.02	220	37147	37518247
LEV16	-0.006	0.02	209	37147	37518247
LEV15	-0.006	0.02	198	37147	37518247
LEV14	-0.006	0.02	187	37147	37518247
LEV13	-0.006	0.02	176	37147	37518247
LEV12	-0.006	0.02	165	37147	37518247
LEV11	-0.006	0.02	154	37147	37518247
LEV10	-0.006	0.02	143	37147	37518247
LEV9	-0.006	0.02	132	37147	37518247
LEV8	-0.006	0.02	121	37147	37518247
LEV7	-0.006	0.02	110	37147	37518247
LEV6	-0.018	-0.89	99	74265	43066903
LEV5-MECH2	-0.001	-1.142	88	105031	70128787
LEV5-MECH1	0.003	3.778	77	70321	60506797
LEV4-AMEN2	-0.018	3.51	66	69892	62247105
LEV3-AMEN1	-0.027	3.301	44	81786	70558842
LEV2-COM	-0.027	3.301	22	81786	70469890
LEV1-LOBBY	0.015	3.542	0	78502	68687100

Notes:

1. Heights are measured from ground level.
2. The x and y coordinates of the masses are measured from the report centre shown in Figure 8.

TABLE 2a SUMMARY OF ACCELERATION RESPONSES – CONFIGURATION 1

VARIABLE	10-YEAR RETURN PERIOD, Damping, ζ = 1.5%				
	Case 1	**Case 2**	**Case 3**	**Case 4**	**Case 5**
X Acceleration (milli-g)	15.5	16.5	18	14	12
Y Acceleration (milli-g)	25.5	27	28.5	23	21
Torsional Acceleration (milli-g)	4	4.5	4.5	3.5	3.5
Centroidal Acceleration (milli-g)	27	29	30	24.5	22
Resultant Acceleration (milli-g)	27	29	30.5	25	22
Torsion Velocity (milli-rads/sec)	1.1	1.3	1.6	0.9	0.8

VARIABLE	10-YEAR RETURN PERIOD, Damping, ζ = 2%				
	Case 1	**Case 2**	**Case 3**	**Case 4**	**Case 5**
X Acceleration (milli-g)	13	14.5	15.5	12	10.5
Y Acceleration (milli-g)	22	23.5	24.5	20	18
Torsional Acceleration (milli-g)	3.5	3.5	4	3	3
Centroidal Acceleration (milli-g)	23.5	25	26	21.5	19
Resultant Acceleration (milli-g)	23.5	25	26.5	21.5	19
Torsion Velocity (milli-rads/sec)	1.0	1.1	1.3	0.8	0.7

Notes:

1. Accelerations and Torsion Velocity are calculated at 592.5’ above Ground, corresponding to the LV48.
2. Torsional acceleration is expressed as linear acceleration at a distance of 43.5’ from the centre of coordinates.
3. Centroidal accelerations are the combination of X and Y accelerations with an appropriate joint action factor.
4. Resultant accelerations are the combination of X, Y and T accelerations with an appropriate joint action factor.
5. Damping as specified.
6. Periods:

MODE	PERIOD (seconds)				
	Case 1	**Case 2**	**Case 3**	**Case 4**	**Case 5**
1	6.57	7.23	7.88	5.91	5.26
2	5.32	5.85	6.38	4.79	4.26
3	2.36	2.60	2.83	2.12	1.89

TABLE 2b SUMMARY OF ACCELERATION RESPONSES – CONFIGURATION 1 (SUPPLEMENTED DAMPING)

VARIABLE	10-YEAR RETURN PERIOD, Damping, ζ = 3%				
	Case 1	Case 2	Case 3	Case 4	Case 5
X Acceleration (milli-g)	11	12	12.5	10	8.5
Y Acceleration (milli-g)	18	19	20	16.5	14.5
Torsional Acceleration (milli-g)	3	3	3.5	2.5	2.5
Centroidal Acceleration (milli-g)	19	20.5	21.5	17.5	15.5
Resultant Acceleration (milli-g)	19	20.5	21.5	17.5	15.5
Torsion Velocity (milli-rads/sec)	0.8	0.9	1.1	0.6	0.6

VARIABLE	10-YEAR RETURN PERIOD, Damping, ζ = 4%				
	Case 1	Case 2	Case 3	Case 4	Case 5
X Acceleration (milli-g)	9.5	10	11	8.5	7.5
Y Acceleration (milli-g)	15.5	16.5	17.5	14	12.5
Torsional Acceleration (milli-g)	2.5	2.5	3	2.5	2
Centroidal Acceleration (milli-g)	16.5	17.5	18.5	15	13.5
Resultant Acceleration (milli-g)	16.5	18	18.5	15	13.5
Torsion Velocity (milli-rads/sec)	0.7	0.8	1	0.6	0.5

VARIABLE	10-YEAR RETURN PERIOD, Damping, ζ = 5%				
	Case 1	Case 2	Case 3	Case 4	Case 5
X Acceleration (milli-g)	8.5	9	10	7.5	6.5
Y Acceleration (milli-g)	14	15	15.5	12.5	11.5
Torsional Acceleration (milli-g)	2	2.5	2.5	2	2
Centroidal Acceleration (milli-g)	14.5	16	16.5	13.5	12
Resultant Acceleration (milli-g)	15	16	16.5	13.5	12
Torsion Velocity (milli-rads/sec)	0.6	0.7	0.9	0.5	0.4

Notes:

1. Accelerations and Torsion Velocity are calculated at 592.5’ above Ground, corresponding to the LV48.
2. Torsional acceleration is expressed as linear acceleration at a distance of 43.5’ from the centre of coordinates.
3. Centroidal accelerations are the combination of X and Y accelerations with an appropriate joint action factor.
4. Resultant accelerations are the combination of X, Y and T accelerations with an appropriate joint action factor.
5. Total damping as specified.
6. Periods:

MODE	PERIOD (seconds)				
	Case 1	Case 2	Case 3	Case 4	Case 5
1	6.57	7.23	7.88	5.91	5.26
2	5.32	5.85	6.38	4.79	4.26
3	2.36	2.60	2.83	2.12	1.89

TABLE 3a SUMMARY OF LOADS – CONFIGURATION 1

VARIABLE	50-YEAR RETURN PERIOD, Damping, ζ = 1.5%				
	Case 1	Case 2	Case 3	Case 4	Case 5
X Moment (lb-ft)	6.83E+08	7.17E+08	7.40E+08	6.46E+08	5.99E+08
Y Moment (lb-ft)	8.58E+08	8.77E+08	8.86E+08	8.26E+08	7.84E+08
Torsion (lb-ft)	9.38E+06	9.45E+06	9.79E+06	9.17E+06	8.96E+06

VARIABLE	50-YEAR RETURN PERIOD, Damping, ζ = 2.0%				
	Case 1	Case 2	Case 3	Case 4	Case 5
X Moment (lb-ft)	6.16E+08	6.43E+08	6.61E+08	5.86E+08	5.46E+08
Y Moment (lb-ft)	7.76E+08	7.93E+08	7.98E+08	7.51E+08	7.18E+08
Torsion (lb-ft)	8.96E+06	9.03E+06	9.24E+06	8.75E+06	8.54E+06

Notes:

1. Moments are calculated about the Ground level.
2. Damping as specified.
3. Periods:

MODE	PERIOD (seconds)				
	Case 1	Case 2	Case 3	Case 4	Case 5
1	6.57	7.23	7.88	5.91	5.26
2	5.32	5.85	6.38	4.79	4.26
3	2.36	2.60	2.83	2.12	1.89

TABLE 3b SUMMARY OF LOADS – CONFIGURATION 2 (REDUCED SHELTERING)

VARIABLE	50-YEAR RETURN PERIOD, Damping, ζ = 1.5%				
	Case 1	**Case 2**	**Case 3**	**Case 4**	**Case 5**
X Moment (lb-ft)	7.28E+08	7.77E+08	7.84E+08	6.67E+08	6.12E+08
Y Moment (lb-ft)	9.73E+08	9.24E+08	9.14E+08	1.00E+09	9.59E+08
Torsion (lb-ft)	1.02E+07	1.06E+07	1.11E+07	9.78E+06	9.39E+06

VARIABLE	50-YEAR RETURN PERIOD, Damping, ζ = 2.0%				
	Case 1	**Case 2**	**Case 3**	**Case 4**	**Case 5**
X Moment (lb-ft)	6.42E+08	6.77E+08	6.96E+08	6.04E+08	5.61E+08
Y Moment (lb-ft)	8.47E+08	8.12E+08	8.19E+08	8.68E+08	8.33E+08
Torsion (lb-ft)	9.47E+06	9.81E+06	1.02E+07	9.13E+06	8.81E+06

Notes:

1. Moments are calculated about the Ground level.
2. Damping as specified.
3. Periods:

MODE	PERIOD (seconds)				
	Case 1	**Case 2**	**Case 3**	**Case 4**	**Case 5**
1	6.57	7.23	7.88	5.91	5.26
2	5.32	5.85	6.38	4.79	4.26
3	2.36	2.60	2.83	2.12	1.89

TABLE 3c ASCE 7-10 CALCULATED 50-YEAR LOADS (EXPOSURE B)

VARIABLE	50-YEAR RETURN PERIOD, Damping, ζ = 2.0%
	Case 1
X Moment (lb-ft)	6.58E+08
Y Moment (lb-ft)	8.23E+08

Notes:

1. Moments are calculated about the Ground level.
2. Damping as specified.
3. Periods:

MODE	PERIOD (seconds)
	Case 1
1	6.57
2	5.32

TABLE 4a 50-YEAR EQUIVALENT STATIC WIND LOADS, DAMPING, ζ= 1.5% (CASE 1) – CONFIGURATION 1

		281 Fifth Avenue, New York		
Floor	**Floor Height (ft)**	**X Direction (lb)**	**Y Direction (lb)**	**Torsion Direction (lb-ft)**
T.O. TOWER	674	42100	52100	221058
T.O. BH	660.2	56800	69600	315060
LEV52-ROOF	646.5	82800	102000	445216
LEV51-TANK	630	88200	110000	355346
LEV50-MECH2	619	92100	116000	364643
LEV49-MECH1	608	71800	89600	385303
LEV48	592.5	51800	63900	310928
LEV47	577	50200	62200	305763
LEV46	561.5	48600	60300	301631
LEV45	546	47000	58400	296466
LEV44	530.5	45300	56400	290268
LEV43	515	43700	54400	285103
LEV42-DUPL	499.5	41900	52400	277872
LEV41-DUPL	484	36400	45700	245850
LEV40	473	31700	40100	215893
LEV39	462	30700	38800	211761
LEV38	451	29700	37500	206597
LEV37	440	28600	36200	200399
LEV36	429	27600	34900	195234
LEV35	418	26600	33600	188003
LEV34	407	25500	32300	183871
LEV33	396	24500	31100	176640
LEV32	385	23600	29800	172508
LEV31	374	22600	28600	166310
LEV30	363	21600	27400	161145
LEV29	352	20700	26200	154947
LEV28	341	19800	25100	149783
LEV27	330	26800	34700	151848
LEV26-MECH2	319	32500	42900	196267
LEV25-MECH1	308	24700	32600	189036
LEV24	297	17600	23000	142552
LEV23	286	16700	21900	137387
LEV22	275	15900	20800	132222
LEV21	264	15000	19700	124991
LEV20	253	14100	18600	120859
LEV19	242	13300	17500	115694
LEV18	231	12400	16400	109496
LEV17	220	11600	15300	104331
LEV16	209	10800	14300	97307
LEV15	198	10000	13300	92245
LEV14	187	9270	12300	87287
LEV13	176	8540	11300	80573
LEV12	165	7840	10400	75511
LEV11	154	7170	9480	70553

LEV10	143	6540	8630	63838
LEV9	132	5940	7830	58880
LEV8	121	5390	7080	53922
LEV7	110	4870	6400	48963
LEV6	99	5770	8130	48653
LEV5-MECH2	88	6120	9030	58054
LEV5-MECH1	77	4490	6400	51546
LEV4-AMEN2	66	5170	7050	59087
LEV3-AMEN1	44	5160	6840	59706
LEV2-COM	22	3860	4920	46484
LEV1-LOBBY	0	1430	1770	19110
Total Base Shear (lb)		1.44E+06	1.82E+06	
Total Base Moment (lb-ft)		6.83E+08	8.58E+08	9.38E+06

1. Heights are measured from Ground level.
2. Loads are to be applied at the centre of coordinates shown in Figure 8.
3. Loading cases are given in Table 5.

TABLE 4b 50-YEAR EQUIVALENT STATIC WIND LOADS, DAMPING, ζ= 2.0% (CASE 1) – CONFIGURATION 1

		281 Fifth Avenue, New York		
Floor	**Floor Height (ft)**	**X Direction (lb)**	**Y Direction (lb)**	**Torsion Direction (lb-ft)**
T.O. TOWER	674	37100	46100	205716
T.O. BH	660	51200	63200	303272
LEV52-ROOF	647	73200	91100	415674
LEV51-TANK	630	77200	96700	336144
LEV50-MECH2	619	79700	101000	336144
LEV49-MECH1	608	63300	79500	358413
LEV48	593	47200	58600	300091
LEV47	577	45800	56900	294789
LEV46	562	44400	55300	290548
LEV45	546	42900	53600	285246
LEV44	531	41500	51800	278883
LEV43	515	40000	50000	273581
LEV42-DUPL	500	38400	48200	267219
LEV41-DUPL	484	33200	41900	235407
LEV40	473	28800	36500	204656
LEV39	462	27900	35300	200414
LEV38	451	27000	34200	196173
LEV37	440	26100	33000	189810
LEV36	429	25200	31900	185569
LEV35	418	24200	30800	179206
LEV34	407	23400	29600	174965
LEV33	396	22500	28500	168602
LEV32	385	21600	27400	164361
LEV31	374	20800	26300	157999
LEV30	363	19900	25200	153757
LEV29	352	19100	24200	148455
LEV28	341	18300	23300	143153
LEV27	330	24100	31200	144213
LEV26-MECH2	319	28800	38100	182388
LEV25-MECH1	308	22200	29300	176025
LEV24	297	16300	21200	135730
LEV23	286	15500	20300	130428
LEV22	275	14700	19300	126187
LEV21	264	14000	18300	119824
LEV20	253	13200	17300	115583
LEV19	242	12400	16300	110281
LEV18	231	11700	15300	104873
LEV17	220	11000	14400	100313
LEV16	209	10200	13500	94163
LEV15	198	9540	12500	89497
LEV14	187	8870	11700	84831
LEV13	176	8210	10800	78681
LEV12	165	7580	9950	74121
LEV11	154	6980	9140	69456

LEV10	143	6400	8370	63411
LEV9	132	5860	7640	58746
LEV8	121	5350	6960	54186
LEV7	110	4880	6330	49626
LEV6	99	5570	7720	48990
LEV5-MECH2	88	5820	8420	56519
LEV5-MECH1	77	4410	6150	50687
LEV4-AMEN2	66	5240	7000	59912
LEV3-AMEN1	44	5360	6960	62351
LEV2-COM	22	4090	5120	49838
LEV1-LOBBY	0	1530	1860	20890
Total Base Shear (lb)		1.31E+06	1.66E+06	
Total Base Moment (lb-ft)		6.16E+08	7.76E+08	8.96E+06

1. Heights are measured from Ground level.
2. Loads are to be applied at the centre of coordinates shown in Figure 8.
3. Loading cases are given in Table 5.

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281 Fifth Avenue Peer Review
Structural Calculations

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May 2016

281 Fifth Avenue Peer Review Structural Calculations

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1. Design Criteria

1.1 Design Criteria

Design criteria on drawing FO-001.01 was checked following New York City Building Code (NYCBC 2014). The design loads and other information pertinent to the structural design required by Sections 1603.1.1 through 1603.1.9 were indicated on drawing FO-001.01 except roof snow load and spectral response coefficients S_{DS} and S_{D1}. We included these loads in our independent calculation. All the drawings and documents that we reviewed are listed in Appendix E.

2. Global Model

2.1 Global Model

A global building model using the software ETABS was provided by WSP. The model was reviewed and updated as necessary to be consistent with the submitted structural drawings and code requirements.

The model was utilized to generate demand for columns, walls, link beams and foundation checks.

(With slabs) (Without slabs)

Figure 2.1 Global Model

2.2 ETABS Model Check

We checked the model loading following the design criteria. We noticed the General Notes in Drawing FO-001 listed seismic coefficients are Ss=0.281 and S1=0.073 which is consistent with the NYC BC 2014, while the ETABS model defines Ss=0.365 and S1=0.071. We updated the seismic coefficients in the model and discovered they do not control the design. The wind loads in the model are consistent with the wind tunnel test results.

We checked the story shears, overturning moments, and building drift under wind tunnel loads. The shear force at the base of the building is 1,860 Kips and the overturning moment is 832,260 Kip-ft. Figure 2.2 compares the building output under wind and earthquake. It shows the building is governed by wind. The largest displacement at the top is 7.6 in (drift=1/1010) in X direction and 14.5 in (drift=1/530) in Y direction.

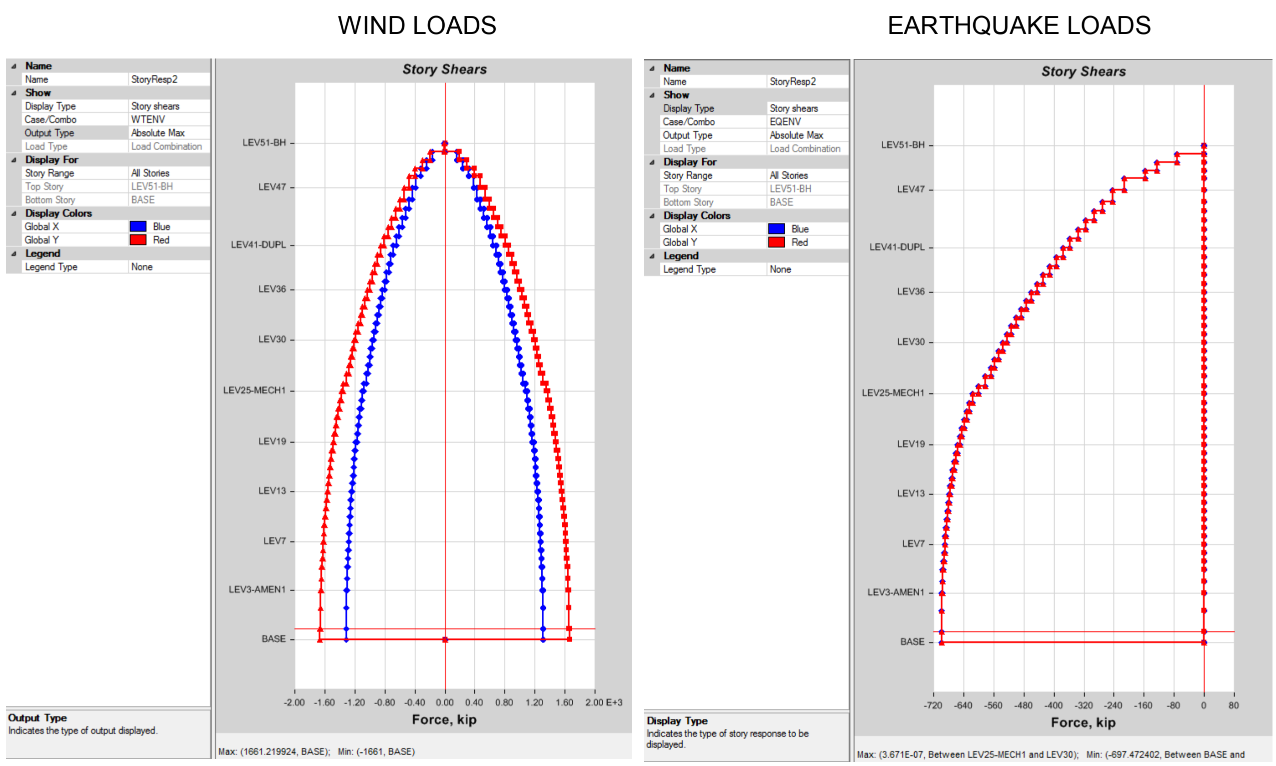

Figure 2.2 Comparison of Global Model Shears under Wind and Earthquake Loads

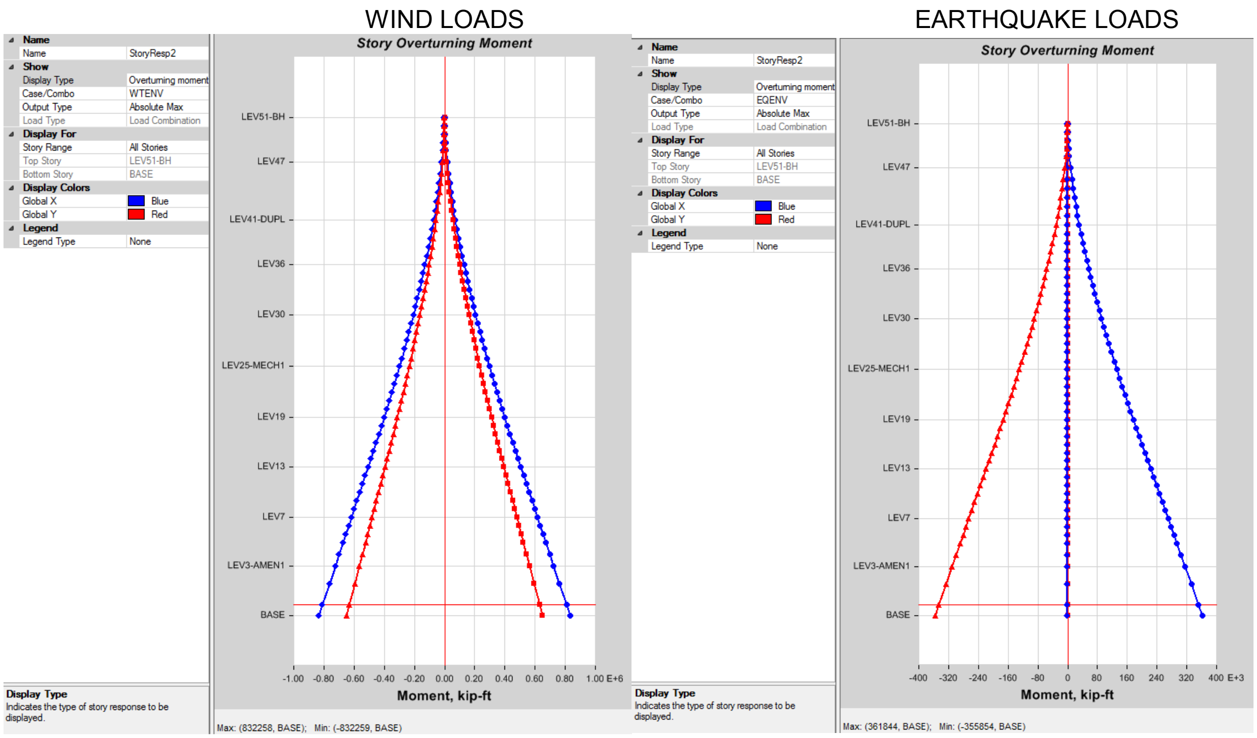

Figure 2.3 Comparison of Global Model Moments under Wind and Earthquake Loads

3. Typical Floor Check

3.1 Punching Shear Check

Typical residential floors 7th -24th (S-070.00) were selected to spot-check punching shear under gravity load combinations alone and under gravity plus wind load combinations. A SAFE model of L20 was built to make the first check for gravity alone. The boundary conditions of the slab beams in the ETABS model were considered as fix-fix to make the check for gravity plus wind.

We reviewed the demand and capacities at Columns 16 and 18. We found some differences in the punching shear DCR depending on the load combinations considered. Table 3.1 lists the DCRs under different load combinations. The DCRs under wind load combinations are somewhat larger than 1.0 without considering potential architectural openings in the future. See Appendix A for a representative calculation of punching shear check.

We questioned EOR about the interconnection of the typical floor slab and perimeter column. The lack of full engagement of the columns into the slab is unusual and the calculations indicate that the resulting connections of the slabs to the columns need to be considered with care.

We understand that this arrangement has been a topic of considerable debate and that EOR plans to use stud rails, a special slab reinforcement, to strengthen many of these connections. Further, we understand that EOR is working with the Architect so that slab openings be avoided, or at least significantly minimized, in the vicinity of these columns.

Based on our review, we believe that it is prudent for WSP to take both of these measures to provide the necessary strength to these typical slab/column connections.

Table 3.1 Punching Shear DCRs

Column No	DCR for Punching Shear		
	1.2D+1.6L	1.2D+1.0L+1.6W	
18	0.88	1.03 (per drawings)	0.38 (if stud rails provided)
16	0.89	1.01 (per drawings)	0.37 (if stud rails provided)

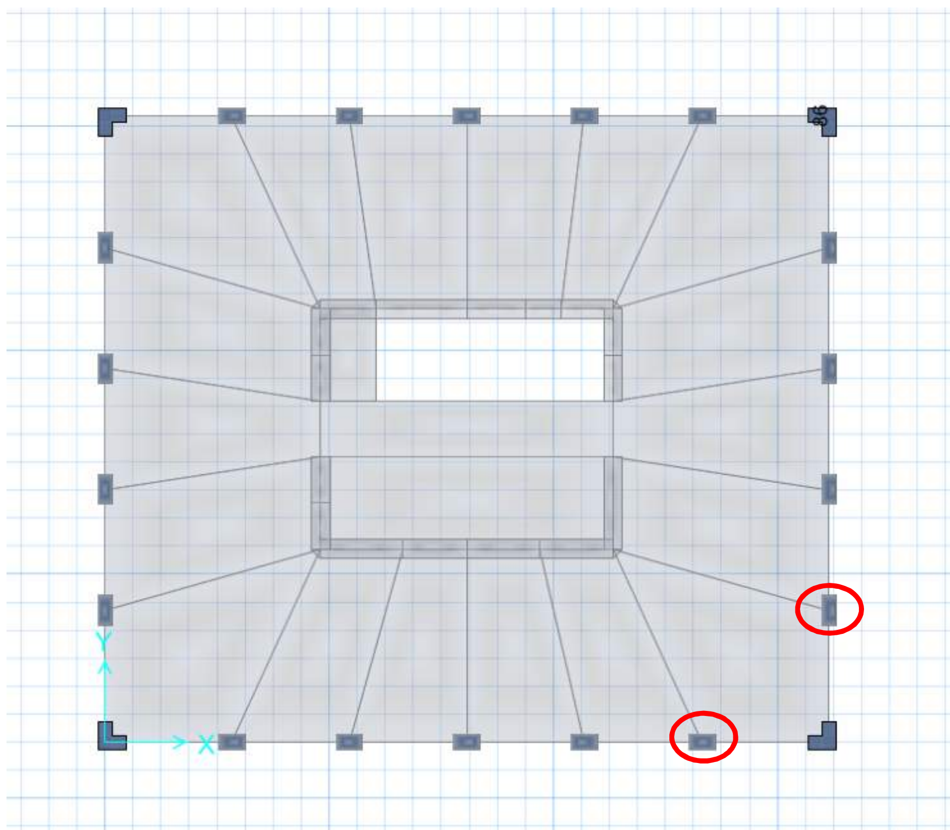

Figure 3.1 L20 SAFE Model (Columns 16 and 18 higlighted)

3.2 Slab Reinforcement Check

We utilized the SAFE model of Level 20 and checked the floor for resisting gravity and the design was found to be adequate. We then extracted lateral loads from the ETABS model by defining the boundary conditions of the Edge beams (defined as 30”x8”) and the Slab beams (defined as 60”x8”) as fix-fix. We superimposed the gravity and lateral loads and again found that the slab thickness and reinforcing to be adequate as shown.

Table 3.2 shows the comparison of column strip reinforcement as an example. 5-#5@12 (from plan) plus 2#5 at edge (from typical details on S-961.00) were provided for most edge column strips. According to ACI 318-11 13.5.3.2, all reinforcement resisting part of the unbalanced moment to be transferred to the column by flexure should be placed between lines that are one and one-half the slab thickness. In this case, 3-#5 was placed within the 1.5h zone, which is 12” for an 8” slab. This is greater than the 2-#5 we calculated was needed to transfer this moment (refer to Table 3.3). The design is therefore adequate as shown.

Table 3.2 Comparison of Column Strip Reinforcement

Location	Total reinforcement provided	Effective for slab to column moment transfer	Notes
Edge column strip reinforcement	5-#5@12 +2 #5 Bars at Edge (S-961.00)	2#5 within 12”	ACI 318-11 13.5.3.2

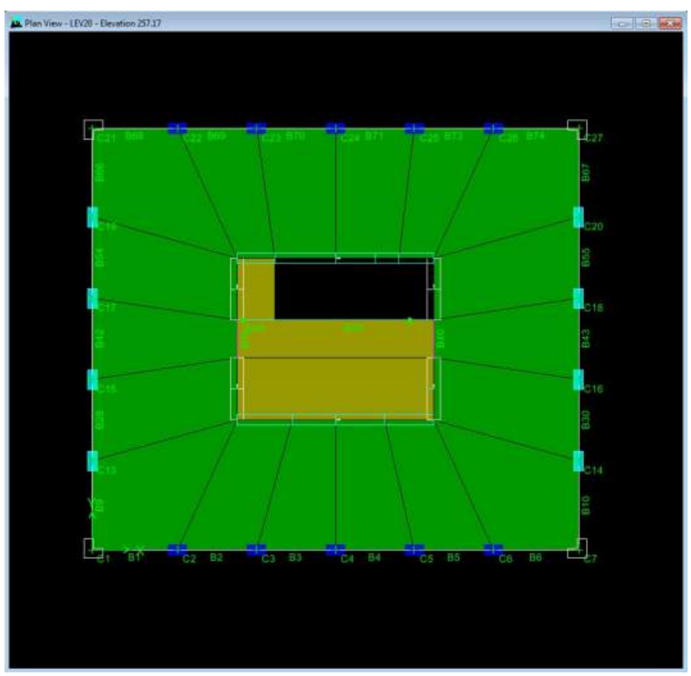

Figure 3.2 L20 ETABS Edge Column Strip label

Table 3.3 Calculation of Column Strip Reinforcement

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7th-24th Floor Edge Column Strip Reinforcement

	Beam	B10	B30	B43	B55	B67		B1	B2	B3	B4	B5	B6
Loads	Mu (kips-ft)	29	13	14	14	16		12	13	11	10	16	11
	Vu (kips)	16	10	10	10	11		9	9	8	8	10	10
Safety Factors	phi_m	0.90	0.90	0.90	0.90	0.90		0.90	0.90	0.90	0.90	0.90	0.90
	phi_v	0.75	0.75	0.75	0.75	0.75		0.75	0.75	0.75	0.75	0.75	0.75
Material Properties	fc' (psi)	7,200	7,200	7,200	7,200	7,200		7,200	7,200	7,200	7,200	7,200	7,200
	Beta1	0.690	0.690	0.690	0.690	0.690		0.690	0.690	0.690	0.690	0.690	0.690
	fy (psi)	60,000	60,000	60,000	60,000	60,000		60,000	60,000	60,000	60,000	60,000	60,000
Geometric Properties	b (in)	30.0	30.0	30.0	30.0	30.0		30.0	30.0	30.0	30.0	30.0	30.0
	h (in)	8.0	8.0	8.0	8.0	8.0		8.0	8.0	8.0	8.0	8.0	8.0
	Clear cover (in)	1.50	1.50	1.50	1.50	1.50		1.50	1.50	1.50	1.50	1.50	1.50
	Total cover to center of bar (in)	2.3	2.3	2.3	2.3	2.3		2.3	2.3	2.3	2.3	2.3	2.3
	d (in)	5.7	5.7	5.7	5.7	5.7		5.7	5.7	5.7	5.7	5.7	5.7
Flexure Design	A	8823.53	8823.53	8823.53	8823.53	8823.53		8823.53	8823.53	8823.53	8823.53	8823.53	8823.53
	B	-307125.00	-307125.00	-307125.00	-307125.00	-307125.00		-307125.00	-307125.00	-307125.00	-307125.00	-307125.00	-307125.00
	C	346783	161460	165219	171451	192179		142101	160016	130215	124001	190355	136372
	As_min (in^2)	0.00	0.00	0.00	0.00	0.00		0.00	0.00	0.00	0.00	0.00	0.00
	As_required (in^2)	1.17	0.53	0.55	0.57	0.64		0.47	0.53	0.43	0.41	0.63	0.45
	ρ_required	0.0068	0.0031	0.0032	0.0033	0.0037		0.0027	0.0031	0.0025	0.0024	0.0037	0.0026
	ρ_max	0.0312	0.0312	0.0312	0.0312	0.0312		0.0312	0.0312	0.0312	0.0312	0.0312	0.0312
	Bar size (1/8 in)	5	5	5	5	5		5	5	5	5	5	5
	Bar diameter	0.625	0.625	0.625	0.625	0.625		0.625	0.625	0.625	0.625	0.625	0.625
	Area (in^2)	0.31	0.31	0.31	0.31	0.31		0.31	0.31	0.31	0.31	0.31	0.31
	Required # bars	4	2	2	2	3		2	2	2	2	3	2
	No. of Layers	1	1	1	1	1		1	1	1	1	1	1
	Bars per layer	4	2	2	2	3		2	2	2	2	3	2
	Clear Spacing (in)	7.83	24.75	24.75	24.75	12.06		24.75	24.75	24.75	24.75	12.06	24.75
	ρ_provided	0.0072	0.0036	0.0036	0.0036	0.0054		0.0036	0.0036	0.0036	0.0036	0.0054	0.0036
Shear Design	Stirrup size (1/8 in)	4	4	4	4	4		4	4	4	4	4	4
	Bar diameter	0.500	0.500	0.500	0.500	0.500		0.500	0.500	0.500	0.500	0.500	0.500
	Bar area (in^2)	0.20	0.20	0.20	0.20	0.20		0.20	0.20	0.20	0.20	0.20	0.20
	ϕVc (kips)	21.7	21.7	21.7	21.7	21.7		21.7	21.7	21.7	21.7	21.7	21.7
	ϕVs,required (kips)	0.0	0.0	0.0	0.0	0.0		0.0	0.0	0.0	0.0	0.0	0.0
	Required Av/s (in)	0.025	0.025	0.025	0.025	0.025		0.025	0.025	0.025	0.025	0.025	0.025
	# Legs of Stirrup	0	0	0	0	0		0	0	0	0	0	0
	s_max (in)	2.8	2.8	2.8	2.8	2.8		2.8	2.8	2.8	2.8	2.8	2.8
	s_required (in)	0.0	0.0	0.0	0.0	0.0		0.0	0.0	0.0	0.0	0.0	0.0
	Provided Spacing (in)	12.0	10.0	10.0	10.0	10.0		12.0	10.0	10.0	10.0	10.0	10.0
Capacity Check	a (in)	0.40	0.20	0.20	0.20	0.30		0.20	0.20	0.20	0.20	0.30	0.20
	ϕMn (kips-ft)	30.3	15.4	15.4	15.4	22.9		15.4	15.4	15.4	15.4	22.9	15.4
	ϕVn (kips)	21.7	21.7	21.7	21.7	21.7		21.7	21.7	21.7	21.7	21.7	21.7
	Flexure Design OK?	OK	OK	OK	OK	OK		OK	OK	OK	OK	OK	OK
	Shear Design OK?	OK	OK	OK	OK	OK		OK	OK	OK	OK	OK	OK
Summary	M	4-5	2-5	2-5	2-5	3-5		2-5	2-5	2-5	2-5	3-5	2-5
	V	0-4@12	0-4@10	0-4@10	0-4@10	0-4@10		0-4@12	0-4@10	0-4@10	0-4@10	0-4@10	0-4@10
	D/C_M	0.95	0.87	0.89	0.93	0.70		0.77	0.86	0.70	0.67	0.69	0.74
	D/C_V	0.75	0.45	0.44	0.47	0.52		0.43	0.42	0.38	0.37	0.47	0.45

4. Column Capacity Check

4.1 Column Tension & Foundation Rock Anchor

Refer to Column Schedule S-950.01 and foundation plan F0-100.01

Locations of anchors under columns do not appear consistent with the locations of column tension splices at foundation level. See Table 4.1 below. Based on this check, we believe anchors should be considered under columns 7, 8, 9, 10 and 11.

Table 4.1 Comparison of Column tension splices and column with anchor below

Column No	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
Columns with tension splice at Foundation level			Y	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y			
Columns with Anchor below		Y		Y	Y	Y						Y	Y	Y		Y

We also reviewed the total uplift on the foundation from the wind applied on the north face of the building. This check also accounts for the dead load moments resulting from the column transfers on the South side of the tower at Level 4. The total uplift in this specific case was adequately counterbalanced by the total capacity of rock anchors provided.

We note that columns 7 and 11 share symmetry for the structural load path, however their tension splice locations in the columns schedule (between the 1st and 25th floors) are not consistent. Based on our observation, the locations of tension splices for Columns 7 and 11 should be similar.

See Table 4.2 below for a summary of factored column tension forces (the effects of construction sequence are considered when summarizing column forces). In this case, we find differences between column tensions in our model and the locations shown in the drawings for a few additional columns.

Based on the above, we recommend EOR to review tension splice locations for all columns and the anchor locations for columns 7 through 11.

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Table 4.2 Column Tension Forces With Constructural Sequence

COLUMN SCHEDULE	C1	C2	C3	C4	C5	C6	C7	C8	C9	C10	C11
ETABS LABLE	C1	C13	C15	C17	C19	C21	C22	C23	C24	C25	C26
LEV50-ROOF	-70	-45	-9	-20	73	79	30	18	12	15	33
LEV49-TANK	139	18	33	9	133	275	126	99	62	88	137
LEV48-MECH2	363	57	40	38	94	261	114	90	53	79	125
LEV48-MECH1	761	24	46	100	44	243	31	58	21	47	43
LEV47	722	8	12	67	26	209	14	32	0	21	27
LEV46	694	0	-18	37	17	172	15	1	-20	-11	30
LEV45	677	0	-31	13	4	143	3	-21	-38	-33	19
LEV44	654	-25	-53	-17	0	115	0	-43	-55	-55	9
LEV43	633	-50	-68	-29	0	98	0	-62	-71	-77	0
LEV42	613	-74	-90	-51	-23	81	-18	-77	-88	-98	0
LEV41-DUPL	594	-99	-112	-73	-45	65	-36	-92	-104	-117	0
LEV40-DUPL	575	-123	-134	-95	-66	48	-53	-107	-120	-131	-20
LEV39	557	-148	-156	-116	-88	32	-70	-121	-135	-145	-38
LEV38	540	-173	-178	-138	-110	16	-86	-136	-151	-159	-56
LEV37	525	-197	-198	-159	-130	2	-102	-150	-166	-173	-73
LEV36	510	-222	-219	-179	-151	-12	-118	-164	-180	-186	-90
LEV35	496	-247	-241	-200	-172	-26	-134	-177	-194	-200	-107
LEV34	482	-272	-262	-221	-193	-40	-150	-192	-209	-213	-124
LEV33	469	-298	-283	-242	-214	-54	-166	-206	-224	-228	-141
LEV32	457	-325	-304	-263	-235	-67	-183	-221	-239	-242	-158
LEV31	444	-352	-326	-284	-257	-81	-200	-235	-254	-257	-176
LEV30	432	-379	-348	-305	-279	-95	-217	-251	-270	-272	-194
LEV29	420	-407	-369	-327	-302	-109	-235	-267	-286	-288	-212
LEV28	409	-436	-392	-349	-324	-124	-254	-283	-302	-305	-231
LEV27	397	-464	-414	-371	-347	-137	-273	-300	-319	-322	-250
LEV26	394	-10	-39	-25	-66	191	28	-11	16	-15	44
LEV25-MECH2	381	239	56	47	65	272	196	166	165	196	191
LEV25-MECH1	378	692	413	258	379	1093	822	751	633	761	832
LEV24	366	663	390	236	356	1078	803	733	615	742	812
LEV23	354	634	367	213	332	1063	784	715	597	724	793
LEV22	342	605	345	191	309	1047	765	697	580	707	774
LEV21	330	577	322	169	286	1033	747	680	564	689	756
LEV20	319	550	300	147	263	1018	729	664	547	673	738
LEV19	307	523	278	125	240	1003	712	647	531	656	721
LEV18	294	497	255	103	217	989	694	631	515	640	703
LEV17	282	471	233	81	194	975	677	615	499	624	686
LEV16	270	445	211	59	171	960	660	599	483	608	668
LEV15	257	419	189	37	148	946	642	583	467	592	651
LEV14	244	394	167	15	124	931	625	567	451	575	634
LEV13	231	369	144	-7	100	917	607	551	435	559	616
LEV12	217	344	122	-29	76	902	590	535	418	543	598
LEV11	203	319	99	-52	52	887	572	519	401	526	580
LEV10	189	294	76	-74	28	872	554	502	384	509	562
LEV9	174	269	53	-97	2	857	535	485	367	492	543
LEV8	158	244	30	-120	-23	841	516	467	348	474	524
LEV7	142	213	1	-144	-49	824	496	448	329	456	504
LEV6	0	220	15	32	258	807	469	420	301	428	478
LEV5-MECH2	0	467	69	154	569	793	456	407	288	414	465
LEV5-MECH1	0	801	187	424	1001	767	414	362	242	370	422
LEV4-AMEN2	0	346	149	398	975	748	386	324	204	331	382
LEV3-AMEN1	0	253	63	315	874	698	322	264	142	272	329
LEV2-COM	0	424	-3	248	803	635	255	200	77	205	268
LEV1-LOBBY	0	505	-50	190	744	593	196	147	22	150	210

LERA

Table 4.2 Column Tension Forces With Constructural Sequence

COLUMN SCHEDULE	C12	C13	C14	C15	C16	C17	C18	C19	C20	C21	C22
ETABS LABLE	C27	C20	C18	C16	C14	C7	C6	C5	C4	C3	C2
LEV50-ROOF	74	73	-18	-6	-46	-71	-75	-69	-69	-69	-75
LEV49-TANK	259	135	16	36	34	117	-1	24	34	30	-8
LEV48-MECH2	245	106	45	41	69	317	53	119	96	138	66
LEV48-MECH1	226	62	107	48	75	688	145	358	299	417	21
LEV47	192	45	73	13	64	653	125	331	276	385	9
LEV46	152	36	43	-18	51	628	107	315	259	365	0
LEV45	123	24	13	-49	40	604	90	299	243	345	0
LEV44	94	13	-17	-79	29	580	73	284	227	323	-4
LEV43	65	2	-47	-110	19	557	56	270	212	302	-8
LEV42	37	0	-76	-140	9	535	39	257	197	282	-11
LEV41-DUPL	20	0	-88	-166	0	514	22	244	183	262	-15
LEV40-DUPL	3	-23	-110	-197	0	492	0	225	169	244	-19
LEV39	-13	-46	-127	-209	0	475	0	210	155	226	-24
LEV38	-28	-68	-149	-229	-29	459	-7	194	142	209	-29
LEV37	-42	-89	-169	-248	-57	446	-13	179	130	193	-33
LEV36	-56	-110	-189	-267	-84	434	-18	165	118	178	-38
LEV35	-70	-131	-210	-287	-111	422	-24	151	106	163	-43
LEV34	-83	-152	-231	-306	-139	411	-30	137	94	149	-47
LEV33	-96	-174	-251	-326	-167	400	-36	124	82	134	-53
LEV32	-109	-195	-272	-346	-196	389	-43	110	70	120	-58
LEV31	-123	-217	-293	-367	-224	378	-50	97	59	106	-65
LEV30	-137	-239	-315	-387	-252	368	-58	84	47	92	-72
LEV29	-151	-261	-336	-408	-280	358	-66	70	34	77	-79
LEV28	-165	-284	-358	-430	-307	347	-75	56	22	63	-88
LEV27	-179	-307	-380	-451	-334	337	-85	42	8	48	-97
LEV26	123	-42	-32	-19	77	337	-101	6	-25	11	-111
LEV25-MECH2	298	80	115	138	421	324	-112	-5	-36	0	-123
LEV25-MECH1	1094	436	271	411	693	325	-130	-40	-68	-35	-139
LEV24	1079	413	248	389	662	314	-141	-55	-82	-51	-150
LEV23	1063	390	226	366	632	302	-153	-70	-96	-66	-161
LEV22	1048	367	203	344	603	291	-165	-84	-110	-81	-173
LEV21	1033	344	181	321	575	280	-177	-99	-124	-96	-185
LEV20	1018	321	159	299	548	269	-190	-113	-137	-111	-197
LEV19	1004	298	137	277	521	257	-203	-128	-151	-126	-210
LEV18	989	276	115	255	494	246	-216	-142	-165	-141	-223
LEV17	975	253	93	233	468	234	-230	-157	-179	-156	-237
LEV16	960	230	72	211	442	222	-244	-172	-193	-171	-251
LEV15	946	207	50	189	417	210	-259	-187	-208	-187	-266
LEV14	931	184	28	166	391	197	-275	-202	-223	-202	-281
LEV13	917	160	6	144	366	184	-291	-218	-238	-219	-297
LEV12	902	137	-17	122	341	171	-308	-234	-254	-235	-314
LEV11	887	112	-39	99	316	157	-326	-251	-270	-252	-332
LEV10	872	88	-61	77	291	143	-345	-268	-287	-270	-351
LEV9	856	63	-84	54	266	128	-364	-286	-305	-288	-370
LEV8	840	38	-107	30	241	112	-385	-305	-323	-307	-391
LEV7	824	12	-130	1	210	97	-412	-329	-347	-332	-418
LEV6	806	265	41	36	241	0	0	0	0	0	0
LEV5-MECH2	792	630	175	89	497	0	0	0	0	0	0
LEV5-MECH1	767	976	474	221	744	0	0	0	0	0	0
LEV4-AMEN2	741	924	428	175	305	0	0	0	0	0	0
LEV3-AMEN1	704	855	366	113	403	0	0	0	0	0	0
LEV2-COM	657	782	297	47	536	0	0	0	0	0	0
LEV1-LOBBY	611	721	240	-11	509	0	0	0	0	0	0

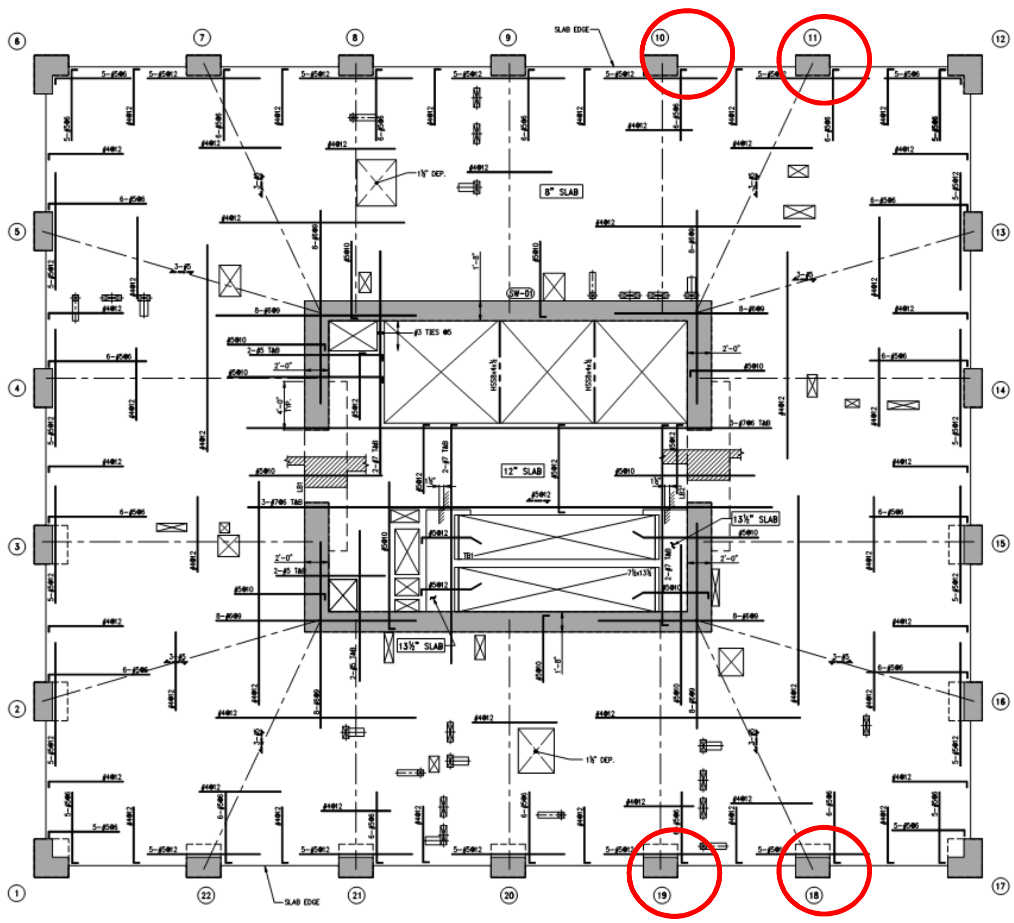
4.2 Column Compression Capacity Check

The capacities of Columns 10, 11, 18 and 19 were checked at critical locations where column sizes or concrete grades were changed or connecting to stiff elements such as outrigger walls.

SPColumn was used to check column capacity. The column loads were taken from the ETABS model. The column capacities were found to be adequate. Table 4.3 shows the summary of the column capacity check results. See appendix B for a representative calculation of column capacity check.

Table 4.3 Column Capacity Check

Column Locations	C19_L6	C10-L24	C18-L6	C18-L40	C11-L24
DCR	0.56	0.38	0.38	0.59	0.95

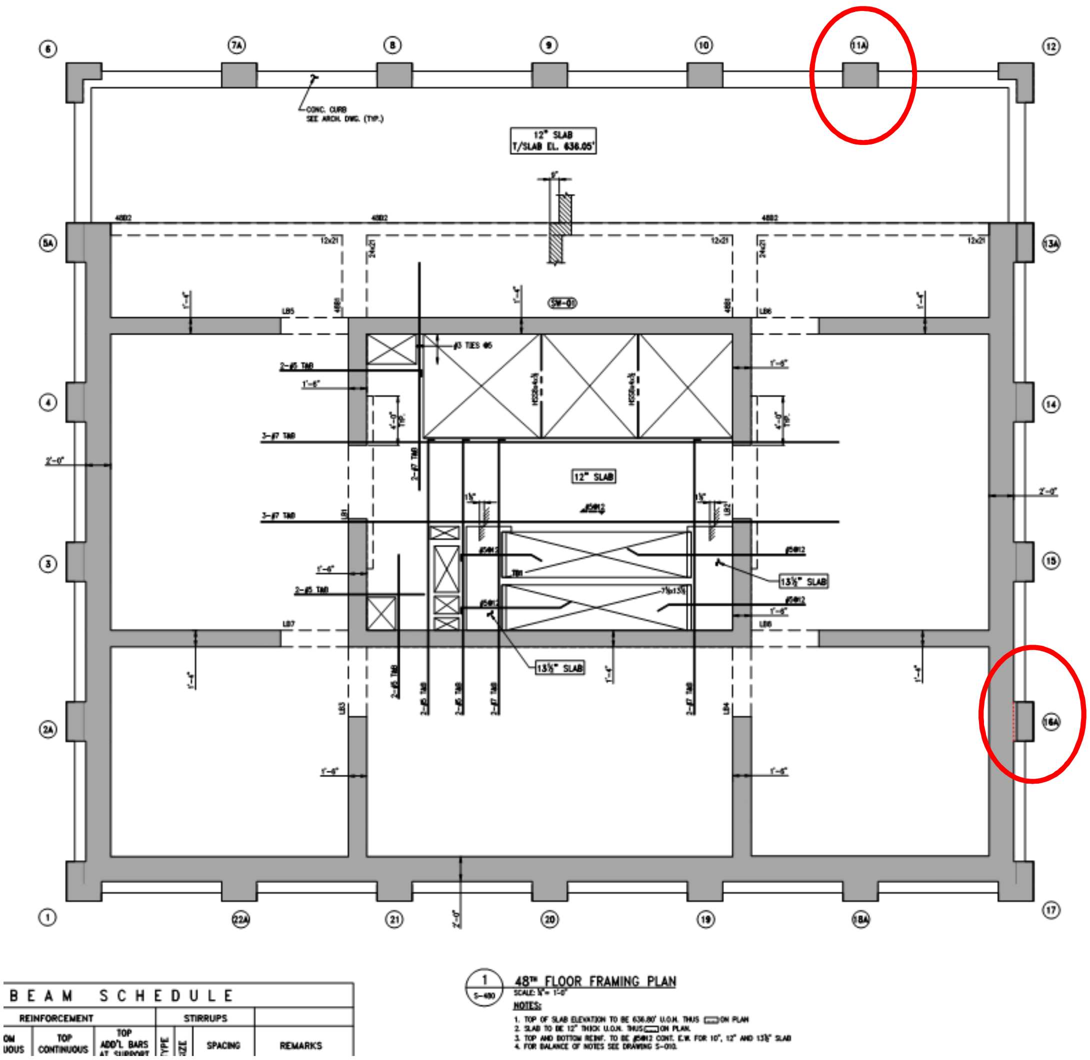


S-070.00

1 7TH TO 24TH FLOOR FRAMING PLAN

4.3 Hanging Column Check

There are columns hanging to the outrigger walls at L48 and L49. Hanging columns 16A (from L40-L48) and 11A (from L42 to L49) were selected to check the tensile capacity and the shear capacity of transferring column forces to outrigger walls. The column capacities were found to be adequate.



REV.	LERA LESLIE E. ROBERTSON ASSOCIATES, RLLP CONSULTING STRUCTURAL ENGINEERS	P 933	DRAWING NO.
	281 Fifth Ave.	DRAFT	
	Hanging Column (16A) & (11A)	DESIGN JY	
		CHECK	5/3/2016

Hanging Column (16A)   L40 - L48 $T_{max} = 75^k$

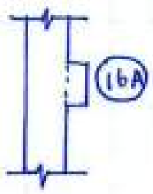
(Considering Construction Sequence

Column Schedule S-950 Column (16A) Reinforcement 6#7

$$\phi_t P_n = \phi_t F_y \cdot A_s = 0.9 \times 60\text{ ksi} \times 6 \times 0.6\text{ in}^2$$

$$= 194^k > 75^k \quad , \quad OK$$

Shear Transfer to Wall @ L48



$$\phi V_c = \phi \cdot 2\sqrt{f'_c}\, bd = 0.75 \times 2 \times \sqrt{10{,}000} \times 38'' \times 10.98' \times 12''/ft\ /1000$$

$$= 751^k > T = 75^k$$

Hanging Column (11A).   L42 - L49. $T_{max} = 137^k$

(Considering Construction Sequence)

Column Schedule S-950 Column (11A) Reinforcement 6-#8

$$\phi_t P_n = \phi_t F_y \cdot A_s = 0.9 \times 60\text{ ksi} \times 6 \times 0.79\text{ in}^2$$

$$= 256^k > T = 137^k \quad , \quad OK$$

Shear Transfer to Wall @ L49

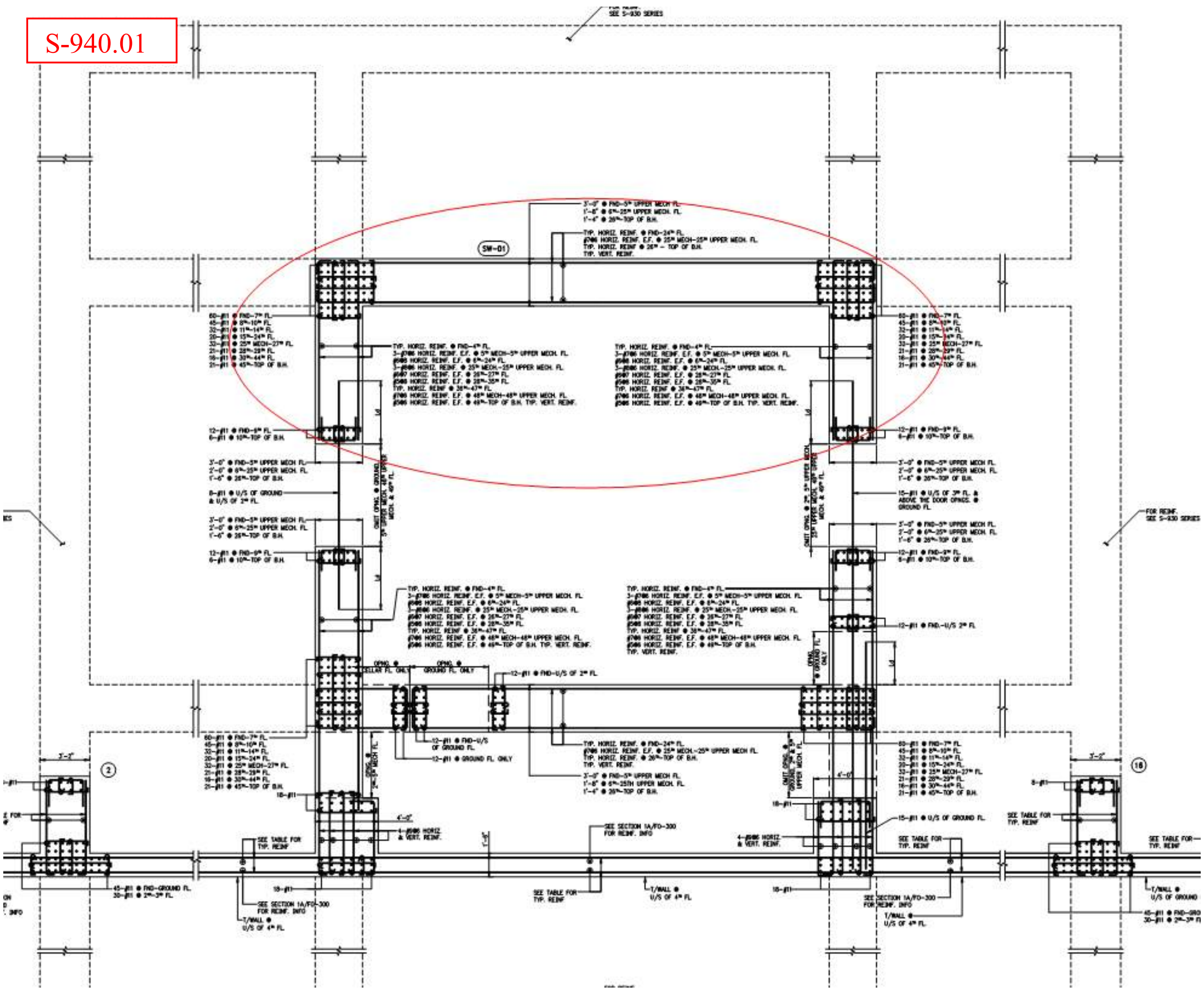
$$\phi V_c = \phi \cdot 2\sqrt{f'_c}\, bd = 0.75 \times 2 \times \sqrt{10{,}000} \times 34'' \times 10.98' \times 12''/ft\ /1000$$

$$= 672^k > T = 137^k \quad . \quad OK$$

5. Shear Wall Capacity Check

5.1 Shear Wall Flexural Capacity Check

Shear Wall SW-01 at ground floor level was checked using CSIColumn. The load combinations were taken from Etabs model. The flexural DCR is 0.35. The capacity of shear wall was found to be adequate. See appendix C for an example shear wall calculation check.



5.2 Shear Wall Shear Capacity Check

Shear Wall SW-01 at ground floor level was checked in design spreadsheet and minimum shear reinforcement was required. Table 5.2 shows the shear capacity check of this wall. The shear reinforcement provided in drawing S-940.01 was minimum. The shear capacity of shear wall was found to be adequate.

Table 5.2 Shear Wall SW-01 Shear Capacity Check

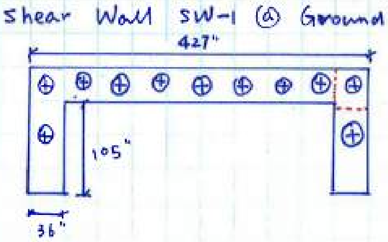
Story	Pier	Load	Loc	P	V2	V3	T	M2	M3	Thicknes (in)	Length (in)	f'c (ksi)	FiVc(Kip)	rho,v (Shear)
LEV1-LOBBY	PB	14D	Top	-29304.9	-57	47.01	-12050.9	28977.79	112265.4	36	394	14	684.4532	min
LEV1-LOBBY	PB	14D	Bottom	-30154.6	-53.65	193.76	-39514.5	16048.68	30659.6	36	394	14	892.683	min
LEV1-LOBBY	PB	12D16L	Top	-29288	-53.69	43.17	-11924.4	29975.69	105010.2	36	394	14	684.9863	min
LEV1-LOBBY	PB	12D16L	Bottom	-30071.8	-51.64	190.39	-39670.1	16375.61	23692.32	36	394	14	965.4288	min
LEV1-LOBBY	PB	09D16WT MAX	Top	653.44	556.65	634.32	60954.51	148590.8	1209843	36	394	14	622.1444	min
LEV1-LOBBY	PB	09D16WT MAX	Bottom	325.81	556.22	873.14	51924.73	233573.9	1295791	36	394	14	621.4893	min
LEV1-LOBBY	PB	09D16WT MIN	Top	-38331.2	-629.94	-573.92	-75457	-111755	-1065502	36	394	14	719.0144	min
LEV1-LOBBY	PB	09D16WT MIN	Bottom	-39096	-625.21	-624.02	-100479	-213372	-1256371	36	394	14	702.1994	min
LEV1-LOBBY	PB	12D10L16WT MAX	Top	-8232.14	541.42	646.2	57375.24	158011.3	1239389	36	394	14	636.6338	min
LEV1-LOBBY	PB	12D10L16WT MAX	Bottom	-8776.51	541.19	929.86	39832.02	238650.1	1300744	36	394	14	635.9777	min
LEV1-LOBBY	PB	12D10L16WT MIN	Top	-47216.8	-645.17	-562.04	-79036.2	-102334	-1035956	36	394	14	747.2367	min
LEV1-LOBBY	PB	12D10L16WT MIN	Bottom	-48198.4	-640.24	-567.3	-112572	-208296	-1251418	36	394	14	723.4799	min
LEV1-LOBBY	PB	12D10L10EQN MAX	Top	-22474.3	-25.27	44.73	8790.183	33598.41	130477.6	36	394	14	629.4122	min
LEV1-LOBBY	PB	12D10L10EQN MAX	Bottom	-23168.8	-23.16	191.33	-18299.2	19103.78	70358.57	36	394	14	648.0679	min
LEV1-LOBBY	PB	12D10L10EQN MIN	Top	-27679.5	-248.18	-115.89	-8930.73	-5802.08	-274528	36	394	14	742.1781	min
LEV1-LOBBY	PB	12D10L10EQN MIN	Bottom	-28474.7	-245.1	-13.58	-28624.4	-43505.5	-391326	36	394	14	701.3305	min
LEV1-LOBBY	PB	12D10L10EQP MAX	Top	-27769.4	144.43	200.07	-13721.8	61900.28	477960.9	36	394	14	649.9558	min
LEV1-LOBBY	PB	12D10L10EQP MAX	Bottom	-28500.2	146.06	376.14	-46365.4	74291.97	440651	36	394	14	655.376	min
LEV1-LOBBY	PB	12D10L10EQP MIN	Top	-32974.6	-78.48	39.45	-31442.7	22499.79	72955.8	36	394	14	791.7043	min
LEV1-LOBBY	PB	12D10L10EQP MIN	Bottom	-33806	-75.88	171.22	-56690.6	11682.74	-21033.4	36	394	14	1273.423	min

5.3 Shear Wall Foundation Tension Capacity Check

Shear Wall SW-01 at ground floor level was checked and minimum shear reinforcement was required and provided in the drawings. The tension capacity of the rock anchor under the shear wall was found to be adequate.

REV.	LERA LESLIE E. ROBERTSON ASSOCIATES, RLLP CONSULTING STRUCTURAL ENGINEERS	P 933	DRAWING NO.
	281 Fifth Ave.	DRAFT	
	Shear Wall & Rock Anchor	DESIGN JY	
		CHECK	5/9/2016

Shear Wall SW-1 @ Ground Floor.



Shear Wall Design Assuming Distributed Vertical Reinf:

$\rho_{vertical} = 0.69\%$

Total As Required: $A_s = (427 \times 36 + 105 \times 36 \times 2) \times 0.69\%$

$= 158 \text{ in}^2$

Shear Wall Vertical Reinforcement Provided in Drawing S-940.01:

144 #11 & 77 #7

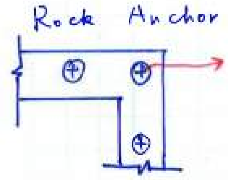
Total As Provided: $A_s = 144 \times 1.56 + 77 \times 0.6 = 270 \text{ in}^2 > 158 \text{ in}^2$

Total As Provided at Boundary Elements:

$A_s = 144 \times 1.56 = 224 \text{ in}^2 > 158 \text{ in}^2$

OK

Rock Anchor check:



Base Reaction Tension Force under Service Load Combination $0.6D + 1.0W$:

$T = 368^k < 616^k$ (Rock Anchor Tension Capacity)

OK

6. Link Beam Capacity Check

6.1 Link Beam Capacity Check

Link beams LB1, LB2, LB4 and LB5 at critical locations where beam sizes and reinforcement were changed were spot-checked in design spreadsheets and the beam strength are found to be adequate. Table 6.1 lists the calculated reinforcement and the provided reinforcement in drawing S-946.00. We did notice that link beam LB2 in S-050.00 should be labeled LB7. See appendix D for a representative calculation of link beam check.

Table 6.1 Summary of Link Beam Capacity Check

LB Mark	Floor	LERA Calculated Reinforcement		WSP_Provided Reinforcement (S-946.00)	
		Flexural	Shear	Flexural	Shear
LB1	Level4	5-11	4-5@4	7#11	4-4@5
	Level7	5-11	4-5@4	7#11	4-4@5
	LEV47	4-11	2-5@12	3#11	2-5@12
LB2	Level1	3-11	4-4@12	2#11	4-4@12
	Level7	5-11	4-6@4	9#11	4-6@4
	LEV47	4-11	2-4@12	3#11	2-4@12
LB4	Level4	7-11	4-5@5	8#11	4-5@5
LB5	Level5	5-11	4-4@12	8#11	4-4@12

7. Outrigger Wall Capacity Check

7.1 Outrigger Wall Capacity Check

The forces for the outrigger wall (highlighted in Figure 7.1 below) at L48 Mech Lower and L48 Mech Upper were extracted from the ETABS model and compared to calculated capacity. We found this outrigger wall to be adequate for the forces.

We noticed this outrigger wall thickness in the ETABS model is 16in, while it shows 24in on drawing S-480. 00. We find that a 16in wall with minimum vertical reinforcement is adequate for shear capacity. If the wall thickness is actually 24in, the vertical reinforcement shown on drawing 7/S-930 as #4@10 will not meet code minimum. The reinforcing would need to be increased to #5@10 at each face.

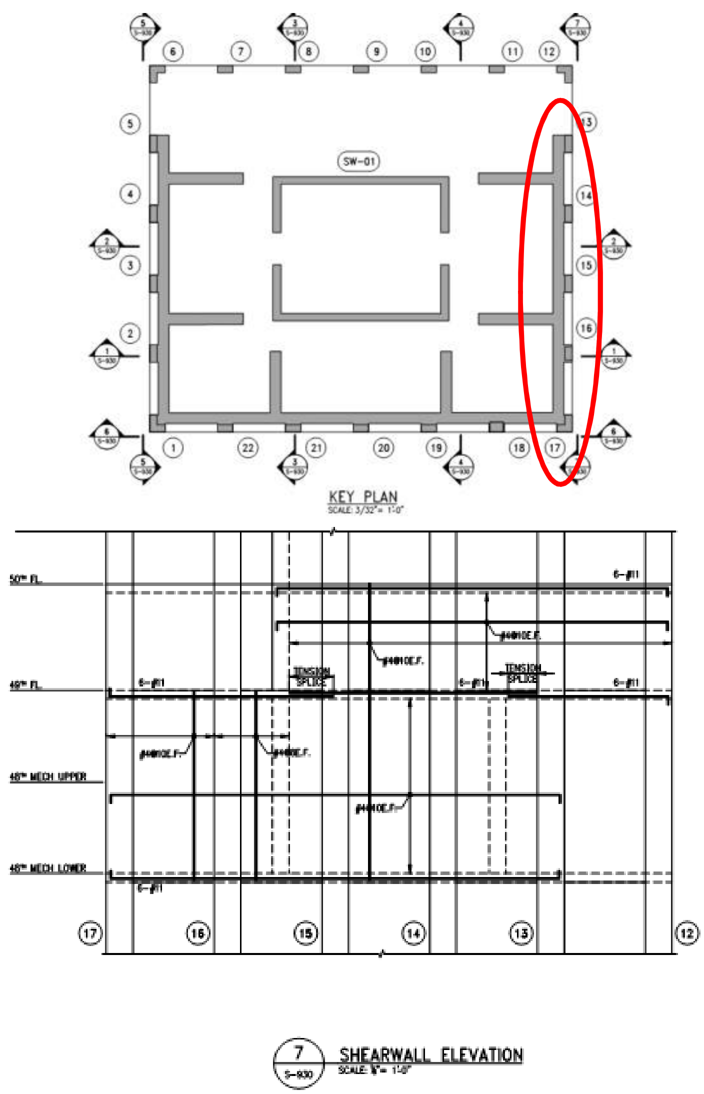


Figure 7.1 Outrigger Wall at L48

REV.	LERA LESLIE E. ROBERTSON ASSOCIATES, RLLP CONSULTING STRUCTURAL ENGINEERS	P 993	DRAWING NO.
	281 5th Ave	DRAFT	
	Outrigger Wall Check	DESIGN JY	
		CHECK	5/9/2016

(wall thickness $t = 24"$)

Outrigger Wall @ L48-Mech Lower & L48-Mech Upper

$\frac{7}{S-930}$

Vertical #4 @ 10 E.F $\longrightarrow \rho_v = \frac{2 \times 0.2}{10" \times 24"} = 0.16\%$

(#4 @ 8 E.F) $\longrightarrow \rho_v = \frac{2 \times 0.2}{8" \times 24"} = 0.2\%$

Horizontal Rebar : Top & Bot : 6-#11 each.

$As_{top} = As_{bot} = 6 \times 1.56 \text{ in}^2 = 9.36 \text{ in}^2$

Outrigger Wall Design in ETABS as Spandrel.

Vertical : $0.248 \text{ in}^2/\text{ft}$ $\rho_v = \frac{0.248 \text{ in}^2}{12" \times 24"} = 0.086\% < 0.16\%$ (0.2%) Provided

Horizontal $As_{top} = As_{bot} = 5.29 \text{ in}^2 < 9.36 \text{ in}^2$ (provided)

OK.

Etabs Spandrel Flexure Design

Story	SpandLbl	StnLoc	TopSteel	TopStlRatio	TopStlCombo	MuTop	BotSteel	BotStlRatio	BotStlCombo	MuBot	Message
LEV49-TANK	OUTRIGGE	Left	5.292	0.0045	12D10L16WT	-14018.77	5.241	0.0044	09D16WT	13885.616	No Message
LEV49-TANK	OUTRIGGE	Right	0.93	0	09D16WT	-2489.642	2.9	0.0025	12D10L16WT	7728.742	No Message

Etabs Spandrel Shear Design

Story	SpandLbl	StnLoc	AVert	AHorz	ShearCombo	Vu	CapPhiVc	CapPhiVs	CapPhiVn	ADiag	DiagReinfCombo	VuDiag	DiagReinf	Message
LEV49-TANK	OUTRIGGER_L48	Left	0.248	0	12D10L16WT	195.257	133.58	61.677	195.257	0	12D10L16WT	195.257	No	No Message
LEV49-TANK	OUTRIGGER_L48	Right	0.24	0	12D10L16WT	187.049	133.366	59.637	193.002	0	12D10L16WT	187.049	No	No Message

8 Edge Column Strips Connecting Columns and Outrigger Walls

8.1. Edge Column Strips Connecting Columns and Outrigger Walls

There are two edge column strips connecting columns and outrigger walls at L6 and L26. The capacities of these edge column strips were checked and they were found to require additional reinforcing (shown below) or alternatively EOR could study an increase to the concrete section. .

Table 8.1 Comparison of Reinforcement in Edge Column Strips

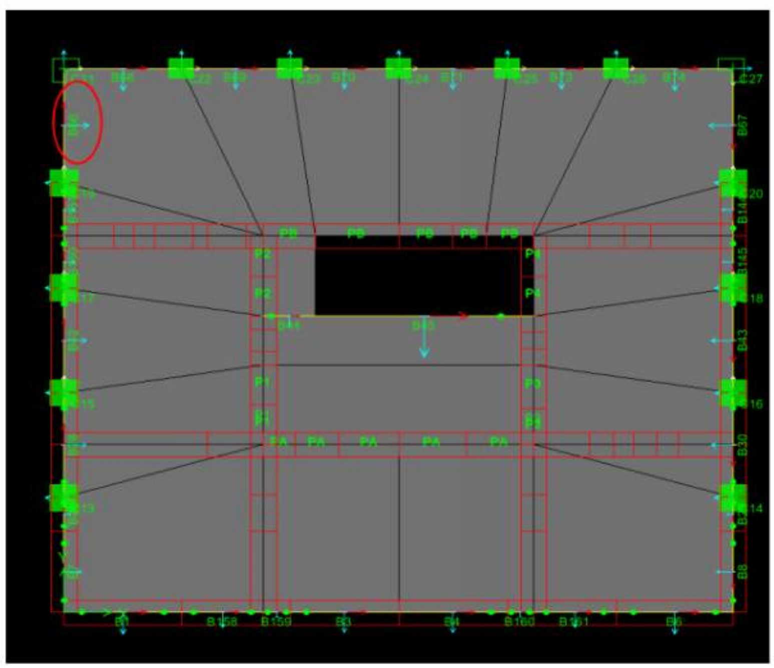
Floor	Slab Thickness	WSP		LERA	
Level 6	10”	T&B #5@10	ρ= 0.31%	5-#5 within 30”	ρ= 0.58%
Level 26	12”	T&B #5@12	ρ= 0.21%	15-#5 within 30”	ρ= 1.49%

LERA

		L6
	Beam	B66
Loads	Mu (kips-ft)	46
	Vu (kips)	14
Safety Factors	phi_m	0.90
	phi_v	0.75
Material Properties	fc' (psi)	8,600
	Beta1	0.650
	fy (psi)	60,000
Geometric Properties	b (in)	30.0
	h (in)	10.0
	Clear cover (in)	1.50
	Total cover to center of bar (in)	2.3
	d (in)	7.7
Flexure Design	A	7387.14
	B	-415125.00
	C	546552
	As_min (in²)	1.07
	As_required (in²)	1.35
	ρ_required	0.0058
	ρ_min	0.0046
	ρ_max	0.0352
	Bar size (1/8 in)	5
	Bar diameter	0.625
	Area (in²)	0.31
	Required # bars	5
	No. of Layers	1
	Bars per layer	5
	Clear Spacing (in)	5.72
	ρ_provided	0.0067
Shear Design	Stirrup size (1/8 in)	4
	Bar diameter	0.500
	Bar area (in²)	0.20
	ϕVc (kips)	32.1
	ϕVs,required (kips)	0.0
	Required Av/s (in)	0.025
	# Legs of Stirrup	0
	s_max (in)	3.8
	s_required (in)	0.0
	Provided Spacing (in)	12.0

Capacity Check	a (in)	0.42
	ϕMn (kips-ft)	51.6
	ϕVn (kips)	32.1
	Flexure Design OK?	OK
	Shear Design OK?	OK
Summary	M	5-5
	V	0-4@12
	D/C_M	0.88
	D/C_V	0.44

Edge Column Strip Connecting Column and Outrigger Walls-Level 6

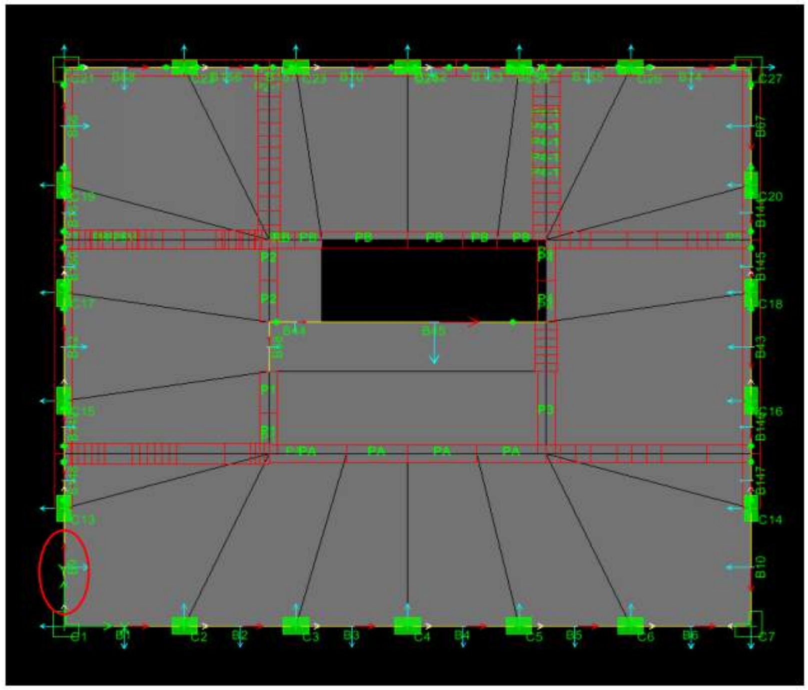


LERA

Edge Column Strip Connecting Column and Outrigger Wall_Level 26

		L26
	Beam	B9
Loads	Mu (kips-ft)	174
	Vu (kips)	38
Safety Factors	phi_m	0.90
	phi_v	0.75
Material Properties	fc' (psi)	7,200
	Beta1	0.690
	fy (psi)	60,000
Geometric Properties	b (in)	30.0
	h (in)	12.0
	Clear cover (in)	1.50
	Total cover to center of bar (in)	2.3
	d (in)	9.7
Flexure Design	A	8823.53
	B	-523125.00
	C	2093556
	As_min (in^2)	1.23
	As_required (in^2)	4.32
	ρ_required	0.0149
	ρ_min	0.0042
	ρ_max	0.0312
	Bar size (1/8 in)	5
	Bar diameter	0.625
	Area (in^2)	0.31
	Required # bars	15
	No. of Layers	1
	Bars per layer	15
	Clear Spacing (in)	1.19
	ρ_provided	0.0158
Shear Design	Stirrup size (1/8 in)	4
	Bar diameter	0.500
	Bar area (in^2)	0.20
	ϕVc (kips)	37.0
	ϕVs,required (kips)	1.4
	Required Av/s (in)	0.025
	# Legs of Stirrup	0
	s_max (in)	4.8
	s_required (in)	0.0
	Provided Spacing (in)	12.0

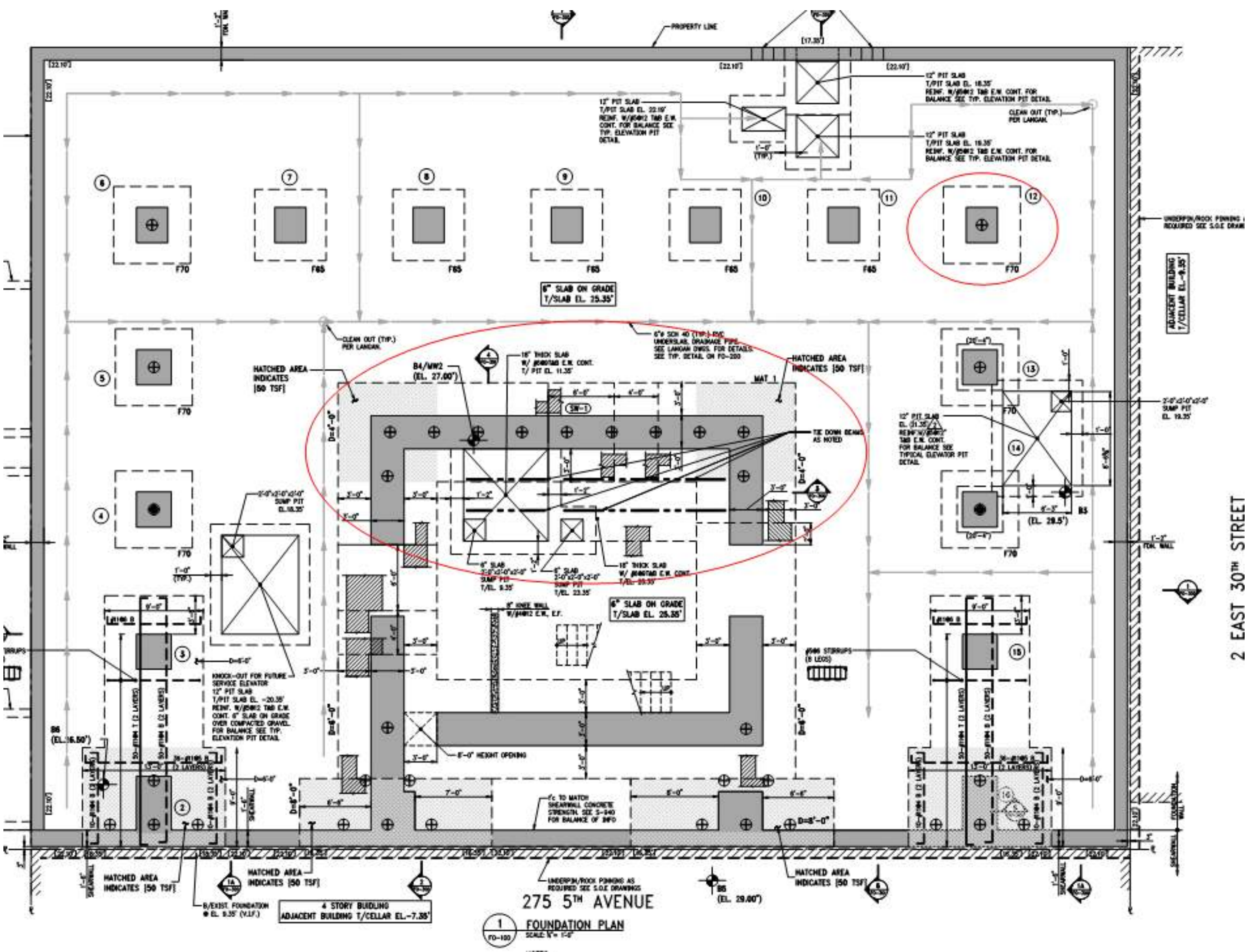
Capacity Check	a (in)	1.50
	ϕMn (kips-ft)	185.0
	ϕVn (kips)	37.0
	Flexure Design OK?	OK
	Shear Design OK?	NG
Summary	M	15-5
	V	0-4@12
	D/C_M	0.94
	D/C_V	1.04



9. Foundation Capacity Check

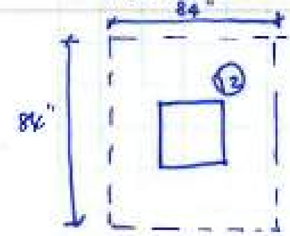
9.1 Bearing Capacity Check

Foundations of Column 12 and core wall SW-01 were checked. The maximum compression stress at F70 under service load is 27 tsf, and the bearing capacity is 40 tsf. The maximum compression stress at the bottom of wall SW-01 footing is 38 tsf and the bearing capacity is 50 tsf. The foundation bearing capacities were found to be adequate.



REV.	LERA LESLIE E. ROBERTSON ASSOCIATES, RLLP CONSULTING STRUCTURAL ENGINEERS	P 933	DRAWING NO.
	281 Fitch Ave.	DRAFT	
	Foundation Check	DESIGN JY	
		CHECK	5/9/2016

Foundation F70 Bearing Capacity Check @ Column (12)



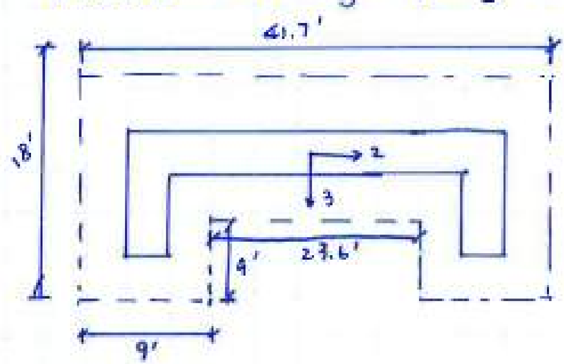
Service Load from Etabs

$P = -2645^{k}$

$\frac{P}{A} = \frac{2645 \times 10^3 \text{ lb}}{7' \times 7'} = 53980 \text{ psf}$

$= 27 \text{ tsf} < 40 \text{ tsf}$, OK

Foundation Bearing Capacity Check @ Wall SW-1



ASD load combination

1.0 D + 1.0 W (Compression)

$P = -33853^{k}$

$M = 64635$ k.ft

$p = \frac{P}{A} + \frac{M}{W}$

$= \frac{33853^{k} \times 10^3}{538.2 \text{ ft}^2} + \frac{64635^{k \cdot ft} \times 10^3}{4381 \text{ ft}^3}$

$= 77653 \text{ psf}$

$= 38 \text{ tsf} < 40 \text{ tsf}$

OK

$W_3 = \frac{1}{6} b_1 h_1^2 - \frac{1}{6} b h^2 = \frac{1}{6} \times 18' \times 41.7^2 - \frac{1}{6} \times 9 \times 23.6^2$

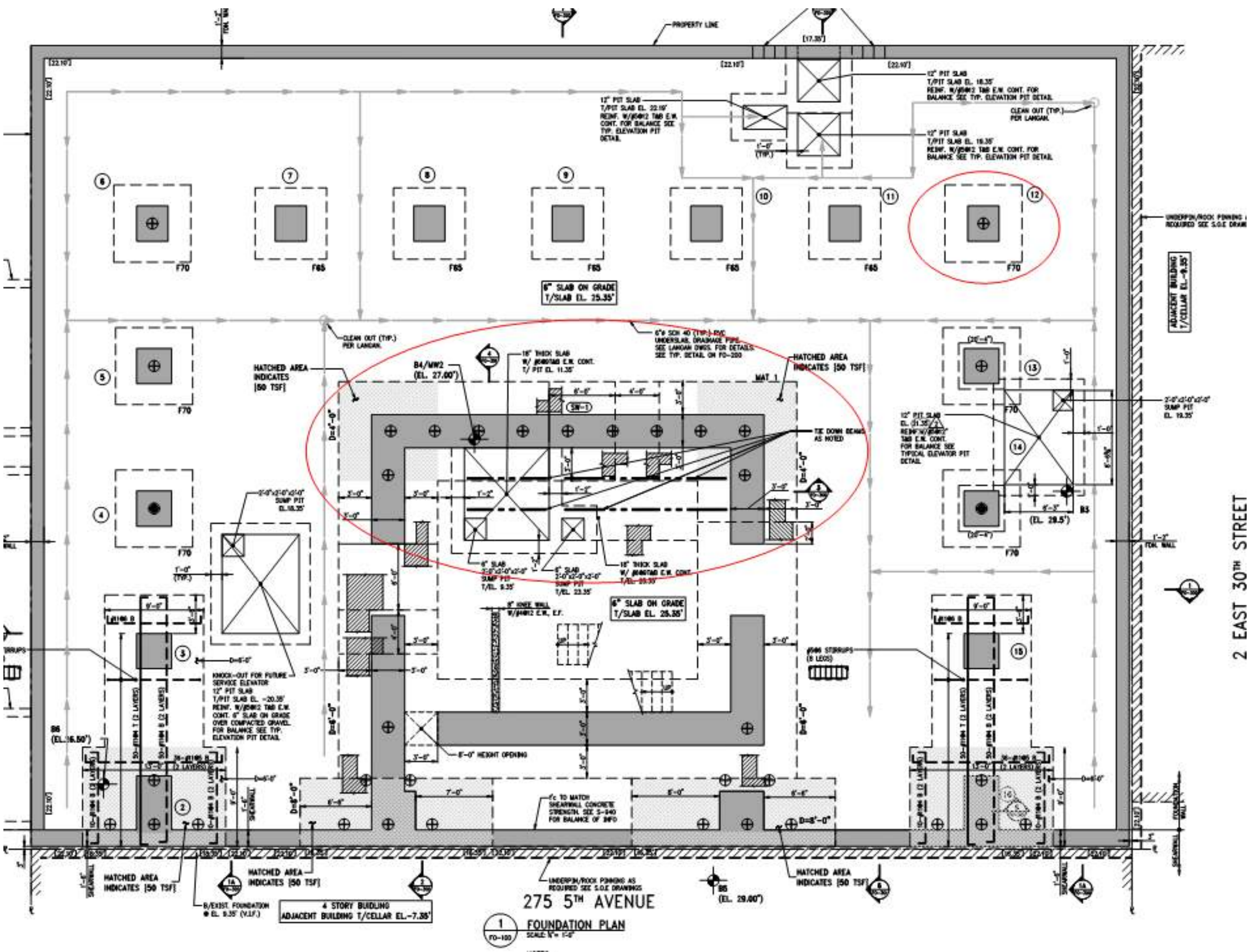
$= 4381 \text{ ft}^3$

$A = b_1 h_1 - b h = 18 \times 41.7 - 9 \times 23.6$

$= 538.2 \text{ ft}^2$

9.2 Tension Capacity Check

Foundations of Column 12 and core wall SW-01 were checked and their tension capacities were found to be adequate. The maximum tension at F70 under service load is 324 Kips, and the rock anchor capacity is 616 Kips. The maximum tension at the bottom of the wall footing is 368 Kips, and the rock anchor capacity is 616 Kips. The foundation bearing capacities were found to be adequate.



REV. | LERA LESLIE E. ROBERTSON ASSOCIATES, RLLP CONSULTING STRUCTURAL ENGINEERS | P 933 | DRAWING NO.

281 Fifth Ave.

Foundation _ Tension Check

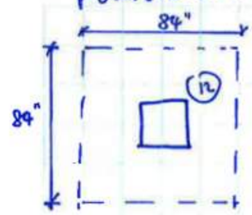
DRAFT

DESIGN JY

CHECK

5/9/2016

Foundation F70 Tension Capacity check @ column 12



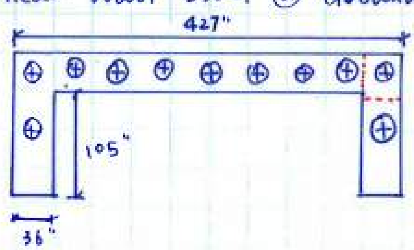
Service Load from Etabs Load combo. 0.6D+1.0W

$P = 324$ kips (Tension)

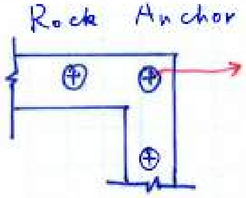
$< 616^k$ (Rock Anchor Tension Capacity)

OK.

Shear Wall SW-1 @ Ground Floor.



Rock Anchor check:



Base Reaction Tension Force under Service Load combination 0.6D + 1.0W:

$T = 368^k < 616^k$ (Rock Anchor Tension (capacity)

OK

DISCLAIMER

The structural review of an existing building requires that assumptions be made regarding existing conditions, some of which may not be verifiable within the constraints afforded to LERA. We have not completed an examination of the building at 281 Fifth Avenue, New York, NY, relying instead on such drawings as have been made available to us and on information that has been provided to us.

The opinions expressed in this report 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 reputable structural engineers of this location. No other warranty, expressed or implied, is made as to the professional advice included in this report.

Respectfully submitted,

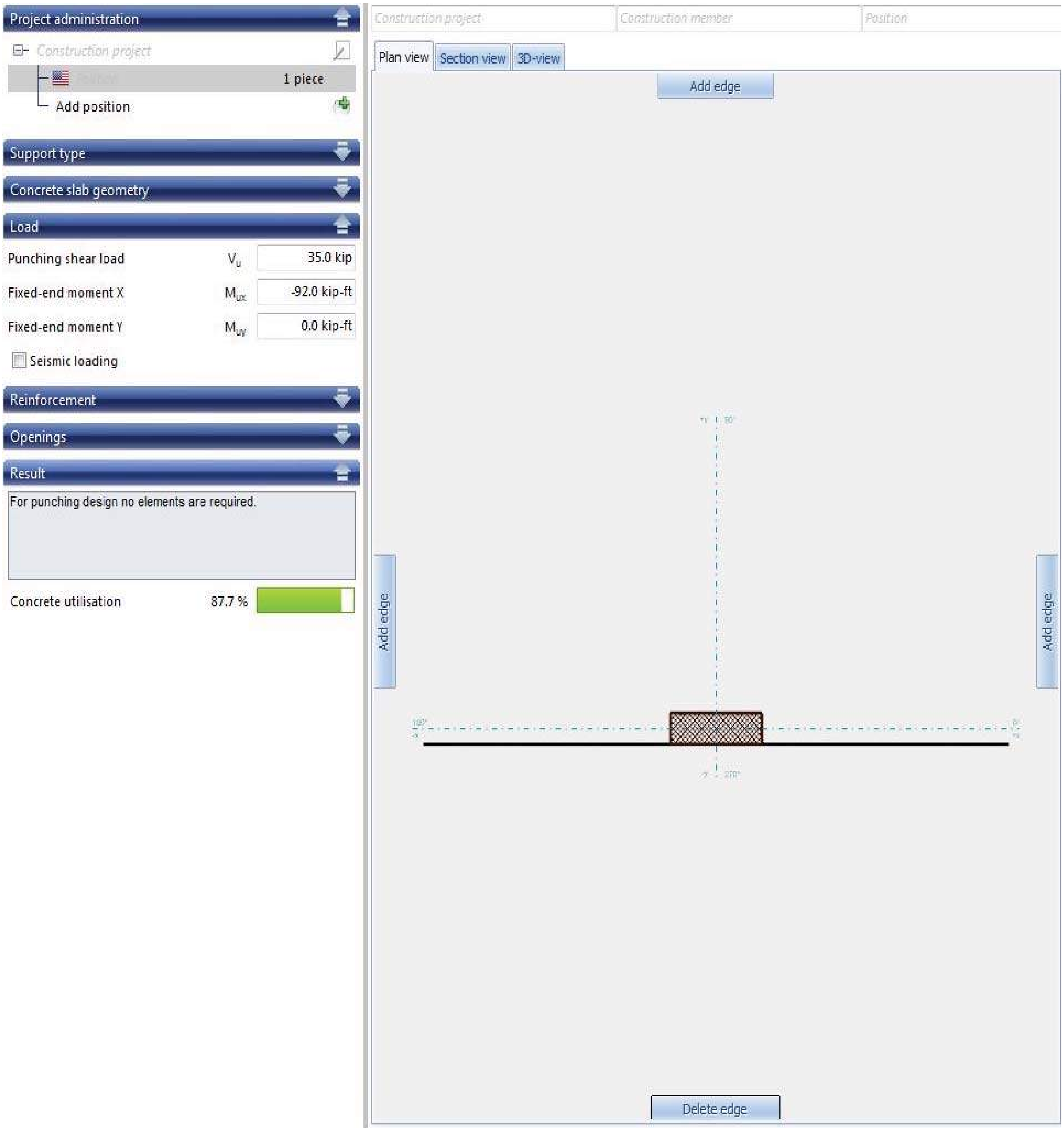
LESLIE E. ROBERTSON ASSOCIATES, R.L.L.P.

WILLIAM J. FASCHAN
Partner-In-Charge

RICHARD B. GARLOCK
Project Director

Appendix A Punching Shear Check Calculation

Column 18 Punching Shear Check Under Gravity Loads_Input



Responsible:	Construction project: Construction member: Position: Date: 七月 18, 2016

JORDAHL® EXPERT Punching shear - Design

1. Input information

Column type	Rectangular edge column						
Column dimension	c_x / c_y	=	34	in	/	10	in
Edge	r_b	=	0	in			
Slab type	In-situ concrete slab						
Slab thickness	h	=	8	in			
Concrete cover top/bottom	c_o / c_u	=	0.75	in	/	0.75	in
Effective depth	d_x / d_y	=	6.5	in	/	6.5	in
Concrete strength	7500 psi						
Density	Normal concrete						
Prestress	f_{pc}	=	0	psi			
Punching shear load	V_u	=	35	kip			
Unbalanced moment	M_{ux} / M_{uy}	=	-92	kip-ft	/	0	kip-ft

2. Output information (ACI 318-14)

2.1 Inner Critical Section (d/2 outside of column face)

2.1.1 Common Properties

Area	A_c	=	435.5	in²
Critical section perimeter	b_0	=	67	in

2.1.2 Natural Axis Properties

Centroid coordinate	e_x / e_y	=	0	in	/	5.63	in
Section moment of inertia	I_x / I_y	=	$7.090 \cdot 10^3$	in^4	/	$1.066 \cdot 10^5$	in^4
Section product of inertia	I_{xy}	=	0	in^4			

2.1.3 Principal Axis Properties

Centroid coordinate	e_1 / e_2	=	-5.63	in	/	0	in
Section moment of inertia	I_1 / I_2	=	$1.066 \cdot 10^5$	in^4	/	$7.090 \cdot 10^3$	in^4
Principal axis rotation	θ	=	90.0	°			
Moment fraction	γ_1 / γ_2	=	0.5382		/	0.1921	
Unbalanced moment	M_{u1} / M_{u2}	=	0	kip-ft	/	92	kip-ft

Responsible:

Construction project:
Construction member:
Position:
Date: 七月 18, 2016

2.1.4 Stresses

Maximum shear stresss	v_u	=	180.8	psi			
	x / y	=	-20.25	in	/	-5	in
Shear resistance (concrete only)	ϕv_c	=	206.3	psi			

Punching shear reinforcement is not required.

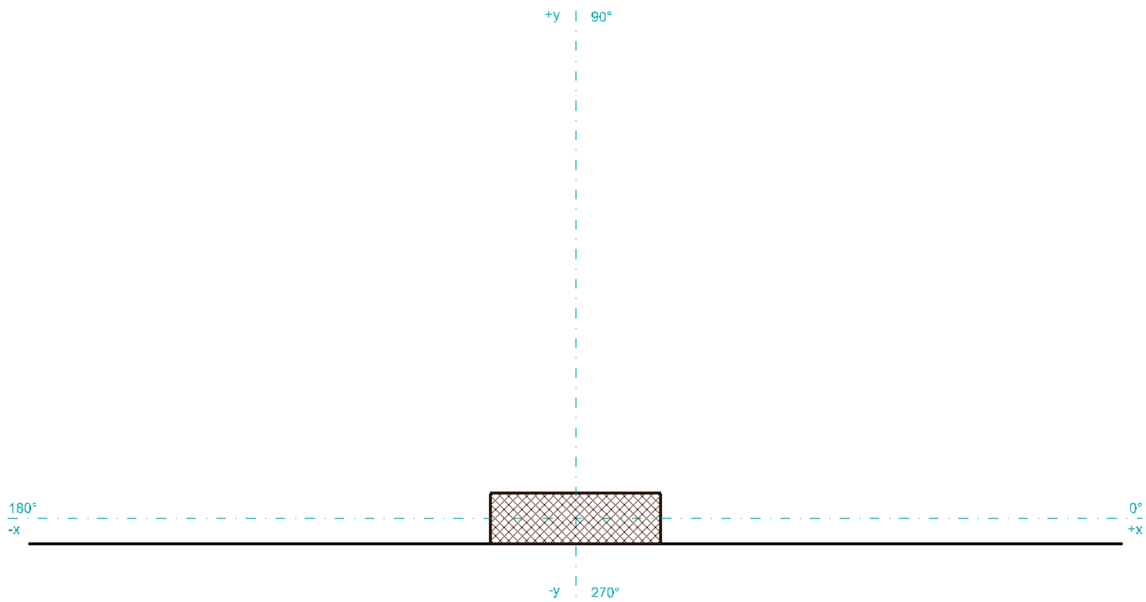
Punching Shear
DCR=180.8/206.3=0.88

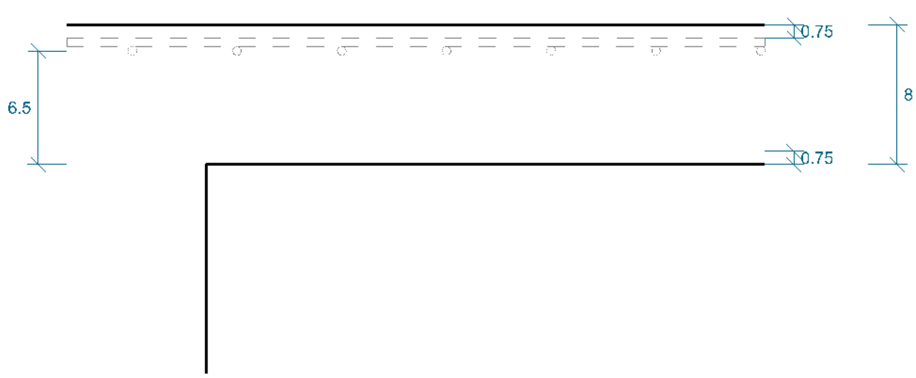
3. Note

- The design against punching shear failure is based on the rules of ACI 318-14.
- This calculation is bases on product specific properties of DECON® Studrails®. Changes, even to similar products, are only possible with new calculations.
- All data have to be checked with the given edge boundaries and the feasibility. DECON assumes no liability for the input data of the user.

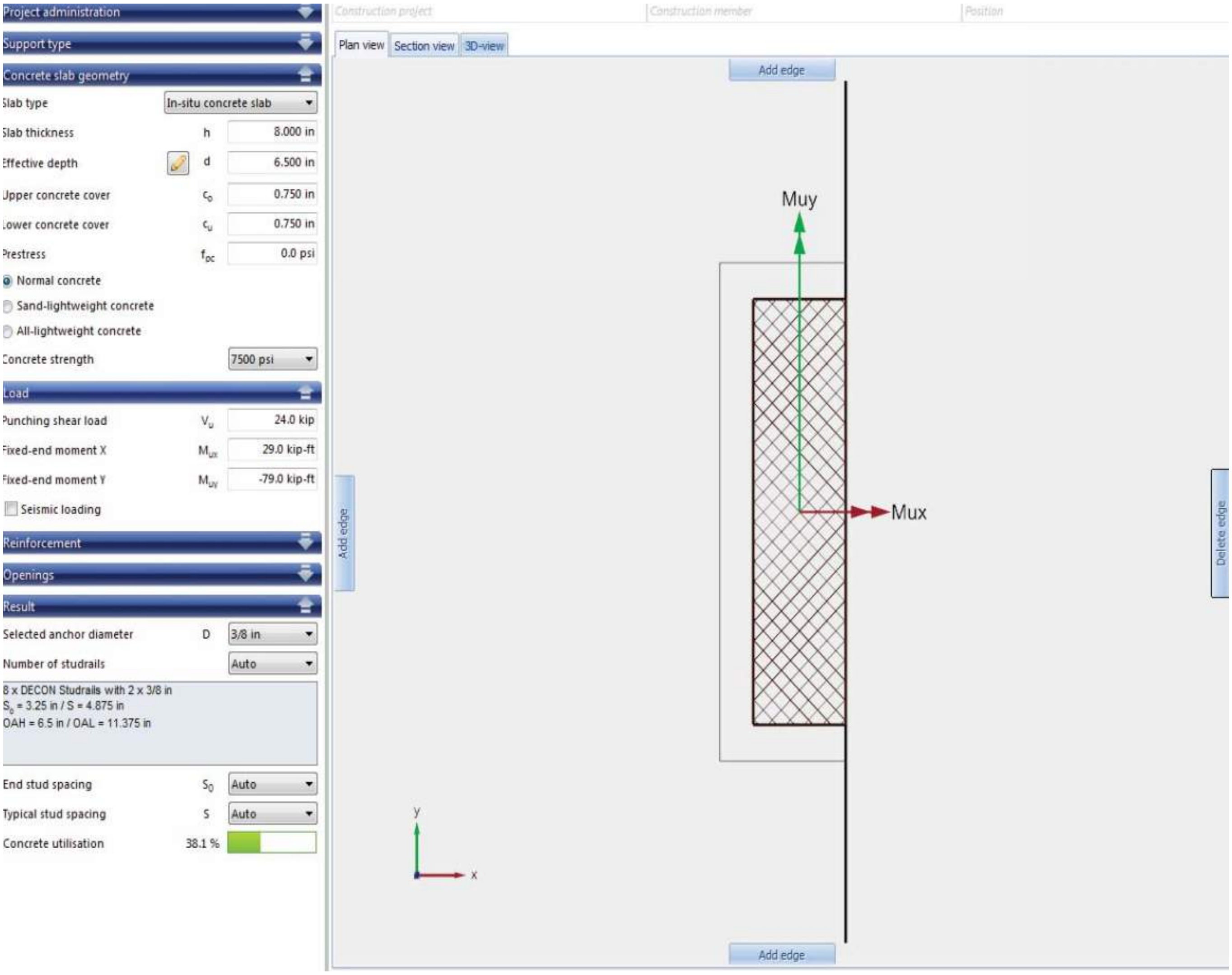
Responsible:

Construction project:
Construction member:
Position:
Date: 七月 18, 2016





Column 18 Punching Shear Check Under Wind Load Combination_Input



Responsible:

Construction project:
Construction member:
Position:
Date: 七月 22, 2016

JORDAHL® EXPERT Punching shear - Design

1. Input information

Column type	Rectangular edge column						
Column dimension	c_x / c_y	=	34	in	/	10	in
Edge	r_b	=	0	in			
Slab type	In-situ concrete slab						
Slab thickness	h	=	8	in			
Concrete cover top/bottom	c_o / c_u	=	0.75	in	/	0.75	in
Effective depth	d_x / d_y	=	6.5	in	/	6.5	in
Concrete strength	7500 psi						
Density	Normal concrete						
Prestress	f_{pc}	=	0	psi			
Punching shear load	V_u	=	23.8	kip			
Unbalanced moment	M_{ux} / M_{uy}	=	-83	kip-ft	/	15	kip-ft

2. Output information (ACI 318-14)

2.1 Inner Critical Section (d/2 outside of column face)

2.1.1 Common Properties

Area	A_c	=	435.5	in²			
Critical section perimeter	b_0	=	67	in			

2.1.2 Natural Axis Properties

Centroid coordinate	e_x / e_y	=	0	in	/	5.63	in
Section moment of inertia	I_x / I_y	=	$7.090 \cdot 10^3$	in⁴	/	$1.066 \cdot 10^5$	in⁴
Section product of inertia	I_{xy}	=	0	in⁴			

2.1.3 Principal Axis Properties

Centroid coordinate	e_1 / e_2	=	-5.63	in	/	0	in
Section moment of inertia	I_1 / I_2	=	$1.066 \cdot 10^5$	in⁴	/	$7.090 \cdot 10^3$	in⁴
Principal axis rotation	θ	=	90.0	°			
Moment fraction	γ_1 / γ_2	=	0.5382		/	0.1921	
Unbalanced moment	M_{u1} / M_{u2}	=	15	kip-ft	/	83	kip-ft

Responsible:

Construction project:
Construction member:
Position:
Date: 七月 22, 2016

2.1.4 Stresses

Maximum shear stresss	v_u	=	212	psi			
	x / y	=	-20.25	in	/	-5	in
Shear resistance (concrete only)	ϕv_c	=	206.3	psi			
Shear resistance (with studrails)	ϕv_n	=	298.3	psi			
Shear resistance (upper limit)	$\phi v_{n,max}$	=	519.6	psi			

Punching Shear
DCR=212/206.3=1.03
without studs

2.2 Outer Critical Section (d/2 outside of reinforced zone)

2.2.1 Common Properties

Area	A_c	=	566.6	in²			
Critical section perimeter	b_0	=	87.2	in			

2.2.2 Natural Axis Properties

Centroid coordinate	e_x / e_y	=	0	in	/	10.607	in
Section moment of inertia	I_x / I_y	=	$2.556 \cdot 10^4$	in⁴	/	$2.433 \cdot 10^5$	in⁴
Section product of inertia	I_{xy}	=	0	in⁴			

2.2.3 Principal Axis Properties

Centroid coordinate	e_1 / e_2	=	-10.607	in	/	0	in
Section moment of inertia	I_1 / I_2	=	$2.433 \cdot 10^5$	in⁴	/	$2.556 \cdot 10^4$	in⁴
Principal axis rotation	θ	=	90.0	°			
Moment fraction	γ_1 / γ_2	=	0.5207		/	0.2189	
Unbalanced moment	M_{u1} / M_{u2}	=	15	kip-ft	/	83	kip-ft

2.2.4 Stresses

Maximum shear stresss	v_u	=	85.6	psi			
	x / y	=	17.846	in	/	16.375	in
Shear resistance	ϕv_c	=	129.9	psi			

3. Elements

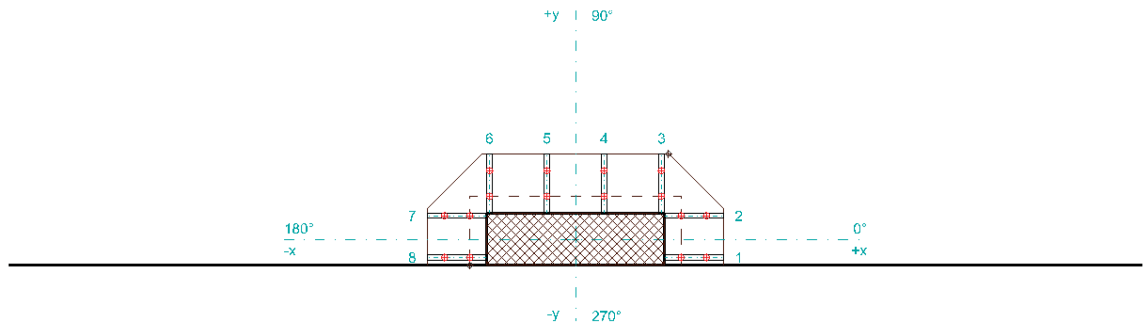
Number of studrails per column		=	8				
Number of studs per studrail		=	2				
Stud diameter	D	=	0.375	in			
Stud spacing	S / S_0	=	4.875	in	/	3.25	in
Overall height of studrail	OAH	=	6.5	in			
Overall length of studrail	OAL	=	11.375	in			

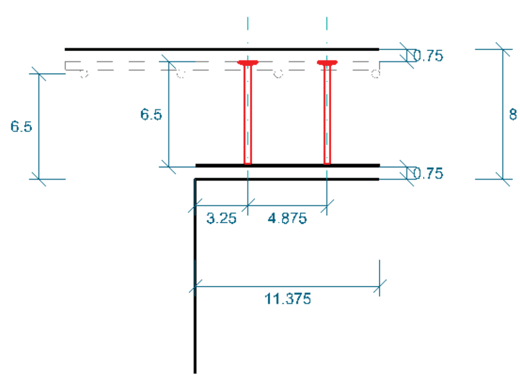
4. Note

- The design against punching shear failure is based on the rules of ACI 318-14.
- This calculation is bases on product specific properties of DECON® Studrails®. Changes, even to similar products, are only possible with new calculations.
- All data have to be checked with the given edge boundaries and the feasibility. DECON assumes no liability for the input data of the user.

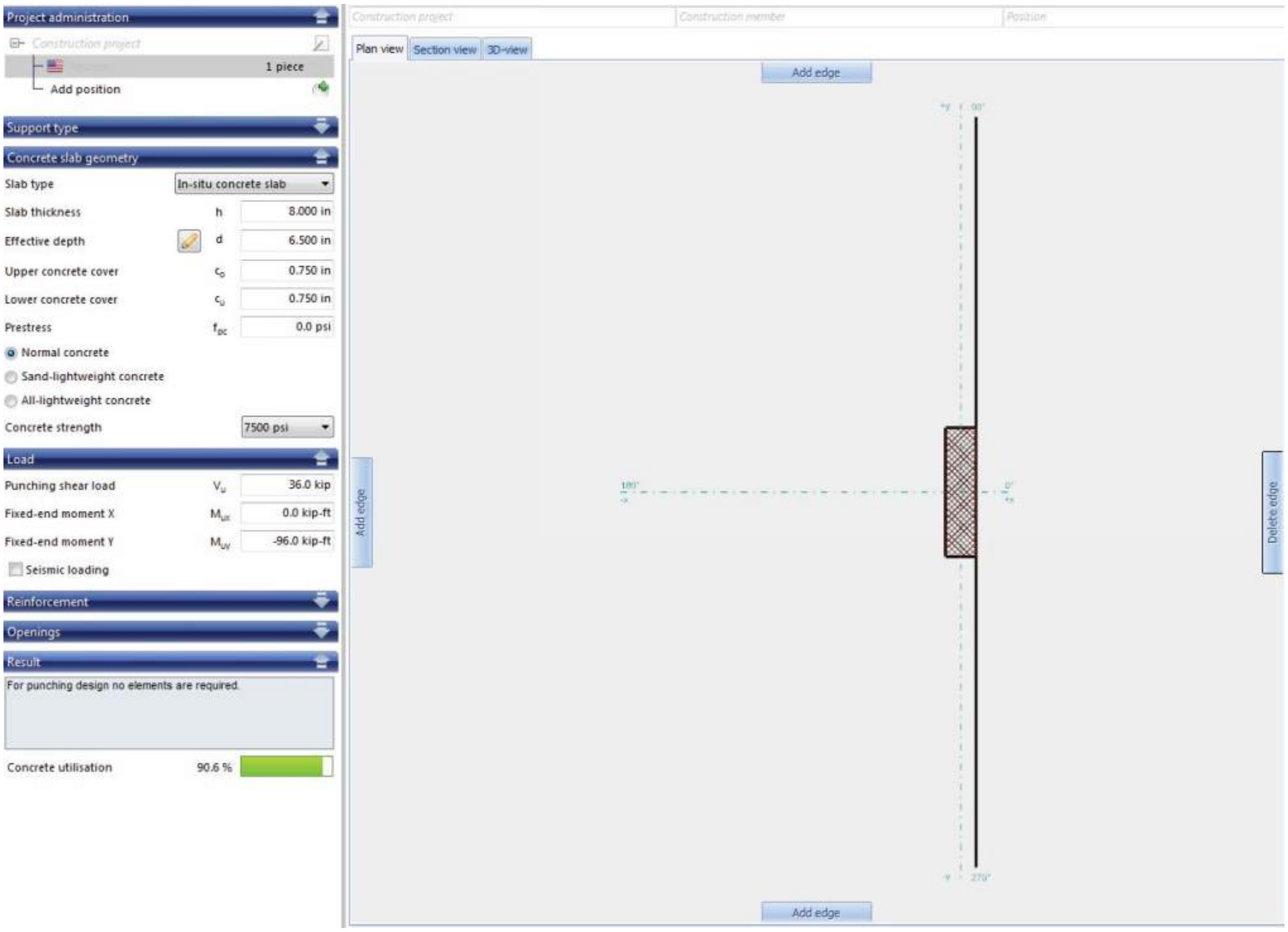
Responsible:

Construction project:
Construction member:
Position:
Date: 七月 22, 2016





Column 16 Punching Shear Check Under Gravity Load_Input



DECON a JORDAHL company

Responsible:	Construction project: Construction member: Position: Date: 七月 22, 2016

JORDAHL® EXPERT Punching shear - Design

1. Input information

Column type	Rectangular edge column						
Column dimension	c_x / c_y	=	9	in	/	38	in
Edge	r_a	=	0	in			
Slab type	In-situ concrete slab						
Slab thickness	h	=	8	in			
Concrete cover top/bottom	c_o / c_u	=	0.75	in	/	0.75	in
Effective depth	d_x / d_y	=	6.5	in	/	6.5	in
Concrete strength	7500 psi						
Density	Normal concrete						
Prestress	f_{pc}	=	0	psi			
Punching shear load	V_u	=	36	kip			
Unbalanced moment	M_{ux} / M_{uy}	=	0	kip-ft	/	-96	kip-ft

2. Output information (ACI 318-14)

2.1 Inner Critical Section (d/2 outside of column face)

2.1.1 Common Properties

Area	A_c	=	448.5	in²			
Critical section perimeter	b_0	=	69	in			

2.1.2 Natural Axis Properties

Centroid coordinate	e_x / e_y	=	-5.575	in	/	0	in
Section moment of inertia	I_x / I_y	=	$1.266 \cdot 10^5$	in^4	/	$5.844 \cdot 10^3$	in^4
Section product of inertia	I_{xy}	=	0	in^4			

2.1.3 Principal Axis Properties

Centroid coordinate	e_1 / e_2	=	5.575	in	/	0	in
Section moment of inertia	I_1 / I_2	=	$1.266 \cdot 10^5$	in^4	/	$5.844 \cdot 10^3$	in^4
Principal axis rotation	θ	=	0.0	°			
Moment fraction	γ_1 / γ_2	=	0.5596		/	0.1546	
Unbalanced moment	M_{u1} / M_{u2}	=	0	kip-ft	/	-96	kip-ft

Responsible:

Construction project:
Construction member:
Position:
Date: 七月 22, 2016

2.1.4 Stresses

Maximum shear stresss	v_u	=	173.3	psi			
	x / y	=	4.5	in	/	22.25	in
Shear resistance (concrete only)	ϕv_c	=	191.4	psi			

Punching shear reinforcement is not required.

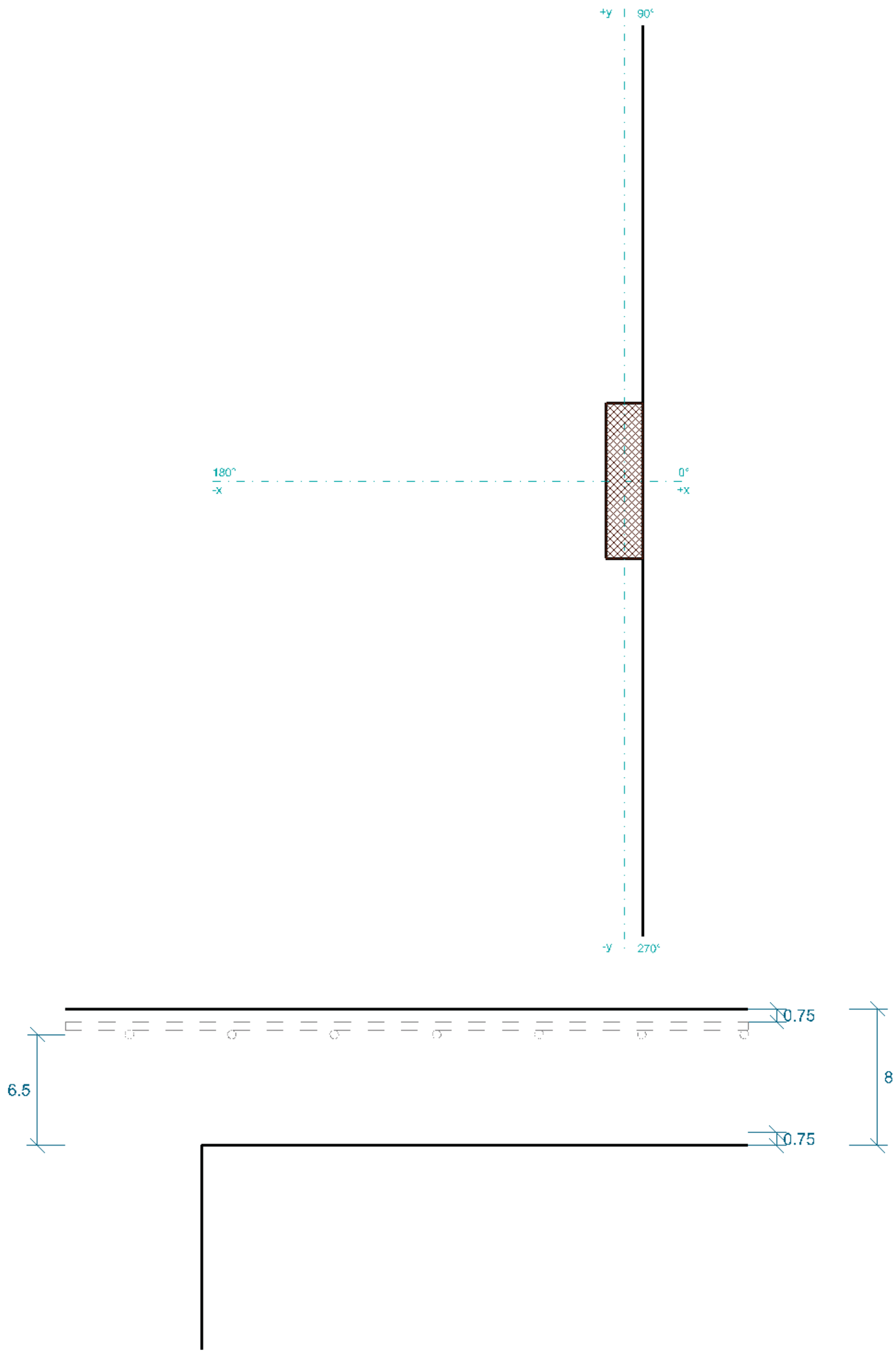
Punching Shear
DCR=173/191=0.9

3. Note

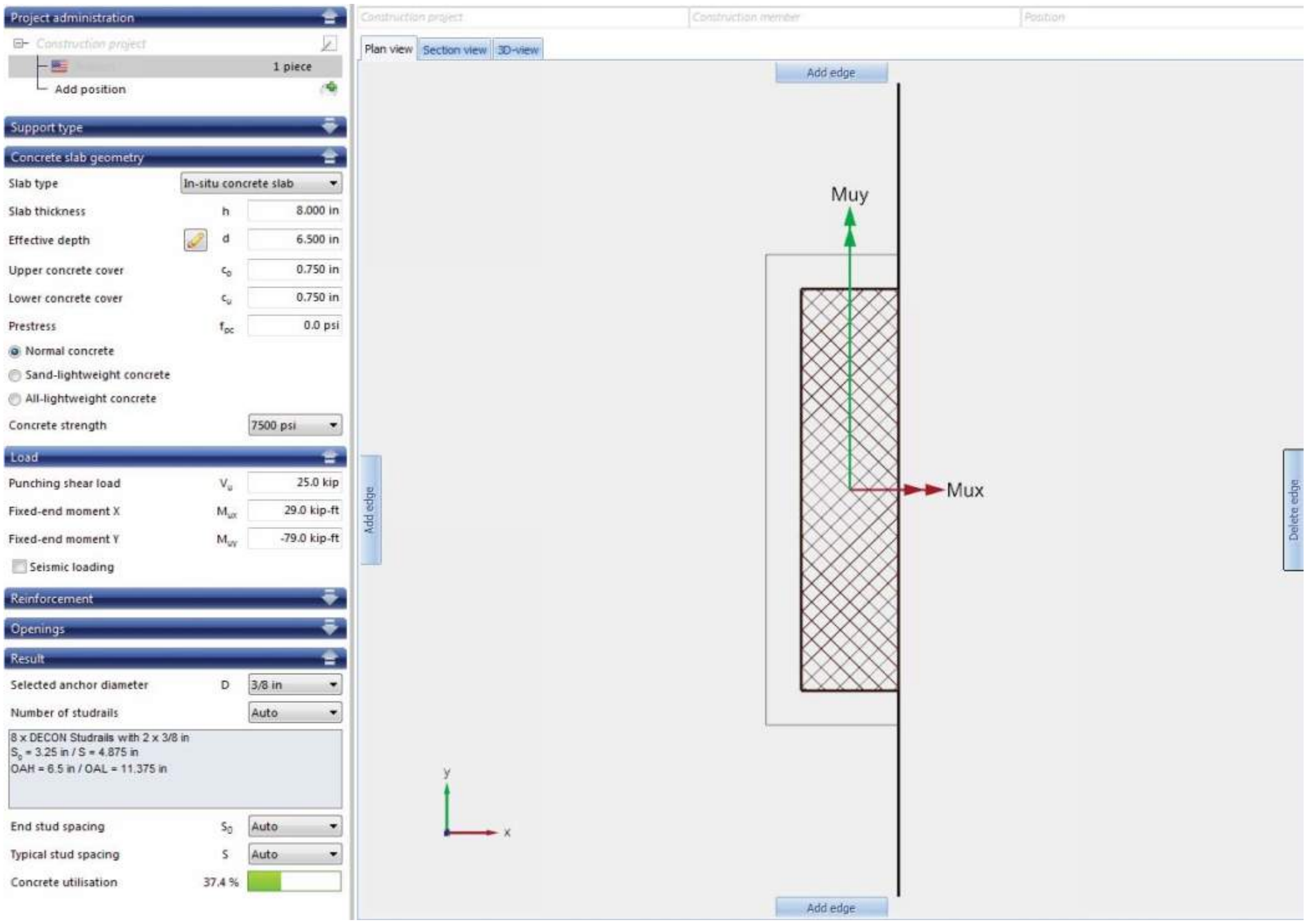
- The design against punching shear failure is based on the rules of ACI 318-14.
- This calculation is bases on product specific properties of DECON® Studrails®. Changes, even to similar products, are only possible with new calculations.
- All data have to be checked with the given edge boundaries and the feasibility. DECON assumes no liability for the input data of the user.

Responsible:

Construction project:
Construction member:
Position:
Date: 七月 22, 2016



Column 16 Punching Shear Check Under Wind Load Combination_Input



Column 16 Punching Shear Check Under Wind Load Combination_Output

Responsible:

Construction project:
Construction member:
Position:
Date: 七月 22, 2016

JORDAHL® EXPERT Punching shear - Design

1. Input information

Column type	Rectangular edge column						
Column dimension	c_x / c_y	=	9	in	/	38	in
Edge	r_a	=	0	in			
Slab type	In-situ concrete slab						
Slab thickness	h	=	8	in			
Concrete cover top/bottom	c_o / c_u	=	0.75	in	/	0.75	in
Effective depth	d_x / d_y	=	6.5	in	/	6.5	in
Concrete strength	7500 psi						
Density	Normal concrete						
Prestress	f_{pc}	=	0	psi			
Punching shear load	V_u	=	25	kip			
Unbalanced moment	M_{ux} / M_{uy}	=	29	kip-ft	/	-79	kip-ft

2. Output information (ACI 318-14)

2.1 Inner Critical Section (d/2 outside of column face)

2.1.1 Common Properties

Area	A_c	=	448.5	in²			
Critical section perimeter	b_0	=	69	in			

2.1.2 Natural Axis Properties

Centroid coordinate	e_x / e_y	=	-5.575	in	/	0	in
Section moment of inertia	I_x / I_y	=	$1.266 \cdot 10^5$	in⁴	/	$5.844 \cdot 10^3$	in⁴
Section product of inertia	I_{xy}	=	0	in⁴			

2.1.3 Principal Axis Properties

Centroid coordinate	e_1 / e_2	=	5.575	in	/	0	in
Section moment of inertia	I_1 / I_2	=	$1.266 \cdot 10^5$	in⁴	/	$5.844 \cdot 10^3$	in⁴
Principal axis rotation	θ	=	0.0	°			
Moment fraction	γ_1 / γ_2	=	0.5596		/	0.1546	
Unbalanced moment	M_{u1} / M_{u2}	=	29	kip-ft	/	-79	kip-ft

2.1.4 Stresses

Maximum shear stresss	v_u	=	194	psi			
	x / y	=	4.5	in	/	22.25	in
Shear resistance (concrete only)	ϕv_c	=	191.4	psi			
Shear resistance (with studrails)	ϕv_n	=	295.3	psi			
Shear resistance (upper limit)	$\phi v_{n,max}$	=	519.6	psi			

Punching Shear DCR=194/191=1.02 without studs

2.2 Outer Critical Section (d/2 outside of reinforced zone)

2.2.1 Common Properties

Area	A_c	=	579.6	in²			
Critical section perimeter	b_0	=	89.2	in			

2.2.2 Natural Axis Properties

Centroid coordinate	e_x / e_y	=	-10.704	in	/	0	in
Section moment of inertia	I_x / I_y	=	$2.758 \cdot 10^5$	in⁴	/	$2.325 \cdot 10^4$	in⁴
Section product of inertia	I_{xy}	=	0	in⁴			

2.2.3 Principal Axis Properties

Centroid coordinate	e_1 / e_2	=	10.704	in	/	0	in
Section moment of inertia	I_1 / I_2	=	$2.758 \cdot 10^5$	in⁴	/	$2.325 \cdot 10^4$	in⁴
Principal axis rotation	θ	=	0.0	°			
Moment fraction	γ_1 / γ_2	=	0.5351		/	0.1970	
Unbalanced moment	M_{u1} / M_{u2}	=	29	kip-ft	/	-79	kip-ft

2.2.4 Stresses

Maximum shear stresss	v_u	=	86.3	psi			
	x / y	=	-15.875	in	/	-19.846	in
Shear resistance	ϕv_c	=	129.9	psi			

3. Elements

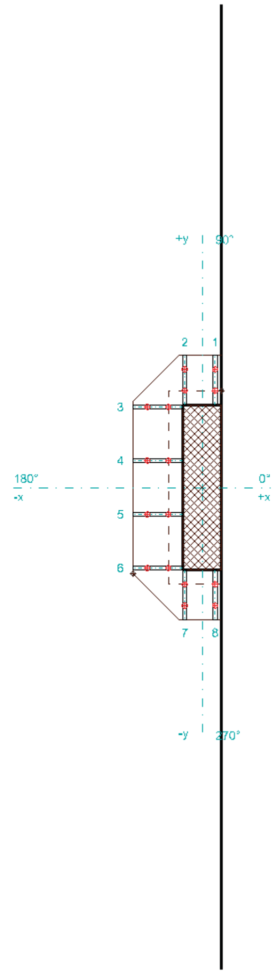
Number of studrails per column		=	8				
Number of studs per studrail		=	2				
Stud diameter	D	=	0.375	in			
Stud spacing	S / S_0	=	4.875	in	/	3.25	in
Overall height of studrail	OAH	=	6.5	in			
Overall length of studrail	OAL	=	11.375	in			

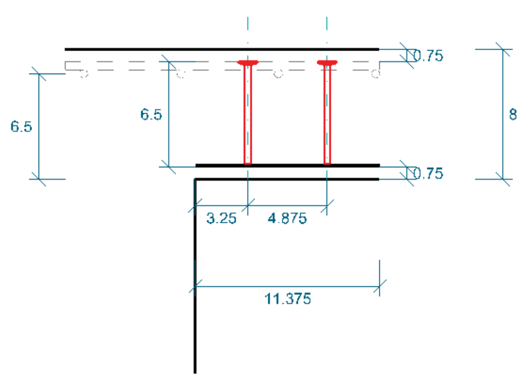
4. Note

- The design against punching shear failure is based on the rules of ACI 318-14.
- This calculation is bases on product specific properties of DECON® Studrails®. Changes, even to similar products, are only possible with new calculations.
- All data have to be checked with the given edge boundaries and the feasibility. DECON assumes no liability for the input data of the user.

Responsible:

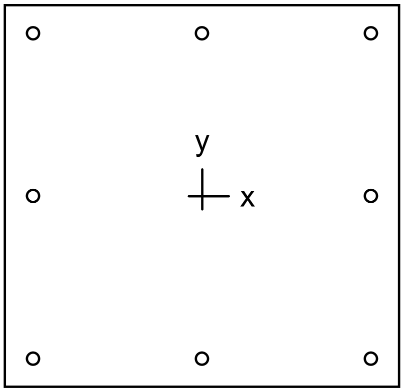
Construction project:
Construction member:
Position:
Date: 七月 22, 2016



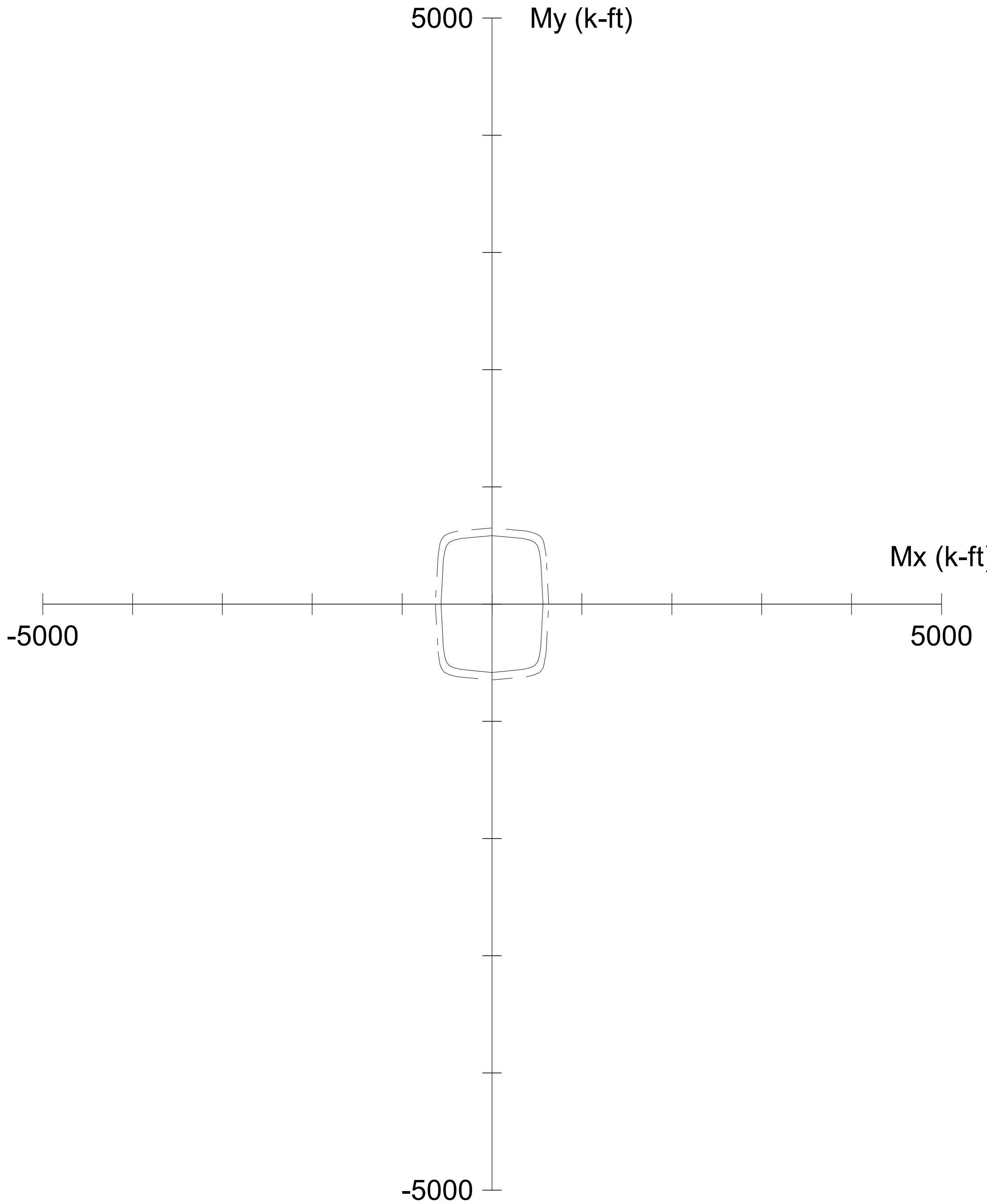


Appendix B Column Capacity Check Calculation

Column 19 at Level 6



34 x 33 in



P = 0 kip

Code: ACI 318-08

Units: English

Run axis: Biaxial

Run option: Investigation

Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 07/18/16

Time: 17:39:55

spColumn v5.00. Licensed to: Leslie E. Robertson Associates. License ID: 64917-1050653-4-2315A-2607D

File: c:\projects\p993 281 5th ave peer review\calculation\column\sp column\column 19-l6.col

Project:

Column:		Engineer:	
f'c = 14 ksi	fy = 60 ksi	Ag = 1122 in^2	8 #9 bars
Ec = 6744 ksi	Es = 29000 ksi	As = 8.00 in^2	rho = 0.71%
fc = 11.9 ksi	e_yt = 0.00206897 in/in	Xo = 0.00 in	Ix = 101822 in^4
e_u = 0.003 in/in		Yo = 0.00 in	Iy = 108086 in^4
Beta1 = 0.65		Min clear spacing = 12.93 in	Clear cover = 1.88 in
Confinement: Tied			
phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65			

```
                                   oooooo               o
                                  oo    oo             oo
               ooooo    oooooo    oo          ooooo    oo      oo   oo   o oooooooooo    o ooooo
              oo    o   oo   oo   oo         oo   oo   oo      oo   oo    oo   oo   oo    oo   oo
              oo        oo   oo   oo         oo   oo   oo      oo   oo    oo   oo   oo    oo   oo
               ooooo    oo   oo   oo         oo   oo   oo      oo   oo    oo   oo   oo    oo   oo
                   oo   oooooo    oo         oo   oo   oo      oo   oo    oo   oo   oo    oo   oo
              o    oo   oo        oo    oo   oo   oo   oo  o   oo   oo    oo   oo   oo    oo   oo
               ooooo    oo         oooooo     ooooo     ooo     ooooo o   oo   oo   oo    oo   oo (TM)
```

==

spColumn v5.00 (TM)

Computer program for the Strength Design of Reinforced Concrete Sections

==

General Information:
====================

File Name: c:\projects\p993 281 5th ave peer review\calculation\column\sp column\column 19-l6.col
Project:
Column: Engineer:
Code: ACI 318-08 Units: English

Run Option: Investigation Slenderness: Not considered
Run Axis: Biaxial Column Type: Structural

Material Properties:
====================

Concrete: Standard Steel: Standard
f'c = 14 ksi fy = 60 ksi
Ec = 6744.34 ksi Es = 29000 ksi
fc = 11.9 ksi Eps_yt = 0.00206897 in/in
Eps_u = 0.003 in/in
Beta1 = 0.65

Section:
========

Rectangular: Width = 34 in Depth = 33 in

Gross section area, Ag = 1122 in^2
Ix = 101822 in^4 Iy = 108086 in^4
rx = 9.52628 in ry = 9.81495 in
Xo = 0 in Yo = 0 in

Reinforcement:
==============

Bar Set: ASTM A615

Size	Diam (in)	Area (in^2)	Size	Diam (in)	Area (in^2)	Size	Diam (in)	Area (in^2)
# 3	0.38	0.11	# 4	0.50	0.20	# 5	0.63	0.31
# 6	0.75	0.44	# 7	0.88	0.60	# 8	1.00	0.79
# 9	1.13	1.00	# 10	1.27	1.27	# 11	1.41	1.56
# 14	1.69	2.25	# 18	2.26	4.00			

Confinement: Tied; #3 ties with #10 bars, #4 with larger bars.
phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65

Layout: Rectangular
Pattern: All Sides Equal (Cover to transverse reinforcement)
Total steel area: As = 8.00 in^2 at rho = 0.71% (Note: rho < 1.0%)
Minimum clear spacing = 12.93 in

8 #9 Cover = 1.5 in

Service Loads:
==============

No.	Load Case	Axial Load kip	Mx @ Top k-ft	Mx @ Bot k-ft	My @ Top k-ft	My @ Bot k-ft
1	Dead	2093.00	106.00	0.00	141.00	0.00
	Live	0.00	0.00	0.00	0.00	0.00
	Wind	0.00	0.00	0.00	0.00	0.00
	EQ	0.00	0.00	0.00	0.00	0.00
	Snow	0.00	0.00	0.00	0.00	0.00
2	Dead	735.00	637.00	0.00	657.00	0.00
	Live	0.00	0.00	0.00	0.00	0.00
	Wind	0.00	0.00	0.00	0.00	0.00
	EQ	0.00	0.00	0.00	0.00	0.00
	Snow	0.00	0.00	0.00	0.00	0.00
3	Dead	1367.00	333.00	0.00	309.00	0.00
	Live	0.00	0.00	0.00	0.00	0.00
	Wind	0.00	0.00	0.00	0.00	0.00
	EQ	0.00	0.00	0.00	0.00	0.00
	Snow	0.00	0.00	0.00	0.00	0.00

Sustained Load Factors:
=======================

Load Case	Factor (%)
Dead	100
Live	0
Wind	0
EQ	0
Snow	0

Load Combinations:

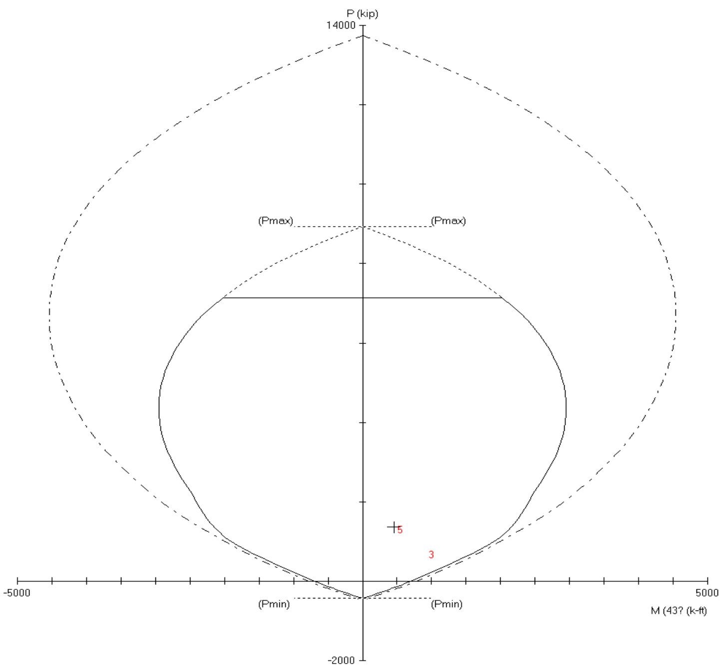
U1 = 1.000*Dead + 1.000*Live + 1.000*Wind + 1.000*EarthQuake + 1.000*Snow

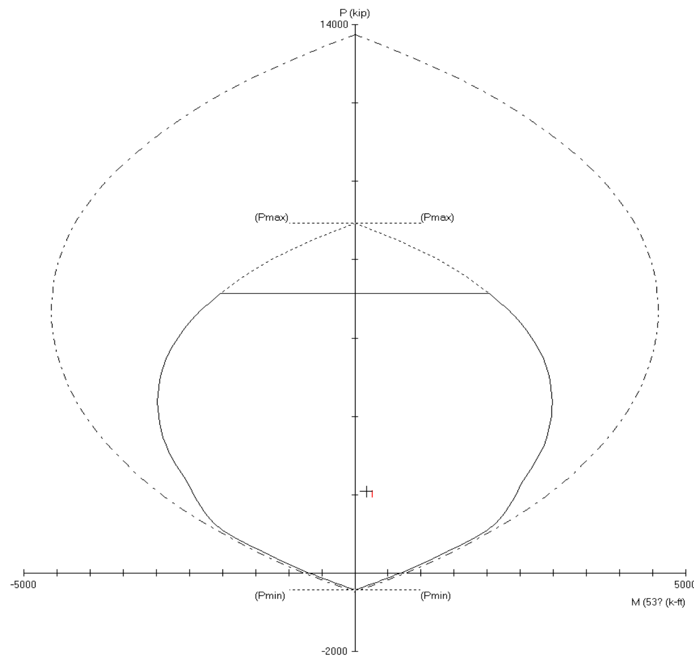
Factored Loads and Moments with Corresponding Capacities:

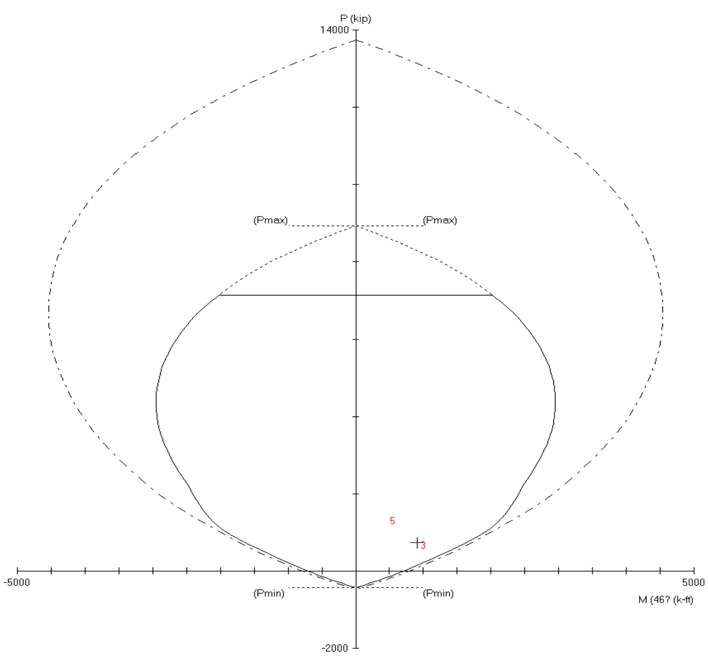
NOTE: Each loading combination includes the following cases:
First line - at column top
Second line - at column bottom

No.	Load Combo	Pu kip	Mux k-ft	Muy k-ft	PhiMnx k-ft	PhiMny k-ft	PhiMn/Mu	NA depth in	Dt depth in	eps_t	Phi
1	1 U1	2093.00	106.00	141.00	1476.89	1964.55	13.933	24.36	43.55	0.00237	0.675
2		2093.00	-0.00	-0.00	2841.95	0.00	999.999	9.43	30.56	0.00672	0.900
3	2 U1	735.00	637.00	657.00	1133.03	1168.61	1.779	14.31	43.75	0.00617	0.900
4		735.00	-0.00	-0.00	1457.32	0.00	999.999	4.00	30.56	0.01995	0.900
5	3 U1	1367.00	333.00	309.00	1579.09	1465.28	4.742	18.75	43.60	0.00398	0.813
6		1367.00	-0.00	-0.00	2152.09	0.00	999.999	6.44	30.56	0.01125	0.900

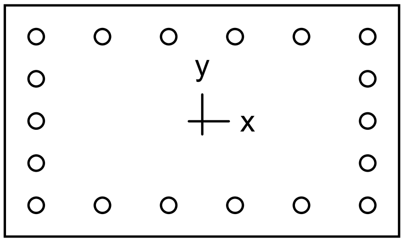
*** End of output ***







Column 10 at Level 24



34 x 20 in

Code: ACI 318-08

Units: English

Run axis: Biaxial

Run option: Investigation

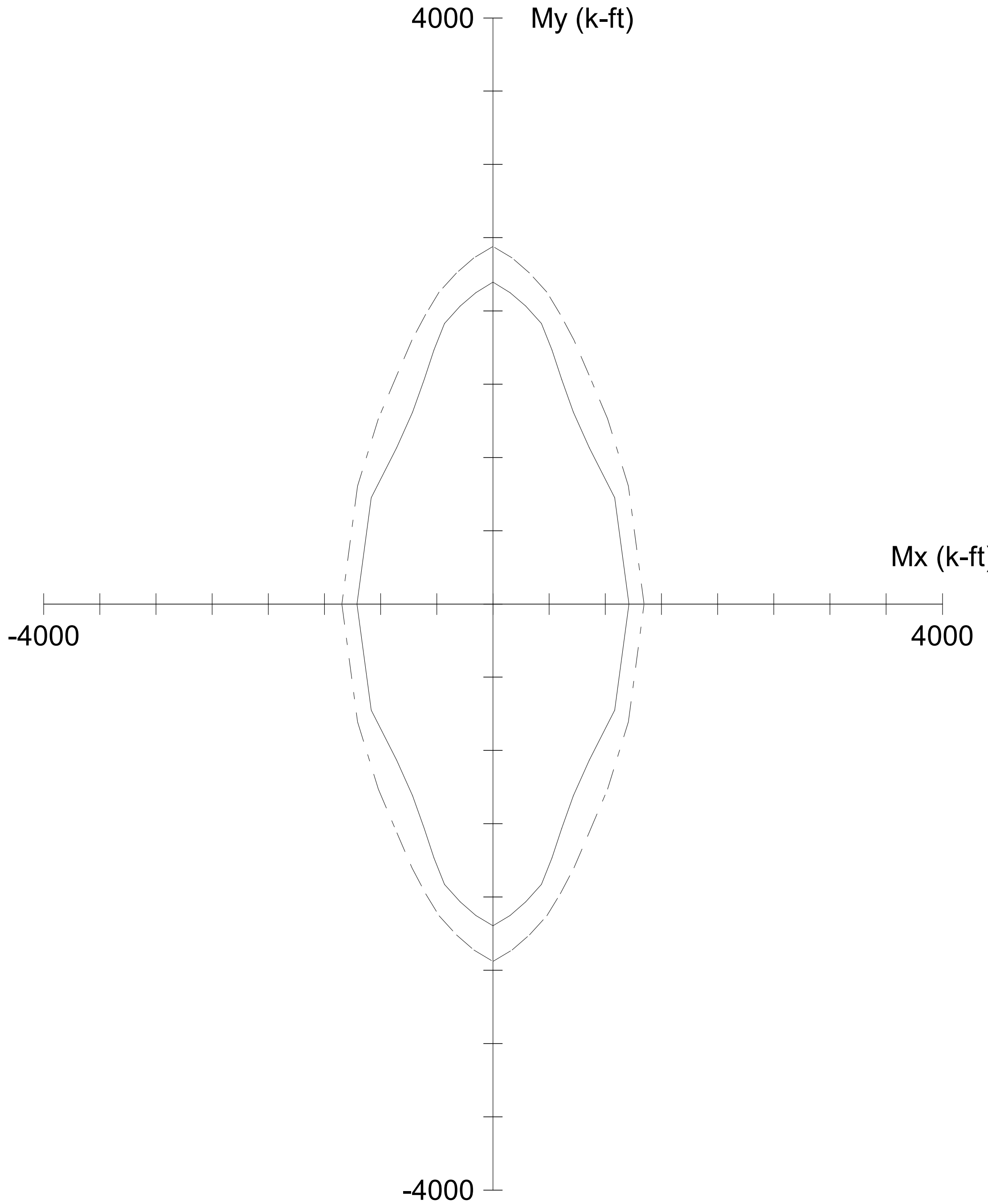
Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 07/18/16

Time: 17:30:05



P = 0 kip

spColumn v5.00. Licensed to: Leslie E. Robertson Associates. License ID: 64917-1050653-4-2315A-2607D

File: c:\projects\p993 281 5th ave peer review\calculation\column\sp column\column 10-l24.col

Project:

Column:		Engineer:	
f'c = 12 ksi	fy = 75 ksi	Ag = 680 in^2	18 #11 bars
Ec = 6244 ksi	Es = 29000 ksi	As = 28.08 in^2	rho = 4.13%
fc = 10.2 ksi	e_yt = 0.00258621 in/in	Xo = 0.00 in	Ix = 22666.7 in^4
e_u = 0.003 in/in		Yo = 0.00 in	Iy = 65506.7 in^4
Beta1 = 0.65		Min clear spacing = 2.24 in	Clear cover = 2.00 in
Confinement: Tied			

phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65

```
                       oooooo               o
                      oo    oo             oo
   ooooo    oooooo    oo          ooooo    oo      oo   oo   o oooooooooo    o ooooo
  oo    o   oo   oo   oo         oo   oo   oo      oo   oo    oo   oo   oo    oo   oo
  oo        oo   oo   oo         oo   oo   oo      oo   oo    oo   oo   oo    oo   oo
   ooooo    oo   oo   oo         oo   oo   oo      oo   oo    oo   oo   oo    oo   oo
       oo   oooooo    oo         oo   oo   oo      oo   oo    oo   oo   oo    oo   oo
  o    oo   oo        oo    oo   oo   oo   oo  o   oo   oo    oo   oo   oo    oo   oo
   ooooo    oo         oooooo     ooooo     ooo     ooooo o   oo   oo   oo    oo   oo (TM)
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spColumn v5.00 (TM)

Computer program for the Strength Design of Reinforced Concrete Sections

==

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General Information:
====================
File Name: c:\projects\p993 281 5th ave peer review\calculation\column\sp column\column 10-l24.col
Project:
Column: Engineer:
Code: ACI 318-08 Units: English

Run Option: Investigation Slenderness: Not considered
Run Axis: Biaxial Column Type: Structural

Material Properties:
====================
Concrete: Standard Steel: Standard
f'c = 12 ksi fy = 75 ksi
Ec = 6244.04 ksi Es = 29000 ksi
fc = 10.2 ksi Eps_yt = 0.00258621 in/in
Eps_u = 0.003 in/in
Beta1 = 0.65

Section:
========
Rectangular: Width = 34 in Depth = 20 in

Gross section area, Ag = 680 in^2
Ix = 22666.7 in^4 Iy = 65506.7 in^4
rx = 5.7735 in ry = 9.81495 in
Xo = 0 in Yo = 0 in

Reinforcement:
==============
Bar Set: ASTM A615

Size	Diam (in)	Area (in^2)	Size	Diam (in)	Area (in^2)	Size	Diam (in)	Area (in^2)
# 3	0.38	0.11	# 4	0.50	0.20	# 5	0.63	0.31
# 6	0.75	0.44	# 7	0.88	0.60	# 8	1.00	0.79
# 9	1.13	1.00	# 10	1.27	1.27	# 11	1.41	1.56
# 14	1.69	2.25	# 18	2.26	4.00			

Confinement: Tied; #3 ties with #10 bars, #4 with larger bars.
phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65

Layout: Rectangular
Pattern: Sides Different (Cover to transverse reinforcement)
Total steel area: As = 28.08 in^2 at rho = 4.13%
Minimum clear spacing = 2.24 in

	Top	Bottom	Left	Right
Bars	6 #11	6 #11	3 #11	3 #11
Cover(in)	1.5	1.5	1.5	1.5

Service Loads:
==============

No.	Load Case	Axial Load kip	Mx @ Top k-ft	Mx @ Bot k-ft	My @ Top k-ft	My @ Bot k-ft
1	Dead	2204.00	190.00	0.00	344.00	0.00
	Live	0.00	0.00	0.00	0.00	0.00
	Wind	0.00	0.00	0.00	0.00	0.00
	EQ	0.00	0.00	0.00	0.00	0.00
	Snow	0.00	0.00	0.00	0.00	0.00
2	Dead	-794.00	201.00	0.00	383.00	0.00
	Live	0.00	0.00	0.00	0.00	0.00
	Wind	0.00	0.00	0.00	0.00	0.00
	EQ	0.00	0.00	0.00	0.00	0.00
	Snow	0.00	0.00	0.00	0.00	0.00

Sustained Load Factors:
=======================

Load Case	Factor (%)
Dead	100
Live	0
Wind	0
EQ	0
Snow	0

Load Combinations:
==================

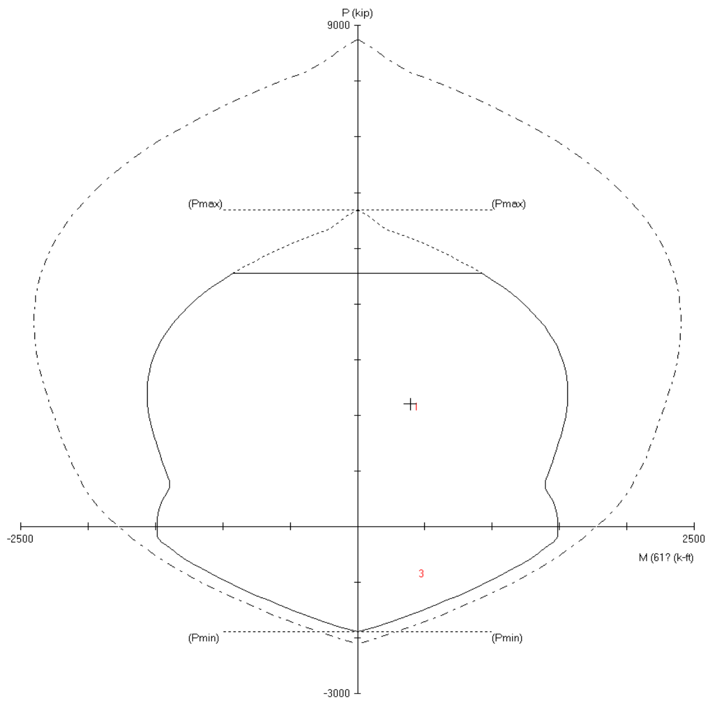
U1 = 1.000*Dead + 1.000*Live + 1.000*Wind + 1.000*EarthQuake + 1.000*Snow

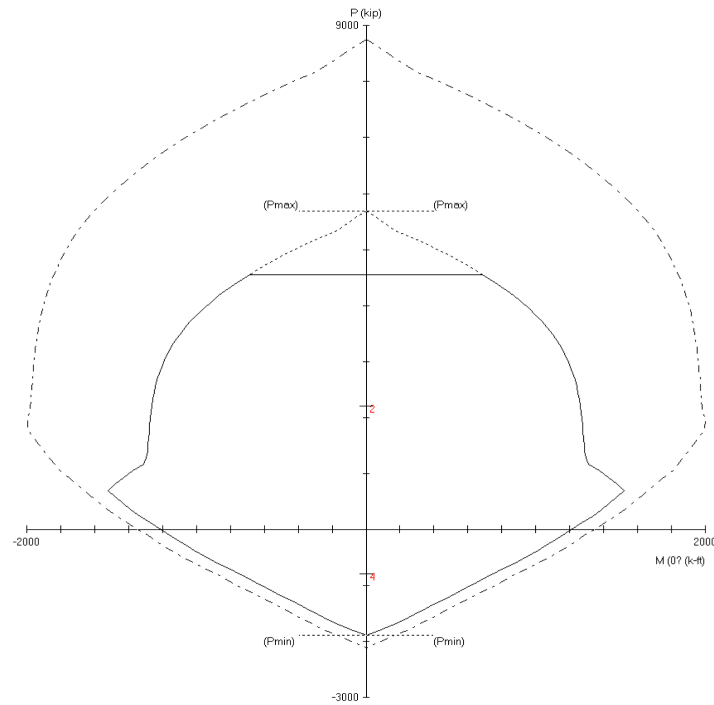
Factored Loads and Moments with Corresponding Capacities:
===

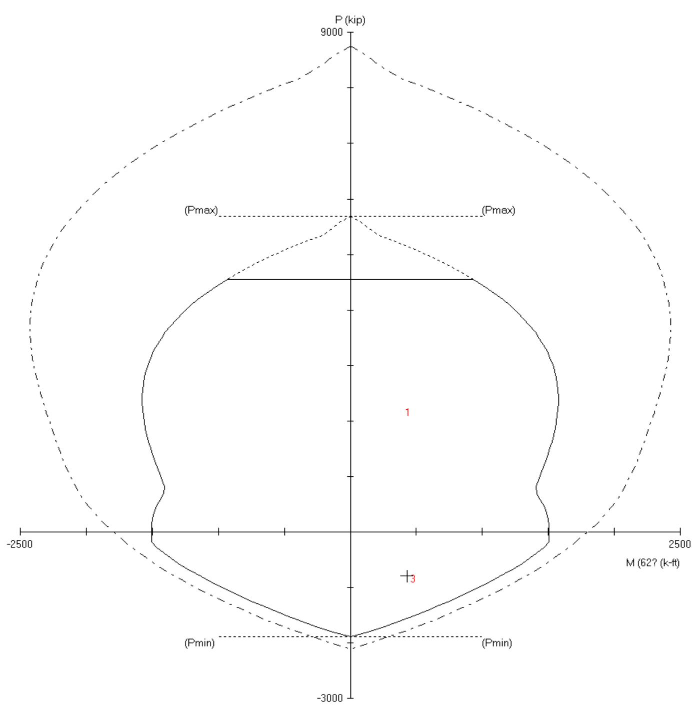
NOTE: Each loading combination includes the following cases:
First line - at column top
Second line - at column bottom

No.	Load Combo	Pu kip	Mux k-ft	Muy k-ft	PhiMnx k-ft	PhiMny k-ft	PhiMn/Mu	NA depth in	Dt depth in	eps_t	Phi
1	1 U1	2204.00	190.00	344.00	754.21	1365.51	3.970	24.01	30.86	0.00086	0.650
2		2204.00	-0.00	-0.00	1263.17	0.00	999.999	13.06	17.30	0.00097	0.650
3	2 U1	-794.00	201.00	383.00	532.06	1013.84	2.647	8.95	31.00	0.00739	0.900
4		-794.00	-0.00	-0.00	750.34	0.00	999.999	2.54	17.30	0.01740	0.900

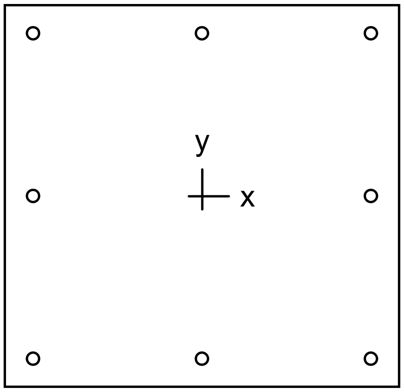
*** End of output ***







Column 18 at Level 6



34 x 33 in

Code: ACI 318-08

Units: English

Run axis: Biaxial

Run option: Investigation

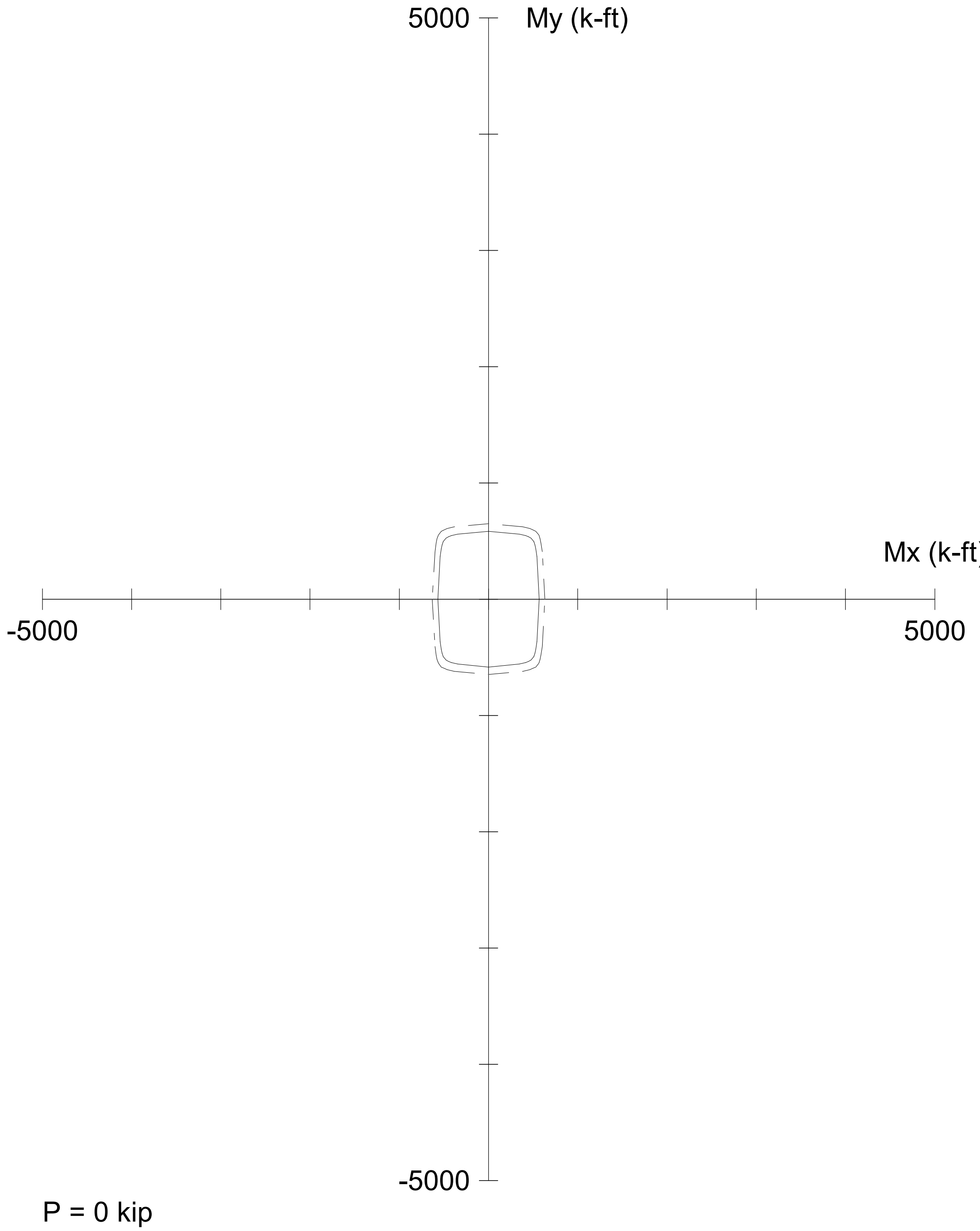
Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 07/18/16

Time: 17:36:45



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File: c:\projects\p993 281 5th ave peer review\calculation\column\sp column\column 18-l6.col

Project:

Column:		Engineer:	
f'c = 14 ksi	fy = 60 ksi	Ag = 1122 in^2	8 #9 bars
Ec = 6744 ksi	Es = 29000 ksi	As = 8.00 in^2	rho = 0.71%
fc = 11.9 ksi	e_yt = 0.00206897 in/in	Xo = 0.00 in	Ix = 101822 in^4
e_u = 0.003 in/in		Yo = 0.00 in	Iy = 108086 in^4
Beta1 = 0.65		Min clear spacing = 12.93 in	Clear cover = 1.88 in

Confinement: Tied

phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65

```
                     oooooo               o
                    oo    oo             oo
 ooooo    oooooo    oo          ooooo    oo      oo   oo   o oooooooooo    o ooooo
oo    o   oo   oo   oo         oo   oo   oo      oo   oo    oo   oo   oo    oo   oo
oo        oo   oo   oo         oo   oo   oo      oo   oo    oo   oo   oo    oo   oo
 ooooo    oo   oo   oo         oo   oo   oo      oo   oo    oo   oo   oo    oo   oo
     oo   oooooo    oo         oo   oo   oo      oo   oo    oo   oo   oo    oo   oo
o    oo   oo        oo    oo   oo   oo   oo  o   oo   oo    oo   oo   oo    oo   oo
 ooooo    oo         oooooo     ooooo     ooo     ooooo o   oo   oo   oo    oo   oo (TM)
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spColumn v5.00 (TM)

Computer program for the Strength Design of Reinforced Concrete Sections

==

General Information:

File Name: c:\projects\p993 281 5th ave peer review\calculation\column\sp column\column 18-l6.col
Project:
Column: Engineer:
Code: ACI 318-08 Units: English

Run Option: Investigation Slenderness: Not considered
Run Axis: Biaxial Column Type: Structural

Material Properties:

Concrete: Standard Steel: Standard
f'c = 14 ksi fy = 60 ksi
Ec = 6744.34 ksi Es = 29000 ksi
fc = 11.9 ksi Eps_yt = 0.00206897 in/in
Eps_u = 0.003 in/in
Beta1 = 0.65

Section:

Rectangular: Width = 34 in Depth = 33 in

Gross section area, Ag = 1122 in^2
Ix = 101822 in^4 Iy = 108086 in^4
rx = 9.52628 in ry = 9.81495 in
Xo = 0 in Yo = 0 in

Reinforcement:

Bar Set: ASTM A615

Size	Diam (in)	Area (in^2)	Size	Diam (in)	Area (in^2)	Size	Diam (in)	Area (in^2)
# 3	0.38	0.11	# 4	0.50	0.20	# 5	0.63	0.31
# 6	0.75	0.44	# 7	0.88	0.60	# 8	1.00	0.79
# 9	1.13	1.00	# 10	1.27	1.27	# 11	1.41	1.56
# 14	1.69	2.25	# 18	2.26	4.00			

Confinement: Tied; #3 ties with #10 bars, #4 with larger bars.
phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65

Layout: Rectangular
Pattern: All Sides Equal (Cover to transverse reinforcement)
Total steel area: As = 8.00 in^2 at rho = 0.71% (Note: rho < 1.0%)
Minimum clear spacing = 12.93 in

8 #9 Cover = 1.5 in

Service Loads:

No.	Load Case	Axial Load kip	Mx @ Top k-ft	Mx @ Bot k-ft	My @ Top k-ft	My @ Bot k-ft
1	Dead	1803.00	191.00	0.00	424.00	0.00
	Live	0.00	0.00	0.00	0.00	0.00
	Wind	0.00	0.00	0.00	0.00	0.00
	EQ	0.00	0.00	0.00	0.00	0.00
	Snow	0.00	0.00	0.00	0.00	0.00
2	Dead	807.00	559.00	0.00	246.00	0.00
	Live	0.00	0.00	0.00	0.00	0.00
	Wind	0.00	0.00	0.00	0.00	0.00
	EQ	0.00	0.00	0.00	0.00	0.00
	Snow	0.00	0.00	0.00	0.00	0.00

Sustained Load Factors:

Load Case	Factor (%)
Dead	100
Live	0
Wind	0
EQ	0
Snow	0

Load Combinations:

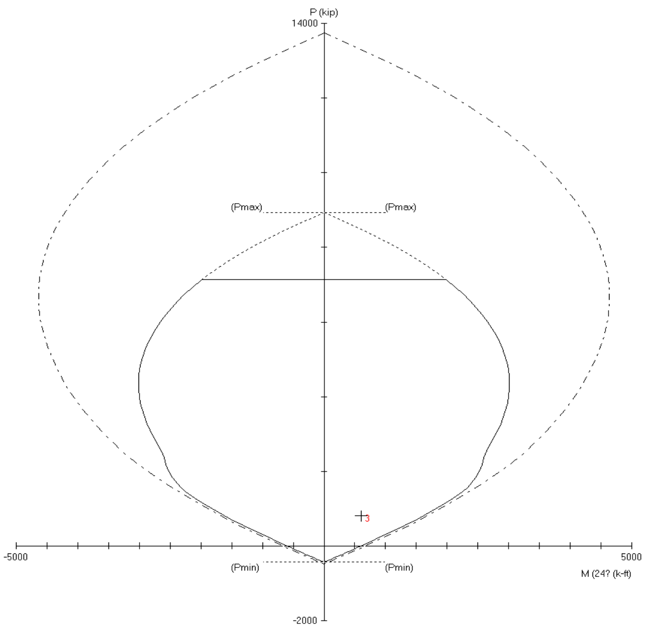
U1 = 1.000*Dead + 1.000*Live + 1.000*Wind + 1.000*EarthQuake + 1.000*Snow

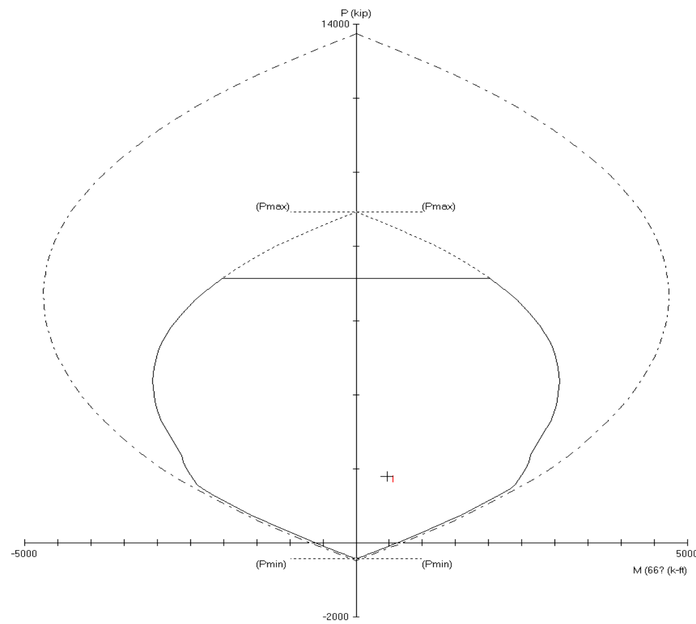
Factored Loads and Moments with Corresponding Capacities:
==

NOTE: Each loading combination includes the following cases:
First line - at column top
Second line - at column bottom

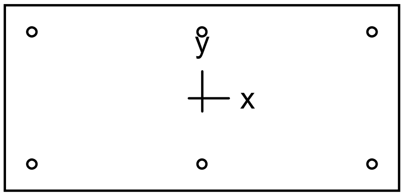
No.	Load Combo	Pu kip	Mux k-ft	Muy k-ft	PhiMnx k-ft	PhiMny k-ft	PhiMn/Mu	NA depth in	Dt depth in	eps_t	Phi
1	1 U1	1803.00	191.00	424.00	1023.01	2270.98	5.356	18.13	40.71	0.00376	0.795
2		1803.00	-0.00	-0.00	2581.78	0.00	999.999	8.21	30.56	0.00817	0.900
3	2 U1	807.00	559.00	246.00	1467.89	645.98	2.626	9.62	36.69	0.00859	0.900
4		807.00	-0.00	-0.00	1540.41	0.00	999.999	4.26	30.56	0.01851	0.900

*** End of output ***





Column 18 at Level 40



34 x 16 in

Code: ACI 318-08

Units: English

Run axis: Biaxial

Run option: Investigation

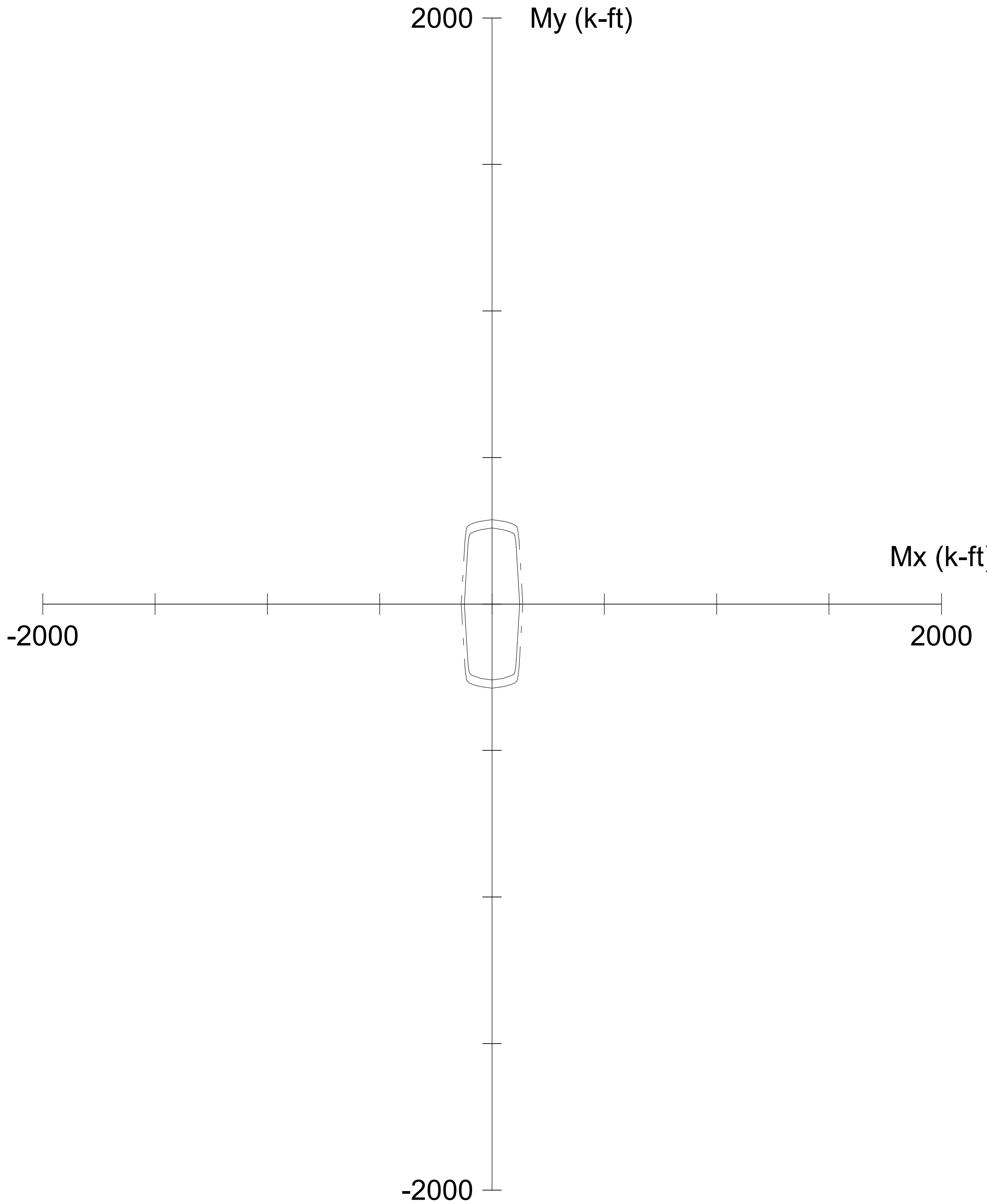
Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 07/18/16

Time: 17:38:40



P = 0 kip

spColumn v5.00. Licensed to: Leslie E. Robertson Associates. License ID: 64917-1050653-4-2315A-2607D

File: c:\projects\p993 281 5th ave peer review\calculation\column\sp column\column 18-l40.col

Project:

Column: Engineer:

f'c = 10 ksi	fy = 60 ksi	Ag = 544 in^2	6 #7 bars
Ec = 5700 ksi	Es = 29000 ksi	As = 3.60 in^2	rho = 0.66%
fc = 8.5 ksi	e_yt = 0.00206897 in/in	Xo = 0.00 in	Ix = 11605.3 in^4
e_u = 0.003 in/in		Yo = 0.00 in	Iy = 52405.3 in^4
Beta1 = 0.65		Min clear spacing = 10.50 in	Clear cover = 1.88 in

Confinement: Tied

phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65

```
                    oooooo          o
                   oo    oo         oo
 ooooo   oooooo    oo         ooooo   oo      oo   oo   o oooooooooo    o ooooo
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oo       oo   oo   oo        oo   oo  oo      oo   oo    oo   oo   oo    oo   oo
 ooooo   oo   oo   oo        oo   oo  oo      oo   oo    oo   oo   oo    oo   oo
     oo  oooooo    oo        oo   oo  oo      oo   oo    oo   oo   oo    oo   oo
o    oo  oo        oo    oo  oo   oo  oo  o   oo   oo    oo   oo   oo    oo   oo
 ooooo   oo         oooooo    ooooo    ooo     ooooo o   oo   oo   oo    oo   oo (TM)
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spColumn v5.00 (TM)

Computer program for the Strength Design of Reinforced Concrete Sections

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General Information:

File Name: c:\projects\p993 281 5th ave peer review\calculation\column\sp column\column 18-l40.col
Project:
Column: Engineer:
Code: ACI 318-08 Units: English

Run Option: Investigation Slenderness: Not considered
Run Axis: Biaxial Column Type: Structural

Material Properties:

Concrete: Standard Steel: Standard
f'c = 10 ksi fy = 60 ksi
Ec = 5700.01 ksi Es = 29000 ksi
fc = 8.5 ksi Eps_yt = 0.00206897 in/in
Eps_u = 0.003 in/in
Beta1 = 0.65

Section:

Rectangular: Width = 34 in Depth = 16 in

Gross section area, Ag = 544 in^2
Ix = 11605.3 in^4 Iy = 52405.3 in^4
rx = 4.6188 in ry = 9.81495 in
Xo = 0 in Yo = 0 in

Reinforcement:

Bar Set: ASTM A615

Size	Diam (in)	Area (in^2)	Size	Diam (in)	Area (in^2)	Size	Diam (in)	Area (in^2)
# 3	0.38	0.11	# 4	0.50	0.20	# 5	0.63	0.31
# 6	0.75	0.44	# 7	0.88	0.60	# 8	1.00	0.79
# 9	1.13	1.00	# 10	1.27	1.27	# 11	1.41	1.56
# 14	1.69	2.25	# 18	2.26	4.00			

Confinement: Tied; #3 ties with #10 bars, #4 with larger bars.
phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65

Layout: Rectangular
Pattern: Sides Different (Cover to transverse reinforcement)
Total steel area: As = 3.60 in^2 at rho = 0.66% (Note: rho < 1.0%)
Minimum clear spacing = 10.50 in

	Top	Bottom	Left	Right
Bars	3 # 7	3 # 7	0 # 7	0 # 7
Cover(in)	1.5	1.5	1.5	1.5

Service Loads:

No.	Load Case	Axial Load kip	Mx @ Top k-ft	Mx @ Bot k-ft	My @ Top k-ft	My @ Bot k-ft
1	Dead	200.00	55.00	0.00	292.00	0.00
	Live	0.00	0.00	0.00	0.00	0.00
	Wind	0.00	0.00	0.00	0.00	0.00
	EQ	0.00	0.00	0.00	0.00	0.00
	Snow	0.00	0.00	0.00	0.00	0.00

Sustained Load Factors:

Load Case	Factor (%)
Dead	100
Live	0
Wind	0
EQ	0
Snow	0

Load Combinations:

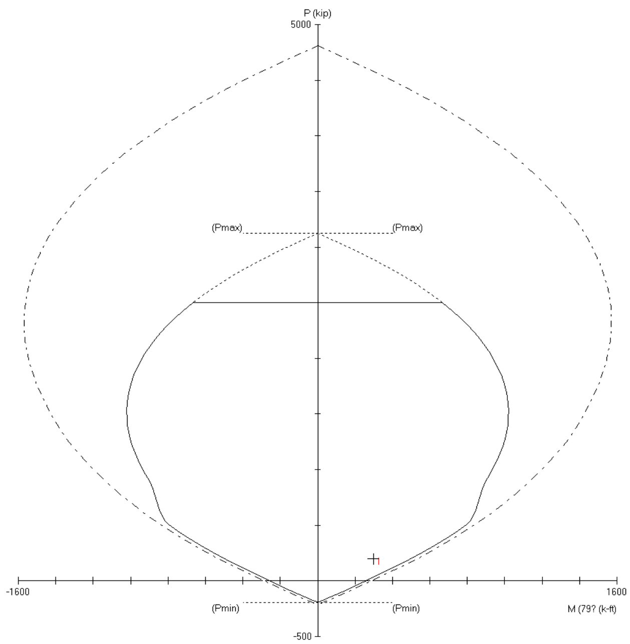
U1 = 1.000*Dead + 1.000*Live + 1.000*Wind + 1.000*EarthQuake + 1.000*Snow

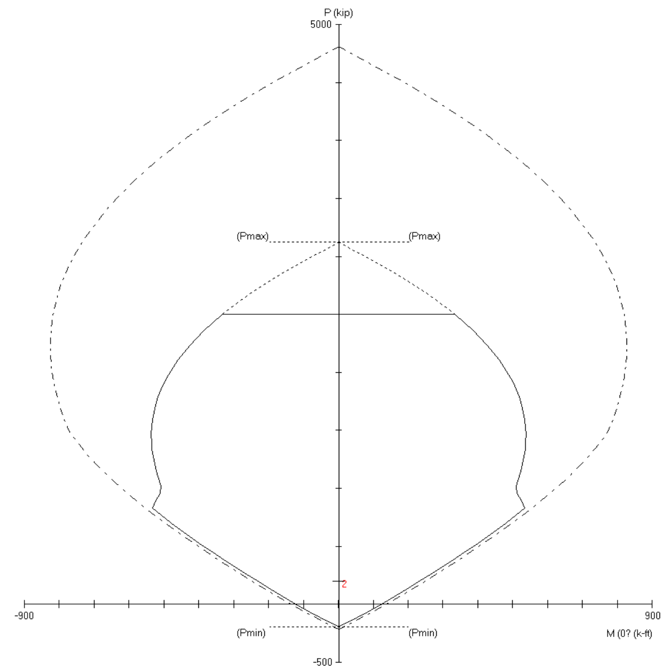
Factored Loads and Moments with Corresponding Capacities:
==

NOTE: Each loading combination includes the following cases:
First line - at column top
Second line - at column bottom

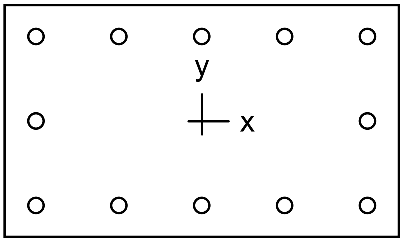
No.	Load Combo	Pu kip	Mux k-ft	Muy k-ft	PhiMnx k-ft	PhiMny k-ft	PhiMn/Mu	NA depth in	Dt depth in	eps_t	Phi
1	1 U1	200.00	55.00	292.00	92.94	493.44	1.690	8.00	34.42	0.00996	0.900
2		200.00	-0.00	-0.00	232.69	0.00	999.999	1.93	13.69	0.01833	0.900

*** End of output ***





Column 11 at Level 24



34 x 20 in

Code: ACI 318-08

Units: English

Run axis: Biaxial

Run option: Investigation

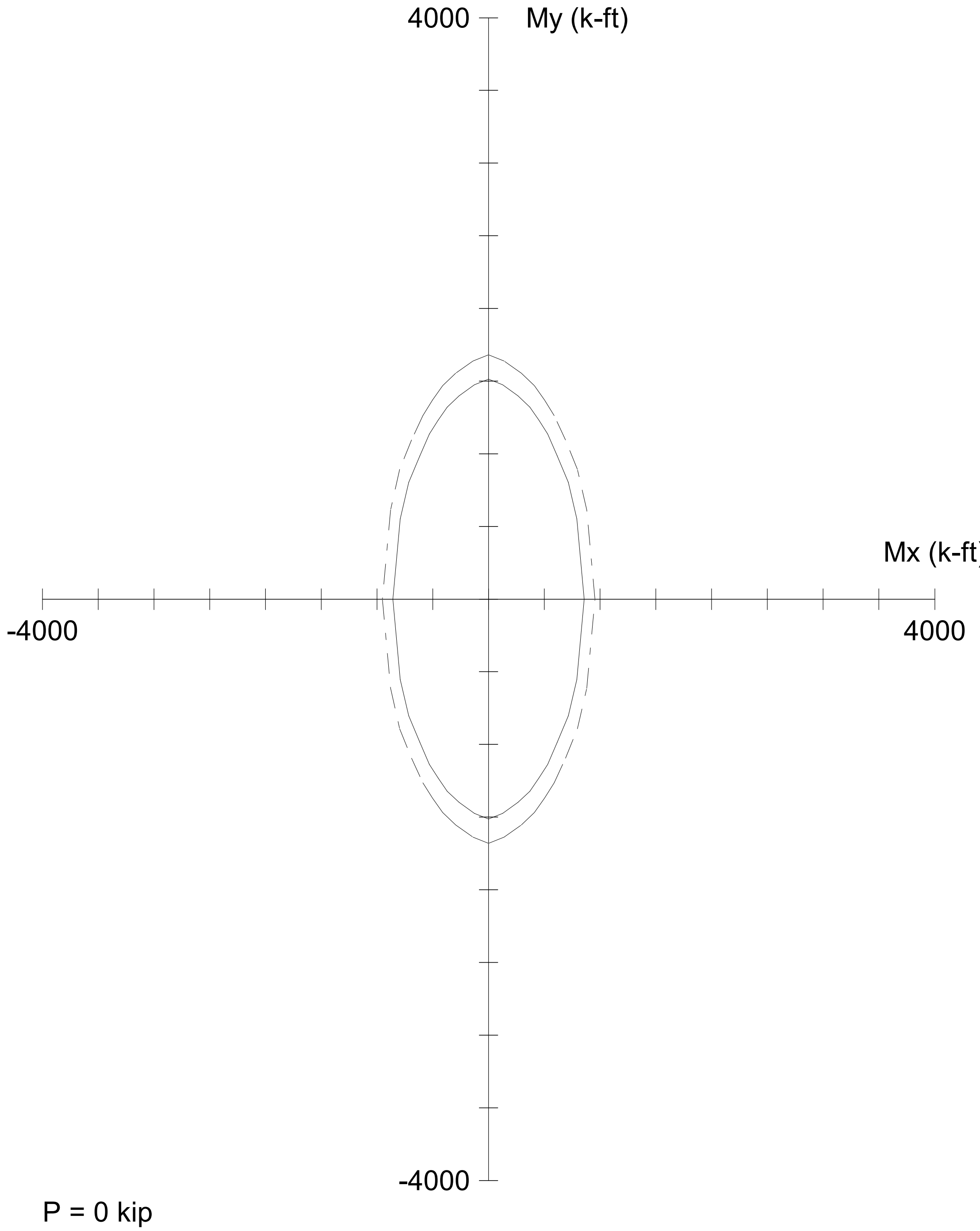
Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 07/20/16

Time: 10:12:15



spColumn v5.00. Licensed to: Leslie E. Robertson Associates. License ID: 64917-1050653-4-2315A-2607D

File: C:\Projects\P993 281 5th Ave Peer Review\Calculation\column\sp column\COLUMN 11-L24.col

Project:

Column: Engineer:

f'c = 12 ksi	fy = 75 ksi	Ag = 680 in^2	12 #11 bars
Ec = 6244 ksi	Es = 29000 ksi	As = 18.72 in^2	rho = 2.75%
fc = 10.2 ksi	e_yt = 0.00258621 in/in	Xo = 0.00 in	Ix = 22666.7 in^4
e_u = 0.003 in/in		Yo = 0.00 in	Iy = 65506.7 in^4
Beta1 = 0.65		Min clear spacing = 5.74 in	Clear cover = 2.00 in

Confinement: Tied

phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65

```
                           oooooo                o
                          oo    oo              oo
   ooooo     oooooo       oo          ooooo     oo       oo   oo   o oooooooooo     o ooooo
  oo    o    oo   oo      oo         oo   oo    oo       oo   oo     oo   oo   oo    oo   oo
  oo         oo   oo      oo         oo   oo    oo       oo   oo     oo   oo   oo    oo   oo
   ooooo     oo   oo      oo         oo   oo    oo       oo   oo     oo   oo   oo    oo   oo
       oo    oooooo       oo         oo   oo    oo       oo   oo     oo   oo   oo    oo   oo
  o    oo    oo           oo    oo   oo   oo    oo  o    oo   oo     oo   oo   oo    oo   oo
   ooooo     oo            oooooo     ooooo      ooo      ooooo o    oo   oo   oo    oo   oo (TM)
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spColumn v5.00 (TM)

Computer program for the Strength Design of Reinforced Concrete Sections

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General Information:

File Name: C:\Projects\P993 281 5th Ave Peer Review\Calculation\column\sp column\COLUMN 11-L24.col
Project:
Column: Engineer:
Code: ACI 318-08 Units: English

Run Option: Investigation Slenderness: Not considered
Run Axis: Biaxial Column Type: Structural

Material Properties:

Concrete: Standard
f'c = 12 ksi
Ec = 6244.04 ksi
fc = 10.2 ksi
Eps_u = 0.003 in/in
Beta1 = 0.65

Steel: Standard
fy = 75 ksi
Es = 29000 ksi
Eps_yt = 0.00258621 in/in

Section:

Rectangular: Width = 34 in Depth = 20 in

Gross section area, Ag = 680 in^2
Ix = 22666.7 in^4 Iy = 65506.7 in^4
rx = 5.7735 in ry = 9.81495 in
Xo = 0 in Yo = 0 in

Reinforcement:

Bar Set: ASTM A615

Size	Diam (in)	Area (in^2)	Size	Diam (in)	Area (in^2)	Size	Diam (in)	Area (in^2)
# 3	0.38	0.11	# 4	0.50	0.20	# 5	0.63	0.31
# 6	0.75	0.44	# 7	0.88	0.60	# 8	1.00	0.79
# 9	1.13	1.00	# 10	1.27	1.27	# 11	1.41	1.56
# 14	1.69	2.25	# 18	2.26	4.00			

Confinement: Tied; #3 ties with #10 bars, #4 with larger bars.
phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65

Layout: Rectangular
Pattern: Sides Different (Cover to transverse reinforcement)
Total steel area: As = 18.72 in^2 at rho = 2.75%
Minimum clear spacing = 5.74 in

	Top	Bottom	Left	Right
Bars	5 #11	5 #11	1 #11	1 #11
Cover(in)	1.5	1.5	1.5	1.5

Service Loads:

No.	Load Case	Axial Load kip	Mx @ Top k-ft	Mx @ Bot k-ft	My @ Top k-ft	My @ Bot k-ft
1	Dead	2229.00	189.00	0.00	465.00	0.00
	Live	0.00	0.00	0.00	0.00	0.00
	Wind	0.00	0.00	0.00	0.00	0.00
	EQ	0.00	0.00	0.00	0.00	0.00
	Snow	0.00	0.00	0.00	0.00	0.00
2	Dead	-832.00	171.00	0.00	500.00	0.00
	Live	0.00	0.00	0.00	0.00	0.00
	Wind	0.00	0.00	0.00	0.00	0.00
	EQ	0.00	0.00	0.00	0.00	0.00
	Snow	0.00	0.00	0.00	0.00	0.00

Sustained Load Factors:

Load Case	Factor (%)
Dead	100
Live	0
Wind	0
EQ	0
Snow	0

Load Combinations:

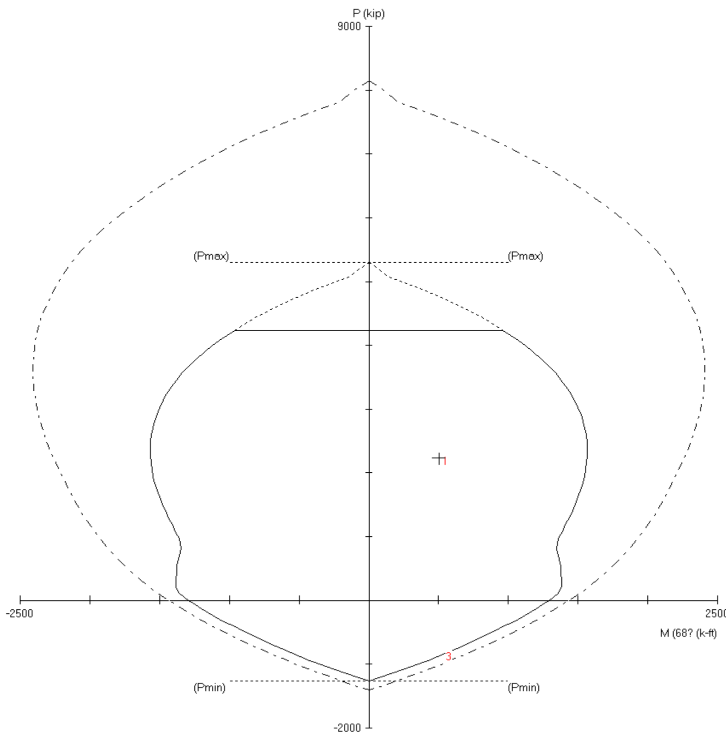
U1 = 1.000*Dead + 1.000*Live + 1.000*Wind + 1.000*EarthQuake + 1.000*Snow

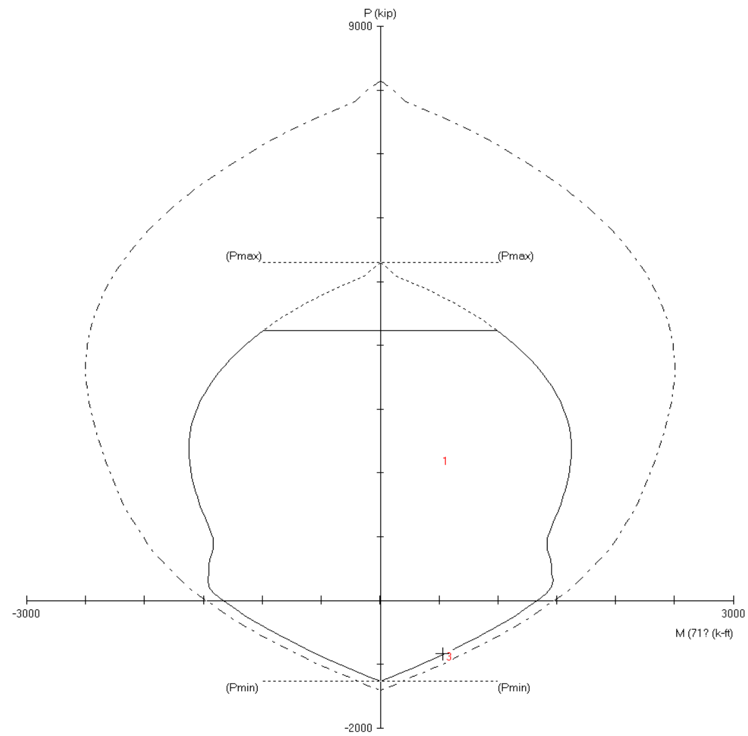
Factored Loads and Moments with Corresponding Capacities:

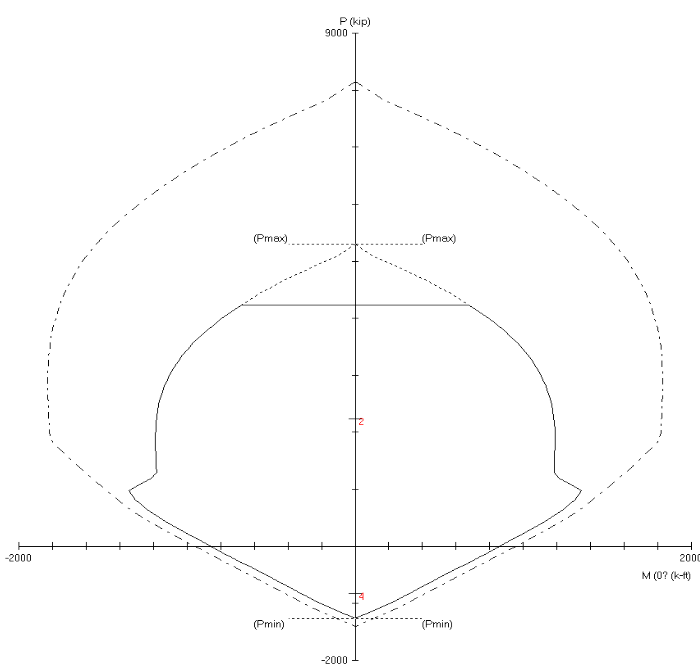
NOTE: Each loading combination includes the following cases:
First line - at column top
Second line - at column bottom

No.	Load Combo	Pu kip	Mux k-ft	Muy k-ft	PhiMnx k-ft	PhiMny k-ft	PhiMn/Mu	NA depth in	Dt depth in	eps_t	Phi
1	1 U1	2229.00	189.00	465.00	588.74	1448.49	3.115	26.00	32.69	0.00077	0.650
2		2229.00	-0.00	-0.00	1182.57	0.00	999.999	13.64	17.30	0.00080	0.650
3	2 U1	-832.00	171.00	500.00	179.59	525.10	1.050	5.63	35.50	0.01603	0.900
4		-832.00	-0.00	-0.00	321.56	0.00	999.999	1.60	17.30	0.02937	0.900

*** End of output ***







Appendix C Shear Wall Flexural Capacity Check Calculation

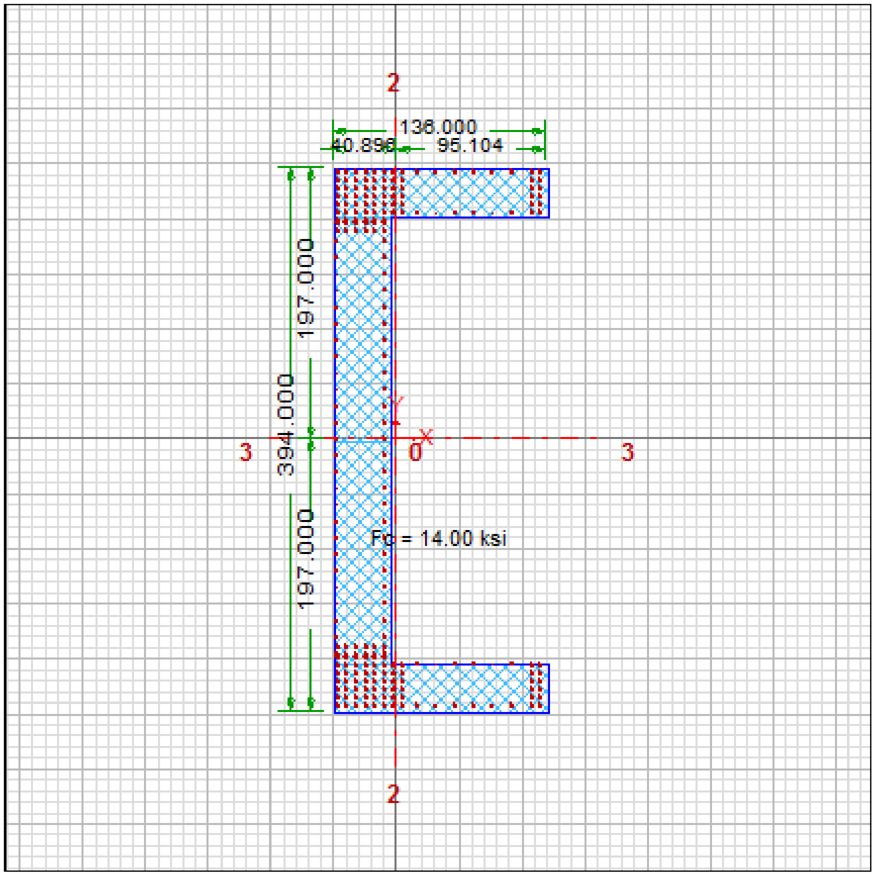
Project Information

Project	
Job No	
Company	
Designer	
Remarks	
Software	CSICOL (Version: 9.0 (Rev. 1))
File Name	C:\Projects\P993 281 5th Ave Peer Review\Calculation\shear wall \L1_SW-01-distributed rebar
Working Units	US (in, kip, k-ft, ksi)
Design Code	ACI-318-11

Column:SW-01

Basic Design Parameters

Caption	= SW-01	
Default Concrete Strength, Fc	= 14.00	ksi
Default Concrete Modulus, Ec	= 6744.00	ksi
Maximum Concrete Strain	= .003	in/in
Rebar Set	= ASTM	
Default Rebar Yeild Strength, Fy	= 60.00	ksi
Default Rebar Modulus, Es	= 28985.51	ksi
Default Cover to Rebars	= 1.500	in
Maximum Steel Strain	= Infinity	
Transverse Rebar Type	= Ties	
Total Shapes in Section	= 1	
Consider Slenderness	= No	



Section Diagram

Cross-section Shapes

Shape	Width in	Height in	Conc Fc ksi	S/S Curve	Rebars
Channel Shape	136.000	394.000	14.000	ACI-Whitney Rectangular	77-#7+144-#11

Rebar Properties

Sr.No	Designation	Area in^2	Cord-X in	Cord-Y in	Fy ksi	S/S Curve
1	#11	1.56	33.000	3.000	60.00	Elasto-Plastic
2	#11	1.56	27.000	3.000	60.00	Elasto-Plastic
3	#11	1.56	21.000	3.000	60.00	Elasto-Plastic
4	#11	1.56	15.000	3.000	60.00	Elasto-Plastic
5	#11	1.56	9.000	3.000	60.00	Elasto-Plastic
6	#11	1.56	3.000	3.000	60.00	Elasto-Plastic
7	#11	1.56	33.000	9.000	60.00	Elasto-Plastic
8	#11	1.56	27.000	9.000	60.00	Elasto-Plastic
9	#11	1.56	21.000	9.000	60.00	Elasto-Plastic
10	#11	1.56	15.000	9.000	60.00	Elasto-Plastic
11	#11	1.56	9.000	9.000	60.00	Elasto-Plastic
12	#11	1.56	3.000	9.000	60.00	Elasto-Plastic
13	#11	1.56	33.000	15.000	60.00	Elasto-Plastic
14	#11	1.56	27.000	15.000	60.00	Elasto-Plastic
15	#11	1.56	21.000	15.000	60.00	Elasto-Plastic
16	#11	1.56	15.000	15.000	60.00	Elasto-Plastic
17	#11	1.56	9.000	15.000	60.00	Elasto-Plastic
18	#11	1.56	3.000	15.000	60.00	Elasto-Plastic
19	#11	1.56	33.000	21.000	60.00	Elasto-Plastic
20	#11	1.56	27.000	21.000	60.00	Elasto-Plastic
21	#11	1.56	21.000	21.000	60.00	Elasto-Plastic
22	#11	1.56	15.000	21.000	60.00	Elasto-Plastic
23	#11	1.56	9.000	21.000	60.00	Elasto-Plastic

24	#11	1.56	3.000	21.000	60.00	Elasto-Plastic
25	#11	1.56	33.000	27.000	60.00	Elasto-Plastic
26	#11	1.56	27.000	27.000	60.00	Elasto-Plastic
27	#11	1.56	21.000	27.000	60.00	Elasto-Plastic
28	#11	1.56	15.000	27.000	60.00	Elasto-Plastic
29	#11	1.56	9.000	27.000	60.00	Elasto-Plastic
30	#11	1.56	3.000	27.000	60.00	Elasto-Plastic
31	#11	1.56	33.000	33.000	60.00	Elasto-Plastic
32	#11	1.56	27.000	33.000	60.00	Elasto-Plastic
33	#11	1.56	21.000	33.000	60.00	Elasto-Plastic
34	#11	1.56	15.000	33.000	60.00	Elasto-Plastic
35	#11	1.56	9.000	33.000	60.00	Elasto-Plastic
36	#11	1.56	3.000	33.000	60.00	Elasto-Plastic
37	#11	1.56	33.000	39.000	60.00	Elasto-Plastic
38	#11	1.56	27.000	39.000	60.00	Elasto-Plastic
39	#11	1.56	21.000	39.000	60.00	Elasto-Plastic
40	#11	1.56	15.000	39.000	60.00	Elasto-Plastic
41	#11	1.56	9.000	39.000	60.00	Elasto-Plastic
42	#11	1.56	3.000	39.000	60.00	Elasto-Plastic
43	#11	1.56	33.000	45.000	60.00	Elasto-Plastic
44	#11	1.56	27.000	45.000	60.00	Elasto-Plastic
45	#11	1.56	21.000	45.000	60.00	Elasto-Plastic
46	#11	1.56	15.000	45.000	60.00	Elasto-Plastic
47	#11	1.56	9.000	45.000	60.00	Elasto-Plastic
48	#11	1.56	3.000	45.000	60.00	Elasto-Plastic
49	#11	1.56	39.000	3.000	60.00	Elasto-Plastic
50	#11	1.56	39.000	9.000	60.00	Elasto-Plastic
51	#11	1.56	39.000	15.000	60.00	Elasto-Plastic
52	#11	1.56	39.000	21.000	60.00	Elasto-Plastic
53	#11	1.56	39.000	27.000	60.00	Elasto-Plastic
54	#11	1.56	39.000	33.000	60.00	Elasto-Plastic
55	#11	1.56	45.000	3.000	60.00	Elasto-Plastic
56	#11	1.56	45.000	9.000	60.00	Elasto-Plastic
57	#11	1.56	45.000	15.000	60.00	Elasto-Plastic
58	#11	1.56	45.000	21.000	60.00	Elasto-Plastic
59	#11	1.56	45.000	27.000	60.00	Elasto-Plastic
60	#11	1.56	45.000	33.000	60.00	Elasto-Plastic
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62	#11	1.56	133.000	9.000	60.00	Elasto-Plastic
63	#11	1.56	133.000	15.000	60.00	Elasto-Plastic
64	#11	1.56	133.000	21.000	60.00	Elasto-Plastic
65	#11	1.56	133.000	27.000	60.00	Elasto-Plastic
66	#11	1.56	133.000	33.000	60.00	Elasto-Plastic
67	#11	1.56	127.000	3.000	60.00	Elasto-Plastic
68	#11	1.56	127.000	9.000	60.00	Elasto-Plastic
69	#11	1.56	127.000	15.000	60.00	Elasto-Plastic
70	#11	1.56	127.000	21.000	60.00	Elasto-Plastic
71	#11	1.56	127.000	27.000	60.00	Elasto-Plastic
72	#11	1.56	127.000	33.000	60.00	Elasto-Plastic
73	#7	0.60	54.000	3.000	60.00	Elasto-Plastic
74	#11	1.56	33.000	391.000	60.00	Elasto-Plastic
75	#11	1.56	27.000	391.000	60.00	Elasto-Plastic
76	#11	1.56	21.000	391.000	60.00	Elasto-Plastic
77	#11	1.56	15.000	391.000	60.00	Elasto-Plastic
78	#11	1.56	9.000	391.000	60.00	Elasto-Plastic
79	#11	1.56	3.000	391.000	60.00	Elasto-Plastic
80	#11	1.56	33.000	385.000	60.00	Elasto-Plastic
81	#11	1.56	27.000	385.000	60.00	Elasto-Plastic
82	#11	1.56	21.000	385.000	60.00	Elasto-Plastic
83	#11	1.56	15.000	385.000	60.00	Elasto-Plastic
84	#11	1.56	9.000	385.000	60.00	Elasto-Plastic
85	#11	1.56	3.000	385.000	60.00	Elasto-Plastic
86	#11	1.56	33.000	379.000	60.00	Elasto-Plastic
87	#11	1.56	27.000	379.000	60.00	Elasto-Plastic
88	#11	1.56	21.000	379.000	60.00	Elasto-Plastic
89	#11	1.56	15.000	379.000	60.00	Elasto-Plastic
90	#11	1.56	9.000	379.000	60.00	Elasto-Plastic
91	#11	1.56	3.000	379.000	60.00	Elasto-Plastic
92	#11	1.56	33.000	373.000	60.00	Elasto-Plastic
93	#11	1.56	27.000	373.000	60.00	Elasto-Plastic
94	#11	1.56	21.000	373.000	60.00	Elasto-Plastic
95	#11	1.56	15.000	373.000	60.00	Elasto-Plastic

96	#11	1.56	9.000	373.000	60.00	Elasto-Plastic
97	#11	1.56	3.000	373.000	60.00	Elasto-Plastic
98	#11	1.56	33.000	367.000	60.00	Elasto-Plastic
99	#11	1.56	27.000	367.000	60.00	Elasto-Plastic
100	#11	1.56	21.000	367.000	60.00	Elasto-Plastic
101	#11	1.56	15.000	367.000	60.00	Elasto-Plastic
102	#11	1.56	9.000	367.000	60.00	Elasto-Plastic
103	#11	1.56	3.000	367.000	60.00	Elasto-Plastic
104	#11	1.56	33.000	361.000	60.00	Elasto-Plastic
105	#11	1.56	27.000	361.000	60.00	Elasto-Plastic
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111	#11	1.56	27.000	355.000	60.00	Elasto-Plastic
112	#11	1.56	21.000	355.000	60.00	Elasto-Plastic
113	#11	1.56	15.000	355.000	60.00	Elasto-Plastic
114	#11	1.56	9.000	355.000	60.00	Elasto-Plastic
115	#11	1.56	3.000	355.000	60.00	Elasto-Plastic
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117	#11	1.56	27.000	349.000	60.00	Elasto-Plastic
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119	#11	1.56	15.000	349.000	60.00	Elasto-Plastic
120	#11	1.56	9.000	349.000	60.00	Elasto-Plastic
121	#11	1.56	3.000	349.000	60.00	Elasto-Plastic
122	#11	1.56	133.000	391.000	60.00	Elasto-Plastic
123	#11	1.56	133.000	385.000	60.00	Elasto-Plastic
124	#11	1.56	133.000	379.000	60.00	Elasto-Plastic
125	#11	1.56	133.000	373.000	60.00	Elasto-Plastic
126	#11	1.56	133.000	367.000	60.00	Elasto-Plastic
127	#11	1.56	133.000	361.000	60.00	Elasto-Plastic
128	#11	1.56	127.000	391.000	60.00	Elasto-Plastic
129	#11	1.56	127.000	385.000	60.00	Elasto-Plastic
130	#11	1.56	127.000	379.000	60.00	Elasto-Plastic
131	#11	1.56	127.000	373.000	60.00	Elasto-Plastic
132	#11	1.56	127.000	367.000	60.00	Elasto-Plastic
133	#11	1.56	127.000	361.000	60.00	Elasto-Plastic
134	#7	0.60	54.000	3.000	60.00	Elasto-Plastic
135	#7	0.60	66.000	3.000	60.00	Elasto-Plastic
136	#7	0.60	78.000	3.000	60.00	Elasto-Plastic
137	#7	0.60	90.000	3.000	60.00	Elasto-Plastic
138	#7	0.60	102.000	3.000	60.00	Elasto-Plastic
139	#7	0.60	114.000	3.000	60.00	Elasto-Plastic
140	#7	0.60	54.000	33.000	60.00	Elasto-Plastic
141	#7	0.60	66.000	33.000	60.00	Elasto-Plastic
142	#7	0.60	78.000	33.000	60.00	Elasto-Plastic
143	#7	0.60	90.000	33.000	60.00	Elasto-Plastic
144	#7	0.60	102.000	33.000	60.00	Elasto-Plastic
145	#7	0.60	114.000	33.000	60.00	Elasto-Plastic
146	#7	0.60	54.000	391.000	60.00	Elasto-Plastic
147	#7	0.60	66.000	391.000	60.00	Elasto-Plastic
148	#7	0.60	78.000	391.000	60.00	Elasto-Plastic
149	#7	0.60	90.000	391.000	60.00	Elasto-Plastic
150	#7	0.60	102.000	391.000	60.00	Elasto-Plastic
151	#7	0.60	114.000	391.000	60.00	Elasto-Plastic
152	#7	0.60	54.000	361.000	60.00	Elasto-Plastic
153	#7	0.60	66.000	361.000	60.00	Elasto-Plastic
154	#7	0.60	78.000	361.000	60.00	Elasto-Plastic
155	#7	0.60	90.000	361.000	60.00	Elasto-Plastic
156	#7	0.60	102.000	361.000	60.00	Elasto-Plastic
157	#7	0.60	114.000	361.000	60.00	Elasto-Plastic
158	#7	0.60	3.000	41.000	60.00	Elasto-Plastic
159	#7	0.60	3.000	53.000	60.00	Elasto-Plastic
160	#7	0.60	3.000	65.000	60.00	Elasto-Plastic
161	#7	0.60	3.000	77.000	60.00	Elasto-Plastic
162	#7	0.60	3.000	89.000	60.00	Elasto-Plastic
163	#7	0.60	3.000	101.000	60.00	Elasto-Plastic
164	#7	0.60	3.000	113.000	60.00	Elasto-Plastic
165	#7	0.60	3.000	125.000	60.00	Elasto-Plastic
166	#7	0.60	3.000	137.000	60.00	Elasto-Plastic
167	#7	0.60	3.000	149.000	60.00	Elasto-Plastic

168	#7	0.60	3.000	161.000	60.00	Elasto-Plastic
169	#7	0.60	3.000	173.000	60.00	Elasto-Plastic
170	#7	0.60	3.000	185.000	60.00	Elasto-Plastic
171	#7	0.60	3.000	197.000	60.00	Elasto-Plastic
172	#7	0.60	3.000	209.000	60.00	Elasto-Plastic
173	#7	0.60	3.000	221.000	60.00	Elasto-Plastic
174	#7	0.60	3.000	233.000	60.00	Elasto-Plastic
175	#7	0.60	3.000	245.000	60.00	Elasto-Plastic
176	#7	0.60	3.000	257.000	60.00	Elasto-Plastic
177	#7	0.60	3.000	269.000	60.00	Elasto-Plastic
178	#7	0.60	3.000	281.000	60.00	Elasto-Plastic
179	#7	0.60	3.000	293.000	60.00	Elasto-Plastic
180	#7	0.60	3.000	305.000	60.00	Elasto-Plastic
181	#7	0.60	3.000	317.000	60.00	Elasto-Plastic
182	#7	0.60	3.000	329.000	60.00	Elasto-Plastic
183	#7	0.60	3.000	341.000	60.00	Elasto-Plastic
184	#7	0.60	33.000	41.000	60.00	Elasto-Plastic
185	#7	0.60	33.000	53.000	60.00	Elasto-Plastic
186	#7	0.60	33.000	65.000	60.00	Elasto-Plastic
187	#7	0.60	33.000	77.000	60.00	Elasto-Plastic
188	#7	0.60	33.000	89.000	60.00	Elasto-Plastic
189	#7	0.60	33.000	101.000	60.00	Elasto-Plastic
190	#7	0.60	33.000	113.000	60.00	Elasto-Plastic
191	#7	0.60	33.000	125.000	60.00	Elasto-Plastic
192	#7	0.60	33.000	137.000	60.00	Elasto-Plastic
193	#7	0.60	33.000	149.000	60.00	Elasto-Plastic
194	#7	0.60	33.000	161.000	60.00	Elasto-Plastic
195	#7	0.60	33.000	173.000	60.00	Elasto-Plastic
196	#7	0.60	33.000	185.000	60.00	Elasto-Plastic
197	#7	0.60	33.000	197.000	60.00	Elasto-Plastic
198	#7	0.60	33.000	209.000	60.00	Elasto-Plastic
199	#7	0.60	33.000	221.000	60.00	Elasto-Plastic
200	#7	0.60	33.000	233.000	60.00	Elasto-Plastic
201	#7	0.60	33.000	245.000	60.00	Elasto-Plastic
202	#7	0.60	33.000	257.000	60.00	Elasto-Plastic
203	#7	0.60	33.000	269.000	60.00	Elasto-Plastic
204	#7	0.60	33.000	281.000	60.00	Elasto-Plastic
205	#7	0.60	33.000	293.000	60.00	Elasto-Plastic
206	#7	0.60	33.000	305.000	60.00	Elasto-Plastic
207	#7	0.60	33.000	317.000	60.00	Elasto-Plastic
208	#7	0.60	33.000	329.000	60.00	Elasto-Plastic
209	#7	0.60	33.000	341.000	60.00	Elasto-Plastic
210	#11	1.56	39.000	391.000	60.00	Elasto-Plastic
211	#11	1.56	39.000	385.000	60.00	Elasto-Plastic
212	#11	1.56	39.000	379.000	60.00	Elasto-Plastic
213	#11	1.56	39.000	373.000	60.00	Elasto-Plastic
214	#11	1.56	39.000	367.000	60.00	Elasto-Plastic
215	#11	1.56	39.000	361.000	60.00	Elasto-Plastic
216	#11	1.56	45.000	391.000	60.00	Elasto-Plastic
217	#11	1.56	45.000	385.000	60.00	Elasto-Plastic
218	#11	1.56	45.000	379.000	60.00	Elasto-Plastic
219	#11	1.56	45.000	373.000	60.00	Elasto-Plastic
220	#11	1.56	45.000	367.000	60.00	Elasto-Plastic
221	#11	1.56	45.000	361.000	60.00	Elasto-Plastic

77-#7+144-#11		
Total Area =	270.73	in^2
Steel Ratio =	1.27	%

Basic Section Properties:

Total Width	= 136.00	in
Total Height	= 394.00	in
Center, Xo	= 0.00	in
Center, Yo	= 0.00	in
X-bar (Right)	= 95.10	in
X-bar (Left)	= 40.90	in
Y-bar (Top)	= 197.00	in
Y-bar (Bot)	= 197.00	in

Transformed Properties:		
Base Material	= Custom	
Area, A	= 2.14E+04	in^2
Inertia, I33	= 4.15E+08	in^4
Inertia, I22	= 2.96E+07	in^4
Inertia, I32	= 0.00E+00	in^4
Radius, r3	= 139.30	in
Radius, r2	= 37.21	in

Additional Section Properties:

Transformed Properties:		
Base Material	= Custom	
Modulus, S3(Top)	= 2.11E+06	in^3
Modulus, S3(Bot)	= 2.11E+06	in^3
Modulus, S2(Left)	= 7.24E+05	in^3
Modulus, S2(Right)	= 3.11E+05	in^3
Plastic Modulus, Z3	= 3.02E+06	in^3
Plastic Modulus, Z2	= 6.38E+05	in^3
Torsional, J	= 8.58E+06	in^4
Shear Area, A3	= 1.07E+04	in^2
Shear Area, A2	= 1.38E+04	in^2
Principal Angle	= 0.00E+00	Deg
Inertia, I33'	= 4.15E+08	in^4
Inertia, I22'	= 2.96E+07	in^4

Framing Along-X

Total C/C Length, Lc	= 12.000	ft
Unsupported Length, Lu	= 10.000	ft
Framing Type	= 4	
Framing Case	= 0	
K Factor, Braced	= 1.00	
Kl/r, Braced	= 3.22	
K Factor, Unbraced	= 1.00	
Kl/r, Unbraced	= 3.22	

Framing Along-Y

Total C/C Length, Lc	= 12.000	ft
Unsupported Length, Lu	= 10.000	ft
Framing Type	= 4	
Framing Case	= 0	
K Factor, Braced	= 1.00	
Kl/r, Braced	= .86	
K Factor, Unbraced	= 1.00	
Kl/r, Unbraced	= .86	

Final Design Loads

Sr.No	Combination	Load Pu kip	Mux-Bot k-ft	Muy-Bot k-ft	Mux-Top k-ft	Muy-Top k-ft
1	14D	3.01E+04	2.55E+03	1.34E+03	9.35E+03	2.42E+03
2	12D16L	3.01E+04	1.97E+03	1.37E+03	8.74E+03	2.50E+03
3	09D16WT MAX	-326.32	107,969.10	19,463.80	100,806.10	12,381.80
4	09D16WT MIN	3.91E+04	-1.05E+05	-1.78E+04	-8.88E+04	-9.31E+03
5	12D10L16WT MAX	8,774.7	108,381.3	19,886.9	103,266.8	13,166.9
6	12D10L16WT MIN	4.82E+04	-1.04E+05	-1.74E+04	-8.63E+04	-8.53E+03

7	12D10L10EQP MAX	2.85E+04	3.67E+04	6.19E+03	3.98E+04	5.16E+03
8	12D10L10EQP MIN	3.38E+04	-1.75E+03	9.74E+02	6.07E+03	1.88E+03
9	12D10L10EQN MAX	2.32E+04	5.85E+03	1.59E+03	1.09E+04	2.80E+03
10	12D10L10EQN MIN	2.85E+04	-3.26E+04	-3.62E+03	-2.29E+04	-4.82E+02

Result Summary

Sr.No	Combination	Pu (kip)	Cap. Ratio-Bot	Cap. Ratio-Top	Remarks
1	14D	3.01E+04	0.217	0.217	Capacity OK
2	12D16L	3.01E+04	0.216	0.216	Capacity OK
3	09D16WT MAX	-326.32	0.572	0.459	Capacity OK
4	09D16WT MIN	3.91E+04	0.281	0.281	Capacity OK
5	12D10L16WT MAX	8,774.7	1.884	0.107	Capacity Not OK
6	12D10L16WT MIN	4.82E+04	0.346	0.346	Capacity OK
7	12D10L10EQP MAX	2.85E+04	0.205	0.205	Capacity OK
8	12D10L10EQP MIN	3.38E+04	0.243	0.243	Capacity OK
9	12D10L10EQN MAX	2.32E+04	0.167	0.167	Capacity OK
10	12D10L10EQN MIN	2.85E+04	0.205	0.205	Capacity OK

Moment Magnification Calculations

14D- Along X

Bracing Condition	= Non-Sway	
Non-Sway Part of Loading:		
Design Load, Pu	= 0.00	kip
Sustained Load, Pud	= 0.00	kip
End Moment, M1	= 0.0	k-ft
End Moment, M2	= 0.0	k-ft
Minimum Moment, Mmin	= 0.0	k-ft
Design Moment, Mc	= 0.0	k-ft
Creep Factor, Bd	= .00	
Section Stiffness, EI For Pcr	= 0.00E+00	k-in^2
Euler Buckling Load, Pcr	= 0.00	kip
Buckling Failure	= Pcr < Pu	

14D- Along Y

Bracing Condition	= Non-Sway	
Non-Sway Part of Loading:		
Design Load, Pu	= 0.00	kip
Sustained Load, Pud	= 0.00	kip
End Moment, M1	= 0.0	k-ft
End Moment, M2	= 0.0	k-ft
Minimum Moment, Mmin	= 0.0	k-ft
Design Moment, Mc	= 0.0	k-ft
Creep Factor, Bd	= .00	
Section Stiffness, EI For Pcr	= 0.00E+00	k-in^2
Euler Buckling Load, Pcr	= 0.00	kip
Buckling Failure	= Pcr < Pu	

12D16L- Along X

Bracing Condition	= Non-Sway	
Non-Sway Part of Loading:		
Design Load, Pu	= 0.00	kip
Sustained Load, Pud	= 0.00	kip
End Moment, M1	= 0.0	k-ft
End Moment, M2	= 0.0	k-ft
Minimum Moment, Mmin	= 0.0	k-ft
Design Moment, Mc	= 0.0	k-ft
Creep Factor, Bd	= .00	
Section Stiffness, EI For Pcr	= 0.00E+00	k-in^2
Euler Buckling Load, Pcr	= 0.00	kip
Buckling Failure	= Pcr < Pu	

12D16L- Along Y

Bracing Condition	= Non-Sway	
Non-Sway Part of Loading:		
Design Load, Pu	= 0.00	kip
Sustained Load, Pud	= 0.00	kip
End Moment, M1	= 0.0	k-ft
End Moment, M2	= 0.0	k-ft
Minimum Moment, Mmin	= 0.0	k-ft
Design Moment, Mc	= 0.0	k-ft
Creep Factor, Bd	= .00	
Section Stiffness, EI For Pcr	= 0.00E+00	k-in^2
Euler Buckling Load, Pcr	= 0.00	kip
Buckling Failure	= Pcr < Pu	

09D16WT MAX- Along X

Bracing Condition	= Non-Sway	
Non-Sway Part of Loading:		
Design Load, Pu	= 0.00	kip
Sustained Load, Pud	= 0.00	kip
End Moment, M1	= 0.0	k-ft
End Moment, M2	= 0.0	k-ft
Minimum Moment, Mmin	= 0.0	k-ft
Design Moment, Mc	= 0.0	k-ft
Creep Factor, Bd	= .00	
Section Stiffness, EI For Pcr	= 0.00E+00	k-in^2
Euler Buckling Load, Pcr	= 0.00	kip
Buckling Failure	= Pcr < Pu	

09D16WT MAX- Along Y

Bracing Condition	= Non-Sway	
Non-Sway Part of Loading:		
Design Load, Pu	= 0.00	kip
Sustained Load, Pud	= 0.00	kip
End Moment, M1	= 0.0	k-ft
End Moment, M2	= 0.0	k-ft
Minimum Moment, Mmin	= 0.0	k-ft
Design Moment, Mc	= 0.0	k-ft
Creep Factor, Bd	= .00	
Section Stiffness, EI For Pcr	= 0.00E+00	k-in^2

Euler Buckling Load, Pcr	= 0.00	kip
Buckling Failure	= Pcr < Pu	

09D16WT MIN- Along X

Bracing Condition	= Non-Sway	
Non-Sway Part of Loading:		
Design Load, Pu	= 0.00	kip
Sustained Load, Pud	= 0.00	kip
End Moment, M1	= 0.0	k-ft
End Moment, M2	= 0.0	k-ft
Minimum Moment, Mmin	= 0.0	k-ft
Design Moment, Mc	= 0.0	k-ft
Creep Factor, Bd	= .00	
Section Stiffness, EI For Pcr	= 0.00E+00	k-in^2
Euler Buckling Load, Pcr	= 0.00	kip
Buckling Failure	= Pcr < Pu	

09D16WT MIN- Along Y

Bracing Condition	= Non-Sway	
Non-Sway Part of Loading:		
Design Load, Pu	= 0.00	kip
Sustained Load, Pud	= 0.00	kip
End Moment, M1	= 0.0	k-ft
End Moment, M2	= 0.0	k-ft
Minimum Moment, Mmin	= 0.0	k-ft
Design Moment, Mc	= 0.0	k-ft
Creep Factor, Bd	= .00	
Section Stiffness, EI For Pcr	= 0.00E+00	k-in^2
Euler Buckling Load, Pcr	= 0.00	kip
Buckling Failure	= Pcr < Pu	

12D10L16WT MAX- Along X

Bracing Condition	= Non-Sway	
Non-Sway Part of Loading:		
Design Load, Pu	= 0.00	kip
Sustained Load, Pud	= 0.00	kip
End Moment, M1	= 0.0	k-ft
End Moment, M2	= 0.0	k-ft
Minimum Moment, Mmin	= 0.0	k-ft
Design Moment, Mc	= 0.0	k-ft
Creep Factor, Bd	= .00	
Section Stiffness, EI For Pcr	= 0.00E+00	k-in^2
Euler Buckling Load, Pcr	= 0.00	kip
Buckling Failure	= Pcr < Pu	

12D10L16WT MAX- Along Y

Bracing Condition	= Non-Sway	
Non-Sway Part of Loading:		
Design Load, Pu	= 0.00	kip
Sustained Load, Pud	= 0.00	kip
End Moment, M1	= 0.0	k-ft
End Moment, M2	= 0.0	k-ft

Minimum Moment, Mmin	= 0.0	k-ft
Design Moment, Mc	= 0.0	k-ft
Creep Factor, Bd	= .00	
Section Stiffness, EI For Pcr	= 0.00E+00	k-in^2
Euler Buckling Load, Pcr	= 0.00	kip
Buckling Failure	= Pcr < Pu	

12D10L16WT MIN- Along X

Bracing Condition	= Non-Sway	
Non-Sway Part of Loading:		
Design Load, Pu	= 0.00	kip
Sustained Load, Pud	= 0.00	kip
End Moment, M1	= 0.0	k-ft
End Moment, M2	= 0.0	k-ft
Minimum Moment, Mmin	= 0.0	k-ft
Design Moment, Mc	= 0.0	k-ft
Creep Factor, Bd	= .00	
Section Stiffness, EI For Pcr	= 0.00E+00	k-in^2
Euler Buckling Load, Pcr	= 0.00	kip
Buckling Failure	= Pcr < Pu	

12D10L16WT MIN- Along Y

Bracing Condition	= Non-Sway	
Non-Sway Part of Loading:		
Design Load, Pu	= 0.00	kip
Sustained Load, Pud	= 0.00	kip
End Moment, M1	= 0.0	k-ft
End Moment, M2	= 0.0	k-ft
Minimum Moment, Mmin	= 0.0	k-ft
Design Moment, Mc	= 0.0	k-ft
Creep Factor, Bd	= .00	
Section Stiffness, EI For Pcr	= 0.00E+00	k-in^2
Euler Buckling Load, Pcr	= 0.00	kip
Buckling Failure	= Pcr < Pu	

12D10L10EQP MAX- Along X

Bracing Condition	= Non-Sway	
Non-Sway Part of Loading:		
Design Load, Pu	= 0.00	kip
Sustained Load, Pud	= 0.00	kip
End Moment, M1	= 0.0	k-ft
End Moment, M2	= 0.0	k-ft
Minimum Moment, Mmin	= 0.0	k-ft
Design Moment, Mc	= 0.0	k-ft
Creep Factor, Bd	= .00	
Section Stiffness, EI For Pcr	= 0.00E+00	k-in^2
Euler Buckling Load, Pcr	= 0.00	kip
Buckling Failure	= Pcr < Pu	

12D10L10EQP MAX- Along Y

Bracing Condition	= Non-Sway

Non-Sway Part of Loading:		
Design Load, Pu	= 0.00	kip
Sustained Load, Pud	= 0.00	kip
End Moment, M1	= 0.0	k-ft
End Moment, M2	= 0.0	k-ft
Minimum Moment, Mmin	= 0.0	k-ft
Design Moment, Mc	= 0.0	k-ft
Creep Factor, Bd	= .00	
Section Stiffness, EI For Pcr	= 0.00E+00	k-in^2
Euler Buckling Load, Pcr	= 0.00	kip
Buckling Failure	= Pcr < Pu	

12D10L10EQP MIN- Along X

Bracing Condition	= Non-Sway	
Non-Sway Part of Loading:		
Design Load, Pu	= 0.00	kip
Sustained Load, Pud	= 0.00	kip
End Moment, M1	= 0.0	k-ft
End Moment, M2	= 0.0	k-ft
Minimum Moment, Mmin	= 0.0	k-ft
Design Moment, Mc	= 0.0	k-ft
Creep Factor, Bd	= .00	
Section Stiffness, EI For Pcr	= 0.00E+00	k-in^2
Euler Buckling Load, Pcr	= 0.00	kip
Buckling Failure	= Pcr < Pu	

12D10L10EQP MIN- Along Y

Bracing Condition	= Non-Sway	
Non-Sway Part of Loading:		
Design Load, Pu	= 0.00	kip
Sustained Load, Pud	= 0.00	kip
End Moment, M1	= 0.0	k-ft
End Moment, M2	= 0.0	k-ft
Minimum Moment, Mmin	= 0.0	k-ft
Design Moment, Mc	= 0.0	k-ft
Creep Factor, Bd	= .00	
Section Stiffness, EI For Pcr	= 0.00E+00	k-in^2
Euler Buckling Load, Pcr	= 0.00	kip
Buckling Failure	= Pcr < Pu	

12D10L10EQN MAX- Along X

Bracing Condition	= Non-Sway	
Non-Sway Part of Loading:		
Design Load, Pu	= 0.00	kip
Sustained Load, Pud	= 0.00	kip
End Moment, M1	= 0.0	k-ft
End Moment, M2	= 0.0	k-ft
Minimum Moment, Mmin	= 0.0	k-ft
Design Moment, Mc	= 0.0	k-ft
Creep Factor, Bd	= .00	
Section Stiffness, EI For Pcr	= 0.00E+00	k-in^2
Euler Buckling Load, Pcr	= 0.00	kip
Buckling Failure	= Pcr < Pu	

12D10L10EQN MAX- Along Y

Bracing Condition	= Non-Sway	
Non-Sway Part of Loading:		
Design Load, Pu	= 0.00	kip
Sustained Load, Pud	= 0.00	kip
End Moment, M1	= 0.0	k-ft
End Moment, M2	= 0.0	k-ft
Minimum Moment, Mmin	= 0.0	k-ft
Design Moment, Mc	= 0.0	k-ft
Creep Factor, Bd	= .00	
Section Stiffness, EI For Pcr	= 0.00E+00	k-in^2
Euler Buckling Load, Pcr	= 0.00	kip
Buckling Failure	= Pcr < Pu	

12D10L10EQN MIN- Along X

Bracing Condition	= Non-Sway	
Non-Sway Part of Loading:		
Design Load, Pu	= 0.00	kip
Sustained Load, Pud	= 0.00	kip
End Moment, M1	= 0.0	k-ft
End Moment, M2	= 0.0	k-ft
Minimum Moment, Mmin	= 0.0	k-ft
Design Moment, Mc	= 0.0	k-ft
Creep Factor, Bd	= .00	
Section Stiffness, EI For Pcr	= 0.00E+00	k-in^2
Euler Buckling Load, Pcr	= 0.00	kip
Buckling Failure	= Pcr < Pu	

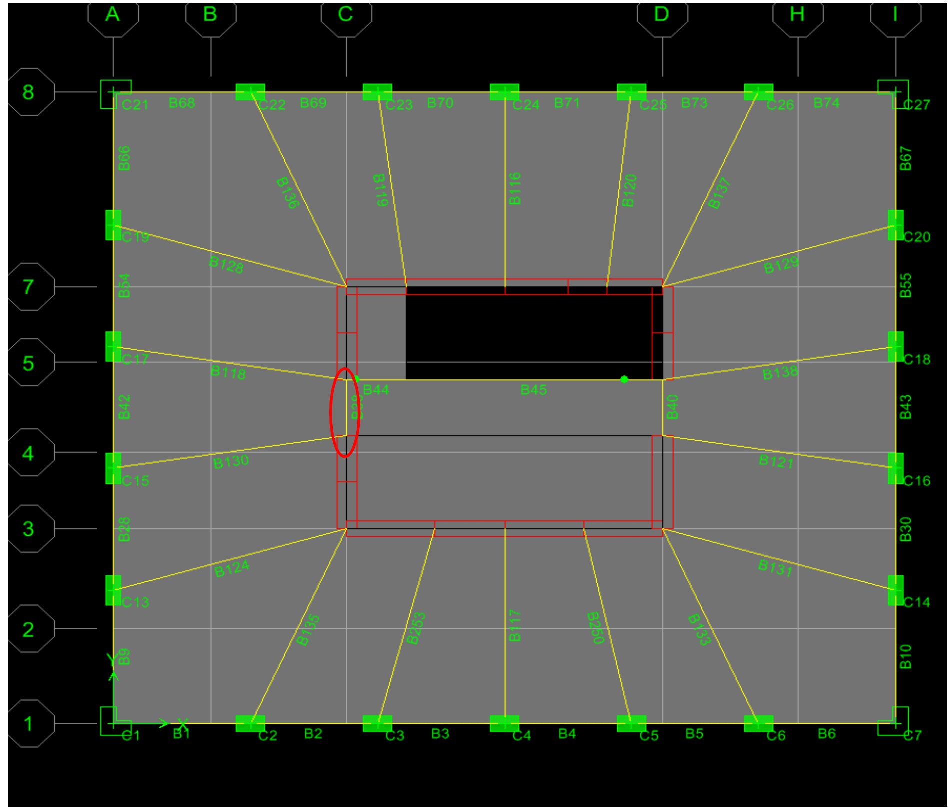
12D10L10EQN MIN- Along Y

Bracing Condition	= Non-Sway	
Non-Sway Part of Loading:		
Design Load, Pu	= 0.00	kip
Sustained Load, Pud	= 0.00	kip
End Moment, M1	= 0.0	k-ft
End Moment, M2	= 0.0	k-ft
Minimum Moment, Mmin	= 0.0	k-ft
Design Moment, Mc	= 0.0	k-ft
Creep Factor, Bd	= .00	
Section Stiffness, EI For Pcr	= 0.00E+00	k-in^2
Euler Buckling Load, Pcr	= 0.00	kip
Buckling Failure	= Pcr < Pu	

Appendix D Link Beam Capacity Check Calculation

	STORY	LB1 LEV4-AMEN2	LB1 LEV7	LB1 LEV47
Loads	Mu (kips-ft)	783	1019	692
	Vu (kips)	264	344	234
Safety Factors	phi_m	0.90	0.90	0.90
	phi_v	0.75	0.75	0.75
Material Properties	fc' (psi)	14,000	14,000	10,000
	Beta1	0.650	0.650	0.650
	fy (psi)	75,000	75,000	75,000
Geometric Properties	b (in)	36.0	36.0	24.0
	h (in)	48.0	30.0	60.0
	Clear cover (in)	1.50	1.50	1.50
	Total cover to center of bar (in)	2.8	2.8	2.8
	d (in)	45.2	27.2	57.2
Flexure Design	A	5908.61	5908.61	12408.09
	B	-3048975.00	-1833975.00	-3858975.00
	C	9393996	12229548	8305644
	As_min (in^2)	7.70	4.63	5.49
	As_required (in^2)	7.70	6.82	5.49
	ρ_required	0.0047	0.0070	0.0040
	ρ_min	0.0047	0.0047	0.0040
	ρ_max	0.0415	0.0415	0.0297
	Bar size (1/8 in)	11	11	11
	Bar diameter	1.410	1.410	1.410
	Area (in^2)	1.56	1.56	1.56
	Required # bars	5	5	4
	No. of Layers	1	1	1
	Bars per layer	5	5	4
	Clear Spacing (in)	6.18	6.18	4.70
	ρ_provided	0.0048	0.0080	0.0046
Shear Design	Stirrup size (1/8 in)	5	5	5
	Bar diameter	0.625	0.625	0.625
	Bar area (in^2)	0.31	0.31	0.31
	ϕVc (kips)	288.6	173.6	205.8
	ϕVs,required (kips)	0.0	170.9	28.3
	Required Av/s (in)	0.024	0.112	0.016
	# Legs of Stirrup	4	4	2
	s_max (in)	22.6	13.6	24.0
	s_required (in)	51.1	11.0	38.3
	Provided Spacing (in)	4.0	4.0	12.0
Capacity Check	a (in)	1.37	1.37	2.30
	ϕMn (kips-ft)	1953.7	1163.2	1968.2
	ϕVn (kips)	288.6	642.5	370.2
	Flexure Design OK?	OK	OK	OK
	Shear Design OK?	OK	OK	OK
Summary	M	5-11	5-11	4-11
	V	4-5@4	4-5@4	2-5@12
	D/C_M	0.40	0.88	0.35
	D/C_V	0.91	0.54	0.63
LB Schedule	M	7#11	7#11	3#11
	V	4-4@5	4-4@5	2-5@12

As required_analysis	3.10		2.17
Schedule provided	6.25		4.68
	Provided is 1/3 greater than required by analysis		Provided is 1/3 greater than required by analysis

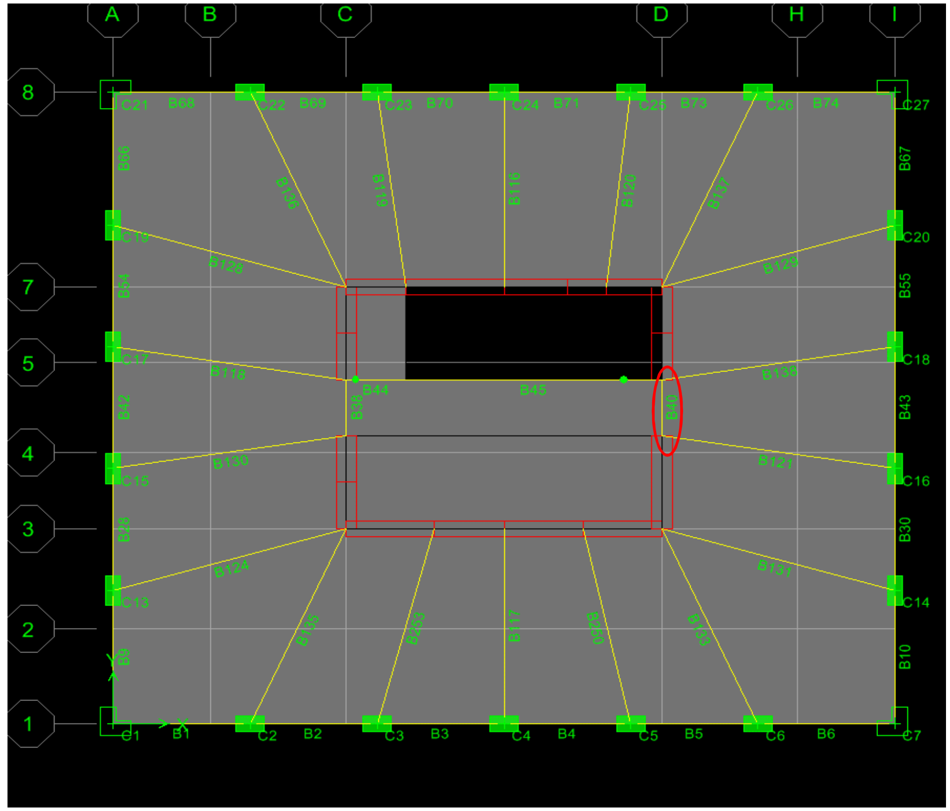


7#11	4-4@5
7#11	4-4@5
3#11	2-5@12

	STORY	LB2 LEV1-LOBBY	LB2 LEV7	LB2 LEV47
Loads	Mu (kips-ft)	288	1029	776
	Vu (kips)	101	348	246
Safety Factors	phi_m	0.90	0.90	0.90
	phi_v	0.75	0.75	0.75
Material Properties	fc' (psi)	14,000	14,000	10,000
	Beta1	0.650	0.650	0.650
	fy (psi)	75,000	75,000	75,000
Geometric Properties	b (in)	36.0	36.0	24.0
	h (in)	30.0	30.0	60.0
	Clear cover (in)	1.50	1.50	1.50
	Total cover to center of bar (in)	2.7	3.0	2.7
	d (in)	27.3	27.0	57.3
Flexure Design	A	5908.61	5908.61	12408.09
	B	-1842412.50	-1825537.50	-3867412.50
	C	3454404	12352632	9314448
	As_min (in^2)	4.65	4.61	5.50
	As_required (in^2)	4.65	6.92	5.50
	ρ_required	0.0047	0.0071	0.0040
	ρ_min	0.0047	0.0047	0.0040
	ρ_max	0.0415	0.0415	0.0297
	Bar size (1/8 in)	11	11	11
	Bar diameter	1.410	1.410	1.410
	Area (in^2)	1.56	1.56	1.56
	Required # bars	3	5	4
	No. of Layers	1	1	1
	Bars per layer	3	5	4
	Clear Spacing (in)	13.89	6.11	4.79
	ρ_provided	0.0048	0.0080	0.0045
Shear Design	Stirrup size (1/8 in)	4	6	4
	Bar diameter	0.500	0.750	0.500
	Bar area (in^2)	0.20	0.44	0.20
	ϕVc (kips)	174.4	172.8	206.3
	ϕVs,required (kips)	0.0	175.3	39.3
	Required Av/s (in)	0.024	0.115	0.016
	# Legs of Stirrup	4	4	2
	s_max (in)	13.6	13.5	24.0
	s_required (in)	32.7	15.3	24.5
	Provided Spacing (in)	12.0	4.0	12.0

Capacity Check	a (in)	0.82	1.37	2.30
	ϕMn (kips-ft)	708.4	1157.7	1972.6
	ϕVn (kips)	174.4	844.9	311.7
	Flexure Design OK?	OK	OK	OK
	Shear Design OK?	OK	OK	OK
Summary	M	3-11	5-11	4-11
	V	4-4@12	4-6@4	2-4@12
	D/C_M	0.41	0.89	0.39
	D/C_V	0.58	0.41	0.79
LB Schedule	M	2#11	9#11	3#11
	V	4-4@12	4-6@4	2-4@12

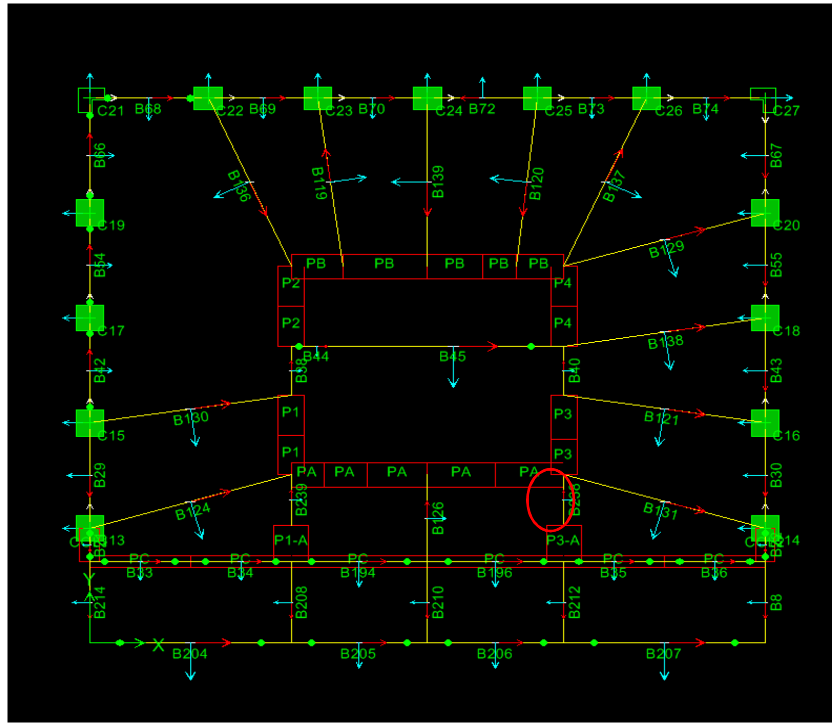
As required_analysis	1.89		2.43
Schedule provided	4.68		4.68
	Provided is 1/3 greater than required by analysis		Provided is 1/3 greater than required by analysis



		LB4
	STORY	LEV4-AMEN2
Loads	Mu (kips-ft)	2305
	Vu (kips)	684
Safety Factors	phi_m	0.90
	phi_v	0.75
Material Properties	fc' (psi)	14,000
	Beta1	0.650
	fy (psi)	75,000
Geometric Properties	b (in)	48.0
	h (in)	48.0
	Clear cover (in)	1.50
	Total cover to center of bar (in)	2.8
	d (in)	45.2
Flexure Design	A	4431.46
	B	-3048975.00
	C	27661500
	As_min (in^2)	10.26
	As_required (in^2)	10.26
	ρ_required	0.0047
	ρ_min	0.0047
	ρ_max	0.0415
	Bar size (1/8 in)	11
	Bar diameter	1.410
	Area (in^2)	1.56
	Required # bars	7
	No. of Layers	1
	Bars per layer	7
	Clear Spacing (in)	5.65
	ρ_provided	0.0050
Shear Design	Stirrup size (1/8 in)	5
	Bar diameter	0.625
	Bar area (in^2)	0.31
	ϕVc (kips)	384.8
	ϕVs,required (kips)	298.9
	Required Av/s (in)	0.118
	# Legs of Stirrup	4
	s_max (in)	22.6
	s_required (in)	10.4
	Provided Spacing (in)	5.0

Capacity Check	a (in)	1.44
	ϕMn (kips-ft)	2733.0
	ϕVn (kips)	1008.4
	Flexure Design OK?	OK
	Shear Design OK?	OK
Summary	M	7-11
	V	4-5@5
	D/C_M	0.84
	D/C_V	0.68

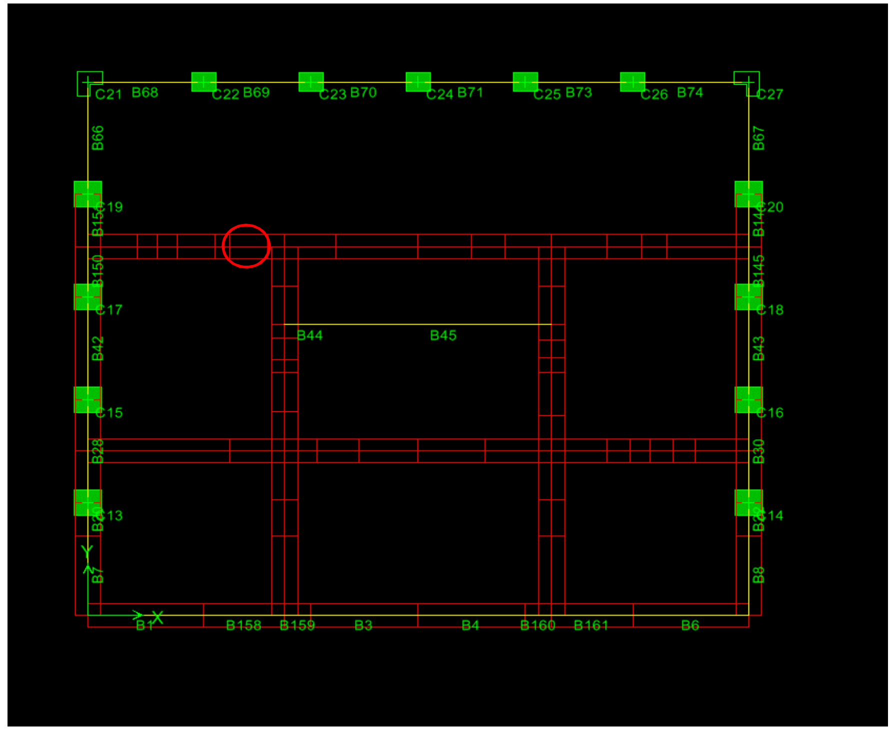
LB Schedule	M	8#11
	V	4-5@5



		LB5
	STORY	LEV5-MECH2
Loads	Mu (kips-ft)	1481
	Vu (kips)	420
Safety Factors	phi_m	0.90
	phi_v	0.75
Material Properties	fc' (psi)	14,000
	Beta1	0.650
	fy (psi)	75,000
Geometric Properties	b (in)	36.0
	h (in)	48.0
	Clear cover (in)	1.50
	Total cover to center of bar (in)	2.7
	d (in)	45.3
Flexure Design	A	5908.61
	B	-3057412.50
	C	17771448
	As_min (in^2)	7.72
	As_required (in^2)	7.72
	ρ_required	0.0047
	ρ_min	0.0047
	ρ_max	0.0415
	Bar size (1/8 in)	11
	Bar diameter	1.410
	Area (in^2)	1.56
	Required # bars	5
	No. of Layers	1
	Bars per layer	5
	Clear Spacing (in)	6.24
	ρ_provided	0.0048
Shear Design	Stirrup size (1/8 in)	4
	Bar diameter	0.500
	Bar area (in^2)	0.20
	ϕVc (kips)	289.4
	ϕVs,required (kips)	130.3
	Required Av/s (in)	0.051
	# Legs of Stirrup	4
	s_max (in)	22.6
	s_required (in)	15.4
	Provided Spacing (in)	12.0

Capacity Check	a (in)	1.37
	ϕMn (kips-ft)	1959.2
	ϕVn (kips)	456.2
	Flexure Design OK?	OK
	Shear Design OK?	OK
Summary	M	5-11
	V	4-4@12
	D/C_M	0.76
	D/C_V	0.92

LB Schedule	M	8#11
	V	4-4@12



Appendix E List of Documents Reviewed

DOCUMENTS REVIEWED

1. **Geotechnical Investigation Report, Proposed 281 Fifth Avenue Development, by Langan Engineering and Environmental Services dated 18 May 2015.**
2. **Structural Design Criteria shown in Drawing FO-001.01 by WSP Group dated 8 April 2016.**
3. **Study of Wind Effects for 281 Fifth Avenue, New York, NY by the Boundary Layer Wind Tunnel Laboratory dated 25 November 2015**
4. **Architectural Drawings by Rafael Vinoly Architects dated 8 April 2016, for DOB Submission as listed below:**

Drawing List			DOB SUBMISSION 4/8/2016
SHEET #	ARCHITECTURAL	SCALE	
A0 Series	GENERAL INFORMATION		
A-001.00	DRAWING LIST	NTS	X
A-002.00	GENERAL NOTES	N/A	X
A-003.00	GENERAL NOTES	N/A	X
A-004.00	SYMBOLS AND ABBREVIATIONS	N/A	X
A-005.00	SITE FLOOD MAP	N/A	X
A-006.00	ACCESSIBILITY DIAGRAM	N/A	X
A1 Series	BUILDING PLANS		
A-100.00	CELLAR FLOOR PLAN LEVEL -01	1/4" = 1'-0"	X
A-101.00	GROUND FLOOR PLAN LEVEL 01	1/4" = 1'-0"	X
A-102.00	COMMERCIAL FLOOR PLAN LEVEL 02	1/4" = 1'-0"	X
A-103.00	AMENITY FLOOR PLAN LEVEL 03 LOUNGE	1/4" = 1'-0"	X
A-104.00	AMENITY FLOOR PLAN LEVEL 04 FITNESS	1/4" = 1'-0"	X
A-105.00	MECH/STRUCTURAL TRANF. FLOOR PLAN LEVEL 05 LOWER	1/4" = 1'-0"	X
A-106.00	MECH/STRUCTURAL TRANF. FLOOR PLAN LEVEL 05 UPPER	1/4" = 1'-0"	X
A-107.00	RESI. TYP FLOOR PLAN LEVEL 6	1/4" = 1'-0"	X
A-108.00	RESI. TYP FLOOR PLAN LEVEL 7-24	1/4" = 1'-0"	X
A-109.00	MECHANICAL FLOOR PLAN LEVEL 25 LOWER	1/4" = 1'-0"	X
A-110.00	MECHANICAL FLOOR PLAN LEVEL 25 UPPER	1/4" = 1'-0"	X
A-111.00	RESI. TYP FLOOR PLAN LEVEL 26-33	1/4" = 1'-0"	X
A-112.00	RESI. TYP FLOOR PLAN LEVEL 34-37	1/4" = 1'-0"	X
A-113.00	RESI. FLOOR PLAN LEVEL 38 DUPLEX LOWER	1/4" = 1'-0"	X
A-114.00	RESI. FLOOR PLAN LEVEL 39 DUPLEX UPPER	1/4" = 1'-0"	X
A-115.00	RESI. FLOOR PLAN LEVEL 40 DUPLEX LOWER	1/4" = 1'-0"	X
A-116.00	RESI. FLOOR PLAN LEVEL 41 DUPLEX UPPER	1/4" = 1'-0"	X
A-117.00	RESI. FLOOR PLAN LEVEL 42 DUPLEX LOWER	1/4" = 1'-0"	X
A-118.00	RESI. FLOOR PLAN LEVEL 43 DUPLEX UPPER	1/4" = 1'-0"	X
A-119.00	RESI. FLOOR PLAN LEVEL 44 DUPLEX LOWER	1/4" = 1'-0"	X
A-120.00	RESI. FLOOR PLAN LEVEL 45 DUPLEX UPPER	1/4" = 1'-0"	X
A-121.00	PENTHOUSE FLOOR PLAN LEVEL 46	1/4" = 1'-0"	X
A-122.00	MECHANICAL FLOOR PLAN LEVEL 47 LOWER	1/4" = 1'-0"	X
A-123.00	MECHANICAL FLOOR PLAN LEVEL 47 UPPER	1/4" = 1'-0"	X
A-124.00	SLOSH TANK PLAN LEVEL 48	1/4" = 1'-0"	X
A-125.00	TOWER ROOF AND EMR LEVEL 49	1/4" = 1'-0"	X
A-126.00	TOWER TOP	1/4" = 1'-0"	X

A2 Series	BUILDING ELEVATIONS		
A-201.00	ELEVATIONS NORTH AND SOUTH	1/32" = 1'-0"	X
A-202.00	ELEVATIONS EAST AND WEST	1/32" = 1'-0"	X
A-203.00	ENLARGED ELEVATIONS NORTH AND WEST	1/8" = 1'-0"	X
A-204.00	ENLARGED ELEVATIONS SOUTH AND EAST	1/8" = 1'-0"	X
A-260.00	THERMAL BUILDING SECTION	1/8" = 1'-0"	X
A3 Series	BUILDING SECTIONS		
A-301.00	OVERALL BUILDING SECTIONS EAST - WEST	1/32" = 1'-0"	X
A-302.00	OVERALL BUILDING SECTIONS NORTH - SOUTH	1/32" = 1'-0"	X
A-303.00	PARTIAL BUILDING SECTIONS 1	1/8" = 1'-0"	X
A-304.00	PARTIAL BUILDING SECTIONS 2	1/8" = 1'-0"	X
A-330.00	STAIR SECTION AND PLANS LEVEL CELLAR TO 24	1/4" = 1'-0"	X
A-331.00	STAIR SECTION AND PLANS LEVEL 25L TO 49	1/4" = 1'-0"	X
A4 Series	BUILDING COMPONENTS		
A-401.00	ENCLOSURE TYP. FLOOR NORTH-SOUTH FACADE	3/8" = 1'-0"	X
A-402.00	ENCLOSURE TYP. FLOOR EAST-WEST FAÇADE	3/8" = 1'-0"	X
A-403.00	ENCLOSURE MECH. FLOOR LVL. 25 NORTH FACADE	3/8" = 1'-0"	X
A-404.00	ENCLOSURE MECH. FLOOR LVL. 25 SOUTH FACADE	3/8" = 1'-0"	X
A-405.00	ENCLOSURE MECH. FLOOR LVL. 25 EAST FACADE	3/8" = 1'-0"	X
A5 Series	DETAILS		
A-501.00	ELEVATOR CAB PLANS AND CHART	1/4" = 1'-0"	X
A-502.00	ELEVATOR HOIST WAY SECTIONS AND DETAILS	1/4" = 1'-0"	X
A-503.00	ELEVATOR LOBBY AND RESIDENTIAL CORRIDOR ENLARGED PLANS	1/4" = 1'-0"	X
A-504.00	PASSENGER ELEVATOR CAB ENLARGED PLANS AND DETAILS 01	1/4" = 1'-0"	X
A-505.00	PASSENGER ELEVATOR CAB ENLARGED PLANS AND DETAILS 02	1/4" = 1'-0"	X
A-506.00	SERVICE ELEVATOR CAB ENLARGED PLANS AND SECTIONS	1/4" = 1'-0"	X
A-507.00	SERVICE ELEVATOR CAB ENLARGED PLANS AND DETAILS	1/4" = 1'-0"	X
A-570.00	TRASH CHUTE DETAILS	3/8" = 1'-0"	X
A7 Series	FINISH SCHEDULES, INTERIOR FINISH & APPLIANCES		
A-701.00	PARTITION TYPE 01	1/4" = 1'-0"	X
A-702.00	PARTITION TYPE 02	1/4" = 1'-0"	X
A-703.00	DOOR TYPES	N/A	X
A-704.00	FINISH SCHEDULE	N/A	X
A-705.00	FINISH SCHEDULE	N/A	X
A-706.00	KITCHEN APPLIANCE/PUMBING SCHEDULE	N/A	X
A-707.00	BATHROOM PLUMBING SCHEDULE	N/A	X

5. Structural Drawings by WSP Group, for DOB Submission, as listed below:

SHEET #	SHEET NAME	NO.	DATE	ISSUE
FO-001.01	General Notes, Legend & Abbreviations	1	2016/4/8	DOB SUBMITTAL
FO-100.01	Foundation Plan	2	2016/4/8	DOB SUBMITTAL
FO-110.01	Mat Foundation Plan	3	2016/4/8	DOB SUBMITTAL
FO-200.01	Typical Foundation Details 1	4	2016/4/8	DOB SUBMITTAL
FO-201.01	Typical Foundation Details 2	5	2016/4/8	DOB SUBMITTAL
FO-202.00	Typical Foundation Details 3	6	2016/4/8	DOB SUBMITTAL
FO-203.00	Typical Foundation Details 4	7	2016/4/8	DOB SUBMITTAL
FO-300.01	Foundation Sections 1	8	2016/4/8	DOB SUBMITTAL
FO-301.00	Foundation Sections 2	9	2016/4/8	DOB SUBMITTAL
S-010.01	Ground Floor Framing Plan	10	2016/4/8	DOB SUBMITTAL
S-020.00	2nd Floor Framing Plan	11	2016/4/8	DOB SUBMITTAL
S-030.00	3rd Floor Framing Plan	12	2016/4/8	DOB SUBMITTAL
S-040.00	4th Floor Framing Plan	13	2016/4/8	DOB SUBMITTAL
S-050.00	5th Floor Framing Plan	14	2016/4/8	DOB SUBMITTAL
S-055.00	5th Floor Mech. Upper Framing Plan	15	2016/4/8	DOB SUBMITTAL
S-060.00	6th Floor Framing Plan	16	2016/4/8	DOB SUBMITTAL
S-070.00	7th To 24th Floor Framing Plan	17	2016/4/8	DOB SUBMITTAL
S-250.00	25th Floor Framing Plan	18	2016/4/8	DOB SUBMITTAL
S-255.00	25th Floor Upper Framing Plan	19	2016/4/8	DOB SUBMITTAL
S-260.00	26th Floor Framing Plan	20	2016/4/8	DOB SUBMITTAL
S-270.00	27th - 37th Floor Framing Plan	21	2016/4/8	DOB SUBMITTAL
S-380.00	38th Floor Duplex Lower Framing Plan	22	2016/4/8	DOB SUBMITTAL
S-390.00	39th Floor Duplex Upper Framing Plan	23	2016/4/8	DOB SUBMITTAL
S-400.00	40th Floor Duplex Lower Framing Plan	24	2016/4/8	DOB SUBMITTAL
S-410.00	41st Floor Duplex Upper Framing Plan	25	2016/4/8	DOB SUBMITTAL
S-420.00	42nd Floor Duplex Lower Framing Plan	26	2016/4/8	DOB SUBMITTAL
S-430.00	43rd Floor Duplex Upper Framing Plan	27	2016/4/8	DOB SUBMITTAL
S-440.00	44th Floor Duplex Lower Framing Plan	28	2016/4/8	DOB SUBMITTAL
S-450.00	45th Floor Duplex Upper Framing Plan	29	2016/4/8	DOB SUBMITTAL
S-460.00	46th Floor Framing Plan	30	2016/4/8	DOB SUBMITTAL
S-470.00	47th Floor Framing Plan	31	2016/4/8	DOB SUBMITTAL
S-480.00	48th Floor Framing Plan	32	2016/4/8	DOB SUBMITTAL
S-485.00	48th Floor Upper Framing Plan	33	2016/4/8	DOB SUBMITTAL
S-490.00	49th Floor Slosh Tank/EMR Framing Plan	34	2016/4/8	DOB SUBMITTAL
S-500.00	50th Floor Roof Framing Plan	35	2016/4/8	DOB SUBMITTAL
S-510.00	Bulkhead Framing Plan	36	2016/4/8	DOB SUBMITTAL

S-930.00	Shearwall Elevation 1	37	2016/4/8	DOB SUBMITTAL
S-931.00	Shearwall Elevation 2	38	2016/4/8	DOB SUBMITTAL
S-932.00	Shearwall Elevation 3	39	2016/4/8	DOB SUBMITTAL
S-940.01	Shearwall Reinforcement Plan	40	2016/4/8	DOB SUBMITTAL
S-945.00	Typical Shear Wall Details	41	2016/4/8	DOB SUBMITTAL
S-946.00	Link Beam Schedule	42	2016/4/8	DOB SUBMITTAL
S-950.01	Column Schedule	43	2016/4/8	DOB SUBMITTAL
S-951.00	Typical Column Details	44	2016/4/8	DOB SUBMITTAL
S-960.00	Typical Superstructure Details 1	45	2016/4/8	DOB SUBMITTAL
S-961.00	Typical Superstructure Details 2	46	2016/4/8	DOB SUBMITTAL
S-962.00	Typical Superstructure Details 3	47	2016/4/8	DOB SUBMITTAL
S-963.00	Typical Superstructure Details 4	48	2016/4/8	DOB SUBMITTAL
S-964.00	Typical Superstructure Details 5	49	2016/4/8	DOB SUBMITTAL
S-970.00	Superstructure Section	50	2016/4/8	DOB SUBMITTAL
S-980.00	Typical Stair Details	51	2016/4/8	DOB SUBMITTAL