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PROJECT PROPOSAL

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Location: City of Yonkers, Westchester County, New York

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Company Profile

Ziba & Associates is a large, privately operated company that is a leading national provider in civil engineering consulting, specializing in medium- to large-scale residential development projects. In order to become a unique and distinct firm within the industry, Ziba & Associates has completed residential developments in varying and challenging sites. We pride ourselves in undertaking the engineering and construction challenges of difficult developments while meeting the requirements of our customers. We maintain a high quality of work, on-time delivery, and safety for all of our projects. Ziba & Associates creates value for clients through the application of knowledge, innovative ideas, and a stern dedication to having the absolute best work ethic and proficient design work.

The level of excellence Ziba & Associates produces in each project is a product of understanding the needs of a client and providing the proper means of excelling to meet those needs. The company's in depth knowledge of the customer's business helps Ziba & Associates deliver specialized assistance in providing for the client's needs. That is what makes Ziba & Associates a leader in customer satisfaction and excellent design work.

Mission Statement

Through the practice of high-quality consulting services at fair market competitive prices, we establish lasting relationships with our clients. We are committed to providing the highest benefit-to-cost ratio in developments, as a result of the quality services and the ideal that clients are our family.

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Survey Sheet

The development site is located in the City of Yonkers in Westchester County. The total square footage is 438,518, roughly 10 acres, designated parcel two. The site is bordering approximately 25 privately owned lots to the west and north. To the southwest of the property is Poly Street, a 4-lane road. To the south is MetroTech Park and east of that is Polytechnic Lake Park which is a municipal open space. Bordering the property is parcel one where a one story stone building is located. A blacktop road extends to parcel two from Poly Road and continues through parcel one. Easements for the benefit of parcel one and two are granted for this blacktop road. Bordering Poly Road to the southwest are existing developments of a garden, a fountain, and a concrete driveway.

Average soil slopes of 30 percent, rock outcrops, and dense vegetation characterize the site. The largest rock outcrop is located on a ridgeline that extends northeast to from the western part of the site, separating the property east and west. There are smaller rock outcrops located on the eastern side, however, a majority of the rock is located on the west side of the division. Dense vegetation and shrubs characterize the eastern part of the site. The site has a maximum elevation of 330 feet and a minimum elevation of 170 feet.

Geological studies have classified the soil type as hydrologic soil group C. Group C soils are sandy clay loam. They have low infiltration rates when thoroughly wetted and consist primarily of soils with a layer that impedes downward movement of water and soils with moderately-to-fine structure. A subsurface investigation revealed that the top 25 feet of soil consists of well-graded sand, with an allowable bearing capacity of 12,000 pounds per square foot and a percolation rate of 4 inches per hour. Below the layer of well-graded sand is a layer of fractured rock with an allowable bearing capacity of 24,000 pounds per square foot. The water table cannot be accurately determined.

This site has several challenges for the development of a residential building. Due to the steep slopes located on site, the building pad must be embedded into the existing soil. Considerations must be made to avoid blasting rock to create a more economic design. Due to the heavily vegetated existing surface, development on the site will considerably alter the existing drainage pattern.

City of Yonkers

The city of Yonkers has an estimated population of 200,000 within an area of 18.3 square miles, 4.5 square miles of which are waterfront. Yonkers is the fourth largest city in New York State and the largest city in Westchester County. It is bordered by New York City to the south and the Hudson River to the west. The population trend has pointed to an increase in population from 1940 to 2010; approximately 24 percent of Westchester county population is located in Yonkers. The main employment sector in Yonkers is professional services providing an estimated 25,000 jobs. The median household income is $47,000 per year, and 43 percent of the population has an income greater than $50,000 per year. The city of Yonkers has 39 public schools, 3 colleges, 100 parks and playgrounds, 2 golf courses, 3 libraries, 3 hospitals, 17 shopping malls, 20 bus routes, and 4 airports within a 20 to 40 minute radius. The average household size is 2.7 persons per family.

Owner Requirements

The owner proposed the following requirements: one building structure with the minimum footprint of 12,000 square feet and a minimum of 12 stories with the top two stories to be set back on all four sides, underground parking below the building must be provided for at least 20 cars, an outdoor recreational area must be provided for the residents of the development measuring approximately 10,000 square feet, and outdoor surface parking for a minimum of 75 cars. If extra capacity is necessary it can be achieved through an independent parking garage structure.

Proposed Development

Based on the previously listed demographics, it is economical to undertake a residential development, where the majority of the housing units will be for middle-income residents, and a couple units for higher income residents. With the increasing trend in population and Yonkers being a suburb of New York City, residential development will have value in the present time and gain value in the future. Our proposed building development is a residential concrete flat plate structure, with 81 apartments. Seventy-nine will be one bedroom units, and two will be penthouses with four bedrooms each. One-bedroom units are ideal because the target population is young professionals and university students who are seeking rental apartments. The structure will be located on the eastern portion of the site where there are no rock outcrops, as blasting rock is not desirable. The building will be embedded into the soil, the front of the building will be at grade and the rear entrance will be located 30 feet above the front entrance. This allows vehicles to enter the underground parking at grade and limits the need for excess retaining structures. Underground parking is provided for 26 vehicles. A two-lane main access road will intersect perpendicularly to Poly Street then turn east towards the development site. Seven thousand five hundred square feet of recreational park space will be provided directly west of the main access road where it intersects Poly Street. An additional 1,500 square feet will be provided in the rear of the building. A retention pond will be located on the eastern side of the main access road across from the 7,500 square foot recreational area. This location was picked because it is located at a minimum relative elevation and the site naturally drains to this point. The main access road will terminate at the eastern parking lot. 76 parking stalls will be provided by two surface parking lots located on the east and west of the building. The western parking lot has a capacity of 50 vehicles, and the eastern parking lot has a capacity of 26 vehicles. Two parking lots were used to limit the distance from the building entrance to the farthest parking stall which was set to a maximum of 200 feet.

Recreational Area

The recreation facility will be divided into two parts, a 7,500 square foot open park space and a 1,500 square feet bocce court. The open park will be located at the site entrance and access will be provided from the adjacent sidewalk that runs along the west side of the main access road. The existing site conditions in this area are relatively flat, therefore grading is not necessary. The area is drained by one catch basin located on the southeast of the area by the sidewalk. The park space will be landscaped with a garden, trees, and lawn. Benches will be provided to accommodate picnicking. The open park will serve the general population of the development and provide an open facility for leisure activities.

The other recreational area will consist of one bocce court and the surrounding area will be open space for spectators. This recreational area will be located at the rear of the building. This area can be accessed via the rear building entrance, which is located on the third floor. This location is relatively flat because it is located between the building and retaining walls. The recreational area will be graded so that the center will be the highest point and water will flow away with the edges then drain into the swales around the building.

Bocce originates from the Romans and is still popularly played in Italy and some Adriatic nations such as Croatia, Montenegro, and southern Bosnia. Bocce is traditionally played on natural soil or asphalt courts. The typical dimensions for a court are 90 feet by 30 feet. Bocce is a sport that uses metal or plastic balls. The strategies employed make the game complex and can become extremely competitive. Sports such as bocce are not well known in the United States and would be more interesting for the residents of this development than the typical residential recreational areas that are provided.

Grading Plan

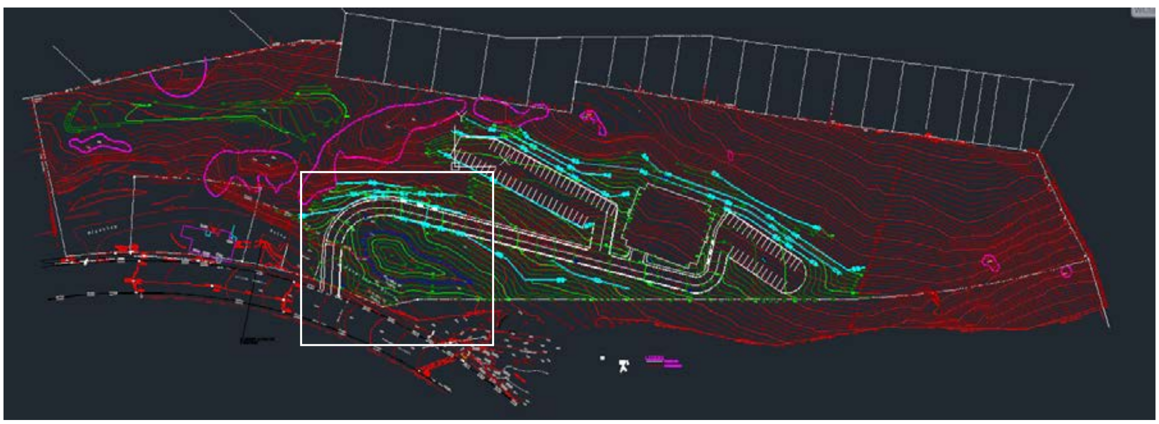
Methodology

We strived to achieve the most economical solution for the given site. In order to achieve this we wanted to work with the natural grade of the site, rather than fight it. The site grade was fairly constant, steeply sloping down from the northeast end of the property toward the southwest end. We wanted to use this to our advantage for grading and also wanted to place our structures parallel to the existing contour lines. The lowest point on the site is near the southern edge of the property line near Concrete Drive and Poly Street. There were several rock outcrops near the western portion of parcel 2. We wanted to avoid these areas as much as possible to avoid blasting rock, which is not economical. All of the elevations along the property line are fixed, since we cannot grade into the neighbor's property.

From chapter two of the New York State Department of Transportation Highway Design Manual, we graded the roads to a maximum of five percent. When ramps were necessary, we graded them to a maximum of 15 percent and kept them to a minimum distance. Ramped areas were designed to be a maximum of 100 feet to ensure the comfort of the drivers. Parking lots and walkways were also designed to be at a maximum grade of five percent. The soil on the property was determined to be well-graded sand by the geotechnical investigation. It was determined that the soil would be stable up to a 25 percent grade. Any higher grades would require soil stabilization. We wanted to avoid soil stabilization wherever possible to reduce the costs of grading. The owner requested that we use retaining walls no higher than six feet above grade. Also, based on local codes, retaining walls are not permitted within 30 feet of the property line.

Process

The first step in the process was to determine where the entrance of the access road would be. The best determined entrance was near the present location of concrete drive where it approaches the property line and Poly Street. This was not only one of the lowest points at the property line, but also contained one of the gentlest slopes at the property line. This area would be ideal to gently ramp up the access road from the entrance from Poly Street, as shown below in figure 1. It would be necessary to achieve proper driver comfort and safety.



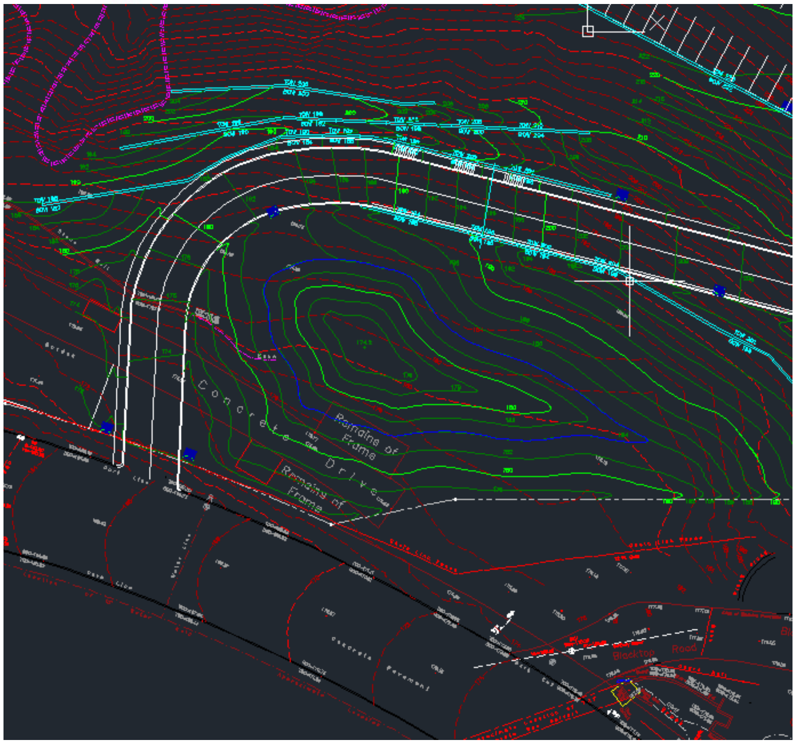
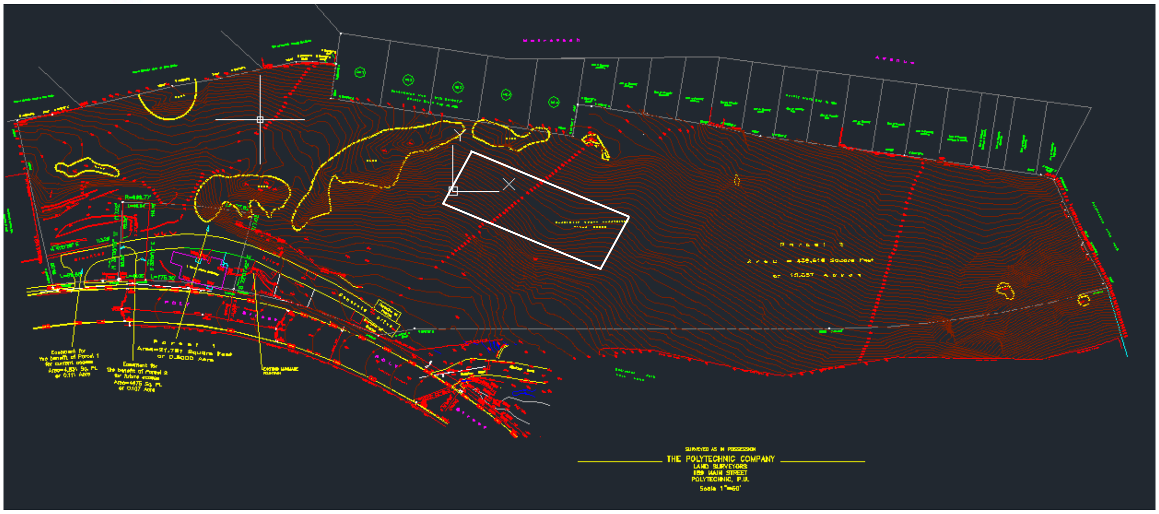


Figure 1

The second step was to determine the location of the building. When determining the location of the building, we kept in mind that all parking lots would be within two hundred feet from the edge of the building. We also kept in mind that the maximum a parking lot could be graded to is five percent, while the maximum soil on our site could be graded to was twenty-five percent. This meant that the parking lots should be located on the gentler existing slopes to avoid fighting the natural grade of the site. We chose the parking lot to be located on the gentlest slope in the middle of parcel two, as shown in figure 2.



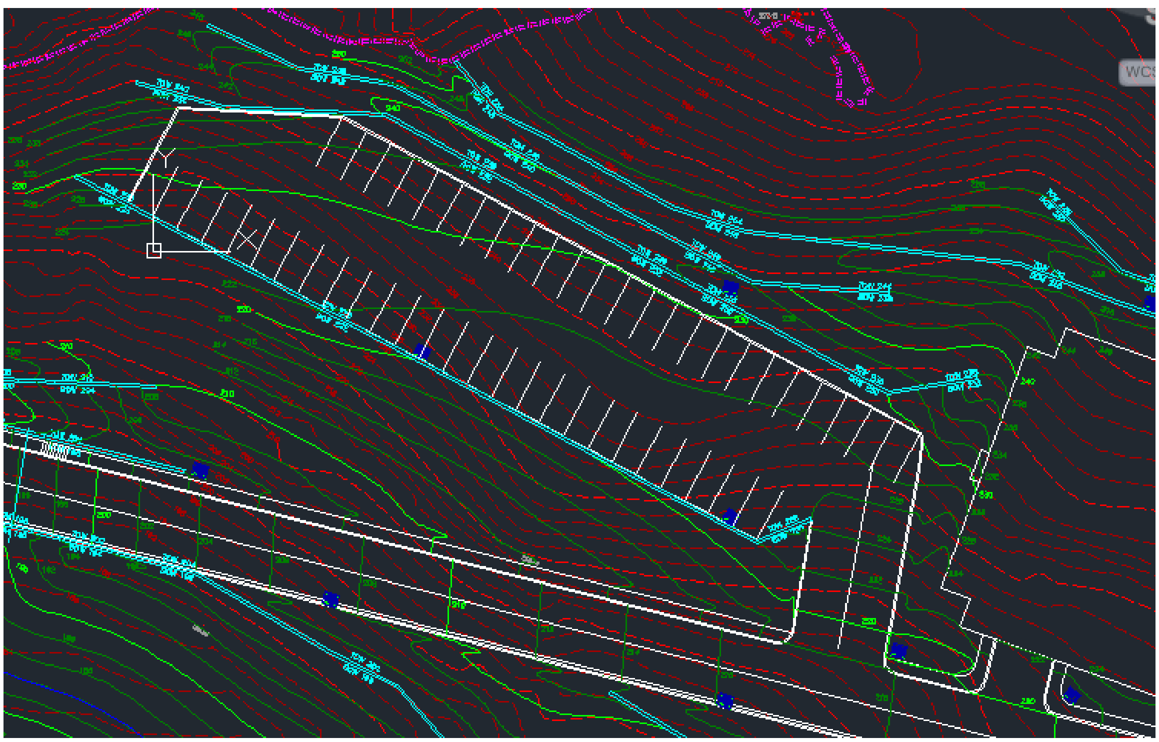
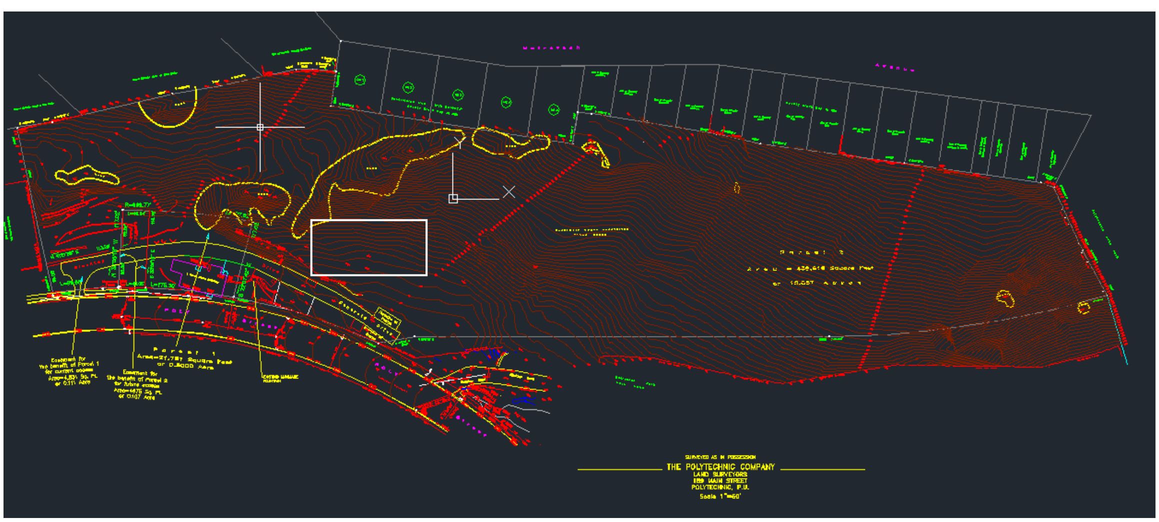


Figure 2

The building was then placed to the east of the parking lot, to avoid getting too close to the bedrock. This would allow us the option to have parking on the east side of the building, if necessary. We placed the footprint of our building parallel to the existing contour lines so the front and back of the building would be relatively flat which would be best for placing entrances to the building. Also, since we planned on placing the first few floors underground, we would have fairly constant slopes along the sides of the building to reduce the cut and fill. These slopes would assist in the drainage of the water away from the building.

The next step was to determine if the road could slope up to the front of the building with the restrictions of the road being five percent and fifteen percent for ramps. We did this by selecting the elevation point of the front of the building to be 222 feet. The entrance of the access road was located at the existing elevation of 172 feet. This meant that the road had to gain 50 feet vertically over the horizontal distance of our road, which was about 500 feet. That would come out to a steady grade of about 10 percent, which is over the standard 5 percent. This meant that a portion of our road had to be ramped at 15 percent. We determined that this portion would be where the natural grade of the site was the steepest, which was near the beginning of the access road close to the area where the road began to curve as shown in figure 3.



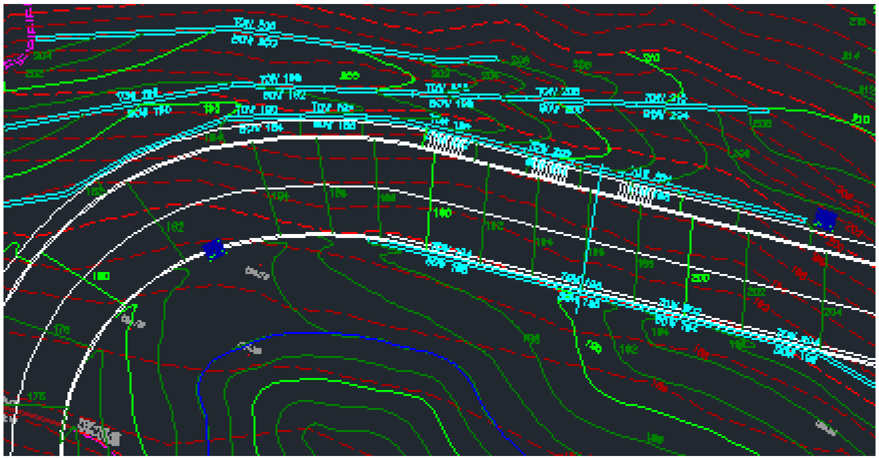


Figure 3

We knew that we would not be able to start the road at 15 percent, so we determined the comfort distance necessary to build up to a 15 percent slope. It was about 100 feet, shown below in the Paving Plan section. We determined we would have the first 100 feet of the road be gently sloped up to a 15 percent grade, followed by a short ramp around 100 feet, and the remainder of the road would be a steady slope of around 5 percent.

Next, we needed to determine if the size of our parking lot was sufficient. We estimated the total amount of parking spots we needed was about 80 spots with the given size and the amount of units in our residential building. The owner also stated that we needed a minimum of 75 spots outside the building. Knowing this, we placed as many spots as possible with the current size of our parking lot. The current size was too small; therefore we needed to add parking. We first attempted adding terraced parking just north of our parking spots. After grading this option, it was determined that it would take up too much space, would need too much cut in elevation, and would be very difficult to meet the minimum grades for parking lots and roads. This option would also require much more cut and retaining walls, therefore it was less economical. Our second solution was to place another parking lot on the east side of the building. We placed the lot parallel to the existing contours. Knowing that we had sufficient parking spots, we began to grade the rest of the site.

The next step was to grade the road starting from the entrance. Since just east of the entrance is one of the lowest portions of our site, we planned the location of the pond to be there. We ran rough estimates for the size of the pond necessary and left a substantial area for the pond just east of the entrance. We planned on placing no retaining walls near the location of the pond, just in case it needed to increase in size. The elevation points along the property line were fixed since we did not want to disturb our neighbor's property. We began by placing spot elevations along the road to meet the necessary road grade. We then connected the elevation on the edge of the road to the elevation on the property line. We used a maximum grade of 25 percent wherever possible. When the grade of the soil exceeded 25 percent we placed a retaining wall. We graded the edges of the road all the way up to the front of the building.

The next step was to grade north of the road. We began by placing ramps from the access road to the parking lots to gain as much elevation as possible and minimize the amount of cut we would need for the parking lots. We also used a grade of 25 percent along the west side of the building to minimize cut. On the east side of the building we ran a slope of 15 percent to reach the same elevation on the north face of the building. We continued grading north of the building and parking lots until we were able to reach the existing grade, adding retaining walls when necessary.

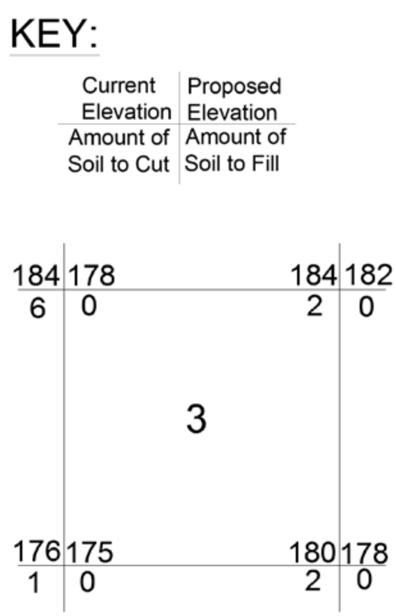
Our next step was to fine-tune our grading plan to ensure for proper drainage along the entire site, using swales when necessary. Using natural drainage with the site grade would reduce the cost and need for infrastructure to handle storm water. We identified critical portions of our site where the drainage would be problematic. We looked for areas that were particularly flat or drained towards our structures and parking lots. Some problematic locations were in between adjacent retaining walls. These problems were adjusted by stepping the retaining walls to allow for water to drain perpendicularly to the face of the walls. Another problematic location was along the property line, where the water was draining onto the neighbor’s property. We are required to keep all of our storm water runoff on our site, so we placed a swale along our property line that drains into our pond. This was a cost effective solution involving as little infrastructure as possible.

The final step was to grade the roads and parking lot to ensure proper drainage. We began by identifying the best place to drain the water to. We decided to drain the parking lots towards the entrances to them and to drain the roads towards the southern face. We then placed the curb heights along the road and parking lot and placed the catch basins on the road where necessary. We also cross graded the road at two percent to ensure there would not be ponding on the road.

Cut and Fill Results

Due to the differences between the current and proposed elevations in the grading plan, there is going to be a large amount of soil that needs to be removed from the site. The elevations along the south-part of the site between the road and the property line are at lower elevations than what has been proposed which means that soil will have to fill the areas to the desired elevation. The only location in this area of the site that has higher elevations than what are proposed is the area where the pond is. The area north of the road and south of the west parking lot is very flat. Therefore, it requires some soil to be filled to the proposed elevations along the western part of this area as well as soil to be cut down to the proposed elevations along the eastern part of this area as it leads to the building. The west parking lot requires soil to be cut along the north side in order to meet the elevations at the south end of the lot. Above the building, the current elevations are much higher than the proposed elevations. Because of this, retaining walls were placed throughout this area to account for the current slopes and to bring them to a desired 25 percent grade. There is a total of 3,091 linear feet of retaining walls throughout the site. The elevation at the south entrance of the building is the same elevation as the slab on grade. There is roughly a 20 foot difference in elevation between the north entrance of the building and the slab on grade elevation with a 25 percent slope along the east and west sides of the building in order for the two elevations to meet. This requires a lot of soil to be removed in order for the building meet the slab on grade elevation from the current elevations.

A grid size of 30 feet by 30 feet was used to determine the cut and fill over the entire site except at the pond location. The slopes varied much more near the pond area, so a grid size of 15 feet by 15 feet was used to determine the cut and fill more accurately. The total amount of soil to be cut is 31,489-cubic yards. With a 15 percent swell factor taken into account, the total volume is 36,212-cubic yards. The amount of fill required is 10,641-cubic yards. This is a cut-to-fill ratio of 2.96. This means that there is nearly three times more soil that needs to be cut to meet the desired elevations than needs to be filled, therefore, there is a significant amount of soil that needs to be removed from the site. After the 6-inches of topsoil is taken out and stored onsite, there will need to be a large amount of soil removed from the site permanently. Using 40-cubic yard size trucks to remove the soil, there will need to be 512 truckloads of soil removed from the site. A sample calculation to find the differences in elevation along the road using the grid method is shown below:



$$A = 30' \times 30' = 900\ sqft.$$

$$Average\ Fill = \frac{6 + 2 + 1 + 2}{4} = 3'$$

Volume of fill for this square area:

$$V = A * h = 900 * 3 = 2700\ cuft.$$

Erosion and Sediment Control Plan

Project Narrative

Project Description

The purpose of this project is to construct a 16-story residential building and two recreational centers with an associated paved road and two parking areas. There will be an on-site retention pond to collect all runoff from the site. Approximately 4.5 acres will be disturbed during the construction. The total area of the site is about 10.6 acres, located in the City of Yonkers, in Westchester County, NY .

Site Description

The site is located on mountainous topography with slopes generally ranging from 20 to 40 percent. There are also various sized rock outcrops scattered predominantly around the western portion of the property. Currently, the site is covered with 6 inches of thick vegetation. According to Table 2.2 Erosion Risk of the New York State Standards and Specifications of Erosion and Sediment Control by the Department of Environmental Conservation, the site, being sandy soil with a grade larger than fifteen percent, if disturbed, is categorized at a high risk of erosion.

Adjacent Property

Land is in residentially zone area. To the West and North of the property, there are nineteen privately owned lots and five vacant lots that are adjacent to the property. These lots are higher in elevation than the lot that is under construction. To the East of the property, there is Polytechnic Lake Park. To the South of the lot, there is a pond (Poly Pond) located within a park (Metrotech Park). Sediment control measures will be taken to prevent damage to or contamination of the park and the pond.

Soils

The soil in the project area is categorized as forest or woodlands, with governing hydrologic soil group (HSG) C.

A subsurface investigation of the site revealed that the top 25 feet of soil consist of well graded sand (SW) with an allowable bearing capacity of 12,000 pounds per square foot, and a percolation rate of 4 in/hr. The sand layer is underlain by a fractured rock layer with an allowable bearing capacity of 24,000 pounds per square foot.

Planned Erosion and Sedimentation Control Practices

Sediment Basin

A sediment basin will be constructed to the south of the property, to the east of the proposed entrance to the site. The water from the disturbed area, approximately 4.5 acres, will be directed through grass and riprap lined channels to the sediment basin before leaving the site. The water from the undisturbed area will drain as it naturally does. See Appendix A.04 for details.

Temporary Gravel Construction Entrance/Exit

A temporary gravel construction entrance will be installed on the southern edge of the property to reduce or eliminate the tracking of sediment onto the existing Poly Street. During wet weather, it may be necessary to wash vehicle tires at this location. The entrance will be graded so that the runoff water will be directed to an excavated drop inlet protection structure. See Appendix A.06 for details.

Temporary Excavated Drop Inlet Protection

There will be eleven excavated drop inlet protection structures will be installed at the proposed locations of the drop inlets. Runoff from the device will be directed into the sediment basin until all excavation procedure have been completed and the pond has been built. (Note: The presence of these devices reduces the sediment load on the sediment basin and provides sediment protection for the pipe. In addition, sediment removal at these points on the site is more convenient than from the basin.) See Appendix A.02 for details.

Land Grading

Heavy grading will be required on the entire disturbed area except for a minimum four foot buffer along both faces of retaining walls. When backfilling the retaining walls, there is a minimum horizontal distance of two-thirds of the wall height away from the wall in which heavy machinery is prohibited from compacting. Instead, the backfill will be compacted using light-hand operated equipment (to prevent wall failure due to extreme horizontal forces.)

A minimum of 15 feet undisturbed buffer zone will be maintained around the perimeter of the disturbed area to reduce water and wind erosion, help contain sediment, and reduce dust.)

Silt Fence

A silt fence will be installed along the southern property line bordering the disturbed area. A total length of 1210 feet of fence will be required. The fence will also protect the topsoil stock pile located to the eastern most point of the disturbed area. See Appendix A.05 for details.

Grass-Lined Channel

A temporary grass-lined channel will be installed along the western and eastern faces of the building. They will meet along the southern face of the building an be directed into the rip rap channel located along the southern property line. There will be a permanent swale located along the western face that leads into a catch basin, and another located along the eastern face of the building that wraps around to the southern face of the building and is directed into another catch basin. See Appendix A.03 for details.

Riprap-Lined Channel

Two temporary riprap lined waterways will be installed on the site. One will be located along the southern property line that runs from east to west into the sediment basin. To control and reduce the sediment that is deposited into the sediment basin, there will be additional areas along the riprap channel horizontally along the bottom of the channel that interrupt the flow only allowing the water to continue along the channels and trapping sediments behind it.

Construction Road Stabilization

As soon as final grade is completed on the entire site and on the entrance road, the proposed location of the road will be stabilized with gravel to provide a protected path for the vehicles that need to access the site. Gravel will also be placed on the West Parking Lot and East Parking Lot until pavement procedures are performed. This will limit the erosion of these areas until paving.

Dust control

Dust will be controlled by segmenting the excavation procedures. The total 4.5 acres of land to be excavated will be broken into sections in which cut and backfilling will be phased appropriately. After final grading has been completed, the disturbed area will be topped with mulch, except for the exterior parking lots and road which will be covered with gravel.

Construction Schedule

1. Obtain plan approval and other applicable permits.
2. Flag the work limits, mark rocks and buffer area for protection.
3. Hold pre-construction conference at least one week prior to starting construction.
4. Install silt fence as the first construction activity.
5. Install sediment basin.
6. Install storm drain with lock and gravel inlet protection at construction entrance/exit.
7. Install temporary gravel construction entrance/exit
8. Clear the entire 4.5 acres of the site that will be excavated, including removing existing structures.
9. Place a mesh on the existing grade to denote a barrier and allow for drainage to store topsoil onsite to the east of the disturbed area within the sediment fence.
10. The fine grading will begin in the north eastern portion of the disturbed area working towards the southwestern corner.

11. Construct retaining walls in 20 foot segments as necessary. Contractor will design temporary sheet piles that will support the soil during the installation of the walls.
12. Rough grade site, stockpile topsoil, construct channels, install catch basin protections. Maintain diversions along top of fill daily.
13. Backfilled areas will be compacted by both heavy machinery and hand machinery in uncompacted layers no larger than 8 inches.
14. After grading elevations are met, mulch the areas immediately.
15. Complete the final grading for the roads and parking. Stabilize with gravel.
16. Install riprap channels and grass lined channels as matching grade.
17. All erosion and sediment control practices will be inspected weekly and after rainfall events. Needed repairs will be made immediately as needed.
18. After the site is stabilized, remove all temporary measures.
19. Add permanent vegetation and replace topsoil after all concrete has been poured.
20. Estimated time before final stabilization – 12 months

Maintenance Plan

1. All erosion and sediment control practices will be checked for stability and operation following every runoff-producing rainfall but in no case less than once every week. Any needed repairs will be made immediately to maintain all practices as designed and installed for their appropriate phase of the project.
2. The sediment basin will be cleaned out when the level of sediment reaches 2.0 ft below the top of the riser. Gravel will be cleaned or replaced when the sediment pool no longer drains properly.

3. Sediment will be removed from within the temporary rip rap channels at the flow interrupters when sediments are no longer being trapped.
4. Riprap channels must be cleaned occasionally when the sediment traps within the bottom of the waterway no longer block sediments.

Drainage Plan

General Narrative

The importance of effective site drainage is critical to the function of a new and existing construction location. Our build site fortunately drains to all one location which makes routing the water towards our primary collection and retention facilities less complicated. However, this also results in increased volumes of runoff which then must be address when considering selection of pipes and drainage infrastructure.

The storm water drainage system is responsible for the following components of building:

1. Safeguard stake holders from illness, injury and protect the various structures assembled on site.
2. Reduce the amount of surface water runoff, external moisture entering habitable structures, discharge of surface grey water systems.
3. Protect adjacent properties from increased water runoff due to the new development.
4. Protect the natural environment from pollutants due to increases traffic facilities and new development and deforestation.

According to the World Health Organization, poor site drainage can result in a tremendous decline in public health and poses a tremendous risk to many locations (Kolsky 134). The risk develops through the formation of stagnant pools of water that is not drained properly as well as polluted runoff that damages water supply infrastructure and contaminates domestic water sources. Harmful runoff entering streams and rivers has the potential to overwhelm natural ecosystems and eventually find its way into reservoirs and aquifers.

Currently, there is much discussion about practices used to reduce storm water runoff using green infrastructure systems. This topic can become charged due to the increased cost to project budgets cause by including green infrastructure to the basis of design. It continues to be a question between the long term advantages versus the initial increased cost of the additions to the base cost of the project. Certain design practices make it mandatory to incorporate a certain percentage of green infrastructures to offset the increased runoff due to development. For example the New York State Department of Environmental Conservation Design Manual for Storm Water Runoff includes values that must be calculated. This value is referred to as the Runoff Reduction Volume (RRv) and is mandatory in considering the design for storm water collection using a storm pond. The RRv is used to determine how much green infrastructure must be used to collect and handle the calculated volume. Today incorporating such infrastructure certainly has proven positive effects, especially when used symbiotically with traffic facilities. The apparent impacts of green roof systems are still not completely clear as there are still some major questions as to their cost to investment values. We assume that for this proposed build location, the minimum requirements for runoff reduction through incorporation of green infrastructure will be in compliance with the NYS DEC design manual.

As discussed in the first paragraph of this narrative, the existing slope on site allows for the ease of collection of storm water runoff. This has proven to be advantageous when deciding upon location for the storm water pond as well as storm water retention facility (located underground). For the safe of this narrative, we will assume that the site orientation is due north of Poly Road and therefore, the entrance to the site is at the southernmost part of the site (downhill). Our pond location allows for natural collection and run off from the regions above the site. We were able to split up the runoff into two main watershed areas, one draining into the pond, the other into a storm water retention facility. Due to the existing slope of our site, the drainage area includes a large portion of the site that is not being developed by our firm. This area of “unaltered” terrain accounts for roughly 25 percent of all runoff within the system. According to the United States Department of Agriculture (USDA) Urban Hydrology for Small Watersheds (Technical Release 55), coefficients for infiltration and runoff were selected in order to reflect appropriate storm water runoff loads.

Preliminary design criteria provided in this submittal was developed using procedures outlined by the New York State Department of Environment Conservation (NYS DEC), United States Department of Agriculture (USDA), New York State Department of Transportation (NYS DoT), as well as *Water Resources Engineering (2010)*, textbook written by Larry W. Mays. Major components of design stem from the NYS DEC Strom Water Management Manual, as well as the USDA TR 55 documents.

Drainage

As previously mentioned, the key to storm water management is effective drainage. On the proposed build location, this is achieved through existing and manmade swales, as well as manmade storm water infrastructure. Our goal was to allow the water to run over as much vegetated area as possible before it reached a catch basin used for collection. This logic supports that a longer time of concentration will significantly lower the intensity of the event. It also further reduces the runoff values due to additional surface area and time to allow for great infiltration. The soil grade on site, Type A well graded sand, is also ideal for allowing infiltration which greatly reduces storm water runoff. Natural swales combined with manmade swales have allowed us to achieve ideal times of concentrations where runoff takes place over vegetated surfaces.

Among the manmade structures aiding storm water collection, the proposed development footprint contains a large number of reinforced concrete retaining walls. The walls are necessary in order achieve grading requirements for the proposed land uses; however, they continue to complicate water drainage. In certain locations, low lying areas between retaining walls make collection in locations necessary and therefore, catch basins are located in these areas. Since the walls must have footing that extend past the frost line (3'-6" below grade) deep precast concrete catch basins were required in order to clear the bottom of the wall and allow for passage of the drainage pipes.

In summary, difficulties presented by the existing grades of the proposed build location are as follows:

- Deep catch basins to allow for pipe clearance of retaining walls

- Catch basins placed in vegetated areas due to unideal grading conditions resulting in drops in elevations between walls

The existing site generally drains toward the developed land and therefore deposits a large portion of rainwater runoff onto the proposed development. This requires storm drainage to be designed for larger areas than actually developed and utilized by the building project. The soil grade on side, well graded sand, is ideal for drainage as well as building; however, slopes steeper than 25 percent of 4:1 require stabilization. The soil is incredibly helpful where drainage is concerned as it allows roughly four inches per hour until it becomes fully saturated which will most likely only happen in a prolonged intense storm. The site soil can be classified as grade A and allows for the maximum advantage of infiltration coefficients allowed by design in each category.

Method

The rough procedure detailed within this section is as follows:

1. Identify natural swales and trends in developed, and undeveloped, landscape to determine appropriate catchment basin placement
2. Research and select appropriate pre-cast manufacture for supply of catch basins
3. Analyze post developed site plan for water runoff and contributing area definition
4. Establish a comprehensive pipe network while remaining sensitive to extreme drops in elevations, engineered obstructions (i.e. retaining walls), and natural obstacles

5. Obtain appropriate IDF curves in order to determine storm water runoff loads on the system for 100 year/ 24-hour storm for the appropriate region (Westchester County, NY)
6. Qualify different types of flow in order to calculate time of concentration for each catch basin
7. Design each pipe through using the design criteria outlined in the USDA storm water design manual
8. Confirm appropriate pipe sizes, slopes and pressure through analysis of hydraulic and energy grade lines
9. Establish appropriate pond design and storm water retention facility to account for storm water runoff

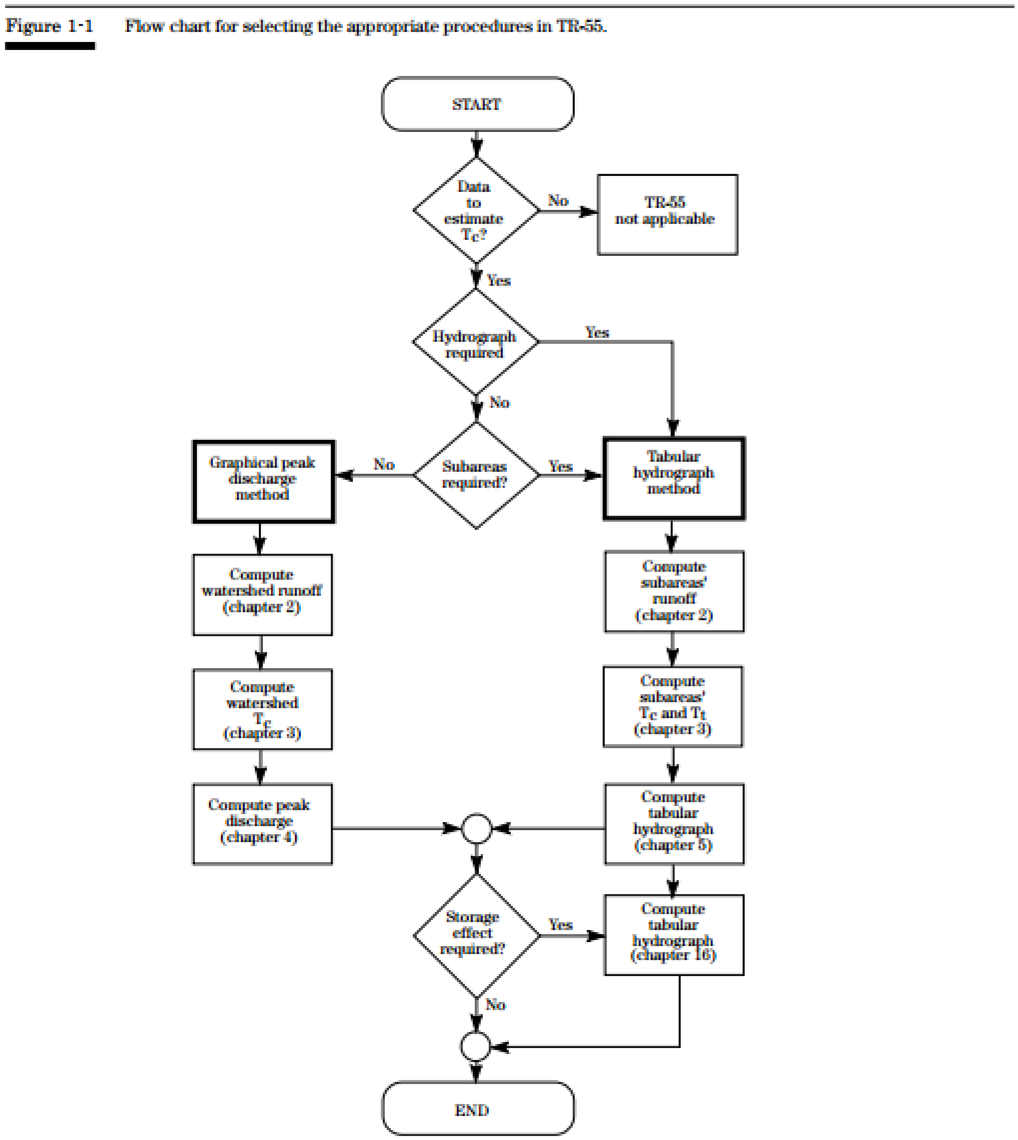
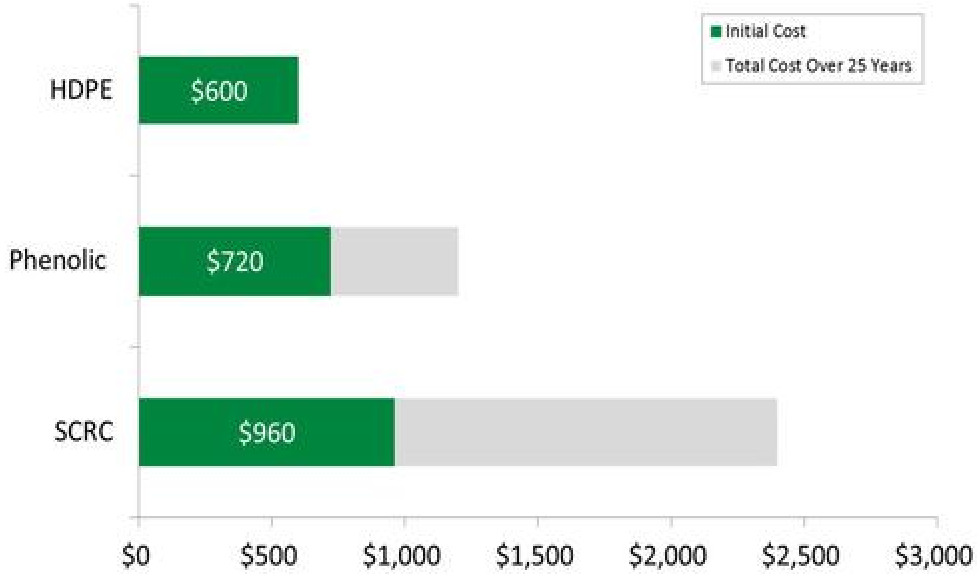


Figure 1-1: USDA TR 55

Precast catch basins designed and provided by Jensen Precast have been recommended for this project due to the ability to provide products that can obtain a depth of 10’-6” below grade. Jensen is also recommended for providing manhole covers, inlets located at traffic facility location and precast headwalls for spillways. The cut sections found in the appendices are examples of recommended products available from this manufacturer. In addition to rectangular grate systems, Jensen Precast also provides options for cylindrical products as well as trench drains. In lieu of structures to be cast in field, precast products allowed our design team to select from quality, pretested products which are manufactured in controlled environments. This provides the client with a quality product backed by a manufactures warranty as well as industry standards in compliance with ASTM and AASHTO requirements.

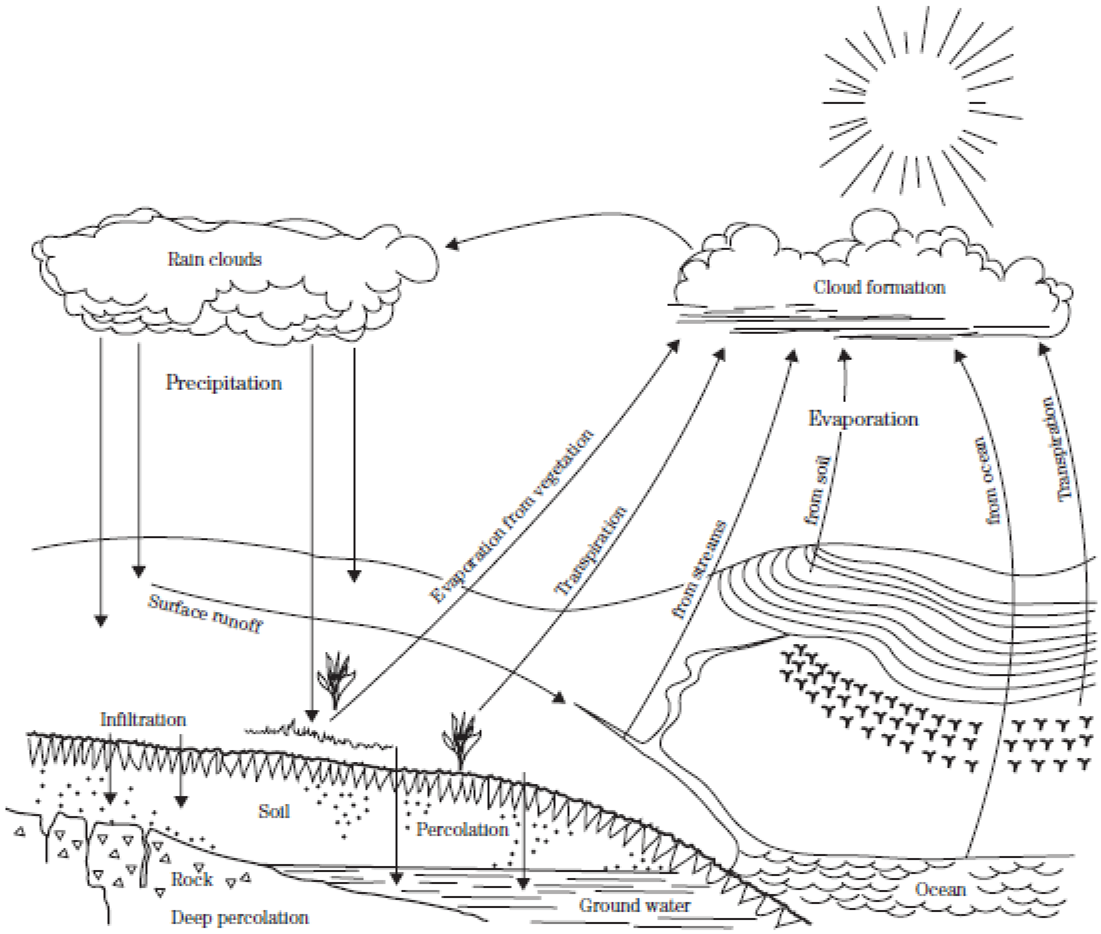
Pipe networks will be installed using High Density Polyethylene pipes. We have researched a recommended supplier who would be appropriate for providing the product for this project. General product specifications provided by the American Concrete Pipe Association allow us to establish design values for using HDPE pipes on site (*see* Appendix). In comparison to concrete pipes, the appropriate class of HDPE pipe performs equal to concrete pipes of the same roughness coefficient for a fraction of the price concerning materials. HDPE pipes are preferred as they tend to be more cost effective as long as their diameters are less than 60 inches. In our case, the soil conditions allow for HDPE to be used as the soil is structurally ideal. Although HDPE pipes are relatively new to the industry when compared to reinforced concrete and metal pipes, the industry claims that they may have a tremendously longer service life than their competitors. It is important to note that installation standards for the proposed HDPE pipes must be to ASTM D 2321 as well as AASHTO section 20 when located adjacent to traffic facilities.



Courtesy of www.scrantonproducts.com

Land Use

Land use is very important to consider for the design of this site. Natural as well as planted vegetation allows for additional infiltration and interception of rainwater which reduces the amount of storm runoff. When calculating the necessary infrastructure to remove excess water during an extreme weather incident, we took into consideration the densely vegetated existing areas, sod planted developed areas, as well as trees planted following construction. Impervious areas have been considered to be rooftop, parking lot, retaining wall surfaces and road infrastructure. These areas may be identified and are classified in further depth within the paving schedule provided by the general civil documents and drawings. These varied areas and their uses are considered especially when calculating the time of concentration which dictates the values use for storm intensity from the IDF curves.



Picture courtesy of USDA National Engineering Handbook

Using the United States Department of Agriculture Part 630 Hydrology National Engineering Handbook, as provided by the Natural Resources Conservation Service, the first component of time of concentration is calculated by using Equation 15-8.

$$T_t = \frac{0.007(n\ell)^{0.8}}{(P_2)^{0.5} S^{0.4}} \qquad \text{(eq. 15–8)}$$

where:
T_t = travel time, h
n = Manning's roughness coefficient (table 15–1)
ℓ = sheet flow length, ft
P_2 = 2-year, 24-hour rainfall, in
S = slope of land surface, ft/ft

This time of concentration refers to sheet flow water, water flow at a depth of less than one inch, over the watershed area collected by a specific catch basin. The travel time, or time of concentration obtained by this equation signifies the very first component of the design criteria. Following this calculation of this value, additional travel times must be considered and added to this value to determine the true time of concentration. This is referred to as the Velocity method of design. The *n*, or Manning's roughness coefficient, is obtained by classifying the percentage of contribution from each type ofter rain and using the values for each in order to calculate the appropriate value.

Table 15–1 Manning's roughness coefficients for sheet flow (flow depth generally ≤ 0.1 ft)

Surface description	n [1]
Smooth surface (concrete, asphalt, gravel, or bare soil)	0.011
Fallow (no residue)	0.05
Cultivated soils:	
Residue cover ≤ 20%	0.06
Residue cover > 20%	0.17
Grass:	
Short-grass prairie	0.15
Dense grasses [2]	0.24
Bermudagrass	0.41
Range (natural)	0.13
Woods: [3]	
Light underbrush	0.40
Dense underbrush	0.80

1 The Manning's n values are a composite of information compiled by Engman (1986).
2 Includes species such as weeping lovegrass, bluegrass, buffalo grass, blue grama grass, and native grass mixtures.
3 When selecting n, consider cover to a height of about 0.1 ft. This is the only part of the plant cover that will obstruct sheet flow.

SCS runoff curve number method

The SCS Runoff Curve Number (CN) method is described in detail in NEH-4 (SCS 1985). The SCS runoff equation is

$$Q = \frac{(P - I_a)^2}{(P - I_a) + S} \qquad \text{[eq. 2-1]}$$

where

Q = runoff (in)
P = rainfall (in)
S = potential maximum retention after runoff begins (in) and
I_a = initial abstraction (in)

Initial abstraction (I_a) is all losses before runoff begins. It includes water retained in surface depressions, water intercepted by vegetation, evaporation, and infiltration. I_a is highly variable but generally is correlated with soil and cover parameters. Through studies of many small agricultural watersheds, I_a was found to be approximated by the following empirical equation:

$$I_a = 0.2S \qquad \text{[eq. 2-2]}$$

By removing I_a as an independent parameter, this approximation allows use of a combination of S and P to produce a unique runoff amount. Substituting equation 2-2 into equation 2-1 gives:

$$Q = \frac{(P - 0.2S)^2}{(P + 0.8S)} \qquad \text{[eq. 2-3]}$$

S is related to the soil and cover conditions of the watershed through the CN. CN has a range of 0 to 100, and S is related to CN by:

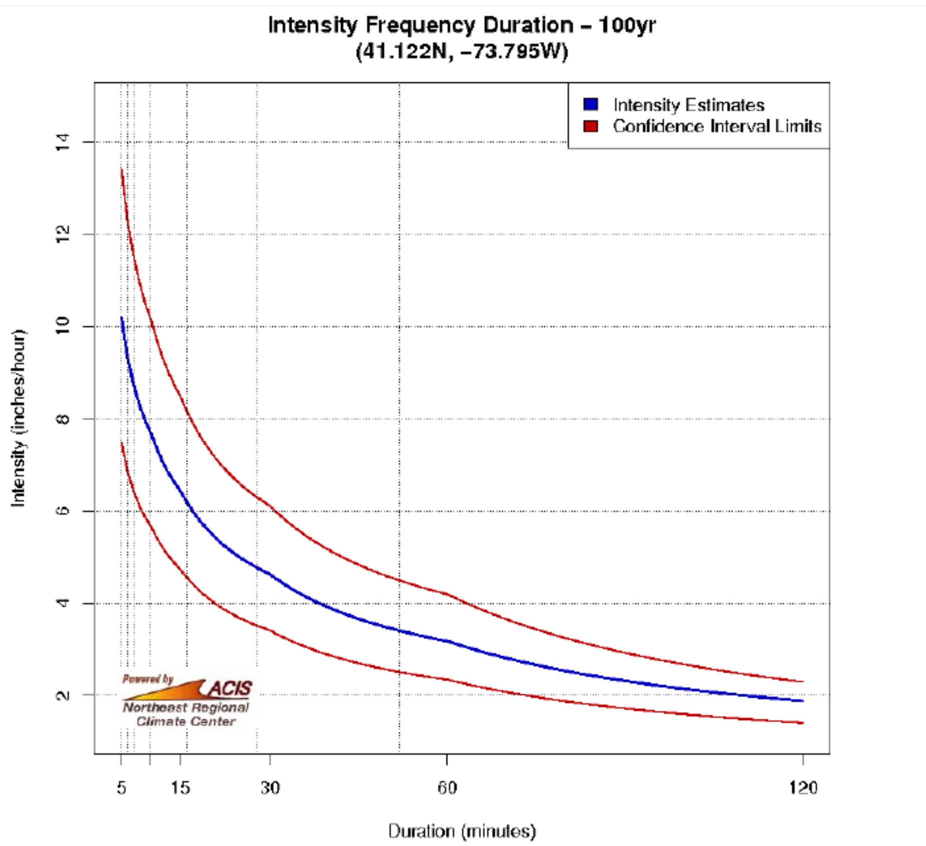
$$S = \frac{1000}{CN} - 10 \qquad \text{[eq. 2-4]}$$

Figure 2-1 and table 2-1 solve equations 2-3 and 2-4 for a range of CN's and rainfall.

For this specific storm water infrastructure, surface flow with transmission losses is considered. From the numerical value obtained the intensity of the event is then extracted from the intensity, duration and frequency curve (See Appendix).

IDF Curves

Intensity, Duration and Frequency curves are used to determine the intensity in inches per hour of a specific storm even. For our design, we have located resource material to allow us to design for the most extreme event required by code, 100 year/ 24 hour event. Once the time of concentration is calculated, that numerical value is used to extract the intensity from the IDF curves and then used to calculate the flow resulting from the storm event from each overland area.



Obtained from http://precip.eas.cornell.edu/

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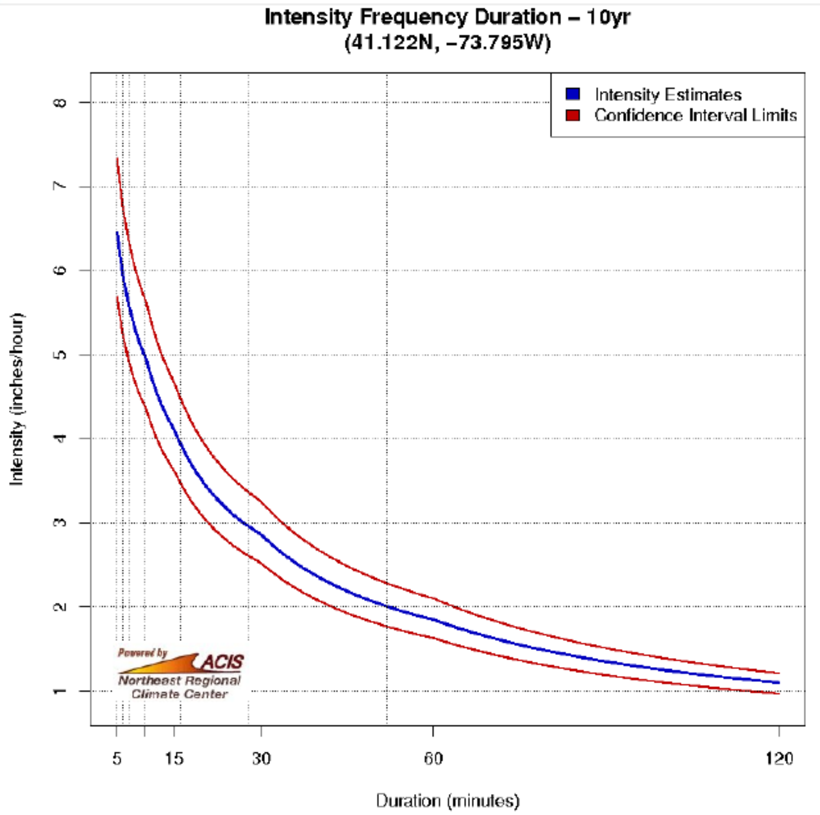


Obtained from http://precip.eas.cornell.edu/

Storm Water Retention

In the event there is not sufficient space on site, or the geographical characteristics of the location make it difficult, in order to have a larger enough pond to house all of the water drained off of the site, a storm water retention facility is required. We have determined that roughly two thirds of the water on the site must be housed in an underground retention center. The following manufacturer has been selected in order to supply the product for the site.

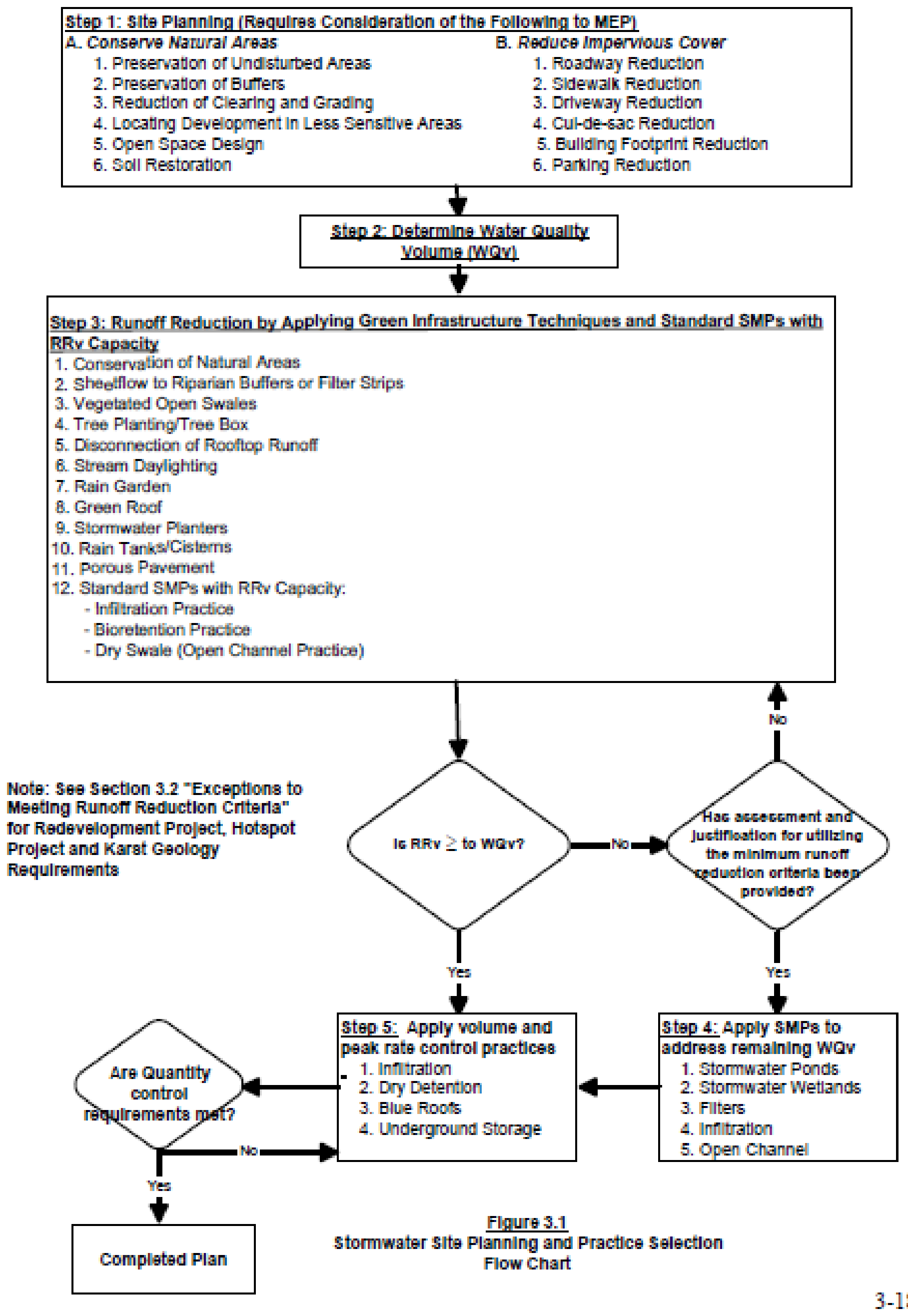


Figure 3.1
Stormwater Site Planning and Practice Selection Flow Chart

3-18

August 2010

NYS DEC Design Manual

Summary of Calculations

Time of Concentration:

$$T_t = \frac{0.007(n * l)^{4/5}}{(P_{100})^{1/2} S^{2/5}}$$

$where$:
$T_t = travel\ time\ of\ water\ over\ land$
$n = Manning's\ roughness\ coeff\ (Table\ 15-1)$
$l = sheet\ flow\ length$
$P_{100} = 100 - yr, 24 - hour\ rainfall\ event$
$S = slope\ of\ watershed\ path$

Slope of Pipe:

$$S_p = \frac{h}{l}$$

$where$:
$h = difference\ of\ pipe\ elevation$
$l = length\ of\ pipe$

Flow (Q):

$Q = ciA$
$where$:
$Q = flow\ from\ contribuiting\ watershed\ area$
$c = runoff\ curve\ number\ or\ retardence\ factor$
$i = intensity\ from\ IDF\ curve\ due\ to\ time\ of\ concentration$
$A = area\ of\ contributing\ portion\ of\ watershed$

Minimum diameter:

$$D_{full} = 12 \times \left(\left(\frac{4^{5/3} Qn}{1.49 \pi S_p} \right)^{\frac{3}{8}} \right)$$

$where$:
$Q = Maximum\ flow\ created\ by\ storm\ event\ in\ the\ pipe$
$n = Manning's\ number\ as\ defined\ by\ pipe\ manufacturer$
$S_p = slope\ of\ the\ pipe$

HGL:

$$HGL = \frac{p}{\gamma} + h$$

$where$:
$HGL = hydraulic\ grade\ line$
$p = static\ pressure$

$\gamma = specific\ weight$
$h = elevation\ height$

<u>EGL:</u>

$$EGL = HGL + \frac{v^2}{2g}$$

$where:$
$EGL = energy\ grade\ line$
$HGL = hydraulic\ grade\ line$
$v = upstream\ velocity$
$g = gravity\ constant$

<u>Velocity:</u>

$$v = \frac{1.49 r^{2/3} S_p^{1/2}}{n}$$

$where:$
$v = average\ velocity$
$r = hydraulic\ radius = \frac{a}{P_w}$

$where:$
$a = cross - sectional\ flow\ area$
$P_w = wetted\ perimeter$

$S = slope\ of\ hydraulic\ gradeline$
$n = Manning's\ coeff\ values\ of\ open\ channel\ flow$

Pipe	Area	From	To	L_p (ft)	S_p	Surface Drainage Area (ft^2)	Surface Drainage Area (Ac)	Incremental Area (ft^2)	c	A*c	sheet flow, n	P100 (in)	Drain Length (L, ft)	Surface Elevation Difference (ft)	S_G	t_{ov} (min) to inlet	V_{min} (fps)	t_p (min)	t_c (min)	t_c Actual	i (in/hr)
A-2	*5*	CB-5	MH-1	89	0.113	8619	0.198	0.198	0.95	0.188	0.012	7.2	123.0	8.0	0.065	0.547	2.5	0.241	0.787	**5.000**	10.19
A-3	*11*	CB-11	MH-1	43	0.082	4190	0.096	0.294	0.51	0.049	0.24	7.2	371.5	29.0	0.078	13.514	2.5	0.127	13.641	**13.641**	6.87
A-4	*n/a*	MH-1	CB-4	94	0.009					0.237							2.5	0.207			6.87
A-5	*13*	CB-13	CB-12	87	0.176	38636.0	0.887	0.887	0.51	0.452	0.24	7.2	315.0	6.0	0.019	20.822	2.5	0.227	21.049	**21.049**	5.58
A-6	*12*	CB-12	CB-2	64	0.167	10473.5	0.240	1.127	0.51	0.575	0.24	7.2	267.0	12.0	0.045	12.940	2.5	0.165	13.105	**21.277**	5.57
A-7	*1*	CB-1	CB-2	89	0.011	7956.0	0.183	0.183	0.95	0.174	0.012	7.2	142.5	7.0	0.049	0.688	2.5	0.341	1.029	**5.000**	10.19
A-8	*2*	CB-2	CB-10	59	0.030	6092.0	0.140	0.322	0.95	0.881	0.012	7.2	360.0	90.0	0.250	0.753	2.5	0.123	0.876	**21.442**	5.55
A-9	*3*	TD-1	CB-10	10	0.290	1941.0	0.045	0.045	0.95	0.042	0.012	7.2	70.8	11.0	0.155	0.248	2.5	0.033	0.281	**5.000**	10.19
A-10	*10*	CB-10	CB-4	25	0.039	3837.0	0.088	0.455	0.51	0.969	0.24	7.2	104.5	21.2	0.203	3.344	2.5	0.086	3.429	**21.527**	5.54
A-11	*4*	CB-4	MH-4	192	0.074	19415.5	0.446	0.901	0.95	1.629	0.012	7.2	219.0	18.0	0.082	0.790	2.5	0.478	1.268	**21.651**	5.5
A-12	n/a	MH-4	pond	138	0.184					1.866										**22.129**	5.44
B-1	*6*	CB-6	MH-2	135	0.137	7474.0	0.172	0.172	0.95	0.163	0.012	7.2	250.0	17.2	0.069	0.943	2.5	0.491	1.434	**5.000**	10.19
B-2	*n/a*	MH-2	MH-3	119	0.060												2.5				
B-3	*7*	TD	CB-9	17	0.136	7982.5	0.183	0.183	0.95	0.174	0.012	7.2	254.0	13.0	0.051	1.074	2.5	0.055	1.130	**5.000**	9.82
B-4	*8+15*	CB-8	CB-9	51	0.037	85710.4	1.968	2.151	0.51	1.178	0.24	7.2	1234.5	141	0.114	30.332	2.5	0.092	30.424	**30.424**	4.73
B-5	*9*	CB-9	MH-3	27	0.054	32360.0	0.743	2.894	0.51	1.556	0.24	7.2	597.5	52.1	0.087	18.909	2.5	0.080	18.990	**30.424**	4.73
B-6	n/a	MH-3	storm	13	0.077	133526.9	3.065			1.719						30.505	2.5				4.73

Q_p (cfs)	Qfull	q/qfull	Diameter Full (in)	Diameter (in)	Diameter Actual (in)	v Actual (fps)	v/vfull 100 yr	vnomo 100 yr	v	vmin	i 10 yr	Q	Q/Qfull	v/vfull 10yr	vnomo 10 yr
1.915	3.750	0.511	7.587	8	**8**	6.13	0.87	13.00	9.13	**4.94**	4.84	0.910	0.243	0.73	12.5
0.337	3.000	0.112	3.239	8	**8**	5.62	0.57	9.10	2.89	**3.13**	4.37	0.214	0.071	0.59	4.9
1.628	2.600	0.626	8.863	8	**8**	7.54	0.87	8.00	5.32	**5.76**		1.124	0.432	0.76	7
2.524	4.300	0.587	7.746	8	**8**	6.34	0.90	13.00	8.19	**4.44**	2.59	1.172	0.272	0.63	13
3.203	8.000	0.400	8.548	10	**10**	6.50	0.80	15.00	7.59	**4.11**	2.57	1.478	0.185	0.69	11
1.768	2.750	0.643	8.747	10	**10**	4.33	0.93	4.30	2.10	**2.28**	4.84	0.840	0.305	0.75	2.8
4.891	11.000	0.445	10.694	15	**15**	7.97	0.80	9.20	3.93	**4.25**	2.57	2.265	0.206	0.77	5.1
0.431	6.800	0.063	3.636	8	**8**	4.94	0.48	19.00	5.61	**3.04**	4.84	0.205	0.030	0.51	11
5.366	12.000	0.447	13.599	15	**15**	4.77	0.80	11.00	7.37	**3.99**	2.57	2.489	0.207	0.67	11
8.960	16.000	0.560	14.634	18	**18**	6.70	0.89	13.90	10.56	**5.72**	2.57	4.187	0.262	0.48	22
10.152	40.000	0.254	12.936	18	**18**	7.98	0.67	22							
1.661	3.400	0.489	6.933	8	**8**	4.59	0.83	10.20	7.64	**4.14**	4.48	0.730	0.215	0.67	11.4
1.661	2.900	0.573	6.238	8	**8**	8.17	0.92	8.20	5.49	**5.95**		0.730	0.252	0.67	8.2
1.710	3.750	0.456	7.022	8	**8**	5.24	0.82	11.80	6.90	**3.74**	4.84	0.843	0.225	0.69	10
5.570	7.000	0.796	10.789	12	**12**	9.27	0.94	9.10	4.40	**4.77**	2.3	2.708	0.387	0.8	5.5
7.362	16.000	0.460	14.431	15	**15**	5.62	0.83	12.50	4.00	**2.16**	2.3	3.580	0.224	0.85	4.7
8.133	26.000	0.313	14.024	18	**18**	6.50	0.75	16.00	9.12	**4.94**	2.3	3.955	0.152	0.57	16

Water Quality Volume:

$$WQ_v = \frac{PR_vA}{12}$$

$where:$

$WQ_v = Water\ Quality\ Volume$

$P = 90\%\ Rainfall\ event\ (Figure\ 4.1 - TR\ 55)$

$R_v = 0.05 + 0.009I$

$where:$

$I = Impervious\ Cover\ (percent)$

$A = Contributing\ area\ to\ pond$

Runoff Reduction Volume:

$$RR_v = \frac{PR_vA_i}{12}$$

$where:$

$WQ_v = Water\ Quality\ Volume$

$P = Rainfall\ event\ (Figure\ 4.1 - TR\ 55)$

$R_v = 0.05 + 0.009I$

$where:$

$I = 100\%$

$A = Total\ area\ of\ impervious\ cover$

24-Hour Extended Detention:

$$Cpv = \frac{\frac{v_s}{v_r} QA}{12}$$

$where:$

$$\frac{v_s}{v_r} = 0.683 - 1.43\left(\frac{q_o}{q_i}\right) + 1.64\left(\frac{q_o}{q_i}\right)^3 - 0.804\left(\frac{q_o}{q_i}\right)^3$$

$where:$

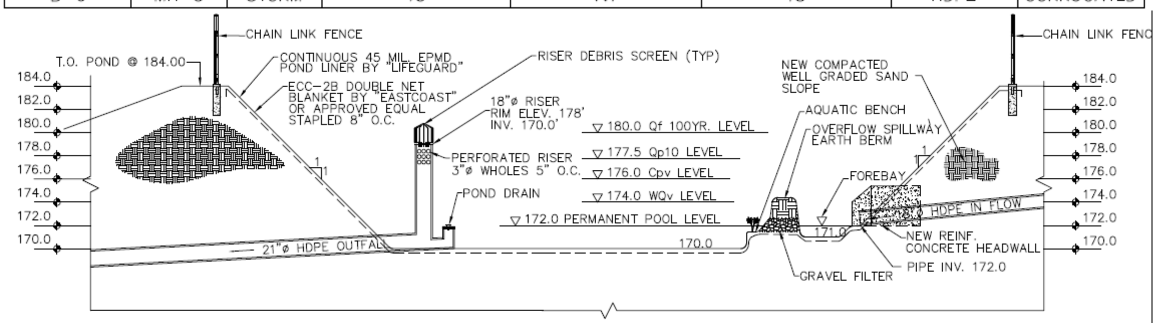
$$\left(\frac{q_o}{q_i}\right) = determined\ Figure\ 8.5, Appendix\ B\ (T = 24hrs, TR - 55)$$

$$Q = determined\ Figure\ 8.3$$

System Summary

PIPE SCHEDULE–NETWORK A							
PIPE	FROM	TO	LENGTH (Lp) FEET	SLOPE (Sp) %	DIAMETER INCHES	MATERIAL/TYPE	
A–1	CB–5	MH–1	89.0	11.3	8	HDPE	CORRUGATED
A–2	CB–11	MH–1	43.0	8.2	8	HDPE	SMOOTH
A–3	MH–1	CB–4	94.0	0.9	8	HDPE	SMOOTH
A–4	CB–13	CB–12	87.0	17.6	8	HDPE	CORRUGATED
A–5	CB–12	CB–2	64.0	16.7	10	HDPE	CORRUGATED
A–6	CB–1	CB–2	89.0	1.1	10	HDPE	SMOOTH
A–7	CB–2	CB–10	59.0	3.0	15	HDPE	SMOOTH
A–8	TD–1	CB–10	10.0	29.0	8	HDPE	CORRUGATED
A–9	CB–10	CB–4	25.0	3.9	15	HDPE	CORRUGATED
A–10	CB–4	MH–4	192.0	7.4	18	HDPE	CORRUGATED
A–11	MH–4	POND	138.0	18.4	18	HDPE	CORRUGATED

PIPE SCHEDULE–NETWORK B							
PIPE	FROM	TO	LENGTH (Lp) FEET	SLOPE (Sp) %	DIAMETER INCHES	MATERIAL/TYPE	
B–1	CB–6	MH–2	135	1.7	8	HDPE	CORRUGATED
B–2	MH–2	MH–3	119	6.0	8	HDPE	SMOOTH
B–3	TD	CB–9	17	13.6	8	HDPE	CORRUGATED
B–4	CB–8	CB–9	51	3.7	12	HDPE	SMOOTH
B–5	CB–9	MH–3	27	5.4	15	HDPE	CORRUGATED
B–6	MH–3	STORM	13	7.7	18	HDPE	CORRUGATED



POND SECTION DETAIL

Paving Plan

Road Characteristics

ZIBA & Associates

The total length of the access road, beginning at Poly Street and extending to the eastern parking lot is 730 feet. The first 584 feet of the road leads to the entrance of the underground parking of the building. There are three curves in the road; the first two, located towards the beginning of the road, are adjacent to each other, and the third one is just before the entrance to the East Parking Lot. Along the centerline of the road, the first curve has a radius of 69’-3” and an arc length of 91’-6”, the second curve has a radius of 119’-0” and an arc length of 39’-4”, and the third curve has a radius of 32’-0” and an arc length of 48’-7”.

Section 43.121.D.2 Article IX: Site Plan Review of the design code provided by the City of Yonkers discusses driveway angle. It states that driveways intended for vehicles going onto a road shall intersect the road at a horizontal angle as near to 90° as site conditions will permit and in no case shall be less than 60° unless acceleration and deceleration lanes are provided. The proposed road is 108° from the horizontal line of the existing road.

According to the design criteria for local roads and streets from the NYSDOT Highway Design Manual, Section 2.7.4.1.A: Design Speed, the range of design speeds of the proposed road on mountainous terrain for an ADT between 250 and 400, is 20-55 mph. According to Exhibit 2-7: Design Criteria for Local Rural Roads of the same manual, the minimum radius of the curve on such a road, considering a maximum superelevation of 6 percent is 81ft. If the design speed is 20 mph and the AADT is 357 (see section “Road Signs” to see the determination of the ADT), the minimum travel lane width is 9 feet. The maximum grade allowable for mountainous terrain is 16 percent. According to the same Highway Design Manual, Section 2.7.4.2.E: Grade, the maximum grade for local streets in residential areas is 15 percent. Section 2.7.4.2.F: Horizontal Curvature requires that for local urban street in heavily built-up residential areas (where the building fronts, drainage, sidewalks, or driveways would be substantially impacted by added superelevation), for a design speed of 20 mph, the minimum curve radius (superelevation at a max of 4 percent) would be 72 ft. According to Exhibit 2-8: Design Criteria for Local Urban Streets, travel lanes with curbing are limited by a minimum width of 10 feet or a desirable width of 11 feet. From the same source, the minimum width of curbed shoulders is zero feet; however it is desirable to have one of 1 to 2 foot width.

Considering all criteria for the road design, it would be most appropriate to post a speed limit of 15 mph. The proposed road is designed with a consistent superelevation of 2 percent, with the radii of 69’-3” and 119’-0” at the entrance of the access road (which accounts for fire access where it would be needed.) The lane widths are 11’-6” and there is 1 foot shoulder on the lowest elevation of the right side of the road. The shoulder is banked at a superelevation of 5 percent. The measurements of proposed variables (posted speed limit, radii of the curves, and superelevation of the road) all fall within the accepted ranges restricted by the codes referenced above.

A ramp is categorized as a stretch of road with a grade within the range of eight to fifteen percent. There is one major ramp on the proposed access road near the entrance from Poly Street, and a minor ramp entering the eastern parking lot. Beginning at Poly Street, the access road is laid out such that a straight way distance of 60’-0” increases at 6.7 percent, followed by the curve of linear 89-0” increasing at an average positive 9.0 percent grade. The first major ramp begins towards the end of the first curve and increases a total of 20 feet in height along 143’-10” of linear length of road, producing an average upgrade of 13.9 percent. A straight road (without turns) continues for 356’-4” at an average increasing grade of 5.6 percent. The second major ramp then begins just after the start of the curve by the East Parking Lot, and rises 10 feet in height along 84’-2” of linear road, producing an average upgrade of 11.9 percent. The profile of the road is shown below including the proposed and existing elevations along the centerline of the proposed road from the entrance of the site to the East Parking Lot.

As required by Section 2.7.4.2.K of the NYSDOT Highway Design Manual, the minimum and maximum travel lane cross slopes in the parking lots are 1.5 percent and 5 percent, respectively.

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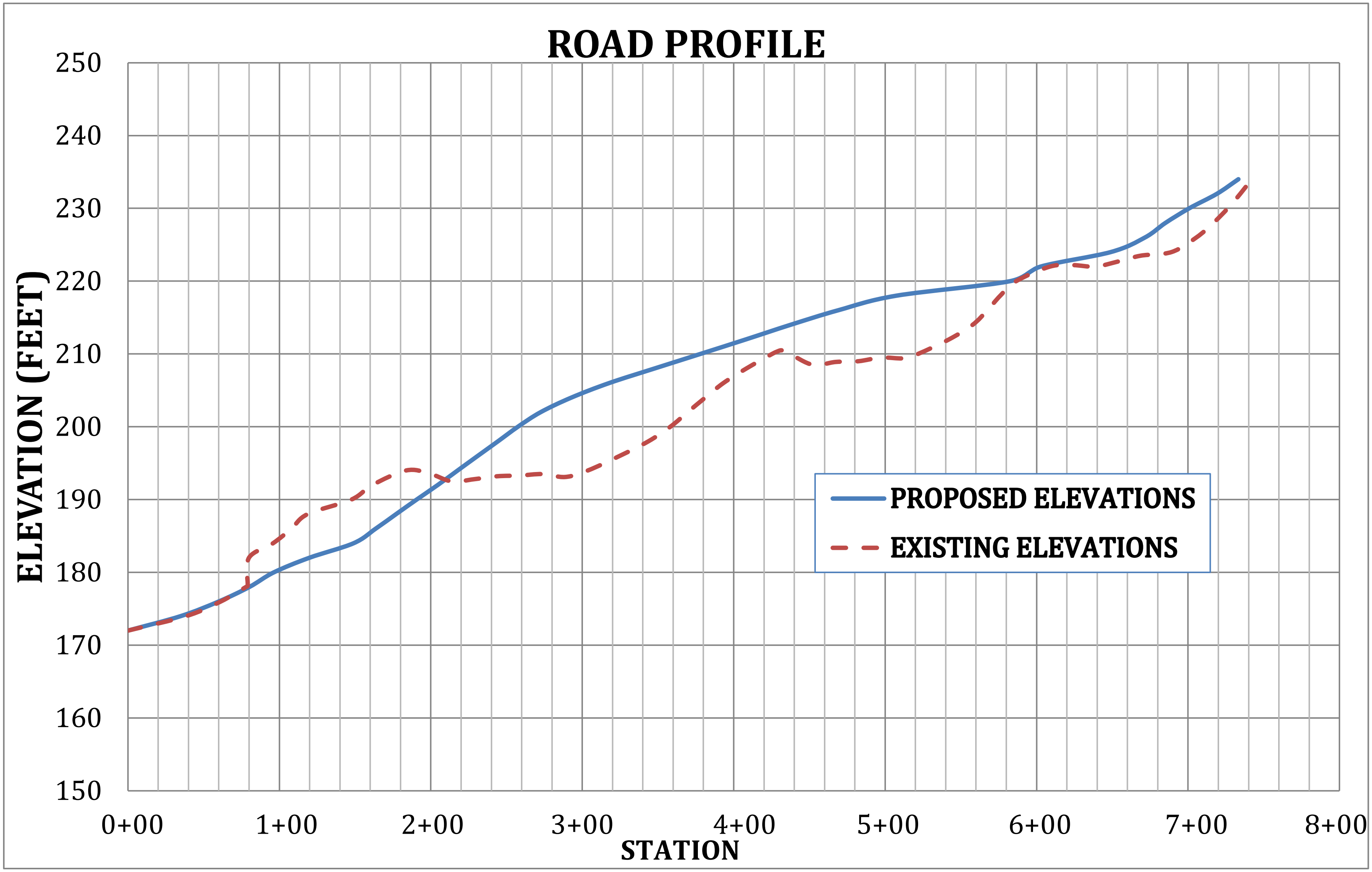

A typical cross-section of would show a schematic of two curbs, two lanes, one shoulder, and one sidewalk. The description that follows is in reference to a cross-section located along the road at the entrance to the western parking lot and facing the eastern lot, but is typical of any cross section that includes a sidewalk. Beginning at the left of the cross-section, there is a 4’-0” sidewalk that is alongside a 0’-6” wide and 0’-6” tall curb. There are then two 11’-6” lanes followed by a 1’-0” shoulder that slopes down at a super elevation of 2 percent. Next is a 0’-6” step up from another curb that is 0’-6” wide, which ties into the grade. The 2 percent bank on the road is adequate for drainage such that any runoff is directed towards the curb and catch basins. The lanes are protected from any buildup of water by the one-foot shoulder that acts like an additional gutter due to the bank of the road. The lane width of 11’-6” is satisfactory for the bypassing of a disabled vehicle in the event that there is one, due to the low volume of traffic on the road. The road itself is composed of an asphalt-macadam base, as described in Section 104.19 Article VI: Pavement Base of the design code provided by the City of Yonkers. The bottom most layer is a 1-foot thick sub base. Above that is a 4-inch gravel fill layer, which is topped by a 7-inch asphalt-macadam base.

Vertical Alignment

Stopping sight distance, S, is a function of the speed the vehicle is travelling (u) in mph, the average driver's perception-reaction time (t) in seconds, the acceleration rate (a) in ft/sec^2, the acceleration due to gravity (g) in ft/sec^2, and the grade of the road (G) as a decimal value:

$$S = 1.47\,u\,t + \left(\frac{u^2}{30\left(\frac{a}{g}\right) - G}\right)$$

For the proposed road, cars would be turning from Poly Street onto the access road, which has an upgrade of percent. This is a typical sag vertical curve. The design speed is u = 20 mph and the grade is 6.7 percent. The assumed values used in the calculations are the average perception-reaction time t = 2.5 sec, the deceleration rate a = 11.2 ft/sec^2, and the acceleration due to gravity g = 32.2 ft/sec^2.

$$S = 1.47 * (20\,mph) * (2.5\text{ s}) + \left(\frac{(20\,mph)^2}{30\left(\frac{11.2\,ft/s^2}{32.2\,ft/s^2}\right) - (0.049)}\right) = 112.01\,ft$$

The absolute difference in grades is represented by the algebraic difference, A:

$$A = |G_2 - G_1|$$

$$A = |(+6.7) - (0)| = 6.7$$

For a Sag vertical curve, the minimum horizontal length of the curve, L, is determined based on its relationship to the stopping sight distance.

For S > L:

$$L_{min} = 2S - \frac{400 + 3.5S}{A}$$

$$L_{min} = 2 * (112.0ft) - \frac{400 + 3.5 * (112.0ft)}{(6.7)} = 105.9\ ft$$

$$S = 112.0ft > L_{min} = 105.9ft \qquad (TRUE)$$

For S < L:

$$L_{min} = \frac{AS^2}{400 + 3.5S}$$

$$L_{min} = \frac{(6.7)(112.0ft)^2}{400 + 3.5(112.0ft)} = 106.2\ ft$$

$$S = 112.0ft > L_{min} = \ 106.2ft \qquad (FALSE)$$

For driver comfort:

$$L_{min} = \frac{Au^2}{46.5}$$

$$L_{min} = \frac{(6.7)(20mph)^2}{46.5} = 57.6ft$$

Based on the above calculations, it can be said that the minimum horizontal length required for the vertical sag curve near the entrance of the access road calculated using a 20 mph design speed is 57.6 feet, based on the design for driver comfort at utmost priority. The horizontal length that was used in the design of the road was 60’-0”.This is adequate if the posted speed limit is below 20 mph.

Parking

The total number of parking spaces required was determined using the Section 43.137.A Article X: Off-Street Parking and Loading of the design code provided by the City of Yonkers. It is stated that the minimum number of required off-street parking spaces for a residential use building is one space per dwelling unit. A supplementary rule of thumb was followed which required an additional one-third of a parking space for every extra bedroom. In the proposed building, there are 79 one-bedroom units, and 2 four-bedroom units, necessitating 83 parking spaces [79+2+ 2*(1/3)*(3) = 83]. According to Section 43-129.A: "In any private open-air parking lot or private parking garage accessory to a multifamily residential use or semipublic parking lot or semipublic parking structure where a total of more than 20 parking spaces are provided to meet the minimum requirements of this chapter, at least 10 percent of such spaces shall be set aside for visitors or shall be unassigned spaces." Since there are 83 required spaces for the residents, at least 92 parking spaces should be provided to account for visitors [(83)*(1+0.10)=92]. To be conservative, a total of 102 parking spaces will be located on the site.

The total number of required handicap spaces is determined based on standards set in 2010 by the Americans with Disabilities Act (ADA). The minimum number of accessible parking spaces is related to the total number of parking spaces required for the building. Since there are 83 proposed spaces, there is a minimum of 4 required accessible parking spaces: 1 van-accessible space with a minimum 96" wide van space and a five to eight foot access aisle on one side, and 3 accessible spaces with a minimum 60” wide space. It is necessary for all accessible parking to be situated as close as possible to the entrances of the building.

The parking is divided into two lots on both sides of the building, as well as an underground parking facility. The West Parking Lot holds 50 spaces, while the East Parking Lot fits 26 spaces. There are 26 spaces located in the underground parking, 4 of which are handicap accessible, since these spaces are nearer to the building entrance than the spaces of the outdoor lots. All parking spaces, regular and handicap accessible, are 9’-0” wide and 18’-0” long.

It was determined that the van accessible parking space would best be positioned directly to the left of the entrance/exit to the underground parking. The other three spaces would be positioned in succession to the right of the entrance. These three spaces are within 22 feet of the stairs or elevator to enter the building. The van accessible space is within 30 feet to entering the building.

For the underground parking facility, the entrance is towards the left of the building. All underground parking spaces are situated towards the outer face of the structure. Upon entering the facility, cars are expected to turn to the right and travel in a counterclockwise circle (the provided turning radius within the facility is 22 feet. By doing so, it is only possible to turn into parking spaces by making right turns.

Road Markings

As denoted by the Manual on Uniform Traffic Control Devices (MUTCD), Chapter 3: Marking, all lines that are drawn onto to road shall be four to six inches in thickness. The proposed plan contains a solid white line 1-foot away from the bottom curb of the road. (Section 3B.06.04 – “if used, right edge of line pavement markings shall consist of a normal solid white line to delineate the right-hand edge of the roadway.”) As described in Section 3B.05, a single solid yellow line shall not be used as a center line marking on a two-way roadway. Therefore, the proposed divider between the two opposing lanes of traffic is a solid double yellow line. Other lines that are painted onto the road way include the white space dividers in the parking lots, the white crosswalks, the white directional arrows in the western parking lot and underground parking facility, and the blue parking space dividers and access zones in the underground parking facility.

Road Signs

Certain signs are required to be placed on the site. Section 5A.-01.01.A of the MUTCD states that a low volume road shall be defined for this Chapter 5 of the Manual (Traffic Control Devices for Low-Volume Roads) as follows: (A) a low-volume road shall be a facility lying outside of built-up areas of cities, towns, and communities, and it shall have a traffic volume of less than 400 AADT; and (B) a low-volume road shall not be a freeway, an expressway, an interchange ramp, a freeway service road, a road on a designated State highway system, or a residential street in a neighborhood. The average annual daily traffic (AADT) can be estimated by assuming that for every parking spot, 3.5 trips will be generated per day. This is a high approximation that generates 357 average daily trips (ADT) [102*3.5 = 357]. The proposed road falls into the category of a low-volume road in this instance, and therefore, can be bound by the limitations set in the chapter.

Section 5B.02 of the MUTCD states that “stop”(R1-1) and “yield”(R2-1) signs should be considered on low-volume roads where engineering judgment or study such that the following condition applies: an intersection of a less-important road with a main road where application of the normal right-of-way rule might not be readily apparent. Based on these regulations, a stop sign will be required at the intersection of the access road and the existing Poly Street. There will be also be a speed limit sign of 15 mph placed at the entrance of the access road. This speed limit is to be maintained throughout the entire road on the site, and driver’s judgment would be used to determine where traffics flows would be less than 15 mph.

Water Supply and Waste Water System

Water Supply System

Fresh Water Supply is necessary to provide a habitable environment for the residents of this development. Fire suppression and safety is incorporated in the water supply system. The system must be able to provide for the necessary demand of the Building occupants, and provide adequate Fire suppression. The municipal water main adjacent to the property is assumed to be a 16 inch supply main operating at 100 psi and providing a total head of 400 feet. Based on these assumptions the velocity in the pipe was calculated using Bernoulli's energy equation and then used to calculate the total flow provided by the municipal water main.

$$Total_{Head} = \frac{P}{\lambda} + Z + \frac{V^2}{2g}$$

$$400\,ft = \frac{100\,\frac{lb}{in^2} \times 144\,\frac{in^2}{ft^2}}{62.4\,\frac{lb}{ft^3}} + 160\,ft + \frac{V^2}{64.4\,\frac{ft}{s^2}}$$

$$V = \sqrt{(400\,ft - 230.77\,ft - 160\,ft) \times 64.4\,\frac{ft}{s^2}} = 24.38\,\frac{ft}{s^2}$$

$$Q = VA = (24.38\,\frac{ft}{s^2}) \times \left(\frac{\pi \times (16in \times \frac{1ft}{12in})^2}{4} \right) = 34.04\,\frac{ft^3}{s} \approx 15277\,\frac{gal}{\min}$$

The municipal supply main is thus capable of providing 15,277 gallons per minute to the development, based on the assumptions made. Fresh water supply pipes must be located 10 feet apart horizontally or four feet apart vertically from sanitary pipes, to prevent contamination. Thus to achieve this the supply pipes were run under the main access road, and placed at elevations lower than the sanitary and drainage pipes. This layout will require vertical riser pipes at the building and the three fire hydrants. Fire hydrants were placed at locations to provide fire suppression at key locations. The first hydrant will be located near the recreational area adjacent to the site entrance. The second and third will be located on the sidewalk adjacent to the two parking facilities.

Fire Suppression

Safety is of the highest concern when calculating the required water supply. The International Organization of Standardization (ISO) provides a formula for calculating the required fire suppression flow for the development.

F = required fire flow in gallons per minute
C = coefficient related to the type of construction
A = total floor area in square feet, including all stories but excluding basements.

$$F = 18 \times C \times \sqrt{A}$$
$$F = 18 \times 0.6 \times \sqrt{3 \times 12496} = 2091.08\ \tfrac{gal}{min}$$
$$F = 0.75 \times 2091.08\ \tfrac{gal}{min} = 1568.25\ \tfrac{gal}{min}$$

For fire resistive buildings the C value is taken as 0.6. For fire resistive structures with protected openings the formula recommends using three standard floors on fire. The equation allows for a reduction of fire flow by 25 percent if the building is a low fire hazard. This development is a concrete residential building, and thus the fire flow was reduced by 25 percent. The adjusted fire flow for the building is calculated to be 1568.25 gallons per minute. A value of 1600 gallons per minute will be used for further calculations.

Demand Estimate

To estimate the fresh water demand for the development, data was taken from the United States Geological Survey (USGS) database. The total domestic water use for Westchester County is 933 million gallons per day, and the total population is 940,807 people. The average water use per person is 99.17 gallons per day per capita.

$$Demand = \frac{93300000\ \frac{gal}{day}}{940807} = 99.17\,\frac{gal}{day \cdot person} \approx 100\,\frac{gal}{day \cdot person}$$

The development will have 79 one-bedroom apartments, and two penthouse units with four bedrooms. The single bedroom units are estimated to house three people per unit, and the penthouse is estimated to house five people per unit. There are two Community rooms with an expected occupancy of 50 people per room, and 10 people using the lobby facilities. A general maintenance value of 750 gallons per day was added to account for general facility operations, and cleaning. Landscaping was estimated to require 10 sprinklers that require 10 gallons of water per day, thus 100 gallons per day in total. The total estimated occupancy of the building is calculated to be 487 people, therefore the calculated average building supply need was 34.34 gallons per minute.

Domestic Water Supply		
Population of Westchester	940,807	Persons
Total Domestic Water Supply	93300000	gal/day
base Supply need	99.17018049	gal/person/day
base Supply need ~	100	gal/person/day
Building Occupancy	257	persons
Community Room 3rd Floor	120	Persons
Community room 1	50	Persons
Community room 2	50	Persons
Lobby	10	Persons
Maintenance	750	gal/day
Average flow	49450	gal/day
Average flow	34.34	gal/min

EPANET Model

The demand for water will peak during the morning and evening hours and the system was designed for this peak demand. The Land Development handbook table 26.4 (p.655) recommends a peak hour factor of 4.50 for residential developments with occupancy of 500 people. The peak flow was calculated to be 154.53 gallons per minute. EPANET was used to model the pressurized supply pipe network. Copper pipe was used which has a roughness coefficient of 130 when calculating for head loss using the Hazen Williams equation. The model was run under different conditions to simulate fires at different locations. The first analysis was conducted with demands for landscaping, building fire suppression, and building peak demand. The following simulations included hydrant flows, however, it is assumed that not all hydrants will be active at the same time. Therefore, the model was conducted three times, once for each hydrant flow. The hydrant flow rate used was 1,500 gallons per minute. For proper building operation, the water supplied to the building must have a pressure greater than 30 psi. Fire hydrants are required to have a minimum pressure of 20 psi, and all hydrant supply pipes must be a minimum of six inches in diameter. The results of the analysis are presented below.

Network Table - Nodes- Fire Flow - Building Peak				
Node ID	Elevation ft	Demand GPM	Head ft	Pressure psi
Municipal Supply	160	-1754.6	400	100
Junc 2	162.9	0	398.5	102.09
Junc 3	166.5	0.07	393.85	98.51
Junc Hydrant1	167.6	0	393.85	98.03
Junc 4	172.29	0	391.35	94.92
Junc 5	178.53	0	388.67	91.05
Junc 6	203.85	0	375.58	74.41
Junc Hydrant2	205.35	0	375.58	73.76
Junc 7	209.45	0	369.79	69.48
Junc 8	209.45	0	369.79	69.48
Junc 9	210.05	0	369.78	69.21
Junc BuildingMAIN	211.75	154.53	368.62	67.97
Junc BuildingFIRE	213.45	1600	362.7	64.67
Junc Hydrant3	215.7	0	369.78	66.76

Network Table - Links-Fire Flow - Building peak					
Link ID	Length ft	Diameter in	Flow GPM	Velocity fps	Unit Headloss ft/Kft
Pipe 1	29	8	1754.6	11.2	51.74
Pipe 2	90	8	1754.6	11.2	51.74
Pipe Hydrant1	11	6	0	0	0
Pipe 3	48.17	8	1754.53	11.2	51.74
Pipe 4	52	8	1754.53	11.2	51.74
Pipe 5	252.92	8	1754.53	11.2	51.74
Pipe Hydrant2	15.02	6	0	0	0
Pipe 6	112	8	1754.53	11.2	51.74
Pipe Fire-Main	40	6	1600	18.16	177.11
Pipe 7	5	8	154.53	0.99	0.57
Pipe 8	12	8	154.53	0.99	0.57
Pipe Building	17	3	154.53	7.01	68.33
Pipe Hydrant3	113	6	0	0	0

The first simulation includes the peak building supply, landscaping, and fire flow demand. These demands will be present in all simulations. Fire flow must always be present to provide suppression in case of a fire. Under these loadings the lowest pressure is at the connection to the building fire supply at a pressure of 64.67 psi, which is adequate.

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Network Table - Nodes- Fire Flow - Building Peak - Hydrant 1				
Node ID	Elevation ft	Demand GPM	Head ft	Pressure psi
Municipal Supply	160	-3254.6	400	100
Junc 2	162.9	0	395.29	100.69
Junc 3	166.5	0.07	380.67	92.8
Junc Hydrant1	167.6	1500	378.94	91.57
Junc 4	172.29	0	378.17	89.21
Junc 5	178.53	0	375.48	85.34
Junc 6	203.85	0	362.4	68.7
Junc Hydrant2	205.35	0	362.4	68.05
Junc 7	209.45	0	356.6	63.76
Junc 8	209.45	0	356.6	63.76
Junc 9	210.05	0	356.59	63.5
Junc BuildingMAIN	211.75	154.53	355.43	62.26
Junc BuildingFIRE	213.45	1600	349.52	58.96
Junc Hydrant3	215.7	0	356.59	61.05

Network Table - Links- Fire Flow - Building Peak - Hydrant 1					
Link ID	Length ft	Diameter in	Flow GPM	Velocity fps	Unit Headloss ft/Kft
Pipe 1	29	8	3254.6	20.77	162.48
Pipe 2	90	8	3254.6	20.77	162.48
Pipe Hydrant1	11	6	1500	17.02	157.16
Pipe 3	48.17	8	1754.53	11.2	51.74
Pipe 4	52	8	1754.53	11.2	51.74
Pipe 5	252.92	8	1754.53	11.2	51.74
Pipe Hydrant2	15.02	6	0	0	0
Pipe 6	112	8	1754.53	11.2	51.74
Pipe Fire-Main	40	6	1600	18.16	177.11
Pipe 7	5	8	154.53	0.99	0.58
Pipe 8	12	8	154.53	0.99	0.57
Pipe Building	17	3	154.53	7.01	68.33
Pipe Hydrant3	113	6	0	0	0

In this simulation the hydrant flow of 1500 gallons per minute was allocated to fire hydrant 1. The total required flow of 3,254.6 gallons per minute from the supply main is below the provided flow thus the system has adequate supply. The lowest pressure is located at the building fire supply connection at 58.96 psi, which is adequate. The pressure at hydrant 1 is 91.57 psi, which is greater than the minimum of 20 psi.

Network Table - Nodes - Fire Flow - Building Peak - Hydrant 2				
Node ID	Elevation ft	Demand GPM	Head ft	Pressure psi
Municipal Supply	160	-3254.6	400	100
Junc 2	162.9	0	395.29	100.69
Junc 3	166.5	0.07	380.67	92.8
Junc Hydrant1	167.6	0	380.67	92.32
Junc 4	172.29	0	372.84	86.9
Junc 5	178.53	0	364.39	80.53
Junc 6	203.85	0	323.3	51.76
Junc Hydrant2	205.35	1500	320.94	50.08
Junc 7	209.45	0	317.5	46.82
Junc 8	209.45	0	317.5	46.82
Junc 9	210.05	0	317.49	46.56
Junc BuildingMAIN	211.75	154.53	316.33	45.32
Junc BuildingFIRE	213.45	1600	310.42	42.02
Junc Hydrant3	215.7	0	317.49	44.11

Network Table - Links - Fire Flow - Building Peak - Hydrant 2					
Link ID	Length ft	Diameter in	Flow GPM	Velocity fps	Unit Headloss ft/Kft
Pipe 1	29	8	3254.6	20.77	162.48
Pipe 2	90	8	3254.6	20.77	162.48
Pipe Hydrant1	11	6	0	0	0
Pipe 3	48.17	8	3254.53	20.77	162.47
Pipe 4	52	8	3254.53	20.77	162.47
Pipe 5	252.92	8	3254.53	20.77	162.47
Pipe Hydrant2	15.02	6	1500	17.02	157.16
Pipe 6	112	8	1754.53	11.2	51.74
Pipe Fire-Main	40	6	1600	18.16	177.11
Pipe 7	5	8	154.53	0.99	0.58
Pipe 8	12	8	154.53	0.99	0.57
Pipe Building	17	3	154.53	7.01	68.33
Pipe Hydrant3	113	6	0	0	0

Hydrant flow at hydrant 2 will result in lower pressures however the lowest pressure is 42.02 psi at the building fire supply connection, which is still adequate. The pressure at hydrant 2 is 50.08 psi which is greater than the minimum of 20 psi that is required.

Network Table - Nodes - Fire Flow - Building Peak - Hydrant 3				
Node ID	Elevation ft	Demand GPM	Head ft	Pressure psi
Municipal Supply	160	-3254.6	400	100
Junc 2	162.9	0	395.29	100.69
Junc 3	166.5	0.07	380.67	92.8
Junc Hydrant1	167.6	0	380.67	92.32
Junc 4	172.29	0	372.84	86.9
Junc 5	178.53	0	364.39	80.53
Junc 6	203.85	0	323.3	51.76
Junc Hydrant2	205.35	0	323.3	51.11
Junc 7	209.45	0	305.1	41.45
Junc 8	209.45	0	304.87	41.35
Junc 9	210.05	0	304.31	40.84
Junc BuildingMAIN	211.75	154.53	303.15	39.6
Junc BuildingFIRE	213.45	1600	298.02	36.64
Junc Hydrant3	215.7	1500	286.55	30.7

Network Table - Links - Fire Flow - Building Peak - Hydrant 3					
Link ID	Length ft	Diameter in	Flow GPM	Velocity fps	Unit Headloss ft/Kft
Pipe 1	29	8	3254.6	20.77	162.48
Pipe 2	90	8	3254.6	20.77	162.48
Pipe Hydrant1	11	6	0	0	0
Pipe 3	48.17	8	3254.53	20.77	162.47
Pipe 4	52	8	3254.53	20.77	162.47
Pipe 5	252.92	8	3254.53	20.77	162.47
Pipe Hydrant2	15.02	6	0	0	0
Pipe 6	112	8	3254.53	20.77	162.47
Pipe Fire-Main	40	6	1600	18.16	177.11
Pipe 7	5	8	1654.53	10.56	46.41
Pipe 8	12	8	1654.53	10.56	46.41
Pipe Building	17	3	154.53	7.01	68.33
Pipe Hydrant3	113	6	1500	17.02	157.16

Flow at hydrant 3 is the critical case. The system experiences the lowest pressure of 30.7 psi at hydrant 3. This is greater than the minimum of 20 psi, thus the system can provide adequate pressure and flow to all necessary junctions.

Waste Water System

Waste water systems are designed to convey sewage generated by land use. In a residential development the waste is generally sanitary sewage such as organic material from kitchens, bathrooms, and laundry facilities. Sewage contains a high level of bacteria and viruses, therefore a sanitary system is required to provide a healthy environment for the development. Sewage is 99.9 percent water and 0.1 percent dissolved and suspended solids, therefore sewage in sanitary pipes have hydraulic characteristics of water. In residential developments the peak flow will occur around the same time as the peak flow for the water supply system, offset by the time of concentration.

The sanitary sewer system must be designed to accommodate the peak flow. The system must also have a minimum velocity during off peak operation in the pipe of one foot per second to prevent any solids from accumulating within the pipe; this is referred to as scouring velocity. The average flow rate from the building supply will be used as the base flow rate for the sanitary system, 34.34 gallons per minute, or 0.0765 cubic feet per second. The peak factor for sanitary systems can be taken from figure 25.2 in the Land Development Handbook (p.602). For a flow of 0.0765 cfs the peak factor is 6.1 and the peak plow is 0.4667 cfs.

$$Q = 6.1 \times 0.0765 \tfrac{ft^3}{s} = 0.4667 \tfrac{ft^3}{s}$$

The flow through sanitary pipes is gravity fed. This means that the total head cannot exceed the crown of the pipe otherwise the pipe will flow under pressure. The system on this site is constrained due to the elevation of the building and the elevation of the municipal sanitary pipe. Manning's equation was used to calculate the required diameter to accommodate the peak flow. Cast iron pipe was selected with a Manning's friction coefficient of 0.013. Based on the required diameter a standard diameter is chosen and the ratios are used to calculate the depth of flow in the selected pipes. Figure 25.6 from the Land Development Handbook was used to determine the depth of flow and velocities.

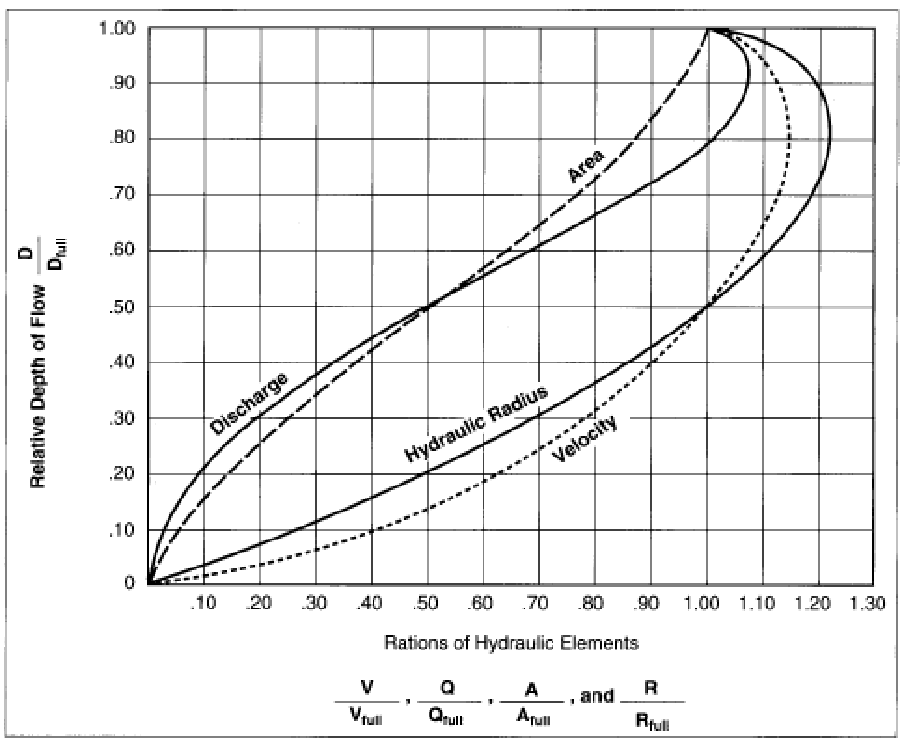


FIGURE 25.6 Nomograph of selected hydraulic elements for circular pipe flowing partially full.

Pipe	Manhole No.		Flow Q (cfs)			Length (ft)	Pipe Invert Elevation (ft)		Slope
	From	To	Average	Peak Factor	Peak		start	end	
1	SB	S2	0.0765	6.1	0.4667	48	222	218	0.08
2	S2	S1	0.0765	6.1	0.4667	250	218	192	0.10
3	S1	S0	0.0765	6.1	0.4667	150	192	169.03	0.15

Pipe	Slope	Manning n value	Diameter (in)	
			Required	Standard
1	0.083	0.013	3.76	4
2	0.104	0.013	3.61	4
3	0.153	0.013	3.36	4

Pipe	Depth of Flow (in)		HGL (ft)		EGL (ft)	
	Depth Peak	Depth average	Start	End	Start	End
1	3.08	1.12	222.26	218.26	222.87	218.87
2	2.88	1	218.24	192.24	218.30	192.95
3	2.48	0.88	192.21	169.24	192.28	170.15

Pipe	Velocity (ft/s)		
	full	Partial	Scouring
1	6.30	6.30	3.53
2	7.03	6.75	3.80
3	8.53	7.68	4.35

The lowest scouring velocity is 3.53 feet per second, which is higher than the required one foot per second. The velocity head is relatively small, the HGL falls within the pipe diameter at peak flow. The EGL is 13.47 inches above the HGL at pipe 3, which has the highest slope. Under peak flow, pipe 1 flows at 77 percent capacity, which is acceptable.

Architects Vision

The most influential designs that are present in the building were inspired from several buildings around the New York City area, which is near the building site in Westchester. The building was kept to a modest height and kept bulky, bringing a sense of modernism to the architectural design. The combination of curtain wall and brick façade was inspired by several recent Manhattan renovation projects, mainly the Chelsea Market building located in the Meat Packing District of Manhattan. The brick veneer was used to maintain a classical appearance. The curtain wall façade was brought into the design in order to inject a modern impression on the building. In addition to injecting a modern impression on the building, the curtain wall façade also allows natural light to infiltrate the vast open apartment spaces. The most impressive portion of the building would be the two duplex penthouse apartments. These apartments are set back from the original building to allow each of the units to have a rather large terrace. There is a full curtain wall system that encloses these apartments, allowing for constant sunlight in all the rooms and breathtaking views.

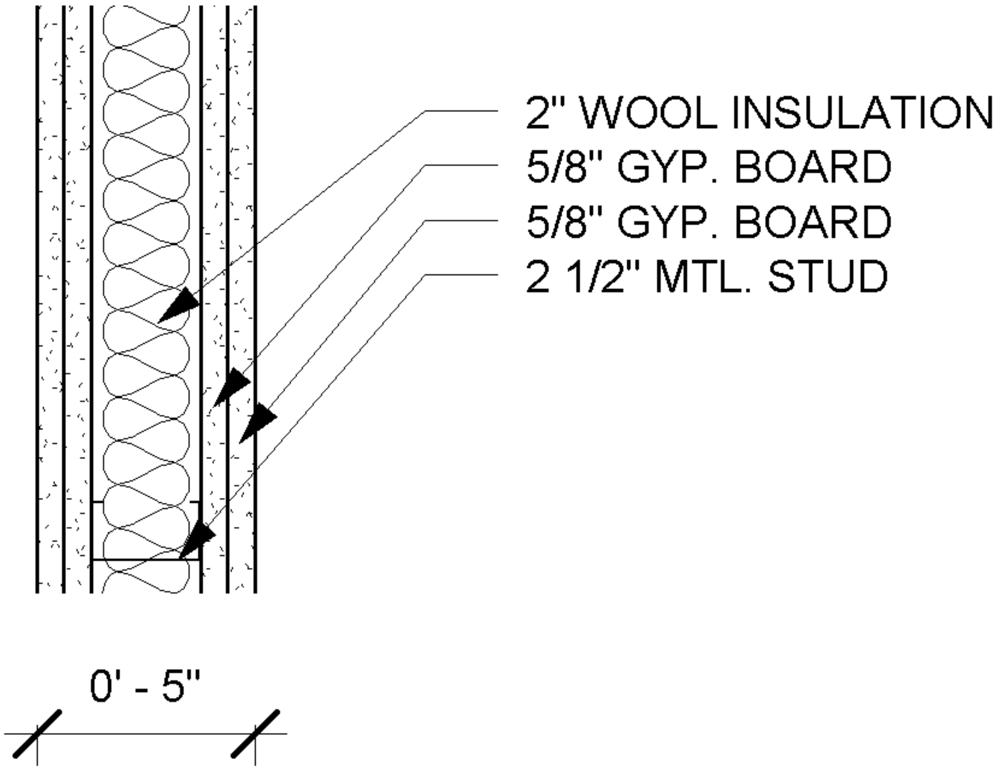
Methodology

The owner requested a residential building that would be an excellent investment and grow in value over the years. In order to make this possible, we wanted to create apartment units that would be very spacious and would be customizable if an owner decides to adjust the floor plan. We also wanted the building to complement the given site. In order to achieve this, we made a much wider building that ran with the grade, as opposed to fighting it. The gross footprint of the building is just over 12,200 square feet. We began designing the building from the core. We estimated that we would need three elevators to serve the estimated size of the building; two passenger elevators and one service elevator. After placing two sets of stairs within the core, we surrounded them by two18-inch thick C-shaped shear walls. These would start from the elevator pit, 4-feet below grade, and run all the way up the height of the building to the roof. The core had a size of 20-feet by 40-feet. Using the maximum column-to-column clear span given to us by the structural engineer (22 feet) we began to lay out the columns from the core in the garage level. We determined that 22 foot maximum spacing would not be sufficient for the passenger cars to maneuver in the building garage. We increased the second floor slab to 10-inch thickness, which gave us a maximum column-to-column clear span of 27.5 feet. We laid out the columns from the core and determined the parking layout. We were able to fit 26 parking spaces, which was more than the owner had requested. We laid the columns for floors two through 16 up the building spaced at around 20 feet. Utilizing the maximum column spacing allowed us to produce generous open floor plans, while keeping the number of columns to a minimum. We determined where columns would not fit into the architect's vision and moved them when necessary. The majority of the columns were hidden in walls and closets throughout the building. Twelve inch by 36 inch rectangular columns were used for their ability be hidden in walls. Circular columns were used on the corners of the buildings where they would be exposed. These were chosen to be aesthetically pleasing and compliment the beautiful landscape view allowed by the glass façade chosen.

The second floor would serve as the main entrance to the building. Since the building would be partially underground, two entrances were provided on the wings of the second floor to allow access to the east and west parking lots. The rest of the second floor was dedicated to building management space, mechanical rooms, storage, and community rooms. The third floor elevation coincided with the grade on the north end of the building. An additional community space was designed on this floor, which also provided access to the bocce court located behind the building. A simplistic symmetrical floor plan with eight units per floor was chosen due to its economic feasibility for floors four through twelve. A symmetrical floor plan allows for the construction of the building to operate quickly and economically. Formwork was made to be as simplistic as possible and allow for reuse and repeatability. The symmetrical floor plans allowed for all of the mechanical, electrical, and plumbing runs to be shared between adjacent units, decreasing the square footage lost. We also chose all of the units, except the two penthouse units to be one bedroom. It was determined that producing eight one-bedroom units on each floor would produce the highest real estate value. The fourteenth floor would contain a setback, at the request of the owner. This floor would be the main entrances to the two four-bedroom penthouse units. Both of the penthouse units have their own roof promenades, which allow the users to appreciate the majestic Yonkers landscape.

Interior Partitions

In order to provide maximal comfort and safety, the type of interior partitions used in the project were carefully chosen so that there will be excellent noise insulation and ample fire protection. The entire interior partitions will be kept to an industry standard in regards to fire rating and will be composed of several layers that make up the partition. The fire resistance on the interior partition assembly will be two hour rated. The partition will consist of a 2.5 inch metal stud core with two layers of 5/8 inch thick gypsum wall board. The assembly will also contain an internal wool mineral insulation that will help with sound insulation and also with fire safety. The image below shows a plan view of the two hour fire rated assembly.


2" WOOL INSULATION
5/8" GYP. BOARD
5/8" GYP. BOARD
2 1/2" MTL. STUD
0' - 5"

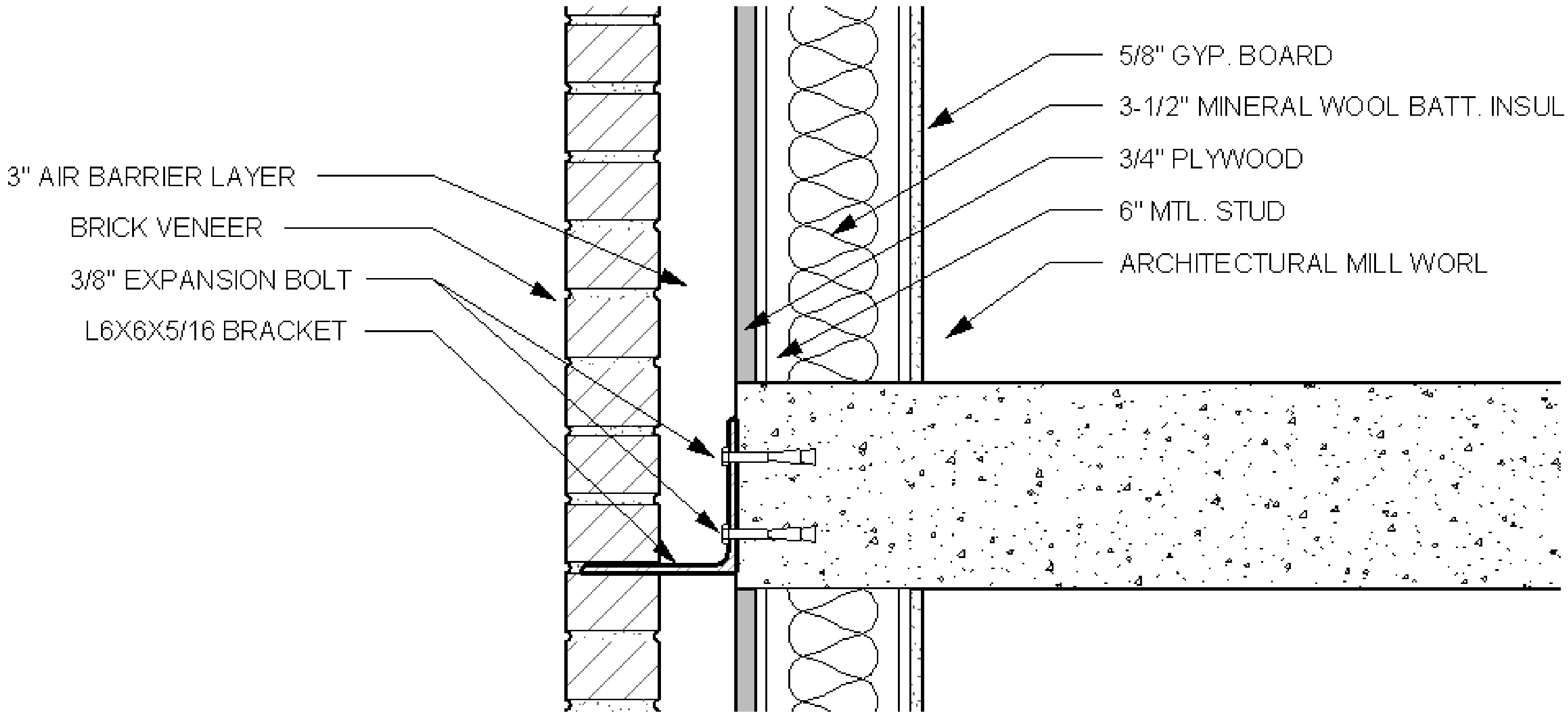
Façade

Curtain Wall

The curtain wall product specifications are provided by FM Grahm Architectural Products, York, PA:

Brick Veneer

The building will feature large amounts of architectural brick veneer along its major exterior walls as well as on the promenade and roof. The brick used will be of standard size and mount to the building using metal angles on every floor slab edge that is exposed to the exterior. The assembly will be rather thick to allow for adequate strength against the exterior elements. The main structural portion of the wall will be a six inch thick metal stud that will be covered by one layer of gypsum wall board on the interior side and one layer of plywood on the exterior weather exposed side. There will be thin sheeting along the exterior to add extra protection from the elements as well as insulation on the interior of the wall. Below is a section view of this wall assembly.


5/8" GYP. BOARD
3-1/2" MINERAL WOOL BATT. INSUL
3/4" PLYWOOD
6" MTL. STUD
ARCHITECTURAL MILL WORL
3" AIR BARRIER LAYER
BRICK VENEER
3/8" EXPANSION BOLT
L6X6X5/16 BRACKET

Structural Plan

The International Building Code 2006 was followed for the design of all structural components of the development site, including the design of the retaining walls. A reinforced concrete frame structure was chosen for the residential building for several reasons. A reinforced concrete building was chosen because they are much stronger, last longer, and allowed the architect more flexibility when designing the units, when compared to steel. Reinforced concrete is much more fire resistant and resistant to corrosion than steel.

A two-way slab flat plate system was used for the entire building. This allowed the architect to utilize larger clear ceiling heights and column-to-column spacing. No exterior beams or drop panels would be used, in order to decrease the costs and time of construction by utilizing minimal formwork. Also, varying column sizes was minimized in order to allow formwork to be reused throughout the entire structure. The flat plate system was designed to resist all gravity loads, including snow, dead, and live loads. The special reinforced concrete shear wall (18-inch on the first floor and 12-inch for the remainder of the building) around the core of the building was designed to withstand all lateral loads, including wind and earthquake. The concrete frame was assumed to resist no lateral loads, but assumed to transfer all of the lateral loads to the shear wall. The slab was assumed to be a rigid diaphragm during the analysis.

All columns, slabs, and beams were designed using linear static analysis. They were designed to resist the gravity loads in the elastic region of the reinforced concrete.

The shear wall was designed to resist the earthquake loads in the inelastic region, possibly leading to inelastic deformations in the event of an earthquake. This was chosen because it is more economical to design reinforced concrete to resist seismic loads in the inelastic region, especially because the residential building is not a particularly high-risk structure such as a hospital. The building was also checked for torsional irregularities between stories.

We decided to use an eight-inch thick slab. From ACI 9.5.32, we were constrained to a maximum column-to-column clear space of 22 feet.

$$8" = \frac{l_n}{33} \rightarrow l_n = 22'$$

The architect determined this would not be sufficient column-to-column clear distances to achieve proper parking in the underground garage, so we increased the slab above the garage, floor 2, to 10-inch thick slab. This would require transfer beams where the columns were offset from the core. The maximum clear spacing was increased to 27.5'.

$$10" = \frac{l_n}{33} \rightarrow l_n = 27.5'$$

Loads

The building was designed with an Occupancy Category of II, from IBC Table 1604.5.

Dead Load

Values for minimum dead loads were taken from ASCE 7-05 Tables C3-1 and C3-2. A value of 150 pounds per cubic foot was used for the unit weight of reinforced concrete. The architect suggested the partitions on each floor to be made up of five-inch thick apparatuses with two layers of 5/8-inch gypsum board on both sides and 2 ½-inch metal stud in between them. A unit weight of 20 pounds per square foot will be used for the partitions. The load in pounds per square foot for the partitions was determined by finding the total load due to partitions on the entire floor and dividing it over the entire area of the floor, as shown. The mechanical engineer estimated a load of eight pounds per square foot for mechanical allowance. Three pounds per square foot was added for miscellaneous loads that may have been overlooked. The architect determined that the majority of the flooring cover on each floor would be tile, so a load of 16 pounds per square foot was used on all floors. Bituminous smooth surface waterproofing (two pounds per square foot) and half-inch rigid insulation (one pound per square foot) were specified by the architect for placement on the roof levels.

A force of 48 pounds per square foot was used for the exterior stud wall with brick veneer. A force of 15 pounds per square foot was used for the exterior glass façade.

The structural engineer decided that it would be much more economical if steel frame partitions with two layers of 5/8 inch gypsum board were used instead of the CMU partitions. They would decrease the dead load on each floor by almost 80 pounds per square foot on average, and still provide a two hour fire rating.

Garage Floor Load	*(psf)*
5" slab (6"/12)(150 psf)	75
Mechanical Allowance	8
Miscellaneous	3
Total	**86psf**

Second Floor Load	*(psf)*
10" slab (10"/12)(150 psf)	125
Mechanical Allowance	8
*Steel Frame with 2 layer 5/8" gypsum board	17
Floor Finish (Ceramic Tile)	16
Miscellaneous	3
Total	**169psf**

*CMU Partitions (Full Grout 8" wythes) with plaster on both sides = 85psf

Floor Area = 12,162 sqft
Wall height = 10'
Linear feet of CMU Wall per floor = 1330'
Total load over entire floor = (1330')(10')(85psf)=1,130,500 lbs
Distributed Load = (1,130,500lbs)/(12,162sqft)=**93 psf, much heavier**

*Steel Frame with 2 layers of 5/8" gypsum board on both sides = 14psf

Floor Area = 12,162 sqft
Wall height = 10'
Linear feet of partitions per floor = 1330'
Total load over entire floor = (1330')(10')(14psf)=186,200 lbs
Distributed Load = (186,200lbs)/(12,162sqft)=**15.3 psf use 17 psf**

Floors 3-12 Loads	*(psf)*
8" slab (8"/12)(150 psf)	100
Mechanical Allowance	8
*Steel Frame with 2 layer 5/8" gypsum board	17
Floor Finish (Ceramic Tile)	16
Miscellaneous	3
Total	**144psf**

*Steel Frame with 2 layers of 5/8" gypsum board on both sides = 14psf

Floor Area = 12,162 sqft
Wall height = 10'
Linear feet of partitions per floor = 1330'
Total load over entire floor = (1330')(10')(14psf)=186,200 lbs

Distributed Load = (186,200lbs)/(12,162sqft)=**15.3 psf use 17 psf**

Floors 14-15 Loads	*(psf)*
8" slab (8"/12)(150 psf)	100
Mechanical Allowance	8
*Steel Frame with 2 layer 5/8" gypsum board	17
Floor Finish (Ceramic Tile)	16
Miscellaneous	3
Total	**144psf**

*Steel Frame with 2 layers of 5/8" gypsum board on both sides = 14psf

Floor Area = 6047.22 sqft

Wall height = 10'

Linear feet of partitions per floor = 716'

Total load over entire floor = (716')(10')(14psf)=100,240 lbs

Distributed Load = (100,240lbs)/(6047.22sqft)=**16.5 psf use 17 psf**

Promenade Floor Load	*(psf)*
8" slab (8"/12)(150 psf)	100
Bituminous Smooth Surface Waterproofing	2
½" Rigid Insulation	1
Floor Finish (Ceramic Tile)	16
Miscellaneous	3
Total	**122psf**

Roof Floor Load	*(psf)*
8" slab (8"/12)(150 psf)	100
Bituminous Smooth Surface Waterproofing	2
½" Rigid Insulation	1
Miscellaneous	3
Total	**106psf**

Live Load

The minimum live loads were determined using the International Building Code Table 1607.1.

Garage Floor Load

The garage is only designed to fit passenger cars, so it will be designed for a live load of 50 pounds per square foot, as stated in Section 1607.6 of the IBC.

Second Floor Load

The second floor is the main entrance floor with the lobby of the building, so it will be designed for a live load of 100 pounds per square foot. There are also light storage units for the residents of the building located on this floor; these areas will be designed for a live load of 125 pounds per square foot. The entire floor will be designed for 125 pounds per square foot for simplicity and to account for any possible alterations in the future.

Third Floor Load

The third floor contains a community center in the northeast corner, which will also provide outdoor access to the bocce courts in the rear of the building. The community room live load minimum is 100 pounds per square foot. The residential units on the floor will be designed for a live load of 40 pounds per square foot.

Floors 4-12 Load

Floors four through twelve have the same floor plans and are all residential dwellings. They are private rooms so it will be designed for a live load of 40 pounds per square foot, except in the core where the elevators are where a live load of 100 pounds per square foot will be used since they are public areas.

Floors 14-15 Load

Floors fourteen and fifteen are all residential penthouse dwellings. They are private rooms so they will be designed for a live load of 40 pounds per square foot, except in the core where the elevators are where a live load of 100 pounds per square foot will be used since they are public areas.

Promenade Floor Load

The promenade will be open to the residents of the penthouse, so it will be designed for a live load of 60 pounds per square foot.

Roof Floor Load

Access to the roof is only for maintenance to be performed, so it will be designed for a live load of 20 pounds per square foot.

Summary

Summary	
Floor	**Load (psf)**
Garage Core	50 100
Second Floor Storage	100 125
Third Floor Community Room	40 100
Floors 4-12 Core	40 100
Floors 14-15 Core	40 100
Roof Promenade	60
Roof	20

Live Load Reduction Factors

Live load reduction will be utilized during the design process for column members in accordance with IBC Section 1607.9. The live load reduction factors utilized are shown below and were taken from IBC Table 1607.9.1.

Live Load Reduction Factors	
Member	**K_{LL}**
Interior Columns	4
Exterior Columns without Cantilever Slabs	4
Edge Columns with Cantilever Slabs	3
Corner Columns with Cantilever slabs	2
Two-Way Slabs	1

Snow Load

The International Building Code Section 1608 stipulates snow loads to be designed in accordance with Chapter 7 of ASCE 7-05. The balanced snow load will first be calculated for the roof areas, followed by the unbalanced snowdrift that will occur on the roof and roof promenade in both the east/west and north/south directions.

Ground Snow Load P_g=30psf (ASCE 7-05 Figure 7-1)
-Yonkers, NY
Exposure Factor C_e=1 (ASCE 7-05 Table 7-2)
-Partially Exposed due to parapets on roof and promenade
Thermal Factor C_t=1 (ASCE 7-05 Table 7-3)
-Due to fact that building will be heated
Importance Factor I=1.0 (ASCE 7-05 Table 7-4)
-Occupancy Category II

Surface Roughness C: Open Terrain with scattered obstructions

Flat Roof Snow Load

$P_f = \text{ } flat\ roof\ snow\ load\ = 0.7C_eC_tIP_g$
$P_f = 0.7 * 1 * 1 * 1 * 30\ psf\ = \mathbf{21\ psf}$

$Snow\ Density = \gamma = 0.13P_g + 14 \leq 30$
$\gamma = 0.13(30psf) + 14 = \mathbf{17.9\ pcf} < 30$

Height of balanced snow load $= h_b = \frac{snow\ load}{\gamma} = \frac{21\ psf}{17.9\ pcf} = 1.173\ ft$
$h_{parapet} = 4\ ft$

Unbalanced Snow Load

Level 16 (Roof) & 17 (Top of Core) Snow Drift

North/South Elevation View

Drift Against Core of Building

Clear Height above balanced snow = $h_c = 10\ ft - 1.173\ ft = 8.827\ ft$

$$\frac{h_c}{h_b} = \frac{8.827\ ft}{1.173\ ft} = 7.52$$

$7.52 > 0.2 \therefore drift\ load\ must\ be\ considered\ against\ building\ core$

Leeward

- $l_u = Length\ higher\ roof\ = 40\ ft$
- $h_d = 0.43\sqrt[3]{l_u}\left(\sqrt[4]{P_g + 10}\right) - 1.5$
- $h_d = 0.43\sqrt[3]{40\ ft}\left(\sqrt[4]{30psf + 10}\right) - 1.5 = 2.2\ ft$
- $Drift\ Height = 2.2\ ft$

Windward

- $l_u = Length\ lower\ roof\ = 26\ ft$
- $h_d = 0.43\sqrt[3]{l_u}\left(\sqrt[4]{P_g + 10}\right) - 1.5$
- $h_d = 0.43\sqrt[3]{26\ ft}\left(\sqrt[4]{30psf + 10}\right) - 1.5 = 1.70\ ft$
- $Drift\ Height = \frac{3}{4}h_d =\ 1.23\ ft$

Design Drift Height=2.2ft

- $P_d = Snow\ Drift\ Load = (h_d + h_b) * \gamma$
- $P_d = (2.2 + 1.173)\ ft * 17.9\ pcf\ = \mathbf{60.4\ psf}$
- $Width = W = 4 * h_d = 4 * 2.2\ ft$
- $\boldsymbol{W = 8.8\ ft}$

Drift Against Parapet

$h_c = 4\ ft - 1.173\ ft = 2.827\ ft$

$$\frac{h_c}{h_b} = \frac{2.827\ ft}{1.173\ ft} = 2.41$$

$2.41 > 0.2 \therefore drift\ load\ must\ be\ considered\ against\ parapet$

Windward

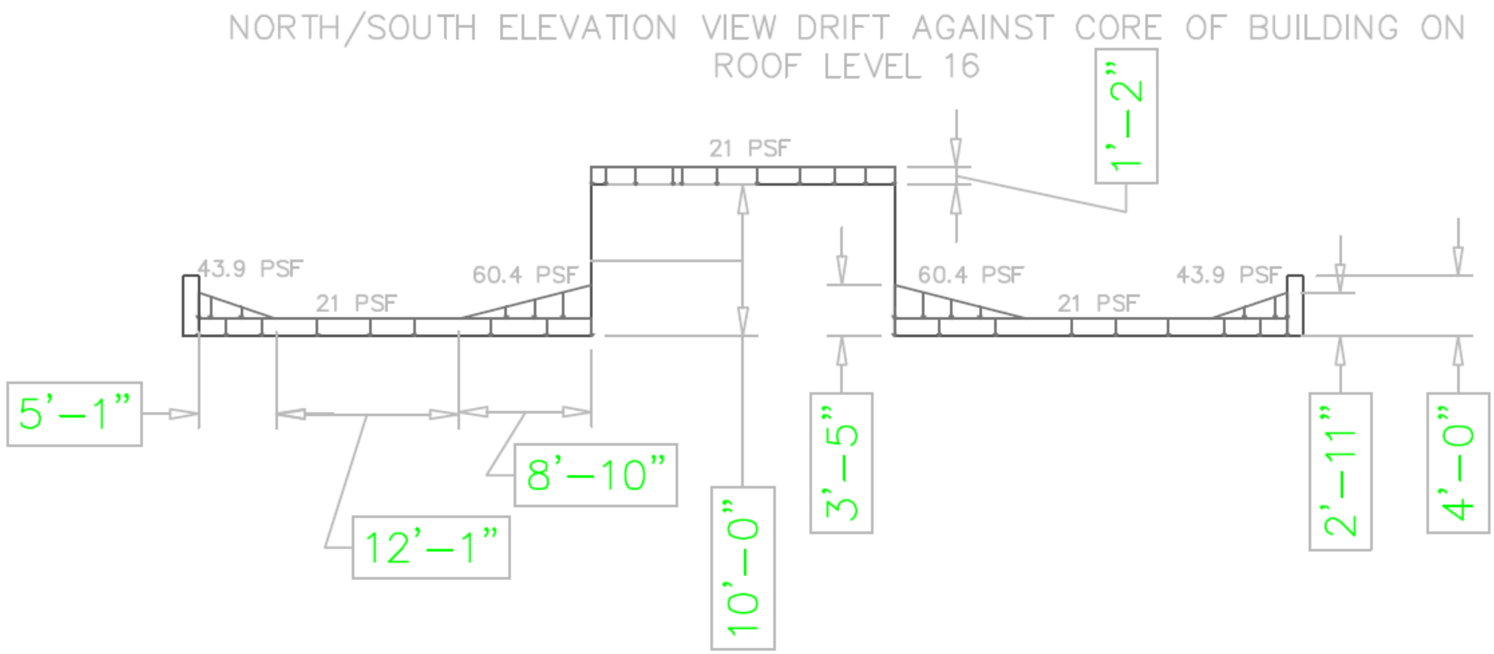
- $l_u = Length\ lower\ roof\ = 26\ ft$
- $h_d = 0.43\sqrt[3]{l_u}\left(\sqrt[4]{P_g + 10}\right) - 1.5$
- $h_d = 0.43\sqrt[3]{26\ ft}\left(\sqrt[4]{30psf + 10}\right) - 1.5 = 1.70\ ft$
- $Drift\ Height = \frac{3}{4}h_d =\ 1.28\ ft$

Design Drift Height=1.28ft

- $P_d = Snow\ Drift\ Load = (h_d + h_b) * \gamma$
- $P_d = (1.28 + 1.173)ft * 17.9\ pcf\ = \mathbf{43.9\ psf}$
- $Width = W\ = 4 * h_d = 4 * 1.28\ ft$
- $\boldsymbol{W = 5.12\ ft}$

Width of Balanced Snow at center (21psf)

$$Width = 26 - 8.8 - 5.12 = \mathbf{12.08ft}$$


NORTH/SOUTH ELEVATION VIEW DRIFT AGAINST CORE OF BUILDING ON
ROOF LEVEL 16
21 PSF
1'–2"
43.9 PSF
21 PSF
60.4 PSF
60.4 PSF
21 PSF
43.9 PSF
5'–1"
8'–10"
12'–1"
10'–0"
3'–5"
2'–11"
4'–0"

East/West Elevation View

Drift Against Core of Building

$h_c = 10\ ft - 1.173\ ft = 8.827\ ft$

$\frac{h_c}{h_b} = \frac{8.827\ ft}{1.173\ ft} = 7.52$

$7.52 > 0.2 \therefore drift\ load\ must\ be\ considered\ against\ building\ core$

Leeward

- $l_u = Length\ higher\ roof = 20\ ft$
- $h_d = 0.43\sqrt[3]{l_u}\left(\sqrt[4]{P_g + 10}\right) - 1.5$
- $h_d = 0.43\sqrt[3]{20\ ft}\left(\sqrt[4]{30psf + 10}\right) - 1.5 = 1.44\ ft$
- $Drift\ Height = 2.2\ ft$

Windward

- $l_u = Length\ lower\ roof = 26\ ft$
- $h_d = 0.43\sqrt[3]{l_u}\left(\sqrt[4]{P_g + 10}\right) - 1.5$
- $h_d = 0.43\sqrt[3]{26\ ft}\left(\sqrt[4]{30psf + 10}\right) - 1.5 = 1.70\ ft$
- $Drift\ Height = \frac{3}{4}h_d = \ 1.23\ ft$

Design Drift Height=1.44 ft

- $P_d = Snow\ Drift\ Load = (h_d + h_b) * \gamma$
- $P_d = (1.44 + 1.173)ft * 17.9\ pcf = \mathbf{46.8psf}$
- $Width = W = 4 * h_d = 4 * 1.44ft$
- $\boldsymbol{W = 5.76\ ft}$

Drift Against Parapet

$h_c = 4\ ft - 1.173\ ft = 2.827\ ft$

$\frac{h_c}{h_b} = \frac{2.827\ ft}{1.173\ ft} = 2.41$

$2.41 > 0.2 \therefore drift\ load\ must\ be\ considered\ against\ parapet$

Windward

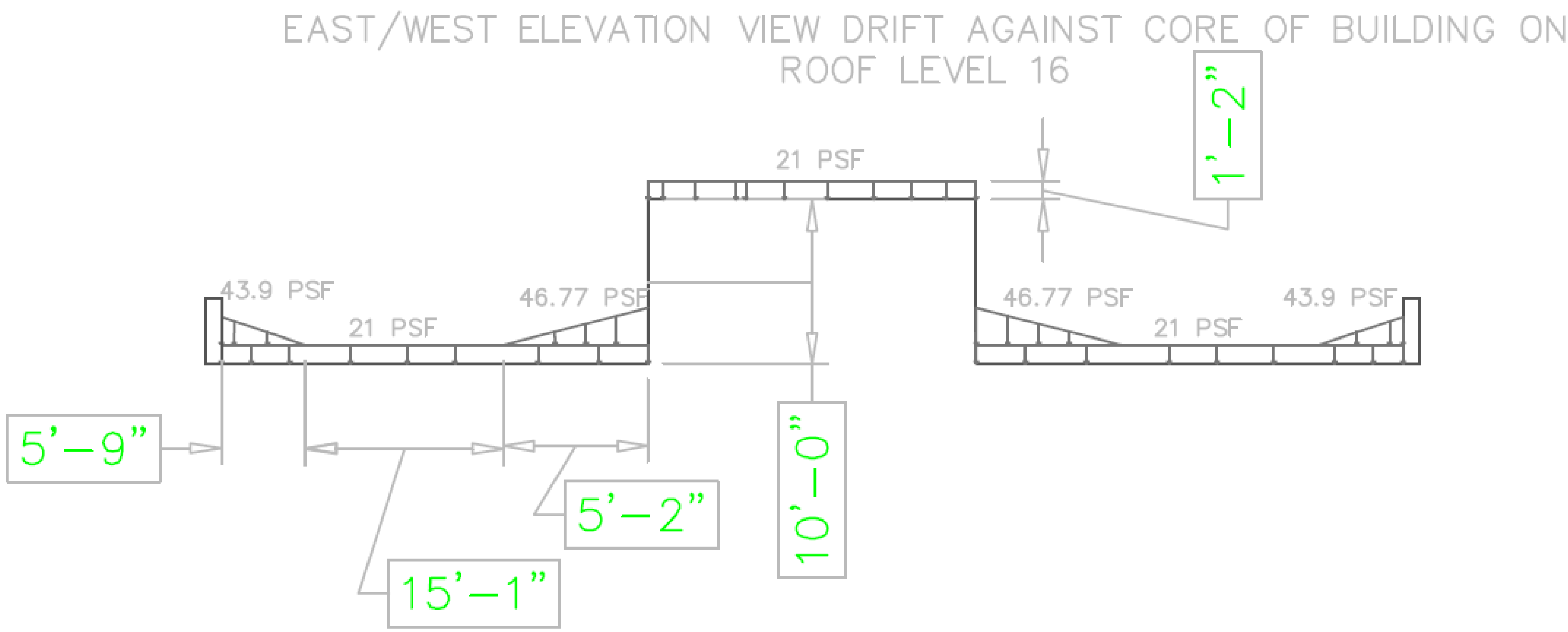
- $l_u = Length\ lower\ roof = 26\ ft$
- $h_d = 0.43\sqrt[3]{l_u}\left(\sqrt[4]{P_g + 10}\right) - 1.5$
- $h_d = 0.43\sqrt[3]{26\ ft}\left(\sqrt[4]{30psf + 10}\right) - 1.5 = 1.70\ ft$
- $Drift\ Height = \frac{3}{4}h_d = \ 1.28\ ft$

Design Drift Height=1.28ft

- $P_d = Snow\ Drift\ Load = (h_d + h_b) * \gamma$
- $P_d = (1.28 + 1.173) ft * 17.9\ pcf = \mathbf{43.9\ psf}$
- $Width = W = 4 * h_d = 4 * 1.28\ ft$
- $\boldsymbol{W = 5.12\ ft}$

Width of Balanced Snow at center (21psf)

$$Width = 26 - 5.76 - 5.12 = \mathbf{15.12 ft}$$


EAST/WEST ELEVATION VIEW DRIFT AGAINST CORE OF BUILDING ON ROOF LEVEL 16
21 PSF
1'–2"
43.9 PSF
46.77 PSF
21 PSF
46.77 PSF
21 PSF
43.9 PSF
5'–9"
5'–2"
10'–0"
15'–1"

Level 14 (Promenade) & 16 (Roof) Snow Drift

North/South Elevation View

Drift Against Core of Building

$h_c = 21.33\ ft - 1.173\ ft = 20.12\ ft$

$\frac{h_c}{h_b} = \frac{20.12\ ft}{1.173\ ft} = 17.18$

$17.18 > 0.2 \therefore drift\ load\ must\ be\ considered\ against\ building\ core$

Leeward

- $l_u = Length\ higher\ roof = 92\ ft$
- $h_d = 0.43\ \sqrt[3]{l_u}\left(\sqrt[4]{P_g + 10}\right) - 1.5$
- $h_d = 0.43\ \sqrt[3]{92\ ft}\left(\sqrt[4]{30psf + 10}\right) - 1.5 = 3.38\ ft$
- $Drift\ Height = 3.38\ ft$

Windward

- $l_u = Length\ lower\ roof = 16\ ft$
- $h_d = 0.43\ \sqrt[3]{l_u}\left(\sqrt[4]{P_g + 10}\right) - 1.5$
- $h_d = 0.43\ \sqrt[3]{16\ ft}\left(\sqrt[4]{30psf + 10}\right) - 1.5 = 1.22\ ft$
- $Drift\ Height = \frac{3}{4}h_d = \ 0.92\ ft$

Design Drift Height=3.38 ft

- $P_d = Snow\ Drift\ Load = (h_d + h_b) * \gamma$
- $P_d = (3.38 + 1.173)ft * 17.9\ pcf = \mathbf{81.5}\boldsymbol{psf}$
- $Width = W = 4 * h_d = 4 * 3.38ft$
- $\boldsymbol{W} = \mathbf{13.52}\ \boldsymbol{ft}$

Drift Against Parapet

$h_c = 4\ ft - 1.173\ ft = 2.827\ ft$

$\frac{h_c}{h_b} = \frac{2.827\ ft}{1.173\ ft} = 2.41$

$2.41 > 0.2 \therefore drift\ load\ must\ be\ considered\ against\ parapet$

Windward

- $l_u = Length\ lower\ roof = 16\ ft$
- $h_d = 0.43\ \sqrt[3]{l_u}\left(\sqrt[4]{P_g + 10}\right) - 1.5$
- $h_d = 0.43\ \sqrt[3]{16\ ft}\left(\sqrt[4]{30psf + 10}\right) - 1.5 = 1.22\ ft$
- $Drift\ Height = \frac{3}{4}h_d = \ 0.92\ ft$

Design Drift Height=0.92ft

ZIBA & Associates

- $P_d = Snow\ Drift\ Load = (h_d + h_b) * \gamma$
- $P_d = (0.92 + 1.173)ft * 17.9\ pcf = \mathbf{37.6\ psf}$
- $Width = W = 4 * h_d = 4 * 0.92\ ft$
- $\mathbf{W = 3.68\ ft}$

Width of Balanced Snow at center (21psf)

$$Width = 16 - 3.68 - 13.52 = \mathbf{-1.2ft\ Not\ possible}(\mathbf{Drifts\ overlap})$$

They intersect at 23.7psf

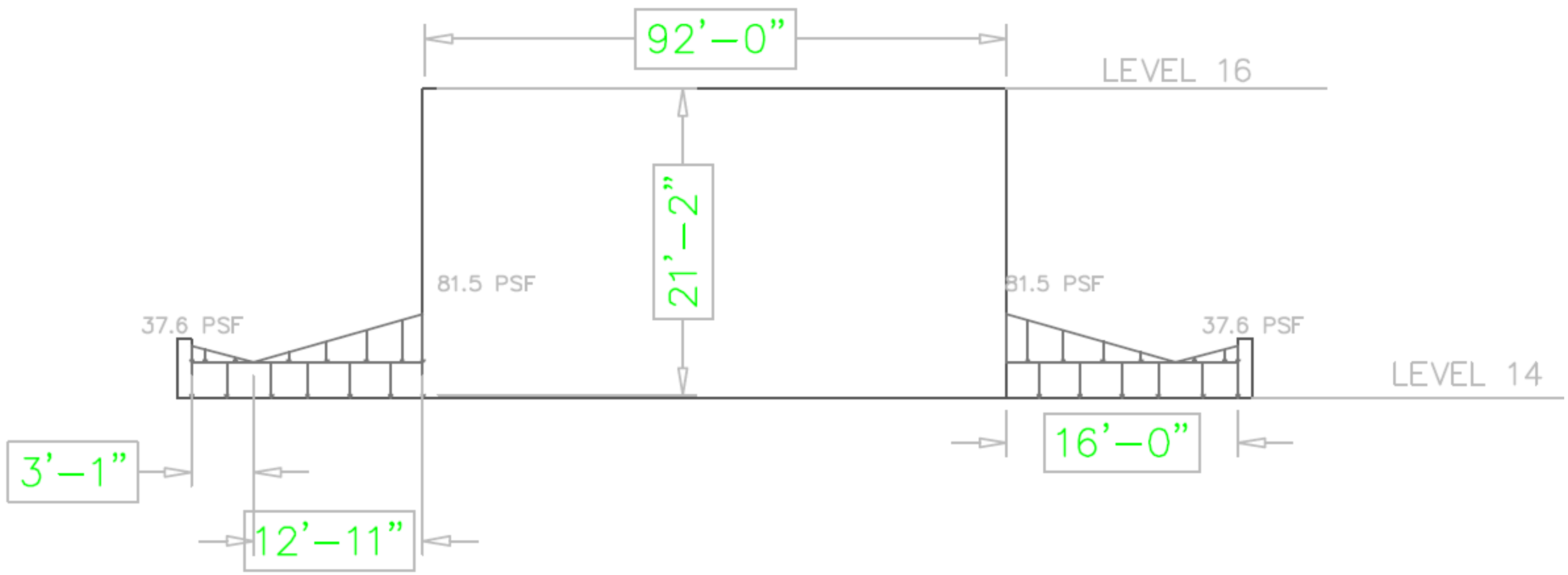
Summary

81.5 psf triangular distribution until 12.92’ from face of core

23.7psf at 12.92’ from face of core

Triangular distribution starting at 12.92’ from face of core 23.7psf until parapet at 37.6psf

NORTH/SOUTH ELEVATION VIEW DRIFT AGAINST CORE OF BUILDING ON LEVEL 16



East/West Elevation View

Drift Against Core of Building

$h_c = 21.33\ ft - 1.173\ ft = 20.12\ ft$

$\frac{h_c}{h_b} = \frac{20.12\ ft}{1.173\ ft} = 17.18$

$17.18 > 0.2 \therefore drift\ load\ must\ be\ considered\ against\ building\ core$

Leeward

- $l_u = Length\ higher\ roof = 72\ ft$
- $h_d = 0.43\sqrt[3]{l_u}\left(\sqrt[4]{P_g + 10}\right) - 1.5$
- $h_d = 0.43\sqrt[3]{72\ ft}\left(\sqrt[4]{30psf + 10}\right) - 1.5 = 3.0\ ft$
- $Drift\ Height = 3\ ft$

Windward

- $l_u = Length\ lower\ roof = 16\ ft$
- $h_d = 0.43\sqrt[3]{l_u}\left(\sqrt[4]{P_g + 10}\right) - 1.5$
- $h_d = 0.43\sqrt[3]{16\ ft}\left(\sqrt[4]{30psf + 10}\right) - 1.5 = 1.22\ ft$
- $Drift\ Height = \frac{3}{4}h_d = \ 0.92\ ft$

Design Drift Height=3 ft

- $P_d = Snow\ Drift\ Load = (h_d + h_b) * \gamma$
- $P_d = (3 + 1.173)ft * 17.9\ pcf = \mathbf{74.7psf}$
- $Width = W = 4 * h_d = 4 * 3ft$
- $\mathbf{W = 12\ ft}$

Drift Against Parapet

$h_c = 4\ ft - 1.173\ ft = 2.827\ ft$

$\frac{h_c}{h_b} = \frac{2.827\ ft}{1.173\ ft} = 2.41$

$2.41 > 0.2 \therefore drift\ load\ must\ be\ considered\ against\ parapet$

Windward

- $l_u = Length\ lower\ roof = 16\ ft$
- $h_d = 0.43\sqrt[3]{l_u}\left(\sqrt[4]{P_g + 10}\right) - 1.5$
- $h_d = 0.43\sqrt[3]{16\ ft}\left(\sqrt[4]{30psf + 10}\right) - 1.5 = 1.22\ ft$
- $Drift\ Height = \frac{3}{4}h_d = \ 0.92\ ft$

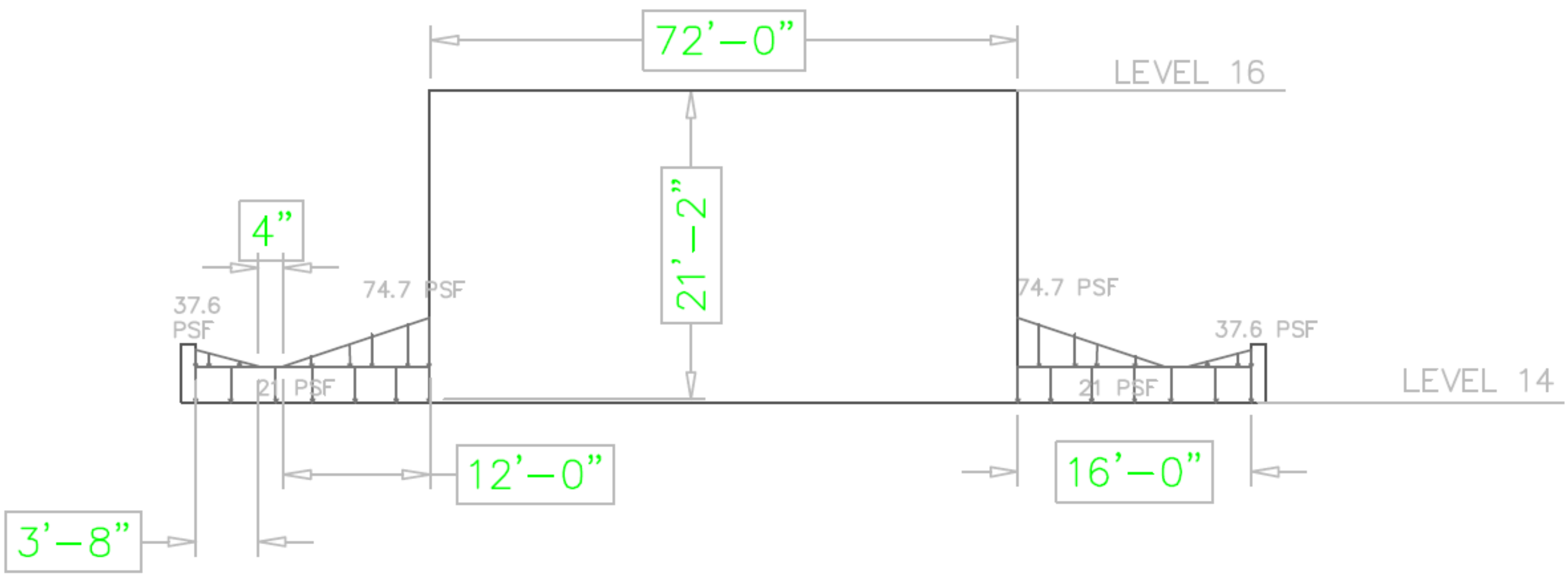
Design Drift Height=0.92ft

- $P_d = Snow\ Drift\ Load = (h_d + h_b) * \gamma$
- $P_d = (0.92 + 1.173) ft * 17.9\ pcf = \mathbf{37.6\ psf}$
- $Width = W = 4 * h_d = 4 * 0.92\ ft$
- $\boldsymbol{W = 3.68\ ft}$

Width of Balanced Snow at center (21psf)

$$Width = 16 - 12 - 3.68 = \mathbf{0.32 ft}$$

EAST/WEST ELEVATION VIEW DRIFT AGAINST CORE OF BUILDING ON LEVEL 14



Wind Load

Wind loads were designed according to Section 1609 of IBC 2006, which stipulates wind loads to be determined according to Chapter 6 of ASCE 7-05. ASCE 7-05's procedure is based on Bernoulli's Theorem. The wind velocities are converted to equivalent static wind pressures on the faces of the building. The dynamic wind loads are modeled as static loads, rather than dynamic.

First the wind was determined travelling in the East/West direction perpendicular to the building, then in the North/South direction. The equivalent static wind forces are applied to the shear walls at each floor of the building. They are assumed to transfer through the rigid diaphragm to the shear walls. The shear walls are designed to withstand 100 percent of the wind forces.

ASCE 7-05 Method 2 (Analytical Procedure) is used to calculate the wind loads
- -Regularly Shaped Building
- -No irregular response characteristics

Gust Effect Factor $G= 0.85$ (ASCE 7-05 Section 6.5.8)
- -Rigid Structure

Internal Pressure Coefficient $GC_{pi}=\pm0.18$ (ASCE 7-05 Figure 6-5)
- -Enclosed Building

Vel. Pressure Exposure Coefficient $K_{zt}=1.0$ (ASCE 7-05 Table 6-3)
- -Flat Terrain

Directionality Factor $K_d=0.85$ (ASCE 7-05 Table 6-4)
- -Main Wind Force Resisting System

Basic Wind Speed $V=100mph$ (ASCE 7-05 Figure 6-1)
- -Yonkers, NY

Importance Factor $I=1.0$ (ASCE 7-05 Table 6-1)
- -Occupancy Category II

Exposure Category C
-Surface Roughness C

Positive (+) refers to wind hitting the face of the building, while negative (-) refers to wind travelling away from the face of the building.

North Elevation View: Wind Travels East/West

Building Length Parallel to wind direction = L=126'
Building Length Perpendicular to wind direction = B=106'

$\frac{L}{B} = 1.19$

Mean height of building = h=162.67'

External Pressure Coefficients C_p Values (ASCE 7-05 Figure 6-6)	
Windward Wall	+0.8
Leeward Wall	-0.46
Side Wall	-0.7

Windward Wall

Design Wind Pressure= $P = q_z(GC_p) - q_h(GC_{Pi})$

$C_p = 0.8(windward\ wall)$

Velocity Pressure = $q_z = 0.00256 * k_z * k_{zt} * k_d * V^2 * I$

$q_h = 0.00256 * 1.39534 * 1.0 * 0.85 * (100mph)^2 * 1.0 = 30.36psf$

$P = q_z(0.85 * 0.8) - q_h(\pm 0.18)$

Leeward Wall

$z = h = 162.67ft$

$C_p = -0.46$

$q_h = 30.36\ psf$

$P = q_h * (GC_p) - q_h(GC_{pi}) = 30.36 * (0.85 * -0.46) - 30.36 * (\pm 0.18)$

$P\ for\ (+GC_{pi}) = \ -17.34\ psf$

$P\ for\ (-GC_{pi}) = \ -6.41\ psf$

Parapet Loads

Parapet Level 15

Parapet Height = 4ft
Top of Parapet Elevation = 156.67 ft
$k_z = 1.385$
$q_p = 0.00256 * k_z * k_{zt} * k_d * V^2 * I$
$q_p = 0.00256 * 1.385 * 1.0 * 0.85 * (100mph)^2 * 1.00 = 30.14\ psf$

$P = q_p(GC_{pn})$
$Windward\ Parapet\ GC_{pn} = +1.5$
$Leeward\ Parapet\ GC_{pm} = -1.0$

$Windward\ Parapet \rightarrow F = P_p * Parapet\ Height = 30.14(1.5)(4) = 180.8 lb/ft$
$F = 180.8\ lb/ft$
$$Wind\ Load = \frac{1}{2} * \frac{135.07 lb}{ft} * 106 ft = 9.58 kip$$

$Leeward\ Parapet \rightarrow F = P_p * Parapet\ Height = 30.14(-1)(4)$
$= -120.56\ lb/ft$
$F = -120.56\ lb/ft$
$$Wind\ Load = \frac{1}{2} * \frac{120.56\ lb}{ft} * 106 ft = -6.39\ kip$$

Parapet Level 13

Parapet Height = 4ft
Parapet Elevation = 135.33 ft
$k_z = 1.34832$
$q_p = 0.00256 * k_z * k_{zt} * k_d * V^2 * I$
$q_p = 0.00256 * 1.34832 * 1.0 * 0.85 * (100mph)^2 * 1.00 = 29.34\ psf$

$P = q_p(GC_{pn})$
$Windward\ Parapet\ GC_{pn} = +1.5$
$Leeward\ Parapet\ GC_{pm} = -1.0$

$Windward\ Parapet \rightarrow F = P_p * Parapet\ Height = 29.34(1.5)(4)$
$= 176.04\ lb/ft$
$F = 176.04\ lb/ft$
$$Wind\ Load = \frac{1}{2} * \frac{176.04 lb}{ft} * 106 ft = 9.33 kip$$

$$Leeward\ Parapet \rightarrow F = P_p * Parapet\ Height = 29.34(-1)(4)$$
$$= -117.36\ lb/ft$$
$$F = -117.36\ lb/ft$$
$$Wind\ Load = \frac{1}{2} * \frac{117.36\ lb}{ft} * 106ft = -6.22\ kip$$

Summary					
Level	**Height (ft)**	**Kz**	**qz (psf)**	**P for + Gcpi**	**P for -Gcpi**
Top of Building	162.67	1.39534	30.36	15.18	26.11
Top of Parapet Leeward				-30.14	
Top of Parapet Windward	156.67	1.385	30.14	45.21	
15	152.67	1.379	30.01	14.94	25.87
14	142.00	1.363	29.66	14.70	25.63
Top of Parapet Leeward				-29.34	
Top of Parapet Windward	135.33	1.34832	29.34	44.01	
13	131.33	1.3383	29.12	14.34	25.27
12	120.67	1.31	28.51	13.92	24.85
11	110.00	1.285	27.96	13.55	24.48
10	99.33	1.259	27.40	13.16	24.09
9	88.67	1.236	26.90	12.82	23.75
8	78.00	1.202	26.16	12.32	23.25
7	67.33	1.15932	25.23	11.69	22.62
6	56.67	1.1167	24.30	11.06	21.99
5	46.00	1.07	23.28	10.37	21.30
4	35.33	1.012	22.02	9.51	20.44
3	24.67	0.93736	20.40	8.40	19.34
15 feet	15.00	0.85	18.50	7.11	18.04
2	14.00	0.85	18.50	7.11	18.04
1	0.00	0.85	18.50	7.11	18.04
Leeward			30.36	-17.34	-6.41

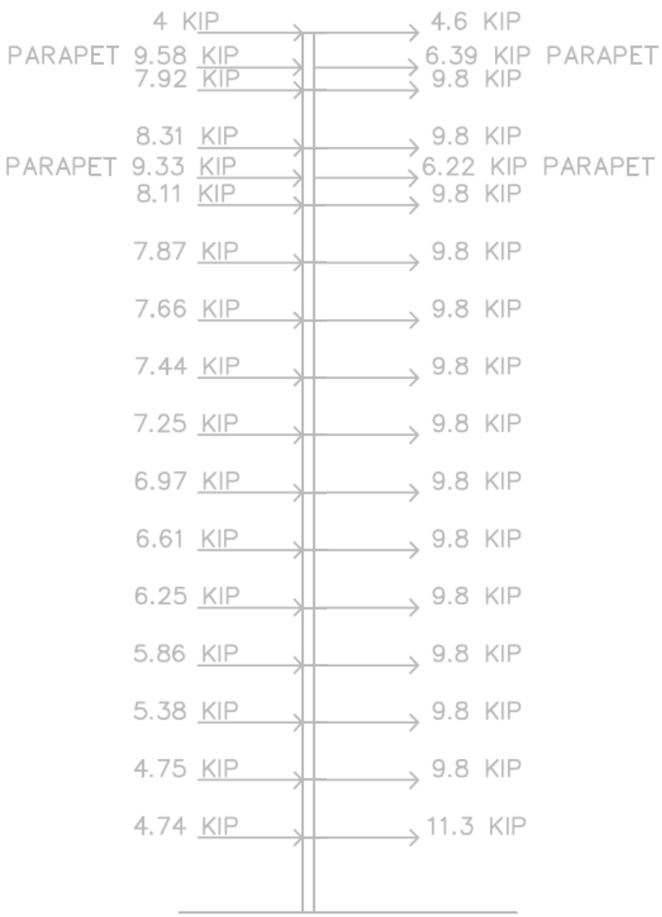
<u>Sample Calculation Force Per Shear Wall (Level 4)</u>

$$Windward\ Force = 9.51psf * \frac{106'}{2} * 10.67' = 5.378kips$$

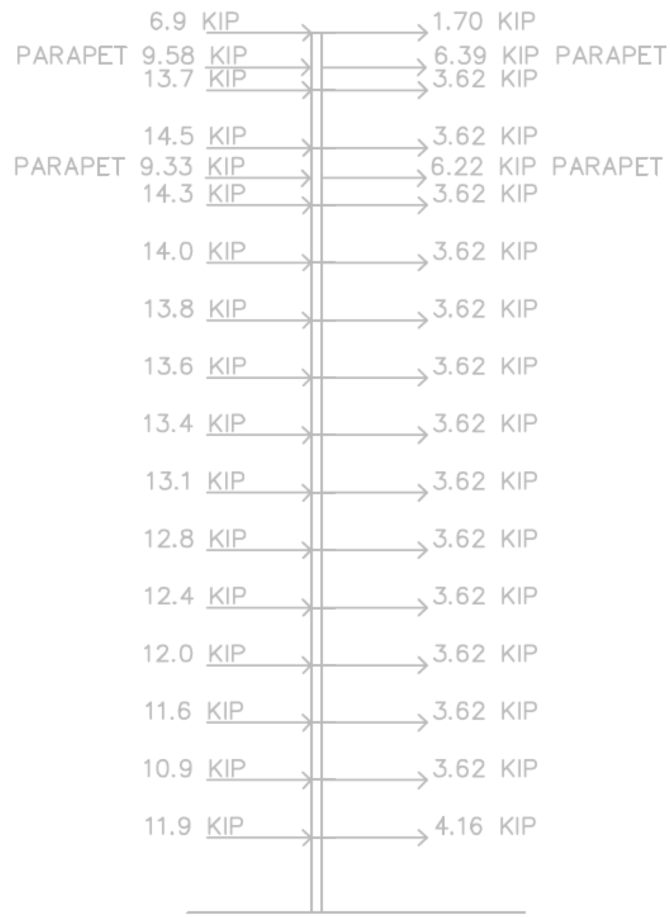
$$Leeward\ Force = 17.34psf * \frac{106'}{2} * 10.67' = 9.806kips$$

Forces Per Shear Wall					
Level	**Height (ft)**	**Force (kip) +Gcpi**	**Force (kip) - Gcpi**	**Leeward Force + Gcpi (kip)**	**Leeward Force – Gcpi (kip)**
Top of Building	162.67	4.0	6.9035	4.6	1.70
Top of Parapet Leeward		-6.39			
Top of Parapet Windward	156.67	9.58			
15	152.67	7.92	13.71	9.8	3.62
14	142.00	8.31	14.49	9.8	3.62
Top of Parapet Leeward		-6.22			
Top of Parapet Windward	135.33	9.33			
13	131.33	8.11	14.28	9.8	3.62
12	120.67	7.87	14.05	9.8	3.62
11	110.00	7.66	13.84	9.8	3.62
10	99.33	7.44	13.62	9.8	3.62
9	88.67	7.25	13.43	9.8	3.62
8	78.00	6.97	13.14	9.8	3.62
7	67.33	6.61	12.79	9.8	3.62
6	56.67	6.25	12.43	9.8	3.62
5	46.00	5.86	12.04	9.8	3.62
4	35.33	5.38	11.56	9.8	3.62
3	24.67	4.75	10.93	9.8	3.62
15 feet	15.00				
2	14.00	4.74	11.88	11.33	4.16
1	0.00				
Leeward					

NORTH ELEVATION VIEW WIND TRAVELS IN THE EAST/WEST DIRECTION
+Gcpi



NORTH ELEVATION VIEW WIND TRAVELS IN THE EAST/WEST DIRECTION
−Gcpi



East Elevation View: Wind Travels North/South

L=106'
B=126'
$\frac{L}{B} = 0.84$
h=162.67'

Wall Pressure Coefficients C_p Values

Windward Wall	+0.8
Leeward Wall	-0.5
Side Wall	-0.7

Windward Wall

$$P = q_z(GC_p) - q_h(GC_{Pi})$$
$$C_p = 0.8(windward\ wall)$$
$$q_z = 0.00256 * k_z * k_{zt} * k_d * V^2 * I$$
$$q_h = 0.00256 * 1.39534 * 1.0 * 0.85 * (100mph)^2 * 1.0 = 30.36psf$$
$$P = q_z(0.85 * 0.8) - q_h(\pm 0.18)$$

Leeward Wall

$$z = h = 162.67ft$$
$$C_p = -0.5$$
$$q_h = 30.36\ psf$$
$$P = q_h * (GC_p) - q_h(GC_{pi}) = 30.36 * (0.85 * -0.5) - 30.36 * (\pm 0.18)$$
$$P\ for\ (+GC_{pi}) = -18.37\ psf$$
$$P\ for\ (-GC_{pi}) = -7.44\ psf$$

Parapet Loads

Parapet Level 15

Parapet Height = 4ft
Parapet Elevation = 156.67 ft
$k_z = 1.385$
$$q_p = 0.00256 * k_z * k_{zt} * k_d * V^2 * I$$
$$q_p = 0.00256 * 1.385 * 1.0 * 0.85 * (100mph)^2 * 1.00 = 30.14\ psf$$

$$P = q_p(GC_{pn})$$
$$Windward\ Parapet\ GC_{pn} = +1.5$$

$Leeward\ Parapet\ GC_{pm} = -1.0$

$Windward\ Parapet \rightarrow F = P_p * Parapet\ Height = 30.14(1.5)(4) = 180.8 lb/ft$
$F = 180.8\ lb/ft$
$$Wind\ Load = \frac{1}{2} * \frac{180.8lb}{ft} * 126ft = 11.39kip$$

$Leeward\ Parapet \rightarrow F = P_p * Parapet\ Height = 30.14(-1)(4)$
$= -120.56\ lb/ft$
$F = -120.56\ lb/ft$
$$Wind\ Load = \frac{1}{2} * \frac{120.56\ lb}{ft} * 126ft = -7.59\ kip$$

<u>Parapet Level 13</u>

Parapet Height = 4ft
Parapet Elevation = 135.33 ft
$k_z = 1.34832$
$q_p = 0.00256 * k_z * k_{zt} * k_d * V^2 * I$
$q_p = 0.00256 * 1.34832 * 1.0 * 0.85 * (100mph)^2 * 1.00 = 29.34\ psf$

$P = q_p(GC_{pn})$
$Windward\ Parapet\ GC_{pn} = +1.5$
$Leeward\ Parapet\ GC_{pm} = -1.0$

$Windward\ Parapet \rightarrow F = P_p * Parapet\ Height = 29.34(1.5)(4)$
$= 176.04\ lb/ft$
$F = 176.04\ lb/ft$
$$Wind\ Load = \frac{1}{2} * \frac{176.04lb}{ft} * 126ft = 11.09kip$$

$Leeward\ Parapet \rightarrow F = P_p * Parapet\ Height = 29.34(-1)(4)$
$= -117.36\ lb/ft$
$F = -117.36\ lb/ft$
$$Wind\ Load = \frac{1}{2} * \frac{117.36\ lb}{ft} * 126ft = -7.93\ kip$$

Summary					
Level	**Height (ft)**	**Kz**	**qz (psf)**	**P for + Gcpi**	**P for -Gcpi**
Top of Building	162.67	1.39534	30.36	15.18	26.11
Top of Parapet Leeward				-30.14	
Top of Parapet Windward	156.67	1.385	30.14	45.21	
15	152.67	1.379	30.01	14.94	25.87
14	142.00	1.363	29.66	14.70	25.63
Top of Parapet Leeward				-29.34	
Top of Parapet Windward	135.33	1.34832	29.34	44.01	
13	131.33	1.3383	29.12	14.34	25.27
12	120.67	1.31	28.51	13.92	24.85
11	110.00	1.285	27.96	13.55	24.48
10	99.33	1.259	27.40	13.16	24.09
9	88.67	1.236	26.90	12.82	23.75
8	78.00	1.202	26.16	12.32	23.25
7	67.33	1.15932	25.23	11.69	22.62
6	56.67	1.1167	24.30	11.06	21.99
5	46.00	1.07	23.28	10.37	21.30
4	35.33	1.012	22.02	9.51	20.44
3	24.67	0.93736	20.40	8.40	19.34
15 feet	15.00	0.85	18.50	7.11	18.04
2	14.00	0.85	18.50	7.11	18.04
1	0.00	0.85	18.50	7.11	18.04
Leeward			30.36	-18.37	-7.44

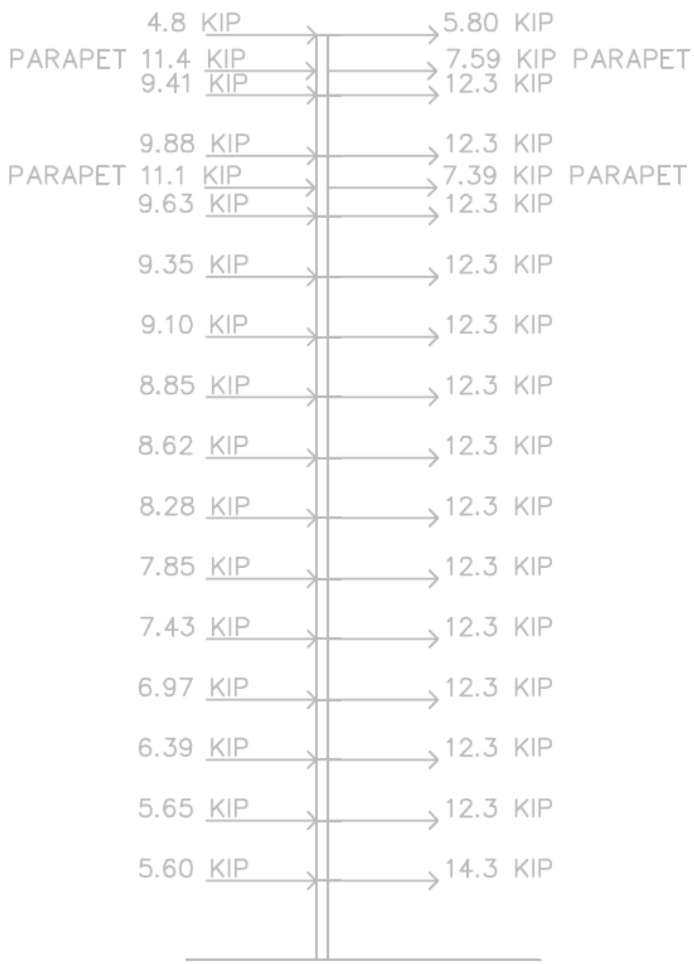
<u>Sample Calculation Force Per Shear Wall (Level 4)</u>

$$Windward\ Force = 9.51psf * \frac{126'}{2} * 10.67' = 6.39kips$$

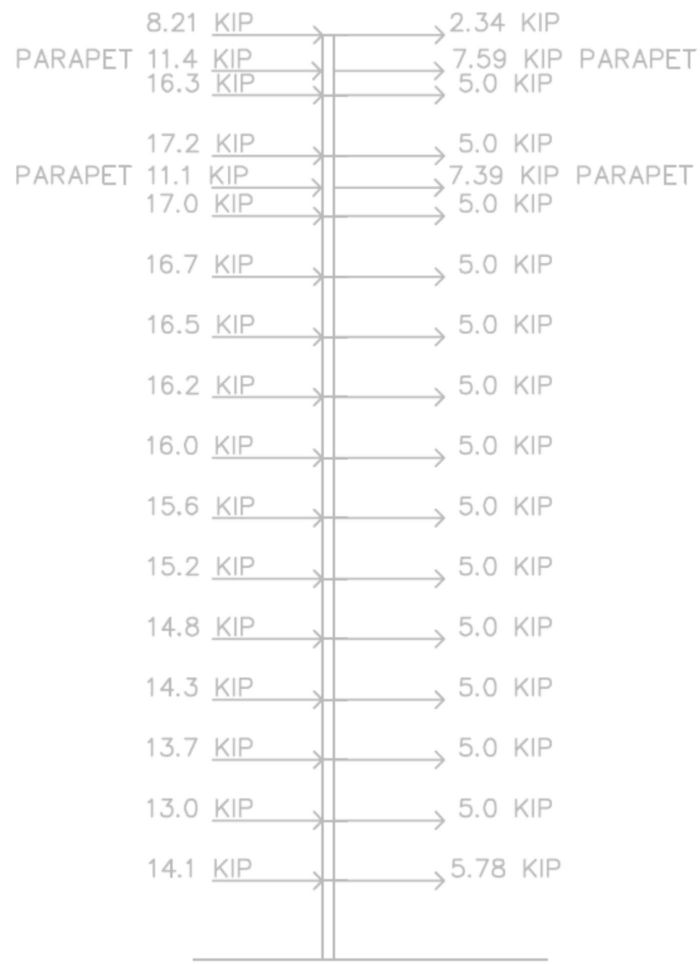
$$Leeward\ Force = 18.37psf * \frac{126'}{2} * 10.67' = 12.35\ kips$$

Forces Per Shear Wall					
Level	**Height (ft)**	**Force (kip) +Gcpi**	**Force (kip) -Gcpi**	**Leeward Force + Gcpi (kip)**	**Leeward Force – Gcpi (kip)**
Top of Building	162.67	4.8	8.207	5.8	2.34
Top of Parapet Leeward		-7.59			
Top of Parapet Windward	156.67	11.39			
15	152.67	9.41	16.30	12.35	5.00
14	142.00	9.88	17.23	12.35	5.00
Top of Parapet Leeward		-7.39			
Top of Parapet Windward	135.33	11.09			
13	131.33	9.63	16.98	12.35	5.00
12	120.67	9.35	16.70	12.35	5.00
11	110.00	9.10	16.45	12.35	5.00
10	99.33	8.85	16.19	12.35	5.00
9	88.67	8.62	15.96	12.35	5.00
8	78.00	8.28	15.62	12.35	5.00
7	67.33	7.85	15.20	12.35	5.00
6	56.67	7.43	14.78	12.35	5.00
5	46.00	6.97	14.31	12.35	5.00
4	35.33	6.39	13.74	12.35	5.00
3	24.67	5.65	12.99	12.35	5.00
15 feet	15.00				
2	14.00	5.60	14.10	14.27	5.78
1	0.00				
Leeward					

EAST ELEVATION VIEW WIND TRAVELS IN THE NORTH/SOUTH DIRECTION
+Gcpi



EAST ELEVATION VIEW WIND TRAVELS IN THE NORTH/SOUTH DIRECTION
−Gcpi



Earthquake Load

Seismic loads were designed according to Section 1613 of IBC 2006, which stipulates wind loads to be determined according to ASCE 7-05. ASCE 7-05's procedure is also known as the equivalent lateral force method. This method converts the motions that a building receives during an earthquake into equivalent lateral static forces. The shear walls are designed to withstand 100 percent of the seismic lateral forces; the slabs are modeled as rigid diaphragms and transfer all seismic loads to the shear walls. Seismic loads were assumed to act at the center of mass of the seismic diaphragm of each floor. There were also modeled five percent to the left and right of the center of mass of the diaphragms to account for any possible accidental eccentricities due to building and construction mistakes.

First the seismic design category of the building was determined. The owner has requested that a minimum seismic design category of C should be used. Then the equivalent weight of each floor was determined and the base shear was distributed throughout the building, according to floor weight.

Calculated loads were compared to the loads determined by ETABS 2013. They were confirmed by the analysis of ETABS 2013.

Design Category

Location: Yonkers, NY 10701

Site Class C (IBC 2006 Table 16.13.5.2)

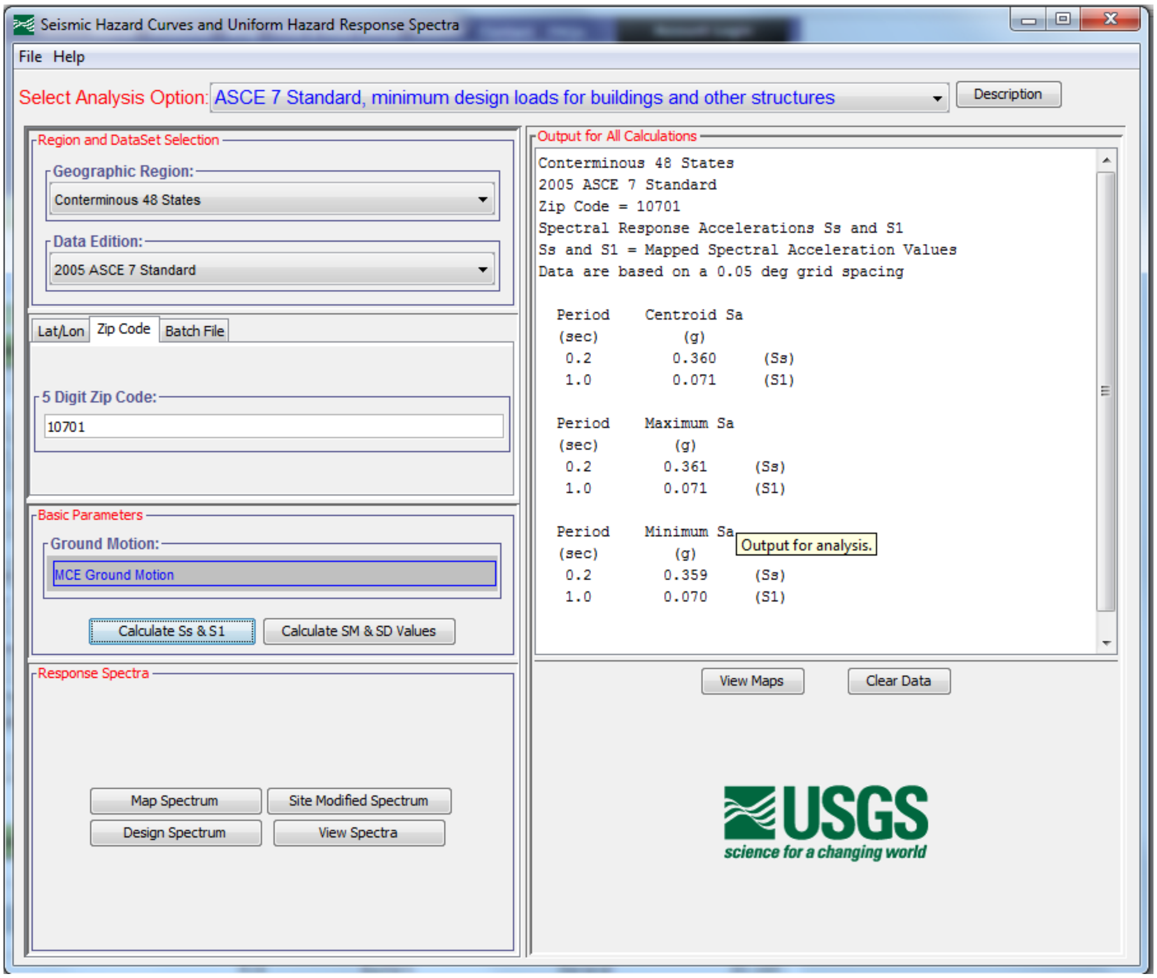
-Very Dense Soil & Soft Rock Beneath

Importance Factor I= 1.0 (ASCE 7-05 Table 11.5-1)

-Occupancy Category II

Building Height of 162.33 feet

From Java Calculator: S_s =0.360g S_1 = 0.071g



Rigid Diaphragm Concrete Building Frame System with Special Reinforced Concrete Shear Wall

Response Modification Coefficient = R = 6 (ASCE 7-05 Table 12.2-1)

System Over strength Factor = $\Omega_0 = 2.5$ (ASCE 7-05 Table 12.2-1)

Deflection Amplification Factor = C_d=5 (ASCE 7-05 Table 12.2-1)

<u>Short Period Response S_{Ds}</u>

From ASCE 7-05 Table 11.4-1 Class C and S_S = 0.360g

F_a=1.2 from interpolation

$$S_{ms} = F_a S_s = 1.2 * 0.360g = 0.432g$$

$$S_{Ds} = \frac{2}{3} S_{ms} = \frac{2}{3} * 0.432g = 0.288g$$

From ASCE 7-05 Table 11.4-2 for Occupancy Category II the **Seismic Design Category is B**

<u>One-Second Period Response S_{D1}</u>

From ASCE 7-05 Table 11.4-2 Class C and S_1 < 0.1

F_v=1.7 from interpolation

$$S_{m1} = F_v S_1 = 1.7 * 0.071g = 0.1207g$$

$$S_{D1} = \frac{2}{3} S_{m1} = \frac{2}{3} * 0.1207g = 0.0805g$$

From ASCE 7-05 Table 11.6-2 for Occupancy Category II and $0.067<S_{D1}<0.133$ the **Seismic Design Category is B**

Final Seismic Design Category is C (minimum required by owner)

From ASCE 7-05 Table 12.6-1 Equivalent Lateral Force Analysis may be used

<u>Seismic Response Coefficient</u>

ASCE 7-05 12.8-2

$$Seismic\ Response\ Coefficient = C_S = \frac{S_{Ds}}{R/I} = \frac{0.288g}{6/1} = 0.048g > 0.01g\ OK$$

<u>Period of Vibration</u>

$$T_a = C_t h_n^x = 0.02 * 162.33^{0.75} = 0.911 seconds$$

T_L=6 seconds ASCE 7-05 Figure 22-15

T > T_L so $C_S \le \frac{S_{D1} T_L}{T^2 \frac{R}{I}} = \frac{0.0805*6}{0.911^2 * \frac{6}{1}} = 0.097\ \ OK$ ASCE 7-05 12.8-4

Horizontal Structural Irregularities Checks (ASCE 7-05 Table 12.3-1)

1. The torsional irregularities of the building were checked in ETABS. There were no torsional irregularities. The largest ratio of maximum story drift to average story drift for all load combinations was 1.14, using ETABS 2013.
2. Reentrant corner irregularities do not exist in our structure.
3. There are no diaphragm discontinuity irregularities. A constant diaphragm of eight inches was used for all floors except for floor two, where a ten-inch diaphragm was used.
4. There are no out of plane offsets in the lateral force resistance system; it is continuous throughout.
5. The lateral force resistance systems are parallel in the structure.

Vertical Structural Irregularities Checks (ASCE 7-05 Table 12.3-2)

1. There are no stiffness-soft story irregularities in the building.
2. There are no mass irregularities in the building; our stories are fairly consistent.
3. There are no vertical geometric irregularities in our building.
4. There are no in-plane discontinuities in lateral force–resisting element irregularities.
5. There are no discontinuities in the lateral strength. This was achieved by keeping the size and location of the shear wall constant throughout the building.

Weight of Building

<u>Loads</u>
Brick Wall Weight = 48psf
Glass Façade Weight = 15psf
Storage Room Live Load = 125psf (25% of storage area live load)
Concrete unit weight = 150pcf
Flat Snow Load=21psf < 30psf so need not be considered in building weight

When determining the building weight, the garage floor was not taken into account because it rests on grade. All other floors were considered. The floor dead load, 25 percent of storage live load, retaining wall weight, masonry façade weight, glass façade weight, column weights, shear wall weights, beam weights, and parapet weights on each floor were accounted for. Below is a summary of the results.

Building Weight Summary			
Floor	**Height (ft)**	**Weight (kips)**	**Total Weight (kips)**
Top of Building Core	162.33	214	214
16 Roof Core		374	1088
16 Roof Access	152.67	714	
15	142.00	1317	1317
Promenade		834	2294
14	131.33	1460	
12	120.67	2562	2562
11	110.00	2562	2562
10	99.33	2562	2562
9	88.67	2562	2562
8	78.00	2562	2562
7	67.33	2562	2562
6	56.67	2562	2562
5	46.00	2562	2562
4	35.33	2562	2562
3	24.67	2789	2789
2	14.00	3937	3937
		TOTAL	**34,701**

Base Shear

$$V = C_s W = 0.048 * 34{,}701 kip = \mathbf{1665.6 kips}$$

Vertical Distribution of Forces (ASCE 7-05 12.8.3)

The vertical distribution factor was determined for each floor in order to determine the seismic force on the floor.

k=1.205 (Interpolated)
Floor 2 Calculations

$$C_{vx} = \frac{w_x h_x^k}{\sum w_i h_i^k} = \frac{3937 kips * 14^{1.205}}{6590938} = 0.01436$$

$$F_x = C_{vx} * V = 0.01436 * 1665.6 kip = \mathbf{23.93 kip}$$

Building Equivalent Seismic Force Summary				
Floor	**Height (ft)**	**Cvx**	**Force (kips)**	**Force per shear wall (kips)**
Top of Building Core	162.33	0.015	24.97	12.48
16 Roof Core	152.67	0.071	117.71	58.86
16 Roof Access				
15	142.00	0.078	130.55	65.28
Promenade	131.33	0.124	206.91	103.46
14				
12	120.67	0.125	208.73	104.37
11	110.00	0.112	186.70	93.35
10	99.33	0.099	165.11	82.55
9	88.67	0.086	143.99	71.99
8	78.00	0.074	123.38	61.69
7	67.33	0.062	103.35	51.67
6	56.67	0.050	83.95	41.98
5	46.00	0.039	65.30	32.65
4	35.33	0.029	47.52	23.76
3	24.67	0.020	33.54	16.77
2	14.00	0.014	23.93	11.96
	TOTAL	**1.0**	**1665.6**	**832.82**

Please Note: S_{Ds}>0.125 so the vertical effects due to earthquake need to be considered as:

$$E_V = 0.2 * S_{DS} * Dead = 0.2 * 0.288 * Dead$$

$$E_H = \rho Q_E = 1.0 * 1665.6kip = 1665.6kip$$

The vertical effects of earthquake were only considered when designing the shear wall.

Slabs

Floors 4-12 Typical

The floor plans and loads on floors four through twelve are the same, so their slab reinforcement will be the same. The dead load on these floors was 144 pounds per square foot and the live load in the private areas was 40 pounds per square foot and 100 pounds per square foot in the core. The brick and glass facades had loads of 480 and 150 pounds per linear foot, respectively.

SAFE 12.2.0 Procedure

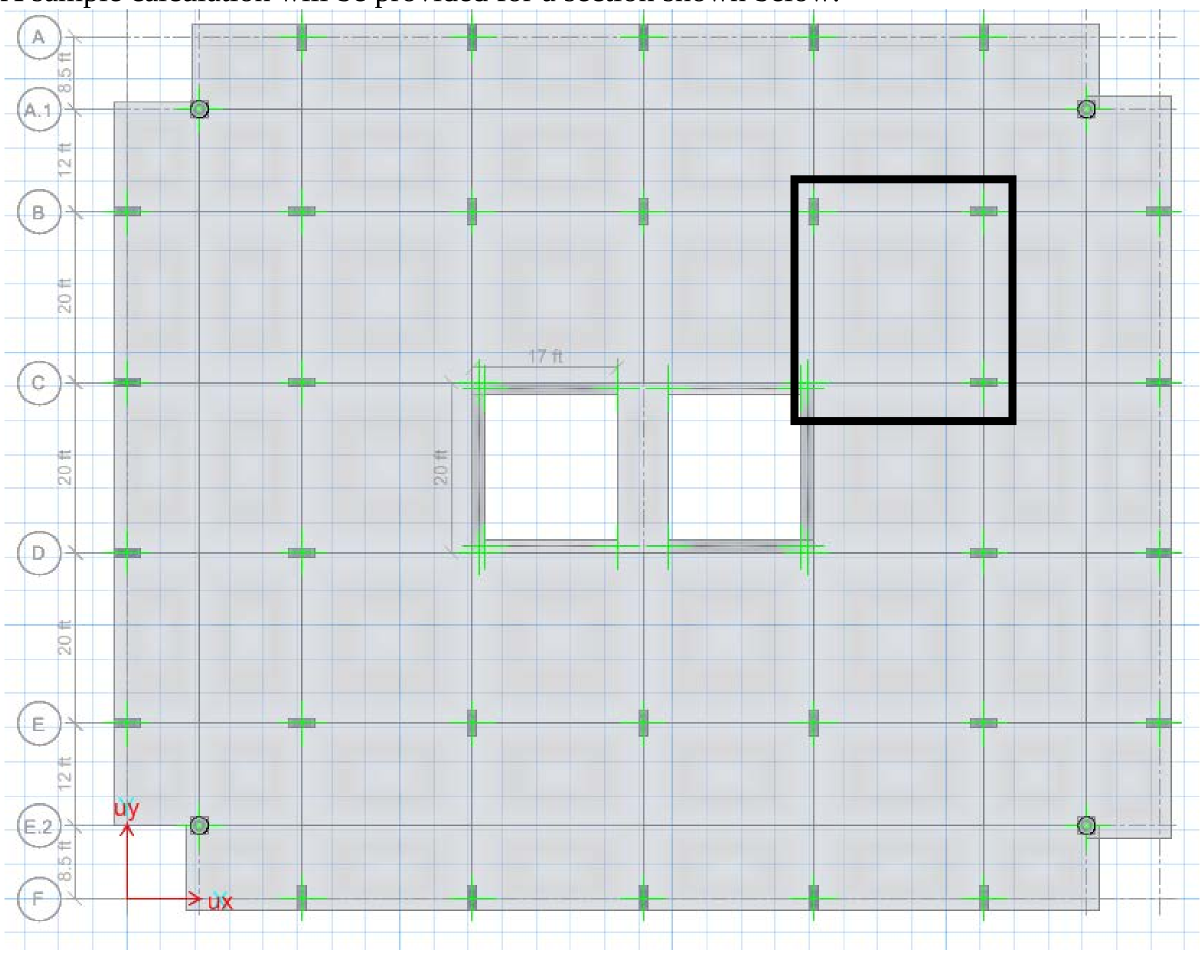
The slabs were designed using SAFE 12.2.0 finite element analysis. The minimum slab thickness of eight inches was used, according to ACI 9.5.32, to avoid the need of deflection analysis.

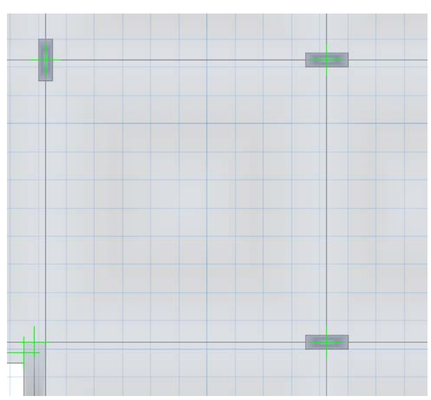
The eight-inch slab and the locations of the columns and shear wall were first drawn. Then stiff elements were drawn wherever a column or wall travelled through the slab. These stiffer elements were modeled to have a thickness of three times the slab (24 inches) in order to more accurately model the behavior of the slab. The shear wall was modeled to have a rigid zone above the wall and not take out of plane motion. A mesh of two feet by two feet was used over the entire slab. Cracking analysis was also checked to be determined during the analysis and design.

The dead load and live loads were then modeled and applied to their respective areas. Analysis was then run and the shear on the slab was modeled. This was checked to make sure the column strips ran along the lines of zero shear, which is also the lines with the largest moments. Punching shear design was checked afterwards. Several of the outer columns failed in punching shear. Several solutions could be made to adjust the punching shear failures: the slab could have been made thicker, the strength of concrete could have been increased, drop panels could have been added, or additional shear reinforcement could have been added to the columns in need. To solve the punching shear issue, it was determined that the most economical solution would be to provide Lenton Steel Fortress Punching Shear Reinforcement System at the columns in need. This is a proprietary system that is a cost effective solution, the specifications are shown in the appendix. The strips were then designed using the slab design option in SAFE. A mesh was used for the bottom layer in both directions. Also, in between the column strips a standard spacing of rebar was determined for each middle strip. Wherever additional bars were needed, they were noted. The number of bars, length of the bars, spacing, and stagger were all specified at each column. This method was used to simplify the design and construction of the two-way slabs. Also, a maximum spacing of 12 inches was used to ensure that cracking would not occur in the slab over time, although the maximum spacing according to the ACI code is 18 inches.

Sample Calculation

A sample calculation will be provided for a section shown below.





The columns are all one foot by three feet and spaced at 20 feet center to center. The shear wall is 12 inches thick.

Check if Direct Design Method may be used

1. There are at least three continuous spans in each direction.
2. The panels are square, and the ratio of long to short side is less than two.
3. Adjacent spans in each direction do not differ by more than one-third of the larger span.
4. Columns are not offset by more than 10 percent of the span length in the direction of offset.
5. All loads are due to gravity and uniform. Also, the ratio of unfactored live to dead loads does not exceed two.
6. There are no beams present along the side.

The slab was already determined to be eight inches thick

Factored Loads

$$1.2Dead + 1.6Live = 1.2(144) + 1.6(40) = \mathbf{236.8psf}$$

Effective Depth (d)

¾ inch cover will be used. #5 rebar will be estimated.

$$d = h - \frac{3}{4} - d_b = 8 - \frac{3}{4} - \frac{5}{8} = \mathbf{6.625\ inches}$$

Punching Shear at Interior Column

Critical section is located at d/2=3.31 inches from face of column.

$$V_u = \left[20 * 20 - \left(\frac{18.625}{12}\right)\left(\frac{42.625}{12}\right)\right] * 236.8psf = \mathbf{93,414\ lbs}$$

Punching Shear limits is the smaller of the following: (ACI 318-11 11.11.2.1)

$$\left(2 + \frac{4}{\beta}\right)\lambda\sqrt{f'_c}b_0 d = \left(2 + \frac{4}{3}\right)1 * \sqrt{4000} * 122.5 * 6.625 = \mathbf{171,093lb}$$

$$\left(2 + \frac{\alpha_s d}{b_0}\right)\lambda\sqrt{f'_c}b_0 d = \left(2 + \frac{40 * 6.625}{122.5}\right)1 * \sqrt{4000} * 122.5 * 6.625 = 213{,}692lb$$

$$(4)\lambda\sqrt{f'_c}b_0 d = (4)1 * \sqrt{4000} * 122.5 * 6.625 = 205{,}311lb$$

$$0.75V_c = 0.75(171093lb) = 128{,}320\ lb > 93{,}414\ lbs\ \boldsymbol{OK}$$

Passes for punching shear

One-way Shear at d from column face

Critical section is located at d=6.625 inches from face of column.
Assume cantilevered at d from column to mid span

$$\frac{20}{2}-\frac{12}{24}-\frac{6.625}{12}=8.95\,feet\ from\ midspan$$

$$Max\ one\ way\ shear=8.95'*1'*236.8psf=\mathbf{2,119\ lb}$$

One way shear limit:

$$\phi(2)\lambda\sqrt{f'_c}b\quad d=0.75(2)1*\sqrt{4000}*12*6.625=7{,}542lb>2{,}119lb\ \boldsymbol{OK}$$

Passes for one way shear

Moments in the East/West Direction

$$Column\ Strip=\frac{1}{4}*20+\frac{1}{4}*20=10'$$
$$Midstrip=20-5-5=10'$$
$$Moment=\frac{236.8psf(19')(20-1.5)^2}{8}=\mathbf{192.48kip-ft}$$

Column Strip Moments ACI 318-11 13.6.4

$$Negative\ Moment=0.75*0.65*192.48kip-ft=\mathbf{93.83kip-ft}$$
$$Positive\ Moment=0.60*0.35*192.48kip-ft=\mathbf{40.42kip-ft}$$

Middle Strip Moments ACI 318-11 13.6.4

$$Negative\ Moment=0.25*0.65*192.48kip-ft=\mathbf{31.3kip-ft}$$
$$Positive\ Moment=0.40*0.35*192.48kip-ft=\mathbf{26.9\ kip-ft}$$

Reinforcement ACI 318-11 10.5.1

$$\rho_{min}=\frac{3\sqrt{f_c'}}{f_y}\geq\frac{200}{f_y}=\frac{3\sqrt{4000}}{60000}=0.003\geq\frac{200}{60000}=\mathbf{0.0033}$$

$$A_{smin}=\rho_{min}bd=0.0033*120*6.625=2.62in^2$$

Minimum reinforcement for shrinkage=0.0018 ACI 7.12.2.1

Max Spacing ACI 318-11 7.6.5

$$spacing=3*8=24<18"$$

Max spacing is 18”
Max spacing in critical sections =2*8=16” ACI 318-13.3.2
A **maximum spacing of 12 inches** will be used over the entire slab to reduce cracking over time.

East/West Direction Layer A				
	Column Strip		**Middle Strip**	
Moment	**Negative**	**Positive**	**Negative**	**Positive**
Mu (lb*ft)	93,830	40,420	31,300	26,900
width b (in)	120	120	120	120
depth d (in)	6.625	6.625	6.625	6.625
Ru=Mu/bd^2	17.82	7.67	5.94	5.11
p	0.002	0.002	0.002	0.002
p min	0.0033	0.0033	0.0033	0.0033
As=pbd	2.62	2.62	2.62	2.62
Bar Selection	9 #5	14 #4	9 #5	14 #4
As	2.79	2.8	2.79	2.8
Spacing (in)	15	12	16	12
Max Spacing (in)	12	12	12	12
Final Selection	11 #5	#4	#5	#4
Spacing (in)	12	12	12	12

Moments in the North/South Direction

$$Column\ Strip = \frac{1}{4} * 20 + \frac{1}{4} * 20 = 10'$$
$$Midstrip = 20 - 5 - 5 = 10'$$
$$Moment = \frac{236.8psf(18.5')(19)^2}{8} = \mathbf{197,683kip - ft}$$

Column Strip Moments ACI 318-11 13.6.4

$$Negative\ Moment = 0.75 * 0.65 * 197.683kip - ft = \mathbf{96.37kip - ft}$$
$$Positive\ Moment = 0.60 * 0.35 * 197.683kip - ft = \mathbf{41.51kip - ft}$$

Middle Strip Moments ACI 318-11 13.6.4

$$Negative\ Moment = 0.25 * 0.65 * 197.683kip - ft = \mathbf{32.12kip - ft}$$
$$Positive\ Moment = 0.40 * 0.35 * 197.683kip - ft = \mathbf{27.67\ kip - ft}$$

North/South Direction Layer B				
	Column Strip		**Middle Strip**	
Moment	**Negative**	**Positive**	**Negative**	**Positive**
Mu (lb*ft)	96,370	41,510	32,120	27,670
width b (in)	120	120	120	120
depth d (in)	6.625	6.625	6.625	6.625
Ru=Mu/bd^2	18.30	7.88	6.10	5.25
p	0.002	0.002	0.002	0.002
p min	0.0033	0.0033	0.0033	0.0033
As=pbd	2.62	2.62	2.62	2.62
Bar Selection	9 #5	14 #4	9 #5	14 #4
As	2.79	2.8	2.79	2.8
Spacing (in)	15	12	16	12
Max Spacing (in)	12	12	12	12
Final Selection	11 #5	#4	#5	#4
Spacing (in)	12	12	12	12

Length of Bars and Stagger ACI 318-11 Fig. 13.3.8

Column Strip Bottom

At least 2 continuous bars must pass through the column on the bottom layers in both directions (#4 at 12")

At edge columns, the bars must extend at least 6" into column on bottom layers

Column Strip Top

All rebar terminating at edge columns must end in a 180 hook (#5) ACI 318-11 12.5

$$8d_b = 8 * \frac{5}{8} = 5'' \; lower\ part\ of\ hook$$

Development length of top bar hooks ACI 318-11 12.5.2

$$l_{dh} = \frac{[0.02 * 60000]}{\sqrt{4000}}\left(\frac{5}{8}\right) = 11.85'' = 12''$$

50 percent of bars on top at edge columns must extend at least 0.3(clear span) from the face

The remaining 50 percent of bars on top at edge columns must extend at least 0.2(clear span) from the face

50 percent of bars on top at interior columns must extend at least 0.3(clear span) from the face

The remaining 50 percent of bars on top at edge columns must extend at least 0.2(clear span) from the face

The same size bar will be used. The size of the bar will be determined as (0.2+0.3)(clear span)+column width. They will be staggered at (0.3-0.2)(clear span). Using the same size bar staggered is much more economical and easier to install, leading to fewer mistakes being made in the field. Also, an odd number of bars was always used, so the first bar would be placed at the center of the column and the remaining bars will be placed to the left and right of the first bar at the determined spacing and stagger.

Simplification of Layout

In order to simplify the layout, several size uniform meshes were used for the bottom layers of reinforcement in both layers. It was determined that #4 bars at 12 inch spacing was the most economical. An alternative was #3 bars at 8 inch spacing, which came out to more steel over the entire floor. Wherever additional bars were needed, they were stated on the drawing.

Also, for column strips the length of the bar and the spacing were determined and noted on the drawings. The total number was not noted, to ensure that no bars or areas would be accidently missed or misplaced during construction.

Transfer Beam

There is a shift in the column lines on floor 2 by 6' for 10 columns around the core. This was due to the fact that the garage needed much more clear space between the shear wall and columns to allow for traffic to maneuver safely in the underground garage. The transfer beam will transfer the loads of all of the columns from floor 2 and above to the column on the garage floor. These are all very critical columns for the structural integrity of the building. SAFE was used to determine the loads on the columns. The point load was then input onto the SAFE model for floor 2 and the beam was designed in SAFE. The beams more or less contained the same forces, so the worst beam was used for design purposes of all of the beams. The design provided by SAFE was confirmed with the hand calculation shown below. The beam was determined to be 52" deep and 36" wide. This means that the beam will drop 42" (3.5 feet) below the 10" slab. This still leaves 10.5 feet clear height in the garage level, which is more than the 8'-2" required for vehicles.

Span is 26’
Depth=h=52”
Design depth=d=50”
Width=b=36”
Column from above is 6’ from below column

$\beta_1 = 0.85$
$f'c = 4000\ psi$
$f_y = 60{,}000\ psi$

Loads

The loads were determined from SAFE model.

Service Loads from columns resting on beam			**Service Floor Loads**	
Dead (kip)	**Live (kip)**	**Snow (kip)**	**Live (ksf)**	**Dead (ksf)**
719.2	192.2	6.7	0.1	0.169

Tributary width =20’

$$Dead = (0.169)(20') + (0.150)(3')\left(\frac{52-10}{12}\right) = 4.96\frac{kip}{ft}$$

$$Live = (0.1)(20') = 2\frac{kip}{ft}$$

<u>Factored Loads</u>

$$1.2Dead + 1.6Live + 0.5Snow$$

$$1.2(719.2) + 1.6(192.2) + 0.5(6.7) = \mathbf{1216.63\ kip}$$

$$1.2(4.96) + 1.6(2) = \mathbf{9.15\frac{kip}{ft}}$$

Maximum positive moment from SAFE was **4,508 kipft** located at point load from column location.

Beam behaves as fixed near the shear wall, and is not necessarily fixed or pinned at column support but behaves somewhat in between.

The AISC Steel Manual Moment calculations were used to estimate whether the moment given by SAFE was accurate.

The maximum moment if the beam was fixed at both ends was determined to be 2,027.6 kipft.
The maximum moment of the beam fixed at one end and pinned at the other was determined to be 5,189 kip ft.

Since the moment determined by SAFE was **4,508 kipft** and falls between the two extremes it seems to be correct.

The negative moment near the shear wall was determined to be **2,995 kipft.**

The negative moment near the column support was determined to be **almost zero,** but the same reinforcement on the shear wall side will be used.

Reinforcement

Positive Reinforcement

$$Moment = 4508\ kip - ft$$

$$a = \frac{A_s f_y}{(0.85 f_c' b_w)} = \frac{60 * A_s}{(0.85 * 4 * 36)} = 0.49As$$

$$A_s = \frac{M_u}{\left(\phi f_y \left(d - \left(\frac{a}{2}\right)\right)\right)} = \frac{4508kft(12)}{0.9 * 60 * \left(50 - \frac{a}{2}\right)}$$

Solving the quadratic equation A_s=22.5in^2

SAFE Specified As= 22.8 in^2

16 # 11 bars will be used As=24.96 in^2

$$\rho = \frac{A_s}{bd} = \frac{24.96}{36 * 50} = 0.0139$$

$$\rho_{max} = 0.85 * \beta_1 * \frac{f_c'}{f_y}\left(\frac{\varepsilon}{\varepsilon + 0.004}\right) = 0.85 * 0.85 * \frac{4}{60} * \left(\frac{0.003}{0.003 + 0.004}\right) = 0.02064$$

$$\rho_{min} = 3 * \frac{\sqrt{f_c'}}{f_y} = 3 * \frac{\sqrt{4000}}{60,000} = 0.00316 \leq \rho_{min} = \frac{200}{f_y} = \frac{200}{60,0000} = 0.00333 \therefore \rho_{min} = 0.00316$$

Quantity of steel is between min and max. OK

Verification of φ for Negative Moment

$$\rho_{limit} = 0.85 * \beta_1 * \frac{f_c'}{f_y}\left(\frac{\varepsilon}{\varepsilon + 0.005}\right) = 0.85 * 0.85 * \frac{4}{60} * \left(\frac{0.003}{0.003 + 0.005}\right) = 0.01806$$

$$\rho = 0.0139 \leq \rho_{limit} = 0.01806 \rightarrow \phi = 0.9$$

Negative Reinforcement

$$Moment = 2995\ kip - ft$$

$$a = \frac{A_s f_y}{(0.85 f_c' b_w)} = \frac{60 * A_s}{(0.85 * 4 * 36)} = 0.49As$$

$$A_s = \frac{M_u}{\left(\phi f_y \left(d - \left(\frac{a}{2}\right)\right)\right)} = \frac{2995kft(12)}{0.9 * 60 * \left(50 - \frac{a}{2}\right)}$$

Solving the quadratic equation A_s=14.3 in^2
SAFE Specified As= 14.5 in^2
10 # 11 bars will be used As=15.6 in^2

$$\rho = \frac{A_s}{bd} = \frac{15.6}{36 * 50} = 0.0087$$

$$\rho_{max} = 0.85 * \beta_1 * \frac{f_c'}{f_y}\left(\frac{\varepsilon}{\varepsilon + 0.004}\right) = 0.85 * 0.85 * \frac{4}{60} * \left(\frac{0.003}{0.003 + 0.004}\right) = 0.02064$$

$$\rho_{min} = 3 * \frac{\sqrt{f_c'}}{f_y} = 3 * \frac{\sqrt{4000}}{60{,}000} = 0.00316 \leq \rho_{min} = \frac{200}{f_y} = \frac{200}{60{,}0000} = 0.00333 \therefore \rho_{min} = 0.00316$$

Quantity of steel is between min and max. OK

Verification of φ for Positive Moment

$$\rho_{limit} = 0.85 * \beta_1 * \frac{f_c'}{f_y}\left(\frac{\varepsilon}{\varepsilon + 0.005}\right) = 0.85 * 0.85 * \frac{4}{60} * \left(\frac{0.003}{0.003 + 0.005}\right) = 0.01806$$

$$\rho = 0.0087 \ \leq \rho_{limit} = 0.01806 \rightarrow \phi = 0.9$$

Shear Reinforcement

Maximum Shear from SAFE = 831 kips

$$\phi V_C = 0.75\left(2\sqrt{f_c'}b_w d\right)\lambda = 0.75 * \left(2 * \sqrt{4000} * 36 * 50\right) * 1 = 227.7k$$

$$\phi V_C = 227.7k < V_d = 831k \ \therefore need\ shear\ reinforcement$$

Assuming #5 Four legged Stirrups ($A_v = 1.24\ in^2$)

$$\phi V_s = V_u - \phi V_c = 831k - 227.7k = 603.3k$$

Check V_s

$$\phi V_s = 603.3 \leq \frac{4\phi\sqrt{f_c'}b_w d}{1000} = \frac{4 * 0.75 * \sqrt{4000} * 50 * 36}{1000} = 341.5k$$

$$\frac{d}{2} = \frac{50}{2} = 25in$$

$$Spacing\ S_{max} = min \quad 24in$$

$$S = \frac{\phi A_v f_y d}{V_u - \phi V_c} = \frac{0.75 * 1.24 * 60 * 50}{603.3} = 4.6\ in$$

$4"\ spacing\ from\ face\ of\ column\ until\ 10\ feet\ from\ face$
$Then\ space\ at\ 12"\ until\ shear\ wall$

Reinforcement Details

Development Length

Development length of top bar hooks ACI 318-11 12.5.2

$$l_{dh} = \frac{[0.02 * 60000]}{\sqrt{4000}}\left(\frac{11}{8}\right) = 26.1\text{=}30"\text{=}\mathbf{2.5'}$$

Development length of #11 bars ACI 318-11 12.2.3

$$l_{dh} = \frac{[60000]}{20\sqrt{4000}}\left(\frac{11}{8}\right) = 65.22"\text{=}\mathbf{5.5'}$$

Class B Splice Tension ACI 318-11 12.15.1

$$1.3 * l_d = 1.3 * 65.22" = 85in \cong 7.5'$$

Splice Compression ACI 318-11 12.16.1

$$0.0005 f_y d_b = 0.0005(60000)\left(\frac{11}{8}\right) = 41.25in \cong 3.5'$$

Columns

There were a total of 36 columns located in the building. The columns were laid out in way to reduce the total number in the building by maximizing the allowable space between them (20 feet to avoid deflection analysis). Given the architectural restrictions and preliminary estimates, the columns were to be one foot by three feet except for corner columns in the building which were circular columns of two feet in diameter. It was determined that columns 9, 13, 24, and 28 needed to be increased in size to 16-inch by 36-inch in order to maintain a minimal percentage of reinforcement. The columns on the garage level were increased wherever they did not interfere with the parking layout. The outer rectangular columns against the retaining wall were increased to 42 inches by 18 inches, while the corner columns were increased to 42 inch square columns. The corner columns were made into square columns to make the formwork easier with the basement wall. All column sizes were kept the same above the garage level. This was done to increase the speed and economy of construction by being able to utilize the same forms on every floor. This was also done to keep the percentage of steel in each column very low due to column-slab intersections being junctions for intersecting rebar. Although the ACI code permits rebar in columns to be a minimum of one percent to a maximum of eight percent, a maximum of four percent was used in our building whenever possible for constructability reasons. Reducing the amount of vertical reinforcement in columns will allow for column slab intersections to be less crowded, lead to fewer mistakes, and allow for the concrete to flow much better when poured. All outer columns terminated at the top of the twelfth floor, at the roof promenade, while the remaining columns continued up until the roof.

First, all of the loads were determined on the columns. Snow, live, and dead loads were accounted for in the design. All of the loads and moments transferred from the slabs to the columns were determined in SAFE for each column. They were tabulated, so the total load on the column at any floor could be determined readily. Column design was performed in Structure Point Column (spColumn). Sample calculations for circular and rectangular columns are shown below. Note that due to the symmetry of the building and loading, there were several columns repeated in the building. For economy, the worst of the columns in these groups was designed and utilized for the others. This will also make construction much faster due to the repetitiveness of rebar layout. It will be much simpler for workers to perform one task several times, rather than several different layouts.

All columns were designed to only resist gravity loads. The shear wall will be designed to resist the lateral loads. The concrete frame is determined to be a braced frame.

Circular Column

To illustrate the design process for the circular columns in the building column (07) A.1-6.2 procedure for the second floor will be shown. The column below the second floor is 14 feet while the column above the floor is 10 feet. The clear height of the column on the second floor is 10 feet. The column was 2 feet in diameter.

Loads

Below is the summary for the loads on the column. The load combination used to design the column was 1.2Dead+1.6Live+0.5Snow. The total factored axial load was determined to be 621.6 kips.

COLUMN A.1-6.2 SECOND FLOOR LOADS				
Floor	**Dead (kip)**	**Live (kip)**	**Snow (kip)**	
3	36.38	19.8	0	
4	36.4	7.5	0	
5	36.4	7.5	0	
6	36.4	7.5	0	
7	36.4	7.5	0	
8	36.4	7.5	0	
9	36.4	7.5	0	
10	36.4	7.5	0	
11	36.4	7.5	0	
12	36.4	7.5	0	
14	33.9	0	8.9	
15	0	0	0	
16	0	0	0	
TOTAL	**397.88**	**87.3**	**8.9**	
Factor	1.2	1.6	0.5	**Total**
FACTORED TOTAL	**477.456**	**139.68**	**4.45**	**621.6 kips**

The moments at the top and bottom of the column are shown below. They were also determined from the SAFE models.

Unfactored Second Floor Moments				
	Dead Load		Live Load	
	Mx (kip-ft)	My (kip-ft)	Mx (kip-ft)	My (kip-ft)
Top	-1.2	1	1.6	-0.81
Bottom	3.4	-3.2	2	-2

Reinforcement

$$f'c = 4000\,psi \quad fy = 60{,}000\,psi$$

$$(Slenderness\ Ratio)\frac{klu}{r} = \frac{(1)(120)}{\frac{12^2}{2}} = 1.66 < 34 - 0\ (\boldsymbol{non\ slender\ member})$$

Factored Moments

$$M_x = 1.2(3.4) + 1.6(2) = 7.28\,kip * ft$$
$$M_y = 1.2(3.2) + 1.6(2) = 7.04\,kip * ft$$

$$Mx = M_x + M_y\left(\frac{h}{b}\right)\left(\frac{1-\beta}{\beta}\right) = 7.28 + 7.04\left(\frac{24}{24}\right)\left(\frac{1-0.6}{0.6}\right) = \mathbf{11.97\ k \cdot ft}$$

P-M Diagrams

Enter P-M Diagrams with:

$$\boldsymbol{Mx = 11.97\ k \cdot ft}$$
$$\boldsymbol{P = 621.6\ kip}$$

$$Kn = \frac{Pu}{\phi f'cAg} = \frac{621.6}{(0.65)(4)(452.4)} = 0.528$$

$$Rn = \frac{Mu}{\phi f'cAgh} = \frac{11.97 * 12}{(0.65)(4)(452.4)(24)} = 0.005$$

Use 2 inch clear cover to shear reinforcement (#4 bars), assume #14 bars or smaller will be used for longitudinal reinforcement.

$$\gamma = \frac{24-2\left(2+0.5+\frac{7}{8}\right)}{24} = 0.718$$ (True if #14 bars or smaller are used)

Use $\gamma = 0.7\ P - M\ Diagrams$

$Use\ graphs\ for\ \gamma = 0.70\ \rho_g = 0.01\ (in\ compression\ zone)$

$$\rho_g = \frac{As}{bd} => As = (0.01)(\pi 12^2) = 4.52\ in^2 \therefore\ \boldsymbol{Use\ 4\#10}\ A_S = 5.08\ in^2$$

$$\rho = 0.0112$$

Verify Strength

$$\frac{e}{h} = \frac{ex}{lx} = \frac{Mu_y}{Pulx} = \frac{7.04 * 12}{621.6 * 24} = 0.0057$$

$$\frac{Pnx_o}{f'cAg} = 0.95 => Pnx = 1719\ k$$

$$\frac{Po}{f'cAg} = 1.02 => Po = 1846\ k$$

$$\frac{e}{h} = \frac{ex}{lx} = \frac{Mu_x}{Pulx} = \frac{7.28 * 12}{621.6 * 24} = 0.0059$$

$$\frac{Pny_o}{f'cAg} = 0.95 => Pny = 1719\ k$$

$$\frac{1}{Pn} = \frac{1}{Pnx_o} + \frac{1}{Pny_o} - \frac{1}{Po} = \frac{1}{1719} + \frac{1}{1719} - \frac{1}{1846} => Pn = 1609\ k$$

$$\phi Pn = (0.65)(1609) = 1045\ k$$

$$(0.80)(0.65)(3600) = 959.92 < 1045\ k$$

$$\boldsymbol{\phi Pn = 1045} > 621.6\ k\ (\boldsymbol{column\ is\ adequate})$$

Transverse Reinforcement

$$s \le \left\{ \begin{matrix} 8d_b = 8(10/8) = 10\ in \\ 24d_{t_b} = 24\left(\frac{1}{2}\right) = 12\ in \\ \frac{1}{2} least\ dimension = 12\ in \\ 12\ in \end{matrix} \right\} use\ 10"\ spacing$$

Use #4 ties spaced at 10” on center

Square Column

To illustrate the design process for square and rectangular columns in the building column (07) A.1-6.2 procedure for the garage floor will be shown. The column above the floor is 10 feet long. The clear height of the column on the garage floor is 14 feet. The column is 2.5 feet by 2.5 feet.

Loads

Below is the summary for the loads on the column. The load combination used to design the column was 1.2Dead+1.6Live+0.5Snow. The total factored axial load was determined to be 713.4 kips.

Garage level Design				
Floor	**Dead (kip)**	**Live (kip)**	**Snow (kip)**	
2	51.2	19	0	
3	36.38	19.8	0	
4	36.4	7.5	0	
5	36.4	7.5	0	
6	36.4	7.5	0	
7	36.4	7.5	0	
8	36.4	7.5	0	
9	36.4	7.5	0	
10	36.4	7.5	0	
11	36.4	7.5	0	
12	36.4	7.5	0	
14	33.9	0	8.9	
15	0	0	0	
16	0	0	0	
TOTAL	**449.08**	**106.3**	**8.9**	
Factor	1.2	1.6	0.5	**TOTAL**
FACTORED TOTAL	**538.896**	**170.08**	**4.45**	**713.4 kips**

Unfactored Garage Floor Moments				
	Dead Load		Live Load	
	Mx (kip-ft)	My (kip-ft)	Mx (kip-ft)	My (kip-ft)
Top	3.4	-3.2	2	-2

Reinforcement

$$f'c = 4000\ psi\ \ fy = 60{,}000\ psi$$

$$(Slenderness\ Ratio)\frac{klu}{r} = \frac{(1)(168)}{(0.3)(30)} = 18.7 < 34 - 0\ (\textbf{non slender member})$$

<u>Factored Moments</u>

$$M_x = 1.2(3.4) + 1.6(2) = 7.28\ kip * ft$$
$$M_y = 1.2(3.2) + 1.6(2) = 7.04\ kip * ft$$

$$Mx = M_x + M_y\left(\frac{h}{b}\right)\left(\frac{1-\beta}{\beta}\right) = 7.28 + 7.04\left(\frac{30}{30}\right)\left(\frac{1-0.6}{0.6}\right) = \mathbf{11.97\ k \cdot ft}$$

<u>P-M Diagrams</u>
Enter P-M Diagrams with:

$$\boldsymbol{Mx = 11.97\ k \cdot ft}$$
$$\boldsymbol{P = 713.4\ kip}$$

$$Kn = \frac{Pu}{\phi f'cAg} = \frac{713.4}{(0.65)(4)(30^2)} = 0.305$$

$$Rn = \frac{Mu}{\phi f'cAgh} = \frac{11.97 * 12}{(0.65)(4)(30^2)(30)} = 0.002$$

Use 2 inch clear cover to shear reinforcement (#4 bars), assume #14 bars or smaller will be used for longitudinal reinforcement.

$$\gamma = \frac{30-2\left(2+0.5+\frac{7}{8}\right)}{30} = 0.775$$ (True if #14 bars or smaller are used)

Use $\gamma = 0.7\ P - M\ Diagrams$

$Use\ graphs\ for\ \gamma = 0.70\ \rho_g = 0.01\ (in\ compression\ zone)$

$$\rho_g = \frac{As}{bd} => As = (0.01)(30^2) = 9.0\ in^2 \therefore\ \ Use\ 8\#10\ A_S = 10.16\ in^2$$

$$\rho = 0.011$$

<u>Verify Strength</u>

$$\frac{e}{h} = \frac{ex}{lx} = \frac{Mu_y}{Pulx} = \frac{7.04 * 12}{713.4 * 30} = 0.039$$

$$\frac{Pnx_o}{f'cAg} = 0.94 => Pnx = 3384\ k$$

$$\frac{Po}{f'cAg} = 1.00 => Po = 3600\ k$$

$$\frac{e}{h} = \frac{ex}{lx} = \frac{Mu_x}{Pulx} = \frac{7.28 * 12}{713.4 * 30} = 0.004$$

$$\frac{Pny_o}{f'cAg} = 0.98 => Pny = 3528\ k$$

$$\frac{1}{Pn} = \frac{1}{Pnx_o} + \frac{1}{Pny_o} - \frac{1}{Po} = \frac{1}{3384} + \frac{1}{3528} - \frac{1}{3600} => Pn = 3320\ k$$

$$\phi Pn = (0.65)(3320) = 2158\ k$$

$$(0.80)(0.65)(3600) = 1872k < 2158\ k$$

$$\boldsymbol{\phi Pn = 1872} > 713.4\ k\ (\boldsymbol{column\ is\ adequate})$$

<u>Transverse Reinforcement</u>

$$s \le \left\{\begin{matrix} 8d_b = 8(14/8) = 14\ in \\ 24d_{t_b} = 24\left(\frac{1}{2}\right) = 12\ in \\ \frac{1}{2}least\ dimension = 15\ in \\ 12\ in \end{matrix}\right\}\ use\ 12"\ spacing$$

Use #4 ties spaced at 12” on center

Splice

Bottom column 8 #10 $d_b = 1.41"$
Top column 4 #10 $d_b = 1.41"$

$$l_{dc} = \frac{0.02 f_y}{\sqrt{f'c}} d_b = \frac{0.02 * 60{,}000 * 10/8}{\sqrt{4000}} = 23.7" \; for \; \#10 \; bars$$

$$l_{sc} = (0.0005) f_y d_b = 0.0005 * 60000 * \frac{10}{8} = \mathbf{37.5" \; for \; \#10 \; bars}$$

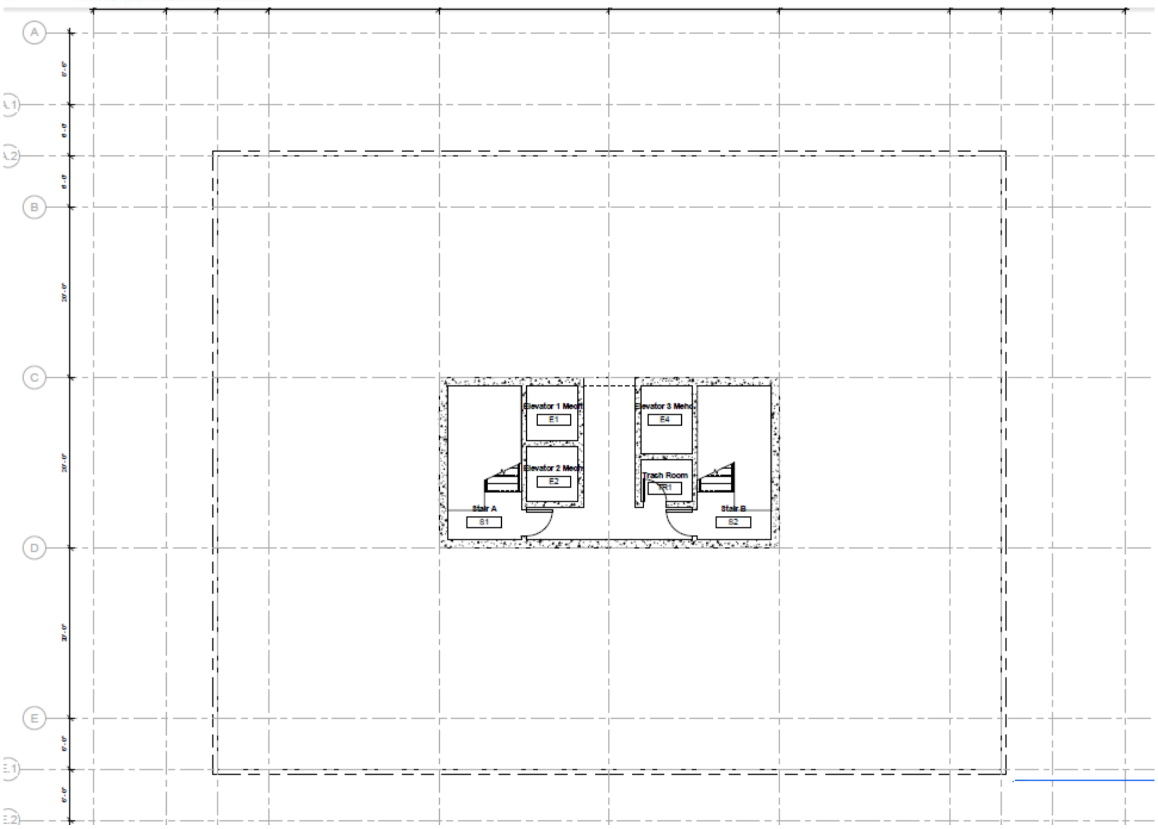
$$Lap \; Splice = 37.5" = \mathbf{3.5'}$$

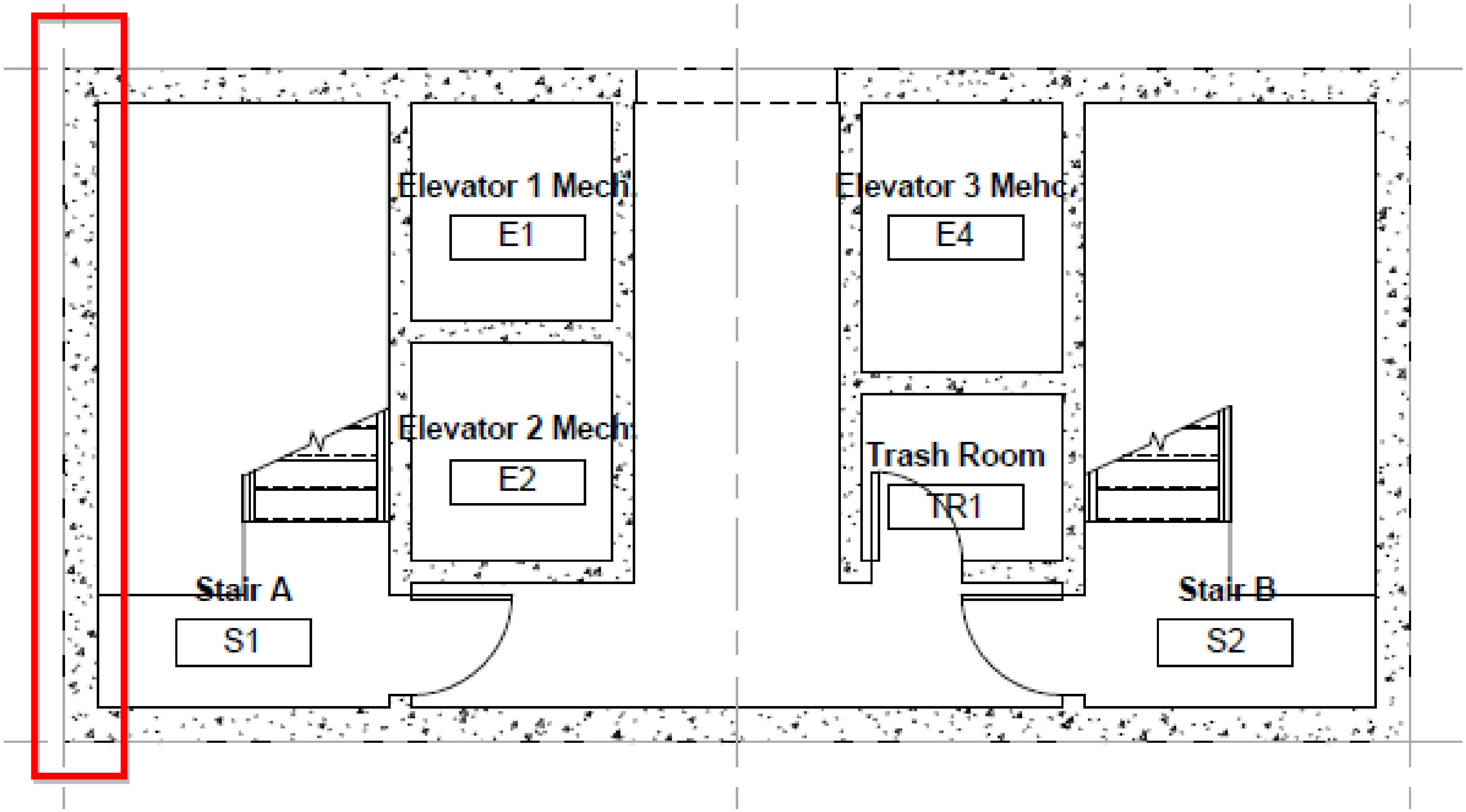
Shear Wall

Two special reinforced C-shaped shear walls were used to resist all of the lateral forces in the structure. They were located around the core of the building as shown below in the figures. The shear wall is 18 inches thick on the garage level and 12 inches thick for the remainder of the building. The shear wall is 18-inches thick on the first floor to accommodate for the large axial loads due to the transfer beams. Also, extra concrete cover was used on the first floor to accommodate for being exposed to weathering and road salt in the garage. The shear wall is 20 feet in length along the north/south axis and 40 feet along the east/west axis. There is a six foot wide by eight foot tall doorway cut out of the shear wall on the north and south faces to provide access to the elevators. For the purpose of designing and modeling the shear wall these openings were considered to be continuously open in the entire building and their added strength was neglected. The shear walls were the entire height of the building from the garage to the top of roof (163’-4”). They also extended eight feet below the garage level to the bottom of the elevator pit, which was not accounted for or modeled when designing the structure.

The shear walls were designed to resist all lateral forces, in addition to gravity loads, in ETABS. They were designed to resist snow, live, dead, earthquake, and wind loads. The gravity load reactions on the shear wall were determined from the SAFE model of each slab. The lateral loads were determined according to IBC 2006. They were determined by hand and also checked in ETABS. The earthquake loads were applied with 0.05 eccentricities in each direction, to account for construction errors in the building. The shear wall was modeled as two C-shaped walls with rigid diaphragms for the slabs at each level in ETABS. The seismic mass of each floor was defined in the ETABS model. Rigid diaphragms were utilized because they would transfer all of the lateral forces to the shear wall, rather than absorb them. The snow, live, and dead loads were then applied to the wall at each level. Since the short period response (S_{DS}) was 0.288, which is larger than 0.125, the vertical effects of a seismic event needed to be taken into account. They were applied at each floor as $0.2S_{DS}$*Dead.

The torsional irregularities were checked according to ASCE 7-05 Table 12.3.1 in ETABS. The maximum story drift was less than 1.2(average story drift) for each floor and load combination (largest was 1.14). There were no horizontal or vertical irregularities in the structure.


A
B
C
D
E
Elevator 1 Mech
E1
Elevator 2 Mech
E2
E4
Trash Room
TR1
Stair A
S1
Stair B
S2


Elevator 1 Mech.
E1
Elevator 2 Mech.
E2
Elevator 3 Mehc.
E4
Trash Room
TR1
Stair A
S1
Stair B
S2

Design

A sample calculation for the western portion of the wall on the first floor in the north/south direction will be shown below; the portion of the wall to be designed is shown in the figure above. The height of the wall was 14 feet. The wall was 20 feet long and 18 inches thick.

Check if Braced Against Side Sway

$$\sum Shearwall\ Stiffness \geq 12 \sum column\ stiffness$$

$$\left(2\frac{18*240^3}{12}+4\frac{204*18^3}{12}\right) \geq 12\left(16\frac{12*36^3}{12}+16\frac{36*12^3}{12}+\pi 24^4\right)$$

$$41{,}868{,}576 \geq 22{,}460{,}940$$

Shear wall is considered to be braced against side sway.

Minimum Thickness of Wall

$$t_{wall} \geq \frac{l_u}{20} = \frac{240}{20} = 12"$$

Thickness of wall =18" is sufficient.

Loads

The controlling load combinations from ETABS were $D_{Wall}6$ and $D_{wall}43$
Flexural Design Controlling Load Case:

$$D_{wall}43 = 0.8Dead + 1.0Seismic$$
$$P_u = 2297kip \qquad M_u = 5{,}655kipft$$

Shear Design Controlling Load Case:

$$D_{wall}6 = 1.2Dead + 1.0Live + 0.5Snow + 1.0Wind$$
$$P_u = 3761.5kip \qquad M_u = 4{,}811kipft \qquad V_u = 332.5kips$$

Reinforcement

Horizontal distributed reinforcement designed according to **ACI 318 Sec 21.9.2.1**

$$Max\ horizontal\ spacing \leq (18in)$$
$$a\ spacing\ of\ 12"\ will\ be\ used$$

Horizontal and longitudinal reinforcement shall be at least:

$$\rho_l \& \rho_t \geq 0.0025$$

Horizontal reinforcement shall be continuous

At least 2 curtains of reinforcement are needed **ACI 318 Sec 21.9.2.2**

Depth (d) =0.8l_{wall}=0.8*240=192” **ACI 318 Sec 21.9.2.3**

Development length of longitudinal bars shall be 1.25*development length in tension

Not less than 2 #5 bars in both directions in 2 layers shall be provided around openings and developed in tension development length **ACI 318 Sec 14.3.7**

Horizontal Reinforcement

Check if 2 rows of #5 @12” is OK

$$\rho_t = \frac{0.31\ in^2 * 2\ row}{12in * 18in} = 0.00287 > 0.0025$$

Vertical Reinforcement

Check if 2 rows of #5 @ 12” is ok for longitudinal bars.

$$\rho_l = 0.0025$$
$$Maximim\ vertical\ spacing \leq \left(\frac{l_w}{3}, 3t_w, 18in\right)$$
$$12"\ spacing\ for\ vertical\ will\ be\ used$$
$$\rho_l = \frac{0.31in^2 * 2\ rows}{18in * 12in} = 0.00287 > 0.0025$$

Weak Axis Load Capacity

Estimate of weak axis axial load capacity

$$\phi P_{n_w} = 0.55\phi f_c' A_g [1 - \left(\frac{kl_c}{32t_w}\right)^2 ; \phi = 0.70\ for\ bearing\ wall$$

$$\phi P_{n_w} = 0.55 * 0.7 * 4000psi * 240in * 18in * [1 - \left(\frac{0.8\left(\left(14\,ft * 12\frac{in}{ft}\right)\right)}{32 * 18in}\right)^2$$

$$\phi P_{n_w} = 6{,}290.6\ kip > 3{,}761.5\ kip\ (Load\ Case\ D_{wall}6)\ OK$$

#5 @12" is OK

Shear

Effective depth =0.8h=0.8=192"

Max Allowable Shear **ACI 318 Sec 21.9.4.1**

$$V_n = A_{cv}\left(\alpha_c\lambda\sqrt{f_c'} + \rho_t f_y\right) = 18 * 192\left(3 * 1\sqrt{4000} + 0.00287 * 60{,}000\right)$$

$$V_n = 1{,}250\ kip > 332.3kip\ (Load\ Case\ D_{wall}6)OK$$

#5 @12" is OK

Concrete Shear Strength

Use minimum calculated V_c

Equation for walls under compressive loads,

$$V_c = 2\sqrt{f_c'}dt_w = 2 * \sqrt{4000} * 192in * 18in$$

$$\boldsymbol{V_c = 437.3\ kip}$$

Equation 11.29 from ACI 318

$$V_c = 3.3\sqrt{f_c'}dt_w + \frac{P_u d}{4 * l_w * t_w} = 3.3\sqrt{4000} * 192in * 18in + \frac{3761.5 * 1000 * 192in}{4 * 240in * 18in}$$

$$V_c = 763.1\ kip$$

Equation 11.30 from ACI 318

$$V_c = \left[0.6\sqrt{f_c'} + \frac{l_w\left(1.25\sqrt{f_c'}\right) + \frac{0.2P_u}{l_w t_w}}{\left(\frac{M_u}{V_u}\right) - \left(\frac{l_w}{2}\right)}\right]dt_w$$

$$V_c = \left[0.6\sqrt{4000} + \frac{240in\left(1.25\sqrt{4000}\right) + \frac{0.2 * 3761.5 * 1000}{240in * 18in}}{\left(\frac{4811.3\ kipft * 12000}{332.5kip * 1000}\right) - \left(\frac{240in}{2}\right)}\right] 192in * 18in$$

$$\phi V_c = 1364.8\ kip$$

$$V_c = 437.3\ kip$$

$$V_u > .5\phi V_c$$

$$332.3kip > 0.5 * 0.75 * 437.3kip = 164kip$$

$$\therefore Use\ minimum\ shear\ reinforcement\ because\ 0.5\phi V_c < V_u < \phi V_c$$

$$\phi V_c = 328\ kip \cong 332kip$$

#5 @12” is OK

<u>Flexural Design</u>

Slender Shear wall

$$\frac{h_w}{l_w} = \frac{163}{20} = 8.17 > 2 \therefore Slender\ shear\ wall$$

Nominal Tension Controlled Limit

$$C_{CTL} = 0.32\beta_1 f_c' d_t t_w = 0.32 * 0.85 * 4000psi * 236in * 18in$$

$$C_{CTL} = 4{,}621.8\ kip$$

Nominal Compression Controlled Limit

$$C_{CCL} = 0.5 f_c' d_t t_w = 0.5 * 0.85 * 4000psi * 236in * 18in$$

$$C_{CCL} = 7{,}221.6kip$$

$$P_u = 2{,}297kip < C_{CTL} = 4{,}621.8\ kip < C_{CCL} = 7{,}221.6\ kip$$

$$\therefore section\ is\ tension\ controlled, \boldsymbol{\phi = 0.9}$$

$$A_{s_{total}} = 19\ bars * 2\ rows * 0.31\ in^2 = 11.78in^2$$

$$\omega = \frac{A_{s_t} f_y}{f_c' l_w t_w} = \frac{11.78\ in^2 * 60000psi}{4000psi * 240in * 18in} = 0.0409$$

$$\alpha = \frac{P_u}{f_c' l_w t_w} = \frac{2297kip * 1000\ lb/kip}{4000psi * 240in * 18in} = 0.133$$

$$\frac{c}{l_w} = \frac{\omega + \alpha}{2\omega + \alpha_1\beta_1} = \frac{0.0409 + 0.133}{2 * 0.0409 + 0.85 * 0.85} = 0.2162$$

$$c = 0.2162 l_w = 0.2162 * 240in = 51.9in$$

$$\begin{aligned} M_n &= 0.5 A_{st} f_y l_w \left(1 + \frac{P_u}{A_s f_y}\right)\left(1 - \frac{c}{l_w}\right) \\ &= 0.5 * 11.78in^2 \\ &* 60{,}000psi(240in)\left(1 + \frac{2{,}297{,}000lb}{11.78in^2 60{,}000psi}\right)(1 - 0.2162) \\ &= 23{,}543.8\ kip * ft \end{aligned}$$

$$\phi M_n = 0.9(23{,}543.8) = 21{,}190\ kip * ft >> 4{,}811\ kip * ft$$

#5 @12” is OK

Summary

2 Rows of #5 @ 12” for longitudinal reinforcement

2 Rows of #5 @ 12” for transverse reinforcement

2 #8 at both ends for longitudinal boundary elements

#3 ties will be placed to tie each longitudinal bar pair

Footings

The soil on the site was determined to be well graded sand with a bearing capacity of 12,000 pounds per square foot. Below the well graded sand there is a fractured rock layer with an allowable bearing capacity of 24,000 pounds per square foot. Since the bearing capacity of the soil is relatively high, isolated shallow footings will be utilized. This would be a more economical solution than using a mat foundation. Also, piles are not necessary due the relatively high bearing capacity of the sand. Continuous shallow wall footings will be used for the retaining wall and shear walls. Square footings were utilized wherever possible to make construction much easier. The frost line in Yonkers is 3.5 feet below grade. Although most of the footings are interior, the bottom of all footings will begin at 3.5 feet below grade. This will be done to ensure that frost heaving will not occur, during and after construction. The first floor garage may not be heated during the winter, which may lead to the possibility of frost heaving. All of the footings were designed so the axial force of the column would act within the middle third of the footing, to ensure the soil would always be in compression. Wherever columns were embedded into the retaining walls, the footings below the column were designed to be isolated from the wall footings, although they will not be once constructed. The footings were designed a depth that would ensure no shear reinforcement would be necessary. They were also designed for in both directions. All footings were also checked if they had the proper bearing capacity strength. Wherever the bearing capacity strength was insufficient, dowels were placed. The size of the dowels was typically chosen to match the size of the reinforcement in the respective column. A sample calculation for column 09 will be shown, which had the largest axial force to resist.

Column 09

Column 09 is a 36 inch by 16 inch interior column with 12 #10 bars for reinforcement. The frost line in Yonkers NY is 3.5 feet below grade. Although this is an interior footing, it will be designed below the frost line because the first floor is an open unheated garage. Also, they will be designed below the frost line just in case the construction of the footings occurs during the winter time. The first 25 feet of soil below grade consists of well graded sand with a bearing capacity of 12 ksf.

Bearing Strength: $q_a = 12{,}000\ psf$
Soil unit weight: $w_s = 120\ pcf\ (assumed)$
Column dimensions: 36" x 16"

$$f'c = 4{,}000\ psi$$
$$f_y = 60{,}000\ psi$$

Size of Footing

Frost line: ~3.5 ft. below grade
Base of footing: 3.5 ft. below grade

Loads (Unfactored) Case 1.2Dead+1.6Live+0.5Snow:

$$DL = 854\ kip$$
$$LL = 217\ kip$$
$$Snow = 21\ kip$$
$$P_a = 1092\ kip\ (Unfactored)$$

Footing Sizing (Service Loads):
Concrete weight

$$150\ pcf$$

Soil bearing strength

$$q_a = 12{,}000\ psf$$
$$q_{net} = q_a - w_c - LiveLoad - DeadLoad$$

$$q_{net} = 12{,}000\ psf - 150\ pcf(3.5\ ft) - 50\ psf - 86psf$$
$$q_{net} = 11{,}339\ psf$$

$$A_{req} = \frac{P_a}{q_{net}} = \frac{1092\ kip}{11.339ksf} = 96.3\ ft^2$$
$$96.3\ ft^2 < 10'x10' = 100\ ft^2 \therefore use\ 10'x\ 10'$$

Footing Depth

(Factored Loads): Case 1.2 DL + 1.6 LL + 0.5 S

$$P_u = (1.2 * 854\ kip + 1.6 * 217\ kip + 0.5 * 21 kip)$$

$$P_u = 1382\ kip$$

$$q_u = \frac{P_u}{A_g} = \frac{1382\ kip}{100\ ft^2} = 13.82 \frac{kip}{ft^2}$$

Punching Shear

Critical Perimeter = $b_o = 2(36 + d) + 2(16 + d)$

$$V_u@d = q_u * (A_{footing} - A_{critical})$$

$$V_u@d = 13.82 \frac{kip}{ft^2} * \frac{10\ ft^2 - (36 + d)\quad(16 + d)}{144 \frac{in^2}{ft^2}}$$

$$Interior\ Column\ \alpha_s = 40$$

$$Rectangular\ Column\ \beta_c = \frac{36}{16} = 2.25$$

For V_c rectangular column, assume $V_c = \left(2 + \frac{4}{\beta}\right) \lambda \sqrt{f_c'} b_o d$ controls

$$V_u = \phi V_c$$

$$1000 \frac{lb}{kip} * \left[13.82 \frac{k}{ft^2} * \left(10^2\ ft^2 - \frac{(36 + d)\quad(16 + d)}{144 \frac{in^2}{ft^2}} \right) \right]$$

$$= 0.75 * \left(2 + \frac{4}{2.25}\right) * 1 * \sqrt{4000} * [2(16 + d) + 2(36 + d)]d$$

$$d \approx 28.4\ in \rightarrow Use\ d = 30\ in.$$

$$b_o = 224\ in.$$

Verify V_c

$$\left(\frac{\alpha_s d}{b_o} + 2\right) \lambda \sqrt{f_c'} b_o d = \left(\frac{40 * 30"}{224\ in} + 2\right) \lambda \sqrt{f'_c} * b_o d$$

$$7.35 \lambda \sqrt{f'_c} * b_o d > 3.78 \lambda \sqrt{f'_c} * b_o d \therefore O.K.$$

Beam Type Shear: 1 Way Shear

$d = 30\ in.$

$c = 16\ in.$

$$V_u@d = 13.82 \frac{kip}{ft^2} \left(\frac{10 ft}{2} - \frac{16 in}{\frac{24 in}{ft}} - \frac{30\ in}{12 \frac{in}{ft}} \right) * 10\ ft$$

$$V_u@d = 253.4\ kip$$

$$V_c = 2\lambda \sqrt{f_c'} b_o d = 2\sqrt{4000} * 10\ ft * 12 \frac{in}{ft} * 30\ in$$

$$V_c = 455.4\ kip$$
$$\phi V_c = 0.75 * 455.4\ kip = 341.6\ kip$$
$$\phi V_c > V_u@d$$

Also adequate for beam shear

Flexural Reinforcement

$$P_u = 1382\ kip$$
$$V_u \cong 0\ kip$$
$$d = 30"$$
$$h = 36"$$
$$M_{u_{shear}} = V_u h = 0\ kip * 36"$$
$$M_{u_{shear}} = 0\ k - in$$
$$M_u = P_u e \rightarrow e = \frac{0\ kip - in}{1382\ kip} = 0\ \therefore loading\ occurs\ in\ middle\ third\ of\ footing$$

Moment will be disregarded because it is so small.

Loading in Kern:

$$q_{max} = \frac{P_u}{AB} + \frac{M_x\left(\frac{B}{2}\right)}{I}; I = \frac{AB^2}{12}$$
$$q_{max} = \frac{1382\ kip}{10^2\ ft^2} + 0 = 13.82\frac{kip}{ft^2}$$

$$M_u = \frac{13.82\frac{k}{ft^2}}{2} * 10\ ft * \left(\frac{10ft}{2} - \frac{16\ in}{12\frac{in}{ft} * 2}\right)^2 = 1297.5\ kip - ft$$
$$A_{s_{required}} = \frac{M_u}{\phi f_y\left(d - \left(\frac{a}{2}\right)\right)}$$
$$A_{s_{required}} = \frac{0.85f'cba}{f_y}$$
$$A_{s_{required}} \approx 9.84\ in^2$$
$$a \approx 1.44\ in.$$

Check ρ and $A_s\ min$

$$\rho_{min} \geq \frac{200}{f_y}, \frac{3\sqrt{f'c}}{f_y}$$
$$\rho_{min} = 0.00333$$
$$A_{s_{min}} = \rho_{min}bd = 0.00333 * 10ft * 12\frac{in}{ft} * 30in = 11.99in^2$$
$$A_{s_{min}} > A_{s_{required}} \therefore A_s = A_{s_{min}}$$

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Spacing limits for slab used to find max spacing

$$Max\ Spacing \leq 3h\ or\ 18"$$
$$3h = 3 * 36 = 108 \therefore s_{max} = 18"$$

Use 28#6 As=12.32in^2

Development length in main reinforcement
Requirements: $Clear\ spacing \geq 2d_{bar}; Clear\ cover \geq d_{bar}$
Simplified Equation:

$$\frac{l_d}{d_{bar}} = \frac{f_y \psi_t \psi_e}{20\sqrt{f'c}\lambda}$$

$\psi_t = 1.0\ for\ lower\ face\ rebar$
$\psi_e = 1.0\ for\ uncoated\ bars$
$\lambda = 1.0\ for\ normal\ weight\ concrete$

$$l_d = \frac{f_y \psi_t \psi_e}{20\sqrt{f'c}\lambda} * d_{bar} = \frac{60000\ psi * 1 * 1}{20\sqrt{4000} * 1} * 0.75in = 35.5\ in$$

Minimum cover for footings=3”

$$l_{embedded} = \frac{\left(10ft * 12\frac{in}{ft}\right)}{2} - \frac{36in}{2} - 3in = 39\ in$$
$$l_{embedded} > l_d \therefore O.K.$$

Total footing depth:

$$h = d + \frac{d_b}{2} + d_b + \min cover$$
$$h = 30\ in + \left(\frac{.75in}{2}\right) + 0.75in + 3in = 34.125\ in \therefore use\ h = 36\ in$$

Dowels

Bearing capacity for column

$$\phi P_n = \phi(0.85)f_c' A_1$$

$$\phi P_n = 0.65(0.85)4000psi * (16in)(36in) = 1272.96\ kip < 1382\ kip$$
$$\therefore reinforcement\ will\ be\ necessary$$

Bearing capacity for footing

$$\phi P_n = \phi(0.85)f'cA_1\sqrt{\frac{A_2}{A_1}} \leq 2\phi(0.85)f_c' A_1$$

1:2 slope

$$A_2 = (2h + column\ width)^2 = \big((2*36in) + (36in)\big)\ \ \big((2*36) + 16\big) = 9504in^2$$
$$= 66\ ft^2$$

$$\sqrt{\frac{A_2}{A_1}} = 4.06$$

$$\phi P_n = 0.65(0.85) * 4000 * 16 * 36 * 4.06 \leq 2 * 0.65(0.85) * 4000 * 36 * 16$$
$$5168.2\ kip \leq 2546\ kip$$
$$\phi P_n = 2546\ kip > 1382\ kip \therefore O.K.$$

Minimum dowel reinforcement

$$min = 0.005 * A_{col} = 0.005 * 36 * 16 = 2.88\ in^2$$

Since $\phi P_n\ for\ column < P_u, dowels\ required\ for\ strength$

$$A_{S_{req}\ dowel} = \frac{P_u - \phi P_n}{\phi f_y}; \phi = 0.65$$

$$A_{S_{req}\ dowel} = \frac{1382kip - 1273kip}{0.65 * 60ksi} = 2.79\ in^2 < 0.005 * A_{col}$$

$$A_{s_{req}\,dowel} = 0.79\ in^2 < 2.88in^2$$

$$\therefore use\ minimum\ reinforcement\ \ 2.88\ in^2$$

Use same size reinforcement in columns due to inadequacy of bearing

4 #10 $\qquad A_s = 5.08\ in^2$

<u>Dowel length</u>

Lap splice in compression

$$l_s = 0.0005 * f_y * d_{dowel}$$

$$l_s = 0.0005 * 60000 * 1.27 = 38.1\ \ in > 12\ in\ spacing$$

$$Use\ l_s = 42" = 3.5'$$

Development length in compression

$$l_{d_{c1}} = 0.02 * \frac{d_b * f_y}{\lambda\sqrt{f_c'}} = 0.02 * \frac{0.75in * 60000psi}{\lambda\sqrt{4000}} = 14.23in$$

$$l_{d_{c2}} = 0.003d_b f_y = 0.0003 * 0.75in * 60000psi = 13.5in$$

$$Use\ l_{d_c} = 15in$$

$$l_s = 42\ in > l_{d_c} = 15in$$

Use splice length from columns b/c column has larger reinforcing bars

Summary

10’ by 10’ footing
36” deep (bottom of footing @ 3.5’ depth below grade)
3” cover on all sides
28 #6 bars in both directions for flexural reinforcement (top layer 30” below top of footing)
4 #10 dowels

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Retaining Walls

In order to meet the desired grade on site, there will need to be retaining walls to account for changes in elevation. Due to an area variance in the city of Yonkers, retaining walls are only permitted to be exposed for 6-feet above grade. The idea was to space the walls so the optimum height would be gained, but each wall would be far enough away to not have to account for its surcharge on the walls below. This is why the minimum distance between each wall is nine feet. There is a 3:1 assumed between each wall when designing which accounts for the worst-case soil slope between walls. The typical wall was designed per linear foot for the case that there was only soil acting on one side of the wall while the other side was fully exposed meaning that only active lateral earth pressure was accounted for. This was done in order to be more conservative. The frost line used in Yonkers is 3.5 feet below grade. This is what determined the full height of the wall and footing assembly of 9.5 feet. The wall design was tested for lateral earth pressure by using the Coulomb Method as well as seismic activity using the Mononobe-Okabe Method. Once the size of the wall was determined to be acceptable according to IBC 1806.1 for active lateral earth pressure, the structural reinforcement was designed using the methods in the ACI code. Since the walls are cantilever walls, the main reinforcement is along the face of the wall holding back the soil. This is due to the shape of the deflection which causes that side to act in tension and the exposed face to act in compression. The reinforcement along the exposed face is to prevent cracking. There is a total of 12 retaining walls combining a total of 3,091 linear feet that will need to be used on site.

There is one wall that is an exception to the typical design which is the wall that the west parking lot rests on. This wall was designed accounting for the surcharge of the live load for the parking lot of 50 psf (according to ASCE 7-05) that will be acting on the wall. At the top of this wall there will be a guardrail to ensure the safety of the vehicles parking as they face a 6-foot drop in elevation. The other eleven walls will use the typical wall design.

Along with the retaining walls used for grading, there are two stories of the building that are below grade. The basement wall along the north side of the building is retaining soil for the entire length and height of the walls whereas the east and west walls have soil sloped along them. Because of this, the north wall is considered the worst case and this was the wall that was designed. Each wall was considered to be simply supported.

In order to maintain proper drainage and keep water away from the walls, there will be 12-inches of gravel backfill between the wall and the soil. There will also be weep holes along the lower part of the wall. This will allow for water to drain down the gravel and through the weep holes to a proper drainage area.

Design Process for Typical Wall

<u>Given:</u>

$c = 0 \rightarrow sand$

$\phi = 30^o\ (conservative\ for\ well\ drained\ sand)$

$\alpha = 18.34^o\ (based\ on\ 1{:}3\ slope)$

$\beta = 90^o$

$\delta = \frac{1}{2}\phi = \frac{1}{2} * 30^o = 15^o$

$\gamma = 115\ pcf\ (conservative\ for\ well\ drained\ sand)$

$k_h = 0.1\ (assumed\ based\ on\ earthquake\ activity\ in\ Yonkers)$

$k_v = 0.5k_h = 0.05$

$$\theta = \tan^{-1}\frac{k_h}{1-k_v} = 6.01^o$$

<u>Lateral Earth Pressure (Coulomb Method):</u>

Seismic activity accounted for using Mononobe-Okabe Method.

$$K_{ae} = \frac{\sin^2(\phi+\beta+\theta)}{\cos\theta\sin^2\beta\sin(\beta+\theta-\delta)\left(1+\sqrt{\frac{\sin(\phi+\delta)\sin(\phi+\theta-\alpha)}{\sin(\beta-\delta-\theta)\sin(\alpha+\beta)}}\right)^2} = 0.5134$$

$$K_a = \frac{\sin^2(\phi+\beta)}{\sin^2\beta\sin(\beta-\phi)\left(1+\sqrt{\frac{\sin(\phi+\delta)\sin(\phi-\alpha)}{\sin(\beta-\delta)\sin(\alpha+\beta)}}\right)^2} = 0.6215$$

$$P_{ae} = \frac{1}{2}\gamma H^2(1-k_v)K_{ae} = 2531.01\frac{lb}{ft}$$

$$P_a = \frac{1}{2}\gamma K_a H^2 = 3225.3\frac{lb}{ft}$$

$$\Delta P_{ae} = P_{ae} - P_a = 694.29\frac{lb}{ft}$$

$$z = \frac{0.6H * \Delta P_{ae} + \frac{H}{3}P_a}{P_{ae}} = 5.60'$$

$$P_{ae} = 2531.01\frac{lb}{ft}$$

$$P_h = 2531.01\cos 15 = 2444.77\frac{lb}{ft}$$

$$P_v = 2531.01\sin 15 = 655.07\frac{lb}{ft}$$

Weights:

$$W_1 = 150\,pcf * 8' * 1' = 1200\frac{lb}{ft} \rightarrow stem\ weight$$

$$W_2 = 150\,pcf * 6.5' * 1.5' = 1462.5\frac{lb}{ft} \rightarrow footing\ weight$$

$$W_3 = 115\,pcf * 8' * 4' = 3680\frac{lb}{ft} \rightarrow soil\ weight$$

$$W_4 = 115\,pcf * \frac{1}{2} * \frac{4'}{3} * 4' = 306.67\frac{lb}{ft} \rightarrow sloped\ soil\ weight$$

$$\sum V = 655.07 + 1200 + 1462.5 + 3680 + 306.67 = 7304.24\frac{lb}{ft}$$

Check Overturn:

All moments were taken about the toe of the footing. IBC1806.1 states that the factor of safety for both overturning and sliding must be greater than 1.5.

$$\sum M_R = 1200(2) + 1462.5\left(\frac{1}{2} * 6.5\right) + 3680(4.5) + 306.67\left(\frac{2}{3} * 4 + 2.5\right) + 655.07(2.5) = 26935.24\ lb - ft$$

$$M_o = 2444.77 * 5.60 = 13690.71$$

$$FS = \frac{\sum M_R}{M_o} = \frac{26935.24}{13690.71} = 1.97 > 1.5 \rightarrow OK$$

Check Sliding:

$$FS = \frac{\sum V \tan\phi}{P_h} = \frac{7304.24 \tan 30}{2444.77} = 1.72 > 1.5 \rightarrow OK$$

The size of the footing is acceptable.

Reinforcement (based on ACI Code):

Stem

Vertical Reinforcement:

$$b = 12"$$

$$d = 12 - 3 - 0.5 = 8.5"$$

$$M_u = 2444.77(5.60 - 1.5) = 10.02\ k - ft$$

$$R_u = \frac{M_u}{\phi b d^2} = \frac{10.02 * 1000 * 12}{0.9 * 12 * 8.5^2} = 154.1\ psi$$

$$\rho = \frac{0.29}{100} = 0.0029$$

For earth-bearing side of the wall:

$$A_s = \rho b d = 0.0029 * 12 * 8.5 = 0.2958\ in^2 \rightarrow \boldsymbol{use\ \#5\ @\ 12"}\ (A_s = 0.31\ in^2)$$

$$A_{s_{min}} = 0.0015 * 12 * 12 = 0.216\ in^2 < 0.31\ in^2 \rightarrow OK$$

Use #4 @ 12" along the exterior (non-earthbearing side).

Temperature and Shrinkage Reinforcement:

$$A_s = 0.002 * 12 * 12 = 0.288\ in^2$$

This is the reinforcement required for both faces of the wall. Divide the reinforcement along both sides of the wall:

$$A_{s_{face}} = 0.5A_s = 0.5(0.288) = 0.144$$

$$\rightarrow \boldsymbol{use\ \#4\ @\ 12"\ along\ both\ faces\ of\ the\ wall}$$

Heel of Footing

Flexural (Lateral) Reinforcement:

$$d = 18 - 3 - 0.5 = 14.5"$$

$$V_u = 1.2\left(8 * 4 * 115 + \frac{1}{2} * \frac{4}{3} * 115 + 150 * 1.5 * 4\right) = 5864\ lb = 5.86\ kip$$

$$M_u = V_u x = 5.86 * \frac{4}{2} = 11.73\ k - ft$$

$$\phi V_c = \phi 2\lambda\sqrt{f'c}bd = \frac{0.75 * 2 * 1\sqrt{4000} * 12 * 14.5}{1000} = 16.51\ kip > V_u \rightarrow d\ OK$$

$$R_u = \frac{M_u}{\phi b d^2} = \frac{11.73 * 12 * 1000}{0.9 * 12 * 14.5^2} = 62.0\ psi$$

$$\rho = \frac{0.12}{100} = 0.0012$$

$$A_s = \rho b d = 0.0012 * 12 * 14.5 = 0.21\ in^2 \rightarrow \boldsymbol{use\ \#5\ @12"}$$

Toe of Footing

Flexural (Lateral) Reinforcement:

$$R_v = \sum V = 7248.69 \frac{lb}{ft}$$

$$a = \frac{M_R - M_o}{R_v} = \frac{25116.2 - 13690.7}{7248.69} = 1.58' < \frac{l}{3} = 2.17'$$

Therefore, the resultant force acts outside of the middle third of the footing which causes the uplift below the footing of the wall to start at the toe and is only distributed along a distance of “3a” along the footing with the maximum pressure under the toe being:

$$q = \frac{2R_v}{3a} = \frac{2(7248.69)}{3(1.58)} = 3.06\ ksf$$

Using similar triangles, the pressures at the distance “a” and at the exterior face of the stem can be found:

$$q_a = 2.04\ kip$$

$$q_{ext} = 2.09\ ksf$$

$$M_u = 1.6\left(\frac{2.09}{2} * 1.5^2 + (3.06 - 2.09) * 1.5 * \frac{1}{2}\left(\frac{2}{3} * 1.5\right)\right) - 1.2\left(1.5 * \frac{150}{1000} * \frac{1.5^2}{2}\right)$$

$$= 4.6\ kip < 10.02\ kip \rightarrow \boldsymbol{use\ \#5\ @\ 12"}$$

For construction, the same reinforcement may be used for the earth-side of the wall and the toe part of the footing.

Shrinkage (Longitudinal) Reinforcement for both heel and toe of footing:

$$A_{s_{min}} = 0.0018bh = 0.0018 * 12 * 12 = 0.26\ in^2$$

Assume that this steel is distributed equally across both faces (top & bottom):

$$A_{s_{face}} = 0.5A_{s_{min}} = 0.5 * 0.26 = 0.13\ in^2 \rightarrow \boldsymbol{choose\ \#4\ @\ 12"}$$

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Design Process for Parking Lot Retaining Wall

Given:

$c = 0 \rightarrow sand$

$\phi = 30^o\ (conservative\ for\ well\ drained\ sand)$

$\alpha = 0$

$\beta = 90^o$

$\delta = \frac{1}{2}\phi = \frac{1}{2} * 30^o = 15^o$

$\gamma = 115\ pcf\ (conservative\ for\ well\ drained\ sand)$

$k_h = 0.1\ (assumed\ based\ on\ earthquake\ activity\ in\ Yonkers)$

$k_v = 0.5k_h = 0.05$

$h = 9.5'$

$$\theta = \tan^{-1}\frac{k_h}{1-k_v} = 6.01^o$$

$$h' = \frac{50\ psf}{115\ pcf} = 0.435'$$

$$H = h + h' = 9.5 + 0.435 = 9.935'$$

Lateral Earth Pressure (Coulomb Method):

Seismic activity accounted for using Mononobe-Okabe Method.

$$K_{ae} = \frac{\sin^2(\phi+\beta+\theta)}{\cos\theta\sin^2\beta\sin(\beta+\theta-\delta)\left(1+\sqrt{\frac{\sin(\phi+\delta)\sin(\phi+\theta-\alpha)}{\sin(\beta-\delta-\theta)\sin(\alpha+\beta)}}\right)^2} = 0.354$$

$$K_a = \frac{\sin^2(\phi+\beta)}{\sin^2\beta\sin(\beta-\phi)\left(1+\sqrt{\frac{\sin(\phi+\delta)\sin(\phi-\alpha)}{\sin(\beta-\delta)\sin(\alpha+\beta)}}\right)^2} = 0.388$$

$$P_{ae} = P_h = \frac{1}{2}\gamma H^2(1-k_v)K_{ae} = 1908.7\frac{lb}{ft}$$

$$P_v = 0$$

$$P_a = \frac{1}{2}\gamma K_a H^2 = 2202.1\frac{lb}{ft}$$

$$\Delta P_{ae} = P_{ae} - P_a = 293.4\frac{lb}{ft}$$

$$z = \frac{0.6H * \Delta P_{ae} + \frac{H}{3}P_a}{P_{ae}} = 3.91'$$

Weights:

$$W_1 = 150\,pcf * 8' * 1' = 1200\frac{lb}{ft} \rightarrow stem\ weight$$
$$W_2 = 150\,pcf * 6.5' * 1.5' = 1462.5\frac{lb}{ft} \rightarrow footing\ weight$$
$$W_3 = 115\,pcf * 8' * 4' = 3680\frac{lb}{ft} \rightarrow soil\ weight$$
$$\sum V = 1200 + 1462.5 + 3680 = 6342.5\frac{lb}{ft}$$

Check Overturn:
All moments were taken about the toe of the footing. IBC1806.1 states that the factor of safety for both overturning and sliding must be greater than 1.5.

$$\sum M_R = 1200(2) + 1462.5(3.25) + 3680(4.5) = 23713.13\ lb - ft$$
$$M_o = 1908.7 * 3.91 = 7463.02\ lb - ft$$
$$FS = \frac{\sum M_R}{M_o} = \frac{23713.13}{7463.02} = 3.18 > 1.5 \rightarrow OK$$

Check Sliding:

$$FS = \frac{\sum V \tan\phi}{P_h} = \frac{6342.5 \tan 30}{1908.7} = 1.92 > 1.5 \rightarrow OK$$

The size of the footing is acceptable.

Reinforcement (based on ACI Code):

Stem

Flexural Reinforcement:

$$b = 12"$$
$$d = 12 - 3 - 0.5 = 8.5"$$
$$M_u = 1.6 * 7463.02 = 11940.8\ lb - ft$$
$$R_u = \frac{M_u}{\phi b d^2} = \frac{11940.8 * 12}{0.9 * 12 * 8.5^2} = 183.6\ psi$$
$$\rho = \frac{0.4}{100} = 0.004$$

For earth-bearing side of the wall:

$$A_s = \rho b d = 0.004 * 12 * 8.5 = 0.41\ in^2 \rightarrow \boldsymbol{use\ \#6\ @\ 12"}\ (A_s = 0.44\ in^2)$$
$$A_{s_{min}} = 0.0015 * 12 * 12 = 0.216\ in^2 < 0.44\ in^2 \rightarrow OK$$

Use #4 @ 12" along the exterior (non-earthbearing side).

Temperature and Shrinkage Reinforcement:

$$A_s = 0.0018 * 12 * 12 = 0.26\ in^2$$

This is the reinforcement required for both faces of the wall. Divide the reinforcement along both sides of the wall:

$$A_{s_{face}} = 0.5A_s = 0.5(0.26) = 0.13 \rightarrow \boldsymbol{use\ \#4\ @\ 12"}$$

Heel of Footing

Flexural (Lateral) Reinforcement:

$$d = 18 - 3 - 0.5 = 14.5"$$

$$V_u = 1.2(8 * 4 * 115 + 150 * 1.5 * 4) = 1.6(50 * 4 * 0.435) = 5.64\,kip$$

$$M_u = V_u x = 5.86 * \frac{4}{2} = 11.27\,k - ft$$

$$\phi V_c = \phi 2\lambda\sqrt{f'c}bd = \frac{0.75 * 2 * 1\sqrt{4000} * 12 * 14.5}{1000} = 16.51\,kip > 5.64\,kip \rightarrow d\ OK$$

$$R_u = \frac{M_u}{\phi bd^2} = \frac{11.27 * 12 * 1000}{0.9 * 12 * 14.5^2} = 59.56\,psi < 183.6\,psi$$

$$\rightarrow \boldsymbol{continue\ \#6@12"\ from\ stem\ of\ wall\ through\ heel\ of\ footing}$$

Toe of Footing

Flexural Reinforcement:

$$R = \sum V = 6342.5\frac{lb}{ft}$$

$$a = \frac{M_R - M_o}{R} = \frac{20419.25 - 7463.02}{6342.5} = 2.04' < \frac{l}{3} = 2.17'$$

Therefore, the resultant force acts outside of the middle third of the footing which causes the uplift below the footing of the wall to start at the toe and is only distributed along a distance of “3a” along the footing with the maximum pressure under the toe being:

$$q = \frac{2R_v}{3a} = \frac{2(6342.5)}{3(2.04)} = 6.21\,ksf$$

Using similar triangles, the pressures at the distance “a” and at the exterior face of the stem can be found:

$$q_a = 3.11\,kip$$

$$q_{ext} = 4.69\,ksf$$

$$M_u = 1.6\left(\frac{4.69}{2} * 1.5^2 + (6.21 - 4.69) * 1.5 * \frac{1}{2}\left(\frac{2}{3} * 1.5\right)\right) - 1.2\left(1.5 * \frac{150}{1000} * \frac{1.5^2}{2}\right)$$

$$= 9.32\,kip < 10.02\,kip \rightarrow \boldsymbol{use\ \#6\ @\ 12"}$$

For construction, the same reinforcement may be used for the earth-side of the wall and the toe part of the footing.

Shrinkage Reinforcement for both heel and toe of footing:

$$A_{s_{min}} = 0.0018bh = 0.0018 * 12 * 12 = 0.26\,in^2$$

Assume that this steel is distributed equally across both faces (top & bottom):

$$A_{s_{face}} = 0.5A_{s_{min}} = 0.5 * 0.26 = 0.13\,in^2 \rightarrow \boldsymbol{choose\ \#4\ @\ 12"}$$

Design Process for Basement Retaining Wall

Given:

$c = 0 \rightarrow sand$

$\phi = 30^o$ $(conservative\ for\ well\ drained\ sand)$

$\gamma = 115\ pcf$ $(conservative\ for\ well\ drained\ sand)$

Top wall: $H_1 = 10' - 8"$

Bottom wall: $H_2 = 14'$

There is a surcharge of 100 psf along soil above basement walls.

Lateral Earth Pressure

Rankine Method:

$$K_a = \frac{1 - \sin\phi}{1 + \sin\phi} = \frac{1 - \sin 30}{1 + \sin 30} = \frac{1}{3}$$

$$h_s = \frac{100psf}{115pcf} = 0.87'$$

$$\sigma_s = K_a \gamma h_s = \frac{1}{3}(115)(0.87) = 33.35\ psf$$

Soil pressure for the top wall:

$$\sigma_1 = K_a \gamma H_1 = \frac{1}{3}(115)(10.67) = 408.9\ psf$$

$$\sigma_2 = K_a \gamma H_T = \frac{1}{3}(115)(24.67) = 945.6\ psf$$

Because the bottom wall is exposed to the largest soil pressure, it is the wall that is designed for.

$$H = 14'$$

$$P_a = \sigma_1 H_2 + \frac{1}{2}(\sigma_2 - \sigma_1)H_2 = 408.9(14) + \frac{1}{2}(945.6 - 408.9)(14)$$

$$= 9481.5 \frac{lb}{LF} of\ wall$$

$$P_s = \sigma_s H_2 = 33.35 * 14 = 466.9 \frac{lb}{LF} of\ wall$$

Equivalent Fluid Pressure:

There is no water table present, but the wall should be designed taking the presence of water into account. The maximum water pressure would be assuming the entire wall is exposed to water:

$$\sigma_w{}' = \gamma_w H_2 = 62.4 * 14 = 873.6\ psf$$

$$P_w{}' = \frac{\sigma_w H}{2} = 873.6 * \frac{14}{2} = 6115.2 \frac{lb}{LF} of\ wall$$

This most likely will not occur, therefore only account for 50% of the max:

$$\sigma_w = \frac{\sigma_w'}{2} = \frac{873.6}{2} = 436.8\ psf$$

$$P_w = \frac{P_w'}{2} = \frac{6115.2}{2} = 3057.6\ psf$$

Assume a 1-LF section of the wall ($b = 12"$). The wall can be treated as a simply supported beam that is 14' long. There are four different loadings along the wall: two are uniformly distributed – the soil pressure from the top wall and the surcharge along the top of the soil. Two are triangularly distributed – the soil pressure along the wall of concern as well as the water pressure. These loads are shown below:

$$\omega_1 = \sigma_w * 1' = 873.6 \frac{lb}{ft} \; along \; wall$$

$$\omega_2 = (\sigma_2 - \sigma_1) * 1' = 536.7 \frac{lb}{ft} along \; wall$$

$$\omega_3 = \sigma_1 * 1' = 408.9 \frac{lb}{ft} along \; wall$$

$$\omega_4 = \sigma_s * 1' = 33.35 \frac{lb}{ft} along \; wall$$

$$M_u = \frac{1.6(\omega_1 + \omega_2)0.128H^2}{2} + \frac{1.6(\omega_3 + \omega_4)H^2}{8} = 45.64 \; k - ft$$

Flexural Reinforcement

Assume $\rho = 0.01 \rightarrow R_u = 332 \; psi; b = 12"$

$$d_{min} = \sqrt{\frac{M_u}{R_u b}} = \sqrt{\frac{45.64 * 12}{0.332 * 12}} = 11.72"$$

Concrete cover for interior face is 0.75". Assume half a bar diameter is 0.5"

$$h = d + 0.75 + 0.5 = 11.72 + 0.75 + 0.5 = 12.97 \rightarrow \text{choose h=14}$$

$$d = 14 - 0.75 - 0.5 = 12.75"$$

$$R_u = \frac{M_u}{bd^2} = \frac{45.64 * 1000 * 12}{12 * 12.75^2} = 280.75 \; psi \rightarrow \rho = 0.0055 > \rho_{min} = 0.0033 \rightarrow OK$$

$$A_s = \rho bd = 0.005 * 12 * 12.75 = 0.8415 \; in^2 \rightarrow \boldsymbol{choose \; \#7 \; @ \; 8"}$$

This reinforcement will be used in the flexural direction for both the interior and earth face.

Temperature and Shrinkage Reinforcement

$$A_{s_{min}} = 0.0018bh = 0.0018 * 12 * 14 = 0.302 \; in^2$$

Assume that this steel is distributed equally across both faces:

$$A_{s_{face}} = 0.5A_{s_{min}} = 0.5 * 0.302 = 0.151 \; in^2 \rightarrow \boldsymbol{choose \; \#4 \; @ \; 12"}$$

Estimate

The estimate was done using two separate methods. The first method uses an estimated cost of $350 per square foot of building. The second method will be a general quantity takeoff with lump sum values for each aspect of the construction, which are quantified and then summed to find the total cost.

Base Estimate

$$Total\_Cost = \$350 \times 164473\,ft^2 = \$57,565,550$$

Quantity Take Off

Building Total (SF) :	164,473	Building Footprint:	12600 SF
		Site Acreage:	10.067
# of Stories:	16		

CSI Division	Description	Quantity	Unit	Unit Cost	Total Cost
Division 1	**General Requirements**				
	Insurance	1	Assumption	$2,500,000.00	$ 2,500,000.00
	Bonds	1	Assumption	$ 150,000.00	$ 150,000.00
	Permits	1	Assumption	$ 100,000.00	$ 100,000.00
	Staffing	4	persons	$ 150,000.00	$ 600,000.00
	Temporary Site Utilities	1	Assumption	$1,500,000.00	$ 1,500,000.00
General Requirements					**$ 4,850,000.00**

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Division 2	**Site Construcion**				
02200	**Site Preparation**				
02230	Site Clearing Clearing and Grubbing	435,600	SF	$ 1.00	$ 435,600.00
02300	**Earthwork**				
02315	Excavation and Fill Cut : Sand/Gravel	2,048 20,480	CY CY-Total	$ 20.00	$ 40,960.00
02315	Excavation and Fill Excavation Sand	31,489	CY	$ 20.00	$ 629,780.00
02315	Backfill Backfill, sand & gravel	10,641	CY	$ 10.00	$ 106,410.00
02500	**Utility Service**				
02530	Sanitary Sewerage Pipes	448	LF	$ 15.00	$ 6,720.00
02530	Sanitary Sewerage Manholes	3	EA	$ 2,000.00	$ 6,000.00
02510	Water Distribution Pipes	800	LF	$ 15.00	$ 12,000.00
02510	Water Distribution Cleanouts	4	EA	$ 2,000.00	$ 8,000.00
02600	**Drainage and Containment**				
02630	Storm Drainage: Storm Drainage Pipe and Fittings Piping, 8" diameter	4,000	LF	$ 15.00	$ 38,720.00
02660	Under Ground Retention	1	EA	$ 200,000.00	$ 200,000.00
02660	Retention Pond	2205	CY	$ 15.00	$ 33,075.00
02700	**Pavements**				
02710	Asphalt Paving	16300	SF	$ 5.00	$ 81,500.00
02775	Sidewalk	1440	CY	$ 200.00	$ 288,000.00
02800	**Site Improvements and Amenities**				
02830	Retaining Wall 9.5' High, 13" Thick Concrete Reinforced	1,985	CY	$ 500.00	$ 992,500.00
02900	**Planting**				
	Landscaping for Open Park Recreation Area	7500	SF	$ 30.00	$ 225,000.00
Site Construction					**$ 3,104,265.00**

Division 3	Concrete						
03300	Cast-In-Place Concrete						
03310	Structural Concrete 36" X 16" Reinforced Rectangular Concrete Column	90	CY	$	600.00	$	54,000.00
03310	Structural Concrete 12" X 36" Reinforced Rectangular Concrete Column	544	CY	$	600.00	$	326,400.00
03310	Structural Concrete 18" X 42" Reinforced Rectangular Concrete Column	6	CY	$	600.00	$	3,600.00
03310	Structural Concrete 24" Reinforced Round Concrete Column	62	CY	$	600.00	$	37,200.00
03310	Structural Concrete Reinforced Concrete Beam	87	CY	$	1,000.00	$	87,000.00
03310	Structural Concrete 10" Slab	4065	CY	$	600.00	$	2,439,000.00
03310	Structural Concrete Concrete Foundation Wall	3235	CY	$	600.00	$	1,941,000.00
03310	30" X 30" Reinforced Concrete Pier	13	CY	$	600.00	$	7,800.00
03310	Structural Concrete Rectangular Footing	500	CY	$	600.00	$	300,000.00
03310	Shear Wall	972	CY	$	500.00	$	486,000.00
Concrete Total						**$**	**5,682,000.00**

Division 4	**Masonry**							
04210	Clay Masonry Unit Brick	164,473	SF	$	20.00	$	3,289,460.00
Masonry Total						**$**	**3,289,460.00**
Division 5	**Metals**						
05500	**Metal Fabrication**						
05550	Stair Treads and Nosing	32	Floor	$	8,000.00	$	256,000.00
Metals Total						**$**	**256,000.00**
Division 7	**Thermal and Moisture Protection**						
07300	**Shingles, Roof Tiles, and Roof Coverings**						
07310	Shingle	8000	SF	$	30.00	$	240,000.00
07050	Thermal Protection and Water Proofing	81472	SF	$	30.00	$	2,444,160.00
Roofing Total						**$**	**2,684,160.00**
Division 8	**Doors and Windows**						
08200	**Wood and Plastic Door**						
08210	Wood Door	840	Each	$	2,000.00	$	1,680,000.00
08500	Windows	415	Each	$	1,500.00	$	622,500.00
08900	Glazed Curtain Wall	1040	SF	$	30.00	$	31,200.00
Doors and Windows Total						**$**	**2,333,700.00**
Division 9	**Finishes**						
09200	**Plaster and Gypsum Board**						
09250	Gypsum Board 5/8"	798000	SF	$	12.00	**$**	**9,576,000.00**
Finishes Total						**$**	**9,576,000.00**

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Divison 10	**Specialities**				
10800	**Toilet, Bath, and Laundry Accessories**				
10810	Toilet Accessories	200	EA	$ 2,500.00	$ 500,000.00
10820	Bath Accessories	100	EA	$ 1,500.00	$ 150,000.00
10830	Laundry Units (Washer/Dryer)	30	EA	$ 2,000.00	$ 60,000.00
Specialties Total					**$ 710,000.00**
Division 13	**Special Construction**				
13900	**Fire Suppression**				
13930	Wet-Pipe Fire Supression Sprinkler System	1	EA	$3,000,000.00	$ 3,000,000.00
13850	Fire Alarms	273	EA	$ 15.00	$ 4,095.00
	Hydrants	3	EA	$ 2,000.00	$ 6,000.00
Fire Suppression Total					**$ 3,010,095.00**
Division 14	**Conveying Systems**				
14200	**Elevator**				
14210	Elevators	3	EA	$ 70,000.00	$ 210,000.00
Conveying Systems Total					**$ 210,000.00**
Division 15	**Mechanical**				
15900	HVAC	164473	SF	$ 40.00	$ 6,578,920.00
15400	Plumbing	164473	SF	$ 25.00	$ 4,111,825.00
Mechanical Total					**$ 10,690,745.00**
Divison 16	**Electrical**				
16050	Electrical	164473	SF	$ 30.00	$ 4,934,190.00
16520	Parking Lighting	14	EA	$ 7,000.00	$ 98,000.00
Electric Total					**$ 5,032,190.00**
Subtotal					$ 51,428,615.00
OH&P					1.15
Grand Total					**$ 59,142,907.25**

Construction Schedule

The development is a located on a 10-acre site in Yonkers, Westchester County. The Building is a cast-in-place flat-plate concrete residential facility, with a total square footage of 164,473 distributed over 16 floors. The contract award date will be July 7, 2014. Submittals will be filed immediately and it is expected to take 30 days for submittals to be completed, however, site mobilization can begin 15 days after the start of submittals. The approval of submittals can also begin 15 days after the notice to proceed (NTP). After site mobilization, the erosion control plan can be implemented followed by rough site grading and surveying. Once the locations of building corners, retaining walls, and road are surveyed, the excavation for the building pad, the pond, the retention system, and the retaining walls. At this point in the schedule, the site work and building construction can run simultaneously.

Several activities can occur simultaneously in site development. The retaining walls can be constructed as catch basins, manholes, and pipes for the underground utilities are being installed. The retaining walls will need to be poured in 2 segments. First the footing is formed and poured then the stem is formed and poured. Underground utility installation is assumed to be complete prior to the completion of the retaining walls. Once all site utilities are installed, and retaining walls are poured, backfill can begin. Landscaping and paving can occur simultaneously as they do not interfere directly, and they will be the last site activities. Site work will be completed on June 26, 2015.

Building excavation will begin directly after surveying. The forms for the wall footing and column footings can be formed together. The forms for the slab on grade will begin 5 days after the forms for the footings are started. The pour for the foundation system can begin after the forms for slab on grade, wall footings, and column footings are all completed. The first floor will require more time to form and pour due to the transfer beams located on this floor. The form and pour for floor two through six will take 40 days. Concrete pours for typical floors were estimated to take three days. The form for the next floor will require five days and there is a necessary two day wait between the end of the pour for one floor and the beginning of the formwork for the next. Therefore, a typical floor will take 10 days. After the sixth floor has been poured, the lower floors will have reached their 28 day strength thus interior framing can begin as the upper floors are being poured. Floor 13 is the last typical floor before the setback and penthouse units, at this point material can be ordered for mechanical, electrical, and plumbing activities (MEP). The material is expected to be on site around the time that the roof of the building is completed and MEP installations can begin. Once the pours are completed, the concrete pump trucks that were necessary to pump concrete to the higher floors can be removed from the site and cranes can be brought in to start installing the exterior cladding. Door and window installations can begin 15 days after the cladding begins. MEP installations are expected to take the longest. Interior finishing can begin once MEP work is completed. The punch list is the last activity before project completion which will be August 23, 2016.

Drawings

Note: The following drawings are not for construction purposes, they are not to scale.

Appendix

A. Sediment and Erosion Control

A.01 – Chain Link Fence

A.02 – Excavated Drop Inlet Protection

A.03 – Grassed Waterway

A.04 – Sediment Basin Details

A.05 – Silt Fence

A.06 – Stabilized Construction Entrance

B. Water

B.01 – 1:1 Erosion Control Blanket

B.02 – 2:1 Erosion Control Blanket

B.03 – Sanitary Manhole Cover

B.04 – Standard Manhole Cover

B.05 – HDPE Storm Water Drain Pipes

B.06 – Cast Iron Sanitary Pipes

B.07 – Hydraulic Elements Graph for Circular Sewers

B.08 – Manning's Equation Nomograph

B.09 – Water Supply Copper Pipe

B.10 – Water Meter

B.11 – Check Valve

B.12 – 10 Year IDF Curve

B.13 – 100 Year IDF Curve

B.14 – Fire Hydrant

B.15 – Curb Inlet

B.16 – Copper Pipe Fittings

B.17 – Traffic Rated Drop Inlet

B.18 – Storm Water Retention Chamber

B.19 – Forebay

C. Traffic

C.01 – Parking Curb

C.02 – Parking Guard Rail

C.03 – Cattle Sign

C.04 – Handicap Parking Sign

C.05 – Pavement Arrow Marking

C.06 – Stop Sign

C.07 – Speed Limit Sign

D. Architectural and Structural

D.01 – 2.5 Inch Metal Stud

D.02 – Gypsum Board

D.03 – Thermal and Acoustic Insulation

D.04 – Brick Veneer

D.05 – Curtain Wall

D.06 – Windows

D.07 – 4000 PSI Concrete Mix

D.08 – 60000 PSI Rebar

D.09 – Lenton Steel Fortress Shear Reinforcement System

NOTE: THIS SPECIFICATION IS TO BE USED AS A GENERAL GUIDELINE. THE SPECIFICATION SHOULD NOT BE CONSIDERED COMPLETE AS THE ARCHITECT IS REQUIRED TO MAKE ADDITIONS AND DELETIONS SO THAT THE SPECIFICATION IS COORDINATED WITH THE DRAWINGS. ITEMS THAT ARE NOT PART OF THE SCOPE OF THIS PROJECT WILL BE DELETED BY THE ARCHITECT; HOWEVER, THE GENERAL FORMAT AND GENERAL STATEMENTS OF THE VARIOUS PARTS OF THIS SECTION WILL REMAIN UNCHANGED.

IF STRUCTURAL SEALANT IS USED TO ATTACH GLAZING TO CURTAIN-WALL ASSEMBLY, USE SECTION 08930 "STRUCTURAL SEALANT GLAZED CURTAIN-WALLS" AS A GUIDE.**SECTION 08 44 13**

GLAZED ALUMINUM CURTAIN-WALLS

PART I - GENERAL

1.1 SUMMARY

Section includes conventional factory glazed and factory assembled, unitized, aluminum curtain-wall assemblies complete with necessary components and accessories.

1.2 PERFORMANCE REQUIREMENTS

General: Comply with performance requirements specified, as determined by testing manufacturer's standard conventional factory glazed and factory assembled curtain -wall assemblies meet or exceed performance requirements designated in AAMA MCWM-1 “Metal Curtain-wall Manual”, as well as requirements otherwise noted herein, without failure due to defective manufacture, fabrication, installation, or other defects in construction.

Conventionally factory glazed and factory assembled curtain -wall units shall withstand movements of supporting structure indicated on Drawings including, but not limited to, story drift, and deflection from uniformly distributed and concentrated live loads.

Failure also includes the following:

Thermal stresses transferring to building structure.

Glass breakage.

Noise or vibration created by wind and thermal and structural movements.

Loosening or weakening of fasteners, attachments, and other components.

Failure of operating units.

NOTE: DETERMINATION OF DESIGN LOADS IS THE RESPONSIBILITY OF THE PROJECT’S ARCHITECT/ENGINEER OF RECORD. INDICATE ON DRAWINGS THE REQUIRED STRUCTURAL DESIGN LOADS AS DETERMINED BY PROJECT’S STRUCTURAL ENGINEER.Structural Loads: All structural components, including meeting rails, mullions and anchors shall comply with the following requirements:

Wind Loads: As indicated on Drawings..

Structural-Test Performance: Provide conventionally glazed aluminum curtain-wall assemblies tested according to ASTM E-330 as follows:

When tested at positive and negative wind-load design pressure of **30** lbf/sq. ft, curtain-wall assemblies will evidence deflection exceeding specified limits.

When tested at 150 percent of positive and negative wind-load design pressure of **30** lbf/sq. ft, curtain-wall assemblies, including anchorage, will not evidence material failures, structural distress, and permanent deformation of main framing members exceeding 0.2 percent of span.

Deflection of Framing Members: At design wind pressure of **30** lbf/sq. ft, as follows:

Deflection Normal to Wall Plane: Limited to 1/175 of clear span for spans up to 13 feet 6 inches and to 1/240 of clear span plus ¼ inch for spans greater than 13 feet 6 inches or an amount that restricts edge deflection of individual glazing lites to ¾ inch, whichever is less.

Deflection Parallel to Glazing Plane: Limited to amount not exceeding that which reduces glazing bite to less than 75 percent of design dimension and that which reduces edge clearance between framing members and glazing or other fixed components to less than 1/8 inch.

Story Drift: Accommodate design displacement of adjacent stories indicated.

Design Displacement: As indicated on Drawings.

Water Penetration under Static Pressure: No evidence of water penetration through fixed glazing and framing areas when tested according to ASTM E 331 at a minimum static-air-pressure differential of 20 percent of positive wind-load design pressure, but not less than 15 lbf/sq. ft.

Maximum Water Leakage: No uncontrolled water penetrating assemblies or water appearing on assemblies' normally exposed interior surfaces from sources other than condensation in compliance with AAMA 501.1. .

Thermal Movements: Allow for thermal movements resulting from the following maximum change (range) in ambient and surface temperatures:

Temperature Change (Range): 120 deg F, ambient; 180 deg F, material surfaces.

Interior Ambient-Air Temperature: 75 deg F.

Energy Performance: curtain-wall vision areas and adjacent framing shall be tested and have energy performance ratings in accordance with NFRC 102 or AAMA 1503 requirements.

Thermal Transmittance (U-factor): overall curtain-wall vision area and adjacent framing shall be less or equal to a U-factor of **0.45** Btu/sq. ft. x h x deg F as determined according to NFRC 100 or AAMA 1503. U Factor is influenced by glass make up.

Solar Heat Gain Coefficient: Provide window units that have a solar heat gain coefficient (SHGC) for overall glazed area of **0.45** or better for north orientation and **0.55** or better for all other orientations as determined in accordance with NFRC 200 or AAMA 1503 . SHGC is influenced by glass make up.

Visible Transmittance: Provide window units that have a visible transmittance value of **0.58** or better for overall glazed area as determined according to ASTM E-1084. Visible transmittance is influenced by glass make up.

Air Infiltration: Maximum air leakage through fixed glazing and framing areas of 0.01 cfm/sq. ft. of fixed wall area as determined according to ASTM E-283 at a minimum static-air-pressure differential of 6.24 lbf/sq. ft.

Condensation Resistance Factor: CRF minimum value for glazing is to be 70.

1.3 SUBMITTALS

Product Data: Submit the manufacturer's specifications, technical product data, performance values, standard details of the products specified, manufacturer's recommendation for installation and the manufacturer's certification of the Installation Subcontractor

Shop Drawings shall be the responsibility of the curtain-wall manufacturer and prepared by the manufacturer. Drawings prepared by others are not acceptable. Shop Drawings shall bear the stamp of a State of _______licensed Professional Engineer, and shall indicate configurations of curtain-wall, windows, system dimensions, profiles, finishes, glass types, accessories, hardware, anchors, fasteners, air and vapor barrier if/as specified and indicated, masonry opening requirements and acceptable tolerances, and details of related adjacent construction.

Submit shop drawings prepared by the curtain-wall manufacturer for each type of product. Include the following- shop drawings not provide by the manufactured will not be allowed:

Indicate plan layout and location of each curtain-wall section and component dimensions. Continue the curtain-wall designation established in the Drawings.

Building plans and elevations to be drawn at a minimum 1/8 inch scale; curtain-wall unit elevations at minimum 1/4 inch scale. Include floor plans, elevations, sections, details, and attachment to other work, operational clearances, required reinforcement and installation details.

Details shall show the following:

Joinery, including concealed welds.

Anchorage fastener types and locations, clips/straps/plates and reinforcing steel as required by structural calculations.

Expansion and contraction provisions.

Glass and glazing.

Flashing and drainage for draining moisture occurring within the assembly to the exterior.

Metal finish.

Sealants, including those selected by the curtain-wall manufacturer.

Submit six final, complete, shop drawing sets to the Architect prior to the start of fabrication. These final shop drawing sets shall incorporate all review comments and notations from previous shop drawing submittals.

Samples: Submit samples, if and as directed by the Architect..

Finish Selection: Submit samples for units with factory-applied color finishes for each type of exposed finish required, in manufacturer's standard sizes.

Product Test Reports / Calculations: Based on evaluation of comprehensive tests performed by a qualified testing agency for curtain-walls, indicating compliance with performance requirements.

Qualification Data: For qualified Installer and testing agency.

NOTE: RETAIN FIRST PARAGRAPH BELOW IF RETAINING PROCEDURES FOR WELDER CERTIFICATION IN "QUALITY ASSURANCE" ARTICLE.Welding certificates.

Sustainable Design Submittals:

Product Data for Credit IEQ 4.1: For sealants used inside of the weatherproofing system, documentation including printed statement of VOC content.

Laboratory Test Reports for Credit IEQ 4: For sealants used inside the weatherproofing system, documentation indicating that products comply with the testing and product requirements of the California Department of Health Services' "Standard Practice for the Testing of Volatile Organic Emissions from Various Sources Using Small-Scale Environmental Chambers."

Field quality-control reports.

Warranties: Samples of special warranties.

1.4 QUALITY ASSURANCE

Manufacturer Qualifications: The manufacturer shall be a firm with a minimum of ten years experience and capable of fabricating glazed aluminum curtain-walls that meet or exceed performance requirements indicated. The curtain-wall manufacturer shall certify in writing that the completed curtain-wall has been fabricated and shop assembled in manufacturer's plant in accordance with the Specifications and the final approved shop drawings. Site built/fabricated curtain wall will not be allowed.

Installer Qualifications: The curtain-wall Installation Subcontractor shall be a firm with a minimum of five years' experience, and certified by the Manufacturer as being trained and approved for the proper installation of the specified aluminum curtain-wall assembly required for this Project.

Testing Agency Qualifications: Qualified according to ASTM E-699 for testing indicated. Field testing to paid for by the owner.

Product Options: Information on Drawings and in Specifications establishes requirements for aesthetic effects and performance characteristics of assemblies. Aesthetic effects are indicated by dimensions, arrangements, alignment, and profiles of components and assemblies as they relate to sightlines, to one another, and to adjoining construction.

Do not revise intended aesthetic effects, as judged solely by Architect, except with Architect's approval. If revisions are proposed, submit comprehensive explanatory data to Architect for review.

NOTE: RETAIN PARAGRAPH BELOW IF WELDING IS REQUIRED. Welding Qualifications: Qualify procedures and personnel according to the following:

AWS D1.1/D1.1M, "Structural Welding Code – Steel."

AWS D1.2/D1.2M, "Structural Welding Code – Aluminum."

NOTE: CHOOSE THE APPLICABLE SUBPARAGRAPH FROM CHOICES BELOW.Mockups: Build mockups to verify selections made under sample submittals and to demonstrate aesthetic effects and set quality standards for fabrication and installation.

Build mockup of typical wall area as shown on Drawings.

Mock up to be for visual inspection – fit and finish

Mock up to be tested at an independent test lab. Pick testing criteria - Testing to include static air, and water, structural, interstitial floor movement.

Testing cost to be paid by the GC.

Pre-installation Conference: Conduct conference at Project site to comply with requirements in Division 01 Section “Project Management and Coordination.” Review methods and procedures related to curtain-wall work including, but not limited to, the following:

Review and finalize construction schedule and verify availability of materials, Installer’s personnel, equipment, and facilities needed to make progress and avoid delays.

Review and discuss the finishing of aluminum curtain-walls that is required to be coordinated with the finishing of other aluminum work for color and finish matching.

Review, discuss, and coordinate the interrelationship of aluminum curtain-wall with other exterior wall components. Include provisions for anchorage, flashing, sealing perimeters, and protecting finishes.

Review and discuss the sequence of work required to construct a watertight and weather tight exterior building envelope.

Inspect and discuss the condition of substrate and other preparatory work performed by other trades.

PROJECT CONDITIONS

Installer to work with GC to ensure correct field dimensions.

DELIVERY, STORAGE AND HANDLING

All curtain-wall materials, components and accessories will be packed, loaded, shipped, unloaded, stored and protected as required by the manufacturer.

1.7 WARRANTY

Special Assembly Warranty: Standard form in which manufacturer agrees to repair or replace components of conventionally glazed aluminum curtain-wall system that do not comply with requirements or that fail in materials or workmanship within specified warranty period.

Failures include, but are not limited to, the following:

Failure to meet performance requirements.

Structural failures including, but not limited to, excessive deflection.

Noise or vibration created by wind and thermal and structural movements.

Water penetration, air infiltration, or condensation through fixed glazing and framing areas.

Failure of operating components.

Deterioration of metals, other materials, and factory applied metal finishes beyond normal weathering.

Warranty Period:

Curtain-wall system and its components: Five (5) years from date of [Preliminary Acceptance] [Substantial Completion] of the Project, as applicable.

Metal Finish: Ten (10) years from date of [Preliminary Acceptance] [Substantial Completion] of curtain-wall system installation. Warranty to align with selected finish

Hardware: Ten (1) years from date of [Preliminary Acceptance] [Substantial Completion] of curtain-wall system installation.

Glazing: Ten (10) years from date of [Preliminary Acceptance] [Substantial Completion] of curtain-wall system installation. Warranty to align with selected glass.

PART 2 - PRODUCTS

2.1 MANUFACTURERS

Manufacturers: Subject to compliance with requirements, provide products by one of the following:

FM Graham Enterprises, LLC

Wausau Windows and Walls

Old Castle Building Envelope

2.2 MATERIALS

Aluminum: Alloy and temper recommended by manufacturer for type of use and finish indicated.

Sheet and Plate: ASTM B-209.

Extruded Bars, Rods, Profiles, and Tubes: ASTM B-221.

Extruded Structural Pipe and Tubes: ASTM B-429.

Structural Profiles: ASTM B-308/B-308M.

NOTE: RETAIN SUBPARAGRAPH BELOW FOR WELDING.Welding Rods and Bare Electrodes: AWS A5.10/A5.10M.

NOTE: RETAIN PARAGRAPH BELOW FOR INTERNAL STEEL REINFORCEMENT OF ALUMINUM FRAMING MEMBERS; REVISE TO SUIT PROJECT.Steel Reinforcement: Manufacturer's standard zinc-rich, corrosion-resistant primer.

Structural Shapes, Plates, and Bars: ASTM A-36.

Cold-Rolled Sheet and Strip: ASTM A-1008.

Hot-Rolled Sheet and Strip: ASTM A-1011.

B. Steel reinforcement to be supplied and fabricated as required by manufacturer.

2.3 FRAMING

Framing Members: Manufacturer's standard extruded, or formed, aluminum framing members of thickness required and reinforced as required to support imposed loads.

Vertical mullions as indicated on Drawings.

Horizontal intermediate framing member as indicated on Drawings

Thermal Break Construction:

Provide manufacturer's thermal-break construction that has been in use for not less than three years and has been tested to demonstrate resistance to thermal conductance and condensation and to show adequate strength and security of glass retention.

Brackets and Reinforcements: Manufacturer's standard high-strength aluminum with non-staining, nonferrous shims for aligning system components or bituminous painted steel as required by manufacturer.

Fasteners and Accessories: Manufacturer's standard corrosion-resistant, non-staining, non-bleeding fasteners and accessories compatible with adjacent materials. Conceal fasteners whenever possible.

Use self-locking devices where fasteners are subject to loosening or turning out from thermal and structural movements, wind loads, or vibration.

Reinforce members as required to receive fastener threads.

Use exposed fasteners with countersunk Phillips screw heads, fabricated from 300 series stainless steel.

Anchors: Anchors that accommodate fabrication and installation tolerances in material and finish; are compatible with adjoining materials and recommended by manufacturer.

RETAIN SUBPARAGRAPH BELOW IF APPLICABLE - REVISE TO SUIT PROJECT.Concrete and Masonry Inserts: Hot-dip galvanized cast-iron, malleable-iron, or steel inserts complying with ASTM A-123 or ASTM A-153 requirements.

Concealed Flashing: Dead-soft, 0.018-inch- thick stainless steel, ASTM A-240 of type recommended by manufacturer.

Framing Sealants:.

Frame joinery sealants shall be suitable for application specified and as tested and approved by curtain-wall manufacturer.

2.4 GLAZING

Glazing General: Comply with Division 8 "Section Glazing."

Glazing method shall be in general accordance with the GANA Glazing Manual for specified glass type, or as approved by the glass fabricator.

Glazing Materials

Glazing Plane: Framing to be designed for glazing from the exterior.

Glazing System: Retained mechanically with gaskets on four sides.

Setting Blocks/Edge Blocking: Provide in sizes and locations recommended by GANA Glazing Manual. Setting blocks used in conjunction with soft-coat low-e glass shall be silicone.

Glazing Gaskets: Manufacturer's standard pressure-glazing system of black, resilient elastomeric glazing gaskets, setting blocks, and shims or spacers. Glazing gaskets shall be non-shrinking, weather-resistant, and compatible with all materials in contact.

Back-bedding tapes, expanded cellular glazing tapes, toe beads, heel beads and cap beads shall meet the requirements of applicable specifications cited in AAMA 800.

Glazing Sealants: As recommended by manufacturer.

All materials and finishes in contact with structural silicone shall be tested for compatibility and approved by the sealant manufacturer for the intended application.

Gaskets in continuous contact with structural silicone shall be extruded silicone or compatible material.

NOTE: RETAIN THIS ARTICLE WHERE VENTING WINDOWS AND/OR DOORS ARE REQUIRED IN PROJECT.Operable Units:

Operable Vents: Provide "zero sightline" projected curtain-wall insert vents complying with the requirements of Division 8 Section "Aluminum Windows", at the locations indicated on the Drawings." Vents to be manufactured and supplied by the curtain wall manufacturer.

Doors: Provide doors complying with the requirements of Division 8 Section "Aluminum-Framed Entrances and Storefronts" at locations indicated on the Drawings."

2.5 ACCESSORY MATERIALS

Bituminous Paint: Cold-applied asphalt-mastic paint complying with SSPC-Paint 12 requirements except containing no asbestos, formulated for 30-mil thickness per coat.

2.6 FABRICATION

Provide manufacturer's standard interlocking curtain-wall assemblies comprised of unitized sections accurately and neatly cut, machined, sealed and fabricated by the curtain wall manufacturer, with units sized for ease of shipping and erection.

Systems using individual field fabricated or field assembled members by the curtain installer are not acceptable unless approved by the architect.

Fabricate system components and its anchorage that, when assembled, have the following characteristics:

Profiles that are sharp, straight, and free of defects or deformations with ends coped or mitered; forming accurately fitted interlocking unitized construction joints at adjacent grid frame members with sharp, straight, well-defined corners and flush sightlines, assembled to form hairline joints. Rigidly secure non movement joints.

Conceal fasteners at vertical to horizontal main framing connections and at miscellaneous trim except as shown on Drawings or otherwise required. All fasteners, anchors, and connection devices are to be concealed from view to greatest extent possible.

Seal all joints, plugs, and components as required to maintain performance characteristics of system as specified. Provide internal guttering system or other means to drain water passing joints, condensation occurring within framing members, and moisture migrating within glazed aluminum curtain-wall to exterior.

Provide physical and thermal isolation of glazing from framing members. Accommodations for thermal and mechanical movements of glazing and framing to maintain required glazing edge clearances.

Provisions for field replacement of glazing from exterior.

NOTE: RETAIN SUBPARAGRAPH BELOW FOR CURVED COMPONENTS IF ANY.Provision of components curved to indicated radii.

NOTE: RETAIN PARAGRAPH BELOW IF WELDING IS REQUIRED.Weld in concealed locations to greatest extent possible to minimize distortion or discoloration of finish. Remove weld spatter and welding oxides from exposed surfaces by descaling or grinding.

Fabricate factory assembled system components and its anchorage to accommodate:

Thermal and mechanical movements of framing and glazing within specified edge clearances without damage.

Vertical story movement, Elastic and inelastic lateral drift, including fabrication tolerances, without damage. All movement shall be accommodated using manufacturer's recommended method of interlocking framing members.

Glass Drainage: Provide weep holes and/or drainage slots within glazing pockets to drain any condensation or accumulating water within the system to exterior.

Snap-On Mullion Covers and Exterior Glazing Beads:

Snap-on covers and glazing beads applied to exterior frame glazing legs to show a sharp, uninterrupted exterior profile between expansion joints.

Allow for horizontal thermal expansion and vertical inter-story movement in cover and bead joinery.

Weather-Stripping:

Install weather-stripping manufactured from suitable material as tested and approved by the curtain-wall manufacturer.

Install weather-stripping in manufacturer's integral slots in framing members.

2.7 ALUMINUM FINISHES

General

Comply with NAAMM's "Metal Finishes Manual" for recommendations for applying and designating finishes. Apply on clean extrusions free from serious surface blemishes; on exposed surfaces visible when installed product's operating vents are closed.

Appearance of Finished Work: Variations in appearance of abutting or adjacent pieces are acceptable if they are within one-half of the range of approved Samples.

NOTE: OF THE THREE FINISH CHOICES PROVIDED BELOW CHOOSE ONE FOR THE SPECIFIC PROJECT:

NOTE: USE THE FOLLOWING AAMA 2604 COMPLIANT 50% FLUOROPOLYMER PAINT FINISH FOR WINDOW REPLACEMENT PROJECTS ONLY. SELECT STANDARD COLOR FROM CHOICES GIVEN BELOW.

High-Performance Organic Finish: AA-C12C42R1x (Chemical Finish: cleaned with inhibited chemicals; Chemical Finish: acid-chromate-fluoride-phosphate conversion coating; Organic coating: as specified below). Prepare, pre-treat, and apply coating to exposed metal surfaces to comply with coating and resin manufacturers' written instructions.

Fluoropolymer Two-Coat System: Manufacturer's standard two-coat, thermo cured system consisting of specially formulated inhibitive primer and fluoropolymer color topcoat containing not less than 50 percent Polyvinylidene fluoride resin by weight; complying with AAMA 2604.

Dry film thickness: minimum 1.2 mils on exposed surfaces, except inside corners and channels.

Color: Project color to be selected from manufacturer standard range of colors

NOTE: USE THE FOLLOWING AAMA 2605 COMPLIANT 70% FLUOROPOLYMER PAINT FINISH FOR NEW CONSTRUCTION PROJECTS ONLY.

High-Performance Organic Finish: AA-C12C42R1x (Chemical Finish: cleaned with inhibited chemicals; Chemical Finish: acid-chromate-fluoride-phosphate conversion coating; Organic Coating: as specified below). Prepare, pre-treat, and apply coating to exposed metal surfaces to comply with coating and resin manufacturers' written instructions.

Fluoropolymer Two-Coat System: Manufacturer's standard two-coat, thermo cured system consisting of specially formulated inhibitive primer and fluoropolymer color topcoat containing not less than 70 percent Polyvinylidene fluoride resin by weight; complying with AAMA 2605.

Dry film thickness: minimum 1.2 mils on exposed surfaces, except inside corners and channels.

Color: chosen from manufacturer's standards.

NOTE: use the following AAMA 611compiant color anodized finis for new construction projects only. Choose standard color from choices given below.

Class I, Color Anodic Finish: AA-M12C22A44 (Mechanical Finish: non-specular as fabricated; Chemical Finish: etched, medium matte; Anodic Coating: Architectural Class I, integrally colored, or electrolytically deposited color coating 0.018 mm or thicker) complying with AAMA 611.

Color: **(Light bronze) (Medium bronze) (Dark bronze) (Black)**

Thickness: AA-M10C22A44 Class I - .7 mils.

Miscellaneous Metal Finishes:

Concealed Steel Items: Galvanized in accordance with ANSI/ASTM A-386 or primed with iron oxide paint.

Apply one coat of bituminous paint to concealed steel surfaces in contact with dissimilar materials.

PART 3 - EXECUTION

EXAMINATION

Comply with all applicable laws, rules and regulations

Prior to commencement of installation, Installer is to perform a thorough field-check, to ensure that construction conditions are correct, that dimensions are correct, and that clearances and construction tolerances between work of this Section and other Trades have been correctly maintained and are in compliance with AAMA " Installation of Aluminum Curtain-walls" manual requirements.

If conditions or dimensions are found to be out of compliance, Installer is to immediately notify the General Contractor in writing describing the unacceptable conditions/dimensions. Proceed with installation only after unsatisfactory conditions have been corrected.

INSTALLATION

Install curtain-wall with skilled workers in accordance with approved shop drawings, specifications, manufacturer's installation instructions.

Install curtain-wall plumb, square and level for proper weathering and operation.

Do not install damaged components.

Fit joints to produce hairline joints free of burrs and distortion.

Install anchors with separators and isolators to prevent metal corrosion and electrolytic deterioration and to prevent impeding movement of moving joints.

NOTE: RETAIN FIRST SUBPARAGRAPH BELOW FOR FIELD WELDING OF COMPONENTS.Weld components in concealed locations to minimize distortion or discoloration of finish. Protect glazing surfaces from welding.

Seal joints watertight unless otherwise indicated.

Metal Protection:

Where aluminum will contact dissimilar metals, protect against galvanic action by painting contact surfaces with primer or by applying sealant or tape or installing nonconductive spacers as recommended by manufacturer for this purpose.

Install components to drain water passing joints, condensation occurring within framing members, and moisture migrating within glazed aluminum curtain-wall to exterior.

Install components plumb and true in alignment with established lines and grades.

NOTE: RETAIN FIRST PARAGRAPH BELOW FOR OPERABLE UNITS.Install operable units level and plumb, securely anchored, and without distortion. Adjust weather-stripping contact and hardware movement to produce proper operation.

Install weather-seal sealant according to sealant manufacturer's written instructions to produce weatherproof joints. Install joint filler behind sealant as recommended by sealant manufacturer.

3.3 ERECTION TOLERANCES

Erection Tolerances: Install glazed aluminum curtain-walls to comply with industry standards.

FIELD QUALITY CONTROL

NOTE: USE THE FOLLOWING IF FIELD TESTING IS GOING TO BE REQUIRED

Testing Agency: Engage a qualified independent testing agency to perform tests and inspections. Specify who is responsible for testing costs align with other specification sections

General: Testing and inspecting of representative areas of conventionally glazed aluminum curtain-walls shall take place as installation proceeds to determine compliance of installed assemblies with specified requirements.

Perform tests in each test area as directed by Architect. Perform at least three tests, prior to 10, 35, and 70 percent completion.

Testing standard: Comply with AAMA 503, including references to ASTM E-783 for Air Infiltration Test and ASTM E 1105 Water Infiltration Test.

Air Infiltration: Areas shall be tested for air leakage of 1.5 times the rate specified for laboratory testing in "Performance Requirements" Article, but not more than 0.09 cfm/sq. ft., whichever is greater, of fixed wall area when tested according to ASTM E-783 at a minimum static-air-pressure differential of 12 lbf/sq. ft.

Test Area: One bay wide, but not less than 30 feet, by one story of structural-sealant-glazed curtain wall.

Water Penetration: Areas shall be tested according to ASTM E-1105 at a minimum uniform and cyclic static-air-pressure differential of 0.80 times the static-air-pressure differential specified for laboratory testing in "Performance Requirements" Article, but not less than 9.0 lbf/sq. ft., and shall not evidence water penetration.

Test Area: One bay wide, but not less than 30 feet, by one story of structural-sealant-glazed curtain wall.

Water Spray Test: Before installation of interior finishes has begun, areas designated by Architect shall be tested according to AAMA 501.2 and shall not evidence water penetration.

Test Area: A minimum area of 75 feet by one story of structural-sealant-glazed curtain wall.

Conventionally glazed aluminum curtain-walls will be considered defective if they do not pass tests and inspections.

Prepare and submit test and inspection reports.

End Specifications.