CIVE 4043 – Senior Design Project – Spring 2016

Management, Design, and Construction for a Mixed Use, Apartment and Retail Development

Group #1, Oklahoma State University

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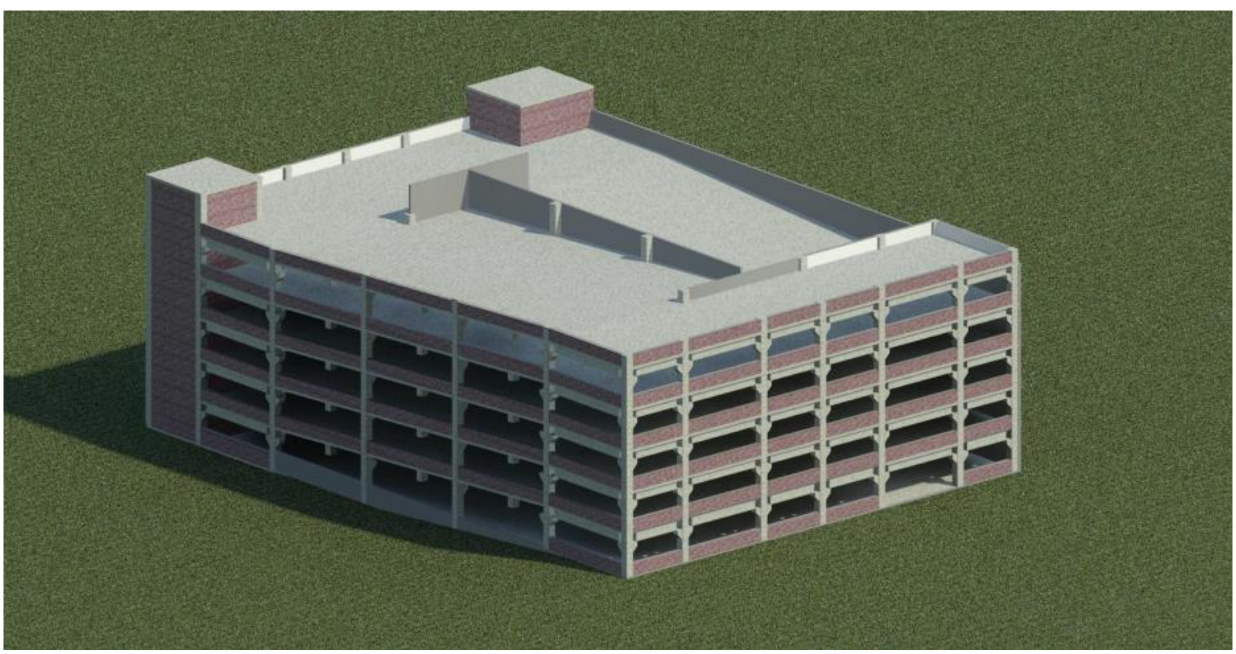

Group 1 Consulting Co.
Stillwater, OK 74074

February 26, 2016

207 Engineering South
Stillwater, OK 74078

Dear Dr. Veenstra,

Our team has completed the site development plans for the proposed apartment complex located at 4th and Ramsay. Attached is the report documenting the engineering calculations and final design recommendations.

As requested by the developer, the report includes the following information:

1. The structural design for the attached parking garage.
2. An analysis of the existing utilities system
3. A hydrological analysis of the site for both pre and post development
4. The proposed grading for the site
5. A transportation analysis of the surrounding area.

Thank you for allowing our group to complete the design and assessment of the project. We look forward to hearing from you and can be contacted by phone at (123-456-7890) or by email (group1@okstate.edu) to answer any further questions.

Sincerely,

Group 1 Consulting Co.
Stillwater, OK 74074

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Introduction

Mega Developers, LLC is undergoing an apartment complex project in the urban core of Stillwater, Oklahoma, close to Oklahoma State University's campus. This project includes the construction of two large apartment buildings that will have retail space, as well as a multi-story parking garage. The two apartment buildings will be five stories tall and have a total of 451 bedrooms. The project site will be fully developed as it will end up having up to 95 percent lot coverage. Our Senior Design group was chosen as the single consultant for the project. Our duties included project management, design, and construction of the project. We were fully responsible for meeting the scope, schedule, budget, and quality objectives of the project including traffic, environment, surveying, design, and construction administration.

Our project design and management tasks included coordinating all aspects of the project to ensure the best value for the developement, good time management, and efficient work. We submitted biweekly progress reports so that the developers were fully aware of everything that was happening. In order to accommodate for the traffic increase in the area, we conducted a traffic analysis to determine if our surrounding intersections needed traffic control modifications. We also had to account for the increase in water demand created by the new apartment complex. Because of this, we were responsible for analyzing the current system and determining whether or not additional piping would be needed to meet the added water demand. We also had to address the increase in water leaving the property. Our responsibilities for the sewer system in connection with the apartment complex included choosing where the new sewer pipes would go, where they would tie into, and then designing the actual piping system. Sewage is not the only water leaving the site. Our stormwater and hydrology systems needed to be designed so that they would have a low impact development. The structural elements of the project were very complex. They included developing a grading plan, designing the foundation, and then creating the entire design and layout of the parking garage. Last but not least, we developed a construction schedule of this apartment complex as well as a cost estimation. After completing all of these tasks, we have designed and constructed an efficient and quality apartment complex.

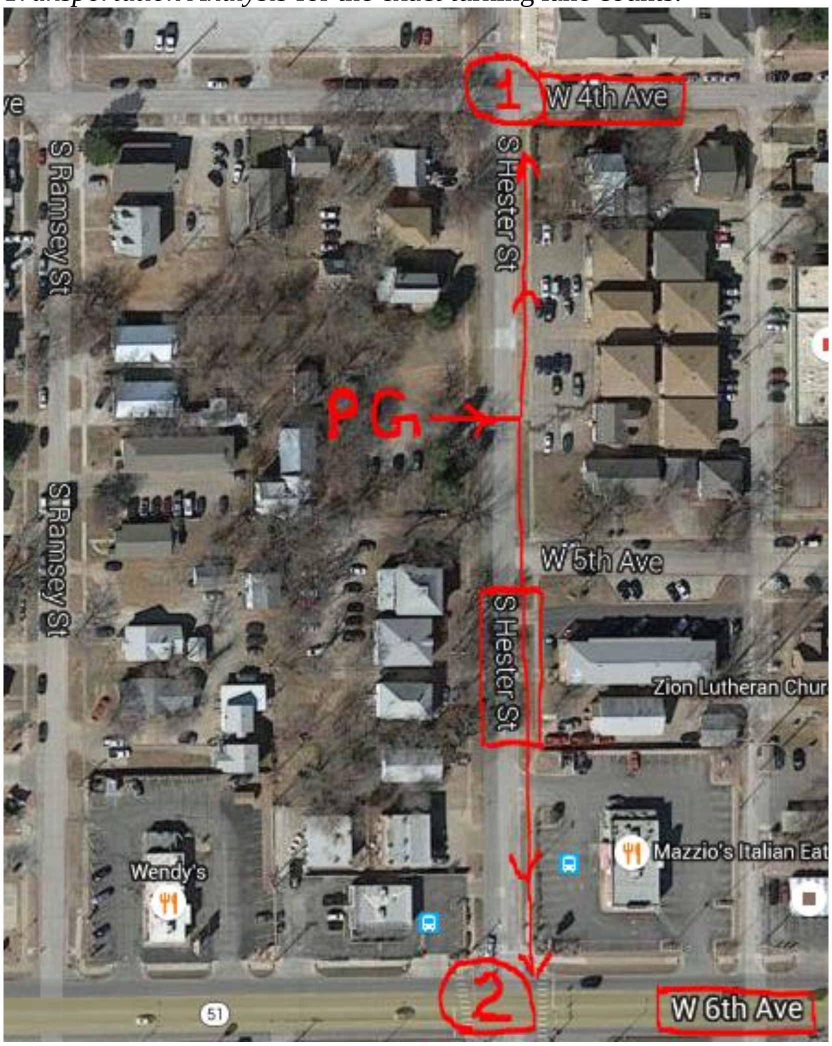
Project Overview

As the lead consultant for the new apartment complex being constructed by Mega Developers, LLC, we have numerous important tasks to complete. These tasks include structural and foundation design for the parking garage, water and sewer utility improvements, hydraulic evaluations, grading plans, and transportation studies.

The structural aspect of the project began with the foundation design. We were given soil data of the surrounding area that allowed for us to create a plan for our cast-in-place piers. From here we completed the rest of the foundation work and moved on to the structural design of the parking garage. This structure was to be constructed out of reinforced concrete. Our responsibilities consisted of full design, including columns, beams, slabs, and walls. We then focused on the water and wastewater improvements. For the water distribution system, we evaluated the current system conditions to see if it could support the additional water demand. We were to decide if any new piping would be necessary. The wastewater system consisted of us designing and placing the new sewage lines and then tieing into an existing system. Our group then conducted a hydraulic evaluation of the site to determine where we would run the stormwater runoff. This was designed with a low impact approach. To finish off the site development, we did the final grading around the apartment complex. Lastly, we completed a traffic analysis to determine if traffic control improvements were needed at the surrounding intersections.

Transportation Analysis

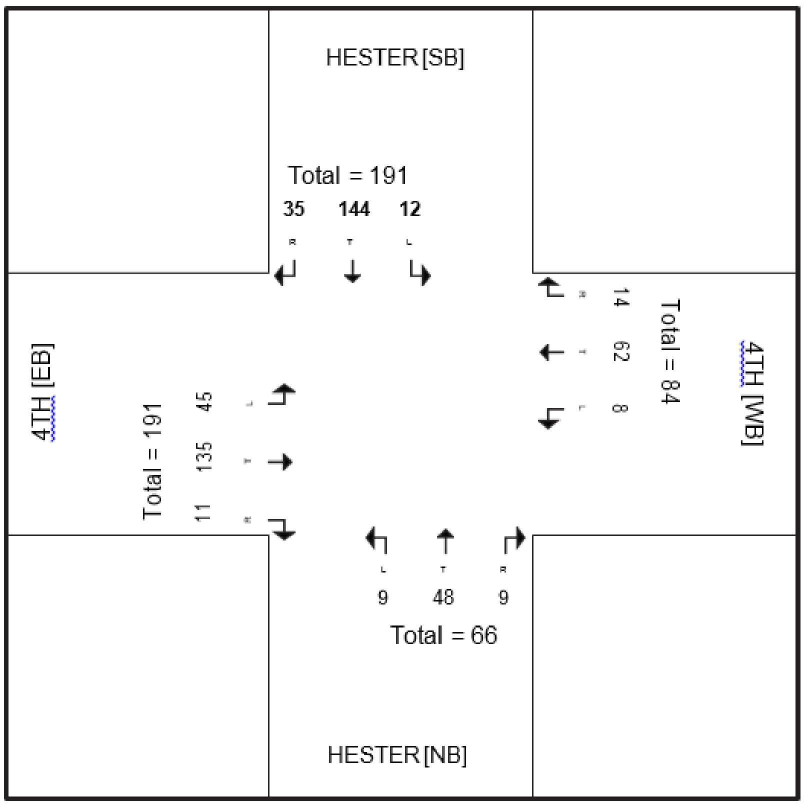
We assumed that the apartment complex has a total of 420 units for a traffic point of view instead of the actual 451 units assuming that a few residents don't drive or that they have shared cars. The total number of trip generated by the construction of the apartment complex was calculated using ITE trip generation rates for an apartment. We assumed that the garage has a single exit on Hester Street. The time of peak traffic flow for the trips generated was calculated as between 4pm and 5pm on a weekday from the traffic flow distribution percentage provided to us in the Traffic Impact Analysis report. See *Appendix A. Transportation Analysis* for the exact turning lane counts.



The parking garage has a single exit on Hester street so we analyzed the traffic intersections at 4th and Hester, and 6th and Hester to determine the current traffic capacity. This was done using the turning movement data for the 2 intersections provided to us in the Traffic Impact Analysis report. We then used a growth rate of 2% per annum to obtain the future traffic demand in the design year of 2020.

4TH and Hester (2016)

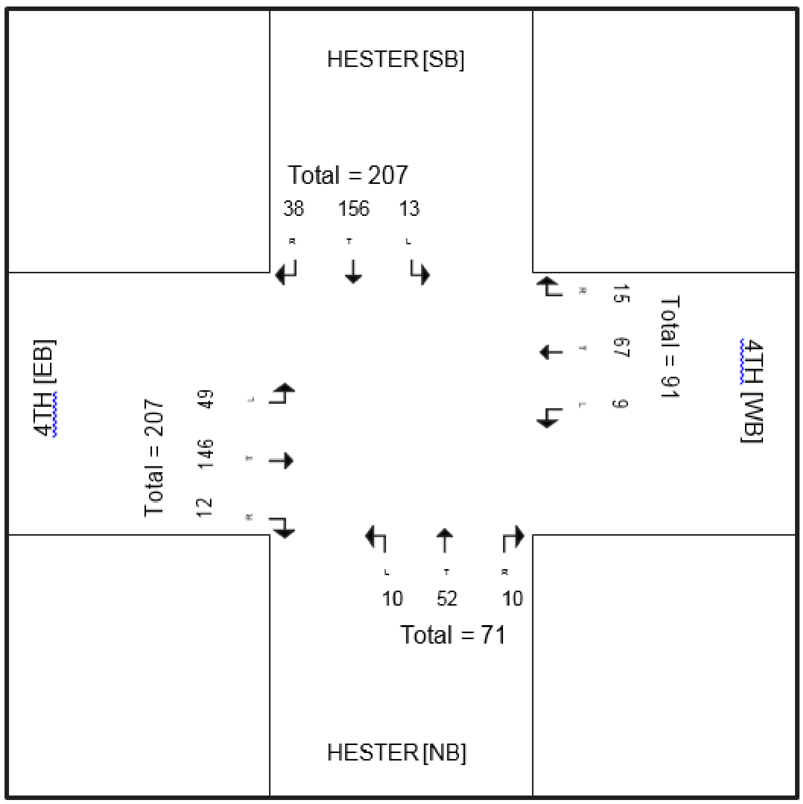
Sum of Critical Lanes:
SB + EB = 382



Turning Movement Data Plot

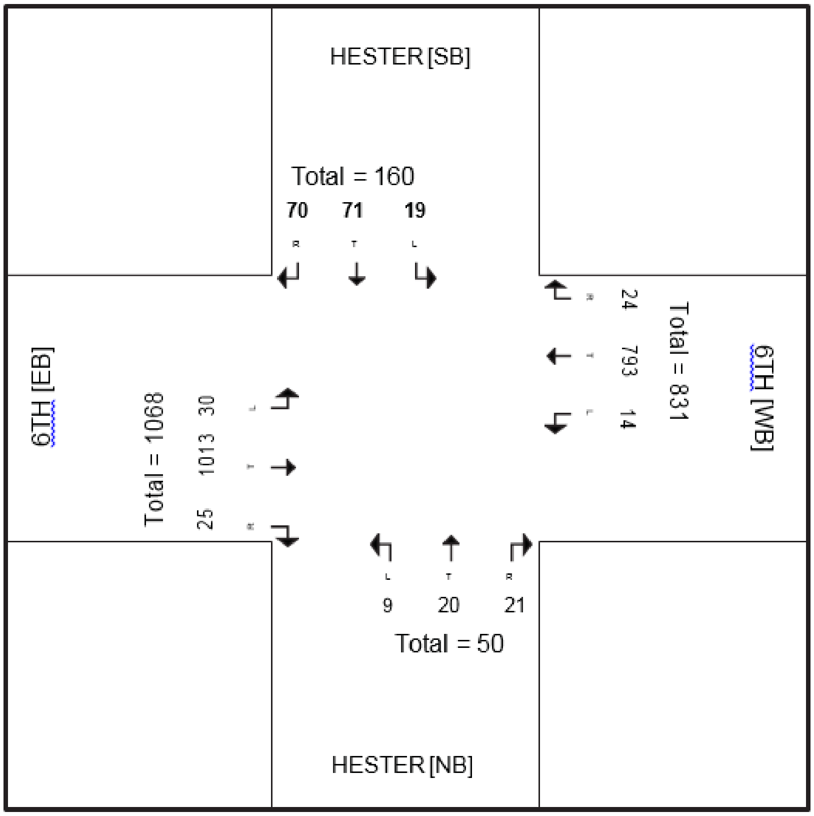
4TH and Hester (2020)

Sum of Critical Lanes:
SB + EB = 413



6TH and Hester (2016)

Sum of Critical Lanes:
SB + EB = 1228



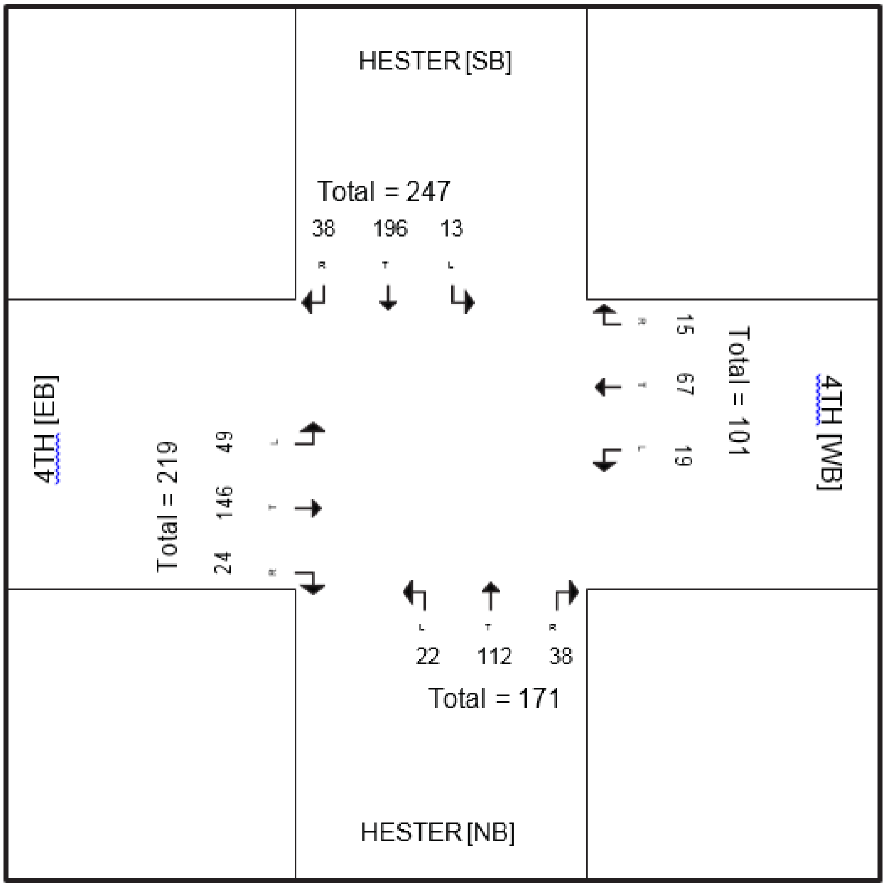
Turning Movement Data Plot

To estimate the traffic demand in 2020 with the additional demand generated by the construction of the apartment complex, we distributed the apartment traffic through the intersections of 4th & Hester and 6th & Hester. We estimated that 65% of the residents will be returning home to the apartments from work or school between 4pm - 5pm on a weekday and thus constitute the incoming traffic. Consequently, we assumed that 35% of the residents will be leaving the parking garage during the peak hour for recreational purposes or for dinner. These percentages were estimated from the ITE trip generation counts for an apartment complex. The apartment traffic was then traffic was then distributed amongst the intersections based on existing 2016 traffic turning movements. See *Appendix A. Transportation Analysis* for further details on traffic counts.These estimated traffic counts for 4th & Hester, 6th & Hester are shown below.

4TH and Hester (2020)

with apartment traffic

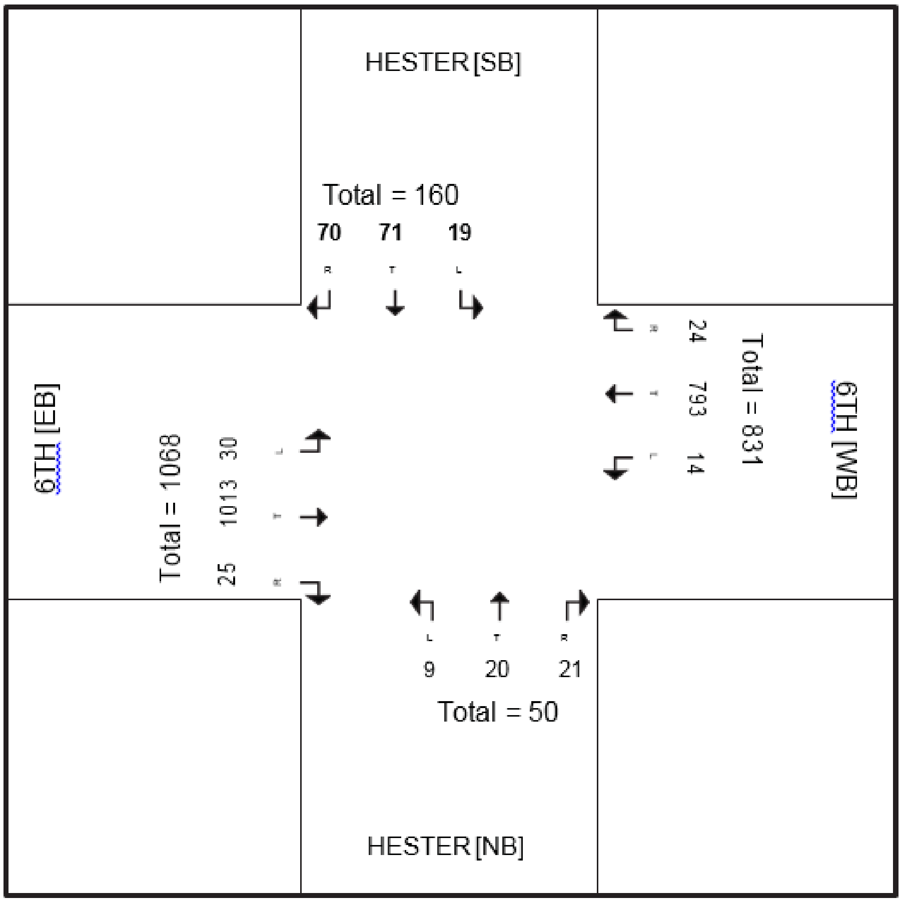
Sum of Critical Lanes:
SB + EB = 465



Turning Movement Data Plot

6TH and Hester (2016)

Sum of Critical Lanes:
SB + EB = 1228



Turning Movement Data Plot

Based on these projected traffic counts in 2020 with the addition of the apartment complex, the traffic growth is not sudden and the current traffic components are sufficient to deal with the increase in demand. Since these counts are based on the worst case scenario with all the residents of the apartment complex entering and leaving the garage during the peak hour between 4pm-5pm, which is unlikely in reality, we can conclude that the increase in traffic will still be at a Level of Service of C for 4th & Hester, and C or borderline D for the intersection at 6th and Hester. This summary of traffic counts is shown in the graph below:

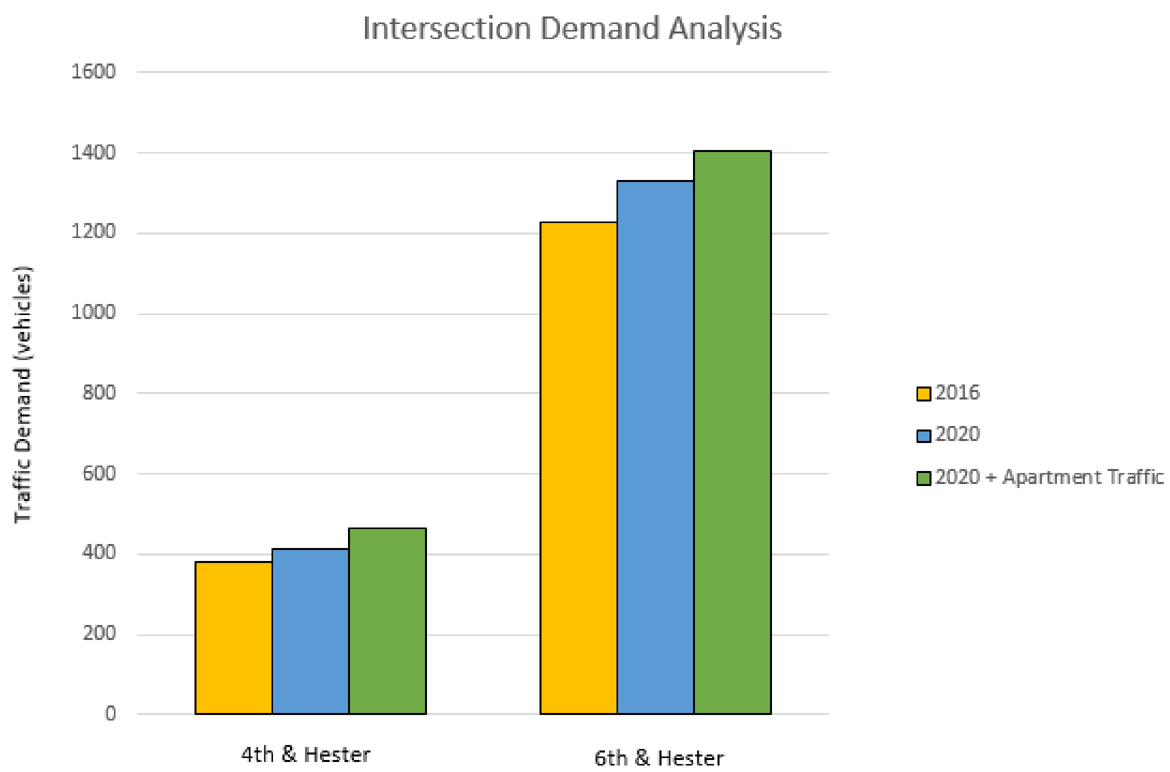


Figure 1. Summary of intersection demand

From the above analysis we can conclude that the current transportation facilities at both intersections are sufficient to meet future demands in 2020. There are 2 recommendations that we would like to make:

1. In the year 2020, pedestrian traffic will substantially increase because of the proximity of the apartment complex to the Oklahoma State campus. Cyclists will also increase. To account for higher pedestrian traffic, an increase in the current width of the sidewalks from 4ft to 6ft would be needed.
2. 4th street needs to be repaved and repainted for a smoother surface finish. All the existing potholes need to be fixed. Repair and maintenance of the currently existing 4th St. would be sufficient to meet future demands. No changes need to be made with traffic signals and signs.

Water Utilities

Our responsibilities for the water distribution system of the apartment complex included analyzing the current system and determining whether or not additional piping would be needed to meet the added water demand.

Our approach with the analyzation of the water system began with taking the given information and creating a schematic of the current system. We were provided with a layout of the water pipes including their size, material, and junction elevations. The lengths of the pipes were acquired from Google Maps. We were given the maximum and average daily usages at each junction. From there we calculated a peak factor and created charts consisting of the hours versus their multipliers. The fire hydrants and their variables were placed into the system as well. The 24-inch water main on University Avenue was designated as the reservoir for our system. The elevation of the reservoir was adjusted so that the fire hydrant pressures in the system would match what was given to us. Once the pressures were correct, we could confirmed that our design was running like the current system. This gave us a hydraulic grade line of 1,075 feet. We then added the additional demand that the apartment complex would bring, including the fire demand. All of these characteristics and values were plugged into the appropriate controls in WaterCAD and the system was run.

We then evaluated the pressures in each junction of the system. According to the Stillwater Engineering Standards, the pressure cannot exceed 100 psi but cannot be below 45 psi. The pressures in our system were far outside of these values. Because our system was not working in the desired range with the added demand, we determined that extra piping would be needed. We decided to add a 10” PVC pipe from the 24” line on University Avenue and run it straight down Ramsey Street to the apartment junction. We chose this size because it was greater than the minimum diameter of 6” and it was able to sufficiently supply the demand of the apartment at a desired pressure. We chose PVC as the material because of its higher performance values, relatively low costs, and its long lifetime. This pipe would be buried 4 feet below the ground level to meet the city standards. A manhole will be placed approximately in the middle of this line, which would be 350 feet South of the intersection at University Avenue and Ramsey Street. This additional piping solved the problem. In our adjusted system, the lowest pressure in the system was found to be 66 psi and the highest was 86 psi. The pressure at the apartment junction was 76 psi. All of these pressures are well within the desired pressure range. The surrounding water distribution system can now sufficiently supply the additional water demand.

The water distribution system schematic and plan and profile views can be seen below. Flex tables and water demand calculations can be found in *Appendix B-1. Water Utilities Tables*

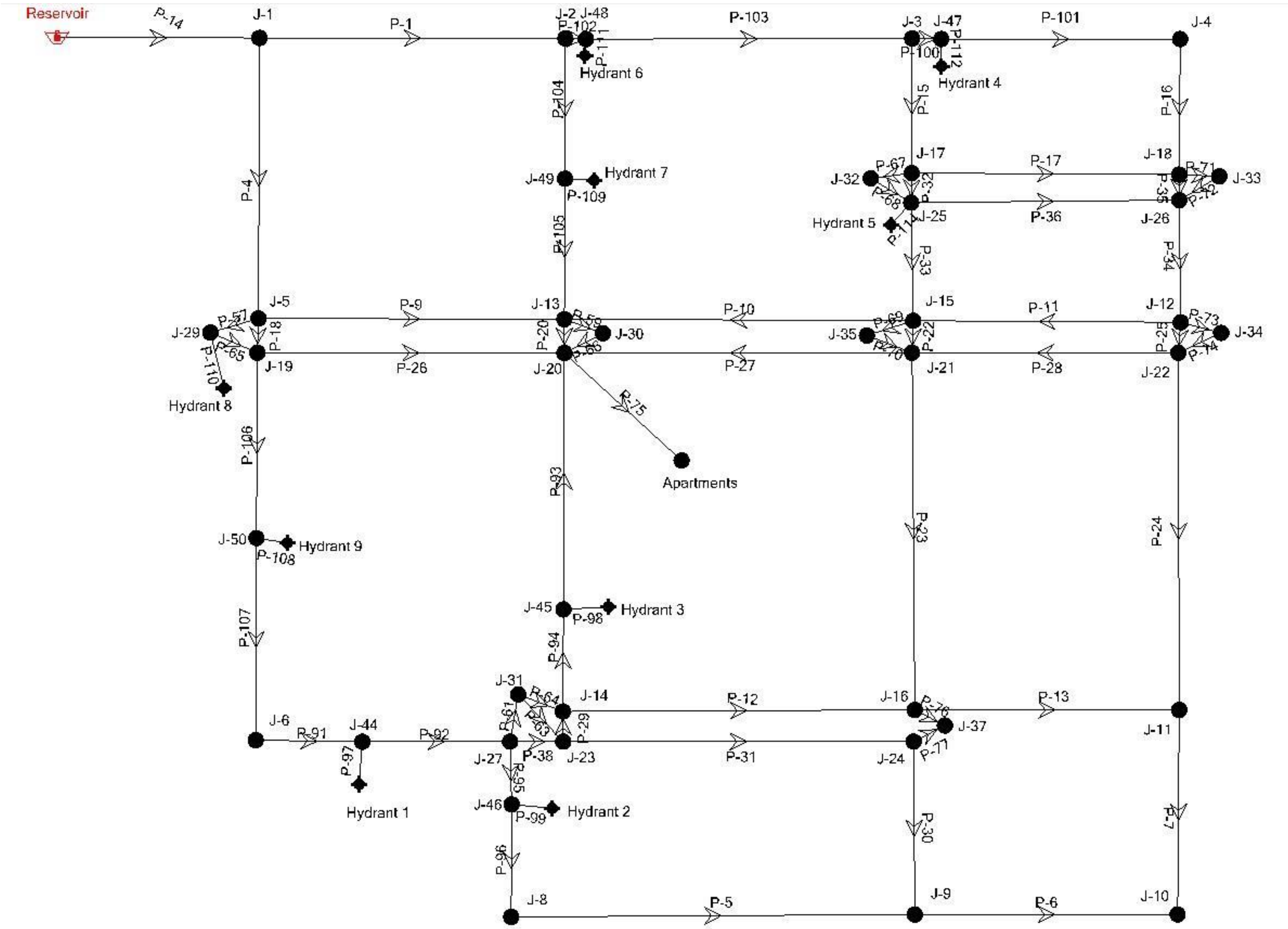


Figure 2-Water Distribution System Schematic

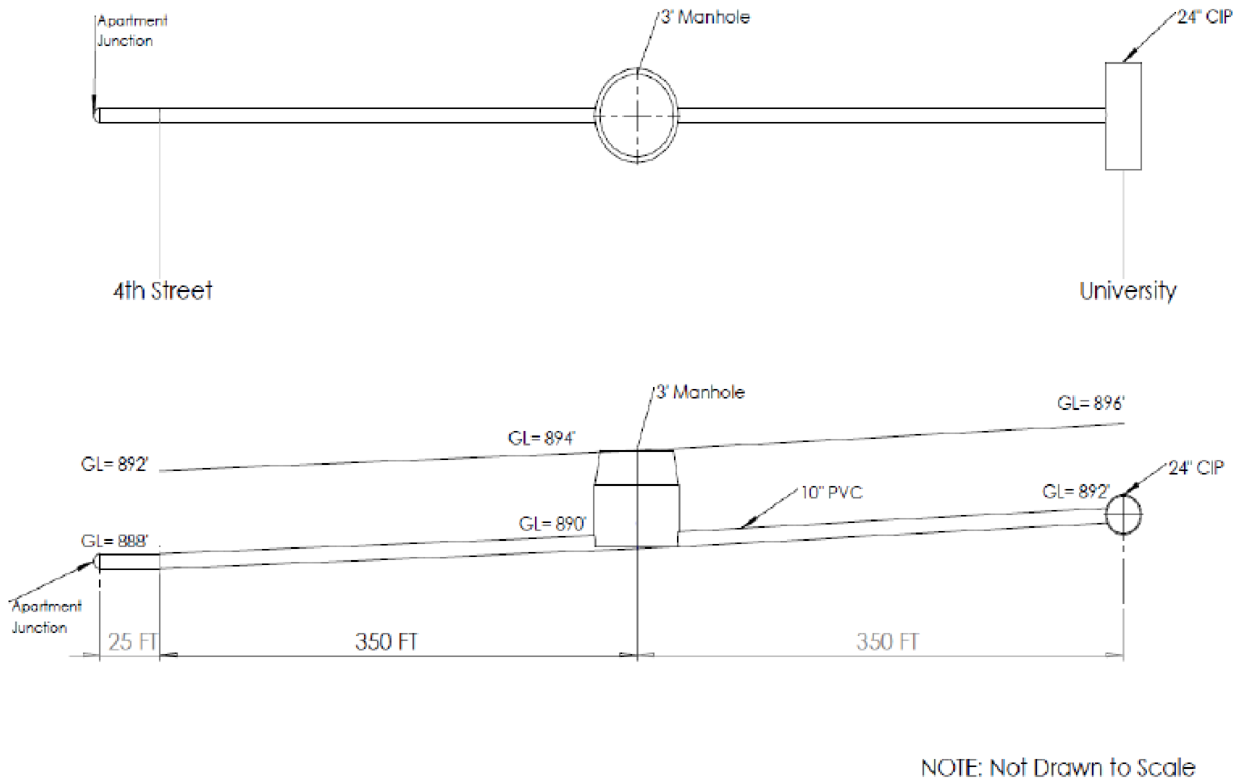


Figure 3-Plan and Profile Views of New Water Line

Wastewater Utilities

Our responsibilities for the sewer system in connection with the apartment complex included choosing where the new sewer pipes would go, where they would tie into, and then designing the actual piping system.

Our goal was to design a gravity-based system so that no pumps would be needed. We decided the place the sewer lines along Fourth Avenue, run them west, and tie into the existing 10-inch line running north and south along Washington Street. Our pipes would start at the intersection of Fourth and Hester, run to Fourth and Ramsey, and then tie into Fourth and Washington. This would allow for the system to flow downhill. These pipes would be buried 3 feet under ground level beneath Fourth Avenue in order to stay within the right of way. We calculated a design flow for the system based on the number of residents in the apartment. The design flow was found to be 0.365 cfs. We were given the elevation at each intersection, along with the length of each section. From here we were able to calculate the slopes and ensure that they were greater than the minimum slope of 0.0033 ft/ft. We then calculated the required diameter of the pipe. For the section running from Hester to Ramsey, we found the needed diameter to be 5.35 inches. However, the Stillwater Engineering Standards state that the minimal sewer pipe size is 8 inches, so this is what they size of our pipe would be. The pipe capacity was then checked to make sure it could support the system, which it could support 2.13 cfs. This is far greater than the needed 0.365 cfs. The maximum and minimum velocities were evaluated to make sure they met the desired range set forth by the city of Stillwater. The maximum average velocity was 5 fps and the maximum peak velocity was 7 fps. The minimum velocity was 2 fps. All of our velocities fell in those ranges. This section had a hydraulic grade line of 899 feet. This process was repeated for the section running from Ramsey to Washington. It was found to have a diameter of 5.81 inches would was bumped up to 8 inches as well and it met all capacity and velocity requirements. This section had a hydraulic grade line of 888 feet. All of the pipes would be PVC material due to its high performance values and reasonable costs. These pipes would be buried at least 3 feet below ground level to meet the city standards. Manholes would be installed at each intersection. This would be 3 feet in diameter so that city engineering standards would be met. There is currently a 6 inch PVC sewer pipe on our development site that we will not need and are choosing to abandon.

The plan and profile views can be seen below. Pipe calculations can be found in Appendix B-2.

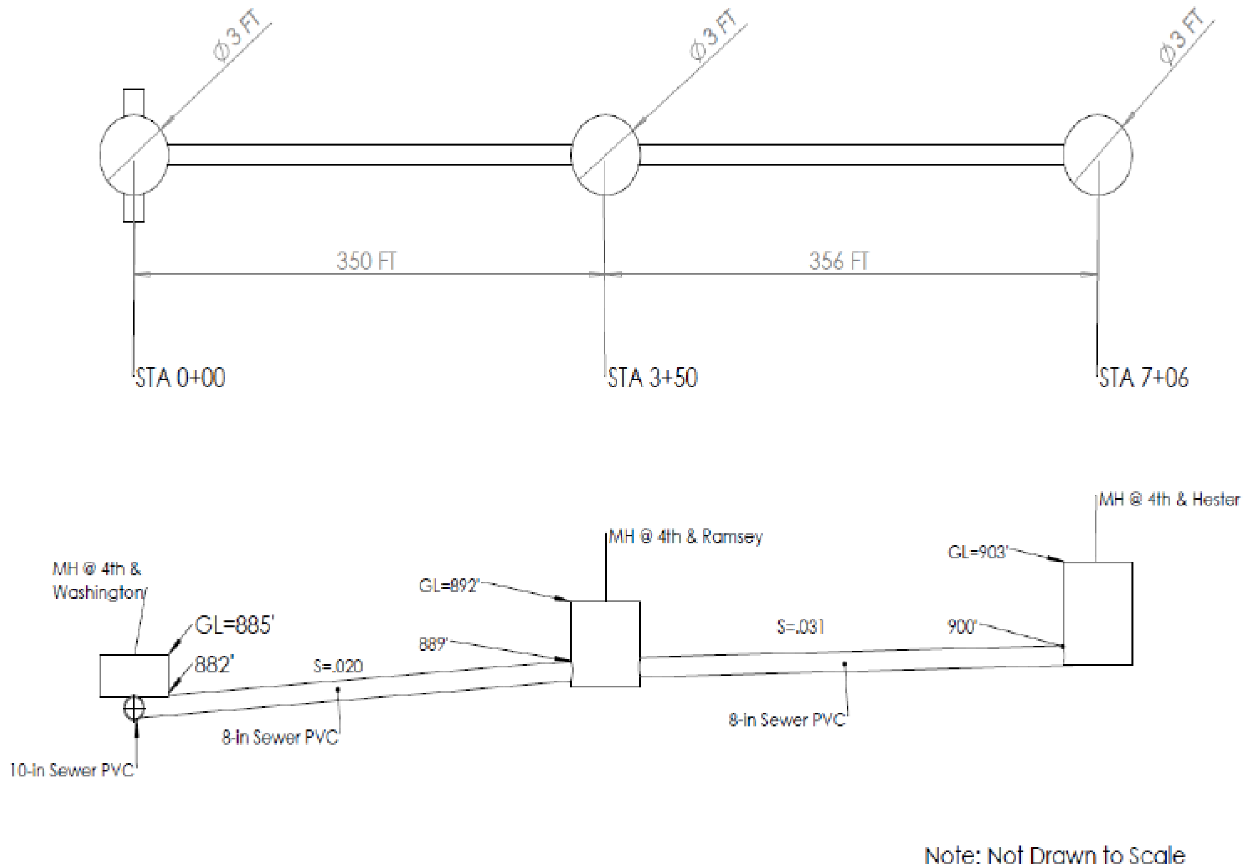


Figure 4- Plan and Profile View of New Sewer Line

Hydrology

Hydrologically we were responsible for determining the hydrological conditions of the site and developing a stormwater management plan. This included determining the site characteristics pertaining to surface runoff, the predevelopment and post-development peak runoff flow rates, and the design and function of a detention facility if necessary. It was the goal of our team to develop an accurate, effective, and feasible hydrological assessment and stormwater detention plan. The team's process and conclusions are presented below.

In order to determine the predevelopment site characteristics our team consulted the NRCS Online Web Soil Survey, the given AutoCad file of the existing site conditions including a topographic survey of the site, and regular site visits. From the NRCS Online Web Soil Survey we were able to determine the soil at our site is Norge Urban Land Complex with 1-5% slopes with a C-grade hydrological classification. Using the existing site autocad file we were given we were able to determine a total area of the site to be approximately 2.5 acres including the sidewalks surrounding the property on the East, North, and West sides extending from the property boundary.

The next step was to determine the watershed characteristics of the site. This included determining the number of control points that where water exits the site, assigning the drainage areas the contribute to each respective control point, and lastly developing the characteristics of each drainage area used to effectively calculate the predevelopment runoff values for the site. Using the existing site AutoCad file combined with site visits we were able to determine three control points where water enters the stormwater drainage system of Stillwater, Oklahoma, and assign the respective areas that contribute to these control points.

The first control point is located on the corner of 4th and Ramsey and accounts for approx. 1.23 acres. From control point 1 the storm water flows west on 4th St. to Washington St., where the water either enters the storm sewer system on Washingston St. or flows South to inlets located at 6th St. The second control point is located at the Southwest corner of the property and has a contributing area of approx. 1.12 acres. As seen, water that contributes to this point exits the property boundary at two locations but is rejoined some distance along Ramsey St. From the second control point water flows South on Ramsey St. until it enters the storm sewer system at an inlet on 6th St. The third control point is located in the Southeast corner of the property and has a contributing area of 0.15acres. This area is mainly consists of the East sidewalk. The water from this control point travels South on Hester street and enters the storm sewer system at inlets located on 6th St. The diagram showing the control points, their respective areas, and flows to the storm sewer are shown in Figure 5. below.

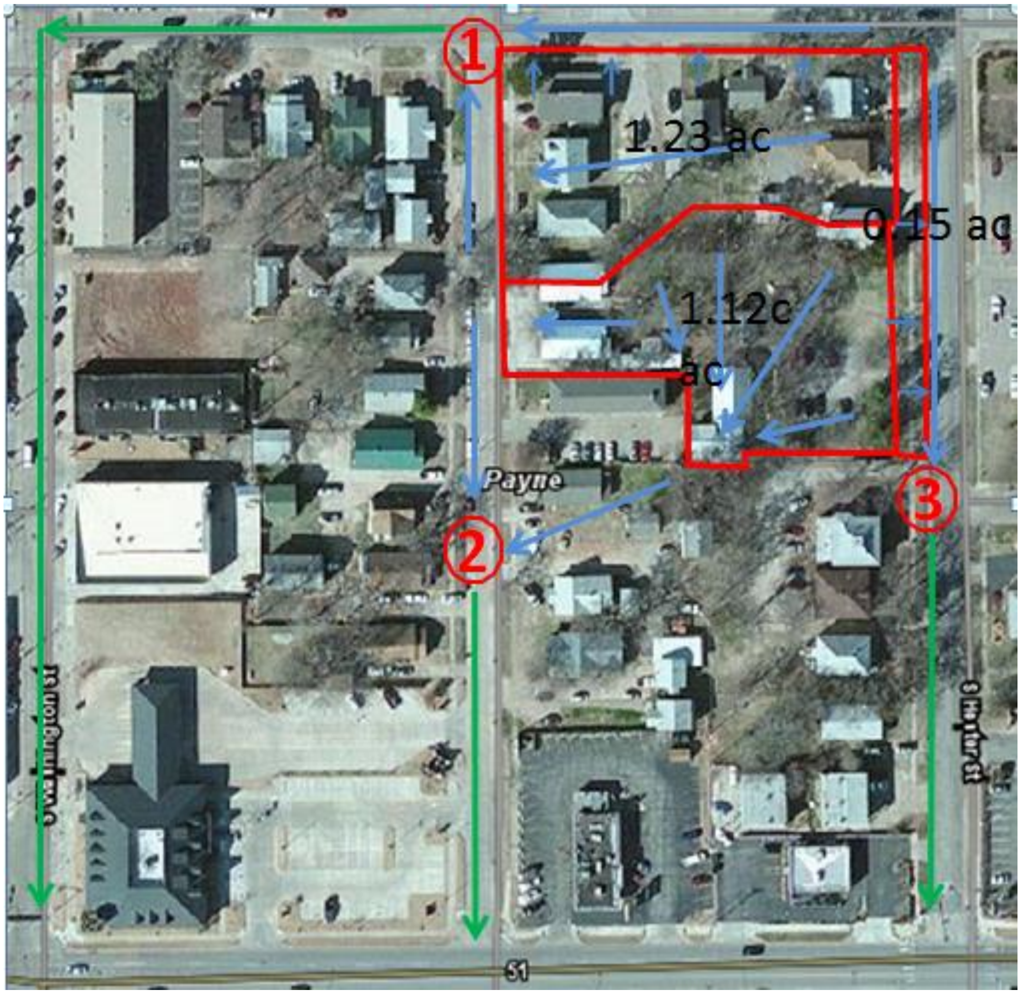


Figure 5- Pre-development flow diagram

Next, WinTR55 small watershed analysis program was used to model each drainage area. The inputs used in the program were calculated using each individual drainage area. The curve-number detail was determined by the program after the area of each type of land cover (impervious, grass, etc.) were calculated using the dimensions presented in the existing site AutoCad file. Time of concentration and reach detail was determined by the program with the input of the type of flow expected at the site. The time of concentration for control point one and three were calculated to be less than 0.1hr causing the program to default to the minimum value of 0.1hr. The inputs and respective output for each control point are presented in Appendix B. Hydrological Inputs and Calculations . Allowing the 100 year-24 hr frequency storm to control the design, the predevelopment peak flow rates for control points 1, 2, and 3 were determined as 12.70 cfs (cubic feet per second), 10.11 cfs, and 1.47 cfs respectively.

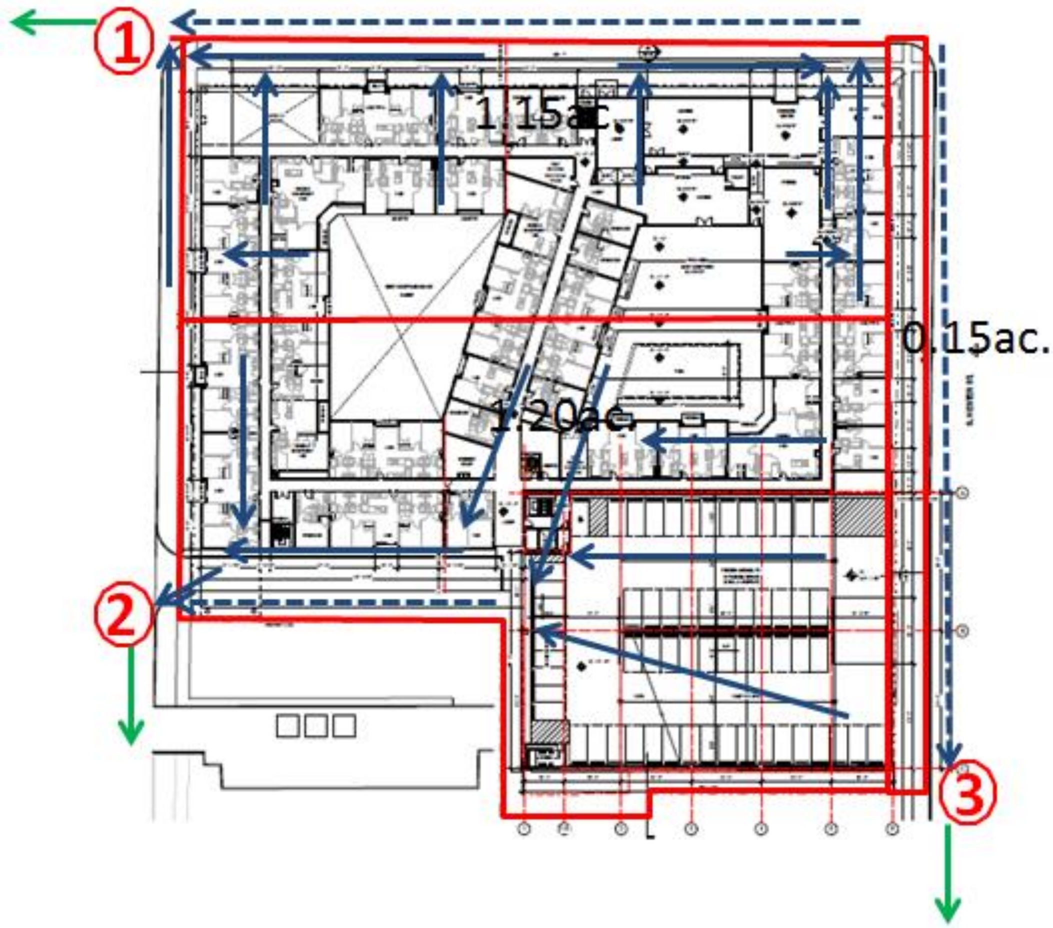


Figure 6 - Post Development Flow Diagram

To determine the post-development peak flow rates the relative same process as described above was used. The same three control points were used and the respective areas were decided in a manner to optimize the amount of water that we were able to flow off of the site. The control points and their assigned areas are presented in figure 7 below. As shown, it is assumed that the area in the north portion will flow from the roof in a manner that it contributes to the area of control point 1 on the corner of 4th and Ramsey. In addition the area in the south, including the parking garage is assumed to flow to control point 2. The area contributing to control point 3 is the same portion of the sidewalk as the predevelopment conditions. WinTR55 small watershed analysis program was employed to determine the post development peak runoff rates and hydrographs. Due to the floor plan of the building to be 95% lot coverage, a conservative CN value of 98 was selected for areas 1 and 2 which assumes the entire area is impervious cover. The reach detail used for the calculation was designed as a very large channel with 1% slope which conservatively allows the water to flow from points in a manner faster than that in the field. This resulted in higher peak flow rates allowing for a conservative design. The inputs and output used for the post-development calculation are detailed in *Appendix C Hydrological Inputs and Calculations* The peak flow rates for the controlling 100yr frequency 24 hour storm for points 1, 2, and 3 were determined as 12.67 cfs, 13.79 cfs, and 1.47 cfs respectively. The peak flow for control point 3 was not changed because it is outside of the property boundary and is not going to undergo further development.

With pre and post development peak flows determined, we were able to determine the need for a stormwater detention facility. Control points 1 and 3 did not exceed the predevelopment conditions and therefore there is no for detention of water that exits these point. For control point 1 this occurred because of the smaller contributing area, and for control point 3 this occurred because the area remained unchanged. For control point 2 however, the post-development exceeds the predevelopment peak runoff rates and therefore detention is required to reduce the peak runoff flow below the predeveloped conditions. In this case we need to ensure the peak runoff at control point 2 does not exceed 10.11 cfs. To do this our team investigated two options, the first would be to detain all of the water that flows to control point 2 and release it at a predesigned rate, and the second would be to detain only a part of the water and release it at the design rate.

To evaluate both options, our team needed to estimate the size of the detention facility for each case. The calculations used to determine the size of detention facility for each case are presented in *Appendix C Hydrological Inputs and Calculation*. In the first case, the detention facility would need at least 1728 cubic feet of volume (approx. 13,000 gal.) and would require routing all water in the drainage area to the detention system and then controlling the release to allow for 10 cfs of flow. The second would detain water from the east most 0.7 acres of the facility and would require a detention volume of 2392 cubic feet of volume and allow for a controlled release of approx. 4.0 cfs.

Looking at the site layout, our team determined the best location for a detention facility would be either an aboveground or belowground system located in the area of the driveway on the southwest corner of the property. This location is would allow for easy access to the detention facility for possible maintenance and allows for greater flexibility in the design. After selecting this option our team selected going with the second option presented above. This option allows for the water from the west 0.5 acres of the drainage area to be directly deposited from the roof to Ramsey St., while the water from the east 0.7 acres to be deposited directly into the detention facility. With this option selected we decided to locate the detention facility at the west most portion of the driveway located in the southwest corner of the property. This allows for the water to flow from the detention facility directly onto the driveway that is designed to carry the flow to Ramsey street. A figure showing the location of the detention facility and the approximate flow of water are presented in figure 7 below.

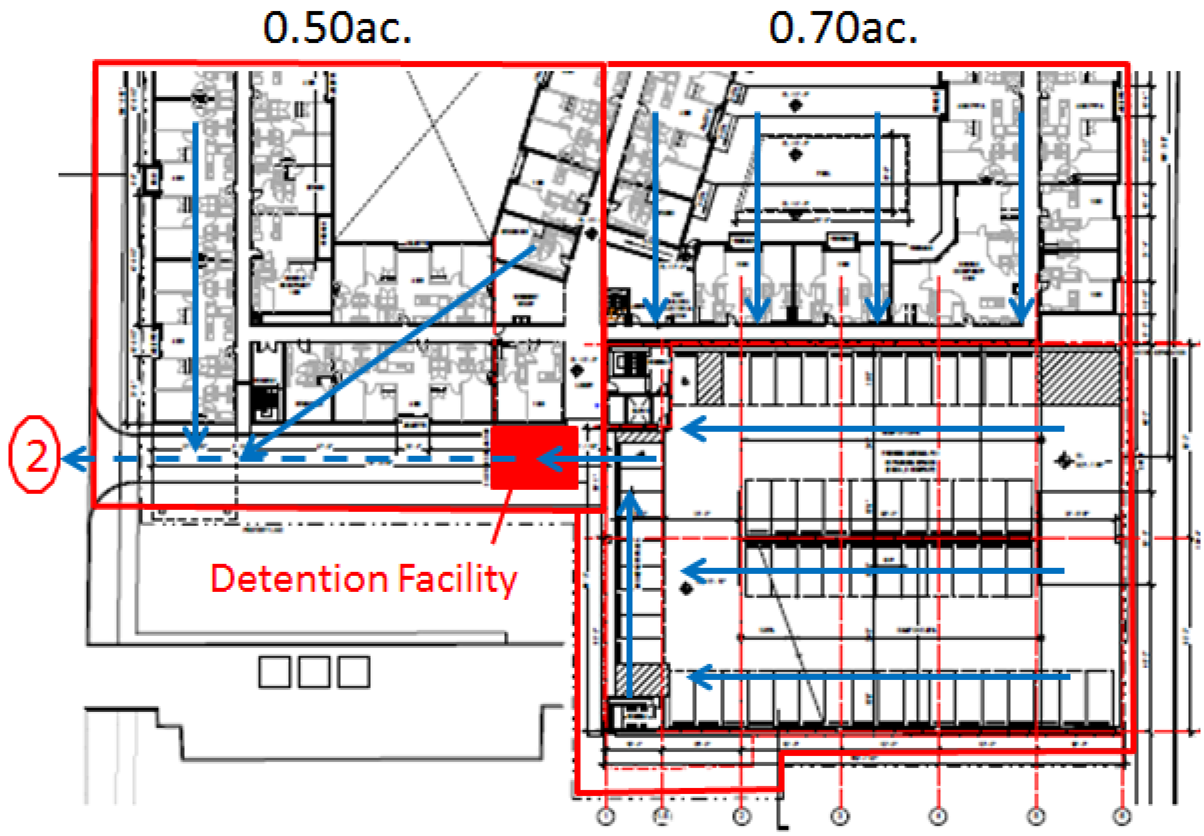


Figure 7 -Flow Diagram w/ Detention Facility

With the detention facility sized and located, we were then able to determine operation of the detention facility. Our team investigated two options, a gravity flow facility and a modified pump flow facility. The gravity flow system would be an above ground system (fig. 8) consisting of at least an 18,000 gal tank with an orifice at the bottom to limit the flow of water to 4 cfs. The tank would need to have an overflow above the 17,900 gal mark in case of malfunction. The tank would have a manhole at the top to allow for entrance into the tank. Due to this being such a large tank to place above ground, and the negative aesthetics of the situation, our team decided to pursue the underground option.

For the underground option, our team selected a modified pump flow facility similar to the one presented in figure 9. In this case, the facility would be a concrete vault approximately 10 x 10 x 25 ft. It would operate by water flowing through a channel that is controlled by an orifice that would only allow 4.0 cfs through the channel. If the channel were to experience more than 4.0 cfs it would fill up and overflow into the tank. The emptying of the tank would be controlled by a pump that would lift the water back into the channel and allow the water to exit the facility. The bottom of the channel would be the same elevation as the road causing the top of the facility to extend approximately 1.5 ft above the ground surface. In order to provide maintenance to the facility a manhole would be installed. In order to drain the facility in less than 4 hours, a pump of at approx. 75 gpm will be needed.

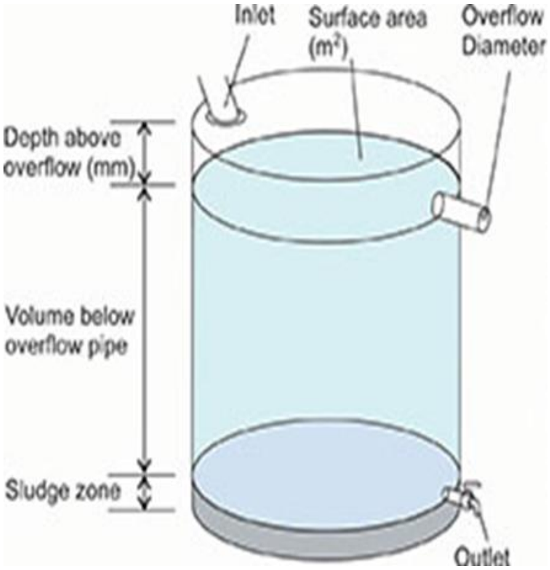


Figure 8 - Above ground detention

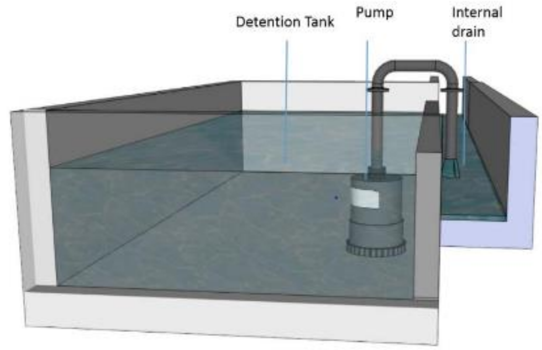


Figure 9- Below ground detention

Based on the following options, our team selected the aboveground option. This is primary due to the increased cost with installing a below ground system and the added maintenance cost included with an underground pumped design.

Structural Design

Structurally, we were tasked with designing the parking garage facility for the project. This included deriving the loads for the structure from International Building Code 2009 and American Society of Civil Engineers/Structural Engineering Institute 7-2010 as well as designing the structural elements such as beams, columns, and piers out of reinforced concrete using the design standards presented in American Concrete Institute 318-2014 and PCI Design Handbook: Precast and Prestressed Concrete 7th Edition. The vast majority of the computations were carried out using Microsoft Excel Spreadsheets derived by the structural design team members.

The layout of the structural elements had to be changed from the original layout due to the required floor to floor clear spacing of ten feet provided by the customer and the required floor to bottom of beam clear spacing of seven feet. With the original number of columns and the span requirements, it was quickly obvious that the provided spacing was not adequate to meet these clear spacing specifications using a reinforced concrete structure. Because our design computations were carried out through excel spreadsheets, the design group was able to optimize a modified column spacing for a reinforced concrete structure that could provide a way to meet these limits without far exceeding the requirements with a non-economical design. The final optimized layout required us to increase the number of columns from 12 to 41. The increase represents the minimum depth that can be achieved with a Prestressed concrete design vs a precast, reinforced design. The change from having the greatest span of more than 60 ft. to one of 32 ft. allowed for an immense decrease in the depth of the cross section based on the limit found in ACI 318-14. This value went from a required minimum depth of more than three and a half feet to a value of two feet two inches. While this change results in an overall increase in the amount of raw materials necessary to construct the parking garage, and thus an increase in cost, the change increases the structural redundancy to improve the safety and reliability of the structure in the event of a localized failure. The final layout is shown in the figure below:

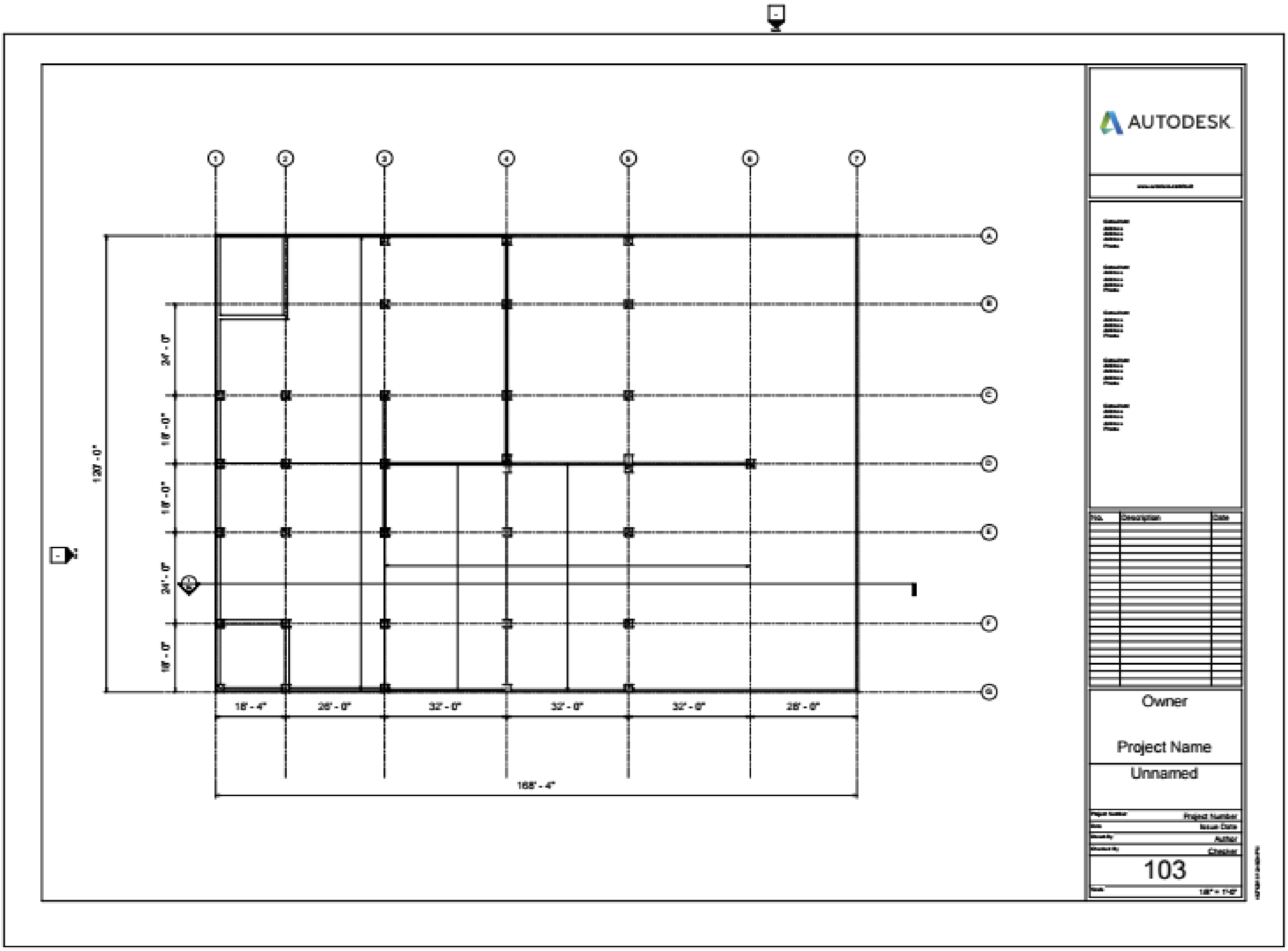


Figure 10. Structural Floor Layout

The ultimate goal during design was to provide a safe and reliable structure that would be capable of handling the loads sustained on the parking garage while retaining the serviceability that the users would require. Load derivation from Chapter 16 of IBC 2009 consisted of determining the Occupancy Category for the structure prior to using the tables to determine the live, snow, wind, rain, and earthquake loads for an Occupancy Category of II and a location in Stillwater, OK. The exact dead load was used in conjunction with an additional 5 psf load to cover the effect of MEP loading. The additional 5 psf was not taken into account in the combinations that included a wind or seismic uplift component, per ASCE 7-10 requirements. The exact loads are presented in Appendix C. Structural Design Calculations

Design for each structural element consisted first of determining the maximum distributed load from the strength design load combinations presented in ACI 318-14 and the tributary width and span length at the location of the element. Preliminary beam dimensions are selected based on these values, such as the beam depth and height. From this, the dead weight of the element can be distributed over the area and included in the calculations for the ultimate load. After this, the ultimate moment and shear were calculated based on pre-determined support conditions, in our case simply supported beams resting on column corbels. From the maximum moment calculated from the load cases, the flexural steel was calculated using the Goal Seek function. Goal Seek was used because the equation in ACI 318-14 for flexural reinforcement is a quadratic function. The exact number of steel bars used was calculated for each iteration for a variety of rebar sizes, which allowed the structural engineer to optimize the beam to require the least amount of material possible while also providing the necessary strength. Shear reinforcement was selected similarly, but unlike the triple shear zone method found in ACI 318-14, a single spacing was selected across the span of the bridge to increase the provided ductility and redundancy. Required shear reinforcement and spacing was calculated using the equations in ACI 318-14 for multiple locations throughout the span to ensure that the provided amount was always adequate for the ultimate value imposed by the load.

The necessary serviceability checks were also performed at each iteration. The clear spacing limit of the maximum of either the diameter of the longitudinal bar chosen or a value of one inch was calculated for a single, double, and triple layer of reinforcing bars. Because the optimal location of reinforcement is further down the depth of the cross section, the least number of layers allowed by the clear spacing limit could be chosen to provide the maximum flexural capacity. Another check necessary for the serviceability of the structure was the deflection limit equal to one sixteenth the value of the span. This value changes for each element, so an excel spreadsheet was necessary to calculate each additional iteration of the design of each element. Deflection in concrete beams is based on the lesser of the cracked or gross moment of inertia, which depends on cross sectional properties, so for each change in these values, no matter how minimal, had an effect on the deflection values of the element. It was necessary to keep in mind how each design element influenced deflection as it was the controlling aspect of most of the design process. Knowing the smaller details of design, as well as keeping the bigger picture in mind was necessary to optimize the elements. The ultimate tensile strain in the longitudinal steel was also calculated to verify that the element was in the tensile failure mode and not in the compression zone. This ensures that in the event of failure, the failure will occur in a slow, ductile fashion, not the drastic, sudden failure that a compression failure event would incur. By ensuring that the element is always in the tension failure zone, it greatly simplified the design process and the optimization of the required reinforcing.

In addition to the clear spacing, ultimate tensile strain of the steel, and the deflection requirements, the necessary lap splices, developmental length, and reinforcing ratio in each member was also calculated. These values are necessary to detail to provide adequate force transfer between elements and to develop the composite action between the steel reinforcing and the precast concrete elements. The reinforcing ratio minimum requirement of 0.015 is provided to ensure adequate composite interaction while the maximum requirement of 0.08 is to prevent the overcrowding of bars in the cross section. Each of the values were compared to the limits as stated in ACI 318-14, with the amount of reinforcing or cross sectional properties changed to verify that each was alright.

Column and beam-column connection was performed in a similar fashion, with similar calculation checks occurring at each step and iteration of the process. Because the corbel connections are simply a short span cantilever beam with an angled compression zone, the design was very straightforward and easily able to draw upon the calculations performed in the beam and girder design calculation sheets. Column design consisted of a similar series of design iterations at each location to provide adequate support for the structure, with each column being a single continuous element upon which precast beams, girders, and slab sections can rest without additional requirements for other supporting elements. This simplified the design of the columns by providing purely axial loading effects without a significant moment applied.

The slab, designed as a rigid diaphragm connecting the exterior simple column and beam supports to the shear wall allowed for complete design of a lateral force resisting system capable of absorbing the loads from wind and seismic events.

The exact calculations for each element, with labels in the heading on each page, can be found in the design spreadsheets attached in *Appendix D 1 Structural Design Calculations*

Foundation Design

The geotechnical investigation provided undrained shear strength and shear angle values for six separate borings taken over the area that the parking garage will sit. Using these values, the foundation design team members were able to calculate the design strength vs. depth for three differently sized piers. A deep foundation, such as drilled, cast in place piers, was chosen primarily for the fact that a shallow foundation would not have been adequate for the soil structure and the high values of load experienced by the large reinforced concrete parking structure. Piers were chosen over driven piles due to the noise level and cost that is required by the equipment necessary to drive piles deep into the earth. Because the structure is nearby residential, commercial, and educational facilities, the noise level of the necessary construction equipment is an important thing to consider. The primary design codes utilized for the foundation design were in Chapter 1810 Deep Foundations of IBC 2009, ACI 336.3R Report on Design and Construction of Drilled Piers, and in ACI 318-14 Chapter 25.

From the structural engineering group members, the maximum axial, moment, and shear values were obtained. Using the maximum moment value, it was determined that a two and a half foot diameter pier would be most cost effective to provide the necessary flexural capacity of the reinforced concrete pier and necessary side bearing force to prevent uplift of the structure. A Microsoft Excel spreadsheet was created in conjunction with the base design strength charts that calculated the required depth of each pier at the location of each column and pier. To do this, each boring location was provided with coordinates in Northing and Easting directions and the depth and design strength determined based on the ultimate axial load. Similarly, each column and wall junction was given coordinates where the South-Westernmost column was chosen as the origin point. From the coordinates, depths, and strengths, an equation was derived in excel that was able to calculate the inverse distance weighted average based on the boring logs that surrounded it. Similar to a weighted average, which calculates a basic interpolation between two points, the inverse distance weighted average is able to calculate the weighted average between multiple points of reference. The main application of the inverse distance weighted average is in interpolating topographical and similar environmental factors, so the application to the varying strength and depth requirements at a point in the middle of four boring locations at different distances allowed for a more precise estimate of the necessary depth and strength provided at each location. This is the reason that the design provides the necessary strength without over utilizing material and construction resources in assuming the worst case throughout the site.

The design for the reinforcing of each pier is the same, due to similar axial loads and to optimize constructability when casting each pier. The design was done using a combination of an excel spreadsheet to determine factors and an interaction diagram obtained from ACI Special Publication 7. The strength safety factor of 0.65 was used to modify the design as noted in ACI 318-14 from when the original publication was produced. The design of eight longitudinal #10 bars with a #5 spiral tie with an inclined pitch of 5.0 inches provided the necessary strength without exceeding the limits imposed by ACI 318-14 Chapter 7 for clear spacing, axial shortening, and minimum reinforcing values, as reflected in the design spreadsheet in *Appendix D 2. Foundation Design Calculations*

Grading Plan

We were asked to create the grading plan for the proposed site. We were given elevations and a topographical map of the site that was from a recent survey. As a team, we worked to provide a grading plan that minimized excess cut and fill as well as one that could accompany the new utility lines that were needed for the project.The existing topography can be seen below

0'= 900' Evelation

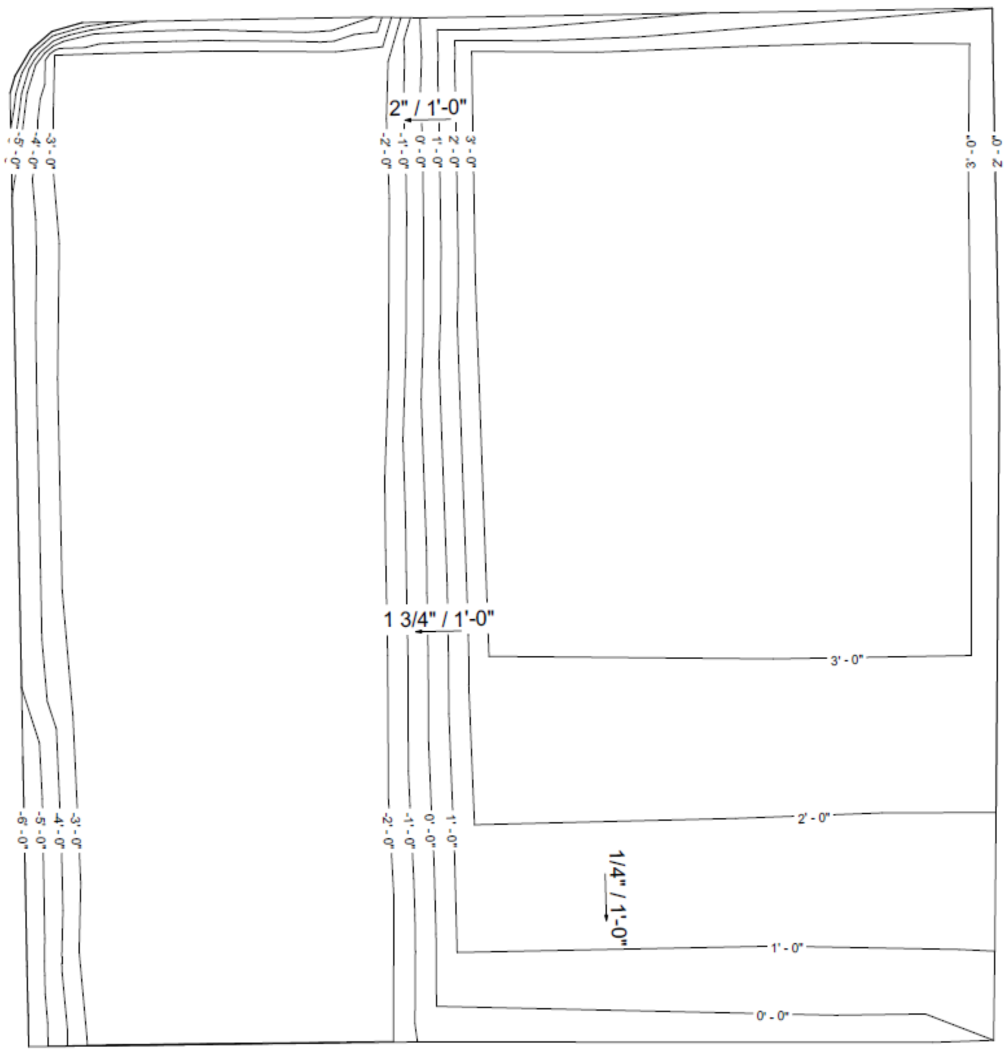


Figure11- Grading plan topography

Figure 12 - Existing site topography

To start the process, we took the elevations given in the construction drawings and used the finished floor elevations to start determining the initial ground elevations for our site. Our lot has 95% building coverage so we decided to place more emphasis on the slab layout rather than the hydrology of the site. The property line does not include the existing sidewalks or roads. We focused mainly on the earthwork within the property line. We set the highest point of the lot on the Northeast corner at 905'. We then compared the proposed elevations with the elevations for our updated utilities. The highest point of our utilities line was also located on the northeast corner at 900'. The proposed elevation allowed for proper cover for the new lines. After checking the utilities height, we used the proposed site layout and began to hand draw the contours we needed for the grading.

To model our grading plan and calculate our total cut and fill, we used a combination of Google Earth and Revit. Google Earth contains existing topography and allows users to take sections and import it into CAD software. Using Revit we were able to manipulate the surface to get the required elevations. The final drawing can be seen below. When we imported the topography from Google Earth, the relative elevation was lost. To correct this in our model, we established a control point on the imported surface and made all the elevations relative to this point. On our grading plan. The elevation 0' is equivalent to 900' in the field. After grading, we had an excess of 3863 cubic yards of cut.

Construction Management

Project Estimation:

We used a unit cost analysis to estimate the overall cost of our parking garage. The four main sections that were analyzed are sewage line installation, water line installation, grading, structural elements, detention facility and interiors. Interiors include electrical wiring, elevators, staircases and fire protection equipment. The parking garage has 6 floors including the basement. The actual floor area is 113,363 sq. ft. but for estimation purposes the area per floor was assumed to be 113,000 sq. ft. The costs for all the components of the garage were calculated using 2011 RS Means. The unit price includes material cost, equipment cost and labour cost. Once the project cost was estimated, the total cost was multiplied by the location factor for Oklahoma. According to RS means, construction in Oklahoma costs 80% of the national average. Once all the costs were calculated, the location factor was applied to get the final project cost.

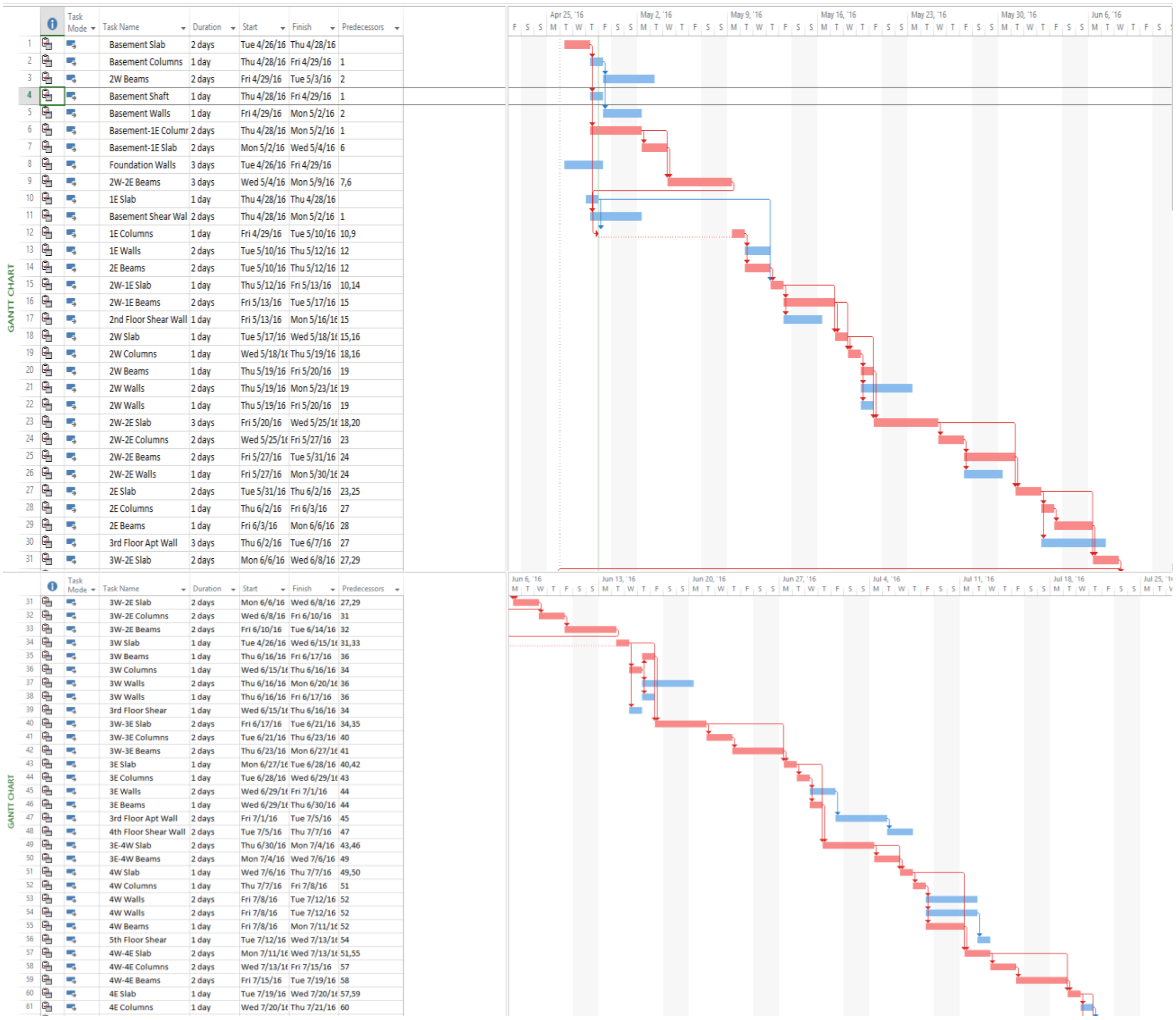
Structural cost was calculated on a linear foot or square foot basis. The length of all the columns were added together and multiplied by the unit price per linear foot. Beams and girders costs were also estimated in a similar way. To obtain the total slab area, the floor area of the garage was multiplied by 6 to account for the floor area of all 6 floors.

The total project cost after accounting for location factors was estimated to be **$ 4470624.48.** All the costs for the structure, grading, utilities and detention are given below.

Sewer Line Installation					
Construction Event	Unit	Unit Price	Amount	Cost	% of total
Construct 8" PVC Sewer Line*	LF	$ 214.00	706	$ 151,084.00	
Construct 3' Manhole	Each	$ 2,500.00	3	$ 7,500.00	
Connect to Existing Line	Each	$ 2,800.00	1	$ 2,800.00	
Total				**$ 161,384.00**	**2.89%**
*Includes excavation, pipe installation, and backfill					
8" PVC pipe costs $12.37/ft					
Water Line Installation					
Construction Event	Unit	Unit Price	Amount	Cost	
Construct 10" PVC Line*	LF	$ 225.00	725	$ 163,125.00	
Construct 3' Manhole	Each	$ 2,500.00	1	$ 2,500.00	
Connect to Existing Line	Each	$ 2,800.00	1	$ 2,800.00	
Total				**$ 168,425.00**	**3.01%**
*Includes excavation, pipe installation, and backfill					
10" PVC pipe costs $19.99/ft					
Structural Elements					
	Unit	Unit price	Amount	Cost	
Slabs	SQ. FT	$ 8.75	113363	$ 991,926.25	
Columns	LF	$ 150.40	2758	$ 414,803.20	
Beams & Girders	LF	$ 139.85	17004	$ 2,378,009.40	
Total				**$ 3,784,738.85**	**67.73%**
Substructure (Grading And Foundation)					
	Unit	Unit price	Amount	Cost	
Basement Excavation	SQ. FT	$ 0.18	113000	$ 20,340.00	
Piers	CY	$ 273.59	31.27	$ 8,555.16	
Slab on Grade	SQ. FT	$ 6.24	114000	$ 711,360.00	
Total				**$ 740,255.16**	**13.25%**
Interiors					
	Unit	Unit price	Amount	Cost	
Stairs	Each	$ 3,750.00	1	$ 3,750.00	
Elevators	Each	$164,575.00	1	$ 164,575.00	
Fire Protection	SQ. FT floor	3.83	78000	$ 298,740.00	
Electrical	SQ. FT floor	3.29	78000	$ 256,620.00	
Total				**$ 719,935.00**	**12.88%**
Detention Facility					
	Unit	Unit price	Amount	Cost	
Detention Facility	CY	365.65	37.03704	$ 13,542.59	**0.24%**
	Total Project Cost			**$ 5,588,280.60**	
Total Project Cost after location adjustment				**$4,470,624.48**	

Project Scheduling:

We used RS Means for the scheduling for the parking garage. Additionally, we used a current construction project to adjust our time for the schedule. There is a parking garage being built on the south side of Stillwater that is of comparable size and material. It will take a total of 183 days to complete the structural elements of the parking garage. That is assuming an eight hour work day and not working on Saturday and Sunday.

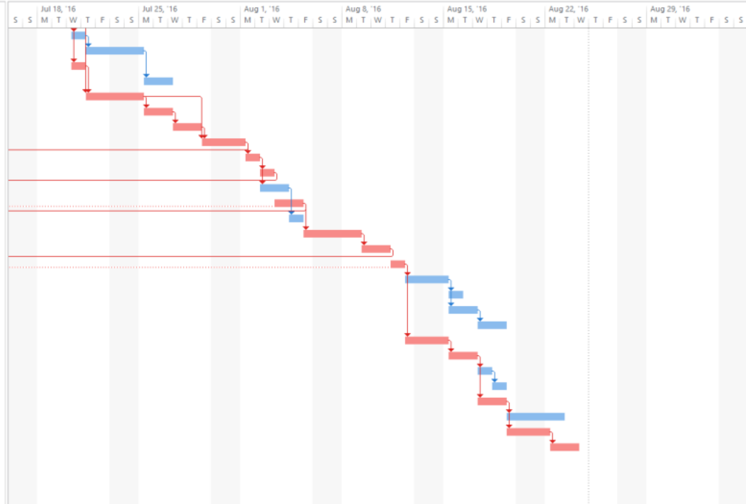


	Task Mode	Task Name	Duration	Start	Finish	Predecessors
1		Basement Slab	2 days	Tue 4/26/16	Thu 4/28/16	
2		Basement Columns	1 day	Thu 4/28/16	Fri 4/29/16	1
3		2W Beams	2 days	Fri 4/29/16	Tue 5/3/16	2
4		Basement Shaft	1 day	Thu 4/28/16	Fri 4/29/16	1
5		Basement Walls	1 day	Fri 4/29/16	Mon 5/2/16	2
6		Basement-1E Columr	2 days	Thu 4/28/16	Mon 5/2/16	1
7		Basement-1E Slab	2 days	Mon 5/2/16	Wed 5/4/16	6
8		Foundation Walls	3 days	Tue 4/26/16	Fri 4/29/16	
9		2W-2E Beams	3 days	Wed 5/4/16	Mon 5/9/16	7,6
10		1E Slab	1 day	Thu 4/28/16	Thu 4/28/16	
11		Basement Shear Wal	2 days	Thu 4/28/16	Mon 5/2/16	1
12		1E Columns	1 day	Fri 4/29/16	Tue 5/10/16	10,9
13		1E Walls	2 days	Tue 5/10/16	Thu 5/12/16	12
14		2E Beams	2 days	Tue 5/10/16	Thu 5/12/16	12
15		2W-1E Slab	1 day	Thu 5/12/16	Fri 5/13/16	10,14
16		2W-1E Beams	2 days	Fri 5/13/16	Tue 5/17/16	15
17		2nd Floor Shear Wall	1 day	Fri 5/13/16	Mon 5/16/1	15
18		2W Slab	1 day	Tue 5/17/16	Wed 5/18/1	15,16
19		2W Columns	1 day	Wed 5/18/1	Thu 5/19/16	18,16
20		2W Beams	1 day	Thu 5/19/16	Fri 5/20/16	19
21		2W Walls	2 days	Thu 5/19/16	Mon 5/23/1	19
22		2W Walls	1 day	Thu 5/19/16	Fri 5/20/16	19
23		2W-2E Slab	3 days	Fri 5/20/16	Wed 5/25/1	18,20
24		2W-2E Columns	2 days	Wed 5/25/1	Fri 5/27/16	23
25		2W-2E Beams	2 days	Fri 5/27/16	Tue 5/31/16	24
26		2W-2E Walls	1 day	Fri 5/27/16	Mon 5/30/1	24
27		2E Slab	2 days	Tue 5/31/16	Thu 6/2/16	23,25
28		2E Columns	1 day	Thu 6/2/16	Fri 6/3/16	27
29		2E Beams	1 day	Fri 6/3/16	Mon 6/6/16	28
30		3rd Floor Apt Wall	3 days	Thu 6/2/16	Tue 6/7/16	27
31		3W-2E Slab	2 days	Mon 6/6/16	Wed 6/8/16	27,29

	Task Mode	Task Name	Duration	Start	Finish	Predecessors
31		3W-2E Slab	2 days	Mon 6/6/16	Wed 6/8/16	27,29
32		3W-2E Columns	2 days	Wed 6/8/16	Fri 6/10/16	31
33		3W-2E Beams	2 days	Fri 6/10/16	Tue 6/14/16	32
34		3W Slab	1 day	Tue 4/26/16	Wed 6/15/1	31,33
35		3W Beams	1 day	Thu 6/16/16	Fri 6/17/16	36
36		3W Columns	1 day	Wed 6/15/1	Thu 6/16/16	34
37		3W Walls	2 days	Thu 6/16/16	Mon 6/20/1	36
38		3W Walls	1 day	Thu 6/16/16	Fri 6/17/16	36
39		3rd Floor Shear	1 day	Wed 6/15/1	Thu 6/16/16	34
40		3W-3E Slab	2 days	Fri 6/17/16	Tue 6/21/16	34,35
41		3W-3E Columns	2 days	Tue 6/21/16	Thu 6/23/16	40
42		3W-3E Beams	2 days	Thu 6/23/16	Mon 6/27/1	41
43		3E Slab	1 day	Mon 6/27/1	Tue 6/28/16	40,42
44		3E Columns	1 day	Tue 6/28/16	Wed 6/29/1	43
45		3E Walls	2 days	Wed 6/29/1	Fri 7/1/16	44
46		3E Beams	1 day	Wed 6/29/1	Thu 6/30/16	44
47		3rd Floor Apt Wall	2 days	Fri 7/1/16	Tue 7/5/16	45
48		4th Floor Shear Wall	2 days	Tue 7/5/16	Thu 7/7/16	47
49		3E-4W Slab	2 days	Thu 6/30/16	Mon 7/4/16	43,46
50		3E-4W Beams	2 days	Mon 7/4/16	Wed 7/6/16	49
51		4W Slab	1 day	Wed 7/6/16	Thu 7/7/16	49,50
52		4W Columns	1 day	Thu 7/7/16	Fri 7/8/16	51
53		4W Walls	2 days	Fri 7/8/16	Tue 7/12/16	52
54		4W Walls	2 days	Fri 7/8/16	Tue 7/12/16	52
55		4W Beams	1 day	Fri 7/8/16	Mon 7/11/1	52
56		5th Floor Shear	1 day	Tue 7/12/16	Wed 7/13/1	54
57		4W-4E Slab	2 days	Mon 7/11/1	Wed 7/13/1	51,55
58		4W-4E Columns	2 days	Wed 7/13/1	Fri 7/15/16	57
59		4W-4E Beams	2 days	Fri 7/15/16	Tue 7/19/16	58
60		4E Slab	1 day	Tue 7/19/16	Wed 7/20/1	57,59
61		4E Columns	1 day	Wed 7/20/1	Thu 7/21/16	60

GANTT CHART

		Task Mode	Task Name	Duration	Start	Finish	Predecessors
61			4E Columns	1 day	Wed 7/20/1€	Thu 7/21/16	60
62			4E Walls	2 days	Thu 7/21/16	Mon 7/25/1€	61
63			4E Beams	1 day	Wed 7/20/1€	Thu 7/21/16	60
64			5th Floor Apt Wall	2 days	Mon 7/25/1€	Wed 7/27/1€	62
65			5W-4E Slab	2 days	Thu 7/21/16	Mon 7/25/1€	60,63
66			5W-4E Columns	2 days	Mon 7/25/1€	Wed 7/27/1€	65
67			5W-4E Beams	2 days	Wed 7/27/1€	Fri 7/29/16	66
68			5W Slab	1 day	Fri 7/29/16	Mon 8/1/16	65,67
69			5W Columns	1 day	Mon 8/1/16	Tue 8/2/16	68
70			5W Beams	1 day	Tue 8/2/16	Wed 8/3/16	69
71			5W Walls	2 days	Tue 8/2/16	Thu 8/4/16	69
72			5W-5E Slab	2 days	Tue 4/26/16	Fri 8/5/16	68,70
73			6th Floor Shear Wall	1 day	Thu 8/4/16	Fri 8/5/16	71
74			5W-5E Columns	2 days	Fri 8/5/16	Tue 8/9/16	72
75			5W-5E Beams	2 days	Tue 8/9/16	Thu 8/11/16	74
76			5E Slab	1 day	Tue 4/26/16	Fri 8/12/16	72,75
77			5E Columns	1 day	Fri 8/12/16	Mon 8/15/1€	76
78			5E Beams	1 day	Mon 8/15/1€	Tue 8/16/16	77
79			5E Walls	2 days	Mon 8/15/1€	Wed 8/17/1€	77
80			6th Floor Apt Wall	2 days	Wed 8/17/1€	Fri 8/19/16	79
81			5E-6W Slab	1 day	Fri 8/12/16	Mon 8/15/1€	76
82			6W Slab	2 days	Mon 8/15/1€	Wed 8/17/1€	81
83			6th Floor Shaft Walls	1 day	Wed 8/17/1€	Thu 8/18/16	82
84			Shaft Roofs	1 day	Thu 8/18/16	Fri 8/19/16	83
85			6W-6E	2 days	Wed 8/17/1€	Fri 8/19/16	82
86			6W Walls	2 days	Fri 8/19/16	Tue 8/23/16	85
87			6E Slab	1 day	Fri 8/19/16	Mon 8/22/1€	85
88			6E Walls	2 days	Mon 8/22/1€	Wed 8/24/1€	87



Appendix A - Transportation Analysis

Determining Peak Hour capacity using the traffic distribution on a weekday.

	PM Peak % Per Hour	Traffic Volume
12 A.M.	0.48	1
1:00	0.32	1
2:00	0.28	1
3:00	0.28	1
4:00	0.27	1
5:00	0.63	2
6:00	2.19	5
7:00	4.82	12
8:00	4.97	12
9:00	4.72	12
10:00	5.43	14
11:00	6.24	16
12:00	6.58	16
13:00	6.81	17
14:00	6.72	17
15:00	7.17	18
16:00	8.45	21
17:00	9.97	**25**
18:00	7.39	18
19:00	5.33	13
20:00	4.62	12
21:00	3.38	8
22:00	2.01	5
23:00	0.92	2
Total	100	250
Total Check		250

Peak Hour counts for Hester & 4th, Hester & 6th for 2016 and 2020

Project	Senior Design
Engineer:	Group 1
Date:	4/23/2016
Traffic studies	Capacity Analysis

4th and Hester				
NB	Left	Center	Right	Total
2016	9	48	9	66
2020	10	52	10	71
Apt. Traffic	12	60	28	100
Total	22	112	38	171
SB	Left	Center	Right	Total
2016	12	144	35	191
2020	13	156	38	207
Apt. Traffic	0	40	0	40
Total	13	196	38	247
WB	Left	Center	Right	Total
2016	8	62	14	84
2020	9	67	15	91
Dist. T	10	0	0	10
Total	19	67	15	101
EB	Left	Center	Right	Total
2016	45	135	11	191
2020	49	146	12	207
Apt. Traffic	0	0	12	12
Total	49	146	24	219

6th and Hester				
NB	Left	Center	Right	Total
2016	9	20	21	50
2020	10	22	23	54
Apt. Traffic	0	10	0	10
Total	10	32	23	64
SB	Left	Center	Right	Total
2016	19	71	70	160
2020	21	77	76	173
Apt. Traffic	11	26	26	63
Total	32	103	102	236
WB	Left	Center	Right	Total
2016	14	793	24	831
2020	15	858	26	900
Apt. Traffic	0	0	10	10
Total	15	858	36	910
EB	Left	Center	Right	Total
2016	30	1013	25	1068
2020	32	1097	27	1156
Apt. Traffic	12	0	0	12
Total	44	1097	27	1168

Sum of Critical Lanes	Traffic
2016	**382**
2020	**413**
2020 + Apt Traffic	**465**

Sum of Critical Lanes	Traffic
2016	**1228**
2020	**1329**
2020 + Apt Traffic	**1404**

Appendix B-1 Water Utilities Tables and Calculations

Water Demand:

Residential Flow = 451 residents x 150 gal/unit/day = 67,650 GPD

To Accommodate for Retail Flow = + 10,000 GPD = 80,000 GPD

Design Flow = 80,000 GPD = 3,333 GPH = 55.55 GPM

Design Flow + Fire Demand = 80,000 GPD + 8,640,000 GPD = 8,720,000 GPD = 363,333.33 GPH

Fire Demand:

8,640,000 GPD = 360,000 GPH = 6,000 GPM

Intersection	Max Day Usage (GPH)	Max Day Usage (GPM)	Average Usage (GPH)	Average Usage (GPM)	Peak Factor
3rd and Hester	223.80	3.73	95.01	1.58	2.4
3rd and Knoblock	281.60	4.69	103.46	1.72	2.7
4th and Hester	282.73	4.71	132.80	2.21	2.1
4th and Knoblock	421.65	7.03	100.39	1.67	4.2
4th and Ramsey	338.83	5.65	126.86	2.11	2.7
4th and Washington	1002.60	16.71	314.95	5.25	3.2
6th and Hester	289.47	4.82	128.28	2.14	2.3
6th and Knoblock	506.00	8.43	200.30	3.34	2.5
6th and Ramsey	418.00	6.97	165.24	2.75	2.5
6th and Washington	706.17	11.77	262.48	4.37	2.7
7th and Hester	268.02	4.47	53.88	0.90	5.0
7th and Knoblock	164.40	2.74	69.57	1.16	2.4
7th and Ramsey	132.00	2.20	48.89	0.81	2.7

Apartment Unit + Fire		363333.33	6055.56	2.5

JUNCTION TABLE		
Junction	**Elevation (ft)**	**Pressure (psi)**
1	889	78.3
2	896	75.3
3	903	72.2
4	915	67.0
5	884	80.4
6	878	83.0
8	870	86.4
9	886	79.5
10	896	75.2
11	896	75.2
12	914	67.4
13	892	76.9
14	883	80.8
15	903	72.1
16	899	73.9
17	906	70.9
18	917	66.2
19	884	80.4
20	892	76.9
21	903	72.1

22	914	67.4
23	883	80.8
24	899	73.9
25	906	70.9
26	917	66.2
27	883	80.8
29	884	80.4
30	892	76.9
31	883	80.8
32	906	70.9
33	917	66.2
34	914	67.4
35	903	72.1
37	899	73.9
44	880	82.0
45	886	79.7
46	878	82.8
47	904	71.7
48	896	75.1
49	894	76.1
50	881	81.6
Apartment Complex	892	76.9

HYDRANT TABLE

Hydrant	Elevation (ft)	Pressure (psi)
1	878	83.0
2	897	74.7
3	881	81.7
4	904	71.8
5	907	70.5
6	897	74.8
7	896	75.2
8	884	80.4
9	878	83.0

PIPE TABLE			
Pipe	Length (ft)	Diameter (in)	Material
1	355	8	PVC
4	677	8	DIP
5	393	8	PVC
6	366	8	PVC
7	381	8	PVC
9	373	8	ACP
10	386	8	CIP
11	425	8	ACP
12	386	10	CIP
13	397	10	CIP
14	2000	24	CIP
15	253	8	PVC
16	253	6	PVC
17	395	4	CIP
23	733	4	CIP
24	750	4	CIP

26	373	6	ACP
27	386	6	ACP
28	425	8	ACP
30	360	6	PVC
31	345	6	PVC
33	400	4	CIP
34	400	4	CIP
36	395	6	DIP
91	140	8	ACP
92	195	8	ACP
93	533	8	CIP
94	212	8	CIP
95	126	8	PVC
96	224	8	PVC
101	369	8	PVC
103	351	8	PVC
104	341	4	CIP
105	342	4	CIP
106	348	8	DIP
107	379	8	DIP

Appendix B-2 Wastewater Utilities Tables and Calculations

Design Flow = 0.365 cfs
Minimum Slope = 0.0043 ft/ft (Table 7.3)
n = 0.013

Hester to Ramsey:

Length = 356 feet
Change in Elevation = 903 ft – 892 ft = 11 ft
Slope = 0.031 ft/ft = 3.1 % (Design Slope)

Pipe Size:

Diameter = $\left(\frac{Qn}{0.465 \, x \, S^{0.5}}\right)^{0.375} = \left(\frac{0.365 \, x \, 0.013}{0.465 \, x \, 0.031^{0.5}}\right)^{0.375}$ = 4.13 inches

Half-Full:

Diameter = $1.73\left(\frac{Qn}{S^{0.5}}\right)^{8/3} = 1.73\left(\frac{0.365 \, x \, 0.013}{0.031^{0.5}}\right)^{8/3}$ = 5.35 inches

City Standard Minimum Pipe Size = 8 inches

Buried 3 feet below ground surface, PVC pipe

Pipe Capacity:

Flow = $\frac{0.463}{n} D^{8/3} S^{1/2} = \frac{0.463}{0.013}(8/12)^{8/3}(0.031)^{1/2}$ = 2.13 cfs

Hydraulic Elements Chart: (n = constant)

$\frac{q}{Q} = \frac{0.365}{2.13}$ = 0.1716 → $\frac{d}{D}$ = 0.28 → d = 0.28 x 8 = 2.24 inches

Velocity-Full = $\frac{Q}{A} = \frac{2.13}{\pi\left(\frac{4}{12}\right)^2}$ = 6.1 ft/s (Max Velocity) < 7 ft/s

From $\frac{d}{D}$ = 0.28: $\frac{v}{V}$ = 0.75 → v = 4.58 ft/s (Average Velocity) < 5 ft/s

<u>Ramsey to Washington:</u>

Length = 350 feet
Change in Elevation = 7 ft
Slope = 0.020 ft/ft = 2.0 % (Design Slope)

Pipe Size:

Diameter = $\left(\frac{Qn}{0.465\,x\,S^{0.5}}\right)^{0.375} = \left(\frac{0.365\,x\,0.013}{0.465\,x\,0.020^{0.5}}\right)^{0.375}$ = 4.48 inches

Half-Full:

Diameter = $1.73\left(\frac{Qn}{S^{0.5}}\right)^{8/3} = 1.73\left(\frac{0.365\,x\,0.013}{0.020^{0.5}}\right)^{8/3}$ = 5.81 inches

City Standard Minimum Pipe Size = <u>8 inches</u>

Buried 3 feet below ground surface, PVC pipe

Pipe Capacity:

Flow = $\frac{0.463}{n} D^{8/3} S^{1/2} = \frac{0.463}{0.013}(8/12)^{8/3}(0.020)^{1/2}$ = 1.71 cfs

Hydraulic Elements Chart: (n = constant)

$\frac{q}{Q} = \frac{0.365}{1.71}$ = 0.2135 → $\frac{d}{D}$ = 0.32 → d = 0.32 x 8 = 2.56 inches

Velocity-Full = $\frac{Q}{A} = \frac{1.71}{\pi\left(\frac{4}{12}\right)^2}$ = <u>4.9 ft/s (Max Velocity)</u> < 7 ft/s

From $\frac{d}{D}$ = 0.32: $\frac{v}{V}$ = 0.80 → v = <u>3.92 ft/s (Average Velocity)</u> < 5 ft/s

Appendix C - Hydrological Inputs and Calculations

Predevelopment Peak Runoff Inputs and Output

Control Point	Area	Weighted CN	Time Conc.	100 yr. Peak Flow	Peak Time
1	1.23	88	0.100 hr	12.70 cfs	11.93 hr
2	1.12	85	0.163 hr	10.11 cfs	11.97 hr
3	0.15	85	0.100 hr	1.47 cfs	11.93 hr

Post-Development Peak Runoff Inputs and Output

Control Point	Sub-Area	Area	Weighted CN	Time Conc.	100 yr. Peak Flow	Peak Time
1	A	0.55	98	0.100 hr	6.07	11.93
	B	0.6	98	0.100 hr	6.63	11.93
				TOTAL	12.67	11.93

*It was assumed area B exited the roof at the NE corner of the building and traveled west on 4th St. to the contol point.

Control Point	Sub-Area	Area	Weighted CN	Time Conc.	100 yr. Peak Flow (cfs)	Peak Time (hr)
2	D	0.55 Ac	98	0.100 hr	6.07	11.93
	E	0.30 Ac	98	0.100 hr	3.32	11.93
	F	0.40 Ac	98	0.100 hr	4.44	11.93
				TOTAL	13.79	

*It was modeled that area E and F flow into a reach that travels along the driveway in the SW corner of the property to the control point

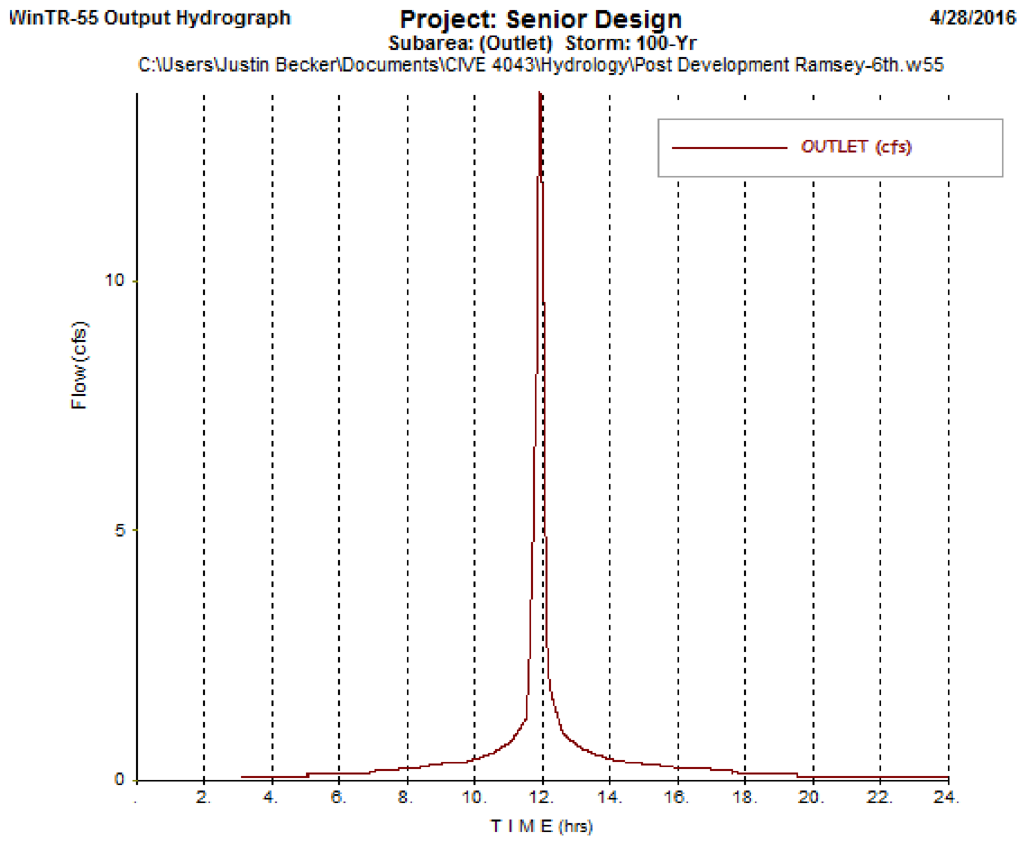
Control Point	Sub-Area	Area	Weighted CN	Time Conc.	100 yr. Peak Flow (cfs)	Peak Time (hr)
3	G	0.15 Ac	85	0.150 hr	1.47	11.93
				TOTAL	1.47	

Sizing of Detention Facility

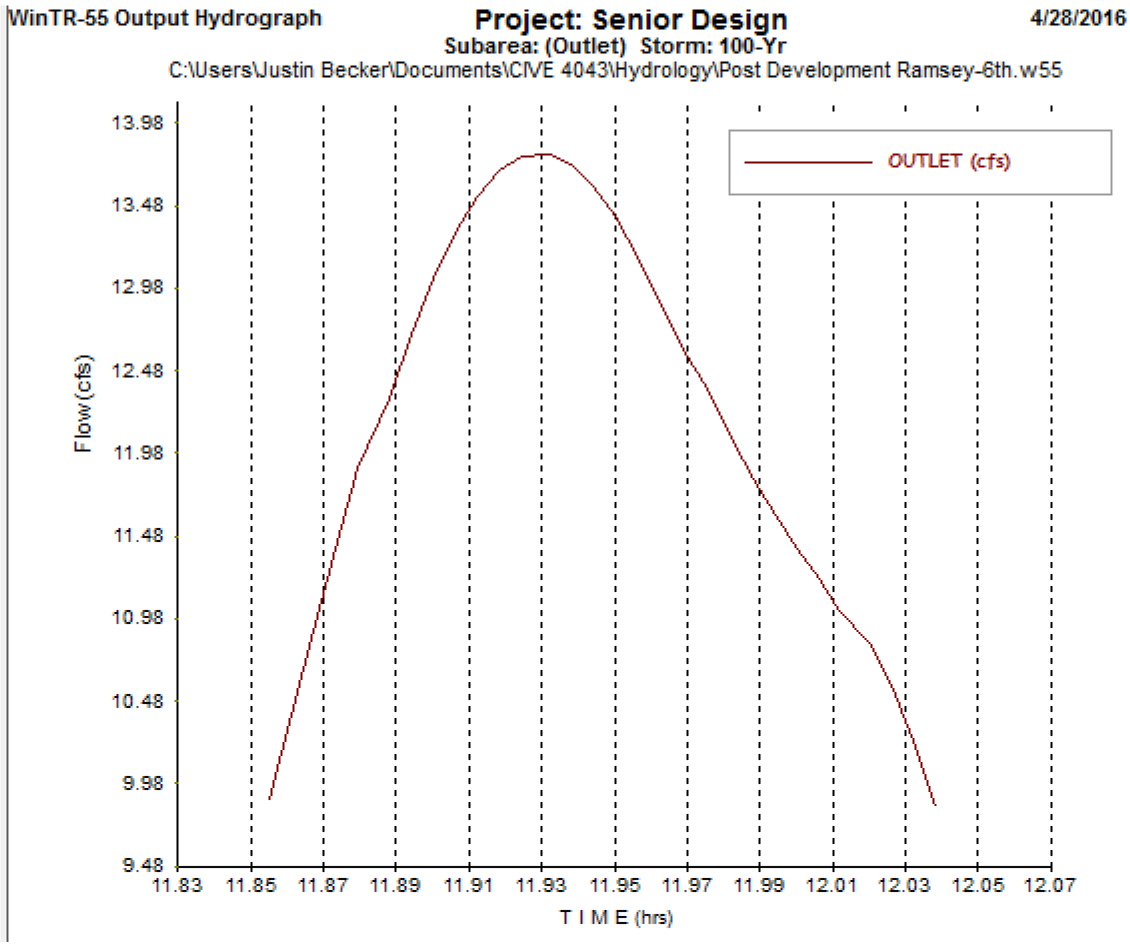
Case 1- Detaining all water to control point 2

In order to properly size the detention facility the team used the hydrograph generated from WinTR55 to determine the the volume of water exceeding the allowable flow of water.

Assuming the allowable outflow is 10 cfs, we determined the area under the hydrograph that was greater than 10 cfs. The hydrograph used in case 1 is below.



Using the zoom function in the program, we determined the time intervals where the hydrograph was above 10 cfs. The portion of the graph used for analysis is presented below.



The time interval used for analysis was t1-11.85hr and t2=12.04. We were then able to calculate the area under the curve by assuming the graph closely resembles that of a parabola.

Area=2/3bh

b=12.04 - 11.85 = 0.19 hr = 684 sec.

h=13.79 - 10.00 = 3.79 cfs

Area=(⅔)*(684s)*(3.79cfs) = 1728.25 cubic ft = 12,928 gal

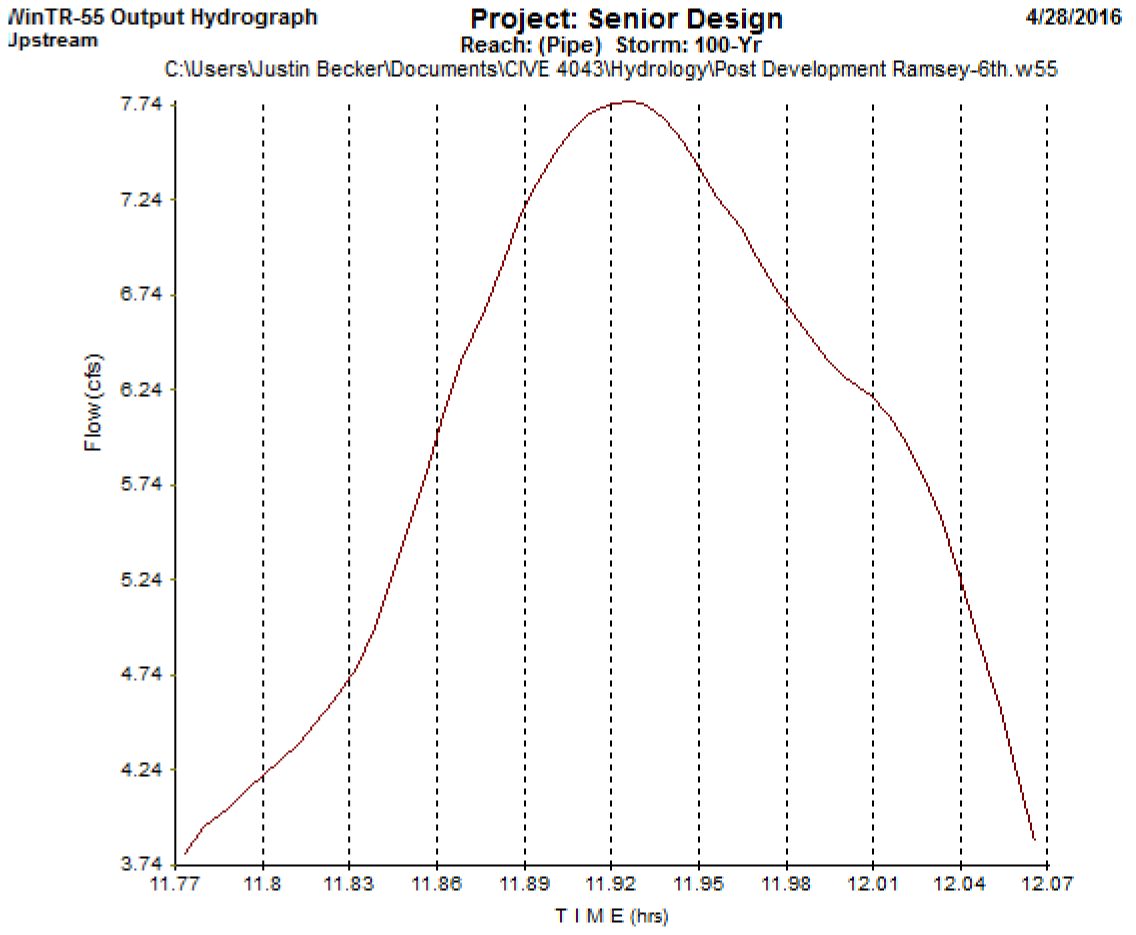
Volume of Detention = Area under Curve = 12,928 gal

Case 2-Detain only East 0.70 Ac.

The volume for case two was determined in the same manner with different constraints. The peak flow from this area is 7.76 cfs. To determine the maximum allowable flow we assumed the work case scenario. This assumes west 0.50 ac was at peak flow. The allowable flow would then equal the maximum peak flow from the west 0.5 ac subtracted from the maximum allowable flow from the control point.

Allowable flow from Area = Max allowed from control point - peak flow from 0.5 ac.
= 10.11 cfs - 6.07 cfs = 4.04 cfs

The same process was used as presented in Case 1 to determine the area under the hydrograph. A picture of the hydrograph used is presented below.



Assuming the area is equal to that of a parabola we can determine the volume of detention needed. By zooming in closer on each section one is able to obtain more accurate results.

Volume = Area under hydrograph

Area = ⅔ bh

b=12.06-11.795= 0.27 hr = 954 s.

h=7.76-4.04 = 3.72 cfs

Area = ⅔ * 954s * 3.72 cfs = 2365.92 cubic feet = 17,698 gal.

Volume = Area = <u>17,698 gal</u>

Appendix D-1 Structural Design Calculations

Project	Senior Design Parking Garage
Engineer:	Group 1
Date:	3/25/2016
Beam/Column Location:	Slab Design

Load Combinations		
Case	Equation	Load (psf)
1	1.4D	112.0
2a.	1.2D+1.6L+0.5(Lr)	170.0
2b.	1.2D+1.6L+0.5(S)	165.0
2c.	1.2D+1.6L+0.5(R)	186.0
3a.	1.2D+1.6(Lr)+0.5(L)	148.0
3b.	1.2D+1.6(Lr)+0.5(+W)	128.0
3c.	1.2D+1.6(Lr)+0.5(-W)	110.5
3d.	1.2D+1.6(S)+0.5(L)	132.0
3e.	1.2D+1.6(S)+0.5(+W)	112.0
3f.	1.2D+1.6(S)+0.5(-W)	94.5
3g.	1.2D+1.6(R)+0.5(L)	199.2
3h.	1.2D+1.6(R)+0.5(+W)	179.2
3i.	1.2D+1.6(R)+0.5(-W)	161.7
4a.	1.2D+1.0(+W)+0.5L+0.5(Lr)	126.0
4b.	1.2D+1.0(+W)+0.5L+0.5(S)	121.0
4c.	1.2D+1.0(+W)+0.5L+0.5(R)	142.0
4d.	1.2D+1.0(-W)+0.5L+0.5(Lr)	91.0
4e.	1.2D+1.0(-W)+0.5L+0.5(Lr)	86.0
4f.	1.2D+1.0(-W)+0.5L+0.5(Lr)	107.0
5	1.2D+1.0E+0.5L+0.2S	114.6
6a.	0.9D+1.0(-W)	37.0
6b.	0.9D+1.0(+W)	72.0
7	0.9D+1.0E	68.6
Max		199.2
Min		0

Loads (psf)	
Dead	80.00
Live	40
Wind (Pressure)	0
Wind (Suction)	-35
Roof Live	20
Snow	10
Seismic	-3.41
Rain	52

Max Load	
Tributary Width (ft)	1
Tributary Length (ft)	10
Distributed Load (klf)	0.20
Point Load (Kips)	1.99

Max Uplift	
Tributary Width (ft)	1
Tributary Length (ft)	10
Distributed Load (klf)	0.00
Point Load (Kips)	0.00

Beam Reactions for Simply Supported	
Max Moment, M_u, K-ft	2.5
Max Shear, V_u, Kips	1.0

Additional Notes and Assumptions
5 psf MEP load

Project	Senior Design Parking Garage
Engineer:	Group 1
Date:	3/25/2016
Beam/Column Location:	Slab Design

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Flexure Design		
Span Width, W	ft	10.00
28-Day Concrete Strength,f'c	ksi	5.00
Steel Yield Strength, fy	ksi	60.00
Minimum Thickess, h	in	6.00
Beam Width,b	in	12.00
Depth of Reinforcing, d=0.9h	in	3.00
Self Weight, Wsw	$\#/ft^2$	75.00
Max Moment, Mu	k-ft	2.49
Max Shear, Vu	k	1.00
Goal Seek	2.06	
Design Moment, Mn	k-ft	2.77
Area of Steel Required, As	in^2	0.14
Number of Bars Required		
Size 4	0.20	1.0
Size 5	0.31	1.0
Size 6	0.44	1.0
Size 7	0.60	1.0
Size 8	0.79	1.0
Size 9	1.00	1.0
Size 10	1.27	1.0
Size 11	1.56	1.0
Size 14	2.25	1.0
Size 18	4.00	1.0
Size of Bar Chosen	Size 4	
Total Area of Steel, As	in2	0.20
Moment Capacity, Mn	k-ft	2.88
Okay?	Yes	

Checks	
Strain, ε_t	0.0306
B1	0.8
c, in.	0.294118
a, in.	0.235294
dt, in.	4.00
Clear Space, in.	5.75
Reinforcing Ratio, ρ	0.005556
Minimum ρ_1	0.003536
Minimum ρ_2	0.003333
Okay?	Yes
Double Layer Check	
Clear Space, in.	
Okay?	
Triple Layer Check	
Clear Space, in.	
Okay?	
Assumptions and Notes	
Long term deflection multiplier,ζ, is 2.0 for loads sustained greater than 60 months.	

Project	Senior Design Parking Garage
Engineer:	Group 1
Date:	3/25/2016
Beam/Column Location:	Slab Design

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Deflection Check, D+L			Deflection Check, D		
Deflection Limit	in.	0.5	Deflection Limit	in.	0.33
Long Term Multipler	ζ	2	Long Term Multipler	ζ	2
Service Load	psf	120.00	Service Load	psf	80.00
Service Load	klf	0.12	Service Load	klf	0.08
Steel Modulus of Elasticity	ksi	29000	Steel Modulus of Elasticity	ksi	29000
Concrete Modulus of Elasticity	ksi	4074	Concrete Modulus of Elasticity	ksi	4074
Gross Moment of Inertia	$in.^4$	378	Gross Moment of Inertia	$in.^4$	378
Cracked Inertia	$in.^4$	12	Cracked Inertia	$in.^4$	12
Centroid	in.	3.00	Centroid	in.	3.00
Effective Inertia	$in.^4$	18736	Effective Inertia	$in.^4$	18736
Cracking Moment	k-ft.	5.6	Cracking Moment	k-ft.	5.6
Moment at Service Load	k-ft.	1.5	Moment at Service Load	k-ft.	1
Deflection at Service Load	in.	0.04	Deflection at Service Load	in.	0.02
OKAY?		Yes	OKAY?		Yes

Transverse Steel Requirements			Development and Splice Length		
Area of Steel	in^2	0.130	Development Length, ld	ft	1.00
Spacing	in	12.0	Splice Length, ls	ft	1.30
Use #4 bars in a 12"X12" Grid Mat					

Project	Senior Design Parking Garage
Engineer:	Group 1
Date:	3/25/2016
Beam/Column Location:	Slab Design

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Shear		
Max Shear at d from Support, Vu	kips	0.95
Beam Width, b	inches	12.00
Depth of Reinforcement, d	inches	3.00
28-Day Concrete Strength,f'c	ksi	5.00
Steel Yield Strength, fy	ksi	60.00
Concrete Shear Strength, Vc	ksi	5.1
Factored Vc	ksi	3.82
Required Steel Shear Strength, ΦVs	ksi	0.00
Adequacy Check	ksi	15.27
Distance of Max Reinf. From Support	ft.	-14.17
Distance of Max Reinf. From Midspan	ft.	19.17
Size of Chosen Stirrup	Size 4	
Spacing Requirement	inches	-9.401
Max Permissible Spacing in Critical Sec.	inches	1.5
Shear Strength Provided in Critical Sec.	kips	0.82
Number of Stirrups in Critical Sec.	N_1	21
Distance of Min. Reinforcement	ft.	-4.58
Length of Min Reinforcement	ft.	9.58
Length of No Reinforcement	ft.	9.58
Number of Stirrups in Min. Reinforcing	N_2	89
Nominal Shear Strength for Critical Sec., ΦVn	kips	0.82
Nominal Shear Strength for Min. Section, ΦVn	kips	28.91
OKAY?	Yes	

Equations	
Distance	Vu
0.00	1.00
0.25	0.95
1.00	0.80
2.00	0.60
3.00	0.40
4.00	0.20
5.00	0.00
5.00	0.00
5.00	0.00

Distance	ΦVn
0.00	1.91
5.00	1.91
5.00	1.91
5.00	1.91
5.00	1.91
5.00	1.91

No Reinforcement Necessary in Slab

Project	Senior Design Parking Garage
Engineer:	Group 1
Date:	3/25/2016
Beam/Column Location:	Slab Design

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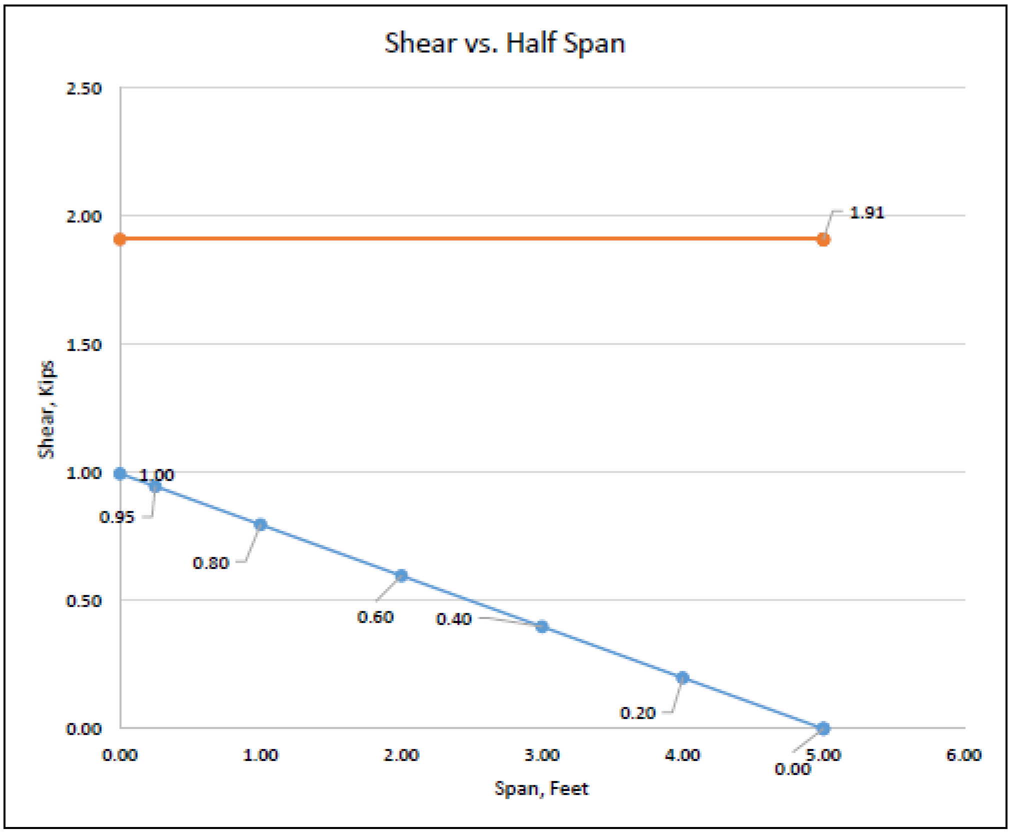

Project	Senior Design Parking Garage
Engineer:	Group 1
Date:	3/25/2016
Beam/Column Location:	Roof Typical #1 Beam

Load Combinations		
Case	Equation	Load (psf)
1	1.4D	128.3
2a.	1.2D+1.6L+0.5(Lr)	184.0
2b.	1.2D+1.6L+0.5(S)	179.0
2c.	1.2D+1.6L+0.5(R)	200.0
3a.	1.2D+1.6(Lr)+0.5(L)	162.0
3b.	1.2D+1.6(Lr)+0.5(+W)	142.0
3c.	1.2D+1.6(Lr)+0.5(-W)	124.5
3d.	1.2D+1.6(S)+0.5(L)	146.0
3e.	1.2D+1.6(S)+0.5(+W)	126.0
3f.	1.2D+1.6(S)+0.5(-W)	108.5
3g.	1.2D+1.6(R)+0.5(L)	213.2
3h.	1.2D+1.6(R)+0.5(+W)	193.2
3i.	1.2D+1.6(R)+0.5(-W)	175.7
4a.	1.2D+1.0(+W)+0.5L+0.5(Lr)	140.0
4b.	1.2D+1.0(+W)+0.5L+0.5(S)	135.0
4c.	1.2D+1.0(+W)+0.5L+0.5(R)	156.0
4d.	1.2D+1.0(-W)+0.5L+0.5(Lr)	105.0
4e.	1.2D+1.0(-W)+0.5L+0.5(Lr)	100.0
4f.	1.2D+1.0(-W)+0.5L+0.5(Lr)	121.0
5	1.2D+1.0E+0.5L+0.2S	128.1
6a.	0.9D+1.0(-W)	47.5
6b.	0.9D+1.0(+W)	82.5
7	0.9D+1.0E	78.6
Max		213.2
Min		0

Loads (psf)	
Dead	91.67
Live	40
Wind (Pressure)	0
Wind (Suction)	-35
Roof Live	20
Snow	10
Seismic	-3.91
Rain	52

Max Load	
Tributary Width (ft)	10
Tributary Length (ft)	18
Distributed Load (klf)	2.13
Point Load (Kips)	38.38

Max Uplift	
Tributary Width (ft)	10
Tributary Length (ft)	18
Distributed Load (klf)	0.00
Point Load (Kips)	0.00

Beam Reactions for Simply Supported	
Max Moment, M_u, K-ft	86.3
Max Shear, V_u, Kips	19.2

Additional Notes and Assumptions
6" slab and 5 psf MEP load

Project	Senior Design Parking Garage
Engineer:	Group 1
Date:	3/25/2016
Beam/Column Location:	Roof Typical #1 Beam

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Flexure Design		
Span Width, W	ft	18.00
28-Day Concrete Strength,f'c	ksi	5.00
Steel Yield Strength, fy	ksi	60.00
Thickess, h	in	14.00
Beam Width,b	in	8.00
Depth of Reinforcing, d=0.9h	in	12.00
Self Weight, Wsw	$\#/ft^2$	11.67
Max Moment, Mu	k-ft	86.35
Max Shear, Vu	k	19.19
Goal Seek	95.93917454	
Design Moment, Mn	k-ft	95.94
Area of Steel Required, As	in^2	1.85
Number of Bars Required		
Size 4	0.20	10.0
Size 5	0.31	6.0
Size 6	0.44	5.0
Size 7	0.60	4.0
Size 8	0.79	3.0
Size 9	1.00	2.0
Size 10	1.27	2.0
Size 11	1.56	2.0
Size 14	2.25	1.0
Size 18	4.00	1.0
Size of Bar Chosen	Size 10	
Total Area of Steel, As	in^2	2.54
Moment Capacity, ΦMn	k-ft	111.54
Okay?	Yes	

Checks	
Strain, ε_t	0.00589
B1	0.8
c, in.	5.602941
a, in.	4.482353
dt, in.	12.00
Clear Space, in.	0.46
Reinforcing Ratio, ρ	0.026458
Minimum ρ_1	0.003536
Minimum ρ_2	0.003333
Okay?	No
Double Layer Check	
Clear Space, in.	1.73
Okay?	Yes
Triple Layer Check	
Clear Space, in.	
Okay?	
Assumptions and Notes	
Long term deflection multiplier,ζ , is 2.0 for loads sustained greater than 60 months.	

Project	Senior Design Parking Garage
Engineer:	Group 1
Date:	3/25/2016
Beam/Column Location:	Roof Typical #1 Beam

Deflection Check, D+L			Deflection Check, D		
Deflection Limit	in.	0.9	Deflection Limit	in.	0.6
Long Term Multipler	ζ	2	Long Term Multipler	ζ	2
Service Load	psf	131.67	Service Load	psf	91.67
Service Load	klf	1.31666667	Service Load	klf	0.916667
Steel Modulus of Elasticity	ksi	29000	Steel Modulus of Elasticity	ksi	29000
Concrete Modulus of Elasticity	ksi	4074	Concrete Modulus of Elasticity	ksi	4074
Gross Moment of Inertia	$in.^4$	3750	Gross Moment of Inertia	$in.^4$	3750
Cracked Inertia	$in.^4$	1736	Cracked Inertia	$in.^4$	1736
Centroid	in.	7.61	Centroid	in.	7.61
Effective Inertia	$in.^4$	1873	Effective Inertia	$in.^4$	1873
Cracking Moment	k-ft.	21.8	Cracking Moment	k-ft.	21.8
Moment at Service Load	k-ft.	53.325	Moment at Service Load	k-ft.	37.125
Deflection at Service Load	in.	0.81	Deflection at Service Load	in.	0.57
OKAY?		Yes	OKAY?		Yes

Development and Splice Length		
Development Length, ld	ft	4.94
Splice Length, ls	ft	6.43

Project	Senior Design Parking Garage
Engineer:	Group 1
Date:	3/25/2016
Beam/Column Location:	Roof Typical #1 Beam

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Shear		
Max Shear at d from Support, Vu	kips	17.06
Beam Width, b	inches	8.00
Depth of Reinforcement, d	inches	12.00
28-Day Concrete Strength,f'c	ksi	5.00
Steel Yield Strength, fy	ksi	60.00
Concrete Shear Strength, Vc	ksi	13.6
Factored Vc	ksi	10.18
Required Steel Shear Strength, ΦVs	ksi	6.87
Adequacy Check	ksi	40.73
Distance of Max Reinf. From Support	ft.	4.22
Distance of Max Reinf. From Midspan	ft.	4.78
Size of Chosen Stirrup	Size 3	
Spacing Requirement	inches	8.64
Max Permissible Spacing in Critical Sec.	inches	6.00
Shear Strength Provided in Critical Sec.	kips	20.08
Number of Stirrups in Critical Sec.	N_1	18
Nominal Shear Strength for Critical Sec., ΦVn	kips	20.08
OKAY?	Yes	

Equations	
Distance	Vu
0.00	19.19
1.00	17.06
1.80	15.35
3.60	11.51
5.40	7.68
7.20	3.84
9.00	0.00

Distance	ΦVn
0.00	20.08
9.00	20.08

Actual Spacing, in.
6.00

Project	Senior Design Parking Garage
Engineer:	Group 1
Date:	3/25/2016
Beam/Column Location:	Roof Typical #1 Beam

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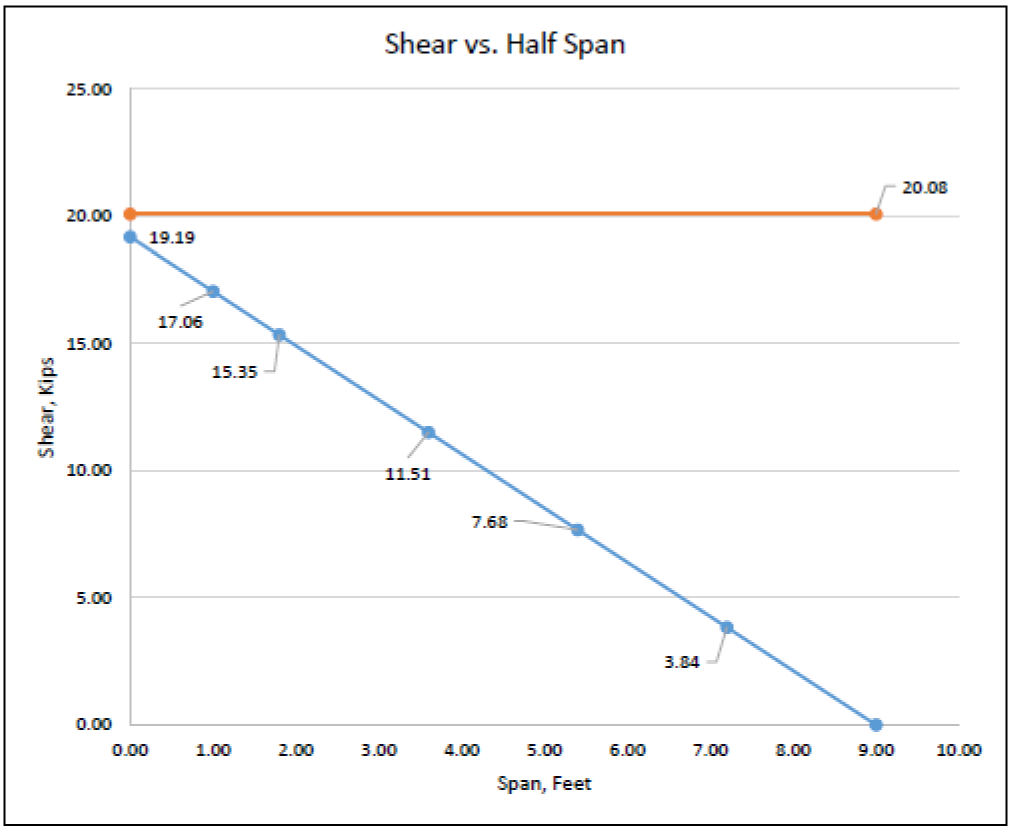

Project	Senior Design Parking Garage
Engineer:	Group 1
Date:	3/25/2016
Beam/Column Location:	Roof Typical #1 Beam

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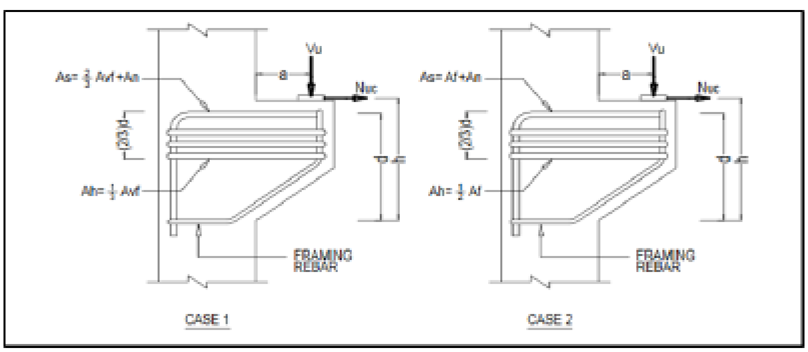
Corbel Connection Design

Max Shear, Vu	kips	19.2
Corbel Width, b	in.	10
Corbel Height, h	in.	10
Effective Corbel Height, d	in.	8
28-Day Concrete Strength,f'c	ksi	5.00
Steel Yield Strength, fy	ksi	60.00
Size Okay?	Yes	
Eccentricity	in.	6
Coefficient of Friction	μ	0.6
Shear Friction Steel, Avf	$in.^2$	0.71
Ultimate Horizontal Force, Nu	kips	3.84
Tension Steel, An	$in.^2$	0.09
Max Moment on Corbel, Mu	kip-ft	12.792
Flexure Steel, Af	$in.^2$	0.50
Angle btwn Comp and Tens, β	°	48.58
Depth of Whitney Stress Block, a	in.	1.21
Distribution Case?	Case 2	
Required Area of Steel, As	$in.^2$	0.59
Supported Beam Width	in.	18.00
Corbel Depth	in.	21.00
Area of Horizontal Ties	$in.^2$	0.25
Depth of Ties from Tension Steel	in.	5.33

Bar Sizes

Tension Reinforcement		
Size 4	0.20	3
Size 5	0.31	2
Size 6	0.44	2
Size 7	0.60	1
Size 8	0.79	1
Size 9	1.00	1
Size 10	1.27	1
Size 11	1.56	1
Size 14	2.25	1
Size 18	4.00	1
Chosen Size		Size 5
Horizontal Reinforcement		
Size 4	0.20	2
Size 5	0.31	1
Size 6	0.44	1
Size 7	0.60	1
Size 8	0.79	1
Size 9	1.00	1
Chosen Size		Size 4
Reinf. Ratio, ρ		0.008
Min. Ah		0.25
Okay?		Yes



Project	Senior Design Parking Garage
Engineer:	Group 1
Date:	3/25/2016
Beam/Column Location:	Roof Typical #2 Beam

Load Combinations

Case	Equation	Load (psf)
1	1.4D	138.3
2a.	1.2D+1.6L+0.5(Lr)	192.5
2b.	1.2D+1.6L+0.5(S)	187.5
2c.	1.2D+1.6L+0.5(R)	208.5
3a.	1.2D+1.6(Lr)+0.5(L)	170.5
3b.	1.2D+1.6(Lr)+0.5(+W)	150.5
3c.	1.2D+1.6(Lr)+0.5(-W)	133.0
3d.	1.2D+1.6(S)+0.5(L)	154.5
3e.	1.2D+1.6(S)+0.5(+W)	134.5
3f.	1.2D+1.6(S)+0.5(-W)	117.0
3g.	1.2D+1.6(R)+0.5(L)	221.7
3h.	1.2D+1.6(R)+0.5(+W)	201.7
3i.	1.2D+1.6(R)+0.5(-W)	184.2
4a.	1.2D+1.0(+W)+0.5L+0.5(Lr)	148.5
4b.	1.2D+1.0(+W)+0.5L+0.5(S)	143.5
4c.	1.2D+1.0(+W)+0.5L+0.5(R)	164.5
4d.	1.2D+1.0(-W)+0.5L+0.5(Lr)	113.5
4e.	1.2D+1.0(-W)+0.5L+0.5(Lr)	108.5
4f.	1.2D+1.0(-W)+0.5L+0.5(Lr)	129.5
5	1.2D+1.0E+0.5L+0.2S	136.3
6a.	0.9D+1.0(-W)	53.9
6b.	0.9D+1.0(+W)	88.9
7	0.9D+1.0E	84.7
Max		221.7
Min		0

Loads (psf)

Dead	98.75
Live	40
Wind (Pressure)	0
Wind (Suction)	-35
Roof Live	20
Snow	10
Seismic	-4.21
Rain	52

Max Load

Tributary Width (ft)	10
Tributary Length (ft)	24
Distributed Load (klf)	2.22
Point Load (Kips)	53.21

Max Uplift

Tributary Width (ft)	10
Tributary Length (ft)	24
Distributed Load (klf)	0.00
Point Load (Kips)	0.00

Beam Reactions for Simply Supported

Max Moment, M_u, K-ft	159.6
Max Shear, V_u, Kips	26.6

Additional Notes and Assumptions

6" slab and 5 psf MEP load

Project	Senior Design Parking Garage
Engineer:	Group 1
Date:	3/25/2016
Beam/Column Location:	Roof Typical #2 Beam

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Flexure Design

Span Width, W	ft	24.00
28-Day Concrete Strength,f'c	ksi	5.00
Steel Yield Strength, fy	ksi	60.00
Thickess, h	in	18.00
Beam Width,b	in	10.00
Depth of Reinforcing, d=0.9h	in	16.00
Self Weight, Wsw	$\#/ft^2$	18.75
Max Moment, Mu	k-ft	159.62
Max Shear, Vu	k	26.60
Goal Seek	177.3596323	
Design Moment, Mn	k-ft	177.36
Area of Steel Required, As	in^2	2.49
Number of Bars Required		
Size 4	0.20	13.0
Size 5	0.31	9.0
Size 6	0.44	6.0
Size 7	0.60	5.0
Size 8	0.79	4.0
Size 9	1.00	4.0
Size 10	1.27	2.0
Size 11	1.56	2.0
Size 14	2.25	2.0
Size 18	4.00	1.0
Size of Bar Chosen	Size 9	
Total Area of Steel, As	in^2	4.00
Moment Capacity, ΦMn	k-ft	237.18
Okay?	Yes	

Checks

Strain, ε_t	0.006375
B1	0.8
c, in.	7.058824
a, in.	5.647059
dt, in.	16.00
Clear Space, in.	0.162667
Reinforcing Ratio, ρ	0.025
Minimum ρ_1	0.003536
Minimum ρ_2	0.003333
Okay?	No
Double Layer Check	
Clear Space, in.	1.37
Okay?	Yes
Triple Layer Check	
Clear Space, in.	
Okay?	
Assumptions and Notes	
Long term deflection multiplier,ζ , is 2.0 for loads sustained greater than 60 months.	

Project	Senior Design Parking Garage
Engineer:	Group 1
Date:	3/25/2016
Beam/Column Location:	Roof Typical #2 Beam

Deflection Check, D+L			Deflection Check, D		
Deflection Limit	in.	1.2	Deflection Limit	in.	0.8
Long Term Multipler	ζ	2	Long Term Multipler	ζ	2
Service Load	psf	138.75	Service Load	psf	98.75
Service Load	klf	1.3875	Service Load	klf	0.9875
Steel Modulus of Elasticity	ksi	29000	Steel Modulus of Elasticity	ksi	29000
Concrete Modulus of Elasticity	ksi	4074	Concrete Modulus of Elasticity	ksi	4074
Gross Moment of Inertia	$in.^4$	10144	Gross Moment of Inertia	$in.^4$	10144
Cracked Inertia	$in.^4$	4811	Cracked Inertia	$in.^4$	4811
Centroid	in.	9.84	Centroid	in.	9.84
Effective Inertia	$in.^4$	5318	Effective Inertia	$in.^4$	5318
Cracking Moment	k-ft.	45.6	Cracking Moment	k-ft.	45.6
Moment at Service Load	k-ft.	99.9	Moment at Service Load	k-ft.	71.1
Deflection at Service Load	in.	0.96	Deflection at Service Load	in.	0.68
OKAY?		Yes	OKAY?		Yes

Development and Splice Length		
Development Length, ld	ft	4.92
Splice Length, ls	ft	6.39

Project	Senior Design Parking Garage
Engineer:	Group 1
Date:	3/25/2016
Beam/Column Location:	Roof Typical #2 Beam

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Shear		
Max Shear at d from Support, Vu	kips	23.65
Beam Width, b	inches	10.00
Depth of Reinforcement, d	inches	16.00
28-Day Concrete Strength,f'c	ksi	5.00
Steel Yield Strength, fy	ksi	60.00
Concrete Shear Strength, Vc	ksi	22.6
Factored Vc	ksi	16.97
Required Steel Shear Strength, ΦVs	ksi	6.68
Adequacy Check	ksi	67.88
Distance of Max Reinf. From Support	ft.	4.35
Distance of Max Reinf. From Midspan	ft.	7.65
Size of Chosen Stirrup	Size 3	
Spacing Requirement	inches	11.86
Max Permissible Spacing in Critical Sec.	inches	8.00
Shear Strength Provided in Critical Sec.	kips	26.87
Number of Stirrups in Critical Sec.	N_1	18
Nominal Shear Strength for Critical Sec., ΦVn	kips	26.87
OKAY?	Yes	

Equations	
Distance	Vu
0.00	26.60
1.33	23.65
2.40	21.28
4.80	15.96
7.20	10.64
9.60	5.32
12.00	0.00

Distance	ΦVn
0.00	26.87
12.00	26.87

Actual Spacing, in.
8.00

Project	Senior Design Parking Garage
Engineer:	Group 1
Date:	3/25/2016
Beam/Column Location:	Roof Typical #2 Beam

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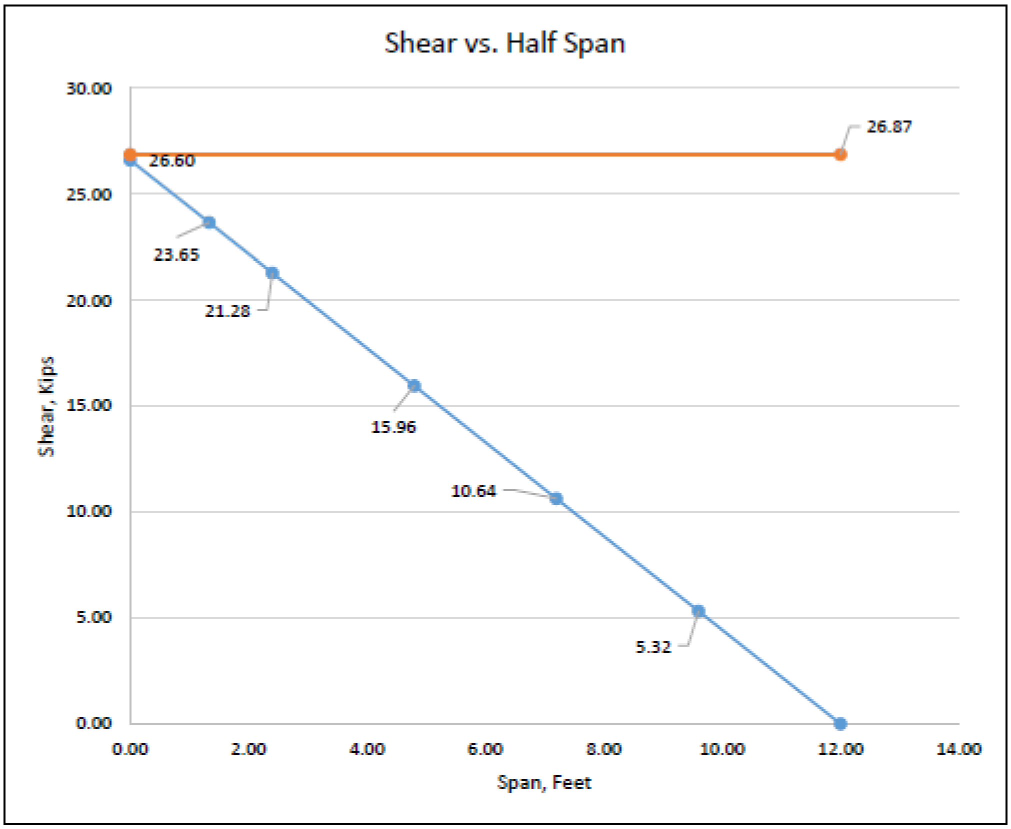
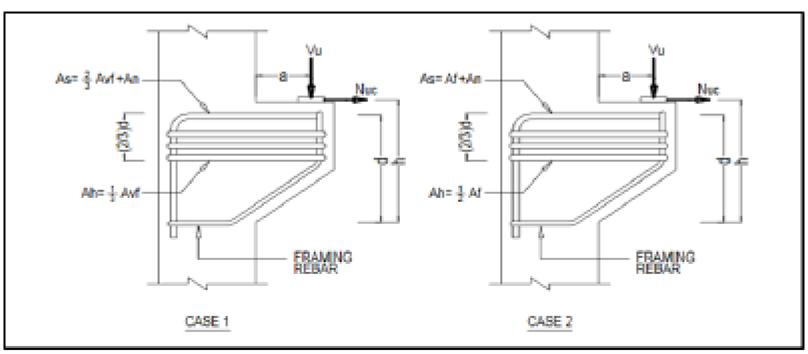


Project	Senior Design Parking Garage
Engineer:	Group 1
Date:	3/25/2016
Beam/Column Location:	Roof Typical #2 Beam

Page:	6/6

Corbel Connection Design		
Max Shear, Vu	kips	26.6
Corbel Width, b	in.	10
Corbel Height, h	in.	10
Effective Corbel Height, d	in.	8
28-Day Concrete Strength,f'c	ksi	5.00
Steel Yield Strength, fy	ksi	60.00
Size Okay?	Yes	
Eccentricity	in.	6
Coefficient of Friction	μ	0.6
Shear Friction Steel, Avf	$in.^2$	0.99
Ultimate Horizontal Force, Nu	kips	5.32
Tension Steel, An	$in.^2$	0.12
Max Moment on Corbel, Mu	kip-ft	17.736
Flexure Steel, Af	$in.^2$	0.70
Angle btwn Comp and Tens, β	°	48.58
Depth of Whitney Stress Block, a	in.	1.68
Distribution Case?	Case 2	
Required Area of Steel, As	$in.^2$	0.81
Supported Beam Width	in.	18.00
Corbel Depth	in.	21.00
Area of Horizontal Ties	$in.^2$	0.35
Depth of Ties from Tension Steel	in.	5.33

Bar Sizes		
Tension Reinforcement		
Size 4	0.20	5
Size 5	0.31	3
Size 6	0.44	2
Size 7	0.60	2
Size 8	0.79	2
Size 9	1.00	1
Size 10	1.27	1
Size 11	1.56	1
Size 14	2.25	1
Size 18	4.00	1
Chosen Size		Size 5
Horizontal Reinforcement		
Size 4	0.20	2
Size 5	0.31	2
Size 6	0.44	1
Size 7	0.60	1
Size 8	0.79	1
Size 9	1.00	1
Chosen Size		Size 4
Reinf. Ratio, ρ		0.012
Min. Ah		0.35
Okay?		Yes



Project	Senior Design Parking Garage
Engineer:	Group 1
Date:	3/25/2016
Beam/Column Location:	Roof Girder

Load Combinations

Case	Equation	Load (psf)
1	1.4D	165.3
2a.	1.2D+1.6L+0.5(Lr)	215.7
2b.	1.2D+1.6L+0.5(S)	210.7
2c.	1.2D+1.6L+0.5(R)	231.7
3a.	1.2D+1.6(Lr)+0.5(L)	193.7
3b.	1.2D+1.6(Lr)+0.5(+W)	173.7
3c.	1.2D+1.6(Lr)+0.5(-W)	156.2
3d.	1.2D+1.6(S)+0.5(L)	177.7
3e.	1.2D+1.6(S)+0.5(+W)	157.7
3f.	1.2D+1.6(S)+0.5(-W)	140.2
3g.	1.2D+1.6(R)+0.5(L)	244.9
3h.	1.2D+1.6(R)+0.5(+W)	224.9
3i.	1.2D+1.6(R)+0.5(-W)	207.4
4a.	1.2D+1.0(+W)+0.5L+0.5(Lr)	171.7
4b.	1.2D+1.0(+W)+0.5L+0.5(S)	166.7
4c.	1.2D+1.0(+W)+0.5L+0.5(R)	187.7
4d.	1.2D+1.0(-W)+0.5L+0.5(Lr)	136.7
4e.	1.2D+1.0(-W)+0.5L+0.5(Lr)	131.7
4f.	1.2D+1.0(-W)+0.5L+0.5(Lr)	152.7
5	1.2D+1.0E+0.5L+0.2S	158.6
6a.	0.9D+1.0(-W)	71.3
6b.	0.9D+1.0(+W)	106.3
7	0.9D+1.0E	101.2
Max		244.9
Min		0

Loads (psf)

Dead	118.06
Live	40
Wind (Pressure)	0
Wind (Suction)	-35
Roof Live	20
Snow	10
Seismic	-5.03
Rain	52

Max Load

Tributary Width (ft)	21
Tributary Length (ft)	32
Distributed Load (klf)	5.14
Point Load (Kips)	164.55

Max Uplift

Tributary Width (ft)	21
Tributary Length (ft)	32
Distributed Load (klf)	0.00
Point Load (Kips)	0.00

Beam Reactions for Simply Supported

Max Moment, M_u, K-ft	658.2
Max Shear, V_u, Kips	82.3

Additional Notes and Assumptions
6" slab and 5 psf MEP load

Project	Senior Design Parking Garage
Engineer:	Group 1
Date:	3/25/2016
Beam/Column Location:	Roof Girder

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Flexure Design		
Span Width, W	ft	32.00
28-Day Concrete Strength,f'c	ksi	5.00
Steel Yield Strength, fy	ksi	60.00
Minimum Thickess, h	in	26.00
Beam Width,b	in	14.00
Depth of Reinforcing, d=0.9h	in	24.00
Self Weight, Wsw	$\#/ft^2$	18.06
Max Moment, Mu	k-ft	658.20
Max Shear, Vu	k	82.28
Goal Seek	731.3399999	
Design Moment, Mn	k-ft	731.34
Area of Steel Required, As	in^2	7.18
Number of Bars Required		
Size 4	0.20	36.0
Size 5	0.31	24.0
Size 6	0.44	17.0
Size 7	0.60	12.0
Size 8	0.79	10.0
Size 9	1.00	9.0
Size 10	1.27	8.0
Size 11	1.56	5.0
Size 14	2.25	4.0
Size 18	4.00	2.0
Size of Bar Chosen	Size 10	
Total Area of Steel, As	in2	10.16
Moment Capacity, ΦMn	k-ft	863.0707
Okay?	Yes	

Checks	
Strain, ε_t	0.005388
B1	0.8
c, in.	12.80672
a, in.	10.24538
dt, in.	24.00
Clear Space, in.	-0.16571
Reinforcing Ratio, ρ	0.030238
Minimum ρ_1	0.003536
Minimum ρ_2	0.003333
Okay?	No
Double Layer Check	
Clear Space, in.	1.306667
Okay?	Yes
Triple Layer Check	
Clear Space, in.	2.595
Okay?	Yes
Assumptions and Notes	
Long term deflection multiplier,ζ, is 2.0 for loads sustained greater than 60 months.	

Project	Senior Design Parking Garage
Engineer:	Group 1
Date:	3/25/2016
Beam/Column Location:	Roof Girder

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Deflection Check, D+L			Deflection Check, D		
Deflection Limit	in.	1.6	Deflection Limit	in.	1.066667
Long Term Multipler	ζ	2	Long Term Multipler	ζ	2
Service Load	psf	158.06	Service Load	psf	118.06
Service Load	klf	3.31916667	Service Load	klf	2.479167
Steel Modulus of Elasticity	ksi	29000	Steel Modulus of Elasticity	ksi	29000
Concrete Modulus of Elasticity	ksi	4074	Concrete Modulus of Elasticity	ksi	4074
Gross Moment of Inertia	$in.^4$	45402	Gross Moment of Inertia	$in.^4$	45402
Cracked Inertia	$in.^4$	26436	Cracked Inertia	$in.^4$	26436
Centroid	in.	14.60	Centroid	in.	14.60
Effective Inertia	$in.^4$	27078	Effective Inertia	$in.^4$	27078
Cracking Moment	k-ft.	137.4	Cracking Moment	k-ft.	137.4
Moment at Service Load	k-ft.	424.853333	Moment at Service Load	k-ft.	317.3333
Deflection at Service Load	in.	1.42	Deflection at Service Load	in.	1.06
OKAY?		Yes	OKAY?		Yes

Development and Splice Length		
Development Length, ld	ft	6.55
Splice Length, ls	ft	8.51

Project		Senior Design Parking Garage
Engineer:		Group 1
Date:		3/25/2016
Beam/Column Location:		Roof Girder

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Shear		
Max Shear at d from Support, Vu	kips	71.99
Beam Width, b	inches	14.00
Depth of Reinforcement, d	inches	24.00
28-Day Concrete Strength,f'c	ksi	5.00
Steel Yield Strength, fy	ksi	60.00
Concrete Shear Strength, Vc	ksi	47.5
Factored Vc	ksi	35.64
Required Steel Shear Strength, ΦVs	ksi	36.35
Adequacy Check	ksi	142.55
Distance of Max Reinf. From Support	ft.	9.07
Distance of Max Reinf. From Midspan	ft.	6.93
Size of Chosen Stirrup	Size 4	
Spacing Requirement	inches	5.942
Max Permissible Spacing in Critical Sec.	inches	12
Shear Strength Provided in Critical Sec.	kips	78.84
Number of Stirrups in Critical Sec.	N_1	38
Nominal Shear Strength for Critical Sec., ΦVn	kips	78.84
OKAY?	Yes	

Equations	
Distance	Vu
0.00	82.28
2.00	71.99
3.20	65.82
6.40	49.37
9.60	32.91
12.80	16.46
16.00	0.00

Distance	ΦVn
0.00	78.84
16.00	78.84

Actual Spacing, in.
5.00

Project	Senior Design Parking Garage
Engineer:	Group 1
Date:	3/25/2016
Beam/Column Location:	Roof Girder

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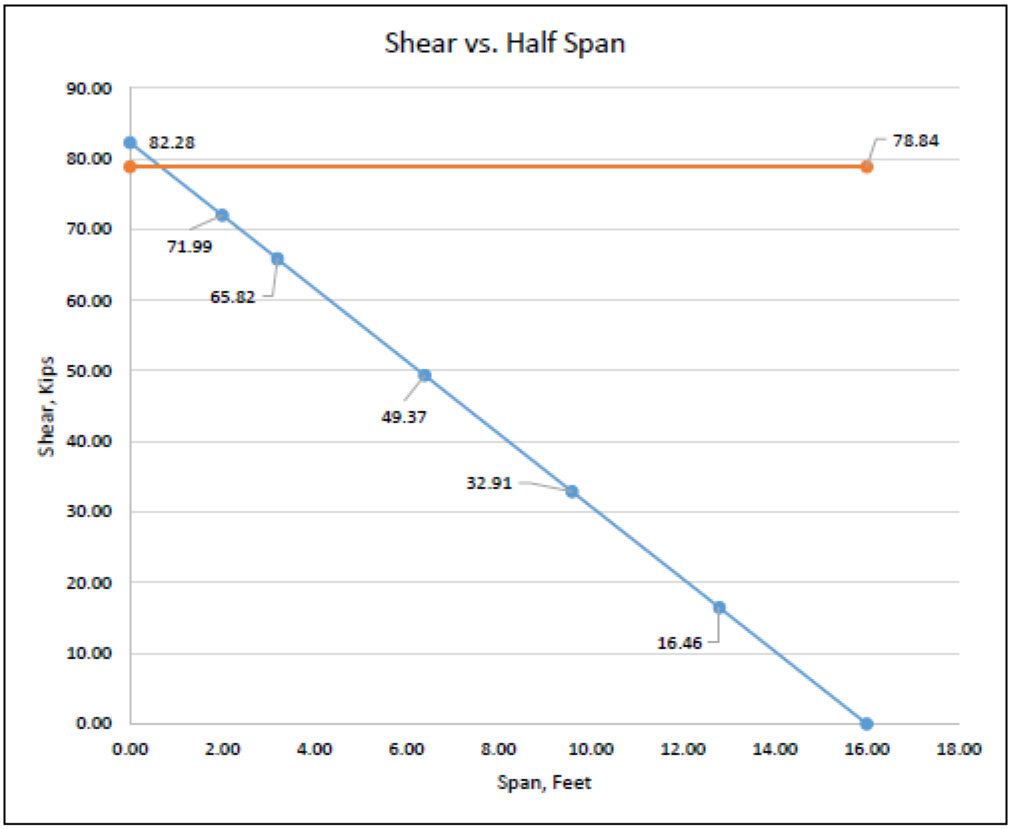
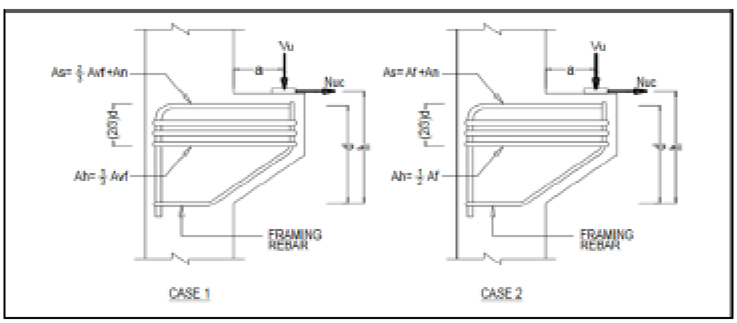


Project	Senior Design Parking Garage
Engineer:	Group 1
Date:	3/25/2016
Beam/Column Location:	Roof Girder

Corbel Connection Design		
Max Shear, Vu	kips	82.3
Corbel Width, b	in.	10
Corbel Height, h	in.	15
Effective Corbel Height, d	in.	13
28-Day Concrete Strength,f'c	ksi	5.00
Steel Yield Strength, fy	ksi	60.00
Size Okay?	Yes	
Eccentricity	in.	6
Coefficient of Friction	μ	0.6
Shear Friction Steel, Avf	$in.^2$	3.05
Ultimate Horizontal Force, Nu	kips	16.46
Tension Steel, An	$in.^2$	0.37
Max Moment on Corbel, Mu	kip-ft	54.85013
Flexure Steel, Af	$in.^2$	1.32
Angle btwn Comp and Tens, β	°	61.50
Depth of Whitney Stress Block, a	in.	4.44
Distribution Case?	Case 1	
Required Area of Steel, As	$in.^2$	2.40
Supported Beam Width	in.	18.00
Corbel Depth	in.	21.00
Area of Horizontal Ties	$in.^2$	1.02
Depth of Ties from Tension Steel	in.	8.67

Bar Sizes		
Tension Reinforcement		
Size 4	0.20	12
Size 5	0.31	8
Size 6	0.44	6
Size 7	0.60	4
Size 8	0.79	4
Size 9	1.00	3
Size 10	1.27	2
Size 11	1.56	2
Size 14	2.25	2
Size 18	4.00	1
Chosen Size		Size 5
Horizontal Reinforcement		
Size 4	0.20	6
Size 5	0.31	4
Size 6	0.44	3
Size 7	0.60	2
Size 8	0.79	2
Size 9	1.00	2
Chosen Size		Size 4
Reinf. Ratio, ρ		0.019
Min. Ah		1.02
Okay?		Yes



Project	Senior Design Parking Garage
Engineer:	Group 1
Date:	3/25/2016
Beam/Column Location:	Typical #1 Beam

Load Combinations

Case	Equation	Load (psf)
1	1.4D	128.3
2a.	1.2D+1.6L+0.5(Lr)	174.0
2b.	1.2D+1.6L+0.5(S)	174.0
2c.	1.2D+1.6L+0.5(R)	174.0
3a.	1.2D+1.6(Lr)+0.5(L)	130.0
3b.	1.2D+1.6(Lr)+0.5(+W)	110.0
3c.	1.2D+1.6(Lr)+0.5(-W)	110.0
3d.	1.2D+1.6(S)+0.5(L)	130.0
3e.	1.2D+1.6(S)+0.5(+W)	110.0
3f.	1.2D+1.6(S)+0.5(-W)	110.0
3g.	1.2D+1.6(R)+0.5(L)	130.0
3h.	1.2D+1.6(R)+0.5(+W)	110.0
3i.	1.2D+1.6(R)+0.5(-W)	110.0
4a.	1.2D+1.0(+W)+0.5L+0.5(Lr)	130.0
4b.	1.2D+1.0(+W)+0.5L+0.5(S)	130.0
4c.	1.2D+1.0(+W)+0.5L+0.5(R)	130.0
4d.	1.2D+1.0(-W)+0.5L+0.5(Lr)	130.0
4e.	1.2D+1.0(-W)+0.5L+0.5(Lr)	130.0
4f.	1.2D+1.0(-W)+0.5L+0.5(Lr)	130.0
5	1.2D+1.0E+0.5L+0.2S	126.1
6a.	0.9D+1.0(-W)	82.5
6b.	0.9D+1.0(+W)	82.5
7	0.9D+1.0E	78.6
Max		174.0
Min		0

Loads (psf)

Dead	91.67
Live	40
Wind (Pressure)	0
Wind (Suction)	0
Roof Live	0
Snow	0
Seismic	-3.91
Rain	0

Max Load

Tributary Width (ft)	10
Tributary Length (ft)	18
Distributed Load (klf)	1.74
Point Load (Kips)	31.32

Max Uplift

Tributary Width (ft)	10
Tributary Length (ft)	18
Distributed Load (klf)	0.00
Point Load (Kips)	0.00

Beam Reactions for Simply Supported

Max Moment, M_u, K-ft	70.5
Max Shear, V_u, Kips	15.7

Additional Notes and Assumptions
6" slab and 5 psf MEP load

Project	Senior Design Parking Garage
Engineer:	Group 1
Date:	3/25/2016
Beam/Column Location:	Typical #1 Beam

Page:	2/6

Flexure Design

Span Width, W	ft	18.00
28-Day Concrete Strength,f'c	ksi	5.00
Steel Yield Strength, fy	ksi	60.00
Thickess, h	in	14.00
Beam Width,b	in	8.00
Depth of Reinforcing, d=0.9h	in	12.00
Self Weight, Wsw	$\#/ft^2$	11.67
Max Moment, Mu	k-ft	70.47
Max Shear, Vu	k	15.66
Goal Seek	78.30000356	
Design Moment, Mn	k-ft	78.30
Area of Steel Required, As	in^2	1.46
Number of Bars Required		
Size 4	0.20	8.0
Size 5	0.31	5.0
Size 6	0.44	4.0
Size 7	0.60	3.0
Size 8	0.79	2.0
Size 9	1.00	2.0
Size 10	1.27	2.0
Size 11	1.56	1.0
Size 14	2.25	1.0
Size 18	4.00	1.0
Size of Bar Chosen	Size 10	
Total Area of Steel, As	in^2	2.54
Moment Capacity, ΦMn	k-ft	111.54
Okay?	Yes	

Checks

Strain, ε_t	0.00589
B1	0.8
c, in.	5.602941
a, in.	4.482353
dt, in.	12.00
Clear Space, in.	0.46
Reinforcing Ratio, ρ	0.026458
Minimum ρ_1	0.003536
Minimum ρ_2	0.003333
Okay?	No
Double Layer Check	
Clear Space, in.	1.73
Okay?	Yes
Triple Layer Check	
Clear Space, in.	
Okay?	
Assumptions and Notes	
Long term deflection multiplier,ζ , is 2.0 for loads sustained greater than 60 months.	

Project		Senior Design Parking Garage
Engineer:		Group 1
Date:		3/25/2016
Beam/Column Location:		Typical #1 Beam

Deflection Check, D+L			Deflection Check, D		
Deflection Limit	in.	0.9	Deflection Limit	in.	0.6
Long Term Multipler	ζ	2	Long Term Multipler	ζ	2
Service Load	psf	131.67	Service Load	psf	91.67
Service Load	klf	1.31666667	Service Load	klf	0.916667
Steel Modulus of Elasticity	ksi	29000	Steel Modulus of Elasticity	ksi	29000
Concrete Modulus of Elasticity	ksi	4074	Concrete Modulus of Elasticity	ksi	4074
Gross Moment of Inertia	$in.^4$	3750	Gross Moment of Inertia	$in.^4$	3750
Cracked Inertia	$in.^4$	1736	Cracked Inertia	$in.^4$	1736
Centroid	in.	7.61	Centroid	in.	7.61
Effective Inertia	$in.^4$	1873	Effective Inertia	$in.^4$	1873
Cracking Moment	k-ft.	21.8	Cracking Moment	k-ft.	21.8
Moment at Service Load	k-ft.	53.325	Moment at Service Load	k-ft.	37.125
Deflection at Service Load	in.	0.81	Deflection at Service Load	in.	0.57
OKAY?		Yes	OKAY?		Yes

Development and Splice Length		
Development Length, ld	ft	4.94
Splice Length, ls	ft	6.43

Project	Senior Design Parking Garage
Engineer:	Group 1
Date:	3/25/2016
Beam/Column Location:	Typical #1 Beam

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Shear		
Max Shear at d from Support, Vu	kips	13.92
Beam Width, b	inches	8.00
Depth of Reinforcement, d	inches	12.00
28-Day Concrete Strength,f'c	ksi	5.00
Steel Yield Strength, fy	ksi	60.00
Concrete Shear Strength, Vc	ksi	13.6
Factored Vc	ksi	10.18
Required Steel Shear Strength, ΦVs	ksi	3.74
Adequacy Check	ksi	40.73
Distance of Max Reinf. From Support	ft.	3.15
Distance of Max Reinf. From Midspan	ft.	5.85
Size of Chosen Stirrup	Size 3	
Spacing Requirement	inches	15.89
Max Permissible Spacing in Critical Sec.	inches	6.00
Shear Strength Provided in Critical Sec.	kips	20.08
Number of Stirrups in Critical Sec.	N_1	18
Nominal Shear Strength for Critical Sec., ΦVn	kips	20.08
OKAY?	Yes	

Equations	
Distance	Vu
0.00	15.66
1.00	13.92
1.80	12.53
3.60	9.40
5.40	6.26
7.20	3.13
9.00	0.00

Distance	ΦVn
0.00	20.08
9.00	20.08

Actual Spacing, in.
6.00

Project		Senior Design Parking Garage
Engineer:		Group 1
Date:		3/25/2016
Beam/Column Location:		Typical #1 Beam

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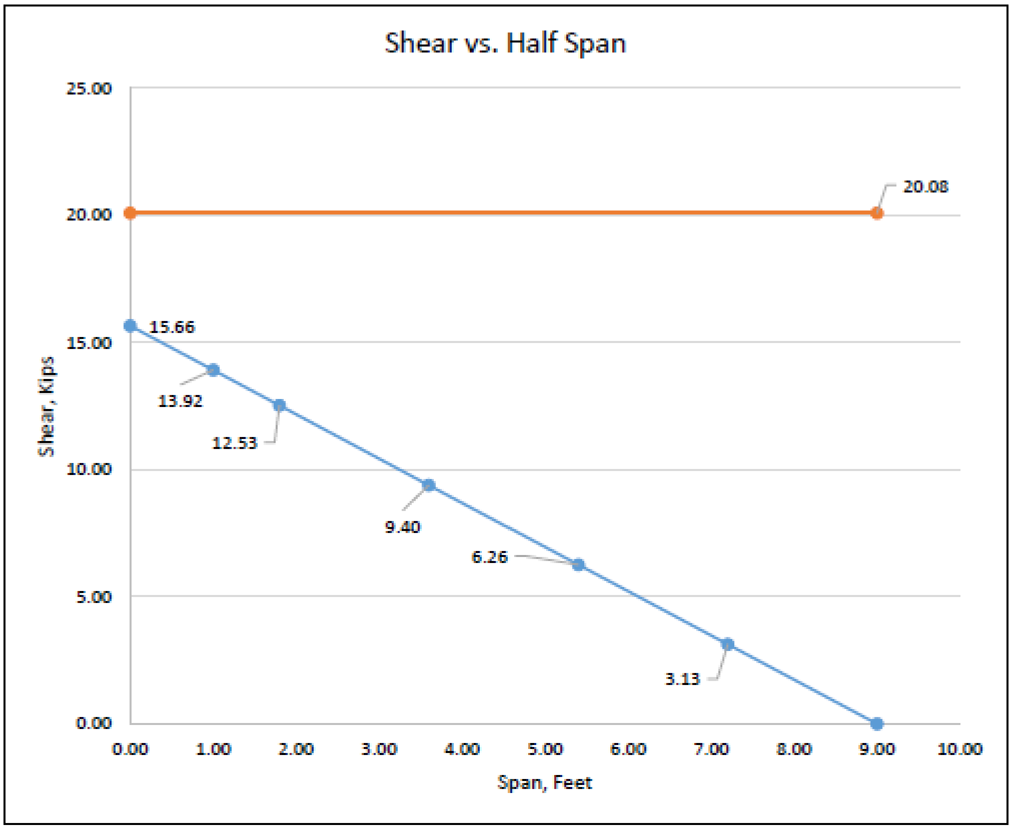

Project	Senior Design Parking Garage
Engineer:	Group 1
Date:	3/25/2016
Beam/Column Location:	Typical #1 Beam

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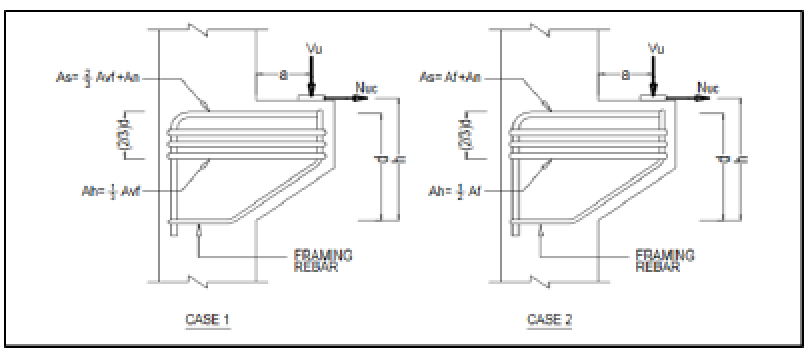
Corbel Connection Design		
Max Shear, Vu	kips	15.7
Corbel Width, b	in.	10
Corbel Height, h	in.	10
Effective Corbel Height, d	in.	8
28-Day Concrete Strength,f'c	ksi	5.00
Steel Yield Strength, fy	ksi	60.00
Size Okay?	Yes	
Eccentricity	in.	6
Coefficient of Friction	μ	0.6
Shear Friction Steel, Avf	in.^{2}	0.58
Ultimate Horizontal Force, Nu	kips	3.13
Tension Steel, An	in.^{2}	0.07
Max Moment on Corbel, Mu	kip-ft	10.44
Flexure Steel, Af	in.^{2}	0.41
Angle btwn Comp and Tens, β	°	48.58
Depth of Whitney Stress Block, a	in.	0.99
Distribution Case?	Case 2	
Required Area of Steel, As	in.^{2}	0.48
Supported Beam Width	in.	18.00
Corbel Depth	in.	21.00
Area of Horizontal Ties	in.^{2}	0.20
Depth of Ties from Tension Steel	in.	5.33

Bar Sizes		
Tension Reinforcement		
Size 4	0.20	3
Size 5	0.31	2
Size 6	0.44	2
Size 7	0.60	1
Size 8	0.79	1
Size 9	1.00	1
Size 10	1.27	1
Size 11	1.56	1
Size 14	2.25	1
Size 18	4.00	1
Chosen Size		Size 5
Horizontal Reinforcement		
Size 4	0.20	2
Size 5	0.31	1
Size 6	0.44	1
Size 7	0.60	1
Size 8	0.79	1
Size 9	1.00	1
Chosen Size		Size 4
Reinf. Ratio, ρ		0.008
Min. Ah		0.20
Okay?		Yes



Project	Senior Design Parking Garage
Engineer:	Group 1
Date:	3/25/2016
Beam/Column Location:	Typical #2 Beam

Load Combinations

Case	Equation	Load (psf)
1	1.4D	138.3
2a.	1.2D+1.6L+0.5(Lr)	182.5
2b.	1.2D+1.6L+0.5(S)	182.5
2c.	1.2D+1.6L+0.5(R)	182.5
3a.	1.2D+1.6(Lr)+0.5(L)	138.5
3b.	1.2D+1.6(Lr)+0.5(+W)	118.5
3c.	1.2D+1.6(Lr)+0.5(-W)	118.5
3d.	1.2D+1.6(S)+0.5(L)	138.5
3e.	1.2D+1.6(S)+0.5(+W)	118.5
3f.	1.2D+1.6(S)+0.5(-W)	118.5
3g.	1.2D+1.6(R)+0.5(L)	138.5
3h.	1.2D+1.6(R)+0.5(+W)	118.5
3i.	1.2D+1.6(R)+0.5(-W)	118.5
4a.	1.2D+1.0(+W)+0.5L+0.5(Lr)	138.5
4b.	1.2D+1.0(+W)+0.5L+0.5(S)	138.5
4c.	1.2D+1.0(+W)+0.5L+0.5(R)	138.5
4d.	1.2D+1.0(-W)+0.5L+0.5(Lr)	138.5
4e.	1.2D+1.0(-W)+0.5L+0.5(Lr)	138.5
4f.	1.2D+1.0(-W)+0.5L+0.5(Lr)	138.5
5	1.2D+1.0E+0.5L+0.2S	134.3
6a.	0.9D+1.0(-W)	88.9
6b.	0.9D+1.0(+W)	88.9
7	0.9D+1.0E	84.7
Max		182.5
Min		0

Loads (psf)

Dead	98.75
Live	40
Wind (Pressure)	0
Wind (Suction)	0
Roof Live	0
Snow	0
Seismic	-4.21
Rain	0

Max Load

Tributary Width (ft)	10
Tributary Length (ft)	24
Distributed Load (klf)	1.83
Point Load (Kips)	43.80

Max Uplift

Tributary Width (ft)	10
Tributary Length (ft)	24
Distributed Load (klf)	0.00
Point Load (Kips)	0.00

Beam Reactions for Simply Supported

Max Moment, M_u, K-ft	131.4
Max Shear, V_u, Kips	21.9

Additional Notes and Assumptions
6" slab and 5 psf MEP load

Project	Senior Design Parking Garage
Engineer:	Group 1
Date:	3/25/2016
Beam/Column Location:	Typical #2 Beam

Page:	2/6

Flexure Design		
Span Width, W	ft	24.00
28-Day Concrete Strength,f'c	ksi	5.00
Steel Yield Strength, fy	ksi	60.00
Thickess, h	in	18.00
Beam Width,b	in	10.00
Depth of Reinforcing, d=0.9h	in	16.00
Self Weight, Wsw	$\#/ft^2$	18.75
Max Moment, Mu	k-ft	131.40
Max Shear, Vu	k	21.90
Goal Seek	146.00	
Design Moment, Mn	k-ft	146.00
Area of Steel Required, As	in^2	2.00
Number of Bars Required		
Size 4	0.20	11.0
Size 5	0.31	7.0
Size 6	0.44	5.0
Size 7	0.60	4.0
Size 8	0.79	3.0
Size 9	1.00	4.0
Size 10	1.27	2.0
Size 11	1.56	2.0
Size 14	2.25	1.0
Size 18	4.00	1.0
Size of Bar Chosen	Size 9	
Total Area of Steel, As	in^2	4.00
Moment Capacity, ΦMn	k-ft	237.18
Okay?	Yes	

Checks	
Strain, ε_t	0.006375
B1	0.8
c, in.	7.058824
a, in.	5.647059
dt, in.	16.00
Clear Space, in.	0.162667
Reinforcing Ratio, ρ	0.025
Minimum ρ_1	0.003536
Minimum ρ_2	0.003333
Okay?	No
Double Layer Check	
Clear Space, in.	1.37
Okay?	Yes
Triple Layer Check	
Clear Space, in.	
Okay?	
Assumptions and Notes	
Long term deflection multiplier,ζ , is 2.0 for loads sustained greater than 60 months.	

Project	Senior Design Parking Garage
Engineer:	Group 1
Date:	3/25/2016
Beam/Column Location:	Typical #2 Beam

Deflection Check, D+L			Deflection Check, D		
Deflection Limit	in.	1.2	Deflection Limit	in.	0.8
Long Term Multipler	ζ	2	Long Term Multipler	ζ	2
Service Load	psf	138.75	Service Load	psf	98.75
Service Load	klf	1.3875	Service Load	klf	0.9875
Steel Modulus of Elasticity	ksi	29000	Steel Modulus of Elasticity	ksi	29000
Concrete Modulus of Elasticity	ksi	4074	Concrete Modulus of Elasticity	ksi	4074
Gross Moment of Inertia	$in.^4$	10144	Gross Moment of Inertia	$in.^4$	10144
Cracked Inertia	$in.^4$	4811	Cracked Inertia	$in.^4$	4811
Centroid	in.	9.84	Centroid	in.	9.84
Effective Inertia	$in.^4$	5318	Effective Inertia	$in.^4$	5318
Cracking Moment	k-ft.	45.6	Cracking Moment	k-ft.	45.6
Moment at Service Load	k-ft.	99.9	Moment at Service Load	k-ft.	71.1
Deflection at Service Load	in.	0.96	Deflection at Service Load	in.	0.68
OKAY?		Yes	OKAY?		Yes

Development and Splice Length		
Development Length, ld	ft	4.92
Splice Length, ls	ft	6.39

Project	Senior Design Parking Garage
Engineer:	Group 1
Date:	3/25/2016
Beam/Column Location:	Typical #2 Beam

Page:	4/6

Shear		
Max Shear at d from Support, Vu	kips	19.47
Beam Width, b	inches	10.00
Depth of Reinforcement, d	inches	16.00
28-Day Concrete Strength,f'c	ksi	5.00
Steel Yield Strength, fy	ksi	60.00
Concrete Shear Strength, Vc	ksi	22.6
Factored Vc	ksi	16.97
Required Steel Shear Strength, ΦVs	ksi	2.50
Adequacy Check	ksi	67.88
Distance of Max Reinf. From Support	ft.	2.70
Distance of Max Reinf. From Midspan	ft.	9.30
Size of Chosen Stirrup	Size 3	
Spacing Requirement	inches	31.73
Max Permissible Spacing in Critical Sec.	inches	8.00
Shear Strength Provided in Critical Sec.	kips	26.87
Number of Stirrups in Critical Sec.	N_1	18
Nominal Shear Strength for Critical Sec., ΦVn	kips	26.87
OKAY?	Yes	

Equations

Distance	Vu
0.00	21.90
1.33	19.47
2.40	17.52
4.80	13.14
7.20	8.76
9.60	4.38
12.00	0.00

Distance	ΦVn
0.00	26.87
12.00	26.87

Actual Spacing, in.
8.00

Project	Senior Design Parking Garage
Engineer:	Group 1
Date:	3/25/2016
Beam/Column Location:	Typical #2 Beam

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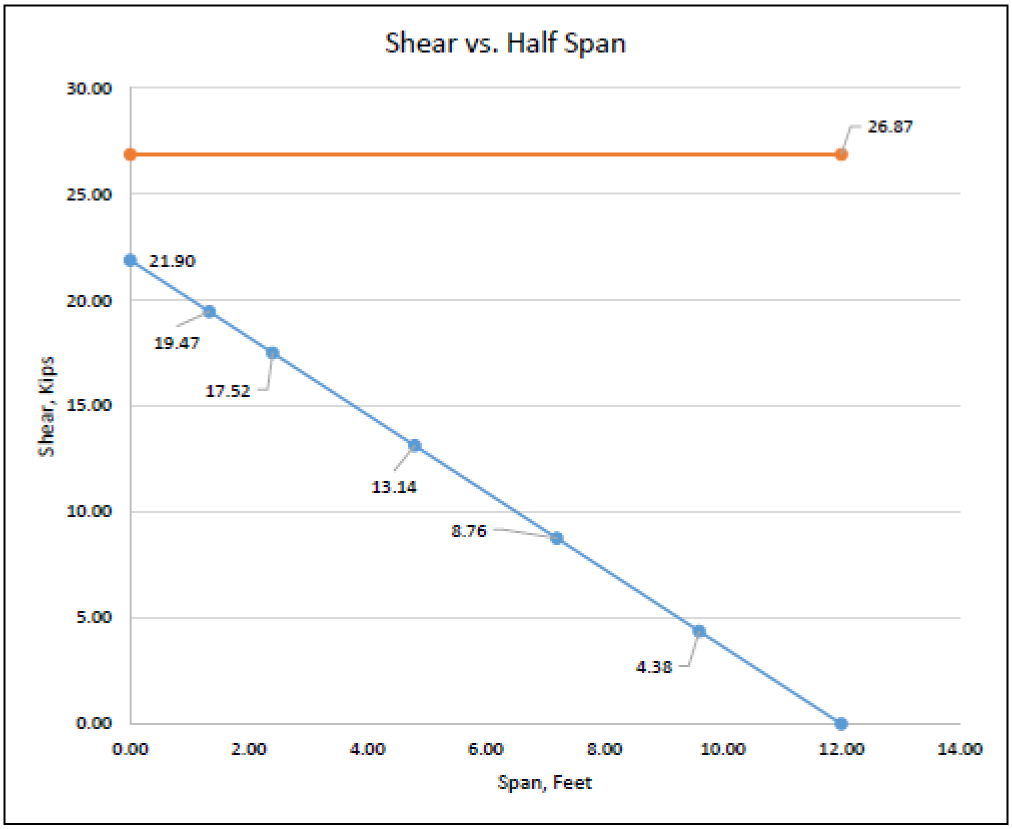

Project	Senior Design Parking Garage
Engineer:	Group 1
Date:	3/25/2016
Beam/Column Location:	Typical #2 Beam

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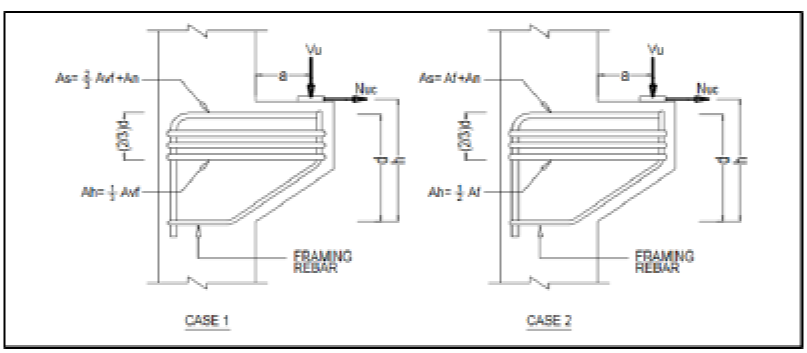
Corbel Connection Design

Max Shear, Vu	kips	21.9
Corbel Width, b	in.	10
Corbel Height, h	in.	10
Effective Corbel Height, d	in.	8
28-Day Concrete Strength,f'c	ksi	5.00
Steel Yield Strength, fy	ksi	60.00
Size Okay?	**Yes**	
Eccentricity	in.	6
Coefficient of Friction	µ	0.6
Shear Friction Steel, Avf	$in.^2$	0.81
Ultimate Horizontal Force, Nu	kips	4.38
Tension Steel, An	$in.^2$	0.10
Max Moment on Corbel, Mu	kip-ft	14.6
Flexure Steel, Af	$in.^2$	0.57
Angle btwn Comp and Tens, β	°	48.58
Depth of Whitney Stress Block, a	in.	1.38
Distribution Case?	**Case 2**	
Required Area of Steel, As	$in.^2$	0.67
Supported Beam Width	in.	18.00
Corbel Depth	in.	21.00
Area of Horizontal Ties	$in.^2$	0.29
Depth of Ties from Tension Steel	in.	5.33

Bar Sizes

Tension Reinforcement		
Size 4	0.20	4
Size 5	0.31	3
Size 6	0.44	2
Size 7	0.60	2
Size 8	0.79	1
Size 9	1.00	1
Size 10	1.27	1
Size 11	1.56	1
Size 14	2.25	1
Size 18	4.00	1
Chosen Size		Size 5
Horizontal Reinforcement		
Size 4	0.20	2
Size 5	0.31	1
Size 6	0.44	1
Size 7	0.60	1
Size 8	0.79	1
Size 9	1.00	1
Chosen Size		Size 4
Reinf. Ratio, ρ		0.012
Min. Ah		0.29
Okay?		**Yes**



Project	Senior Design Parking Garage
Engineer:	Group 1
Date:	3/25/2016
Beam/Column Location:	Girder

Loads (psf)	
Dead	118.06
Live	40
Wind (Pressure)	0
Wind (Suction)	0
Roof Live	0
Snow	0
Seismic	-5.03
Rain	0

Load Combinations

Case	Equation	Load (psf)
1	1.4D	165.3
2a.	1.2D+1.6L+0.5(Lr)	205.7
2b.	1.2D+1.6L+0.5(S)	205.7
2c.	1.2D+1.6L+0.5(R)	205.7
3a.	1.2D+1.6(Lr)+0.5(L)	161.7
3b.	1.2D+1.6(Lr)+0.5(+W)	141.7
3c.	1.2D+1.6(Lr)+0.5(-W)	141.7
3d.	1.2D+1.6(S)+0.5(L)	161.7
3e.	1.2D+1.6(S)+0.5(+W)	141.7
3f.	1.2D+1.6(S)+0.5(-W)	141.7
3g.	1.2D+1.6(R)+0.5(L)	161.7
3h.	1.2D+1.6(R)+0.5(+W)	141.7
3i.	1.2D+1.6(R)+0.5(-W)	141.7
4a.	1.2D+1.0(+W)+0.5L+0.5(Lr)	161.7
4b.	1.2D+1.0(+W)+0.5L+0.5(S)	161.7
4c.	1.2D+1.0(+W)+0.5L+0.5(R)	161.7
4d.	1.2D+1.0(-W)+0.5L+0.5(Lr)	161.7
4e.	1.2D+1.0(-W)+0.5L+0.5(Lr)	161.7
4f.	1.2D+1.0(-W)+0.5L+0.5(Lr)	161.7
5	1.2D+1.0E+0.5L+0.2S	156.6
6a.	0.9D+1.0(-W)	106.3
6b.	0.9D+1.0(+W)	106.3
7	0.9D+1.0E	101.2
Max		205.7
Min		0

Max Load	
Tributary Width (ft)	21
Tributary Length (ft)	32
Distributed Load (klf)	4.32
Point Load (Kips)	138.21

Max Uplift	
Tributary Width (ft)	21
Tributary Length (ft)	32
Distributed Load (klf)	0.00
Point Load (Kips)	0.00

Beam Reactions for Simply Supported	
Max Moment, M_u, K-ft	552.8
Max Shear, V_u, Kips	69.1

Additional Notes and Assumptions
6" slab and 5 psf MEP load

Project	Senior Design Parking Garage
Engineer:	Group 1
Date:	3/25/2016
Beam/Column Location:	Girder

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Flexure Design		
Span Width, W	ft	32.00
28-Day Concrete Strength,f'c	ksi	5.00
Steel Yield Strength, fy	ksi	60.00
Minimum Thickess, h	in	26.00
Beam Width,b	in	14.00
Depth of Reinforcing, d=0.9h	in	24.00
Self Weight, Wsw	$\#/ft^2$	18.06
Max Moment, Mu	k-ft	552.83
Max Shear, Vu	k	69.10
Goal Seek	614.2599997	
Design Moment, Mn	k-ft	614.26
Area of Steel Required, As	in^2	5.83
Number of Bars Required		
Size 4	0.20	30.0
Size 5	0.31	19.0
Size 6	0.44	14.0
Size 7	0.60	10.0
Size 8	0.79	8.0
Size 9	1.00	8.0
Size 10	1.27	8.0
Size 11	1.56	4.0
Size 14	2.25	3.0
Size 18	4.00	2.0
Size of Bar Chosen	Size 10	
Total Area of Steel, As	in2	10.16
Moment Capacity, Mn	k-ft	958.9674
Okay?	Yes	

Checks	
Strain, ε_t	0.005388
B1	0.8
c, in.	12.80672
a, in.	10.24538
dt, in.	24.00
Clear Space, in.	-0.16571
Reinforcing Ratio, ρ	0.030238
Minimum ρ_1	0.003536
Minimum ρ_2	0.003333
Okay?	No
Double Layer Check	
Clear Space, in.	1.306667
Okay?	Yes
Triple Layer Check	
Clear Space, in.	2.595
Okay?	Yes
Assumptions and Notes	
Long term deflection multiplier,ζ , is 2.0 for loads sustained greater than 60 months.	

Project	Senior Design Parking Garage
Engineer:	Group 1
Date:	3/25/2016
Beam/Column Location:	Girder

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Deflection Check, D+L			Deflection Check, D		
Deflection Limit	in.	1.6	Deflection Limit	in.	1.066667
Long Term Multipler	ζ	2	Long Term Multipler	ζ	2
Service Load	psf	158.06	Service Load	psf	118.06
Service Load	klf	3.31916667	Service Load	klf	2.479167
Steel Modulus of Elasticity	ksi	29000	Steel Modulus of Elasticity	ksi	29000
Concrete Modulus of Elasticity	ksi	4074	Concrete Modulus of Elasticity	ksi	4074
Gross Moment of Inertia	$in.^4$	45402	Gross Moment of Inertia	$in.^4$	45402
Cracked Inertia	$in.^4$	26436	Cracked Inertia	$in.^4$	26436
Centroid	in.	14.60	Centroid	in.	14.60
Effective Inertia	$in.^4$	27078	Effective Inertia	$in.^4$	27078
Cracking Moment	k-ft.	137.4	Cracking Moment	k-ft.	137.4
Moment at Service Load	k-ft.	424.853333	Moment at Service Load	k-ft.	317.3333
Deflection at Service Load	in.	1.42	Deflection at Service Load	in.	1.06
OKAY?		Yes	OKAY?		Yes

Development and Splice Length		
Development Length, ld	ft	6.55
Splice Length, ls	ft	8.51

Project	Senior Design Parking Garage
Engineer:	Group 1
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Beam/Column Location:	Girder

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Shear		
Max Shear at d from Support, Vu	kips	60.47
Beam Width, b	inches	14.00
Depth of Reinforcement, d	inches	24.00
28-Day Concrete Strength,f'c	ksi	5.00
Steel Yield Strength, fy	ksi	60.00
Concrete Shear Strength, Vc	ksi	47.5
Factored Vc	ksi	35.64
Required Steel Shear Strength, ΦVs	ksi	24.83
Adequacy Check	ksi	142.55
Distance of Max Reinf. From Support	ft.	7.75
Distance of Max Reinf. From Midspan	ft.	8.25
Size of Chosen Stirrup	Size 4	
Spacing Requirement	inches	8.700
Max Permissible Spacing in Critical Sec.	inches	12
Shear Strength Provided in Critical Sec.	kips	62.64
Number of Stirrups in Critical Sec.	N_1	24
Nominal Shear Strength for Critical Sec., ΦVn	kips	62.64
OKAY?	Yes	

Equations	
Distance	Vu
0.00	69.10
2.00	60.47
3.20	55.28
6.40	41.46
9.60	27.64
12.80	13.82
16.00	0.00

Distance	ΦVn
0.00	62.64
16.00	62.64

Actual Spacing, in.
8.00

Project	Senior Design Parking Garage
Engineer:	Group 1
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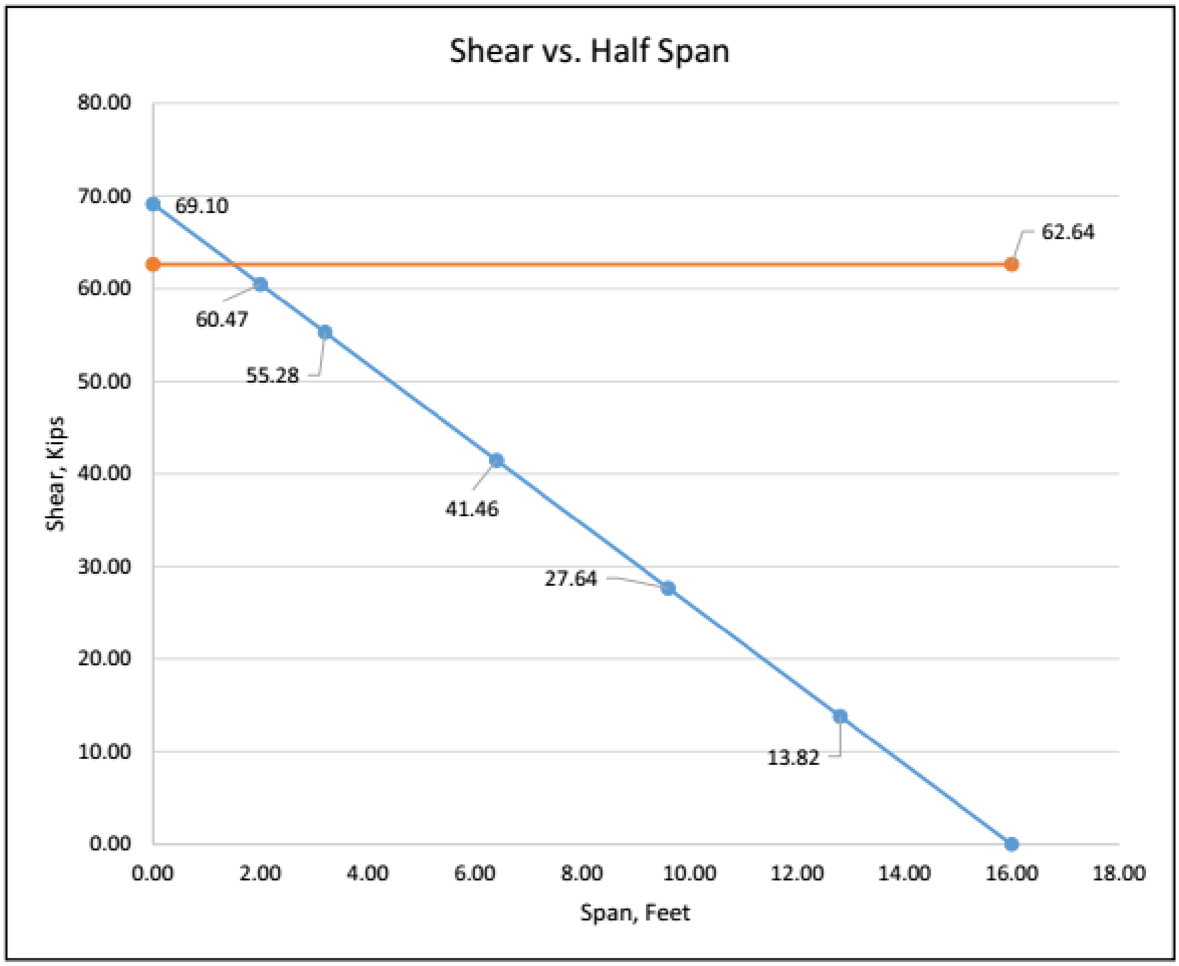
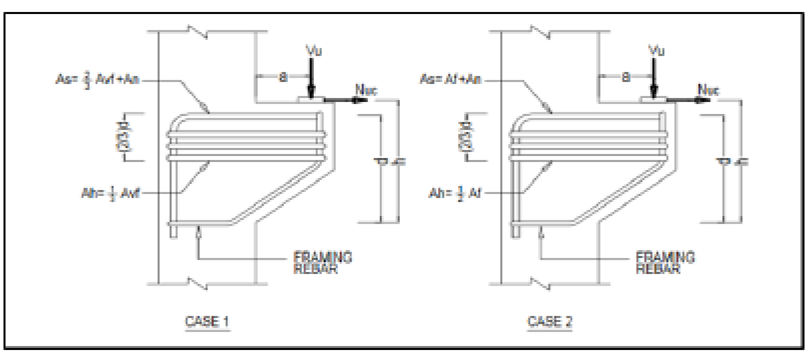


Project	Senior Design Parking Garage
Engineer:	Group 1
Date:	3/25/2016
Beam/Column Location:	Girder

Corbel Connection Design		
Max Shear, Vu	kips	69.1
Corbel Width, b	in.	10
Corbel Height, h	in.	15
Effective Corbel Height, d	in.	13
28-Day Concrete Strength,f'c	ksi	5.00
Steel Yield Strength, fy	ksi	60.00
Size Okay?	Yes	
Eccentricity	in.	6
Coefficient of Friction	μ	0.6
Shear Friction Steel, Avf	in.^{2}	2.56
Ultimate Horizontal Force, Nu	kips	13.82
Tension Steel, An	in.^{2}	0.31
Max Moment on Corbel, Mu	kip-ft	46.06933
Flexure Steel, Af	in.^{2}	1.11
Angle btwn Comp and Tens, β	°	61.50
Depth of Whitney Stress Block, a	in.	3.73
Distribution Case?	Case 1	
Required Area of Steel, As	in.^{2}	2.01
Supported Beam Width	in.	18.00
Corbel Depth	in.	21.00
Area of Horizontal Ties	in.^{2}	0.85
Depth of Ties from Tension Steel	in.	8.67

Bar Sizes		
Tension Reinforcement		
Size 4	0.20	11
Size 5	0.31	7
Size 6	0.44	5
Size 7	0.60	4
Size 8	0.79	3
Size 9	1.00	3
Size 10	1.27	2
Size 11	1.56	2
Size 14	2.25	1
Size 18	4.00	1
Chosen Size		Size 5
Horizontal Reinforcement		
Size 4	0.20	5
Size 5	0.31	3
Size 6	0.44	2
Size 7	0.60	2
Size 8	0.79	2
Size 9	1.00	1
Chosen Size		Size 4
Reinf. Ratio, ρ		0.017
Min. Ah		0.85
Okay?		Yes



Project	Senior Design Parking Garage
Engineer:	Group 1
Date:	3/25/2016
Beam/Column Location:	Typical Column

Loads and Reactions		
Max Factored Load 1 at Roof Level	kips	73
Roof Level 1 Eccentricity	in.	6
Resulting Max Moment	kip-ft	0
Height of Roof Reaction 1	ft	50
Max Factored Load 2 at Roof Level	kips	73
Roof Level 2 Eccentricity	in.	6
Resulting Max Moment	kip-ft	0
Height of Roof Reaction 2	ft	50
Max Factored Load 1 at Floor Level	kips	70
Floor Level 1 Eccentricity	in.	6
Resulting Max Moment	kip-ft	0
Height of Floor Load 1	ft	35
Max Factored Load 2 at Floor Level	kips	70
Floor Level 2 Eccentricity	in.	6
Resulting Max Moment	kip-ft	0.00
Height of Floor Load 2	ft	35
Ultimate Axial Load at Ground, Pu	kips	739.3
Ultimate Moment at Ground	kip-ft	0.00
Concrete Compressive Strength, fc'	ksi	4
Steel Yield Strength, fy	ksi	60
Gamma	γ	0.72
C-C Spacing of Longitudinal Steel, γh	in.	15.87

Sizes and Checks	
Width	22
Depth	22
Moment of Inertia	19521.3
Gross Area	484
Radius of Gyration	6.35
Stiffness	18.90
Stiffness Limit	34
Non-Slender?	Yes
e/h	0.27
Initial Rho	0.015
Required Area	335.27
Required Steel Area	7.26
Bar Size	Number of Bars
Size 4	37
Size 5	24
Size 6	17
Size 7	13
Size 8	10
Size 9	8
Size 10	6
Chosen Size	Size 9
Provided Steel Area	8.00
Tie Size	Tie Spacing
Size 3	18.00
Size 4	18.05
Size 5	18.05

Appendix D-2 Foundation design calculations

Boring Log 1 Nu = 9

Depth (ft)	Cu (psf)	Alpha	fs(ksf) = alpha * Cu	qtip = Cu*Nu
0-9	1000	0.55	0.55	9
9-14.5	1250	0.55	0.6875	11.25
14.5-51	18000	0.35	6.3	162

Boring Log 2

Depth (ft)	Cu (psf)	Alpha	fs(ksf) = alpha * Cu	qtip = Cu*Nu
0-8	750	0.55	0.4125	6.75
8-15.5	2750	0.55	1.5125	24.75
15.5-51	18000	0.35	6.3	162

Boring Log 3

Depth (ft)	Cu (psf)	Alpha	fs(ksf) = alpha * Cu	qtip = Cu*Nu
0-8	3000	0.55	1.65	27
8-16.0	7500	0.42	3.15	67.5
16-51.5	18000	0.35	6.3	162

Boring Log 4

Depth (ft)	Cu (psf)	Alpha	fs(ksf) = alpha * Cu	qtip = Cu*Nu
0-8	2350	0.55	1.2925	21.15
8-16.0	4250	0.45	1.9125	38.25
16-51.5	18000	0.35	6.3	162

Boring Log 5

Depth (ft)	Cu (psf)	Alpha	fs(ksf) = alpha * Cu	qtip = Cu*Nu
0-7	900	0.55	0.495	8.1
7-19.5	1000	0.55	0.55	9
19.5-50	18000	0.35	6.3	162

Boring Log 6

Depth (ft)	Cu (psf)	Alpha	fs(ksf) = alpha * Cu	qtip = Cu*Nu
0-8	8000	0.4	3.2	72
8-15.0	2500	0.55	1.375	22.5
15-18	13500	0.38	5.13	121.5
18- 53.5	18000	0.35	6.3	162

Boring Log #1					
Depth	Skin Friction, ksi	Tip Bearing, ksi	Design Load for 2' O.D.	Design Load for 2.5' O.D.	Design Load for 3' O.D.
1	0.55	9	15.86	24.24	34.38
2	0.55	9	17.58	26.40	36.97
3	0.55	9	19.31	28.55	39.56
4	0.55	9	21.04	30.71	42.15
5	0.55	9	22.77	32.87	44.75
6	0.55	9	24.49	35.03	47.34
7	0.55	9	26.22	37.19	49.93
8	0.55	9	27.95	39.35	52.52
9	0.55	9	29.67	41.51	55.11
10	0.6875	11.25	35.36	49.72	66.29
11	0.6875	11.25	37.52	52.42	69.53
12	0.6875	11.25	39.68	55.12	72.77
13	0.6875	11.25	41.84	57.82	76.01
14	0.6875	11.25	44.00	60.52	79.25
15	6.3	162	291.65	444.04	628.23
16	6.3	162	311.43	468.77	657.90
17	6.3	162	331.21	493.50	687.57
18	6.3	162	350.99	518.22	717.24
19	6.3	162	370.78	542.95	746.92
20	6.3	162	390.56	567.68	776.59
21	6.3	162	410.34	592.41	806.26
22	6.3	162	430.12	617.13	835.94
23	6.3	162	449.90	641.86	865.61
24	6.3	162	469.69	666.59	895.28
25	6.3	162	489.47	691.32	924.96
26	6.3	162	509.25	716.04	954.63
27	6.3	162	529.03	740.77	984.30
28	6.3	162	548.81	765.50	1013.97
29	6.3	162	568.60	790.23	1043.65
30	6.3	162	588.38	814.95	1073.32
31	6.3	162	608.16	839.68	1102.99
32	6.3	162	627.94	864.41	1132.67
33	6.3	162	647.72	889.14	1162.34
34	6.3	162	667.51	913.86	1192.01
35	6.3	162	687.29	938.59	1221.69
36	6.3	162	707.07	963.32	1251.36
37	6.3	162	726.85	988.05	1281.03
38	6.3	162	746.63	1012.77	1310.70

39	6.3	162	766.42	1037.50	1340.38
40	6.3	162	786.20	1062.23	1370.05
41	6.3	162	805.98	1086.96	1399.72
42	6.3	162	825.76	1111.68	1429.40
43	6.3	162	845.54	1136.41	1459.07
44	6.3	162	865.33	1161.14	1488.74
45	6.3	162	885.11	1185.87	1518.42
46	6.3	162	904.89	1210.59	1548.09
47	6.3	162	924.67	1235.32	1577.76
48	6.3	162	944.45	1260.05	1607.43
49	6.3	162	964.24	1284.78	1637.11
50	6.3	162	984.02	1309.50	1666.78

Boring Log #2					
Depth	Skin Friction, ksi	Tip Bearing, ksi	Design Load for 2' O.D.	Design Load for 2.5' O.D.	Design Load for 3' O.D.
1	0.4125	6.75	11.89	18.18	25.79
2	0.4125	6.75	13.19	19.80	27.73
3	0.4125	6.75	14.48	21.42	29.67
4	0.4125	6.75	15.78	23.03	31.62
5	0.4125	6.75	17.07	24.65	33.56
6	0.4125	6.75	18.37	26.27	35.50
7	0.4125	6.75	19.66	27.89	37.44
8	0.4125	6.75	20.96	29.51	39.39
9	1.5125	24.75	53.97	67.95	110.10
10	1.5125	24.75	58.72	73.88	117.22
11	1.5125	24.75	63.47	79.82	124.34
12	1.5125	24.75	68.22	85.76	131.47
13	1.5125	24.75	72.97	91.69	138.59
14	1.5125	24.75	77.72	97.63	145.72
15	1.5125	24.75	82.46	103.57	152.84
16	6.3	162	317.73	476.64	667.35
17	6.3	162	337.51	501.37	697.02
18	6.3	162	357.29	526.10	726.69
19	6.3	162	377.07	550.82	756.37
20	6.3	162	396.86	575.55	786.04
21	6.3	162	416.64	600.28	815.71
22	6.3	162	436.42	625.01	845.39
23	6.3	162	456.20	649.73	875.06
24	6.3	162	475.98	674.46	904.73
25	6.3	162	495.77	699.19	934.41
26	6.3	162	515.55	723.92	964.08
27	6.3	162	535.33	748.64	993.75
28	6.3	162	555.11	773.37	1023.42
29	6.3	162	574.89	798.10	1053.10
30	6.3	162	594.68	822.83	1082.77
31	6.3	162	614.46	847.55	1112.44
32	6.3	162	634.24	872.28	1142.12
33	6.3	162	654.02	897.01	1171.79
34	6.3	162	673.80	921.74	1201.46
35	6.3	162	693.59	946.46	1231.14
36	6.3	162	713.37	971.19	1260.81
37	6.3	162	733.15	995.92	1290.48
38	6.3	162	752.93	1020.65	1320.15

39	6.3	162	772.71	1045.37	1349.83
40	6.3	162	792.50	1070.10	1379.50
41	6.3	162	812.28	1094.83	1409.17
42	6.3	162	832.06	1119.56	1438.85
43	6.3	162	851.84	1144.28	1468.52
44	6.3	162	871.62	1169.01	1498.19
45	6.3	162	891.41	1193.74	1527.87
46	6.3	162	911.19	1218.47	1557.54
47	6.3	162	930.97	1243.19	1587.21
48	6.3	162	950.75	1267.92	1616.88
49	6.3	162	970.53	1292.65	1646.56
50	6.3	162	990.32	1317.38	1676.23

Boring Log #3					
Depth	Skin Friction, ksi	Tip Bearing, ksi	Design Load for 2' O.D.	Design Load for 2.5' O.D.	Design Load for 3' O.D.
1	1.65	27	47.57	72.71	103.15
2	1.65	27	52.75	79.19	110.92
3	1.65	27	57.93	85.66	118.69
4	1.65	27	63.11	92.14	126.46
5	1.65	27	68.30	98.62	134.24
6	1.65	27	73.48	105.09	142.01
7	1.65	27	78.66	111.57	149.78
8	1.65	27	83.84	118.04	157.55
9	3.15	67.5	157.31	229.76	315.45
10	3.15	67.5	167.21	242.12	330.29
11	3.15	67.5	177.10	254.49	345.13
12	3.15	67.5	186.99	266.85	359.96
13	3.15	67.5	196.88	279.21	374.80
14	3.15	67.5	206.77	291.58	389.63
15	3.15	67.5	216.66	303.94	404.47
16	3.15	67.5	226.55	316.31	419.31
17	6.3	162	394.70	572.85	782.80
18	6.3	162	414.48	597.58	812.48
19	6.3	162	434.26	622.31	842.15
20	6.3	162	454.04	647.04	871.82
21	6.3	162	473.83	671.76	901.49
22	6.3	162	493.61	696.49	931.17
23	6.3	162	513.39	721.22	960.84
24	6.3	162	533.17	745.95	990.51
25	6.3	162	552.95	770.67	1020.19
26	6.3	162	572.74	795.40	1049.86
27	6.3	162	592.52	820.13	1079.53
28	6.3	162	612.30	844.86	1109.21
29	6.3	162	632.08	869.58	1138.88
30	6.3	162	651.86	894.31	1168.55
31	6.3	162	671.65	919.04	1198.22
32	6.3	162	691.43	943.77	1227.90
33	6.3	162	711.21	968.49	1257.57
34	6.3	162	730.99	993.22	1287.24
35	6.3	162	750.77	1017.95	1316.92
36	6.3	162	770.56	1042.68	1346.59
37	6.3	162	790.34	1067.40	1376.26
38	6.3	162	810.12	1092.13	1405.94

39	6.3	162	829.90	1116.86	1435.61
40	6.3	162	849.68	1141.59	1465.28
41	6.3	162	869.47	1166.31	1494.95
42	6.3	162	889.25	1191.04	1524.63
43	6.3	162	909.03	1215.77	1554.30
44	6.3	162	928.81	1240.50	1583.97
45	6.3	162	948.59	1265.22	1613.65
46	6.3	162	968.38	1289.95	1643.32
47	6.3	162	988.16	1314.68	1672.99
48	6.3	162	1007.94	1339.41	1702.67
49	6.3	162	1027.72	1364.13	1732.34
50	6.3	162	1047.50	1388.86	1762.01

Boring Log #4					
Depth	Skin Friction, ksi	Tip Bearing, ksi	Design Load for 2' O.D.	Design Load for 2.5' O.D.	Design Load for 3' O.D.
1	1.2925	21.15	37.26	56.96	80.80
2	1.2925	21.15	41.32	62.03	86.89
3	1.2925	21.15	45.38	67.10	92.98
4	1.2925	21.15	49.44	72.18	99.06
5	1.2925	21.15	53.50	77.25	105.15
6	1.2925	21.15	57.56	82.32	111.24
7	1.2925	21.15	61.61	87.40	117.33
8	1.2925	21.15	65.67	92.47	123.41
9	1.9125	38.25	98.53	141.92	192.83
10	1.9125	38.25	104.53	149.43	201.84
11	1.9125	38.25	110.54	156.94	210.84
12	1.9125	38.25	116.54	164.44	219.85
13	1.9125	38.25	122.55	171.95	228.86
14	1.9125	38.25	128.55	179.46	237.87
15	1.9125	38.25	134.56	186.96	246.87
16	1.9125	38.25	140.56	194.47	255.88
17	6.3	162	354.63	522.77	722.70
18	6.3	162	374.41	547.50	752.38
19	6.3	162	394.20	572.23	782.05
20	6.3	162	413.98	596.95	811.72
21	6.3	162	433.76	621.68	841.39
22	6.3	162	453.54	646.41	871.07
23	6.3	162	473.32	671.14	900.74
24	6.3	162	493.11	695.86	930.41
25	6.3	162	512.89	720.59	960.09
26	6.3	162	532.67	745.32	989.76
27	6.3	162	552.45	770.05	1019.43
28	6.3	162	572.23	794.77	1049.11
29	6.3	162	592.02	819.50	1078.78
30	6.3	162	611.80	844.23	1108.45
31	6.3	162	631.58	868.96	1138.12
32	6.3	162	651.36	893.68	1167.80
33	6.3	162	671.14	918.41	1197.47
34	6.3	162	690.93	943.14	1227.14
35	6.3	162	710.71	967.87	1256.82
36	6.3	162	730.49	992.59	1286.49
37	6.3	162	750.27	1017.32	1316.16
38	6.3	162	770.05	1042.05	1345.84

39	6.3	162	789.84	1066.78	1375.51
40	6.3	162	809.62	1091.50	1405.18
41	6.3	162	829.40	1116.23	1434.85
42	6.3	162	849.18	1140.96	1464.53
43	6.3	162	868.96	1165.69	1494.20
44	6.3	162	888.75	1190.41	1523.87
45	6.3	162	908.53	1215.14	1553.55
46	6.3	162	928.31	1239.87	1583.22
47	6.3	162	948.09	1264.60	1612.89
48	6.3	162	967.87	1289.32	1642.57
49	6.3	162	987.66	1314.05	1672.24
50	6.3	162	1007.44	1338.78	1701.91

Boring Log #5					
Depth	Skin Friction, ksi	Tip Bearing, ksi	Design Load for 2' O.D.	Design Load for 2.5' O.D.	Design Load for 3' O.D.
1	0.495	8.1	14.27	21.81	30.94
2	0.495	8.1	15.83	23.76	33.28
3	0.495	8.1	17.38	25.70	35.61
4	0.495	8.1	18.93	27.64	37.94
5	0.495	8.1	20.49	29.58	40.27
6	0.495	8.1	22.04	31.53	42.60
7	0.495	8.1	23.60	33.47	44.93
8	0.55	9	26.74	37.84	50.70
9	0.55	9	28.46	40.00	53.29
10	0.55	9	30.19	42.15	55.88
11	0.55	9	31.92	44.31	58.47
12	0.55	9	33.65	46.47	61.07
13	0.55	9	35.37	48.63	63.66
14	0.55	9	37.10	50.79	66.25
15	0.55	9	38.83	52.95	68.84
16	0.55	9	40.55	55.11	71.43
17	0.55	9	42.28	57.27	74.02
18	0.55	9	44.01	59.42	76.61
19	0.55	9	45.73	61.58	79.20
20	6.3	162	296.70	450.35	635.80
21	6.3	162	316.48	475.08	665.48
22	6.3	162	336.26	499.81	695.15
23	6.3	162	356.04	524.54	724.82
24	6.3	162	375.83	549.26	754.49
25	6.3	162	395.61	573.99	784.17
26	6.3	162	415.39	598.72	813.84
27	6.3	162	435.17	623.45	843.51
28	6.3	162	454.95	648.17	873.19
29	6.3	162	474.74	672.90	902.86
30	6.3	162	494.52	697.63	932.53
31	6.3	162	514.30	722.36	962.21
32	6.3	162	534.08	747.08	991.88
33	6.3	162	553.86	771.81	1021.55
34	6.3	162	573.65	796.54	1051.22
35	6.3	162	593.43	821.27	1080.90
36	6.3	162	613.21	845.99	1110.57
37	6.3	162	632.99	870.72	1140.24
38	6.3	162	652.77	895.45	1169.92

39	6.3	162	672.56	920.18	1199.59
40	6.3	162	692.34	944.90	1229.26
41	6.3	162	712.12	969.63	1258.94
42	6.3	162	731.90	994.36	1288.61
43	6.3	162	751.68	1019.09	1318.28
44	6.3	162	771.47	1043.81	1347.95
45	6.3	162	791.25	1068.54	1377.63
46	6.3	162	811.03	1093.27	1407.30
47	6.3	162	830.81	1118.00	1436.97
48	6.3	162	850.59	1142.72	1466.65
49	6.3	162	870.38	1167.45	1496.32
50	6.3	162	890.16	1192.18	1525.99

Boring Log #6					
Depth	Skin Friction, ksi	Tip Bearing, ksi	Design Load for 2' O.D.	Design Load for 2.5' O.D.	Design Load for 3' O.D.
1	3.2	72	123.09	189.19	269.41
2	3.2	72	133.14	201.75	284.48
3	3.2	72	143.18	214.31	299.56
4	3.2	72	153.23	226.87	314.63
5	3.2	72	163.28	239.43	329.70
6	3.2	72	173.33	251.99	344.77
7	3.2	72	183.38	264.55	359.84
8	3.2	72	193.42	277.11	374.92
9	1.375	22.5	120.03	161.07	206.53
10	1.375	22.5	124.34	166.47	213.01
11	1.375	22.5	128.66	171.87	219.49
12	1.375	22.5	132.98	177.26	225.96
13	1.375	22.5	137.30	182.66	232.44
14	1.375	22.5	141.61	188.06	238.91
15	1.375	22.5	145.93	193.45	245.39
16	5.13	121.5	317.47	373.24	619.27
17	5.13	121.5	333.58	393.37	643.43
18	5.13	121.5	349.69	413.51	667.60
19	6.3	162	433.05	620.80	840.33
20	6.3	162	452.84	645.53	870.01
21	6.3	162	472.62	670.25	899.68
22	6.3	162	492.40	694.98	929.35
23	6.3	162	512.18	719.71	959.03
24	6.3	162	531.96	744.44	988.70
25	6.3	162	551.75	769.16	1018.37
26	6.3	162	571.53	793.89	1048.05
27	6.3	162	591.31	818.62	1077.72
28	6.3	162	611.09	843.35	1107.39
29	6.3	162	630.87	868.07	1137.06
30	6.3	162	650.66	892.80	1166.74
31	6.3	162	670.44	917.53	1196.41
32	6.3	162	690.22	942.26	1226.08
33	6.3	162	710.00	966.98	1255.76
34	6.3	162	729.78	991.71	1285.43
35	6.3	162	749.57	1016.44	1315.10
36	6.3	162	769.35	1041.17	1344.78
37	6.3	162	789.13	1065.89	1374.45
38	6.3	162	808.91	1090.62	1404.12

39	6.3	162	828.69	1115.35	1433.79
40	6.3	162	848.48	1140.08	1463.47
41	6.3	162	868.26	1164.80	1493.14
42	6.3	162	888.04	1189.53	1522.81
43	6.3	162	907.82	1214.26	1552.49
44	6.3	162	927.60	1238.99	1582.16
45	6.3	162	947.39	1263.71	1611.83
46	6.3	162	967.17	1288.44	1641.51
47	6.3	162	986.95	1313.17	1671.18
48	6.3	162	1006.73	1337.90	1700.85
49	6.3	162	1026.51	1362.62	1730.52
50	6.3	162	1046.30	1387.35	1760.20

Factored Dead Load on the Pier Cap, kips			
Depth	2' O.D. Pier	2.5' O.D. Pier	3' O.D. Pier
1	0.66	1.03	1.48
2	1.32	2.06	2.97
3	1.98	3.09	4.45
4	2.64	4.12	5.93
5	3.30	5.15	7.42
6	3.96	6.18	8.90
7	4.62	7.21	10.39
8	5.28	8.24	11.87
9	5.93	9.27	13.35
10	6.59	10.30	14.84
11	7.25	11.33	16.32
12	7.91	12.36	17.80
13	8.57	13.39	19.29
14	9.23	14.42	20.77
15	9.89	15.45	22.25
16	10.55	16.49	23.74
17	11.21	17.52	25.22
18	11.87	18.55	26.71
19	12.53	19.58	28.19
20	13.19	20.61	29.67
21	13.85	21.64	31.16
22	14.51	22.67	32.64
23	15.17	23.70	34.12
24	15.83	24.73	35.61
25	16.49	25.76	37.09
26	17.14	26.79	38.57
27	17.80	27.82	40.06
28	18.46	28.85	41.54
29	19.12	29.88	43.03
30	19.78	30.91	44.51
31	20.44	31.94	45.99
32	21.10	32.97	47.48
33	21.76	34.00	48.96
34	22.42	35.03	50.44
35	23.08	36.06	51.93
36	23.74	37.09	53.41
37	24.40	38.12	54.90
38	25.06	39.15	56.38

39	25.72	40.18	57.86
40	26.38	41.21	59.35
41	27.04	42.24	60.83
42	27.69	43.27	62.31
43	28.35	44.30	63.80
44	29.01	45.33	65.28
45	29.67	46.36	66.76
46	30.33	47.39	68.25
47	30.99	48.42	69.73
48	31.65	49.46	71.22
49	32.31	50.49	72.70
50	32.97	51.52	74.18

Pier Design				
Location	Northing	Easting	Depth	Provided Strength
A1	120	0	29	790.23
A2	120	18.25	29	790.23
A3	120	44.25	29	790.23
A4	120	76.25	29	792.45
A5	120	108.25	29	796.52
A6	120	140.25	29	798.10
A7	120	168.25	29	798.10
B1	102	0	29	790.23
B2	102	18.25	29	790.23
B3	102	44.25	29	791.45
B4	102	76.25	29	792.50
B5	102	108.25	29	794.60
B6	102	140.25	29	798.10
B7	102	168.25	29	798.10
C1	78	0	28	792.50
C2	78	18.25	28	794.60
C3	78	44.25	27	795.60
C4	78	76.25	28	793.80
C5	78	108.25	28	794.20
C6	78	140.25	28	794.28
C7	78	168.25	28	794.28
D1	60	0	28	787.60
D2	60	18.25	27	790.20
D3	60	44.25	26	795.40
D4	60	76.25	26	794.40
D5	60	108.25	28	794.89
D6	60	140.25	28	794.28
D7	60	168.25	28	795.40
E1	42	0	29	795.64
E2	42	18.25	29	795.26
E3	42	44.25	29	794.24
E4	42	76.25	29	796.32
E5	42	108.25	28	794.54
E6	42	140.25	27	794.48
E7	42	168.25	26	795.36
F1	18	0	34	794.89
F2	18	18.25	34	796.98
F3	18	44.25	34	795.47
F4	18	76.25	32	795.54
F5	18	108.25	30	794.32
F6	18	140.25	28	796.11
F7	18	168.25	26	796.10
G1	0	0	34	796.05
G2	0	18.25	34	795.65

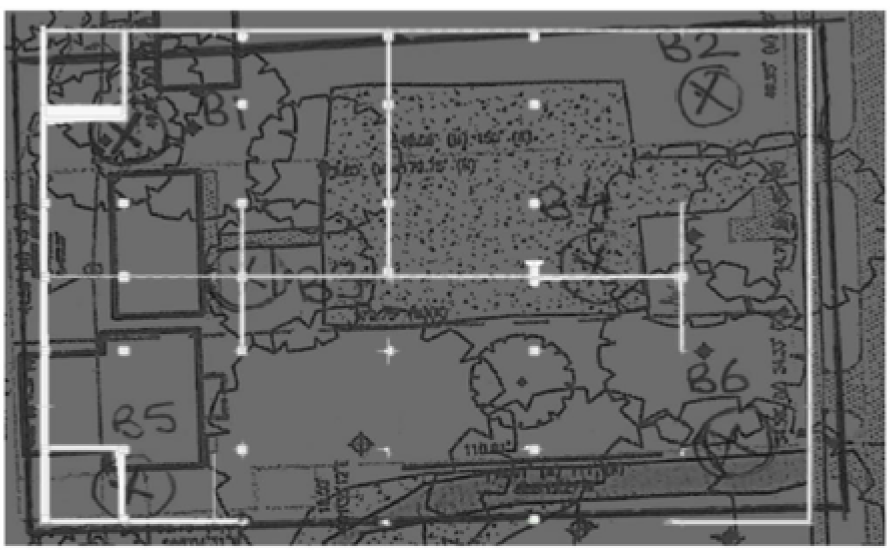
G3	0	44.25	33	794.58
G4	0	76.25	32	796.65
G5	0	108.25	30	794.39
G6	0	140.25	28	796.54
G7	0	168.25	26	796.54

Design Capacity				
Boring	Northing	Easting	Depth	Strength
1	98.0	18.0	29	790.23
2	104.0	145.0	29	798.10
3	60.0	44.5	26	795.40
4	63.0	115.0	28	794.28
5	8.0	20.0	34	796.54
6	19.0	150.0	26	793.89

Max Axial Load	739.3
Max Shear Load	73.9
Max Moment	554.48

kn	Rn	Ast
0.2789	0.0836	10.16
Reinforcement is based on 8 #10 bars		

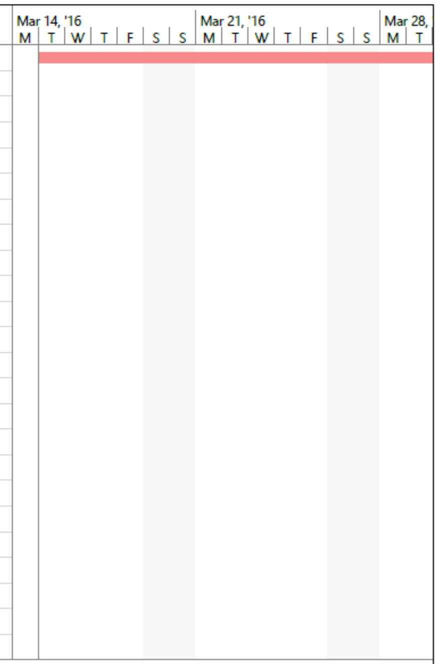
Pier Reinforcement Dimensions	
Diameter	2.5'
Percent of Diameter to Reinforcing, γ	0.9
Number of Longitudinal Bars	8
Size of Longitudinal Bars	#10
Spiral Pitch	5.00
Spiral Bar Size	#5
Spiral Clear Distance	4.375
Spiral Reinforcement Ratio	0.0078



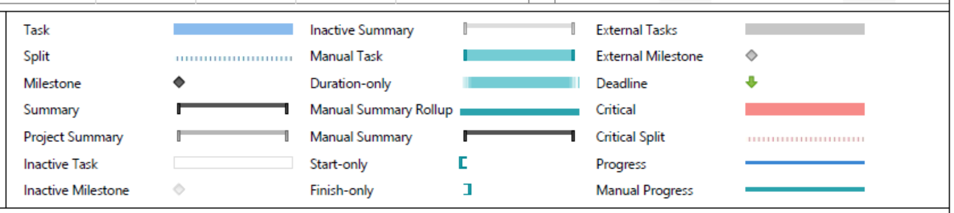
Appendix E- Construction Management

Project Schedule using Microsoft Project.

ID		Task Mode	Task Name	Duration	Start	Finish	Predecessors
1			Foundation Piers	29 days	Tue 3/15/16	Fri 4/22/16	
2			Basement Slab	2 days	Mon 4/25/16	Tue 4/26/16	1
3			Basement Columns	2 days	Wed 4/27/16	Thu 4/28/16	2
4			2W Beams	2 days	Fri 4/29/16	Mon 5/2/16	3
5			Basement Shaft	2 days	Wed 4/27/16	Thu 4/28/16	2
6			Basement Walls	2 days	Fri 4/29/16	Mon 5/2/16	3
7			Basement-1E Column	4 days	Tue 5/3/16	Fri 5/6/16	8
8			Basement-1E Slab	2 days	Fri 4/29/16	Mon 5/2/16	2,3
9			Foundation Walls	3 days	Tue 5/3/16	Thu 5/5/16	8
10			2W-2E Beams	3 days	Mon 5/9/16	Wed 5/11/16	8,7
11			1E Slab	1 day	Thu 4/28/16	Tue 5/3/16	8
12			Basement Shear Walls	3 days	Tue 5/3/16	Thu 5/5/16	4
13			1E Columns	2 days	Fri 4/29/16	Fri 5/13/16	11,10
14			1E Walls	3 days	Mon 5/16/16	Wed 5/18/16	13
15			2E Beams	2 days	Mon 5/16/16	Tue 5/17/16	13
16			2W-1E Slab	2 days	Wed 5/18/16	Thu 5/19/16	11,15
17			2W-1E Beams	3 days	Fri 5/20/16	Tue 5/24/16	16
18			2nd Floor Shear Wall	3 days	Fri 5/20/16	Tue 5/24/16	16
19			2W Slab	1 day	Wed 5/25/16	Wed 5/25/16	16,17
20			2W Columns	2 days	Thu 5/26/16	Fri 5/27/16	19,17
21			2W Beams	2 days	Mon 5/30/16	Tue 5/31/16	20
22			2W Walls	2 days	Mon 5/30/16	Tue 5/31/16	20
23			2W Walls	2 days	Mon 5/30/16	Tue 5/31/16	20
24			2W-2E Slab	3 days	Wed 6/1/16	Fri 6/3/16	19,21



Project: Scheduling
Date: Fri 4/29/16



ID		Task Mode	Task Name	Duration	Start	Finish	Predecessors
25			2W-2E Columns	2 days	Mon 6/6/16	Tue 6/7/16	24
26			2W-2E Beams	3 days	Wed 6/8/16	Fri 6/10/16	25
27			2W-2E Walls	3 days	Wed 6/8/16	Fri 6/10/16	25
28			2E Slab	2 days	Mon 6/13/16	Tue 6/14/16	24,26
29			2E Columns	1 day	Wed 6/15/16	Wed 6/15/16	28
30			2E Beams	2 days	Thu 6/16/16	Fri 6/17/16	29
31			3rd Floor Apt Wall	3 days	Wed 6/15/16	Fri 6/17/16	28
32			3W-2E Slab	2 days	Mon 6/20/16	Tue 6/21/16	28,30
33			3W-2E Columns	3 days	Wed 6/22/16	Fri 6/24/16	32
34			3W-2E Beams	3 days	Mon 6/27/16	Wed 6/29/16	33
35			3W Slab	1 day	Tue 4/26/16	Thu 6/30/16	32,34
36			3W Beams	2 days	Tue 7/5/16	Wed 7/6/16	37
37			3W Columns	2 days	Fri 7/1/16	Mon 7/4/16	35
38			3W Walls	2 days	Tue 7/5/16	Wed 7/6/16	37
39			3W Walls	1 day	Tue 7/5/16	Tue 7/5/16	37
40			3rd Floor Shear	1 day	Fri 7/1/16	Fri 7/1/16	35
41			3W-3E Slab	2 days	Thu 7/7/16	Fri 7/8/16	35,36
42			3W-3E Columns	2 days	Mon 7/11/16	Tue 7/12/16	41
43			3W-3E Beams	3 days	Wed 7/13/16	Fri 7/15/16	42
44			3E Slab	1 day	Mon 7/18/16	Mon 7/18/16	41,43
45			3E Columns	1 day	Tue 7/19/16	Tue 7/19/16	44
46			3E Walls	2 days	Wed 7/20/16	Thu 7/21/16	45
47			3E Beams	2 days	Wed 7/20/16	Thu 7/21/16	45
48			3rd Floor Apt Wall	2 days	Fri 7/22/16	Mon 7/25/16	46

Mar 14, '16 M T W T F S S | Mar 21, '16 M T W T F S S | Mar 28, M T

Project: Scheduling
Date: Fri 4/29/16

Task	Inactive Summary	External Tasks
Split	Manual Task	External Milestone
Milestone	Duration-only	Deadline
Summary	Manual Summary Rollup	Critical
Project Summary	Manual Summary	Critical Split
Inactive Task	Start-only	Progress
Inactive Milestone	Finish-only	Manual Progress

ID		Task Mode	Task Name	Duration	Start	Finish	Predecessors
49			4th Floor Shear Wall	2 days	Tue 7/26/16	Wed 7/27/16	48
50			3E-4W Slab	2 days	Fri 7/22/16	Mon 7/25/16	44,47
51			3E-4W Beams	3 days	Tue 7/26/16	Thu 7/28/16	50
52			4W Slab	1 day	Fri 7/29/16	Fri 7/29/16	50,51
53			4W Columns	1 day	Mon 8/1/16	Mon 8/1/16	52
54			4W Walls	2 days	Tue 8/2/16	Wed 8/3/16	53
55			4W Walls	2 days	Tue 8/2/16	Wed 8/3/16	53
56			4W Beams	1 day	Tue 8/2/16	Tue 8/2/16	53
57			5th Floor Shear	1 day	Thu 8/4/16	Thu 8/4/16	55
58			4W-4E Slab	2 days	Wed 8/3/16	Thu 8/4/16	52,56
59			4W-4E Columns	2 days	Fri 8/5/16	Mon 8/8/16	58
60			4W-4E Beams	3 days	Tue 8/9/16	Thu 8/11/16	59
61			4E Slab	1 day	Fri 8/12/16	Fri 8/12/16	58,60
62			4E Columns	1 day	Mon 8/15/16	Mon 8/15/16	61
63			4E Walls	2 days	Tue 8/16/16	Wed 8/17/16	62
64			4E Beams	1 day	Mon 8/15/16	Mon 8/15/16	61
65			5th Floor Apt Wall	2 days	Thu 8/18/16	Fri 8/19/16	63
66			5W-4E Slab	2 days	Tue 8/16/16	Wed 8/17/16	61,64
67			5W-4E Columns	2 days	Thu 8/18/16	Fri 8/19/16	66
68			5W-4E Beams	3 days	Mon 8/22/16	Wed 8/24/16	67
69			5W Slab	1 day	Thu 8/25/16	Thu 8/25/16	66,68
70			5W Columns	1 day	Fri 8/26/16	Fri 8/26/16	69
71			5W Beams	1 day	Mon 8/29/16	Mon 8/29/16	70
72			5W Walls	2 days	Mon 8/29/16	Tue 8/30/16	70

Mar 14, '16 M T W T F S S | Mar 21, '16 M T W T F S S | Mar 28, M T

Project: Scheduling
Date: Fri 4/29/16

Task	Inactive Summary	External Tasks
Split	Manual Task	External Milestone
Milestone	Duration-only	Deadline
Summary	Manual Summary Rollup	Critical
Project Summary	Manual Summary	Critical Split
Inactive Task	Start-only	Progress
Inactive Milestone	Finish-only	Manual Progress

ID		Task Mode	Task Name	Duration	Start	Finish	Predecessors	Mar 14, '16 M T W T F S S Mar 21, '16 M T W T F S S Mar 28, M T
73			5W-5E Slab	3 days	Tue 4/26/16	Thu 9/1/16	69,71	
74			6th Floor Shear Wall	1 day	Wed 8/31/16	Wed 8/31/16	72	
75			5W-5E Columns	2 days	Fri 9/2/16	Mon 9/5/16	73	
76			5W-5E Beams	2 days	Tue 9/6/16	Wed 9/7/16	75	
77			5E Slab	1 day	Tue 4/26/16	Thu 9/8/16	73,76	
78			5E Columns	1 day	Fri 9/9/16	Fri 9/9/16	77	
79			5E Beams	1 day	Mon 9/12/16	Mon 9/12/16	78	
80			5E Walls	2 days	Mon 9/12/16	Tue 9/13/16	78	
81			6th Floor Apt Wall	2 days	Wed 9/14/16	Thu 9/15/16	80	
82			5E-6W Slab	1 day	Fri 9/9/16	Fri 9/9/16	77	
83			6W Slab	2 days	Mon 9/12/16	Tue 9/13/16	82	
84			6th Floor Shaft Walls	2 days	Wed 9/14/16	Thu 9/15/16	83	
85			Shaft Roofs	1 day	Fri 9/16/16	Fri 9/16/16	84	
86			6W-6E	2 days	Wed 9/14/16	Thu 9/15/16	83	
87			6W Walls	2 days	Fri 9/16/16	Mon 9/19/16	86	
88			6E Slab	2 days	Fri 9/16/16	Mon 9/19/16	86	
89			6E Walls	2 days	Tue 9/20/16	Wed 9/21/16	88	

Project: Scheduling
Date: Fri 4/29/16

Task	Inactive Summary	External Tasks
Split	Manual Task	External Milestone
Milestone	Duration-only	Deadline
Summary	Manual Summary Rollup	Critical
Project Summary	Manual Summary	Critical Split
Inactive Task	Start-only	Progress
Inactive Milestone	Finish-only	Manual Progress