Western Michigan University Main Campus Parking Garage

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WMU Main Campus Student Parking Structure

Sponsored By WGI

Senior Design Team

Brendan Graham
Andy Oviedo
Faris Zayed
Acknowledgements

We would like to thank WGI for sponsoring our project. Specifically, we would like to thank Andrew Kong, P.E. and Trey Just, P.E. of WGI for taking the time to meet with us regularly and for putting us on the right track for success in our project. Andrew’s and Trey’s insights really allowed us to grasp the professional and conceptual aspects of our project.

We would like to thank our faculty advisor, Dr. Shao, for always being willing to answer our questions as they developed. Her knowledge of structural engineering truly provided us with the tools and resources necessary to complete our project. We would also like to thank Dr. Hu for providing us with access to the RSMeans software that assisted us in completing our project estimate. In addition, a special thanks to all our professors not mentioned. We were truly blessed to have exceptional professors that helped prepare us to excel in our senior design project. Lastly, we would like to thank our senior design coordinators, Dr. Hains and Mrs. Hammond, for making this a fun and enlightening experience.
Executive Summary

Student parking when attending class seems to be an issue at most universities. At Western Michigan University, it is a particular issue near the Haworth College of Business and Rood Hall. Students regularly struggle to find parking spaces in a timely manner when attending class in this area. Parking in this area currently consists solely of asphalt parking lots. It was our goal to both increase parking capacity and green space in this area. We will do this by designing a multi-level parking garage that will provide a more efficient use of parking space. As you will see later on, our project will not only increase the parking capacity on the lot we chose to build by approximately 50%, but it will also free up hundreds of square feet of additional space that can be used as green space to create a more environmentally sustainable and aesthetically pleasing area. Our report will detail the process that led us to create our parking structure. It will detail the type of structure we chose to use and why. It will also show the structural, geotechnical, and traffic design processes and all requirements, codes, and specifications used in our project. There will be a detailed estimate and construction schedule, as well as the sustainability factors we took into consideration. The primary software used in our project were SAP2000 for structural design purposes, AutoCAD for design drawings, Mathcad for lengthy equation calculations, RSMeans for estimating, and Microsoft Project for scheduling. It is estimated that our project will take approximately 122 calendar days to complete and cost $5,323,526.
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1 Introduction
1.1 Understanding the Project

For years Western Michigan University students have experienced problems with parking on Student Lot 72W and Lot 61, located on WMU’s main campus near the Haworth College of Business and Rood Hall. Figure 1 shows all parking lots in this area, who is allowed to park in what areas, and images showing the parking state of the areas designated as commuter (student) parking during peak parking hours.

Lot 61 is primarily designated for the parking of WMU Employees, with a few rows reserved for student parking. During peak parking hours, the small area allotted for student parking in Lot 61 is always at or near capacity. Unable to find a spot, students may drive to Lot 72W with hopes of finding a place to park, only to find themselves driving in circles as that lot is also typically at or near capacity during peak parking hours. During these peak parking hours, students must wait for someone to leave the parking lot and free up a spot, at which point the fight for who will be the first to claim it begins. This painstaking process leads to student tardiness and frustration. To avoid tardiness, or to avoid missing class entirely, some students may take the risk and park in non-student spaces. However, as many students have witnessed firsthand, this regularly results in being served with expensive parking tickets, another burden for the typical, monetarily-strained college student.
Between the loss of in-person education time and parking tickets, students can often be out hundreds if not thousands of dollars a year, simply because the proper parking capacity does not exist. To tackle this issue, we have designed a multi-level parking structure to occupy a portion of Lot 72W on the corner of Ring Rd N and Business Ct. The parking structure will increase the parking capacity of Lot 72W by approximately 50% and is designed to serve only students, particularly those in the Haworth Business college and those involved in the math and science courses at Rood Hall. The structure shows a commitment to student development which can lead to increased enrollment and enrollment revenue. It is also an additional selling point for the Haworth College of Business which already sits on Princeton Review’s list of best business schools.

1.2 Constraints

Because the construction of our parking structure is on an existing student parking lot, there will be few significant environmental or social impacts. The social impact will be very minimal as we are simply taking an area, already designated for student parking, and building a student parking garage. The largest constraint will be time. Since the existing lot will be out of commission during construction, the already strained parking situation in the area will be significantly worse. Fortunately, we will have the summer months, May through August, where few students are on campus, to complete construction. However, we are aware that this is a very tight window and provides a significant constraint to the scope of work that we will be able to undertake. If our project was to run into the beginning of the fall semester, students would be impacted as their parking options would be even more limited than before. Lastly, the way this project was procured did not involve specific budgetary limits. That being said, our team did not neglect the fact that cost is a constraint in every engineering endeavor. This project was no exception. Taking cost into account as a constraint allowed us to provide the best product at the most efficient price.
1.3 Analysis of Alternatives

Before we began structural design of our parking structure, we had to consider what type of structure we wanted to build. Generally speaking, there are 3 types of structures used in large scale buildings required to carry heavy loads. Those structure types include steel supported structures, cast-in-place concrete structures, and pre-cast concrete structures. In the following table, Table 1, we break the different structure types down into 7 different consideration factors and ranked them on scale of 1-5, with different considerations carrying different weights. For the purpose of this scale, the higher the score the better. The maximum score any structure type can receive is 50.

### Table 1: Structure Type Design Considerations and Rankings

<table>
<thead>
<tr>
<th>Consideration</th>
<th>Weight</th>
<th>Steel</th>
<th>Cast-in-Place Concrete</th>
<th>Precast Concrete</th>
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</thead>
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<tr>
<td>Flexural Strength</td>
<td>1</td>
<td>5</td>
<td>4</td>
<td>4</td>
</tr>
<tr>
<td>Compressive Strength</td>
<td>1</td>
<td>5</td>
<td>5</td>
<td>5</td>
</tr>
<tr>
<td>Overall Constructability</td>
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<td>3</td>
<td>7.5</td>
<td>7.5</td>
</tr>
<tr>
<td>Construction Time</td>
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<td>7.5</td>
<td>2.5</td>
<td>12.5</td>
</tr>
<tr>
<td>Cost</td>
<td>2</td>
<td>8</td>
<td>4</td>
<td>8</td>
</tr>
<tr>
<td>Durability</td>
<td>1</td>
<td>3</td>
<td>5</td>
<td>3</td>
</tr>
<tr>
<td>Environmental Impact</td>
<td>1</td>
<td>5</td>
<td>4</td>
<td>3</td>
</tr>
<tr>
<td><strong>Total Score (50 = max)</strong></td>
<td></td>
<td>36.5</td>
<td>32</td>
<td>43</td>
</tr>
</tbody>
</table>

As you can see in the table, we broke the structure types down into flexural strength, compressive strength, overall constructability, construction time, cost, durability, and environmental impacts. Overall, the precast concrete structure received the highest score. With regards to flexural strength, concrete itself has a very low tensile strength but, because of the ability to place steel rebar within the concrete, it still has a relatively high flexural strength capability. For overall constructability, both the cast-in-place and precast concrete structures received exceptionally high scores because all aspects of a parking garage can be made of concrete. Steel received a lower score because it cannot be used for the slabs of our parking structure, and thus, we would still require a significant amount of concrete to construct the slabs. Steel is also not ideal for the staircases we will require. For construction time, precast concrete received the highest score because placing precast concrete structures is essentially a matter of playing Tetris. All members are pre-designed to sit directly on top of the others via precast haunches. Steel received a lower score because while it would be fairly quick and easy to construct the steel frame, connecting the concrete slabs to the frames would be much more intricate and time consuming, especially on the inclined ramp sections. Cast-in-place concrete
received the worst score possible for construction time because of its requirement for concrete field curing. Overall, precast concrete and steel have similar construction costs while cast-in-place concrete tends to be more expensive due to additional construction time and the need for concrete formwork. With regards to durability, cast-in-place concrete would be the most durable structure due to concrete being a more durable material over time than steel, especially in the corrosive environments that our parking structure will be subject to. Precast concrete and steel received the same score because, while concrete is generally more durable over time, the precast concrete structure will have additional joints relative to cast-in-place concrete. These joints will make it subject to additional corrosive and weathering conditions that the cast-in-place concrete structure will not be subject to. Lastly, the steel structure received the highest sustainability score because steel is a recyclable structural material while concrete is not. The cast-in-place concrete structure received a slightly higher score than the precast concrete structure with regards to sustainability because its additional durability will allow it to last for longer.

After taking all these considerations into account, and when considering our most pressing constraint, time, we decided to select a precast concrete structure. We believe that this will provide us with a more than capable parking structure and give us the best chance of being able to construct our parking structure in the 4-month window that we have.
1.4 **Scope of Work**

1.4.1 **Structural Scope**

The structural scope included an overall design of the structure as well as a structural analysis of the parking garage. This included understanding the various loads that act on the structure throughout its life and how those loads are carried throughout the structure. Our structure consists of precast reinforced concrete members: beams, columns, and pre-topped double-tees which act as the floor slab system. These are the main structural components of the parking structure we designed. Our structural scope does not include the design of any member connections or bearing plates. Drafts of the design, and the design members, are displayed throughout this report using AutoCAD. All members are designed to accommodate all building codes enforced by the state of Michigan.

The structural scope for the design of the precast double-tees is entirely limited to the pre-designed members available in the Precast/ Prestressed Concrete Institute Handbook. Using the loads we calculated to be present on our parking structure, we selected previously designed precast double-tees with given dimensions and reinforcement. We selected these double-tees using the appropriate PCI loading tables and made sure to select members that have load capacities greater than the calculated applied loads that will be present on our parking structure.

The structural scope for the design of the beams involved using the PCI Handbook loading tables for beams to select general section dimensions that would be adequate to support the loads applied to them. We then designed customized reinforcing steel, different from the reinforcement steel listed in the PCI Handbook. We did this for a few reasons. First, we wanted to display what we’ve learned in our structural design courses during our time at WMU. Secondly, while the PCI handbook gives the amount and type of reinforcing steel in certain cross sections, it does not give the reinforcing steel configuration. It also only uses no. 4 rebar which we decided was not ideal for our requirements. Lastly, as you will see later, one of the beam’s loading requirement was not met by any pre-designed PCI Handbook beam at the length we required. Because of this, we decided to design our own reinforcing steel that we could assure would be adequate. We only considered flexure and shear when designing the beams used in our parking structure. Design for torsion was not included in our scope.

The structural scope for the design of the columns included using SAP2000 to select adequate column dimensions and then using the PCI Handbook interaction curves for precast, reinforced columns to select adequate longitudinal reinforcing steel. We also designed for the longitudinal reinforcing ties.
1.4.2 Geotechnical Scope

The geotechnical scope included designing the foundational supports for our parking structure. A geotechnical report was procured from SME for a location very near to our construction site. We used this geotechnical report to determine the most suitable type of foundational system and necessary size, depth, and quantity of the components making up that system. Our geotechnical scope includes why we chose to use a driven pile/pile cap foundational system, the size, amount, and location of our driven piles, and the dimensions and layouts of our pile caps. Not included in our geotechnical scope is reinforcing steel design for the pile caps as our experience in that area is limited. Drafts of the foundation design was executed and displayed using AutoCAD.

1.4.3 Transportation Scope

The transportation scope including the design of our structure’s overall parking geometry. This include effectively utilizing the structures floor area to provide the maximum amount of parking spaces while maintaining a comfortable driving experience throughout the structure. The parking geometry adheres to MDOT’s Geometric Design Guidance manual prepared by traffic and safety.

1.4.4 Estimating and Scheduling Scope

The estimating scope included utilizing RSMeans by including all necessary line items of materials to calculate the total cost of the parking structure. Many factors are considered in a given line item for a specific activity including raw materials cost, equipment fees, and labor fees.

The scheduling scope included developing the GANTT chart/construction schedule for the summer months of May through August of 2022. While developing this chart, many tasks were linked with one another as certain tasks cannot not be started until preceding tasks are completed. The activity durations and overall schedule duration is based off a standard five-day work week and does not account for potential weekend working days.
1.5 Deliverables

- Excavation and Grading Quantities and Elevations
- Structural Analysis
  - Load Calculations
  - SAP2000 Analysis
  - PCI Design Tables and Processes Used in Selection of Precast Double-Tees
  - Mathcad Calculations Used in Beam Design
  - PCI Interaction Curves and Processes Used in Selection of Columns
- Structural Design
  - Cross Section Dimensions and Steel Reinforcement for Precast Double-Tees
  - AutoCAD Design Drawings Depicting All Reinforcement and Relevant Section Dimensions for Beams and Columns
- Geotechnical Analysis
  - Soil Profile Analysis
  - Soil Bearing Capacity Analysis
- Geotechnical Design
  - Design and Capacities of Selected Driven Piles
  - Pile Cap Design
  - AutoCAD Design Drawings Depicting Driven Pile/Pile Cap Foundations
- Traffic Design
  - AutoCAD Drawings Depicting Parking Layout for All Levels
- Detailed Estimate
  - RS Means estimates exported to spreadsheet form including material, labor, and equipment fees.
- Construction Schedule
  - Microsoft Project Gantt chart outlining all tasks of construction.
- Promotional Video
- Project Presentation
- Project Poster
2 Site Plan

2.1 Site Logistics Plan

During the preconstruction/planning phase, there are parameters that need to be fulfilled prior to starting any work. One of those parameters is the development of a document called a Site Logistics Plan. The site logistics plan is important for maintaining safety and productivity. This plan displays where the site fence outlining construction limits will go, where field personnel trailers will be placed, and designated material delivery and laydown areas. Directly to the right of the site fence will be where all equipment, trucks, and materials come into the site. For this coordination, it is necessary to have a full-time traffic flagger, required site and road signage, and awareness on site to ensure safety between construction personnel and the public.

Figure 2: Site Logistics Plan
2.2 Excavating and Grading

The previous parking lot was laid out on top of an uneven subsurface. In order to achieve a level subsurface for construction, excavation and grading will occur onsite prior to construction. To determine the volume of the soil that must be excavated, it was important to establish a goal elevation to be reached across the site. This was achieved by taking elevation measurements at multiple cross sections across our construction site. Using online topography resources, elevations for various locations on our construction site were taken. The four corners of our construction zone were measured and are shown below in Figure 3.

NW – 911 ft  NE – 913 ft  SW – 911 ft  SE – 913 ft

Figure 3 - Elevations on Site

It was determined that excavating our site to a finished level-elevation of 908 feet would be ideal. This finished grade elevation was selected in order to combat the frost line and better protect the pile caps in our structure. In the state of Michigan, footings are required to be a minimum of 42 inches deep, measured from the finished grade to the bottom of the footing. To reach this desired depth, the East side of the site will be cut down 5 feet and the West side 3 feet. These dimensions were used to establish a typical cross section of the excavation site shown in Figure 4.

The total volume that will need to be excavated was found by multiplying the area shown in Figure 4 by the width of the structure. The final volume to be excavated was determined to be 7218 yd³. That calculation was performed as follows:
Volume to be Excavated

(3ft +5ft) * (280ft) * (1/2) = 1120 ft²

(1120 ft²) * (174ft) = 194,880 ft³

(194,880 ft³) / (27 ft³/ yd³) = 7217.78 yd³
3 Structural Analysis

3.1 Analysis and Selection of Precast Double-Tees

In the vertical loads calculated below, we used ASCE7-10, “Minimum Design Loads for Buildings and Other Structures,” and selected the appropriate loads and LRFD load combination equations to determine the controlling loads present on our parking structure. For the 3rd level double-tee, we determined that LRFD load combination 3 would control. This was due to the snow load present on the top level of our parking structure.

3rd Level Loads on Double-Tees

Snow Load = 35 psf (obtained from ASCE7-10)
Live Load = 40 psf (obtained from sponsor)
Wind Load = 7.83 psf (calculated from ASCE7-10 procedures & shown in Figure 1 of Appendix II)

\[
LC3 = 1.6(35) + 40 = 96 \text{ psf}
\]

2nd Level Loads on Double Tees

For the 2nd Level double-tee, we determined that LRFD load combination 2 would control, as the only load type present on this level is a live load.

Live Load = 40 psf (obtained from sponsor)

\[
LC2 = 1.6(40) = 64 \text{ psf}
\]

Precast Double-Tee Selection

We then used the load tables in the Precast/Prestressed Concrete Institute Design Handbook (8th Ed) to select the precast double tee sections that would be suitable to carry these loads. The loading tables shown in the PCI handbook include both members with no additional concrete topping, and members topped with 3 inches of normal weight concrete. For the selection, we only considered members with the 3-inch additional normal weight concrete topping, as that is standard for most parking structures. It should be noted that the PCI Design Handbook states in Chapter 3 section 3.1 that, “For deck components with composite topping, 15 lb/ft² of the capacity shown is assumed as superimposed dead load.” This means that the precast double-tee we choose must be able to withstand the required controlling load plus an additional 15 psf. Tables 2 and 3 show the PCI double-tee loading tables used in the selection process.
3rd Level Double-Tee Member Selection

Table 2: PCI Loading Table Used in 3rd Level Double-Tee Selection

![Table Image]

Using the 58-foot required member span, we selected a 10DT32+3 188-S double-tee for the 3rd level of our parking structure with 10 representing the member width in feet, DT representing double-tee, 32+3 representing the member height in inches with the additional 3-inch topping. The 188-S portion of the member description is represented in Figure 5. The type of strand, or rebar, used is no. 4 bar.

![Figure 5: Strand Pattern Designation]

This member supports up to 118 psf which is greater than sum of the calculated required load of 96 psf plus the additional 15 psf specified in 3.3.1 of the PCI Handbook.
2nd Level Double-Tee Member Selection

Table 3: PCI Loading Table Used in 2nd Level Double-Tee Selection

Using the 58-foot required member span, we selected a 10DT32+3 148-S double-tee for the 2nd level of our parking structure. The only difference between the member selected for the 3rd level and the member selected for the 2nd level is the number of strands being equal to 14 instead of 18. This member supports up to 94 psf which is greater than the sum of the calculated required load of 64 psf plus the additional 15 psf specified in 3.3.1 of the PCI Handbook. Figure 1 in Appendix I shows the section dimensions of the selected double-tees.
3.2 Analysis and Selection of Precast Beams

When designing the beams for our parking structure, we used a multi-step process of first using the PCI loading tables (Tables 4-7 in this report) for inverted T-beams and L-beams to select the dimensions of the precast concrete beams we will design for. Then, using SAP2000 software, we used those dimensions to determine the minimum required reinforcing steel necessary to withstand the loads present on the structure. One important thing to note is that the stems from the double-tees shown in Figure 1 of Appendix I sit directly on the extruding portions of the beams, shown in Figure 2 of Appendix I, and the loads are transferred as point loads to the beams rather than distributed loads. However, when selecting the appropriate beam section dimensions from the PCI handbook, we treated the loads on the beams as distributed loads, because the load capacities given in the PCI loading tables are in lb/ft. When designing for the reinforcing steel, we did treat each double-tee stem as point loads acting directly on the beams for better accuracy. Overall, it was determined that our parking structure would require 4 different types of beams. It is important to note that, for the loading tables used in this section, Chapter 3 section 3.1 of the PCI Design Handbook states that “For beams, 50% of the capacity shown in the load table is assumed as superimposed dead load.” This means that one-half of the load shown in the PCI load tables must be greater than the entirety of the calculated required controlling loads. All beams, with the exception of the beams shown in Figure 6 are subjected to one half of the force from the 58-foot span width of the double-tee stems. The beams highlighted in red in Figure 6 are subjected to the entire force of the 58-foot span width of the double-tee stems.

Figure 6: Inverted T-Beam Sections Subjected to Full Span Width Loading
3\textsuperscript{rd} Level L-Beams Not Highlighted in Figure 6

Required loading = 96 psf x 58/2’ = 2784 lb/ft

Table 4: PCI Loading Table Used in Selection of 3\textsuperscript{rd} Level L-Beam

<table>
<thead>
<tr>
<th>Designation</th>
<th>Number strand</th>
<th>$y$, in.</th>
<th>Span, ft</th>
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<td>26LB20</td>
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<tr>
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Using the 40-foot beam span length, we selected a 26LB40 section, with the 26 representing the bottom beam width in inches, LB representing that the beam is a L-beam, and 40 representing the overall beam height in inches. The overall dimensions can be found in Figure 2 in appendix I with $h_1$ being 24 inches and $h_2$ being 16 inches. These beams will be referred to as “3\textsuperscript{rd} Level L-Beams” as we move forward. This beam section is able to carry one half of the 5720 lb/ft shown in Table 4, which is greater than the 2784 lb/ft that it will be required to carry. However, these are just preliminary numbers used to get a general idea of the overall beam dimensions necessary to meet the loading requirements. In section 3.3.1 we will design custom reinforcing steel for the beam dimensions specified above and give exact loading capacities and requirements.
2nd Level L-Beams Not Highlighted in Figure 6

Required loading = 64 psf × 58/2’ = 1856 lb/ft

Using the 40-foot beam span length, we selected a 26LB36 section, with the 26 representing the bottom beam width in inches, LB representing that the beam is a L-beam, and 36 representing the overall beam height in inches. The overall dimensions can be found in Figure 2 of Appendix I with h₁ being 24 inches and h₂ being 12 inches. These beams will be referred to as “2nd Level L-Beams” as we move forward. This beam section is able to carry one half of the 4610 lb/ft shown in Table 5, which is greater than the 1856 lb/ft that it will be required to carry. However, these are just preliminary numbers used to get a general idea of the overall beam dimensions necessary to meet the loading requirements. In section 3.3.1 we will design custom reinforcing steel for the beam dimensions specified above and give exact loading capacities and requirements.

Table 5: PCI Loading Table Used in Selection of 2nd Level L-Beam

<table>
<thead>
<tr>
<th>Designation</th>
<th>Number strand</th>
<th>( y ), in.</th>
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<th>18</th>
<th>20</th>
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</tbody>
</table>
| Table 5: PCI Loading Table Used in Selection of 2nd Level L-Beam

Using the 40-foot beam span length, we selected a 26LB36 section, with the 26 representing the bottom beam width in inches, LB representing that the beam is a L-beam, and 36 representing the overall beam height in inches. The overall dimensions can be found in Figure 2 of Appendix I with \( h_1 \) being 24 inches and \( h_2 \) being 12 inches. These beams will be referred to as “2nd Level L-Beams” as we move forward. This beam section is able to carry one half of the 4610 lb/ft shown in Table 5, which is greater than the 1856 lb/ft that it will be required to carry. However, these are just preliminary numbers used to get a general idea of the overall beam dimensions necessary to meet the loading requirements. In section 3.3.1 we will design custom reinforcing steel for the beam dimensions specified above and give exact loading capacities and requirements.
3rd Level Inverted T-Beams Highlighted in Figure 6

Required loading = 96 psf x 58’ = 5568 lb/ft

Using the 40-foot beam span length, we selected a 40IT44 section, with the 40 representing the bottom beam width in inches, IT representing that the beam is an inverted T-beam, and 44 representing the overall beam height in inches. The overall dimensions can be found in Figure 2 of Appendix I with $h_1$ being 28 inches and $h_2$ being 16 inches. These beams will be referred to as “3rd Level T-Beams” as we move forward. This beam section is able to carry one half of the 9950 lb/ft shown in Table 6, which is actually slightly less than the 5568 lb/ft that it will be required to carry. Despite this, we decided to proceed with these beam section dimensions, as it is the largest beam section provided in the PCI loading table at the required span length. Later, we will accommodate for the lower than required strength with additional reinforcement steel for the beam dimensions specified above. This is calculated in section 3.3.1.

---

Table 6: PCI Loading Table Used in Selection of 3rd Level Inverted T-Beam

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</tr>
</tbody>
</table>
2nd Level T-Beams Highlighted in Figure 6

Required Loading = 64 psf x 58’ = 3712 lb/ft

Table 7: PCI Loading Table Used in Selection of 2nd Level Inverted T-Beam

| Designation | Number strand | Span, ft | 20 | 22 | 24 | 26 | 28 | 30 | 32 | 34 | 36 | 38 | 40 | 42 | 44 | 46 | 48 | 50 |
|-------------|---------------|----------|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|
| 40IT20      | 18            | 2.22     | 8420| 8680| 5680| 4760| 4380| 3440| 2980| 2560| 2220| 1940| 1680| 1480| 1310| 1150| 1010|   |
| 40IT24      | 22            | 2.67     | 9990| 8280| 6960| 5650| 5050| 4600| 3800| 2890| 2540| 2240| 1980| 1750| 1550| 1380|   |
| 40IT28      | 26            | 3.08     | 9670| 8200| 7070| 6120| 5130| 4670| 4110| 3640| 3230| 2870| 2560| 2290| 2050|   |
| 40IT32      | 30            | 3.33     | 9520| 8260| 7220| 6350| 5610| 4980| 4440| 3970| 3580| 3190| 2880|   |
| 40IT36      | 32            | 3.50     | 9410| 8290| 7430| 6550| 5840| 5230| 4710| 4250| 3870| 3500|   |
| 40IT40      | 38            | 4.32     | 8940| 7120| 6380| 5780| 5200| 4700|   |
| 40IT44      | 40            | 4.40     | 9950| 8910| 8200| 7330| 6550| 5940|   |
| 40IT48      | 44            | 4.87     | 9690| 8720| 7910| 7190|   |
| 40IT52      | 46            | 5.05     | 9490| 8640|   |

Using the 40-foot beam span length, we selected a 40IT40 section, with the first 40 representing the bottom beam width in inches, IT representing that the beam is an inverted T-beam, and the second 40 representing the overall beam height in inches. The overall dimensions can be found in Figure 2 of Appendix I with $h_1$ being 24 inches and $h_2$ being 16 inches. These beams will be referred to as “2nd Level T-Beams” as we move forward. This beam section is able to carry one half of the 7960 lb/ft shown in Table 7, which is greater than the 3712 lb/ft that it will be required to carry. However, these are just preliminary numbers used to get a general idea of the overall beam dimensions necessary to meet the loading requirements. In section 3.3.1 we will design custom reinforcing steel for the beam dimensions specified above and give exact loading capacities and requirements.
3.2.1 SAP2000 Loading Calculations

When designing the reinforcing steel used in the beams of our parking structure, we used SAP2000 software and broke our parking structure down into two-dimensional frames. When looking at Figure 6 in the section above, you can see that there are 4 individual frames that span the length of our parking structure. We refer to the two inner frames as “Inside Frame” and the two outer frames as “Outside Frame” and assume that the two inner and two outer frames are identical respectively. The frame configurations, as well as the designated beam type definitions, can be seen in Figures 3 and 4 of Appendix I. It should be noted that SAP2000 only allows for the input of rectangular beam sections, so the beam sections input into SAP2000 exclude the extruding portions of the beam sections shown in Figure 2 of Appendix I. It is assumed, however, that these additional concrete portions would only further contribute to the structural capacity of the beams. Figures 5 and 6 in Appendix I show how all loads calculated in this section are applied over the entirety of the two different frames. The highlighted loads in this section indicate the loads that are directly applied to the frames as shown in Figures 5 and 6 of Appendix I.

Vertical Loading on 3rd Level L-Beams

As discussed above, the loads applied on the beams are applied as point loads from the stems of the double-tees that rest on the extruding portions of the beams. In addition, there is a wall load resulting from a 3-foot high, 1-foot-thick brick wall that spans around the entirety of our parking structure and on both sides of the inner ramp. We treat this load as a distributed load across the beams. We chose not to factor this load as it is very small relative to the load applied from the double-tee. It is assumed that factoring those loads will more than account the additional wall load factoring.

Brick Weight = 5 lb

Brick Volume = .04167 ft³

Wall Load = 3’(1’)/.04167 ft³ x 5 lb = \(0.36 \text{ k/ft}\)

Snow Load = 35 psf

Live Load = 40 psf

Wind Load = 7.83 psf (Figure 1 Appendix II)

Dead Load from Double-Tee = 102 psf

\[ \text{LC3} = 1.2(102) + 1.6(35) + 40 = 218 \text{ psf} \times 5 \;' \times 29' = 31.6 \text{ k per double-tee stem} \]
**Vertical Loading on 2\textsuperscript{nd} Level L-Beams**

Wall Load = \(0.36 \text{ k/ft}\)

Live Load = 40 psf

Dead Load from Double-Tee = 102 psf

LC2 = 1.2(102) = 1.6(40) = 186 psf x 5' x 29' = 27.1 k per double tee stem

**Vertical Loading on 3\textsuperscript{rd} Level T-Beam**

The loading on the 3\textsuperscript{rd} level T-beam will be twice that of the 3\textsuperscript{rd} level L-beam and will be equal to 63.2 k. These beams are however not subject to wall loading.

**Vertical Loading on 2\textsuperscript{nd} Level T-Beam**

The loading on the 2\textsuperscript{nd} level T-beam will be twice that of the 2\textsuperscript{nd} level L-beam and will be equal to 54.2 k. These beams are however not subject to wall loading.

**Horizontal Loading on Frames**

Horizontal wind load calculations can be found in Figure 1 of Appendix II
3.2.2 Design of Reinforcing Steel

All beams in our parking structure are simply supported, meaning that they are supported by a pin-roller connection combination. This means that all maximum moments occur at the center of the beam spans and all maximum shears occurs at the ends of the beams. Figures 7 and 8 in Appendix I show the shear and moment distributions on the frames with blue colors representing positive values and red colors representing negative values. For simplicity purposes, we only considered Grade 60 no. 9 bars for use in the flexural steel design. No. 4 bars were used for all shear steel design, however, the T-Beams required Grade 60 steel while the L-Beams only required Grade 40 steel.

Reinforcement Design of 3rd Level L-Beams

The location and applied loadings on the 3rd Level L-Beams can be found in Figures 3-6 of Appendix I. Figures 1 and 2 of Appendix III show the AutoCAD drawings for the flexural and shear steel design of the 3rd Level L-Beams with all applicable dimensions.

Minimum Required Flexural Steel from SAP2000

\[
As > .068209 \text{ ft}^2 \times 12^2 = 9.82 \text{ in}^2
\]

Select 12 no. 9 bars

\[
As = 12 \text{ in}^2 > 9.82 \text{ in}^2
\]

In this case we could have used 10 no. 9 bars, but we felt that was too close to the minimum requirement, so we selected 2 additional bars to be cautious.

Flexural Steel Design

\[
W_u = (.36 + .15(1.5)(3.33)) = 1.11 \text{ k/ft}
\]

\[
V_u = \frac{31.7(8)}{2} + \frac{1.11(40)}{2} = 149 \text{ k}
\]
Figure 7 shows the shear force distribution along the 3rd Level L-Beam. The figure only shows half of the beam span but, because it is a simply supported beam with uniform loading across its span, the other half of the beam span would have values equal and opposite to the values shown in Figure 7. The maximum moment is known to be equal to the area under the shear force diagram. The calculated required flexural capacity and calculations verifying that the 3rd Level L-Beams meet all ACI requirements for flexural steel can be found in Figure 2 of Appendix II.

Shear Steel Design

Figure 8 shows the factored shear force used for calculation of required shear steel. The calculations verifying that the 3rd Level L-Beams meet all ACI requirements for shear steel can be found in Figure 3 of Appendix II.
Reinforcement Design of 2nd Level L-Beams

The location and applied loadings on the 2nd Level L-Beams can be found in Figures 3-6 of Appendix I. Figures 3 and 4 of Appendix III show the AutoCAD drawings for the flexural and shear steel design of the 2nd Level L-Beams with all applicable dimensions.

Minimum Required Flexural Steel from SAP2000

As > .067028 ft² x 12² = 9.65 in²

Select 12 no. 9 bars

As = 12 in² > 9.65 in²

In this case we could have used 10 no. 9 bars, but we felt that was too close to the minimum requirement, so we selected 2 additional bars to be cautious.

Flexural Steel Design

\[ W_u = (0.36 + 0.15(1.5)(3)) = 1.035 \text{ k/ft} \]

\[ V_u = 27.1(8)/2 + 1.035(40)/2 = 129 \text{ k} \]

![Shear Force Distribution](image)

*Figure 9: Shear Force Distribution on 2nd Level L-Beam*
Figure 9 shows the shear force distribution along the 2\textsuperscript{nd} Level L-Beam. The figure only shows half of the beam span but, because it is a simply supported beam with uniform loading across its span, the other half of the beam span would have values equal and opposite to the values shown in Figure 9. The maximum moment is known to be equal to the area under the shear force diagram. The calculated required flexural capacity and calculations verifying that the 2\textsuperscript{nd} Level L-Beams meet all ACI requirements for flexural steel can be found in Figure 4 of Appendix II.

**Shear Steel Design**

![Shear Force Diagram](image)

**Figure 10: Factored Shear Force Distribution on 2\textsuperscript{nd} Level L-Beam**

Figure 10 shows the factored shear force used for calculation of required shear steel. The calculations verifying that the 2\textsuperscript{nd} Level L-Beams meet all ACI requirements for shear steel can be found in Figure 5 of Appendix II.
Reinforcement Design of 3rd Level T-Beams

The location and applied loadings on the 3rd Level T-Beams can be found in Figures 3-6 of Appendix I. Figures 5 and 6 of Appendix III show the AutoCAD drawings for the flexural and shear steel design of the 3rd Level T-Beams with all applicable dimensions.

Minimum Required Flexural Steel from SAP2000

\[ As > 0.11541 \text{ ft}^2 \times 12^2 = 16.62 \text{ in}^2 \]

Select 18 no. 9 bars

\[ As = 18 \text{ in}^2 > 16.62 \text{ in}^2 \]

Flexural Steel Design

\[ W_u = 0.15(2)(3.667) = 1.1 \text{ k/ft} \]

\[ V_u = 31.7(16)/2 + 1.1(40)/2 = 275 \text{ k} \]

Figure 11 shows the shear force distribution along the 3rd Level T-Beam. The figure only shows half of the beam span but, because it is a simply supported beam with uniform loading across its span, the other half of the beam span would have values equal and opposite to the values shown in Figure 11. The maximum moment is known to be equal to the area under the shear force diagram. The calculated required flexural capacity and calculations verifying that the 3rd Level T-Beams meet all ACI requirements for flexural steel can be found in Figure 6 of Appendix II.
Figure 12 shows the factored shear force used for calculation of required shear steel. The calculations verifying that the 3rd Level T-Beams meet all ACI requirements for shear steel can be found in Figure 7 of Appendix II.
Reinforcement Design of 2nd Level T-Beams

The location and applied loadings on the 2nd Level T-Beams can be found in Figures 3-6 of Appendix I. Figures 7 and 8 of Appendix III show the AutoCAD drawings for the flexural and shear steel design of the 2nd Level T-Beams with all applicable dimensions.

Minimum Required Flexural Steel from SAP2000

\[ A_s > 0.11162 \text{ ft}^2 \times 12^2 = 16.1 \text{ in}^2 \]

Select 18 no. 9 bars

\[ A_s = 18 \text{ in}^2 > 16.1 \text{ in}^2 \]

Flexural Steel Design

\[ W_u = 0.15(2)(3.333) = 1 \text{ k/ft} \]

\[ V_u = 27.1(16)/2 + 1(40)/2 = 237 \text{ k} \]

Figure 13: Shear Force Distribution on 2nd Level T-Beam

Figure 13 shows the shear force distribution along the 2nd Level T-Beam. The figure only shows half of the beam span but, because it is a simply supported beam with uniform loading across its span, the other half of the beam span would have values equal and opposite to the values shown in Figure 13. The maximum moment is known to be equal to the area under the shear force diagram. The calculated required flexural capacity and calculations verifying that the 2nd Level T-Beams meet all ACI requirements for flexural steel can be found in Figure 8 of Appendix II.
Figure 14 shows the factored shear force used for calculation of required shear steel. The calculations verifying that the 2nd Level T-Beams meet all ACI requirements for shear steel can be found in Figure 9 of Appendix II.
Spandrel Beam Design

In order to support the wall loads present around the outside of our parking structure, we designed spandrel beams to be placed at ends of the parking structure, highlighted in red in Figure 15. We used a beam height of 24 inches, a beam width of 18 inches, and a beam length of 58 feet. These beams were designed by hand and SAP2000 software was not used in the process. The calculations showing required flexural capacity and shear reinforcement and calculations verifying that the spandrel beams meet all ACI requirements for flexure and shear can be found in Figures 10 and 11 of Appendix II respectively. The final flexural and shear AutoCAD drawings, along with all relevant dimensions can be found in figures 9 and 10 of Appendix III.

Figure 15: Spandrel Beam Locations
3.3 Analysis and Selection of Precast Columns

Using the applied loads on the outside and inside frames shown in Figures 5 and 6 of Appendix I respectively, we began a trial-and-error process in SAP2000 to select the proper column sizes for our parking structure. For simplicity, we only considered square column options. The process included inputting set column dimension for each individual column, analyzing the frame as a whole, and then using the design feature in SAP2000 to design the column steel reinforcement. If the selected size of the columns were not adequate for the applied loads they would appear red. We continued this process until the SAP2000 software determined that all column sizes were adequate. The results of this process and the location of the selected column sizes are shown below. All columns are short columns and are not subjected to slenderness effects.

**18”x 18” Columns**

Figure 16 shows the locations of the columns with the smallest selected dimensions. These columns are subjected to smaller applied loads because of their corner locations, and thus do not require as large of a gross area.

![Figure 16: 18”x 18” Column Locations](image-url)
24”x 24” Columns

Figure 17 shows the locations of the columns with the largest selected dimensions. These columns are subjected to larger loads than any other columns in the structure due to their proximity to the end-bays. They are required to carry both L-Beams and the end-bay T-beams which support much larger loads than any other beams in our parking structure.
20”x 20” Columns

Figure 18 shows the most typical column size. This column size makes 24 out of the 32 columns in the entire structure. The columns not circled are all subjected to identical loadings. The columns that are circled are all subjected to identical loadings respectively, but differ from the non-circled columns in that they carry half of the T-beam loads. In contrast, the non-circled columns carry 1 full L-beam load. These loads are identical, because the T-beams carry twice the load as the L-beams, thus, ½ of a T-Beam load equals 1 full L-beam load. However, the circled columns carry the spandrel beams. The loads resulting from these beams are small respective to the other loads, but do present an additional load on those columns. Given that the spandrel beam loads are small relative to the L-beam and T-beam loads, all columns highlighted in figure 18, both circled and non-circled, are subjected to similar overall loadings. The circled columns are simply subjected to slightly larger loadings than the non-circled columns resulting from the spandrel beams.

Figure 18: 20”x 20” Column Locations
3.3.1 Design of Reinforcing Steel

For simplicity, we decided to design all columns with the 3 different column sizes identically, this meaning all 18”x 18” columns have identical reinforcement, all 20”x 20” columns have identical reinforcement, and all 24”x 24” columns have identical reinforcement. To do this, we used SAP2000 to identify the specific columns for each different column size that are subjected to the largest maximum moments and axial loads. These columns, their respective maximum moments, and the reinforcement we selected for them are shown below.

18”x 18” Columns

Because loading and placement of all 18”x 18” columns are identical and symmetrical respectively, the specific location of the column with the maximum axial force and bending moment is quite literally dependent on whichever way the wind blows and, thus, is not shown. However, the maximum axial force taken from SAP2000 is equal to 181 k, and the maximum bending moment is equal to 19 k-ft. Using the PCI handbook interaction curve for “Precast, Reinforced Columns,” we determined the required reinforcing steel as follows.

![Figure 19: 18”x 18” PCI Interaction Curve for Precast, Reinforced Columns](image)

Using the interaction curve shown in figure 19, we used the maximum axial force (y axis) and maximum moment (x-axis) and determined the point at which those values intersected. In this case, it was less than the minimum required $\rho$ value of 1% so we selected the lowest level of reinforcement greater than 1%, 4 no. 9 bars, provided in the table. For the column ties, we selected no. 4 ties which met the standard for ACI section 25.7.2.2 and used ACI section 25.7.2.1 to determine a maximum tie spacing equal to the least of:
16(d_{ℓ} longitudinal bar) = 16(1.128) = 18”

48(d_{b} of tie bar) = 48(1/2) = 24”

Least dimension of member = 18”

Though 18 inches was the maximum determined value, we decided to select no. 4 ties spaced at 16 inches to be cautious. The AutoCAD drawings for the longitudinal steel and steel ties for the 18”x 18” columns can be found in Figure 11 of Appendix III.

24”x 24” Columns

Because loading and placement of all 24”x 24” columns are also identical and symmetrical respectively, the specific location of the column with the maximum axial force and bending moment is conditions dependent and is not shown. However, the maximum axial force taken from SAP2000 is equal to 436 k, and the maximum bending moment is equal to 51 k-ft. Using the PCI handbook interaction curve for “Precast, Reinforced Columns,” we determined the required reinforcing steel as follows.

Figure 20: 24”x 24” PCI Interaction Curve for Precast, Reinforced Columns
Using the interaction curve shown in figure 20, we used the maximum axial force (y axis) and maximum moment (x-axis) and determined the point at which those values intersected. In this case, it was less than the minimum required ρ value of 1%. However, because this particular column is fairly large, we decided not to select the 4 no. 11 bars shown as the minimum required reinforcement, but rather to select 8 no. 8 bars. This gave us a ρ value of 1.1% but provided more reasonable spacing of the longitudinal bars. For the column ties, we selected no. 4 ties which met the standard for ACI section 25.7.2.2 and used ACI section 25.7.2.1 to determine a maximum tie spacing equal to the least of:

\[ 16(d_b \text{ longitudinal bar}) = 16(1) = 16'' \]
\[ 48(d_b \text{ of tie bar}) = 48(1/2) = 24'' \]

Least dimension of member = 24”

In this case, we did elect to choose no. 4 bars spaced at 16 inches. The AutoCAD drawings for the longitudinal steel and steel ties can be found in Figure 12 of Appendix III.

**20” x 20” Columns**

In the case of the 20” x 20” columns, certain columns are subjected to slightly different loadings than others, and thus the location of the column subjected to the maximum axial force and moment is dependent on the difference in loading. Figure 9 of Appendix I shows where the 20” x 20” column with the greatest maximum moment and axial force is located. As you can see in the figure, the leeward outside column (highlighted in red) in the inside frame is subjected to the largest axial force and maximum moment. These values, taken from SAP2000, are equal to 340 k and 37 k-ft respectively. Using the PCI handbook interaction curve for “Precast, Reinforced Columns,” we determined the required reinforcing steel as follows.
Using the interaction curve shown in figure 21, we used the maximum axial force (y-axis) and maximum moment (x-axis) and determined the point at which those values intersected. In this case, it was less than the minimum required ρ value of 1% so we selected the lowest level of reinforcement greater than (or equal to) 1%, 4 no. 9 bars, provided in the table. For the column ties, we selected no. 4 ties which met the standard for ACI section 25.7.2.2 and used ACI section 25.7.2.1 to determine a maximum tie spacing equal to the least of:

16\((d_b\text{ longitudinal bar}) = 16(1.128) = 18''

48\((d_b\text{ of tie bar}) = 48(1/2) = 24''

Least dimension of member = 20''

Though 18 inches was the maximum determined spacing, we decided to select no. 4 ties spaced at 16 inches to be cautious. The AutoCAD drawings for the longitudinal steel and steel ties can be found in Figure 13 of Appendix III.
### 3.4 Structural Design of Ramp

We decided to design the ramp in our parking structure as a separate structural system. The upper-level ramp connecting the 2\textsuperscript{nd} and 3\textsuperscript{rd} levels uses the same precast double-tee, 10DT32+3 188-S, as the 3\textsuperscript{rd} level precast double-tee selected above, as they are subjected to similar loading. The lower-level ramp connecting the ground level and 2\textsuperscript{nd} level uses the same precast double-tee, 10DT32+3 148-S, as the 2\textsuperscript{nd} level precast double-tee as, they are subjected to similar loading. Unlike the rest of our parking structure, however, the stems from the precast double tees do not sit on perpendicular beams, but rather they sit directly on a system of columns, spaced at 5-foot increments in conjunction with the 5-foot double-tee stem spacing, along the length of the ramp. This system is shown below in figure 22.

![Ramp Layout](image)

**Figure 22: Ramp Layout**

The two ramps connecting the ground level to the 2\textsuperscript{nd} Level and the 2\textsuperscript{nd} Level to the 3\textsuperscript{rd} Level span a length of 200 feet and rise a height of 12 feet each. This gives an angle of inclination equal to \( \tan^{-1}(12/200) \approx 3.434 \) degrees which is less than the 6-degree maximum inclination that allows parking to be available on a ramp. This allows us to have parking spaces along the ramp, increasing its overall parking capacity. Figure 23 below shows the vertical and lateral loading that the columns are subjected to as a result of the inclined ramp. Figure 23 also shows 2 horizontal
beams connecting the two columns. These beams exist to transfer the lateral loads and keep the structure from swaying. For the selection of these beams, we used a trial-and-error process in SAP2000 to minimize deformation while using an economical amount of material. The final determination was to use one beam, no more than 6 feet from the top of the columns, and no more than 12 feet from the bottom of the columns. If this was not possible, a second beam would be introduced to meet the requirements. This meant that any column-set taller than 18 feet would have 2 connecting beams, and any column-set shorter than 18 feet would only have 1 connecting beam. We refer to the connecting beam-column sets as H-Columns. Figure 23 shows the H-Column configuration with the tallest columns. Their heights are equal to 24 feet, and they support the double-tee stems located at the top of both ramp sections. The beams are spaced at 6 feet apart with the top beam being 6 feet from the top of the columns in accordance with the design standards listed above. It is assumed that this configuration is subjected to the most extreme bending and loading conditions and any columns shorter, with the same dimensions and reinforcing steel, will be adequate.

Figure 23: H-column Loading
Using SAP2000 we determined that 14” x 14” columns and 20” x 14” beams would be adequate to support the applied loads from the double-tee stems. For the H-Pile columns we selected 4 no. 7 bars, $A_s = 4(0.6) = 2.4$ in$^2$, for the longitudinal reinforcement which was greater than the minimum reinforcing steel required according to SAP2000 of $2.02$ in$^2$. The maximum longitudinal reinforcement tie spacing is equal to the least of:

- $16(d_b \text{ longitudinal bar}) = 16(7/8) = 14”$
- $48(d_b \text{ of tie bar}) = 48(1/2) = 24”$
- Least dimension of member = 14”

So, we selected no. 4 ties with a tie spacing of 14 inches. For the 20” x 14” connecting beams we selected 2 no. 7 bars both in the top and bottom of the beams due to their fixed connections with the columns and subsequent positive and negative moments. The $A_s = 2(0.6)$ in$^2$ was greater than the minimum requirement from SAP2000 of $A_s = 0.866$ in$^2$ in both the top and bottom of the beam. Shear reinforcement in the beams was not required as the maximum $V_u$ in the beams obtained from SAP2000 was equal to 10.413 k which when divided by $\Phi = .75$ gave a value less than $V_c/2$.

$$V_c/2 = 2(5000)(14)(17.5)/2 = 17.3 \text{ k} < 10.413/.75 = 13.9 \text{ k}$$

Figure 14 of Appendix III shows the AutoCAD drawings for all reinforcement and relevant dimensions used in the H-Columns.
As you can see in Figure 24, the maximum deformation of the H-Column is 0.0119 feet or 0.143 inches which is a very reasonable deformation. It can also be assumed that since the ramp resides within the outer structure that it will be braced from lateral deformation by that outer structure as those frames have slightly smaller deformation as you will see in the following section.

Figure 24: Lateral Deformation of H-Columns
3.5 Selection and Configuration of Bracing

In order to support our parking structure against lateral wind forces, we provided braces at certain locations along the length of the frames. We selected the brace locations using a trial-and-error process in SAP2000. We first used the AISC Steel Construction Manual to select the lightest W-A992 steel member available at the required unbraced length, and then checked for adequacy in SAP2000. It should be noted that given the short height of our parking structure, in conjunction with the large base, the overall lateral forces on our parking structure are minimal, and thus it was determined that the lightest available member was adequate. Using table 6-2 in the AISC manual, and the unbraced brace length of \( L_c = (40^2 + 12^2)^{1/2} = 41.76' \), it was determined that a W10x49 member made of A992 steel was the lightest available member at the length we required. We then used several different bracing arrangements within the individual frame sections and came up with the following arrangement. The location, overall deformation, and adequacy check of both the inside and outside frames are shown in figures 25 and 26 respectively.

![Figure 25: Deformation and Adequacy Check for Inside Frame](image)
As you can see, the largest deformation in the inside frame occurs near the two-outside column/beam connections. This overall max deformation is roughly equal to 0.0077 feet which equates to 0.0924 inches. This is a very reasonable number in concrete design. The figure also shows that SAP2000 determined the frame to be stable as there were 0 eigenvalues found during analysis.

Figure 26 shows that, in the outside frames, the max overall deformation occurs along the 3rd level interior beams and is approximately equal to 0.0063 feet or 0.0756 inches. Again, this is a very reasonable number. The figure also shows that SAP2000 found 0 eigenvalues in the outside frames and thus determined them to be stable as well.
The selected bracings only laterally support our parking structure along its 280-foot length. In order to laterally support our parking structure along its 174-foot width, we will be placing 4 36-foot-long concrete shear walls in the locations indicated in figure 27 below. These walls will ensure that our parking structure is stable in all directions, and they will not interrupt traffic flow in any way. It is also worth noting that the staircases located in the corners of our parking structure will also help to laterally brace it.

Figure 27: Shear Wall Locations
4 Geotechnical Analysis

4.1 Analysis of Soil Conditions

For the determination of the site soil conditions, we were provided a geotechnical report used in the construction of the Haworth College of Business. The site of this report was approximately 700 feet from the center of our construction site, so it is assumed that the same soil conditions exist at both sites. The structure to be constructed for the provided geotechnical report was also a 3-level building so recommendations from the report are assumed to be applicable to our parking structure.

Soil conditions

There are essentially two different soil layers at our construction site. The soil boring logs in the geotechnical report linked in the “Resources and References” section of this report show 3 layers, however, the only difference between the bottom and middle layers is that the bottom layer is a fine to medium sand, while the middle layer is a fine to coarse sand. Both layers are natural sand deposits, light brown, medium dense, damp, and are poorly graded sands (SP), so we consider these two layers to be the same for the purpose of calculating soil bearing capacity. The top layer is a mixed sand and clay fill. This top layer was determined in the geotechnical report to, “be unsuitable in their present condition for support of foundations.” As a result, we determined the most optimal foundational system would be to use driven piles that would allow the weight of our parking structure to be transferred into the natural sand layer. Using the 8 soil boring logs provided to us, we calculated an average layer depth to create one uniform profile to be used. The top sand-clay fill is 12 feet deep with an effective unit weight, $\gamma'$, of 122 lb/ft$^3$, angle of friction, $\phi'$, of 35 degrees, and an average N value of 10 blows/ft. The bottom natural sand layer extends to a depth of 35’. This is the minimum depth of the provided soil boring logs. We use this as the maximum depth we will drive our piles to. The natural sand layer has a $\gamma'$ of 120 lb/ft$^3$, a $\phi'$ of 37.9 degrees, and an average N value of 20 blows/ft. The calculations for these values are shown in Table 8. Figure 10 of Appendix I shows the overall soil profile for our site.

<table>
<thead>
<tr>
<th>Boring Log</th>
<th>Depths of Top Layer</th>
<th>Average N of Top Layer</th>
<th>Average N of Bottom Layer</th>
</tr>
</thead>
<tbody>
<tr>
<td>B1</td>
<td>14</td>
<td>9</td>
<td>20</td>
</tr>
<tr>
<td>B2</td>
<td>7</td>
<td>17.7</td>
<td>19.8</td>
</tr>
<tr>
<td>B3</td>
<td>13</td>
<td>5.75</td>
<td>23.8</td>
</tr>
<tr>
<td>B4</td>
<td>23</td>
<td>7.8</td>
<td>20.2</td>
</tr>
<tr>
<td>B5</td>
<td>12</td>
<td>9</td>
<td>21.3</td>
</tr>
<tr>
<td>B6</td>
<td>9</td>
<td>6</td>
<td>17.2</td>
</tr>
<tr>
<td>B7</td>
<td>10</td>
<td>15.5</td>
<td>20.2</td>
</tr>
<tr>
<td>B8</td>
<td>6</td>
<td>9.5</td>
<td>18.6</td>
</tr>
<tr>
<td>average values</td>
<td>11.75</td>
<td>10.03125</td>
<td>20.1375</td>
</tr>
</tbody>
</table>

Table 8: Layer Depth and N-Value Calculations

round to 12” round to 10 blows/ft round to 20 blows/ft
\( \gamma' \) and \( \phi' \) for Top Sand and Clay Fill Layer

Using Table 9 from Bowels, *Foundation Analysis*, we determined that, for a cohesive soil with an N value of 10, the \( \gamma' \) to be equal to 122 lb/ft\(^3\).

Using Table 10 from geotechdata.info we determined that, for a clayey sand, \( \phi' \) is approximately equal to 30 – 40 degrees. We decided to select an average value of 35 degrees.

\[
\gamma' = 120 + \frac{10-8}{32-8}(140-120) = 122 \text{ lb/ft}^3
\]

\( \gamma' \) and \( \phi' \) for Bottom Natural Sand Layer

Using Table 11 from Bowels, *Foundation Analysis*, we determined that, for granular soils with an N value of 20, the \( \gamma' \) to be equal to 120 lb/ft\(^3\).

\[
\gamma' = 120 + \frac{10-8}{32-8}(140-120) = 122 \text{ lb/ft}^3
\]
\[ \gamma' = 110 + \frac{20-10}{30-10}(130-110) = 120 \text{ lb/ft}^3 \]

To calculate \( \phi' \) for the bottom natural sand layer, we used Skempton’s equation for calculating \( N_{60} \) and Hatanaka and Uchida’s equation relating \( N_{60} \) to \( \phi' \). Table 12 show the values for the factors used in Skempton’s equation.

**Table 12: Values Used in Skempton’s Equation**

| Factor                  | Equipment Variables | Value  
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Borehole diameter</td>
<td>65–115 mm (2.5–4.5 in)</td>
<td>1.00</td>
</tr>
<tr>
<td>factor, ( C_b )</td>
<td>150 mm (6 in)</td>
<td>1.05</td>
</tr>
<tr>
<td></td>
<td>200 mm (8 in)</td>
<td>1.15</td>
</tr>
<tr>
<td>Sampling method factor</td>
<td>Standard sampler</td>
<td>1.00</td>
</tr>
<tr>
<td>factor, ( C_s )</td>
<td>Sampler without liner (not recommended)</td>
<td>1.20</td>
</tr>
<tr>
<td>Rod length factor</td>
<td>3–4 m (10–13 ft)</td>
<td>0.75</td>
</tr>
<tr>
<td>factor, ( C_r )</td>
<td>4–6 m (13–20 ft)</td>
<td>0.85</td>
</tr>
<tr>
<td></td>
<td>6–10 m (20–30 ft)</td>
<td>0.95</td>
</tr>
<tr>
<td></td>
<td>&gt;10 m (&gt;30 ft)</td>
<td>1.00</td>
</tr>
</tbody>
</table>

Borehole diameter = 2”

Standard Sampler

Rod length > 30’

USA Safety Donut

\[ N_{60} = 0.55(1)(1)(20)/0.6 = 18.33 \text{ round down to } 18 \]

\[ \phi' = (20 \times 18)^{0.5} + 20 = 37.9 \text{ degrees.} \]
4.2 Design of Driven Piles

The force that the piles will be required to carry is equal to the applied loadings on the structure and the weight of all structural components that are transferred into the given piles. For the driven pile design, we selected a factor of safety equal to 2, so the piles must be able to carry at least twice the load that is applied to them. The calculations for the toe bearing capacity of the bottom natural sand layer, of which the driven piles will be embedded into, can be found in Figure 12 of Appendix II.

18”x 18” Column Pile Design

\[ P_u = 27.1(4) + 31.7(4) + .36(20+29) + .15(29)(2)(1.5) + .15(20)(744/144) + .15(20)(848/144) + .15(18^2/144)(24) + .15(5.25^2)(2.5) = 321k \]

For the 18”x 18” columns we selected 14-inch diameter piles. The toe bearing capacity of the soil from these columns is equal to the soil bearing capacity times the cross-section area of the pile minus the weight of the pile. The 27.5-foot height of the pile represents the 35-foot depth of the pile minus the pile cap depth and the 5-foot depth of the ground level of our parking structure.

\[ P_t = 100\pi(7/12)^2 - .15\pi(7/12)^2(27.5) = 102.7 k \]

The side friction capacity of the soil is equal to the amount of upward friction that each layer of the soil profile in contact with the pile can carry. A table summary of the calculations for the side friction capacity of the soil is shown in Table 13.

<table>
<thead>
<tr>
<th>Layer (ft)</th>
<th>(K_0)</th>
<th>(\sigma_z) (psf)</th>
<th>(\beta)</th>
<th>(f_n) (psf)</th>
<th>(A_s) (sf)</th>
<th>(f_nA_s) (k)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-12</td>
<td>0.426</td>
<td>427</td>
<td>0.5</td>
<td>213.5</td>
<td>16.5</td>
<td>3.49</td>
</tr>
<tr>
<td>12-22</td>
<td>0.386</td>
<td>1454</td>
<td>0.532</td>
<td>774</td>
<td>36.7</td>
<td>28.4</td>
</tr>
<tr>
<td>22-35</td>
<td>0.386</td>
<td>2835</td>
<td>0.532</td>
<td>1508</td>
<td>47.6</td>
<td>71.7</td>
</tr>
<tr>
<td>Sum</td>
<td></td>
<td></td>
<td></td>
<td>103.6</td>
<td></td>
<td>386</td>
</tr>
</tbody>
</table>

\[ P_n = 102.7 + 103.6 = 206 k \]

By selecting 4 14-inch diameter piles, the total force that the piles will be able to carry is equal to 4(206) = 824 k which gives is a factor of safety equal to 2.57>2.
20"x 20" Column Pile Design

\[ P_u = 27.1(8) + 31.7(8) + .36(40) + .15(40)(744/144) + .15(40)(848/144) + .15(20^2/144)(24) + .15(7^2)(3.5) = 587k \]

For the 20”x 20” columns we selected 20-inch diameter piles. The toe bearing capacity of the soil from these columns is equal to the soil bearing capacity times the cross-section area of the pile minus the weight of the pile. The 26.5-foot height of the pile represents the 35-foot depth of the pile minus the pile cap depth and the 5-foot depth of the ground level of our parking structure.

\[ P_t = 100\pi(10/12)^2 - .15\pi(10/12)^2(26.5) = 209.5 k \]

The side friction capacity of the soil is equal to the amount of upward friction that each layer of the soil profile in contact with the pile can carry. A table summary of the calculations for the side friction capacity of the soil is shown in Table 14.

<table>
<thead>
<tr>
<th>Layer (ft)</th>
<th>( K_0 )</th>
<th>( \sigma_z ) (psf)</th>
<th>( \beta )</th>
<th>( fn ) (psf)</th>
<th>( As ) (sf)</th>
<th>fnAs (k)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-12</td>
<td>0.426</td>
<td>427</td>
<td>0.5</td>
<td>213.5</td>
<td>18.3</td>
<td>3.91</td>
</tr>
<tr>
<td>12-22</td>
<td>0.386</td>
<td>1454</td>
<td>0.532</td>
<td>774</td>
<td>52.4</td>
<td>40.6</td>
</tr>
<tr>
<td>22-35</td>
<td>0.386</td>
<td>2835</td>
<td>0.532</td>
<td>1508</td>
<td>68.1</td>
<td>103</td>
</tr>
<tr>
<td>Sum</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>147.5</td>
</tr>
</tbody>
</table>

\[ P_n = 209.5 + 147.5 = 357 k \]

By selecting 4 20-inch diameter piles, the total force that the piles will be able to carry is equal to \( 4(357) = 1428 \) k which gives is a factor of safety equal to \( 2.43>2 \).
**24”x 24” Column Pile Design**

\[ P_u = 27.1(12) + 31.7(12) + .15(20)(1216/144) + .15(20)(1312/144) + .15(20)(744/144) + .15(20)(848/144) + .15(24^2/144)(24) + .15(8^2)(5) = 854k \]

For the 24”x24” columns we selected 24-inch diameter piles. The toe bearing capacity of the soil from these columns is equal to the soil bearing capacity times the cross-section area of the pile minus the weight of the pile. The 25-foot height of the pile represents the 35-foot depth of the pile minus the pile cap depth and the 5-foot depth of the ground level of our parking structure.

\[ P_t = 100\pi(12/12)^2 - .15\pi(12/12)^2(25) = 301.7 \text{k} \]

The side friction capacity of the soil is equal to the amount of upward friction that each layer of the soil profile in contact with the pile can carry. A table summary of the calculations for the side friction capacity of the soil is shown in Table 15.

Table 15: Side Friction Calculation for 24” Driven Pile

<table>
<thead>
<tr>
<th>Layer (ft)</th>
<th>K₀</th>
<th>σ₀ (psf)</th>
<th>β</th>
<th>fn (psf)</th>
<th>As (sf)</th>
<th>fnAs (k)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-12</td>
<td>0.426</td>
<td>427</td>
<td>0.5</td>
<td>213.5</td>
<td>12.56</td>
<td>2.68</td>
</tr>
<tr>
<td>12-22</td>
<td>0.386</td>
<td>1454</td>
<td>0.532</td>
<td>774</td>
<td>62.83</td>
<td>48.6</td>
</tr>
<tr>
<td>22-35</td>
<td>0.386</td>
<td>2835</td>
<td>0.532</td>
<td>1508</td>
<td>81.68</td>
<td>123</td>
</tr>
<tr>
<td>Sum</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>174.3</td>
</tr>
</tbody>
</table>

\[ P_n = 301.7 + 174.3 = 476 \text{k} \]

By selecting 4 24-inch diameter piles, the total force that the piles will be able to carry is equal to 4(476) = 1904 k which gives is a factor of safety equal to 2.23>2.
4.3 Design of Pile Caps

Using the equations shown in Figure 28 from thestructuralworld.com we were able to design the required dimensions of the pile caps and the spacing of the connected piles. In these equations, s represents the center to center spacing of the piles and α represents the spacing factor equal to $2 - 3$ depending on soil conditions. We used an average value of 2.5 for α. AutoCAD drawings for all 3 different pile cap/driven pile configurations calculated in this section can be found in Figures 15-17 of Appendix III.

\[ s = \alpha \times \text{pile } \]

\[ B = (2.5 + 1)(14/12) + 300/304.8 = 5.07' \text{ round up to 5.25'} \]

\[ d = 14(25.4) = 356 \text{ mm} < 550 \text{ mm} \]

\[ H = 2(14/12) = 2.33' \text{ round up to 2.5'} \]
**20”x 20” Column Pile Caps with 20” Diameter Piles**

\[ s = 2.5 \times \frac{20}{12} = 4.17’ \text{ round up to 4.25’} \]

\[ B = (2.5 + 1) \times \frac{20}{12} + \frac{300}{304.8} = 6.82’ \text{ round up to 7’} \]

\[ d = 20 \times 25.4 = 508 \text{ mm} < 550 \text{ mm} \]

\[ H = 2 \times \frac{20}{12} = 3.33’ \text{ round up to 3.5’} \]

**24”x 24” Column Pile Caps with 24” Diameter Piles**

\[ s = 2.5 \times \frac{24}{12} = 5’ \]

\[ B = (2.5 + 1) \times \frac{24}{12} + \frac{300}{304.8} = 7.98’ \text{ round up to 8’} \]

\[ d = 24 \times 25.4 = 610 \text{ mm} > 550 \text{ mm} \]

\[ H = \frac{1}{3}[8(2) - (600/304.8)] = 4.68’ \text{ round up to 5’} \]
4.4 Design of Ramp Driven Piles and Pile Caps

For the design of the driven piles used to support the ramp, we used 1 pile to support each column along the ramp. Each pile will be offset 2.5 feet from the columns which are spaced at 5-foot increments along the ramp in conjunction with the double-tee stems. Figure 29 shows how each pile transfers roughly half of the loading from the column to its left and half of the loading from the column to its right, or 1 full column load.

\[
P_u = 27.1 + 31.7 + 0.15(24)(14/12)^2 + 0.15(2)(5/2-7/12)(20/12)(14/12) + 0.15(5)(2)(2) = 67.8 \text{ k}
\]

For the ramp driven piles, we selected piles with a 12-inch diameter. The toe bearing capacity of the soil from these columns is equal to the soil bearing capacity times the cross-section area of the pile minus the weight of the pile. The 28-foot height of the pile represents the 35-foot depth of the pile minus the pile cap depth, 2 feet, and the 5-foot depth of the ground level of our parking structure.
\[ P_t = 100\pi(6/12)^2 - 0.15\pi(6/12)^2(28) = 75.2 \text{ k} \]

The side friction capacity of the soil is equal to the amount of upward friction that each layer of the soil profile in contact with the pile can carry. A table summary of the calculations for the side friction capacity of the soil is shown in Table 16.

**Table 16: Side Friction Calculation for 12” Driven Pile**

<table>
<thead>
<tr>
<th>Layer (ft)</th>
<th>( K_0 )</th>
<th>( \sigma_z ) (psf)</th>
<th>( \beta )</th>
<th>( f_n ) (psf)</th>
<th>( A_s ) (sf)</th>
<th>( f_n A_s ) (k)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-12</td>
<td>0.426</td>
<td>427</td>
<td>0.5</td>
<td>213.5</td>
<td>15.71</td>
<td>3.354085</td>
</tr>
<tr>
<td>12-22</td>
<td>0.386</td>
<td>1454</td>
<td>0.532</td>
<td>774</td>
<td>31.42</td>
<td>24.31908</td>
</tr>
<tr>
<td>22-35</td>
<td>0.386</td>
<td>2835</td>
<td>0.532</td>
<td>1508</td>
<td>40.84</td>
<td>61.58672</td>
</tr>
<tr>
<td>Sum</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>89.25989</td>
</tr>
</tbody>
</table>

\[ P_n = 75.2 + 89.3 = 165 \text{ k} \]

By selecting 12-inch diameter piles spaced at every 5 feet along the length of the ramp and along both sides of the ramp, the total force that the piles will be able to carry is equal to 165 k, which gives us a factor of safety, \( P_n/P_u \), equal to 2.19>2.

Figure 18 of Appendix III shows the AutoCAD drawing depicting the ramp driven pile/pile cap configuration.
5. Traffic Design

A lot goes into the development of an effective parking layout. The first step is the most basic; how big does a single parking spot need to be? MDOT (Michigan Department of Transportation) specifies the minimum parking layout dimensions for four different angles of parking. The most basic parking angle is 90°, at this angle a parking stall is required to be at least 8.5 feet wide and 18 feet deep. Other angles include 45°, 52°30’, and 60°. These are often used where one-way traffic operations exist. One-way traffic in parking garages is utilized in garages where there is simply not enough space for two-way traffic. Our site allows enough space for two-way traffic so our team decided that our design would incorporate two-way traffic, in which, 90° parking angles are utilized. From there, the development of the overall parking was pieced together. To develop the overall parking layout, the parking structure was broken down into three bays spanning east/west and two bays spanning north/south, as shown in Figure 30. The size of the bays is typical to a parking garage of similar dimensions. This allows for optimal space for parking and adequate drive lanes.

![Figure 30: Parking Structure Breakdown](image)

Western Michigan University

Drawing by: Andy Dviedo

Department of Engineering

Scale: 1:1400

Date: 2/22/22

Sheet 1 of 1
Bays 1, 2, and 3, span 200 feet and are 58 feet wide. These bays make up the majority of the parking area to be had. At the north (top) and south (bottom) wall of each bay, are 6 columns. Each bay will have a row of parking stalls lined along each row of columns. The dimensions of the drive lanes can then be verified by subtracting the overall width of the bay by the two rows of parking stalls in each bay. In this case the parking bay is 58 feet wide, and two rows of 18 feet deep parking stalls equates to a drive lane that is 22 feet wide. The minimum drive lane width for a two-way parking garage, according to the City of Kalamazoo, is 20 feet. The width of the stall must be a minimum of 8.5 feet. Using a minimum width of 8.5 feet, maximizes the number of stalls per row. Another factor in determining the number of stalls per row are the columns along which the row of stalls is lined up. These columns protrude outward from the wall and will reduce the area of parking available. The columns are set 40 feet apart from each other. To avoid any vehicular damage to the columns, it is important to not have parking stalls touching the columns. For this to happen four 8.5 feet wide parking stalls will sit in between the columns, providing ample room for the columns to occupy. With this parking configuration, bays 1-3 can hold 40 vehicles while providing a drive lane that can accommodate two-way traffic.

The two end bays span 174 feet and are 40 feet wide. Each of these bays are not wide enough to support two-way traffic and two rows of parking. Also occupying these end bays are two stair towers shown in Figure 29. A similar process of determining the number of stalls was used to develop the parking geometry in these bays. 14 stalls will fit along the outer walls while provided space for the columns. Additionally, 4 stalls can be added to the end bays in between the stair towers and bays 1 & 3. The final total number of stalls on this floor plan is 156. It is important to note that this is only the case on the second level of the parking garage. The final parking layout at level 2 is shown below in Figure 31.
The first and third levels will have less space for available parking since the ramp bay, Bay 2 in Figure 30, is only half available for parking at these levels. This reduction equates to a total of 136 stalls on level 3. Level 3 parking layout is shown in Figure 21 of Appendix III. In addition to the ramp reduction, the available space for parking at level 1 is reduced by the entrance and exit on the east wall. Parking space at level 1 will have another slight reduction because this level will hold the required handicapped parking per the Americans with Disabilities Act (ADA). These stalls are required to be 8 feet wide and must have at least 5 feet of available space on either side of the stall per ADA parking requirements. This 5-foot space requirement at each ADA parking space means there will be slightly less available parking at level 1. The number of required handicap stalls is related to the number of total stalls in the garage. With the configuration of level 2 shown above in Figure 31, and with the addition of levels 1 & 3, our parking structure will have a total of 420 parking stalls. ADA requires that for a parking garage with 401-500 stalls, 8 of those stalls are required to be ADA accessible. The final parking geometry for level 1 is shown in Figure 32 below.
Figure 32: Level 1 Parking Layout
6. Cost Estimation

It is necessary to implement accurate cost estimations for any project. These estimates were created using RS Means, a software used for cost estimating, and put into a master spreadsheet organized by trade. Cost estimates consider all the components that make up the project, preconstruction fees, material, equipment and labor fees. Items involved in material fees include all concrete utilized during the construction, this consists of the precast design members. Equipment and labor fees were determined based on information provided by RS Means for real-time market rates. By researching similar projects, this estimate was developed to include all aspects of the total price of the work that is to be performed. Structural material quantities were used to create the line-item estimates for the most important and most expensive parts of the project. Some of the items that were included in the estimate included excavation of existing soil, fine grading, precast concrete erection, pavement striping, trees, grass, bushes/shrubs, EV charging stations, signage for the structure, and emergency alarm systems with ADA compliance. The overall estimated total cost of this project is $5,323,526. Table 17 below illustrates the schedule of values for this project.

Table 17: Project Schedule of Values

<table>
<thead>
<tr>
<th>Description</th>
<th>Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>Equipment Costs</td>
<td>$195,979</td>
</tr>
<tr>
<td>Material Costs</td>
<td>$3,551,054</td>
</tr>
<tr>
<td>Labor Costs</td>
<td>$1,500,558</td>
</tr>
<tr>
<td>Miscellaneous Costs</td>
<td>$75,933</td>
</tr>
<tr>
<td><strong>ESTIMATED TOTAL COST</strong></td>
<td><strong>$5,323,526</strong></td>
</tr>
</tbody>
</table>

* Please refer to Appendix IV for complete detailed cost estimates. *
7. Construction Scheduling

The construction schedule was created using Microsoft Project and looking at other project’s construction schedules that are like this. This was created by including predecessors to multiple tasks which can hinder or accelerate the construction of the structure. The major constraint that was provided on this project is the timeframe of a bit under four months to complete this project to substantial completion. The full project schedule showing all tasks and relevant durations can be found in Appendix V of this report.

7.1 Work Breakdown Structure

To complete the parking structure within the tight time constraint of 4 months it is critical that precast panels are ordered ahead of time due to long lead times. The erection of the precast concrete panels is a swift and quick process that comprises the main structure. First, the construction crews will start with site mobilization and preparation. This includes setting up the laydown area for all subcontractors involved, tying in site temporary power, connecting the construction manager’s site office, which is typically a trailer to provide power, all site fencing, site signage, establishing material drop off routes, etc. Next, the earthwork subcontractor is to provide all tree protection per Western Michigan University’s standards and ensure all sediment control measures are properly in place. Once those tasks are complete, excavation work is to begin on the site which includes milling of the existing asphalt and soils. Following the removal of material, the earthwork contractor will begin compacting and grading the soil to appropriate density and slope. Following the grading the earthwork subcontractor will drive pile and install the pile caps. The purpose of the pile caps is to ensure that the structure will hold up and the soil underneath will not fail unexpectedly. Following the installation of the piles and pile caps, the mechanical, electrical, and plumbing subcontractors will tie-in to utilities and route under slab conduit through the limits. Following the MEP tie-in foundation walls will be formed to kick off structural development. Once the foundation walls are put in place, the slab on grade will be formed to provide the first drivable surface in our structure. After the slab-on-grade is complete it will take the steel contractor and precast concrete contractor working concurrently for 10 days to erect the first level columns. Following the completion of the columns being placed, the beams on level 1 will be placed. The construction schedule is outlined in detail in sequential order, and it is imperative that no setbacks happen during construction to ensure adequate parking is provided on the main campus before the influx of students in September.
7.2 Schedule Scope Breakdown

To fulfill the construction needs of this project, multiple skilled trades will be required to be on site concurrently to achieve timely substantial completion. The necessary trades needed to complete this project include excavators, landscapers, precast concrete installers, site concrete placers, structural steel workers, glazers, joint sealant applicants, plumbers, mechanical workers, and electricians. The following are the skilled trades listed with tasks on the schedule that they are involved in contributing to:

- **Sitework and Excavation:**
  - Site Prep/Mobilization
  - Excavation/Milling of Existing Land
  - Grading of Soil

- **Landscaping:**
  - Tree Installation
  - Shrub Installation
  - Grass Seeding/Sodding

- **Precast Concrete:**
  - Foundation Walls
  - Slab on Grade
  - Erect Columns
  - Set Beams
  - Place Double-Tees
  - Precast Concrete Panels

- **Site Concrete:**
  - Driven Piles and Installation of Pile Caps
  - Site Work (Approaches, Sidewalk)

- **Structural Steel:**
  - Steel Framing
  - Prefabricated Stairs

- **Glazers:**
  - Curtainwall System
    - Glazing on Curtainwall System

- **Joint Sealant:**
  - Sealants on all Precast Concrete Connections

- **Painters:**
  - Pavement Striping
• Plumbers:
  o Under Slab MEP
  o MEP Rough-in
  o Site Utilities
• Mechanical:
  o MEP Rough-in
  o Mechanical Devices
• Electricians:
  o Under Slab MEP
  o MEP Rough-in
  o Site Utilities
  o Electrical Devices
8 Sustainability Considerations

The introduction of a parking structure to expand parking availability will have an immediate positive impact regarding the social and economic aspects of sustainability. Eliminating the stress and lost time students face due to parking capacity strain will allow them to be more prepared for, and spend more time in, class. This will eventually lead to the development of more elite students and eventually more productive members of society. Economically, a well-managed parking structure means more money for the university. Because more spaces are available, the number of students willing to drive their own cars to class will increase, meaning more revenue in the form of parking permits. In addition, making a large investment into the well-being of the students will show a commitment by the university to ensuring student success, which could lead to an uptick in enrollment.

When considering environmental sustainability, there are two main positive impacts of our parking structure. The first being the decrease in vehicle emissions in the area. Vehicle emissions will decrease because students will no longer need to circle lot 72W, and other student lots in the area, to find an available parking or leave their engine idling. They may now drive into a parking structure and park. No more wasting gas, and no more unnecessary release of vehicle emissions.

The second environmental impact will be the increase of green space. By increasing the amount of parking spaces vertically, we can reduce the overall footprint of the previous lot. Because our parking structure and subsequent exterior parking area will not require the entire existing space in Lot 72W, it will introduce additional green space in the area. By providing 420 parking spaces in our parking structure and an additional 106 exterior parking spaces, we both met our goal of increasing Lot 72W’s capacity by greater 50% and introduced 14,000 square feet of additional green space to the area. This final parking layout along with the additional green space is shown below in Figure 33. The precast concrete nature of our parking structure also allows for the possibility of vertical additions in the future, which could increase the green space in the area even more. There should be no other significant impact as no animal habitats will be disturbed, no water sources are in play, no farmland will be affected, and no historic monuments will be disturbed.
Figure 33: Final Site Layout
Conclusions

In conclusion, our team recommends that a 3-level precast concrete parking structure be built on Lot 72W near the Haworth College of Business and Rood Hall on Western Michigan University’s main campus. Our team recommends that this parking structure be constructed using a combination of precast double-tees 10-foot in width and 58-foot in span. The beams used in the structure should be a combination of precast L-beams, inverted T-beams, and rectangular spandrel beams, and all columns should have square dimensions. The structure should be braced laterally using a combination of precast concrete shear walls and structural steel. The parking structure should have an overall width and length equal to 174 feet and 280 feet respectively.

Our team recommends that the foundational support of the parking structure should be comprised of circularly dimensioned, precast concrete driven piles. These piles should be driven to a depth of 35 feet below the existing ground level and pile caps should be used to transfer the column loads to the driven piles.

Our team recommends that the additional portion of the lot not used for the parking structure should be split between an asphalt pavement that will be used for additional parking, and the introduction of additional green space to the area. We have determined that a parking structure of this type will be able to increase Lot 72W’s capacity by approximately 54%, significantly relieving the stress students are subjected to when trying to attend class in that area.

Our team estimates that, once we break ground on the site, it will take approximately 122 calendar days to complete construction. We estimate that, in total, this parking structure will cost $5,323,526.
Resources and references

- Precast/Prestressed Design Handbook, 7th Edition

- Precast/Prestressed Concrete Institute Design Tables & Charts

- American Society of Civil Engineers/Structural Engineering Institute 7-10
  - file:///C:/Users/CAE-USER/Downloads/ASCE7_10_Lo.pdf

- American Concrete Institute Reinforced Concrete Design Handbook

- Geotechnical Report
  - file:///C:/Users/CAE-USER/Downloads/K10475-RPT-061287.pdf

- Bowels Foundation Analysis, 5th Edition

- Typical Values of Soil Friction Angle
  - https://www.geotechdata.info/parameter/angle-of-friction

- Pile Cap Design Criteria
  - https://www.thestructuralworld.com/2018/07/20/pile-cap-design/
- Local Elevation Data – Kalamazoo County
- [https://en-us.topographic-map.com/maps/j1y/Kalamazoo/](https://en-us.topographic-map.com/maps/j1y/Kalamazoo/)
Appendix I – Figures Referenced in Report Body

Appendix I Figure 1: Dimensions of Selected Double-Tee Sections

Appendix I Figure 2: General Dimensions of Selected Beam Sections
Appendix I Figure 3: Inside Frame Member Definitions

Appendix I Figure 4: Outside Frame Member Definitions
Appendix I Figure 5: Applied Loads on the Outside Frame

Appendix I Figure 6: Applied Loads on the Inside Frame
Appendix I Figure 5: Location of 20"x20" Column with the Greatest Maximum Moment and Axial Force (Inside Frame)

Appendix I Figure 7: Shear Distribution on Beams

Appendix I Figure 8: Moment Distribution on Beams
Appendix I Figure 10 - Site Soil Profile
Appendix II – Mathcad Calculations

Wind Load Calculations:

\[ V = 120 \text{ mph} \quad K_{st} = 1 \quad K_d = 0.85 \]

\[ K_z20 = 0.62 \quad K_z30 = 0.81 \quad \text{interpolation} \quad K_z21 = 0.62 + \left( \frac{21 - 20}{50} \right) (0.81 - 0.62) = 0.63 \]

\[ C_p := (-0.9, -0.18) \quad G_{cp} = 1.55 \]

\[ q_z = 0.00256 \cdot K_z21 \cdot V^2 \quad q_z = 0.00256 \cdot 0.63 \cdot 1 \cdot 0.85 \cdot 120^2 = 19.74 \text{ psf} \]

**Roof:**

\[ P = 19.74 \text{ psf} \cdot 0.85 \cdot -0.9 - 19.74 \text{ psf} (-0.55) = -4.2 \text{ psf} \]

\[ P = 19.74 \text{ psf} \cdot 0.85 \cdot -0.9 - 19.74 \text{ psf} (0.55) = -26 \text{ psf} \]

**Controls**

\[ P = 19.74 \text{ psf} \cdot 0.85 \cdot -0.18 - 19.74 \text{ psf} (-0.55) = 7.84 \text{ psf} \]

**Controls**

\[ P = 19.74 \text{ psf} \cdot 0.85 \cdot -0.18 - 19.74 \text{ psf} (0.55) = -13.9 \text{ psf} \]

**Windward:**

\[ K_z = 0.57 \quad (0-15 \text{ ft.}) \quad q_z = 0.00256 \cdot 0.57 \cdot 1 \cdot 0.85 \cdot 120^2 = 17.86 \text{ psf} \]

\[ K_z = 0.62 \quad (20 \text{ ft.}) \quad q_z = 0.00256 \cdot 0.62 \cdot 1 \cdot 0.85 \cdot 120^2 = 19.4 \text{ psf} \]

\[ K_z = 0.63 \quad (21 \text{ ft.}) \quad q_z = 0.00256 \cdot 0.63 \cdot 1 \cdot 0.85 \cdot 120^2 = 19.74 \text{ psf} \]

\[ C_p = 0.8 \quad G = .85 \quad G_{cp} = 0.55 \]

\[(0-15 \text{ ft.}) \quad P = 17.86 \text{ psf} \cdot 0.85 \cdot .8 - 17.86 \text{ psf} (-0.55) = 22 \text{ psf} \]

\[(0-15 \text{ ft.}) \quad P = 17.86 \text{ psf} \cdot 0.85 \cdot .8 - 17.86 \text{ psf} (0.55) = 2.32 \text{ psf} \]

\[(20 \text{ ft.}) \quad P = 19.4 \text{ psf} \cdot 0.85 \cdot .8 - 19.4 \text{ psf} (-0.55) = 23.9 \text{ psf} \]

\[(20 \text{ ft.}) \quad P = 19.4 \text{ psf} \cdot 0.85 \cdot .8 - 19.4 \text{ psf} (0.55) = 2.5 \text{ psf} \]

\[(21 \text{ ft.}) \quad P = 19.74 \text{ psf} \cdot 0.85 \cdot .8 - 19.74 \text{ psf} (-0.55) = 24.3 \text{ psf} \]

\[(21 \text{ ft.}) \quad P = 19.74 \text{ psf} \cdot 0.85 \cdot .8 - 19.74 \text{ psf} (0.55) = 2.57 \text{ psf} \]

**Leeward:**

\[ K_z = 0.57 \quad (0-15 \text{ ft.}) \quad q_z = 0.00256 \cdot 0.57 \cdot 1 \cdot 0.85 \cdot 120^2 = 17.86 \text{ psf} \]

\[ K_z = 0.61 \quad (19 \text{ ft.}) \quad q_z = 0.00256 \cdot 0.61 \cdot 1 \cdot 0.85 \cdot 120^2 = 19.11 \text{ psf} \]

\[ C_p = -0.5 \quad G = .85 \quad G_{cp} = 0.55 \]

\[(0-15 \text{ ft.}) \quad P = 17.86 \text{ psf} \cdot 0.85 \cdot (-0.5) - 17.86 \text{ psf} (-0.55) = -2.23 \text{ psf} \]

\[(0-15 \text{ ft.}) \quad P = 17.86 \text{ psf} \cdot 0.85 \cdot (-0.5) - 17.86 \text{ psf} (0.55) = -17.41 \text{ psf} \]

\[(19 \text{ ft.}) \quad P = 19.11 \text{ psf} \cdot 0.85 \cdot (-5) - 19.11 \text{ psf} (-0.55) = 2.4 \text{ psf} \]

\[(19 \text{ ft.}) \quad P = 19.11 \text{ psf} \cdot 0.85 \cdot (-5) - 19.11 \text{ psf} (0.55) = -18.8 \text{ psf} \]

Appendix II Figure 1: Roof, Windward, and Leeward Wind Load Calculations
Flexural Steel Design L26"x40"

\[ M_u = \frac{149 + 146}{2}(2.5) + \frac{114 + 109}{2}(5) + \frac{77 + 72}{2}(5) + \frac{40 + 35}{2}(5) + \frac{3.1 \times 2.5}{2} = 1490 \]

\[ \begin{align*}
A_s &= 12 \text{ in}^2 \\
F_y &= 60 \text{ ksi} \\
f'_c &= 5 \text{ ksi} \\
b_w &= 18 \text{ in} \\
d &= 36.37 \text{ in} \\
M_u &= 1.49 \times 10^3
\end{align*} \]

\[ a = \frac{A_s \cdot F_y}{0.85 \cdot f'_c \cdot b_w} = 9.41 \text{ in} \]

\[ c = \frac{a}{0.85} = 11.1 \text{ in} \]

\[ \varepsilon_s = \frac{(d - c)}{c} (0.003) = 0.007 > 0.005 \quad \text{Tension Controlled} \]

\[ C_c = 2.5 \text{ in} \]

\[ S = 15 \left( \frac{40000}{2 \cdot 60000} \right) - 2 \cdot 2.5 = 10 \text{ in} \]

\[ S = 12 \left( \frac{40000}{2 \cdot 60000} \right) = 12 \text{ in} \]

\[ S = \frac{18 - 2 \cdot 2.5}{5} = 2.6 \text{ in} \]

\[ d_b = 1.128 \text{ in} \]

\[ C_s = S - d_b = 1.472 \text{ in} \]

\[ \phi M_n = \frac{0.9 \cdot A_s \cdot F_y \cdot \left( d - \frac{a}{2} \right)}{12} \]

\[ \phi M_n = \frac{0.9 \cdot 12 \cdot 60 \cdot \left( 36.37 - \frac{9.41}{2} \right)}{12} = 1710 \text{ ft-kip} \]

Appendix II Figure 2: Flexural Steel Design Calculations for the 3rd Level L-Beam
Shear Reinforcement L26"x40"

Critical Section: \( \frac{36.37 \text{ in}}{12 \text{ in}} = 3.03 \text{ ft} \)

\[
\frac{V_s}{\phi} \cdot d = 152 \left( \frac{3.03 - 2.5}{5} \right) (152 - 145) = 151 \ k \]

\( f'_c := 5000 \ \text{psi} \quad b_w := 18 \ \text{in} \quad d := 36.37 \ \text{in} \quad F_y := 40000 \ \text{psi} \)

\( V_c := 2 \cdot \sqrt{5000 \ \text{psi} \cdot 18 \ \text{in} \cdot 36.37 \ \text{in}} = 92.58 \ \text{kip} \)

\[
\frac{V_s}{\phi} = 151 \ \text{kip} \quad \frac{V_v}{\phi} > \frac{V_c}{2}
\]

Stirrups Required

Use No. 4 Bar Double Leg Stirrups

\( A_s := 0.4 \ \text{in}^2 \quad \frac{V_u}{\phi} = 276 \ \text{kip} \quad V_c := 137.02 \ \text{kip} \)

\( F_y := 40 \ \text{ksi} \quad d := 36.37 \ \text{in} \)

\( S := \frac{(0.4 \ \text{in}^2 \cdot 40000 \ \text{psi} \cdot 36.37 \ \text{in})}{151 \ \text{kip} - 92.58 \ \text{kip}} = 9.96 \ \text{in} \)

Controls

\( S_{\text{max}} := \frac{36.37 \ \text{in}}{2} = 18.2 \ \text{in} \)

\( S := \frac{(0.4 \ \text{in}^2 \cdot 40000 \ \text{psi})}{0.75 \cdot \sqrt{5000 \ \text{psi} \cdot 18 \ \text{in}}} = 16.76 \ \text{in} \)

Use No. 4 Bar Double Leg Stirrups Spaced @ 9"

\[
\frac{V_v}{\phi} > \frac{V_c}{2} \quad @ \ 17.5 \ \text{ft}
\]

Cut off Stirrups @ 17.5 ft from each end

Appendix II Figure 3: Shear Steel Design Calculations for the 3rd Level L-Beam
Flexural Steel Design L26"x36"

\[ M_u = \frac{(129 + 127)}{2} (2.5) + \frac{(99 + 94)}{2} (5) + \frac{(67 + 62)}{2} (5) + \frac{(35 + 30)}{2} (5) + \frac{(3.1 \cdot 2.5)}{2} = 1291 \text{ft} \cdot \text{kip} \]

- \( A_s = 12 \text{ in}^2 \)
- \( F_y = 60 \text{ ksi} \)
- \( f_c' = 5 \text{ ksi} \)
- \( b_w = 18 \text{ in} \)
- \( d = 32.37 \text{ in} \)
- \( M_u = 1.291 \times 10^3 \)

\[ a = \left( \frac{A_s \cdot F_y}{0.85 \cdot f_c' \cdot b_w} \right) = 9.41 \text{ in} \]

\[ c = \frac{a}{0.85} = 11.1 \text{ in} \]

\[ \epsilon_a = \left( \frac{d - c}{c} \right) = 0.003 = 0.0058 > 0.005 \quad \text{Tension Controlled} \]

\[ C_t = 2.5 \text{ in} \]

\[ S = 15 \left( \frac{40000}{2 \cdot 60000} \right) = 2.5 = 10 \text{ in} \]

\[ S = 12 \left( \frac{40000}{3 \cdot 60000} \right) = 12 \text{ in} \]

\[ S = \left( \frac{18 - 2 \cdot 2.5}{5} \right) = 2.6 \text{ in} \quad < 10 \text{ in} \quad \text{ok} \]

\[ d_b = 1.128 \text{ in} \quad C_s = S - d_b = 1.472 \text{ in} \quad > 1.128 \quad \text{ok} \]

\[ \phi M_n = \frac{0.9 \cdot A_s \cdot F_y \cdot \left( \frac{d - a}{2} \right)}{12} \]

\[ \phi M_n = \frac{0.9 \cdot 12 \cdot 60 \cdot \left( 32.37 - \frac{9.41}{2} \right)}{12} = 1494 \text{ ft} \cdot \text{kip} \]

Appendix II Figure 4: Flexural Steel Design Calculations for the 2nd Level L-Beam
Shear Reinforcement L26" x 36"

Critical Section: \[ \frac{32.37 \text{ in}}{12 \text{ in}} = 2.7 \text{ ft} \]

\[ \frac{V_u}{\phi d} = 132 - \frac{(2.7 - 2.5)}{5} (132 - 125) = 131.7 \text{ k} \]

\[ f_c = 5000 \text{ psi}, \quad b_w = 18 \text{ in}, \quad d = 32.37 \text{ in}, \quad F_{yt} = 40000 \text{ psi} \]

\[ V_c = 2 \cdot \sqrt{5000} \text{ psi} \cdot 18 \text{ in} \cdot 32.37 \text{ in} = 82.4 \text{ kip} \]

\[ \frac{V_u}{\phi} = 131.7 \text{ kip}, \quad \frac{V_c}{\phi} > \frac{V_c}{2} \quad \text{Stirrups Required} \]

Use No. 4 Bar Double Leg Stirrups

\[ A_s = 0.4 \text{ in}^2, \quad \frac{V_u}{\phi} = 131.7 \text{ kip}, \quad V_c = 82.4 \text{ kip} \]

\[ F_{yt} = 40 \text{ ksi}, \quad d = 36.37 \text{ in} \]

\[ S = \frac{(0.4 \text{ in}^2 \cdot 40000 \text{ psi} \cdot 32.37 \text{ in})}{131.7 \text{ kip} - 82.4 \text{ kip}} = 10.5 \text{ in} \quad \text{Controls} \]

\[ S_{\text{max}} = \frac{32.37 \text{ in}}{2} = 16.2 \text{ in} \]

\[ S = \frac{(0.4 \text{ in}^2 \cdot 40000 \text{ psi})}{0.75 \cdot \sqrt{5000} \text{ psi} \cdot 18 \text{ in}} = 16.76 \text{ in} \]

Use No. 4 Bar Double Leg Stirrups Spaced @ 10"

\[ \frac{V_u}{\phi} > \frac{V_c}{2} \quad \text{Interpolate} \quad @ 16.65 \text{ ft} \]

Cut off Stirrups @ 16.65 ft from each end

Appendix II Figure 5: Shear Steel Design Calculations for the 2nd Level L-Beam
Appendix II Figure 6: Flexural Steel Design Calculations for the 3rd Level T-Beam
Shear Reinforcement T40"x44"

Critical Section = \( \frac{40.37 \text{ in}}{12 \text{ ft}} = 3.36 \text{ ft} \)

\[
\frac{V_u}{\phi} \cdot d = 277 - \frac{{(3.36 - 2.5)}}{5}{(277 - 271)} = 276 \text{ kip}
\]

\( f'_c = 5000 \text{ psi} \quad b_v = 24 \text{ in} \quad d = 40.37 \text{ in} \quad F_{yt} = 60000 \text{ psi} \)

\( V_u = 2 \cdot \sqrt{5000 \text{ psi}} \cdot 24 \text{ in} \cdot 40.37 \text{ in} = 137.02 \text{ kip} \)

\[
\frac{V_u}{\phi} = 276 \text{ kip} \quad \frac{V_v}{\phi} = \frac{V_c}{2} \quad \text{Stirrups Required}
\]

Use No. 4 Bar Double Leg Stirrups

\( A_s = 0.4 \text{ in}^2 \quad \frac{V_u}{\phi} = 276 \text{ kip} \quad V_c = 137.02 \text{ kip} \)

\( F_{yt} = 40 \text{ ksi} \quad d = 36.37 \text{ in} \)

\( S := \frac{(0.4 \text{ in}^2 \cdot 60000 \text{ psi} \cdot 40.37 \text{ in})}{276 \text{ kip} - 137.02 \text{ kip}} = 6.97 \text{ in} \quad \text{Controls} \)

\( S_{\text{max}} := \frac{40.37 \text{ in}^2}{2} = 20.2 \text{ in} \)

\( S := \frac{(0.4 \text{ in}^2 \cdot 60000 \text{ psi})}{0.75 \cdot \sqrt{5000} \text{ psi} \cdot 24 \text{ in}} = 18.86 \text{ in} \)

Use No. 4 Bar Double Leg Stirrups Spaced @ 6"

\[
\frac{V_u}{\phi} > \frac{V_c}{2} \quad @ \ 17.5 \text{ ft}
\]

Cut off Stirrups @ 17.5 ft from each end

Appendix II Figure 7: Shear Steel Design Calculations for the 3rd Level T-Beam
Flexural Steel Design T40" x 40"

\[ M_u = \frac{237 + 235}{2} \times (2.5) + \frac{180 + 175}{2} \times (5) + \frac{121 + 116}{2} \times (5) + \frac{62 + 57}{2} \times (5) + \frac{3 \times 2.5}{2} = 2371 \text{ ft-kip} \]

- \( A_s = 18 \text{ in}^2 \)
- \( F_y = 60 \text{ ksi} \)
- \( f'c = 5 \text{ ksi} \)
- \( b_w = 24 \text{ in} \)
- \( d = 36.37 \text{ in} \)
- \( M_u = 2371 \text{ ft-kip} \)

\[ a = \frac{A_s \cdot F_y}{0.85 \cdot f'c \cdot b_w} = 10.6 \text{ in} \]

\[ e = \frac{a}{0.85} = 12.5 \text{ in} \]

\[ \varepsilon_s = \frac{(d - e)}{e} \cdot 0.003 = 0.0058 \gg 0.005 \text{ Tension Controlled} \]

\[ S_1 = 15 \left( \frac{40000}{2 \cdot 60000} \right) = 2.25 = 10 \text{ in} \quad C_c = 2.5 \text{ in} \]

\[ S_2 = 12 \left( \frac{40000}{2 \cdot 60000} \right) = 12 \text{ in} \]

\[ S_3 = \frac{(24 - 2.5)}{8} = 2.375 \text{ in} \ll 10 \text{ in} \text{ ok} \]

\[ d_s = 1.128 \text{ in} \quad C_s = S - d_s = 1.247 \text{ in} \ll 1.128 \text{ ok} \]

\[ \phi M_n = \frac{0.9 \cdot A_s \cdot F_y \cdot (d - e)}{12} \]

\[ \phi M_n = \frac{0.9 \times 18 \times 60}{2} \left( \frac{36.37 - 10.6}{2} \right) = 2517 \text{ ft-kip} \gg M_u \text{ ok} \]

\[ A_{min} = \frac{3 \cdot \sqrt{5000}}{60000} \times 24 \times 36.37 \approx 3.09 \text{ in}^2 \ll 18 \text{ in}^2 \text{ ok} \]
Shear Reinforcement T40"x40"

Critical Section:
\[
\frac{36.37 \text{ in}}{12 \text{ in/ft}} = 3.03 \text{ ft}
\]

\[
\frac{V_u}{\Phi} @ d = 240 - \left(\frac{3.03 - 2.5}{5}\right) (240 - 233) = 239 \text{ k}
\]

\[
f' = 5000 \text{ psi} \quad b_w = 24 \text{ in} \quad d = 36.37 \text{ in} \quad F_{yt} = 60000 \text{ psi}
\]

\[
V_c = 2 \sqrt{5000 \text{ psi} \cdot 24 \text{ in} \cdot 36.37 \text{ in}} = 123.44 \text{ kip}
\]

\[
\frac{V_u}{\Phi} = 239 \text{ kip} \quad \frac{V_v}{\Phi} > \frac{V_c}{2}
\]

Stirrups Required
Use No. 4 Bar Double Leg Stirrups

\[
A_s = 0.4 \text{ in}^2 \quad \frac{V_u}{\Phi} = 239 \text{ kip} \quad V_c = 123.44 \text{ kip}
\]

\[
F_{yt} = 40 \text{ ksi} \quad d = 36.37 \text{ in}
\]

\[
S = \left(\frac{0.4 \text{ in}^2 \cdot 60000 \text{ psi} \cdot 36.37 \text{ in}}{239 \text{ kip} - 123.44 \text{ kip}}\right) = 7.55 \text{ in}
\]

 Controls

\[
S_{\text{max}} = \frac{36.37 \text{ in}}{2} = 18.2 \text{ in}
\]

\[
S = \frac{(0.4 \text{ in}^2 \cdot 60000 \text{ psi})}{0.75 \cdot \sqrt{5000 \text{ psi} \cdot 24 \text{ in}}} = 18.86 \text{ in}
\]

Use No. 4 Bar Double Leg Stirrups Spaced @ 7"

\[
\frac{V_v}{\Phi} > \frac{V_c}{2} \quad @ 17.5 \text{ ft}
\]

Cut off Stirrups @ 17.5 ft from each end

Appendix II Figure 9: Shear Steel Design Calculations for the 2nd Level T-Beam
Appendix II Figure 10: Flexural Steel Design Calculations for the Spandrel Beam

Flexural Steel Design End Beam

Assume \( h = 24" \) \( b = 18" \)

\[
\begin{align*}
 \phi M_u &= \frac{0.9 \cdot A_s \cdot F_y \cdot \left( \frac{d}{2} - \frac{a}{2} \right)}{12} \\
\phi M_u &= \frac{0.9 \cdot 60 \cdot \left( \frac{21.5}{2} - \frac{4.28}{2} \right)}{12} = 523 \text{ ft} \cdot \text{kip} \quad \square > M_u \quad \text{ok}
\end{align*}
\]

\[
\begin{align*}
 A_s &= \frac{5.46 \text{ in}^2}{0.9 \cdot 60 \cdot 9^{1/4} \cdot 21.5} = 5.46 \text{ in}^2 \\
 A_s &= \frac{5.46 \text{ in}^2}{0.85 \cdot f' \cdot b_w} = 4.28 \text{ in} \\
 F_y &= 60 \text{ ksi} \\
f' &= 5 \text{ ksi} \\
b_w &= 18 \text{ in} \\
d &= 36.37 \text{ in} \\
 M_u &= 475 \text{ ft} \cdot \text{kip} \\
 a &= \frac{(A_s \cdot F_y)}{0.85 \cdot f' \cdot b_w} = 4.28 \text{ in} \\
c &= \frac{4.28}{0.85} = 5.04 \text{ in} \\
\epsilon_c &= \frac{(21.5 - 5.04)}{5.04} (0.003) = 0.0098 > 0.005 \text{Tension Controlled}
\end{align*}
\]

\[
\begin{align*}
 S &= 15 \left( \frac{40000}{2 \cdot 60000} \right) - 2 - 2.5 - 10 \text{ in} \quad C_r = 2.5 \text{ in} \\
 S &= 12 \left( \frac{40000}{2 \cdot 60000} \right) = 12 \text{ in} \\
 S &= \frac{18 - 2.2.5}{5} = 2.6 \text{ in} \quad \square < 10 \text{ in} \quad \text{ok}
\end{align*}
\]

\[
\begin{align*}
 a &= 4.28 \text{ in} \\
d_b &= 1.128 \text{ in} \\
 C_s &= S - d_b = 1.472 \text{ in} \quad \square > 1.128 \quad \text{ok}
\end{align*}
\]

\[
\begin{align*}
 A_{\text{min}} &= \frac{3 \cdot \sqrt{5000}}{6000} \cdot 18 \cdot 21.5 = 1.37 \text{ m}^2 \\
 &\square < 18 \text{ in}^2 \quad \text{ok}
\end{align*}
\]
Shear Reinforcement End Beam

Critical Section: \[\frac{21.5 \text{ in}}{12 \text{ in}} = 1.79 \text{ ft}\]

\[V_u = \frac{(1.13 \cdot 58)}{2} = 32.8k\]

\[V_u @ d = 0 + \frac{(29 - 1.79)}{29} \cdot 43.7 = 41k\]

\[f'_c = 5000 \text{ psi} \quad b_w = 18 \text{ in} \quad d = 21.5 \text{ in} \quad F_{yt} = 60000 \text{ psi}\]

\[V_c = 2 \cdot \sqrt{5000 \text{ psi} \cdot 18 \text{ in} \cdot 21.5 \text{ in}} = 54.73 \text{ kip}\]

\[\frac{V_u}{\phi} = 41 \text{ kip} \quad \frac{V_v}{\phi} > \frac{V_c}{2}\]

**Stirrups Required**

Use No. 4 Bar Double Leg Stirrups

\[A_s = 0.4 \text{ in}^2 \quad \frac{V_u}{\phi} = 239 \text{ kip} \quad V_c = 123.44 \text{ kip}\]

\[F_{yt} = 40 \text{ ksi} \quad d = 36.37 \text{ in}\]

\[S_{max} = \frac{21.5 \text{ in}}{2} = 10.8 \text{ in}\]

Use 10"

Use No. 4 Bar Double Leg Stirrups Spaced @ 10"
Appendix II Figure 12: Bearing Capacity of Natural Sand Soil Layer Used in Driven Pile Design
Appendix III - AutoCAD Drawings

Appendix III Figure 1: Flexural Steel Design of 3rd Level L-Beam

Appendix III Figure 2: Shear Steel Design of 3rd Level L-Beam
Appendix III Figure 3: Flexural Steel Design of 2nd Level L-Beam

Appendix III Figure 4: Shear Steel Design of 2nd Level L-Beam
Appendix III Figure 5: Flexural Steel Design of 3rd Level T-Beam

Appendix III Figure 6: Shear Steel Design of 3rd Level T-Beam
Appendix III Figure 7: Flexural Steel Design of 2nd Level T-Beam

Appendix III Figure 8: Shear Steel Design of 2nd Level T-Beam
Appendix III Figure 9: Flexural Steel Design of Spandrel Beam

Appendix III Figure 10: Shear Steel Design of Spandrel Beam
Appendix III Figure 11: 18”x18” Column Design
Appendix III Figure 12: 24"x24" Column Design
Appendix III Figure 13: 20”x20” Column Design
Appendix III Figure 14: Ramp H-Column Design
Appendix III Figure 15: Driven Pile/Pile Cap Configuration Supporting 18"x18" Columns

Appendix III Figure 16: Driven Pile/Pile Cap Configuration Supporting 20"x20" Columns
Appendix III Figure 17: Driven Pile/Pile Cap Configuration Supporting 18”x18” Columns

Appendix III Figure 18: Driven Pile/Pile Cap Configuration Supporting the Ramp
Appendix III Figure 19: Level 1 Parking Layout
Appendix III Figure 20: Level 2 Parking Layout
Appendix III Figure 21: Level 3 Parking Layout
## Appendix IV - Detailed Cost Estimate

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<td>cement Type I), placing and finishing</td>
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<th>Subtotal 7</th>
<th>Subtotal 8</th>
<th>Subtotal 9</th>
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<td>Metal parking bumpers, pipe bollards, concrete filled/painted, 8' L x 4' D hole, 6&quot; diam.</td>
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<td>$471.75</td>
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<td>Prestressed concrete piles, 24&quot; diameter, 5&quot; wall, priced using 200 piles, 50' long, unless specified otherwise, excludes pile caps or mobilization</td>
<td>520</td>
<td>$102.27</td>
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<td>Mobilization or demobilization, crane, large lattice boom, requiring assembly</td>
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<td>$7,347.45</td>
<td>$8,190.00</td>
<td>$15,537.45</td>
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<td>$16,380.00</td>
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<td>Prestressed concrete piles, 12&quot; diameter, 2-3/8&quot; wall, priced using 200 piles, 50' long, cylinder, unless specified otherwise, excludes pile caps or mobilization</td>
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<td>$47.34</td>
<td>$46.51</td>
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<td>Precast beam, L shaped, 20' span, 18&quot; x 36&quot;, includes material only</td>
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<td>Precast beam, tee shaped, 20' span, 24&quot; x 44&quot;, includes material only</td>
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<td>$111.38</td>
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<td>Precast column, to 12' high, 16&quot; x 16&quot;, 3000 psi</td>
<td>120</td>
<td>$263.46</td>
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<td>$297.65</td>
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<td>$20,737.92</td>
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<td>Electric vehicle charging, wall mounted, heavy duty, no RFID</td>
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<td>$804.64</td>
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<td>$903.54</td>
<td>$8,465.60</td>
<td>$569.80</td>
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<tr>
<td>Field personnel, superintendent, maximum</td>
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<tr>
<td>Field personnel, field engineer, senior engineer, maximum</td>
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<td>Temporary lighting, 40,000 S.F. building, 8 strings, incl. service lamps, wiring and outlets</td>
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<td>Temporary fencing, wire mesh, on 100mm x 100mm posts, 8' high</td>
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<tr>
<td>Project signs, sign, high intensity reflectorized, buy, excl. posts</td>
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<td>$</td>
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<td>$</td>
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<tr>
<td>Joint sealants, rigid joint sealants, tapes, sealant, PVC foam adhesive, 1/16&quot; x 1&quot;</td>
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<td>$</td>
<td>$</td>
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<td>Firestopping, construction joints, concrete/CMU wall joints, 2&quot; wide</td>
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<td>75</td>
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<td>Utility area drain, catch basins or manholes frames and covers, cast iron, heavy traffic, 24&quot; diameter, 400 lb., excluding footing &amp; excavation</td>
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<td>Stormwater management, allowance, add per S.F. of impervious surface</td>
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<td>Deciduous trees, honeylocust, balled &amp; burlapped (B&amp;B), 10'-12', 1-1/2&quot; caliper, in prepared beds</td>
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<td>$701.80</td>
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<tr>
<td>B&amp;B, 8' - 10', in prepared beds</td>
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<td>580.25</td>
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<td>28,753.08</td>
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<tr>
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<tr>
<td>Sodding, bluegrass sod, on level ground, 1&quot; deep, 8 M.S.F.</td>
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<td>$</td>
<td>$</td>
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<td>$</td>
<td>$</td>
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<td>Smoke and carbon monoxide alarm battery operated photoelectric low profile</td>
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<tr>
<td>Detection system, fire alarm, detector, rate of rise, excl. wires &amp; conduit</td>
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<td>535.00</td>
<td>79.52</td>
<td>- 614.52</td>
<td>535.00</td>
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<td>72.50</td>
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<td>Cost 4</td>
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<td>Cost 8</td>
<td>Cost 9</td>
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<td>Detection system, visual alarm, ADA type, excluding wires &amp; conduits</td>
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<td>$167.25</td>
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<td>Detection system, strobe &amp; horn, ADA type, excluding wires &amp; conduits</td>
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<td>$216.95</td>
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<td>$277.55</td>
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<td>$120.55</td>
<td>277.55</td>
<td>$7,065.00</td>
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<td>Video surveillance cameras, wireless, hidden in exit signs, clocks, etc, includes receiver</td>
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<td>$279.68</td>
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<td>$9,572.85</td>
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<td>Video surveillance, master monitor station, 3 doors x 5 color monitor with tilt feature, complete</td>
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<td>$319.10</td>
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52,218.81 $65,811.13 $3,551,054.86 $626,011.36 $195,979.67 $5,323,526.68
## Appendix V – Project Schedule GANNT Chart

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<th>ID</th>
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<th>Duration</th>
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<td>Senior Design/Draft Schedule</td>
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<td>Mon 5/2/22</td>
<td>Thu 9/1/22</td>
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<td>1</td>
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<td>1 Site Prep/Mobilization</td>
<td>5 days</td>
<td>Mon 5/2/22</td>
<td>Fri 5/6/22</td>
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<tr>
<td>2</td>
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<td>2 Excavation/Removal of Existing Land</td>
<td>3 days</td>
<td>Mon 5/8/22</td>
<td>Wed 5/11/22</td>
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<tr>
<td>3</td>
<td></td>
<td>3 Grading of Soil</td>
<td>2 days</td>
<td>Thu 5/12/22</td>
<td>Fri 5/13/22</td>
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<tr>
<td>4</td>
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<td>4 Drive Piles and installation of Pile Caps</td>
<td>15 days</td>
<td>Mon 5/16/22</td>
<td>Fri 6/3/22</td>
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<tr>
<td>5</td>
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<td>5 Order Slab MEP</td>
<td>15 days</td>
<td>Mon 5/16/22</td>
<td>Fri 6/3/22</td>
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<tr>
<td>6</td>
<td></td>
<td>6 Foundation Walls</td>
<td>10 days</td>
<td>Mon 6/6/22</td>
<td>Fri 6/17/22</td>
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<tr>
<td>7</td>
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<td>7 Slab On Grade</td>
<td>5 days</td>
<td>Mon 6/6/22</td>
<td>Fri 6/10/22</td>
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<tr>
<td>8</td>
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<td>8 Steel Framing</td>
<td>10 days</td>
<td>Mon 6/13/22</td>
<td>Fri 6/24/22</td>
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<tr>
<td>9</td>
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<td>9 Erect Columns</td>
<td>10 days</td>
<td>Mon 6/13/22</td>
<td>Fri 6/24/22</td>
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<tr>
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<td>10 Set Beams Level 1</td>
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<td>Mon 6/27/22</td>
<td>Wed 6/29/22</td>
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<td>11 Place Double Tees Level 1</td>
<td>2 days</td>
<td>Thu 6/30/22</td>
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<td>12 Level 1 Top Precast Concrete Panel</td>
<td>5 days</td>
<td>Mon 7/4/22</td>
<td>Fri 7/8/22</td>
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<tr>
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<td>13 Level 1 Prefabricated Stair Installation</td>
<td>2 days</td>
<td>Mon 7/11/22</td>
<td>Tue 7/22/22</td>
</tr>
<tr>
<td>14</td>
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<td>14 Erect Level 2 Columns</td>
<td>4 days</td>
<td>Mon 7/11/22</td>
<td>Thu 7/14/22</td>
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<tr>
<td>15</td>
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<td>15 Set Beams</td>
<td>3 days</td>
<td>Fri 7/15/22</td>
<td>Tue 7/19/22</td>
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<td>17 Level 2 Top Precast Concrete Panel</td>
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<td>Mon 8/2/22</td>
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<tr>
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<td>19 Level 2 Roof Precast Concrete Panel</td>
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<td>Thu 8/4/22</td>
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<td>20</td>
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<td>20 Glazing/Curtainwall Systems</td>
<td>10 days</td>
<td>Fri 8/5/22</td>
<td>Thu 8/28/22</td>
</tr>
<tr>
<td>21</td>
<td></td>
<td>21 Structure MEP Rough-ins</td>
<td>20 days</td>
<td>Fri 7/21/22</td>
<td>Thu 8/11/22</td>
</tr>
<tr>
<td>22</td>
<td></td>
<td>22 Site Work (Approaches, Sidewalk, Site Utilities)</td>
<td>30 days</td>
<td>Mon 6/13/22</td>
<td>Fri 7/22/22</td>
</tr>
<tr>
<td>23</td>
<td></td>
<td>23 MEP Systems and Devices Installations</td>
<td>15 days</td>
<td>Fri 8/2/22</td>
<td>Thu 8/9/22</td>
</tr>
<tr>
<td>24</td>
<td></td>
<td>24 Final Landscaping (Trees, Shrubs, Grass)</td>
<td>15 days</td>
<td>Mon 7/25/22</td>
<td>Fri 8/12/22</td>
</tr>
<tr>
<td>25</td>
<td></td>
<td>25 Final Pavement Markings</td>
<td>2 days</td>
<td>Fri 8/5/22</td>
<td>Mon 8/8/22</td>
</tr>
<tr>
<td>26</td>
<td></td>
<td>26 Testing and Inspections</td>
<td>5 days</td>
<td>Tue 8/8/22</td>
<td>Mon 8/15/22</td>
</tr>
<tr>
<td>27</td>
<td></td>
<td>27 Closeout</td>
<td>5 days</td>
<td>Tue 8/9/22</td>
<td>Mon 8/15/22</td>
</tr>
<tr>
<td>28</td>
<td></td>
<td>28 Owner Move-In</td>
<td>3 days</td>
<td>Tue 8/16/22</td>
<td>Thu 8/18/22</td>
</tr>
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