Optimal Civil Designs

Coalburg Road Warehouse Project

Birmingham, AL

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December 9, 2011

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2. Executive Summary

Optimal Civil Designs has prepared a thorough analysis and design for the Coalburg Road warehouse project. Areas of analysis included in the report are transportation, geotechnical, structural, environmental, and construction management.

The transportation team analyzed current traffic conditions, estimated future traffic conditions, and performed a traffic impact study for the warehouse. The team also designed the warehouse parking lot, loading dock, and on-site access roads. All design features met or exceeded specified requirements. It was determined that the warehouse would produce on average 147 additional passenger vehicle and 50 semi-truck trips per day on local traffic. After performing extensive traffic analysis it was determined that the new warehouse was determined to have minimal impact on local traffic.

The geotechnical team analyzed the Report of Geotechnical Explorations provided by Goodwyn, Mills, and Cawood. The team also used a topographic map of the area to compute volumes of soil for cut, fill, and surcharge. The topographic map was also used to determine the best location for the warehouse on the site as well as the dimensions for the building pad, locations of drives and parking lots, and the direction of water flow on the site. The warehouse was positioned to the north side of the job site to take advantage area of the soil that requires no treatment or surcharge, confirmed by three boring holes in that area. Finally, the geotechnical team aided the structural team in the design of interior and exterior footings for the warehouse structure.

The structural team produced a cost effective and functional design for the storage warehouse. The design included a braced frame with steel decking acting as a roof diaphragm, a 9,600 s.f. office space on two floors within the structure of the warehouse, slab on grade designed for lift truck traffic and storage racks, and concrete tilt-up panels for the exterior walls. A cost analysis was performed with varying column spacing to determine the most cost effective spacing for the column bays. The column bays were determined to be 40 ft by 40 ft and used K series open web steel joists and joist girders. HSS square tube columns and cable bracing were also chosen for cost effectiveness. The second floor slab for the office space was designed as a one-way continuous slab, 4 in. thick with steel reinforcing bars. The framing for the second floor office space was determined to be W12X120 for the beams and W18X192 for the girders. The slab on grade was found to be 11 in. thick with welded wire reinforcement placed 2 in. below the surface of the slab to control the effects of shrinkage. The concrete tilt-up panels for the exterior walls included a solid panel, a panel with an opening of 10 ft by 15 ft for the loading dock, and a panel with an opening of 6 ft by 9 ft for a personnel door.

The environmental team assessed the potable water and sanitary sewer demands for the warehouse occupancy and designed a system to meet these demands by using modeling software and identifying where the existing mains were located that run along Coalburg road, the mains sizes, velocities, and flow capacities. The environmental team also designed the storm water drainage system for a 25 year storm and redesigned the retention pond to meet to a capacity of a 50 year storm so that it will have the ability to handle additional facilities on the property or an expansion of the warehouse creating an increase in storm water runoff due to additional impervious surfaces. Also, the environmental team implemented sustainable low impact design characteristics into the project including the use of best management practices for the construction phase to reduce sediment runoff, passive lighting by placing windows in the warehouse, and maximizing pervious surfaces with the use of open channel grass swales and landscaped areas.

The construction management team created a detailed project schedule and used economic reasoning to estimate a total project cost. In doing so, using information from the geotechnical report, the soil for the building slab will be treated with dynamic compaction. This treatment will save the owner approximately $640,000, reducing the overall project cost by 5%. Also taken into consideration was the utilization of available space, using the building slab as a location to construct the 104 tilt-up wall panels. Our team has provided a blueprint schedule to enable the general contractor to successfully complete the project in the estimated seventeen months outlined on the Gantt chart. The total estimated construction time for the warehouse is 17 months with a total cost of $11.9 million.

The proposed site layout for the warehouse is located in Figure ES1 on the following page. The position of the warehouse to the north side of the property allows the owner to more efficiently use their property. The position of the warehouse allowed the team to produce a safe, economic design while specifically leaving room for future expansion. As seen in Figure ES1, the 80,000 ft^2 area to the south of the building is area that could be used for expansion if the owner so chooses. This could be done without changing any of the proposed infrastructures, excluding the building expansion itself.

Optimal Civil Designs has carefully considered the aforementioned areas in designing the warehouse. Our main goal was to produce a safe, cost effective design while meeting or exceeding design standards. We feel that we have been able to achieve these goals.

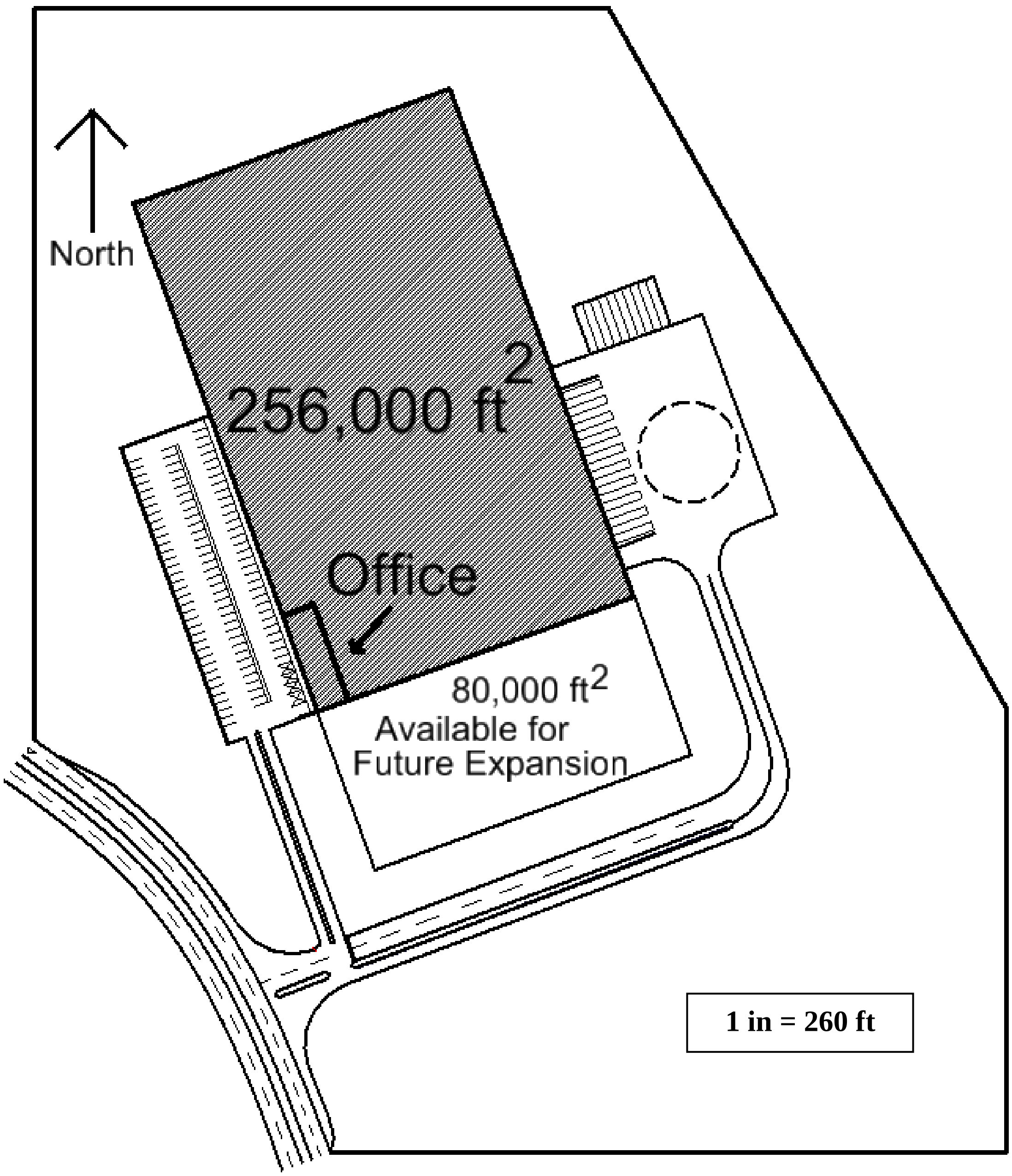

Figure ES1: Site Layout

3. Introduction

Design Constraints

- 250,000 square feet of warehouse space
- Warehouse must have minimum clear height of 20 feet
- 10,000 square feet of office space over two floors
- 10 loading dock bays
- Driveways and loading docks must be able to accommodate WB-62 size semi-trucks
- Future traffic must maintain existing LOS or better at key intersections
- Storm water must be handled on-site
- Water and sewer design must meet all fire and building codes
- Geotechnical and foundations should be designed as to minimize cost and maximum use of existing topography
- Steel structure is to be designed as to minimize overall costs
- Design must meet all codes

A developer wishes to design and construct a 250,000 s.f. warehouse on a parcel of land located on the northwest side of Birmingham, AL. The warehouse will include 10,000 s.f. of office space, contained in two stories and a loading dock with ten bays. The total number of employees working on site will be 110. The job site is roughly 32 acres and is located on Coalburg Road, approximately one mile west of I-65 and two miles south of I-22. Figure I1 shows the location of the warehouse in relation to downtown Birmingham.

The site has roughly 650 ft of Coalburg Road frontage available for access, some of which is located along a curve with elevation change. This brings sight distance and safety into consideration when designing the entrance and exit for the warehouse. The job site contains a significant amount of mine spoils that will need to be treated prior to construction. A full geotechnical report prepared by Goodwyn, Mills and Cawood, Inc. was used as a resource for all geotechnical designs. An existing retention pond of 14.2 acre-feet is located on site and will be utilized to collect storm water runoff from the warehouse.

Organization of the Report

This report includes comprehensive analysis and design in areas including transportation, geotechnical, structural, environmental, and construction management. The transportation team conducted a traffic impact study of the warehouse and designed all on site transportation features. The geotechnical team analyzed existing soil conditions and prepared an excavation and soil treatment plan to adequately support the loads of the warehouse and parking/driveway areas on site. The structural team was given design requirements of square footage and building functionality. The design meets all standards and provides a safe warehouse structure but maintains a cost effect design by minimizing steel use with a 40 ft x40 ft column spacing grid. The environmental team assessed existing hazardous conditions and designed potable water, sanitary sewer, and storm sewer systems. They considered sustainable design methods such as low impact design and the use of renewable resources throughout the scope of the project. Finally, our construction management team developed a detailed project schedule and pricing structure for the warehouse. They carefully priced material vendors, using local vendors and contractors where possible to ensure our bid proposal included the most competitive pricing available in the market.

The engineers at Optimal Civil Design have carefully considered every aspect of the design. Our project proposal provides you the most cost effective design, while satisfying or exceeding required local, state, and federal design standards.

Figure I1: Job Site Location

4. Transportation

Project Background

The construction of a new warehouse will impact traffic on Coalburg Road and at the intersection of Daniel Payne Dr. and Coalburg Rd. A traffic analysis was needed to determine what impact the traffic generated by the warehouse will have not only on Coalburg Rd., but to the intersection as well. The transportation team performed the following:

1) Evaluated the existing traffic conditions in the vicinity of the site.
2) Projected future traffic conditions with site traffic and determine whether roadway improvements will be needed to mitigate any impacts.
3) Determined parking requirements and design of the parking lot.
4) Developed designs for driveways and loading dock areas.

The project study area is shown in Figure T1. The study area is a commercial land use area. Access to the proposed warehouse will be primarily from the south via the Daniel Payne Dr. and Coalburg Rd. intersection, with the remainder of the traffic coming from the north on Coalburg Rd. The speed limits for the roads of interest are 45 mph for Coalburg Rd. and 50 mph for Daniel Payne Dr.

The study accounted for the expected completion of the I-22 corridor to the I-65 interchange in 2014. This opening is expected to reduce the amount of traffic on Coalburg Road and at the intersection of Daniel Payne Drive and Coalburg Road. At the time of this study, significant traffic exits I-22 onto Coalburg Road and travels through the intersection at Daniel Payne Drive to reach I-65. In the future, I-22 will connect directly to I-65 thus alleviating much of this traffic.

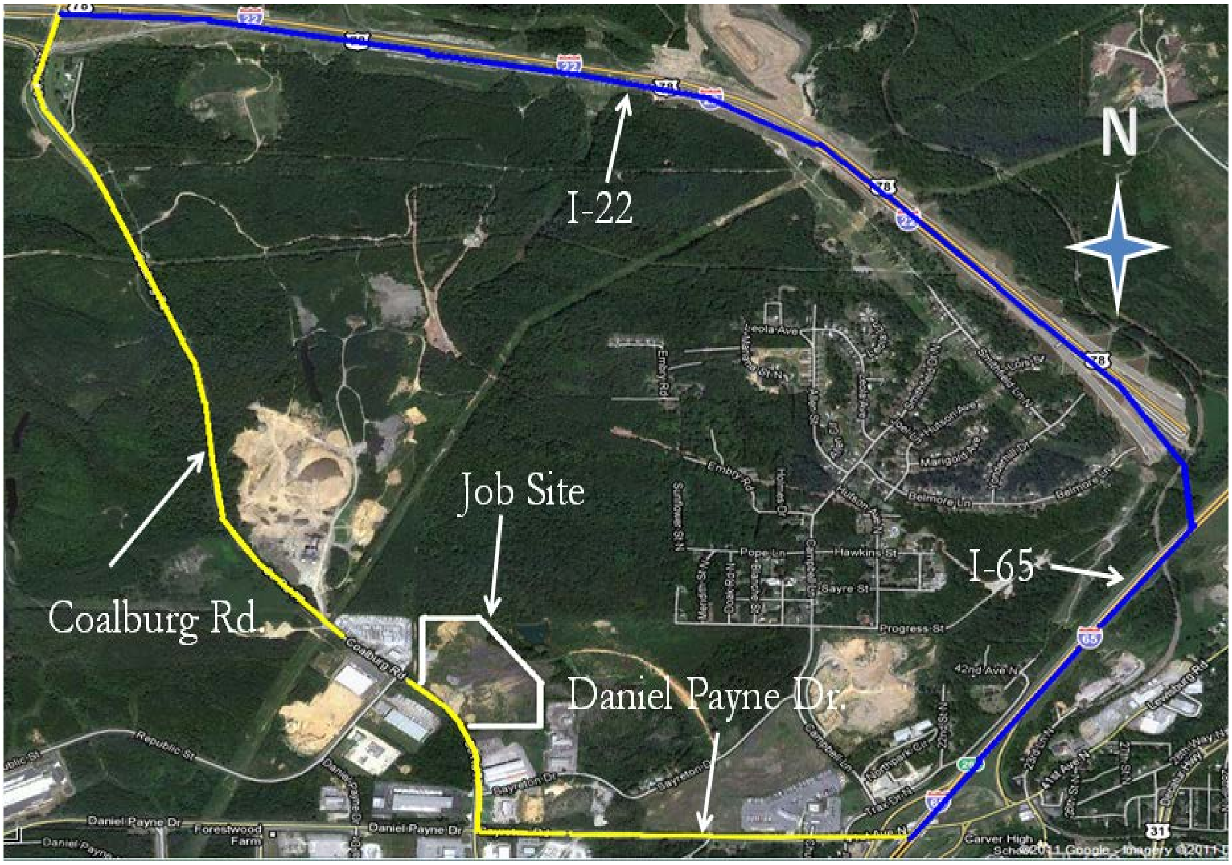

Figure T1. Site location with respect to surrounding area.

Existing Conditions and Analysis

Intersection of Daniel Payne Drive and Coalburg Road

Existing traffic conditions and geometry at the intersection of Coalburg Road and Daniel Payne Driver were observed and recorded, as shown in Figure T2, in order to determine the present Level of Service (LOS). Data collected included signal phasing and timing data, approach speeds, and peak hour turning and thru counts for all vehicle movements. Existing intersection geometry included lane width, and intersection widths. Figure T3 shows the signal phasing and Table T1 shows the signal timing.

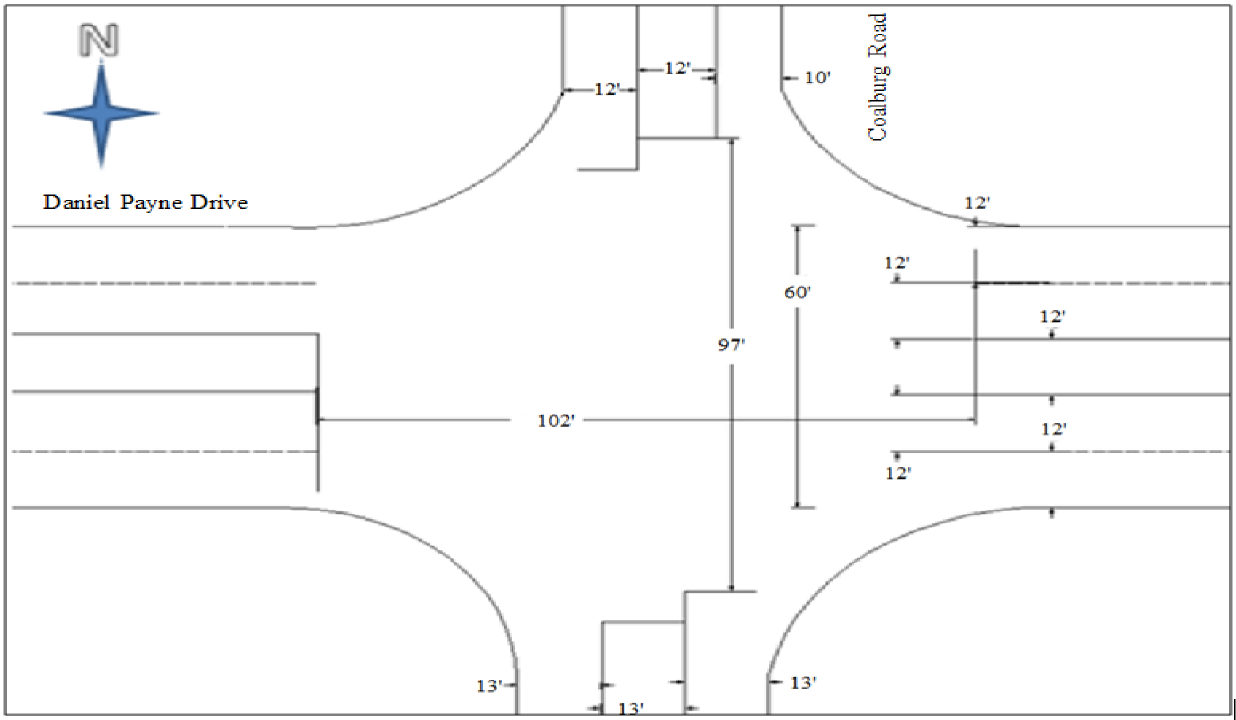

Figure T2. Current Intersection Geometry

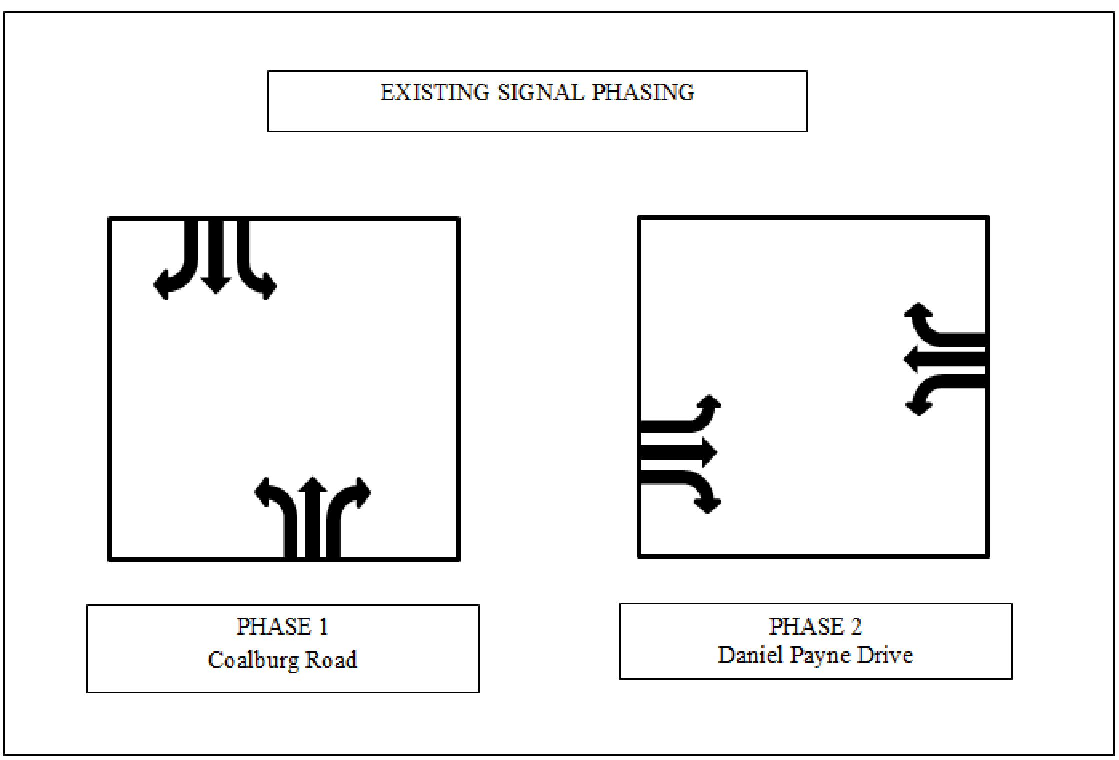

Figure T3: Coalburg Road and Daniel Payne Drive Signal Phasin

Table T1: 2011 - CURRENT SIGNAL TIMING (AM & PM)

Daniel Payne Drive and Coalburg Road Intersection

	Φ1	**Φ2**
Green	30.0	38.0
Yellow	4.5	4.5
All Red	2.0	2.0
Cycle Length	81.0	

The morning traffic count for the intersection was performed between 7:30 – 9:00 A.M., while the afternoon count was performed between 4:30 and 6:00 P.M... The peak hour volumes for each are shown in Figure T4.

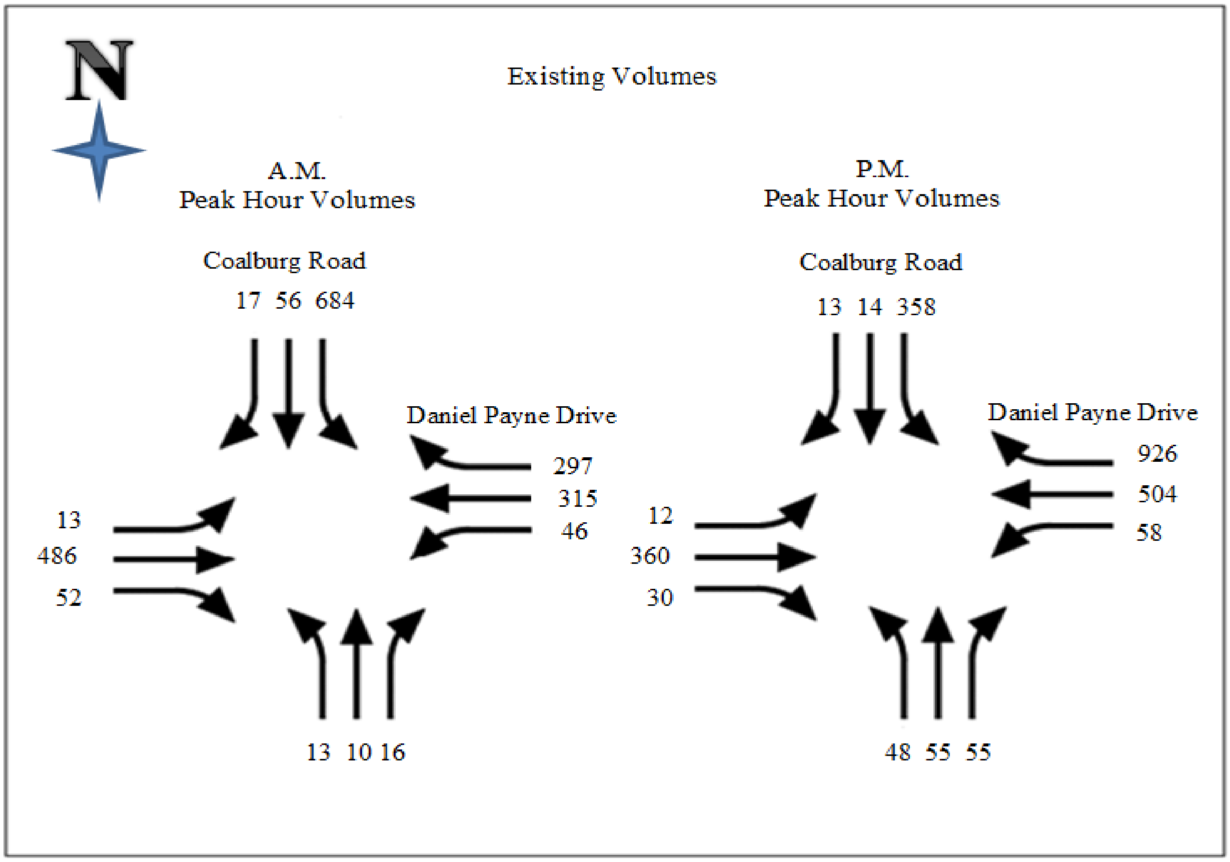

Figure T4: Existing AM & PM Peak Hour Traffic Volumes – Daniel Payne Dr. at Coalburg Rd

The latest edition of Highway Capacity Manual was used to perform a capacity analysis of current conditions and yielded an intersection a LOS of E in the morning peak. This indicates that the intersection is operating near capacity. The low rating of the intersection is directly attributable to the south bound traffic on Coalburg Rd. turning left onto Daniel Payne Dr. A LOS of D was calculated for the afternoon peak, which indicates the presence of decreased travel speeds and increased delays. This rating is directly influenced by the traffic on Daniel Payne Dr. turning right onto Coalburg Rd. These critical movements are shown in Figure T5. The delay and LOS for each movement for the morning and evening can be found in Table T2.

Figure T5: Critical Intersection Movements

Table T2: Delay and LOS by Movement

	AM			
	Coalburg Rd	Daniel Payne Dr	Coalburg Rd	Daniel Payne Dr
	SOUTHBOUND	WESTBOUND	NORTHBOUND	EASTBOUND
Delay (seconds/vehicle)	125.6	21.2	11.7	21.7
Approach LOS	F	C	B	C
Intersection LOS	E			
	PM			
Delay (seconds/vehicle)	21.8	76.9	12.7	19.3
Approach LOS	C	E	B	B
Intersection LOS	D			

Current Traffic Flow for Coalburg Road

The traffic flow for Coalburg Road was found using an automatic tube counter. Analyzing the data collected over a seven day period, the traffic volumes for the morning and the afternoon peak times were found and can be seen in Table T3.

Table T3: Peak Traffic Flow for Coalburg Road (2011)

	NB (vph)			SB (vph)			Bi-Directional	
Time	Auto	Truck	Total	Auto	Truck	Total	Total (vph)	%Trucks
07:00 AM	267	8	275	709	13	722	997	2.1%
05:00 PM	875	12	887	340	3	343	1230	2.9%

Future Conditions and Analysis

The purpose of performing a future analysis was to determine what impact the construction of the proposed warehouse would have on Coalburg Road and the intersection of Danial Payne Drive and Coalburg Road due to:

- Employees of
- Additional traffic generated by the warehouse and office space
- Truck traffic generated by the warehouse

Future Traffic Flow without Warehouse

An analysis year of 2016 was chosen for the future traffic analysis. This would allow us to consider traffic from the proposed warehouse as well as redistributed background traffic that will result from the completion of I-22 to I-65. The traffic flow for the year 2016 was calculated by taking the baseline flow from 2011 and reducing it by 80% to account for the redistribution of traffic to I-65 via the I-22 corridor. These volumes were confirmed using data collected on Coalburg Road prior to the I-22 interchange being open. This data is shown in Table T4.

Table T4: Baseline Traffic Flow for Coalburg Road without Warehouse (2016)

	NB (vph)			SB (vph)			Bi-Directional	
Time	Auto	Truck	Total	Auto	Truck	Total	Total (vph)	% Trucks
07:00 AM	53	2	55	142	3	144	199	2.1%
05:00 PM	175	2	177	68	1	69	246	2.9%

Future Intersection Analysis without Warehouse

Utilizing the future baseline numbers developed in the previous section, each movement from the intersection was increased by 2% per year for five years to assume an approximation for 2016. The traffic flow numbers found for the 2016 traffic flow found in Table T4 were then distributed to the appropriate morning and afternoon turning movements. Utilizing Highway Capacity software, a LOS of B was found for both the morning and afternoon peak traffic times.

A breakdown of each movement can be found in Table T5.

Table T5: 2016 Baseline Intersection Delay and LOS for Daniel Payne Drive and Coalburg Road

AM				
	Coalburg Rd.	Daniel Payne Dr.	Coalburg Rd.	Daniel Payne Dr.
	SOUTHBOUND	WESTBOUND	NORTHBOUND	EASTBOUND
Delay (seconds/vehicle)	16.5	19.0	11.7	21.9
Approach LOS	B	B	B	C
Intersection LOS	B			
PM				
Delay (seconds/vehicle)	12.6	22.6	12.9	19.7
Approach LOS	B	C	B	B
Intersection LOS	B			

Projected Warehouse Trip Generation

Using the ITE Trip Generation Manual, it was calculated the office space would generate 15 vehicle trips during the AM and 18 vehicle trips during the afternoon. Again using the ITE Manual, it was calculated the warehouse would generate 7 trucks per hour during peak working hours. A total of 417 vpd maximum would be generated by the warehouse and office. It was estimated that 80% of the warehouse workers would likely arrive and depart to the south. The same assumption was made for the truck traffic. This is due to the fact that the majority of the building material distributers in the area are located south of the warehouse. With this in mind, the peak hour traffic was distributed accordingly as shown in Figure T6. Calculations for trip generation can be found in the appendix.

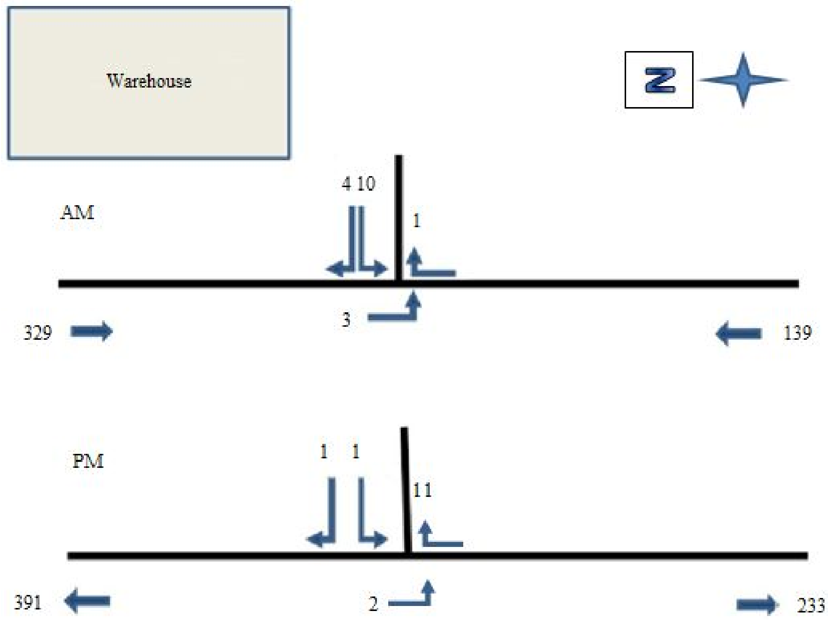

Figure T6: Trip Distribution for AM and PM Peak Hours (vph)

<u>Coalburg Road Traffic Flow</u>

The north bound and south bound traffic was reduced accordingly, and the new truck traffic and office traffic generated by the warehouse were figured into the new flow. These results are shown in Table T6.

Table T6: Peak Traffic Flow for Coalburg Road (2016 with Warehouse)

	NB (vph)			SB (vph)			Bi-Directional	
Time	Auto	Truck	Total	Auto	Truck	Total	Total (vph)	%Trucks
07:00 AM	131	8	139	187	3	190	329	8.1%
05:00 PM	220	3	223	161	7	168	391	6.3%

Effect of Warehouse on Coalburg Road

With the calculated traffic flow for Coalburg Road for 2016 and the trip generation for employees and trucks found, a capacity analysis for the site driveways was performed using Highway Capacity Manual procedures. A LOS of A was found for both the morning and afternoon peak traffic times. This indicates the warehouse will have a minimal effect on Coalburg Road. The results of the analysis can be found in Table T7.

Table T7: Two Way Traffic Analysis Coalburg Rd. and Warehouse

AM			
	Warehouse	Coalburg Rd.	Coalburg Rd.
	Enter	SOUTHBOUND	NORTHBOUND
Delay (seconds/vehicle)	0	11.2	9.4
Approach LOS	A	B	A
Intersection LOS	A		
PM			
	Warehouse	Coalburg Rd.	Coalburg Rd.
	Exit	SOUTHBOUND	NORTHBOUND
Delay (seconds/vehicle)	7.4	10.0	8.8
Approach LOS	A	A	A
Intersection LOS	A		

Daniel Payne Drive and Coalburg Road Intersection Future Analysis with Warehouse (2016)

The traffic generated by the warehouse employees and truck traffic generated by the warehouse were added to the baseline traffic flow numbers and distributed to the corresponding turning movements. The distribution numbers can be found in Figure T7. Using Highway Capacity Analysis Manual, a LOS of C was found for the morning peak, and a LOS of B for the afternoon peak. Both indicate reasonable free-flow operations. The complete LOS summary by movement can be seen in Table T8. By comparing the 2016 LOS for the intersection without the warehouse and the LOS with the warehouse, only the westbound movement shows any change, decreasing the LOS for that movement from a B to a C. The LOS for the intersection increases significantly compared the baseline analysis. The data shows the opening of the I-22 corridor will decrease traffic flow on Coalburg Road and increase the LOS of the intersection significantly, but the construction of the warehouse will have minimal effect on Coalburg Road and at the intersection of Daniel Payne Drive and Coalburg Road.

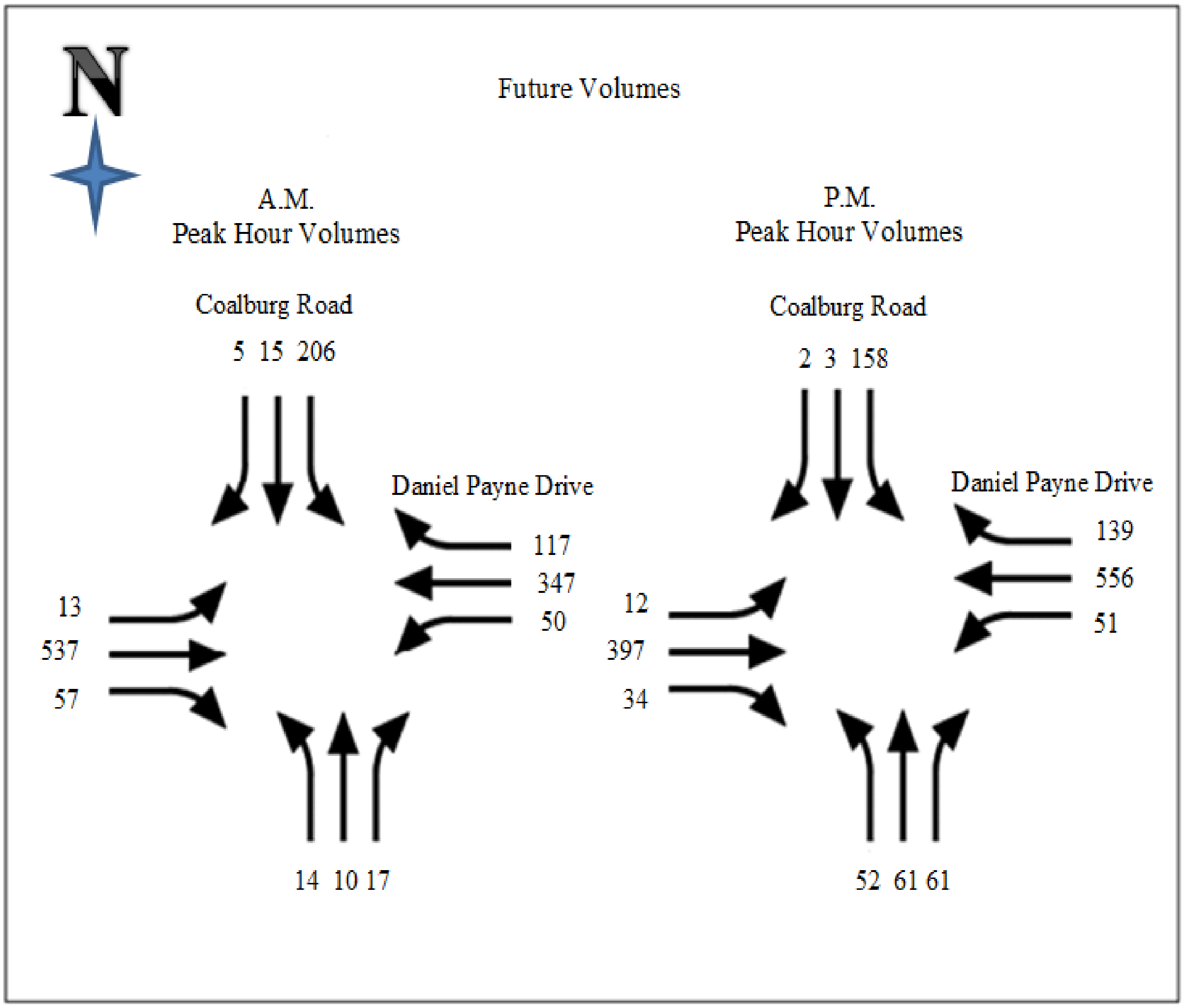

Figure T7: Projected AM & PM Peak Hour Traffic Volumes with Warehouse – Daniel Payne Dr. at Coalburg Rd.

Table T8: Intersection Delay and LOS of Daniel Payne Drive and Coalburg Road by Movement (2016)

AM				
	Coalburg Rd.	Daniel Payne Dr.	Coalburg Rd.	Daniel Payne Dr.
	SOUTHBOUND	WESTBOUND	NORTHBOUND	EASTBOUND
Delay (seconds/vehicle)	16.6	21.3	11.8	21.9
Approach LOS	B	C	B	C
Intersection LOS	C			
PM				
Delay (seconds/vehicle)	15.3	21.8	12.9	19.7
Approach LOS	B	C	B	B
Intersection LOS	B			

Design of On-Site Infrastructure

Parking Lot Design

The passenger car parking lot will be located adjacent to the office complex, allowing for easy entry and exit for employees and visitors. At any given time up to 110 employees could be on site. When designing the parking lot an assumption was made that the number of visitors coming on site will be minimal because the building is a distribution warehouse.

The *City of Birmingham Zoning Ordinance* says the parking lot must contain at least one parking space per three employees. With all assumptions and zoning requirements considered the parking lot was designed for 100 spaces. The 100 parking spaces exceed the minimum of 37 recommended by the City of Birmingham and allows room for growth if the warehouse is expanded in the future. In compliance with American Disability Act requirements, the parking lot includes three 12 foot wide handicap spaces and an additional 15 foot wide handicap space wide enough for van access. The handicap spaces are the located nearest the office door, indicated by the X in Figure T9.

The City of Birmingham Zoning Ordinance requirements state that parking spaces must be at least 9 feet wide and 18 feet deep. The warehouse parking spaces, excluding handicapped, were deigned to be 9.5 feet wide and 18 feet deep. They were designed slightly wider than minimum requirements with the assumption that many employees would be driving larger trucks or SUVs.

The lot will have two aisles. The aisle farthest from the warehouse will have parking on both sides while the aisle closest to the warehouse will only have parking on one side. The aisles will each be 25 feet wide. The aisle width allows for two way traffic and gives automobiles sufficient room to safely back out of the 90^o parking spaces. Traffic will enter and exit the lot through the aisle farthest from the building. This minimizes traffic near the entrance of the warehouse, providing safer conditions for pedestrians entering and exiting the building.

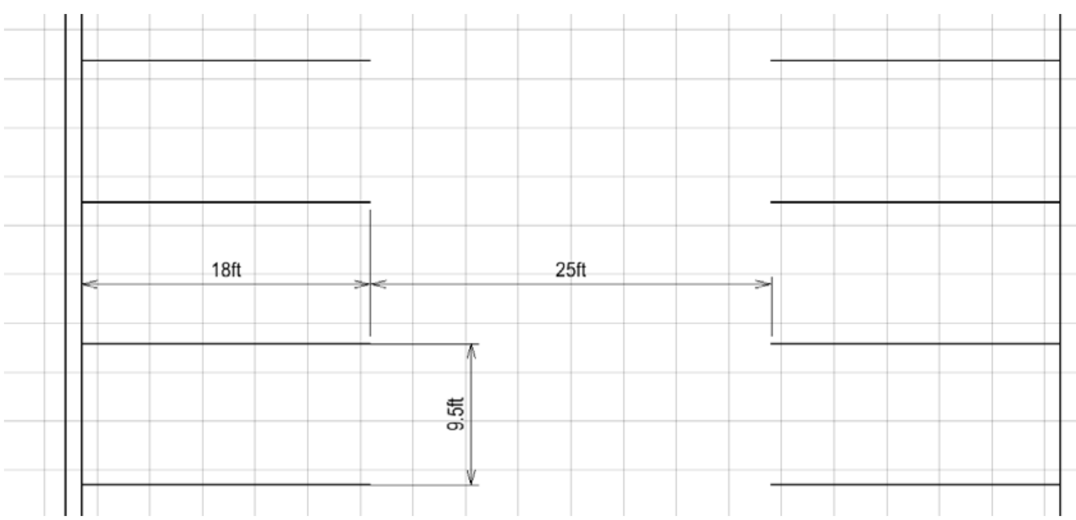

Figure T8: Parking Space Design (feet)

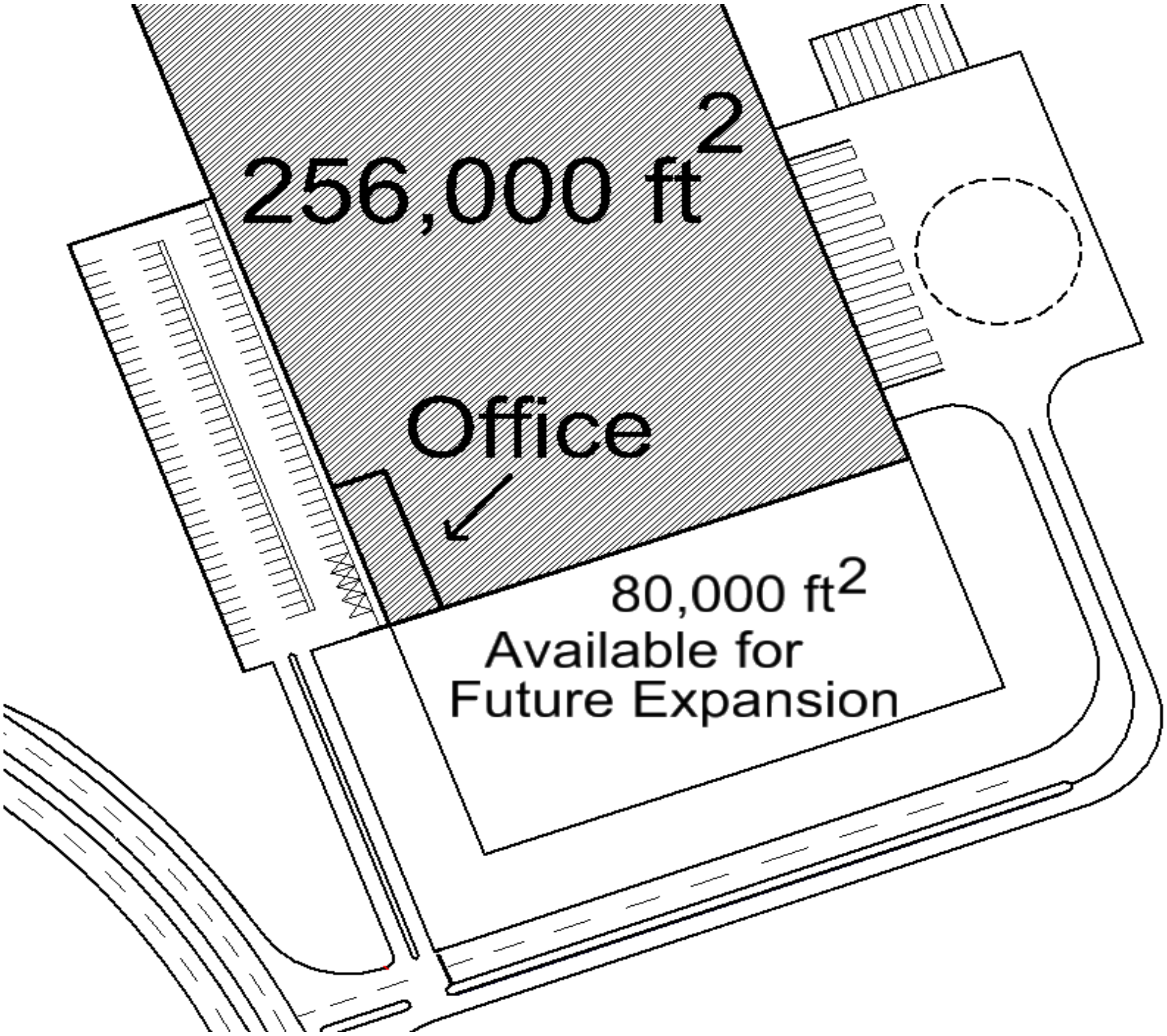

Figure T9: Parking Lot Overview

Loading Dock Design

The project stated that ten semi-truck loading docks be included in the warehouse design. Figure T10 gives an overview of the loading dock design, which will be located in the southeast corner of the warehouse. The entire staging area will be 250 feet wide by 190 feet deep. A 200 foot wide by 50 foot deep concrete pad will be placed at the loading dock to absorb the load of a fully loaded semi-truck. The 50 foot width will accomodate the length of a loaded trailer.

In the event that a semi-trailer needs to be stored we have included a storage area for trailers. The 100 foot by 60 foot area will not be laid with asphalt, but with gravel to reduce cost for this minimally used area. The overflow storage lot will be laid with 6 inches of standard coarse aggregate gravel. RSMeans suggests both 6 inch and 9 inch gravel layers as options for this area. Six inches was chosen to reduce cost as this area is expected to be infrequently used.

The 120 foot diameter turning area in the loading dock area provides sufficient room for a WB-62, or similar, semi-truck to turn. Safety is a concern in a high traffic area such as this so we want truck drivers to have as much maneuvering room as possible.

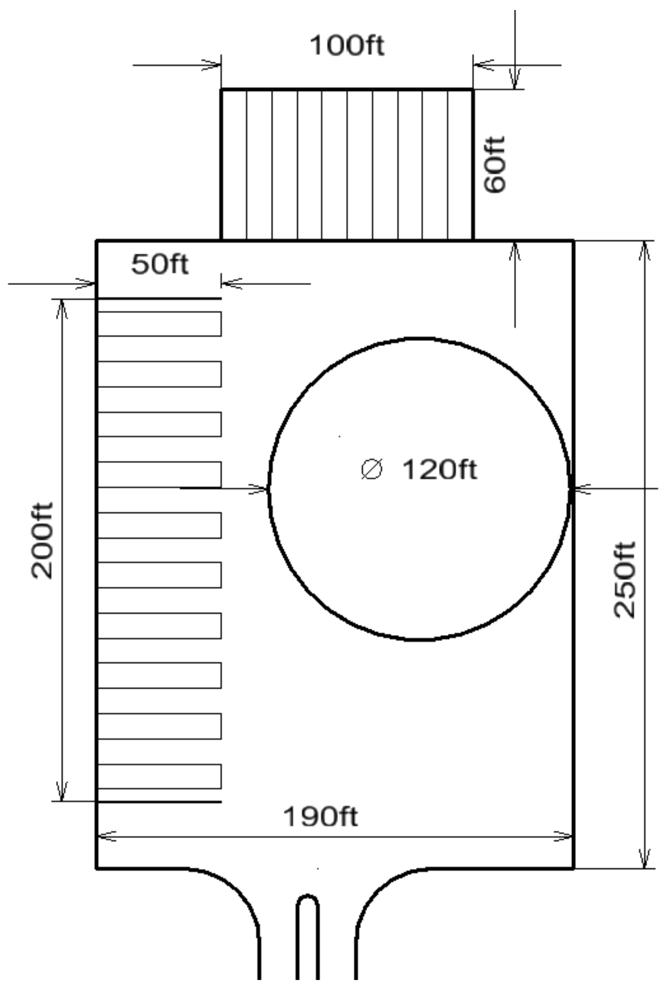

Figure T10: Loading Dock Design

Loading Dock Concrete Pad

The loading dock concrete pad will be a 200 feet by 50 feet slab on grade, as described above. The *AASHTO Guide for Design of Pavement Structures* and the reference book *Designing Floor Slabs on Grade* were used for the design. A 4 inch base layer of coarse aggregate gravel will be laid under the concrete slab.

Using the nomograph method in section 3.2.2 of the *AASHTO Guide for Design of Pavement Structures* the slab thickness was determined to be 8 inches. The nomograph method required several variables be determined in order to find the required thickness. According to the geotechnical report the soil on site is classified by the Unified Classification System as SM-SC with a subgrade modulus of 150 pci. The compressive strength of the concrete used was 4,500 psi, which produced an Elastic Modulus of 3.8 x 10^6 psi. The Modulus of Rupture (MOR) was 604 psi, found by using the Portland Cement Method (PCA) and a compressive strength of 4,500 psi. A drainage coefficient of 1.25 was used based on a "good" quality of drainage and less than 5% of time pavement is exposed to saturated moisture levels. The terminal serviceability life was estimated to be 2.5, which is when 55% of people would state the pad is unacceptable and needs replacement.

The slab has longitudinal steel reinforcement to help control cracking due to shrinkage and temperature changes. The amount of steel reinforcement was determined using the Temperature Method equation:

$$A_s = \frac{12tf_r}{2(\mathrm{f}_s - T\alpha E_s)}$$

A_s = cross-sectional area of steel in square inches per linear foot of slab width

t = slab thickness (8 in)

f_r = tensile strength of concrete, 0.4 * MOR = 0.4 * 604 = (241.6 psi)

fs = allowable steel stress, (45,000,000 psi)

T = range of temperature the slab is expected to be subjected to, 80^o - 15^o = (65^o F)

α = thermal coefficient of concrete, type of coarse aggregate is gravel, (6.0 x 10^{-6} in/in/^{o}F)

E_s = modulus of elasticity of steel, (29 x 10^6 psi)

Roughly 0.344 in^2 of steel reinforcement per linear foot of slab will be needed to resist cracking. Welded wire reinforcement with a designation of 4x4-W5.5xW5.5 was chosen as the reinforcing steel in the loading dock concrete pad. Contraction joints will be placed every 15 feet across the width of the concrete pad. Two inches of cover is necessary between the reinforcement and the surface. A cross section of the slab can be found on the following page.

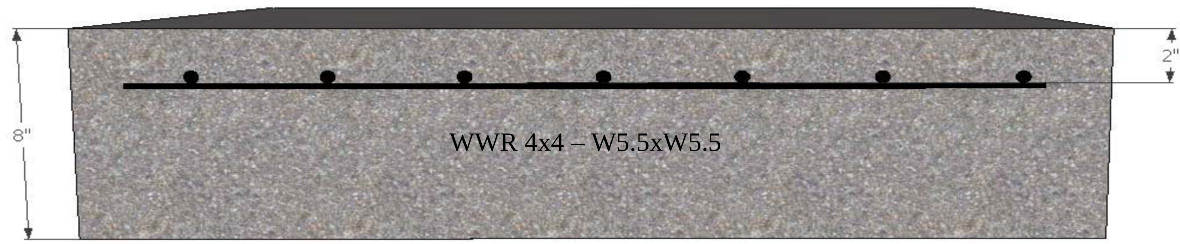

Figure T11 – Loading Dock Concrete Slab Cross Section

Retaining Wall

The loading dock required a retaining wall to account for the 39 inch drop from the bottom of the warehouse slab to the top of the reinforced concrete slab to hold back the soil under the warehouse. With the retaining wall being under five feet tall, it did not need to be engineered, but with the force of the warehouse slab being placed on the top of the retaining wall, all applicable calculations were performed to ensure this added weight would not cause the wall to rotate or overturn from the pressure of the soil and the slab. A diagram of the wall is shown in Figure T12.

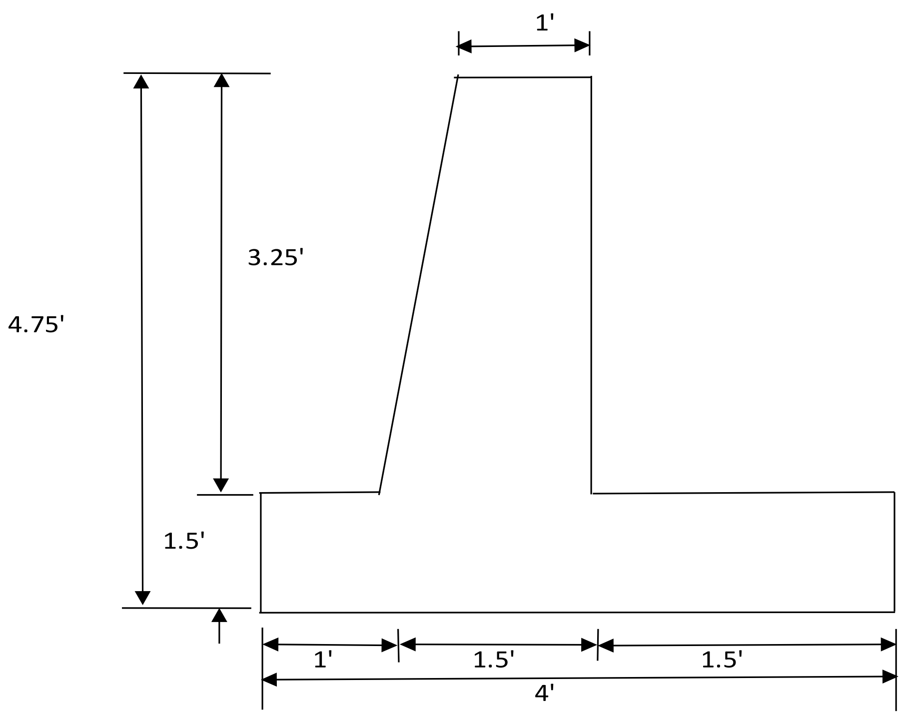

Figure T12: Loading dock retaining wall

The overturning moment was calculated to be 1655 ft – lb, and the Righting moment was calculated to 4908 ft – lb. By dividing the Righting moment by the overturning moment a safety factor of 2.9 was found which is greater than 2 which satisfy code. Using a sliding friction of $\mu = 0.5$ for concrete on soil, the Resisting Force was determined to be 1270ft - lb., then divided by the force causing sliding (817 ft - lb), the Safety Factor of 1.55 was calculated, which is greater than 1.5, which indicate the retaining wall will not slide.

The amount of reinforcement was determined next using the Design of Reinforced Concrete 8th edition ACI 318-08 code edition. Figure T13 shows the size and steel placement for reinforcement. All calculations can be found in the appendix.

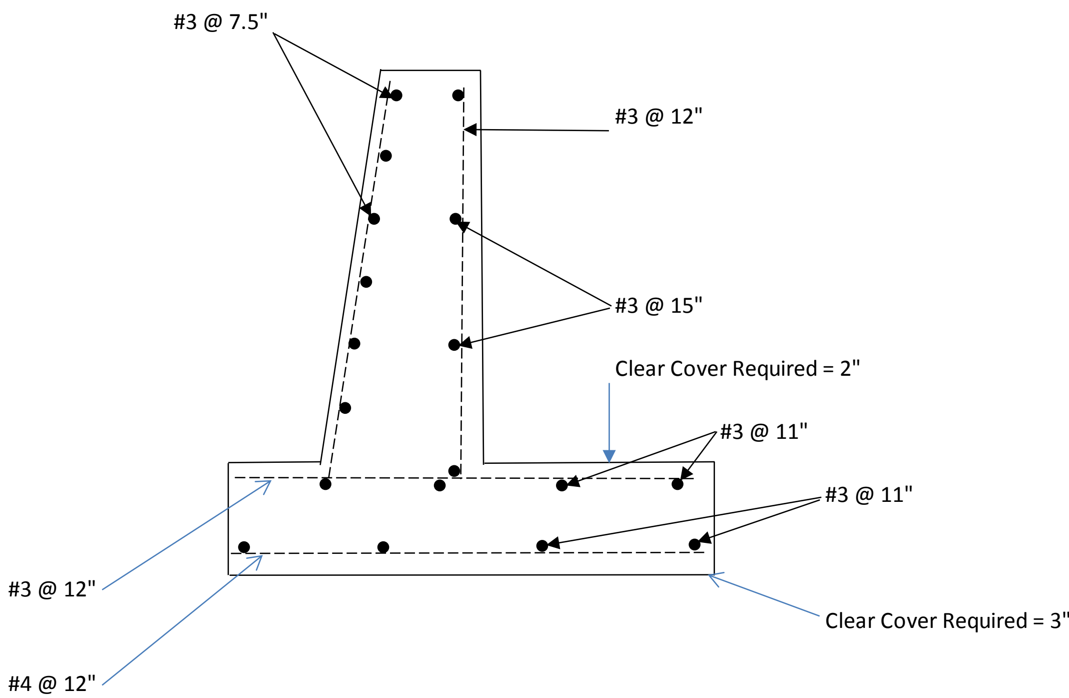

Figure T13: Placement of Reinforcement in Loading Dock Retaining Wall

Entrance and Exit Design

The warehouse will have one entrance shared by passenger car and semi truck traffic. A single entrance was chosen due to limited sight distances to the south along Coalburg Road. The job site is located on a curved section of Coalburg Road with significant elevation change. The topography of the land makes for a challenging entry/exit to the property for those vehicles having to cross north bound traffic.

Laser range finder technology was used to take sight distances were taken from the entrance. The results are shown below in Table T9. North bound traffic on Coalburg Road approaches the warehouse on an upgrade of approximately 5%. Using the *2001 AASHTO Policy of Geometric Design of Streets and Highways*, the minimum stopping sight distance for trucks traveling at 45MPH up a 6% grade is 331 feet. The entrance exceeds the 331 feet required by AASHTO but the proposed 2nd entrance did not. Sight distance to the north did not factor into the decision to use one entrance as the flat, open terrain did not impede the ability of motorists to see. Due to minimum sight distance and passenger safety concerns it was determined that there was not a safe location for a second warehouse entrance. A benefit of using one entrance was dramatically reducing impervious surfaces, which lowered construction costs and limited storm water runoff.

Table T9 – Sight Distance Measurements

	To the South (feet)	To the North (feet)
Main Entrance	591	>1,200

Geometric Design of Entrance

The entrance was designed to allow sufficient room for all types of vehicles to enter and exit the property safely. The warehouse entrance and exit will be separated by an eight foot wide island. The island will help ensure that vehicle movements are made to the appropriate lanes. The entry lane will be 15 feet wide, large enough for WB-62 semi-trucks. The exit lane will have two lanes, one lane being a right turn only and the other being a left turn only. Like the entrance, both exit lanes will be 15 feet wide.

Entering traffic will have unobstructed paths to their destinations in order to reduce traffic congestion near the entrance. Trucks will continue to the rear of the building for loading/unloading while passenger cars cross exiting truck traffic to gain access to the parking lot. To ensure that automobiles will not have to yield to oncoming traffic, exiting trucks will have a stop sign at the intersection. This provides a safer, more efficient entrance for passenger car traffic.

The *2001 AASHTO design manual* establishes specific turning radii for all types of vehicles. The common WB-62 (62 foot wheelbase) semi truck has a minimum turning radius of 45 feet. Assuming that most of the traffic entering the warehouse would be of similar size as a WB-62 the turning radii for the entrance and exit was designed at 75 feet. This allows for a WB-62 semi-truck to comfortably enter and exit the property and also allows a slightly larger truck, if need be, to enter and exit the warehouse.

Acceleration and Deceleration Lanes

Acceleration and deceleration lanes will be utilized for entering and exiting warehouse property. Such lanes are not required but they will be installed as a safety measure for warehouse traffic. They will provide semi-trucks more room to enter the property and gives them space to accelerate and merge into traffic upon exiting. Both lanes will be 12 feet and the *2001 AASHTO design manual* was used as a reference to determine the length. AASHTO recommends the acceleration lane be 560 feet based on an initial vehicle speed of 0 MPH and a highway speed of 45 MPH. This is conservative because the right turn exit lane is designed to have vehicles in motion prior to entering the acceleration lane. The deceleration lane is recommended to be 280 feet based on a highway speed of 45 MPH, a 14 MPH average running speed on exit turn, and a 5%-6% upgrade slope.

Due to limited Coalburg Road frontage available, approximately 600 total feet, the AASHTO suggested lengths for acceleration and deceleration lanes could not be met. However, the available Coalburg Road frontage on the job site was maximized in the design of the acceleration and deceleration lanes. The maximum designable length for the acceleration was determined to be approximately 275 feet and the deceleration was determined to be approximately 130 feet.

On-Site Driveway Design

All onsite driveways will be constructed of asphalt, which will be discussed more in detail in the geotechnical section. The cost analysis of asphalt versus concrete was conducted by our construction management team and it was determined that concrete was not economical based on the amount of impervious surface that was to be laid on property. The only exception to asphalt on the property is the concrete pad at the loading docks, as previously described.

Figure T14 on the following page shows the site layout, including all on site driveways. Outlined in red are lanes designed to be 15 feet wide. These lanes will be subjected toWB-62, or similar, semi-truck traffic. Providing wider lane widths than required by the *AASHTO* design manual provides a safer, easier to maneuver environment for the semi-truck drivers that will operate on site each day. The areas outlined in blue are lanes designed to be 12 feet wide. The 12 foot lane is the preferred width as stated in the *AASHTO* design manual. This allows most cars, even semi-trucks, sufficient room to maneuver safely.

Like the turning radii for the entrance, the turning radius near the loading dock area is 75 feet. This far exceeds the minimum of 45 feet for WB-62 trucks, allowing truck drivers plenty of room to maneuver. The 15 foot lanes and 75 foot turning radii onsite provide safe conditions for truck drivers and workers of the warehouse.

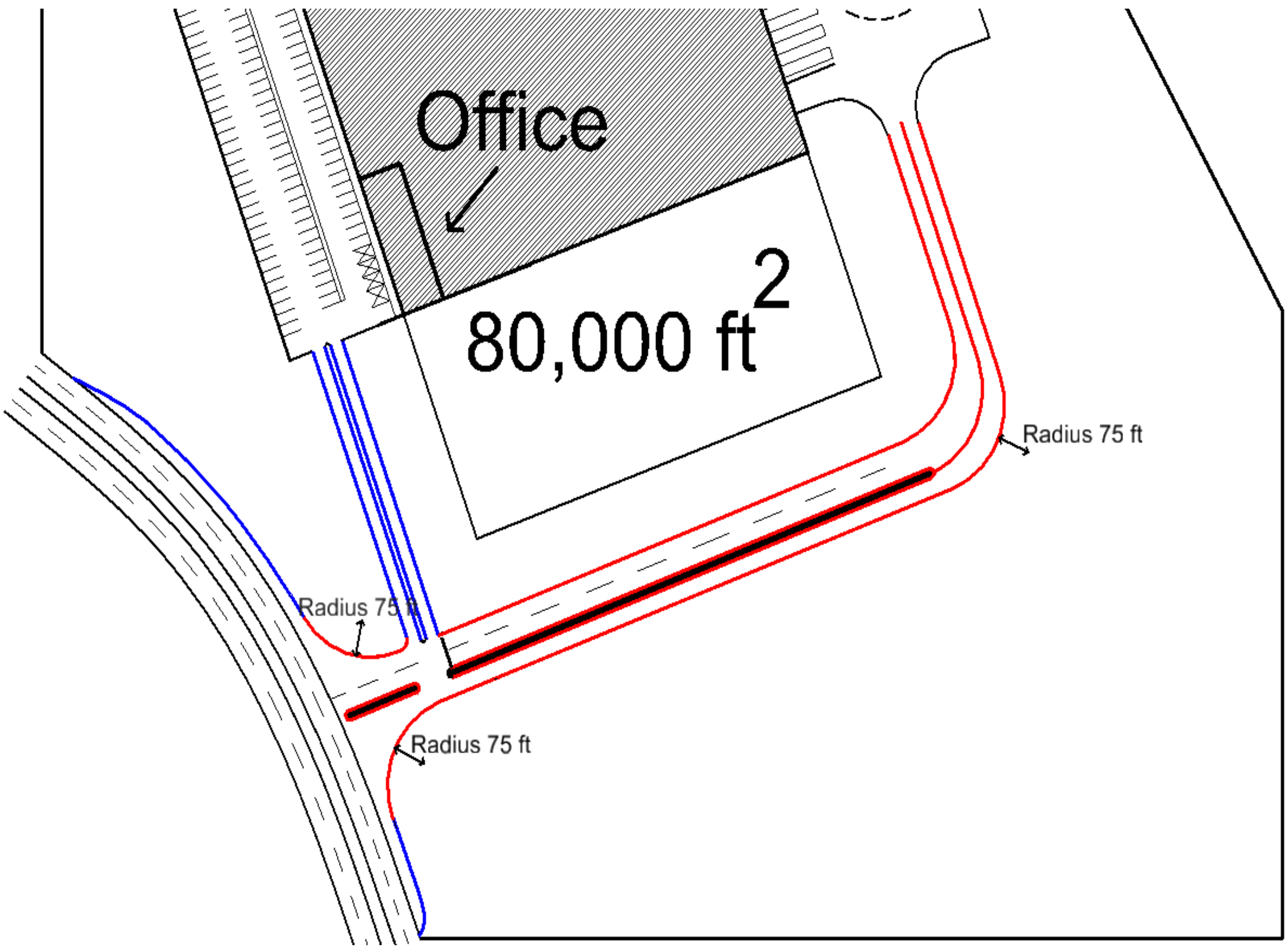

Figure T14 – On Site Driveways

Transportation Conclusion

All analysis of present and future traffic condition imposed on Coalburg Rd and the intersection of Daniel Paine Drive indicate the warehouse and office space will have a minimal impact on local traffic. With addition of acceleration and deceleration lanes to the site entrance and exit, the analysis also concludes that no new roadway improvements other than normal upkeep and maintenance are needed for Coalburg Road. All on site designs meet or exceed local, state, or federal design standards.

5. Geotechnical

Project Information

An evaluation on geotechnical properties has been prepared for th proposed warehouse building. The proposed building will consist of a 250,000 ft^2 warehouse with an additional two-story, 10,000 ft^2 of office space along with 10 truck loading bays, and approximately 100 employee parking spaces.

The following existing conditions analysis provides information about the soil, site area, soil treatment, and pavement types that will be used in the construction of this facility.

Existing Conditions Analysis

Most of the soil at the site consists of underground coal mine spoils. This is made up primarily of medium clayey and silty sands. Residual soils around the site consist of medium to stiff sandy clays and silts and very loose to medium clayey sands. The entire site measures roughly 27 acres. Currently, there is a rough, dirt road running through the proposed site, dividing it north and south. The ground elevation to the north of this road is higher than the ground elevation to the south of the road. The north side of the site contains generally good quality soil, with >75% groundcover. The remaining soil at the site is mainly poor condition (<50% groundcover) with areas of fair (50-75% groundcover) condition soil. At the northeast corner, the site slopes down to an existing detention pond. As the top of the existing slopes leading into the detention pond, deep channels have been cut into the surface soils by erosion. These deep channels range from a few inches deep to over 10 feet and act as drainage channels.

After reviewing the Report of Geotechnical Explorations provided to us by Goodwyn, Mills and Cawood, Inc., it has been determined that boring sites 1, 2, and 14, marked by the yellow area in Figure G1, contains soil adequate for supporting the loads from our proposed warehouse. Because of this, the back end of the warehouse, outlined in Figure G1 by the blue border, will be placed over these three boring sites. This minimizes the area of soil that will need treatment before compaction begins. This area is roughly 43,100 ft^2 or about 1 acre, and, being used for the back end of the warehouse, will reduce the cost of treatment by about $800,000. Situating the warehouse in this location will not only save money in the pre-construction phase, but will also place the warehouse against the back property line, marked in orange in Figure G1. This uses the existing space on the site as effectively as possible, and allows for maximum possible future expansion over the entire site area. The placement of the warehouse near the back property line also allows the design team to integrate an 80,000 ft^2 area to the south of the proposed warehouse, which can be used for future expansion of the warehouse without altering any infrastructure.

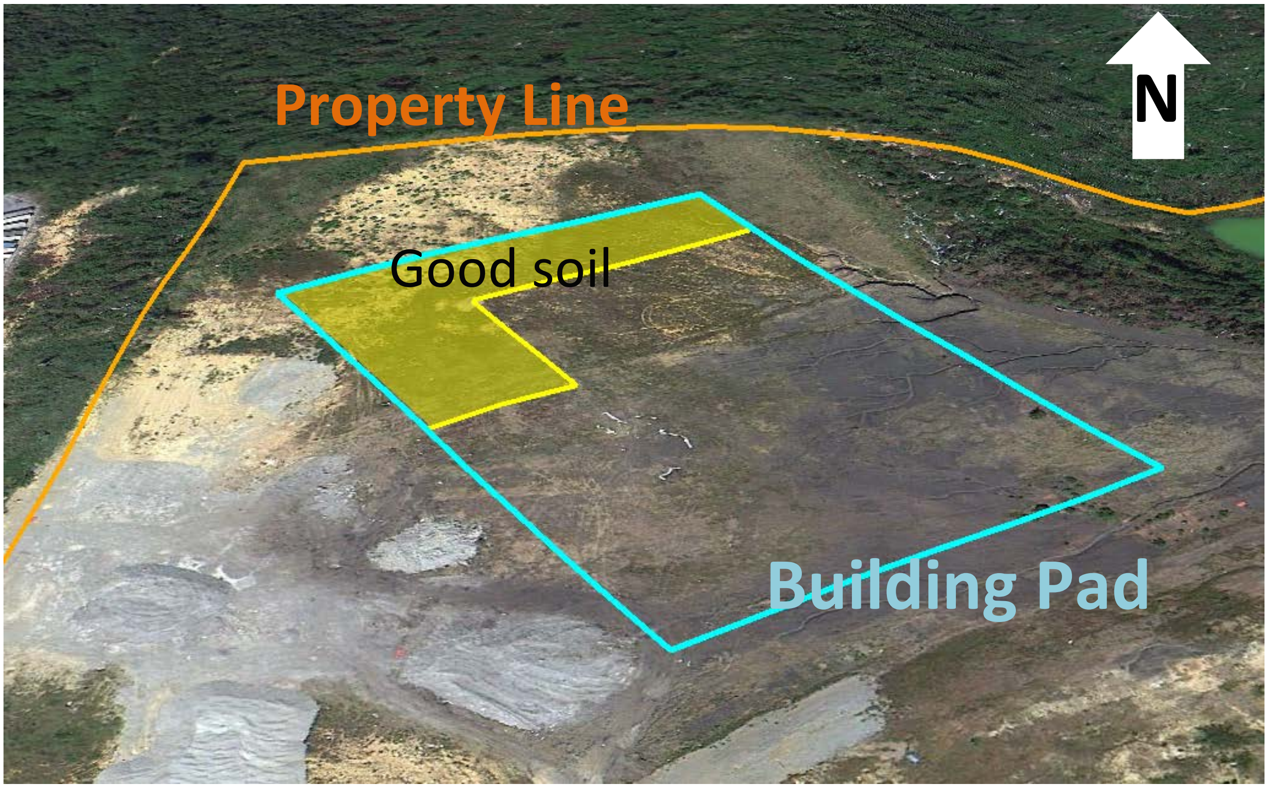

Figure G1: Good Soil Locations

Soil Treatment

The remaining soil at the site will be treated by the use of deep dynamic compaction. Deep dynamic compaction is a method that is used to compact and increase the density of the soil at a construction site prior to construction. This process generally involves the dropping of a heavy weight repeatedly on the ground at regularly spaced intervals. The impact of this weight causes stress waves that aid in the densification of the soil. This was chosen as the most cost effective approach to treating the soil prior to construction beginning. The alternative, cut and fill at a depth of 15 ft, proved to be more expensive and would require a longer time to complete.

Before the dynamic compaction can be started, the site will need to be cut to the proposed subgrade level of 2 ft below the building elevation level of 676.38 ft. The amount of cut required for the building pad and parking area was determined by first deciding on a building elevation of 676.38 ft. This elevation was chosen due to the fact that it allows for all of the cut material to be used as fill material and ultimately remove the need for additional fill material to be brought in from off-site sources. Once this process has been completed, the soil at the site will be ready for deep dynamic compaction.

After the soil has been treated by dynamic compaction, it will be surcharged. This will be done across the building area and a 5 foot extension around the proposed building perimeter with 4 feet of fill. The surcharging will be done for a period of 2-4 months. As stated before, the surcharging and treatment will not be needed in the soil areas marked by borings 1, 2, and 14. For the parking and drive areas, a minimum 2 feet of dense fill material will be placed beneath the proposed subgrade level. All fill material will be required to meet the specifications outlined in Table 1.

Table G1: Specifications for Site Soil

Liquid Limit	**<50%**
Plasticity Index	<25%
Maximum Dry Density (ASTM D-698)	≥98 pcf
Maximum Particle Size	<3 inches

Once the building site has been prepared by dynamic compaction and surcharging, the soil at the site will be suitable for shallow foundations. Foundations have been sized by using a net allowable soil bearing pressure of 3500 psf. According to the Report of Geotechnical Exploration, the total settlements of the foundations due to building loads from the proposed warehouse are expected to be less than 1 inch.

Also, in accordance with the geotechnical report, exterior foundations are to bear a minimum of 18 inches below the final adjacent exterior grade and interior footings will be placed at a nominal depth of 12 inches. The floor slabs of the warehouse will be built on grade, achieving support from proof rolled residual material.

Pavement Areas

For pavement areas subjected to only passenger car traffic, area 1 in Figure G2, standard duty pavement will be used. For areas subjected to both passenger car and heavy truck traffic, area 2 in Figure G2, heavy duty pavement is recommended. For the minimum pavement sections for standard and heavy duty pavement, see Table G2 below.

Table G2: Specifications for Pavement Types

Pavement Type	Thickness	Material
Standard Duty Pavement	1.0 inch	Asphaltic Concrete Surface Course
	3.0 inches	Asphaltic Concrete Binder Course
	6.0 inches	Dense Grade Aggregate Base Stone
Heavy Duty Pavement	2.0 inches	Asphaltic Concrete Surface Course
	4.0 inches	Asphaltic Concrete Binder Course
	10.0 inches	Dense Grade Aggregate Base Stone

For pavement areas subjected to only heavy duty truck traffic, area 3 in Figure G2, rigid pavement will be utilized. This will involve a minimum thickness of 8 in. of concrete with an 8 inch layer of dense grade aggregate placed between the rigid pavement and the proposed subgrade. The concrete used in the rigid pavement will be required to have a minimum 28-day flexural strength of 550 psi, and a minimum 28-day compressive strength of 4000 psi.

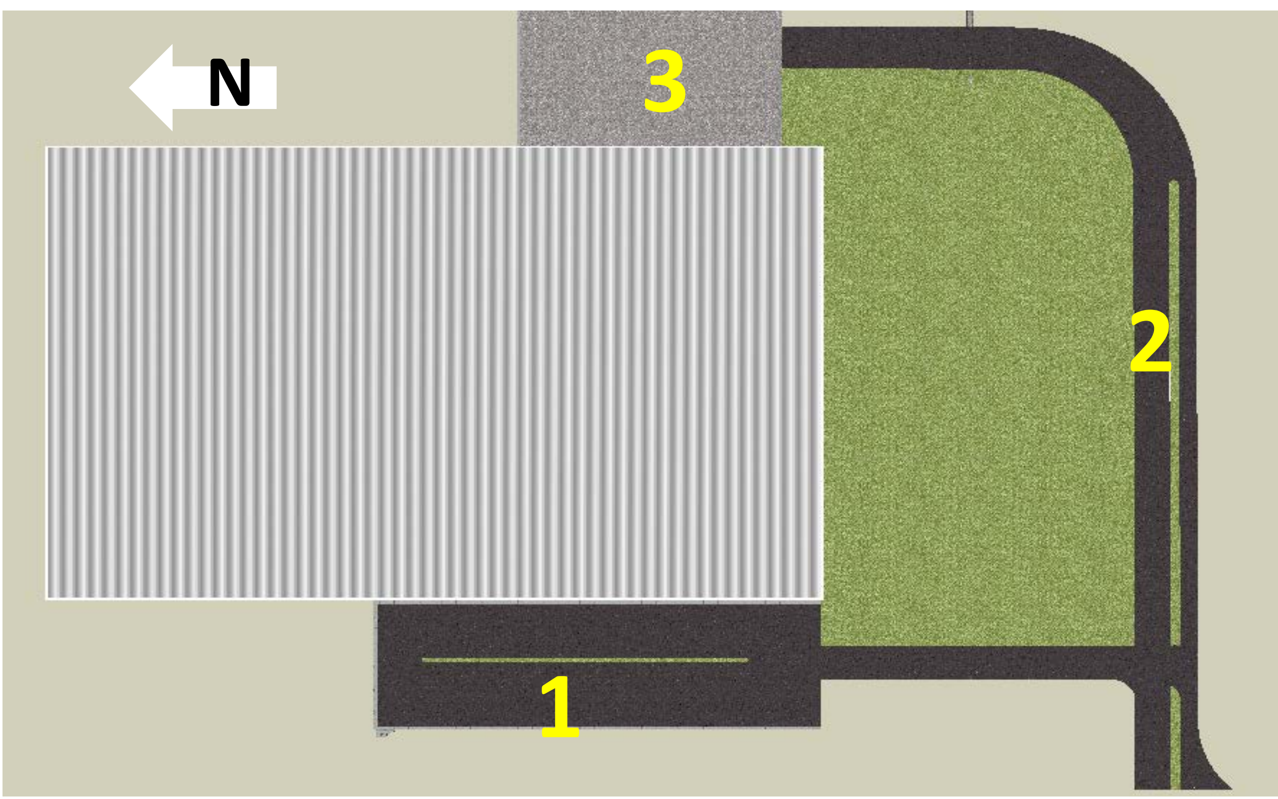

Figure G2: Pavement areas

The parking and drive areas will also require at least 2 ft of compacted fill material or residual dense sands beneath the proposed subgrade level.

Interior Footing Design

Single-column footings have been chosen for the design for the footings supporting the columns for the warehouse structure. Single-column footings isolate the loads from the columns from the warehouse floor slab. In designing the footings, two shear conditions were considered in the column footings: one-way shear and two-way or punching shear. The bending moment of a square footing was also considered with the maximum bending moment occurring at the face of the column. To reduce the risk of the floor slab cracking along the edges of the column footings, a four inch layer of soil will be placed between the top of the column footing and the base of the warehouse floor slab.

The compressive strength of the concrete is to be 3000 psi with Grade 60 reinforcing bars. According to the geotechnical report, the column footings must be a minimum of 18 in. deep. The ACI 318 code states the depth of a footing above the bottom reinforcing bars may be no less than 6 in. and the clear cover must be at least 3in. The column footing was set to be 18 in. thick with a reinforcing depth of 14.5 in. The effective soil pressure was calculated by subtracting the weight of the concrete slab above the footing from the allowable soil pressure of 3500 psi. The effective soil pressure was determined to be 3,137.5 psf. The area of the footing required was calculated by dividing the unfactored combined load of 56.6 kips by the effective soil pressure of 3.1375 ksf. An area of 18.17 ft^2 was determined to be required. In order to make the footing square for simplified forming on the jobsite and more cost-effective, a 5 ft X 5 ft footing was chosen. The bearing pressure of the footing for design strength was determined by dividing the factored dead and live loads by the area of the footing. The bearing pressure for design was calculated to be 3.5 ksf, which will be used to find the ultimate moment on the footing.

As previously mentioned, two shear conditions were considered and checked in the design of the footings. The first shear condition checked was two-way or punching shear. The two-way shear force was determined by subtracting the length of the perimeter of the footing squared from the area of the footing and multiplying by the bearing pressure. The two-way shear force was determined to be 76.19 kips. This shear force was then used to calculate the maximum depth required by the following two equations taking the larger of the two that controls:

$$d = \frac{V_{u2}}{\varphi 4\sqrt{f'_c}b_o}$$

$$\mathbf{d} = \frac{\mathbf{V_{u2}}}{\boldsymbol{\varphi}\left(\frac{\mathbf{40d}}{\mathbf{b_o}} + 2\right)\sqrt{\mathbf{f'_c}}\mathbf{b_o}}$$

By calculating the two equations, the larger maximum depth required was determined to be 5.4”. The column footing depth set is at 14.5 in. which is greater than the 5.4 in. determined for two-way shear, so the design is satisfactory.

The second shear condition checked was for one-way shear. The one-way shear force was determined by multiplying the bearing pressure times the width of the footing times the length between the edge of the footing and the face of the column. The one-way shear force was determined to be 17.5 kips. The shear force was then used to calculate the maximum depth required by the following equation:

$$d = \frac{V_{u1}}{\varphi 2\sqrt{f'_c}b_w}$$

The maximum depth required for one-way shear was calculated to be 3.55 in. This is less than the depth of 14.5 in., so the design is satisfactory for one-way shear.

The maximum bending moment in a square column footing is the same about both axes due to symmetry and occurs at the face of the column. The maximum bending moment is determined in order to determine the area of steel required for reinforcement in each direction. The maximum bending moment was determined to be 42.6 ft-k. This bending moment yields a much lower ρ value than the minimum values stated in ACI 318. Due to the high shear and low ρ values, ACI 318 recommends the steel areas be as large as the flexural minimums of ACI Section 10.5.1. The area of steel was then calculated to be 2.9 in^2 by multiplying the minimum ρ value, footing width, and the depth of reinforcing steel. The development length was also calculated to be 24 in. from center of footing on each side, which leaves more than a 3” clear cover between the end of the reinforcing bars and the edge of the footing. The final column footing design is shown in figure G3.

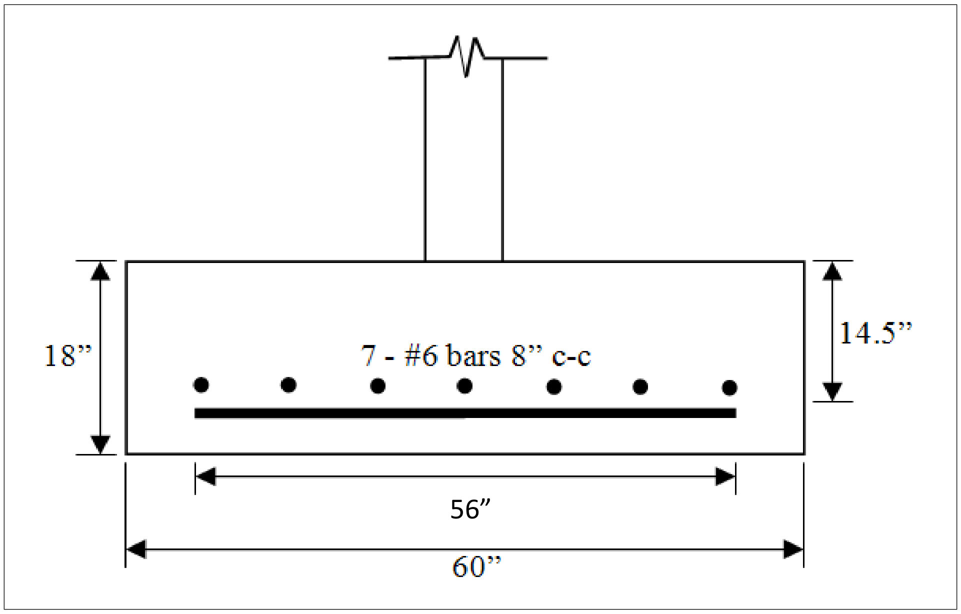

Figure G2: Column Footing Design

Column Base Plate and Anchor Rod Design

The design of the connection of column to base plate to footing was designed to resist the uplift from the MWFRS wind loads. The standard specification for anchor rods is ASTM F1557, and Grade 36 rods were chosen, which is the most common. The regulations of the Occupational Safety and Health Administration (OSHA) require a minimum of four anchor rods in column-base-plate connections. The anchor rods must resist the tensile force due to uplift and the must resist pullout from the concrete.

The uplift on the columns determined from the MWFRS is 40.32 k on each column and the dead load determined from the weight of material on each column is 9.6 k. The required strength due to uplift is 55.87 k determined from the following equation:

$$\text{Uplift } (T_u) = -0.9P_{DL} + 1.6P_{uplift}$$

The required strength per rod was determined to be 14.0 k by dividing the required strength by the minimum number of four rods required by OSHA regulations. It was then determined by the standard specification ASTM F1557 that ¾” Grade 36 bolts to be used. The nominal tensile strength for each rod was calculated by the following equation:

$$\phi R_n = 0.75F_uA_b \qquad \text{(AISC Equation J3-1)}$$

The nominal tensile strength for each rod was determined to be 19.22 k, which is greater than 14.0 k per rod, so the rods are satisfactory.

The anchor rods are to be positioned in the corners of the base plate a vertical and horizontal distance of 2". from the edge of the base plate. The base plate is to be square with the dimensions of 14" by 14". The required thickness of the base plate was determined using the following equation from AISC:

$$t_{min} = l\sqrt{\frac{2P_u}{0.9F_yBN}}$$

It was determined that the thickness required for the base plate is to be ½". The designation for the base plate is to be PL ½ X 14 X 1 ft. 2 in. of A36 steel. The AISC Design Guide 1, Table 2.3 recommends the anchor rod holes be 1-5/16". A detailed drawing of the base plate is illustrated in Figure G4.

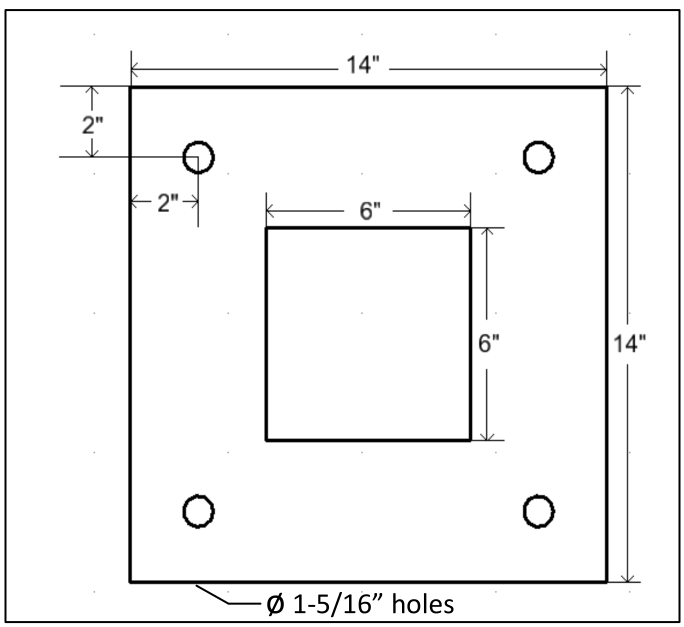

Fig. G3: Base Plate Design

The base plate will be shop welded with E70 electrodes to the bottom of the column for ease of erection in the field. The maximum load experienced by the weld was determined by finding the required tensile strength per rod and dividing it by the effective width required for resisting the moment at the face of the column. The maximum weld load was determined to be 3.62 k/in. The minimum weld size for a column wall thickness of 0.349" is 3/16", per Table J2.4 of AISC Specification. The design strength of the weld was determined by the following equation:

$\phi R_n = \phi F_w A_w$ (AISC Equation J2-4)

$\phi = 0.75$

$F_w = 0.6F_{EXX}\,(1.0+0.5\sin^{1.5}\theta)$

A_w= effective are of the weld, in^2

The design strength of the weld was determined to be 6.26 k/in., which exceeds the maximum weld load of 3.62 k/in., so the weld design is satisfactory.

The stress on the column wall was check for yielding from the forces exerted on it by the welds. The maximum weld load was divided by the wall thickness to yield a wall stress of 12.45 ksi. The maximum design stress on the wall was determined by multiplying 0.9 with the yield stress of the column. The maximum design stress was calculated to be 45 ksi, which is much greater than the stress from the weld, so the design is satisfactory.

The final design check for the base plate and anchor design was for the concrete breakout strength. The needed depth of the anchor (h_{ef}) was determined to be 8" from figure 9.1.6 of AISC Design Guide 7 for the common case of four anchor rods pre-casted in a footing, where a full breakout cone is achieved with ¾" anchor rods. The concrete breakout strength was determined from the following equation:

$$\phi N_{cbg} = \phi \psi_3 24\sqrt{f'_c}\, h_{ef}^{1.5} \frac{A_N}{A_{No}}$$ (ACI 318-05 Appendix D)

$\phi = 0.70$

$\psi_3 = 1.25$; concrete uncracked

A_N= concrete breakout cone area for group

A_{No}=concrete breakout cone area for single anchor

The concrete breakout strength was calculated to be 147.7 k, which is greater than the required 40.32 k for uplift. The design for four, ¾", Grade 36 bolts embedded 8" into the footing is adequate to achieve the anchor strength considering the full breakout strength. The column base and footing detail is illustrated in Figure G5 on the next page.

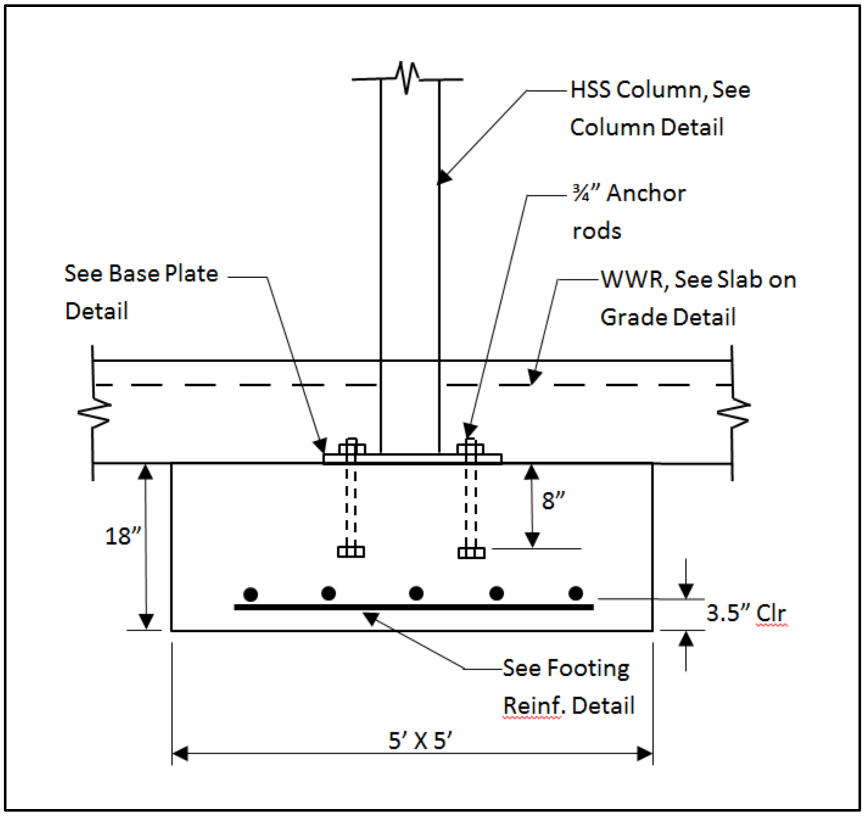

Fig. G4: Column and Footing Detail

Exterior Footing Design

The exterior footings for the proposed warehouse were designed to run the entire length of the warehouse walls. Because of the design of the warehouse, there will be a column footing, as described in the previous section, at every column throughout the exterior wall of the warehouse. The dead load for the wall was calculated to be 2.19 k/lf. Because of this, the minimum depth of the footing was calculated out to be 8 inches. The International Building Code (IBC) manual was used to determine the minimum width of an exterior footing for a three-story structure; this value was found to be at least 18 inches. This footing is defined by ACI as a "deep beam" according to section 10.7.1. According to this section, deep beam footings must be designed as follows:

A. Loaded on one face and supported on the opposite face so that compression struts can develop between the between the loads and the supports and
B. Having either
    a. Clear spans, I_n equal to or less than four times the overall member height, h
    b. Regions loaded with the concentrated loads within 2d from the face of the support.

The ACI code does not quantify the magnitude of the concentrated load needed for the beam to act as a deep beam. However, ACI committee 445 has suggested that a concentrated load that causes 30% or more of the reaction at the support of the beam in question would qualify.

ACI code section 11.7, Deep Beams, gives the same requirements as listed above. Both sections require that deep beams be designed via nonlinear analysis or by strut and tie model. The design equations for V_s given in previous ACI codes are no longer used because they did not have a clearly defined load path and had serious discontinuities as the span-to-depth ratio was varied.

The exterior footings were designed to be 8 inches deep and 18 inches wide. Flexural steel was calculated at .0453 in^2. Because of this, to support the footings for flexure, #4 bars at 18 in. spacing was chosen. Having this as the reinforcement gives 0.133 in^2/ft, exceeding the standard set forth in ACI section 10.6.4. For reinforcement for shrinkage and temperature control, minimum steel area was calculated to be 0.2592 in^2/ft. Because of this, #4 bars at 9 in. spacing were chosen. For a detailed view of the exterior footing, see Figure G5.

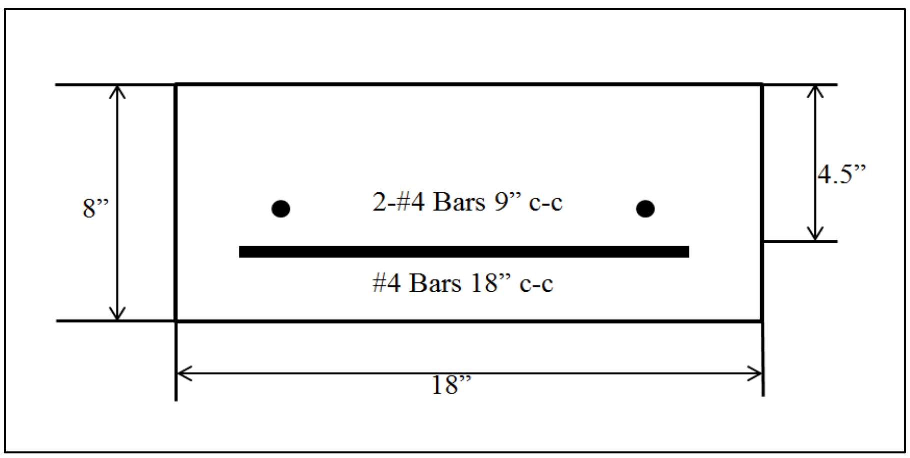

Fig. G5: Exterior Footing Design

6. Structural

Introduction

Two considerations of utmost importance to the structural design of the warehouse on Coalburg Road are public and private safety, and cost-effectiveness. In order to meet expectations of structural integrity and cost-effectiveness, the structural team approached this project by breaking down the design process into the following components: preliminary steel selection and cost analysis, determination of design loads, frame design and member selection, and foundation design. The warehouse will consist of a steel braced frame with a flat roof and concrete tilt-up panels for the sidewalls. The use of a braced frame as opposed to a moment-resisting rigid frame was chosen for several reasons. First of all, bracing is more effective than rigid joints in resisting racking deformation of the frame, and need simpler connections that moment-resisting frames. Braced frames also use less material and therefore add less weight and cost to the overall structure.

Cost Analysis

One of the first steps in designing the warehouse was to select the most cost effective design. The team selected steel members based on theoretical loads that would act on the structure. For a cost-effective design, HSS Columns, K-Series Open Web Steel Joists, and Joist Girders were used in the preliminary analysis. Load and Resistance Factor Design (LRFD) was used to select members based on their required strength. An unfactored dead load of 10 psf and unfactored roof live load of 20 psf were plugged into LRFD factored combinations to determine required strength of members. The steel roof decking was designed in accordance with Steel Deck Institute guidelines. The steel joists and joist-girders were designed using Steel Joist Institute's Specifications for Steel Joists and Joist Girders. The columns were selected from the AISC Steel Construction Manual, 14th Edition (SJI 2010). Steel cables used for lateral bracing were designed capable of carrying LRFD required tension with a factor of safety of 2.

The structural team analyzed various combinations of column spacing and corresponding joist spacing using the previous mentioned references to select members and determine their weights. A joist spacing of 10 ft. o.c. was found to be more efficient than other joist spacings in terms of material cost and design. The cost of steel is directly proportional to its weight, so an analysis of total weight versus column spacing provided an accurate cost analysis. For each combination, the team selected the lightest members with sufficient load carrying capacity. Results of the cost analysis are presented in Figure S1. The most cost effective bay size of 40 ft. by 40 ft. was selected and used to achieve an overall building footprint of 640 ft. by 400 ft. This footprint was selected based on geotechnical and transportation related needs and allowed for a uniform spacing of structural members throughout the frame.

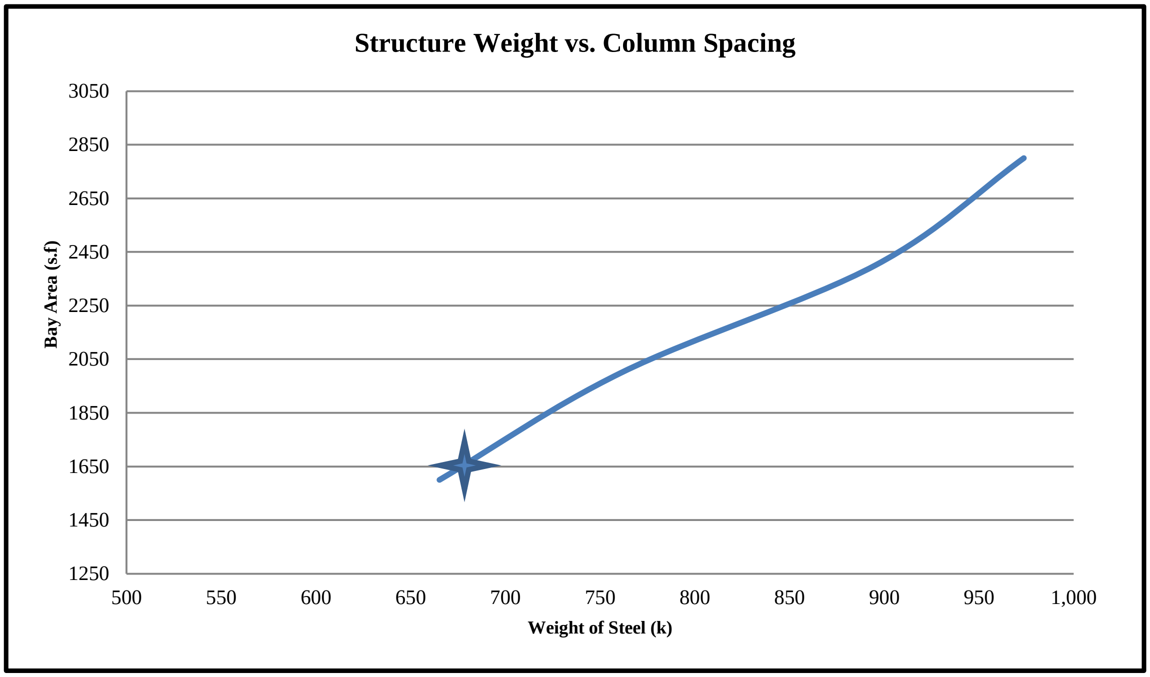

Figure S1: Cost Analysis

Design Loads

The warehouse building designs are based on the 2009 International Building Code (IBC) and the American Society of Civil Engineers (ASCE) 7-10. The minimum design loads for occupancy in Table S1 were used in the design of the warehouse. All loads are uniformly distributed except when it is required that a concentrated load be compared to the distributed load. The worst-case scenario was used for each occupancy category. Other loads on the warehouse floor slab were considered because of expected storage. A lift truck axle load was determined from a typical lift truck with a capacity of 20 kips and a vehicle weight of 22 kips. The rack storage load was also taken into consideration from a typical storage rack 9 ft long by 4 ft wide and five shelves at 100 psf per shelf.

Table S1: Minimum design loads

Occupancy	Uniform (psf)	Concentrated (lb)
Roof Live Load	20	2000
Floor Live Load		
Heavy storage warehouse	250	
Office use	50	2000
Corridors, first level	100	2000
Corridors, above first floor	80	2000
Stairs and exit ways	100	300

The wind loads for the Main Force Resisting System (MWFRS) were determined using the simplified envelope procedure according to ASCE 7-10, Section 28.6. The team used this method because the warehouse fit the criteria of being an enclosed simple, low-rise building with a height less than 60 ft. The warehouse could experience the wind pressures from any side of the building. The exposure factor is a category B with a basic wind speed of 115 mph. The internal pressure coefficient is ± 0.18. The values have been adjusted for building height and exposure category according to ASCE 7-10, Figure 28.1-6. The width of edge strip is 35 ft. Table S2 presents the net wind pressures at each zone according to ACSE 7-10, Section 28.5. The maximum net wind pressure for the roof is an uplift of 25.2 psf. The maximum net wind pressure on the walls is 21 psf. The wind loads for components and cladding (C&C) were determined by the simplified procedure according to ASCE 7-10, Section 30.5. The wind pressures are shown in Table S3. A parapet will be used which means the corner zones will be the same as the edge zones. The values have been adjusted for building height and exposure category according to ASCE 7-10, Figure 30.5-1, which is presented in Figure S3. The width of edge strip (a) is 16 ft. The effective wind area used in determining wall pressures is 100 s.f. for both roofing and wall panels. The plus and minus signs signify pressures acting toward and away from the building surfaces, respectively.

Table S2: MWFRS wind loads (Fig.28.1-6, ASCE 7-10)

Zone	Wind Pressure (psf)
A	21
B	8
C	16
D	8
E	-25.2
F	-14.3
G	-17.5
H	-11.1

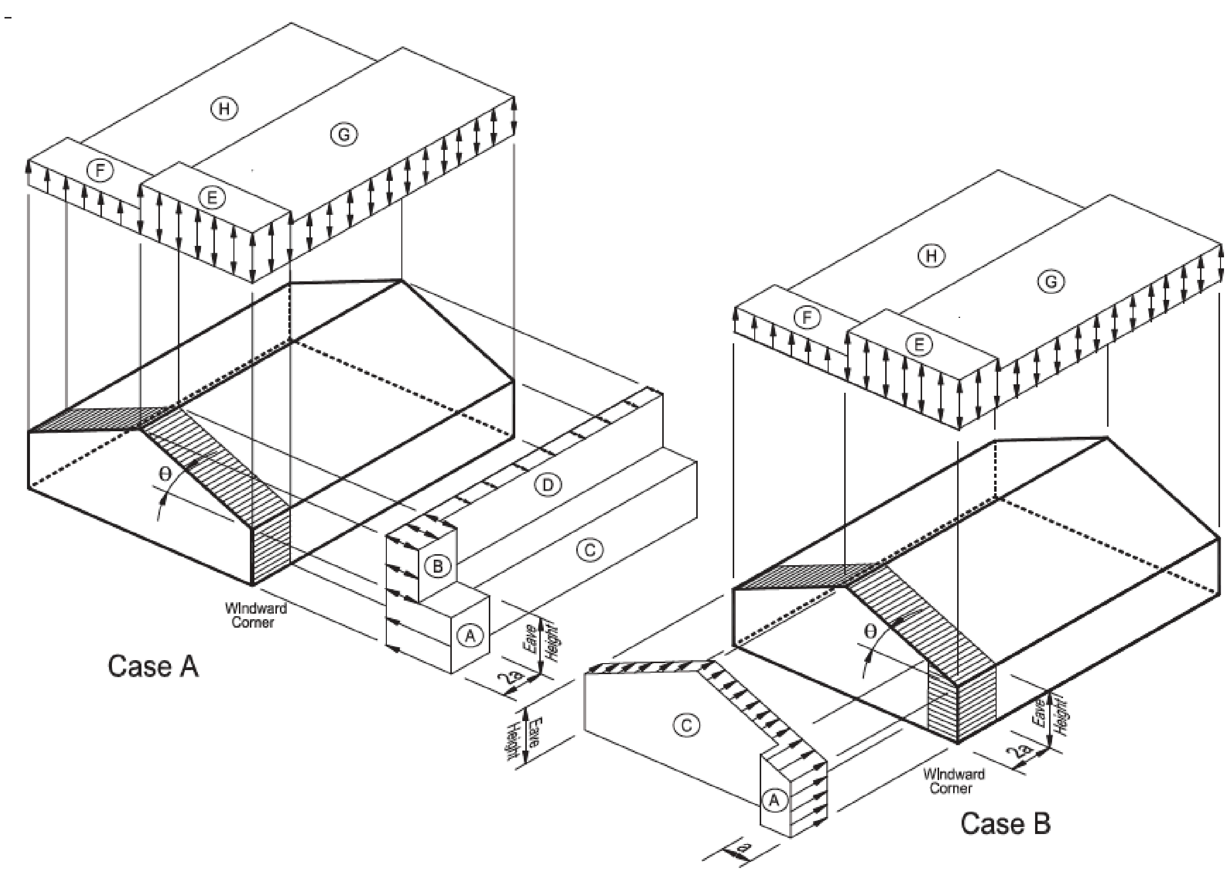

Figure S2: MWFRS – Method 2 (ASCE 7-10, Fig. 28.6-1)

Table S3: Components &Cladding wind loads

Zone	Effective Area (s.f.)	Net Pressures	
1	100	7.7	-21.8
2	100	7.7	-25.8
3	100	7.7	-25.8
4	100	20.2	-22.2
5	100	20.2	-24.7

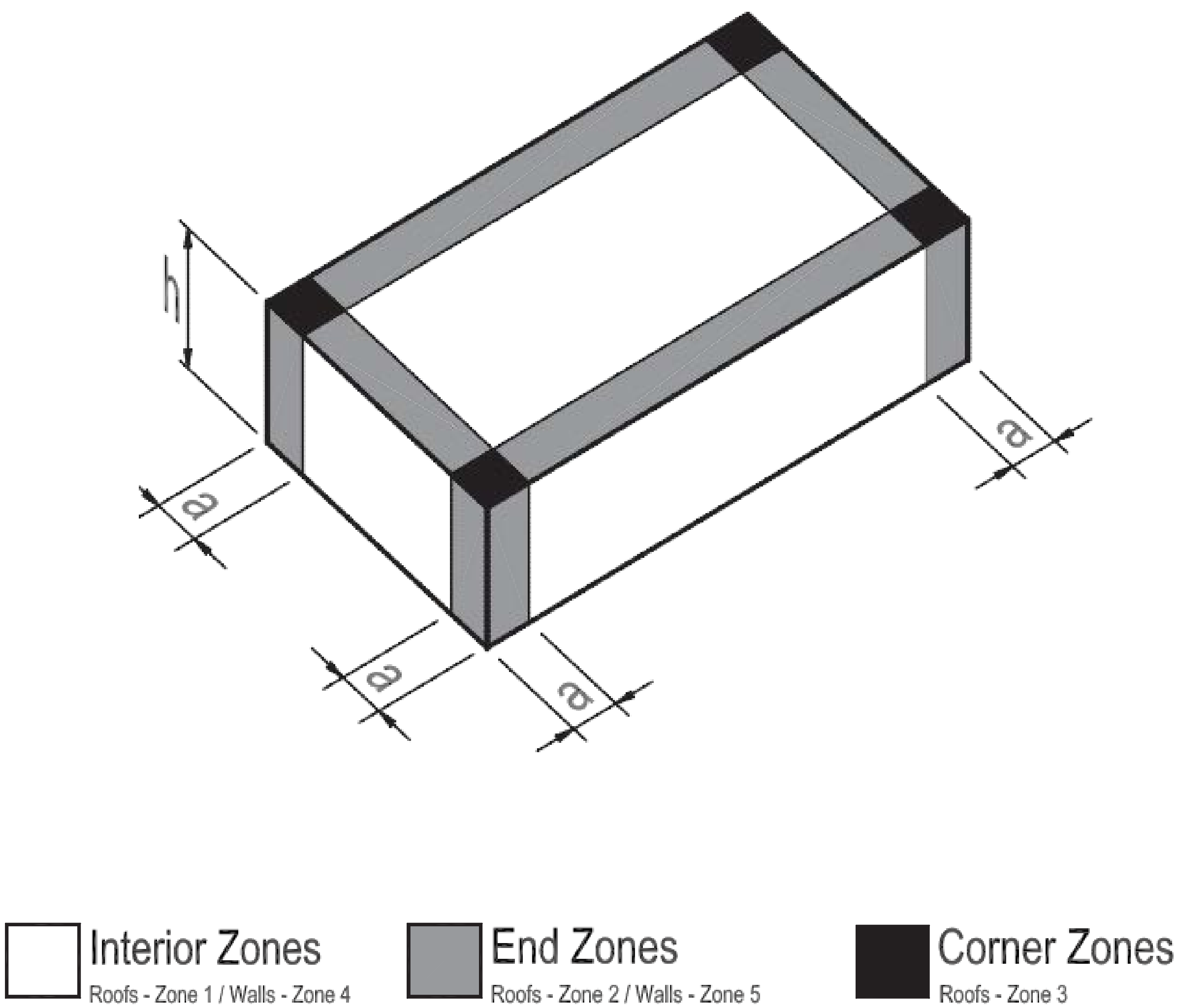

Figure S3: Components and Cladding Method 1 (ASCE 7-10, Figure 30.5-1)

The seismic design loads were also calculated and compared to the design wind loads to verify which design load would control for bracing the structure. The site class for the building footprint was determined by Table 20.3-1 in ASCE 7-10 to be in Site Class C, because the average field standard penetration resistance or the blow count per foot, for the top 100 ft. of soil is greater than 50 blows/ft. The mapped Risk-Targeted Maximum Considered Earthquake (MCE_R) spectral response acceleration parameter at short periods (S_S) was determined to be 0.30 and the mapped MCE_R spectral response acceleration parameter at a period of 1 second was determined to be 0.10 by Figures 22-1 and 22-2 of ASCE 7-10, respectively. The short period site coefficient (F_a) was determined to be 1.2 and the long-period site coefficient (F_v) was determined to be 1.7. The MCE_R spectral response acceleration parameters for short periods (S_{MS}) and at one second (S_{M1}) were calculated by the following equations:

$S_{MS}= F_aS_S$ (ASCE 7-10 Equation 11.4-1)

$S_{M1}= F_vS_1$ (ASCE 7-10 Equation 11.4-2)

The value for S_{MS} was determined to be 0.36 and the value for S_{M1} was determined to be 0.17. The design spectral acceleration parameter at short period, S_{DS}, and at one second period, S_{D1}, was determined by the following:

$S_{DS}= 2/3S_{MS}$ (ASCE 7-10 Equation 11.4-3)

$S_{D1}= 2/3S_{M1}$ (ASCE 7-10 Equation 11.4-4)

The values for S_{DS} and S_{D1} were calculated to 0.24 and 0.11, respectively. According to Tables 11.6-1 and 11.6-2, values for S_{DS} and S_{D1} assign the building in design category B.

The equivalent lateral force procedure was used to determine the seismic forces to be used in this design consideration. The design seismic force is permitted to be applied independently in each of two orthogonal directions as a uniform distributed load. The seismic base shear, V, was determined from the following equation:

$V= C_sW$ (ASCE 7-10 Equation 12.8-1)

C_s= seismic response coefficient

W= the effective seismic weight

The seismic response coefficient was calculated by the following equation:

$C_s= S_{DS}/(R/I_e)$

The seismic force-resisting system for the building frame system is an ordinary steel concentrically braced frame. The response modification coefficient, R, is 3.25, found from ASCE 7-10 Table 12.2-1. The seismic response coefficient was calculated to be 0.0738. The effective seismic weight of the building was determined to be 3,597,947 lbs. The base shear force was found to be 265,529 lbs. After converting this force into a uniform distributed load, the seismic design loads along the long span and short span of the building were determined to be 415 plf and 664 plf, respectively. The wind design load was determined to be 567 plf in the same direction, which means the seismic load will control in the design of the structure.

Roof Drainage

Design of the roof drainage system was considered in order to determine the magnitudes of loads applied to the steel structure. The warehouse roof is being designed as a flat roof with gypsum board fire retardant, EPS foam insulation, and EPDM membrane on top of steel decking. Gypsum board of ½ in. thickness, which, according to ASCE 7-10 Table C3-1, weighs 2 psf per 1-in. thickness will be used, giving it a dead load of 1 psf. Our design accounts for 6-in. of expanded polystyrene foam insulation. According to ASCE 7-10 Table C3-1, the weight of roofing insulation will equal 0.2 psf per inch height of insulation, giving the insulation a dead load of 1.2 psf. The roof drainage system is designed per ASCE 7-10 Section C8 to discharge rain loads from a one hour, 100 year storm (9,984 gals/min) as to neglect a ponding effect on the roof (Technical Paper 40). Twenty-eight 8-inch drains were selected for primary drainage, accompanied by twenty-eight 8-inch secondary drains set at a height of 1 in. Each pair of drains will accommodate a drainage area of 6,400 s.f., and scuppers located on the roof perimeter will drain the remaining 76,800 s.f. of roof area to down spouts located on the exterior walls of the structure. The drainage pattern of the roof can be seen in Figure S4. Decking slopes of ¼ in. length per 12 in. height will be achieved through the tapering of columns at drain inlets. The discharge per drain was calculated as follows:

$Q = 0.0104Ai$ (ASCE 7-10 Equation C8-1)

i = 100-year, 1-hour rainfall (Technical Paper 40)

A = drainage area, ft^2

Design rain load was determined as follows:

$R = 5.2(d_s + d_h)$

d_h = hydraulic head, in (ASCE 7-10, Table C8-1)

d_s = static head, in

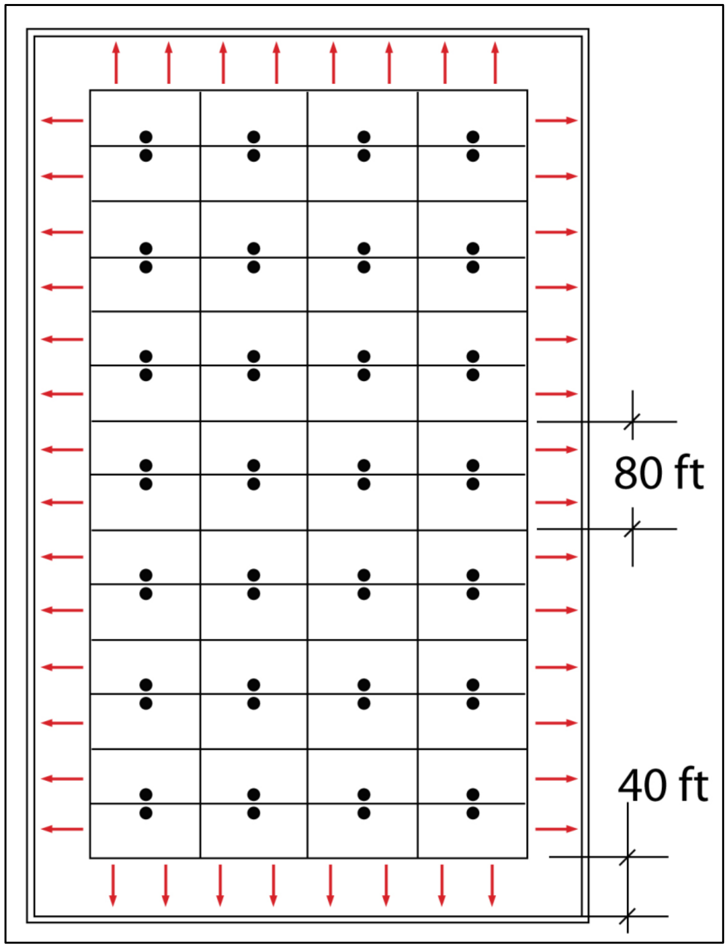

Figure S4: Roof Drainage

Roof Diaphragm

Using LRFD provisions, a uniform factored vertical distributed load of about 49 psf will act on the steel decking. Since the design of the steel structure uses a laterally braced frame, the roofing must be braced in order to resist loading from and transfer loading to the X-bracing of the structure. The steel decking was selected as the most cost effective method of horizontal load transfer through the roof diaphragm (AISC Design Guide 7). Referencing the Vulcraft steel decking catalog, a 3 in., 22 gauge, Type N steel decking with a self-weight of 2.26 psf was selected to resist the maximum vertical loading condition, as well as the maximum diaphragm shear caused by seismic forces. Sheets of 30 ft x 2 ft were selected for shipment. Decking will be connected to supports using 5/8 in. puddle welds and side laps will be connected using #10 threaded fasteners. A 24/4 connecting pattern will be used, with a connection at each trough of the decking (Vulcraft, 2008). For three 10-ft spans, the load causing the maximum deflection of L/240 is 95 psf, exceeding the maximum unfactored load of 35 psf that the decking will be subjected to. Their maximum carrying capacity is 69 psf, which also exceeds the uniform factored distributed load of 45 psf that will act on the decking. The uniform factored load was found using the controlling LRFD loading combination:

w = 1.2D +1.6(Lr or S or R) +.5(L or W) (AISC Equation 2-3c)

D = Dead Load

Lr = Roof Live Load

S = Snow Load

R = Rain Load exclusive of ponding contribution

L = Occupancy Live Load

W = Wind Load

Shear caused by wind loading and seismic activity will be distributed to the braced walls normal to the direction of force. Performing a structural analysis of the roof diaphragm under the influence of seismic forces, it was determined that the maximum shear acting upon the diaphragm will be half of the design seismic load. The shear capacity of selected 3N-22 decking and connections is 351 plf, exceeding the maximum LRFD design shear of 332 plf (Vulcraft 2008). The controlling LRFD load combination for shear was found using the following LRFD equation (AISC 2011):

w = 0.9D + 1.0E (AISC Equation 2-3g)

D = Dead Load

E = Earthquake Load

With 3N-22 decking, deflection of the diaphragm parallel to the short span of the warehouse was found to be 2.06 in. with the original plan of lateral bracing located only on exterior walls. The maximum allowable deflection per SDI specifications is 1 in. (Vulcraft 2008). In order to meet design requirements, it was decided that one more line of bracing would be located mid span of the 640 ft. wall of the structure, splitting the shear diaphragm into two 320 ft x 400 ft sections. Using this design, the diaphragm deflection was found to be 0.52 in., meeting allowable deflection requirements. The shift of shear from the exterior lines of bracing to the interior line of bracing will allow for less bracing on the exterior walls of the structure that are 400 ft in length.

Steel Joists

The steel joists will be spaced at 10 ft o.c. and subjected to a maximum factored load 484 plf. Using the Steel Joist Institute's Load Tables, a conservative joist designation of 28K8 with top chord width of 5 in. was selected, having a maximum factored carrying capacity of 492 plf (SJI 2010). The unfactored uniform load causing a maximum deflection of L/240 for this joist designation is 333 plf, which exceeds the actual unfactored uniform load that should act on these joists, equal to 320 plf. The unfactored uniform load causing a deflection of L/240 was found as suggested by paragraph 3 on page 51 of SJI 43rd Edition Standard Specifications. The uniform factored load was found using the controlling LRFD loading combination in AISC Equation 2-3c. Bottom chords of joists will be braced with three rows of continuous horizontal bridging to resist uplift forces acting on the roof decking, designed per Table 5.4-1 of SJI Specifications. The bridging is designed to be 2x2x1/8 equal leg angles as required by Table 2.6-1a. Joists will be designed with a 5/8" camber per SJI Table 103.6-1 (SJI 2010). Joists will be connected to top chords of joist girders with bearing lengths of 3 in. using two 1/8 in. fillet welds, 1 in. long. They will also be connected using two 3/8 in. ASTM A307 bolts in 9/16 in. standard holes (SJI 2010).

Steel Joist Girders

The steel joist girders will be designed and manufactured to handle point loads distributed from each joist at the top chords. At a joist spacing of 10 ft, the point loads should be 20 kips each. The factored panel point loads were found using the controlling LRFD loading combination in AISC Equation 2-3c. Joist girders with top chord widths of 9 1/8 in., depths of 40 in., capable of carrying point loads of 20 kips and resisting uplift forces from joists of 10.1 kips, were selected with a joist girder designation of 40G4N20K10.1 (SJI 2010). Bottom chords of joist girders will be braced with one row of continuous bottom chord bridging consisting of 2 x 2 x 1/8 in. equal leg angles to resist uplift. According to SJI, joist girders are allowed a maximum unfactored live load deflection of L/240, or, for a 40 ft span, a deflection of 2 in. These joist girders should experience a max deflection of 0.8 in. Deflection was calculated using the following equation per SJI Specifications for Steel Joists and Joist Girders.

$$\Delta = \frac{5wl^4}{384EI}$$

w = unfactored uniform live load, lb/in

l = span, in

E = Steel Modulus of Elasticity, psi

I = Joist Girder Moment of Inertia, in^4

The joist girder moment inertia was calculated using SJI Specifications for Joist Girders:

$I = 0.018NPLd$

N = number of joist spaces

P = panel point load, kip

L = joist girder span, ft

d = joist depth, in.

The connection of steel joist girders to columns was designed as a 9 x 9 ½ x 1 in. plate to accommodate for a minimum bearing length of 4 in. for joist girder extensions. Cap plates will be attached to the tops of HSS columns with 5/16 in. fillet welds (AISC 2011). Each R-type bearing seat of joist girders will be bolted to the plate with two ¾ in. ASTM A307 bolts in 13/16 in. x 1 ½ in. slotted holes. A simple view of the joist girder to column connection is shown in Figure S5. At the connection, joist girders will have the minimum required 7½ in. bearing depth as required by page 180 of SJI Specifications (SJI 2010). AISC Section K1.3b was used in determining limit states of the cap plate to HSS connection (AISC 2011). Available strength of the HSS in local wall yielding due to a concentrated force on the end of a rectangular HSS was calculated using the following equation:

$R_n = 4F_yt[5t_p + N] \leq BF_yt$ (AISC Equation K1-11)

$\phi = 1.0$

N = bearing length of load

Available strength of the HSS in wall local crippling was calculated using the following equation:

$R_n = 0.8t^2[1 + (6N/B)(t/t_p)^{1.5}][EF_yt_p/t]^{0.5}$ (AISC Equation K1-12)

$\phi = 0.75$

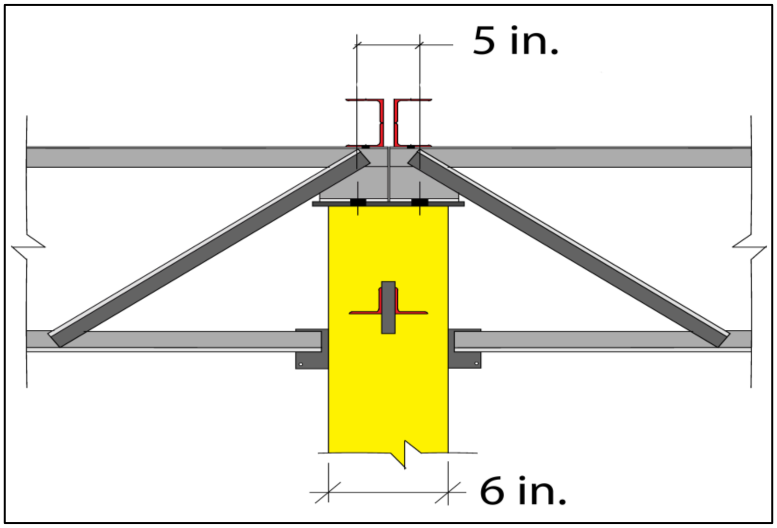

Figure S5: Joist girder to column connection

Steel Columns

The joist girders transfer the loads to which they are subjected, as well as their self-weights, to interior columns as point loads. The LRFD factored load (AISC Equation 2-3c) transferred to these columns, is 80 kips. An HSS 6x6x3/8 square tube column with an axial capacity of 107 kips was selected using the 2011 AISC Steel Construction Manual. This conservative selection was chosen through design checks against limit states of flexural buckling due to axial load, wall plastification due to tensile forces caused by bracing connections, and local yielding of sidewalls due to cap plate bearing.

Because the selected columns are classified as non-slender and compact, the nominal compressive strength based on the limit state of flexural buckling was calculated using the following equation:

$P_n = F_{cr}A_g$ (AISC Equation E3-1)

F_{cr} = Critical stress, ksi

A_g = gross cross-sectional area of column, in^2

Because $KL/r > 4.71\sqrt{(E/F_y)}$:

$F_{cr} = 0.877Fe$ (AISC Equation E3-3)

F_e = elastic buckling stress, ksi

$$F_e = \frac{\pi^2 E}{(\frac{KL}{r})^2} \qquad \text{(AISC Equation E3-4)}$$

E = steel modulus of elasticity, ksi

KL = effective length of column, in

r = column radius of gyration, in

Lateral Bracing

Tension-only bracing was selected to resist lateral deflection of the building frame. This selection was based on research of similar low-rise diaphragm structures. X-bracing applied across 5 bays on each exterior and one interior wall of the structure is sufficient to prevent significant deflection of the roof diaphragm in relation to the structure's slab in the event of maximum seismic or wind loading. The placement of bracing in an exterior wall is shown in Figure S6. Cable bracing will be used because of its available strength in tension. It is also lightweight, aesthetically pleasing, and takes up little space. Diaphragm shear is equivalent to that of seismic shear as determined in the **Design Loads** section of the structural report. The angle from horizontal to bracing is 29.9°. Resultant forces equal 132.8 kips of shear acting along braced walls of the structure. 6 x 19 Uncoated IWRC, improved plow steel wire rope members with 7/8" diameter and having a length of 46.14 ft were designed to resist lateral forces. The breaking load of this cable selection is 69.2 kips. Using a factor of safety of 2, as required by AISC Design Guide 10: Erection Bracing of Low-Rise Structural Steel Frames, the nominal breaking strength of each bracing member is 34.6 kips, exceeding the maximum cable design force of 31 kips that seismic forces would cause in each cable. The design tension in the cables was determined using Figure S7 and the following equations:

$T_{br} = T_x/(\cos\theta)$

T_{br} = cable tension, kips

$\theta = \tan^{-1}(h/L)$ = angle of bracing from horizontal

h = vertical distance between ends of cable, ft

L = horizontal distance between ends of cable, ft

$T_x = V/5$ = x-component of cable tension, kips

V = seismic shear, kips

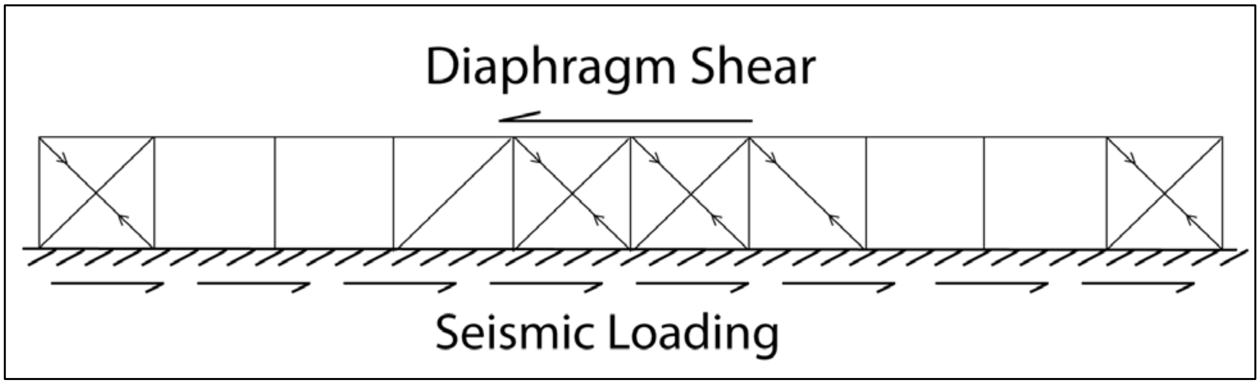

Figure S6: Lateral bracing detail

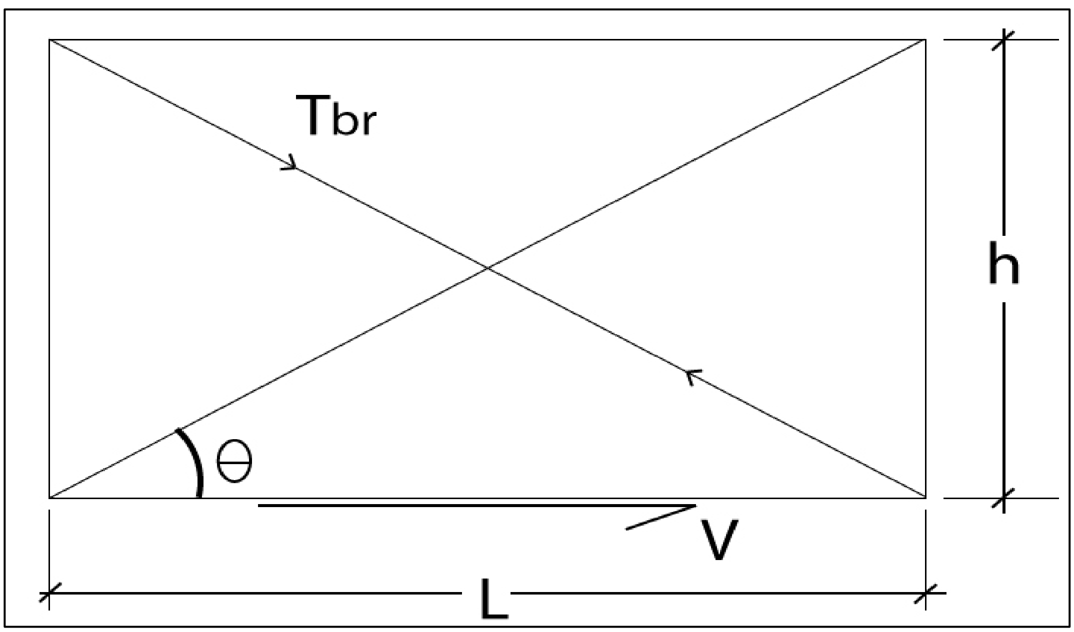

Figure S7: Cable tension

Cables will be pre-stretched to remove constructional elongation. To determined amount of stretch necessary, elastic stretch and constructional stretch were determined using the following equations from AISC Design Guide 10:

$\Delta_1 + \Delta_2$ = cable stretch, ft

$$\Delta_1 = \frac{0.2(\mathrm{NBS} - \mathrm{P})\mathrm{L}}{\mathrm{A}(0.9)\mathrm{E}} \qquad \text{(Equation 5-1)}$$

$$\Delta_2 = \frac{(\mathrm{CDF} - 0.2(\mathrm{NBS}))(\mathrm{L} + \Delta_1)}{\mathrm{A}(\mathrm{E})} \qquad \text{(Equation 5-2)}$$

$$\Delta_{cs} = \frac{\text{Applied Load}}{0.65(\mathrm{NBS})}(\mathrm{CS}\%)(\mathrm{L}) \qquad \text{(Equation 5-3)}$$

NBS = nominal breaking strength, lbs

P = preload, lbs

CDF = cable design force, lbs

L = length, ft

A = net area of cable, in^2

Preload in the cable was determined using the following equation in AISC Design Guide 10 for a cable drape of 3 ¾" for a 7/8" cable diameter:

$$\mathrm{P} = \frac{\mathrm{q}x^2}{8\mathrm{A}\cos\psi} \qquad \text{(Equation 5-4)}$$

q = cable weight, lbs

x = horizontal distance between connection points, ft

A = cable drape, ft

ψ = angle between horizontal and cable, degrees

Based on Table 5.6 of AISC Design Guide 10, a 7/8 in., 6 x 19 cable has the following properties:

Area A = 0.36 in^2

NBS = 69,200 lbs

$\phi = 0.5$

$E = 15{,}000{,}000$ psi

Total elongation was used to determine lateral movement of the tops of columns and to check available cable force against cable forces including PΔ effects of vertical columns loads.

The connections of lateral bracing to columns were designed as plates with a single hole in the center for a 1 ¼ in. pin that attaches to the jaw of the cable. The ends of cables will have clevises with a grips of 0.56 in and pins of 1¼ in. diameter. According to AISC Section D5, the design tensile strength of pin-connected members shall be the lower value obtained according to limit states of tensile rupture, shear rupture, bearing, and yielding. Shear rupture force of the pin was found to be 52.2 kips, exceeding the 32 kip design tension of the steel cable. The pin was checked for shear rupture using the following equation:

$P_n = 0.6F_uA_{sf}$ (AISC Equation D5-2)

$\phi_{sf} = 0.75$

$A_{sf} = 2t(a + d/2)$, in.^{2}

a = shortest distance from edge of pin hole to edge of the member measured parallel to the direction of the force, in.

d = pin diameter, in.,

t = thickness of the plate, in.

The plate was designed using dimensional requirements for pin hole diameter, plate width, and the distance from the pin hole to the edges of the plate, as stated in AISC Section D2. A plate with ½ in. thickness and a width of 6 in. will be attached to the column wall at an angle of 60.1° from the column face. HSS limit states were checked to ensure sufficient wall thickness for the attachment of the plate with the design tensile force acting upon it. A force causing HSS plastification was found to be 46.61 kips, exceeding the 32 kip design tension of the steel cable. The following calculation was used to check for HSS plastification:

$$R_n \sin\theta = \frac{F_y t^2}{\left(1-\frac{t_p}{B}\right)}\left[\frac{2I_b}{B} + 4\sqrt{1-\frac{t_p}{B}Q_f}\right] \qquad \text{(AISC Equation K1-12)}$$

$\phi = 1.00$

R_n = Force causing plastification, kip

θ = angle of plate from column face

F_y = HSS yield strength, ksi

t = HSS wall thickness, in.

t_p = plate thickness, in.

B = plate width, in.

I_b = length of plate attached at column, in.

$Q_f = \sqrt{(1- U^2)}$

$$U = \frac{P_u}{F_y A_g}$$

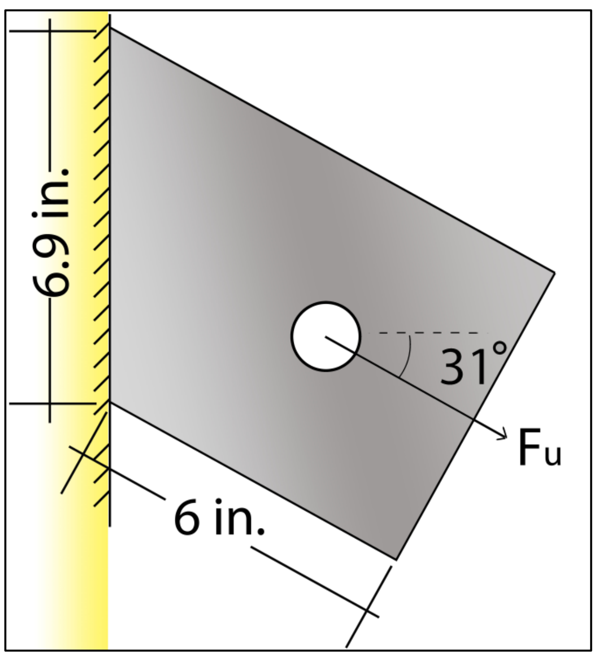

Figure S8: Lateral bracing connection to column

The limit state for punching shear was checked using AISC Equation K1-3. Using this equation, the maximum thickness was calculated and found to be 0.56 in. which exceeded the plate design thickness of ½ in. According to AISC Design Guide 24 Section 6.2, the chord slenderness for rectangular HSS usually falls in between 15 and 25. Slenderness of the selected HSS 6x6x3/8 is equal to 17.2. For the weld connecting the plate to the column, fillet welds will be used. The minimum weld size was found to be 3/16 in. per AISC Table J2.4, and the maximum weld size was found to be 7/16 in. per AISC Section J2.2.b. A weld of 5/16 in. was selected. Design strengths of all the following limit states exceed the design tension of structural cable bracing. Available strength of the fillet weld was found using the following equation:

$\phi R_n = 1.392Dl(1+0.5\sin^{1.5}\theta)$ (AISC Equation J2-5)

$\phi = 0.75$

D = weld size in sixteenths of an inch

l = length of weld, in.

θ = loading angle

Available strength of the plate at weld was calculated using the following equations:

For Tensile Yielding:

$R_n = F_y A_g$ (AISC Equation J4-1)

$\phi = 0.9$

F_y = plate yield stress, ksi

A_g = plate cross-sectional area in tension, in^2

For Tensile Rupture:

$R_n = F_u A_e$ (AISC Equation J4-2)

$\phi = 0.75$

F_u = plate ultimate stress, ksi

A_e = effective area in tension, in^2

For Shear Yielding:

$R_n = 0.6F_y A_{gv}$ (AISC Equation J4-3)

$\phi = 1.0$

F_y = plate yield stress, ksi

A_{gv} = gross area of plate in shear, in^2

For Shear Rupture:

$R_n = 0.6F_uA_{nv}$ (AISC Equation J4-4)

$\phi = 0.75$

F_u = plate ultimate stress, ksi

A_{nv} = net area of plate in shear, in^2

For Block Shear:

$R_n = 0.6F_uA_{nv} + U_{bs}F_uA_{nt} \leq 0.6F_yA_{gv} + U_{bs}F_uA_{nt}$ (AISC Equation J4-5)

U_{bs} determined from AISC Table D3.1

Slab on Grade Design

The warehouse floor slab was designed as a slab on grade, with the minimum design load of 250 psf for a storage warehouse with heavy loading. The building is a storage warehouse, therefore the floor slab was designed for lift truck axle loading and rack storage loading, which act as concentrated loads anywhere on the floor slab. The floor slab classification given in ACI 302 is a Class 6 floor as described as an industrial floor subject to heavy traffic. The slab is to be constructed as Type B, which is a floor slab with shrinkage control reinforcement. The compressive strength recommended for an ACI Class 6 floor slab is 4500 psi at 28 days (Spears 1992). The Portland Cement Association (PCA) Method was used in the design (Ringo and Anderson 1996). The PCA recommends a modulus of rupture coefficient of nine be used. The modulus of rupture for the designed floor slab is 302 psi, with a factor of safety of 2.0 because exact loading is not known. The soil on the site is classified by the Unified Classification System as being SM-SC with a subgrade modulus of 150 pci, according to the geotechnical report, which is a typical subgrade modulus for this classification of soil.

The floor slab was designed using a reasonable lift truck with a weight of 22 kips, a lift capacity of 20 kips, and single wheel spacing of 42 in. PCA design method charts were used because the chart and tables allow slab thickness to be selected for single wheel axle loads and rack support post loading. It was determined from these charts that the slab thickness is to be 11 in.

The floor slab was also designed for rack storage loading. The design used an average size storage rack 8 ft long and 4 ft wide with five shelves with a capacity of 100 psf per shelf. The slab thickness was determined to be 10 in. using the PCA design charts. This slab thickness is less than the design for the lift truck, so the slab thickness will be 11 in.

The slab also has a nominal amount of distributed reinforcement to control the effects of shrinkage and temperature. A construction joint is to be placed every 80 ft., running north-south of the building. The amount of reinforcing steel was determined by the subgrade drag equation:

$$A_s = \frac{FLw}{2f_s}$$

A_s=cross-sectional area of reinforcing steel, in^2

F=1.5, coefficient of friction between base and slab

L=80, slab length between free ends, feet

w=137.5, weight of the concrete slab, psf

f_s=60,000, allowable steel stress, psi

The amount of reinforcing steel was determined to be 0.206 in^2/ft. The reinforcing steel is to be welded wire reinforcement with a designation of 4x4-W5.5xW5.5. The slab showing the reinforcing steel is shown in Figure S9. Control joints are to be sawcut at a depth of 2-¾ in. every 40 ft, running east-west of the building. Isolation joints are to be installed around all column footings to permit vertical and horizontal movements between the floor slab and column footings.

Figure S9: Slab on grade

Office Building Design

The office space will be placed on the interior of the warehouse structure at southwest corner. The office space design consists of 9,600 ft^2 contained on two stories in order to take advantage of the 40 ft by 40 ft column spacing of the overall structure of the warehouse. The dimensions of the office space are to be 120 ft by 40 ft to maximize the floor space of the warehouse. The second floor office space used to design the structural framing is shown in Figure S10. The spaces used as office space were designed with a live load of 50 psf and the hallways/corridors were designed with a live load of 80 psf.

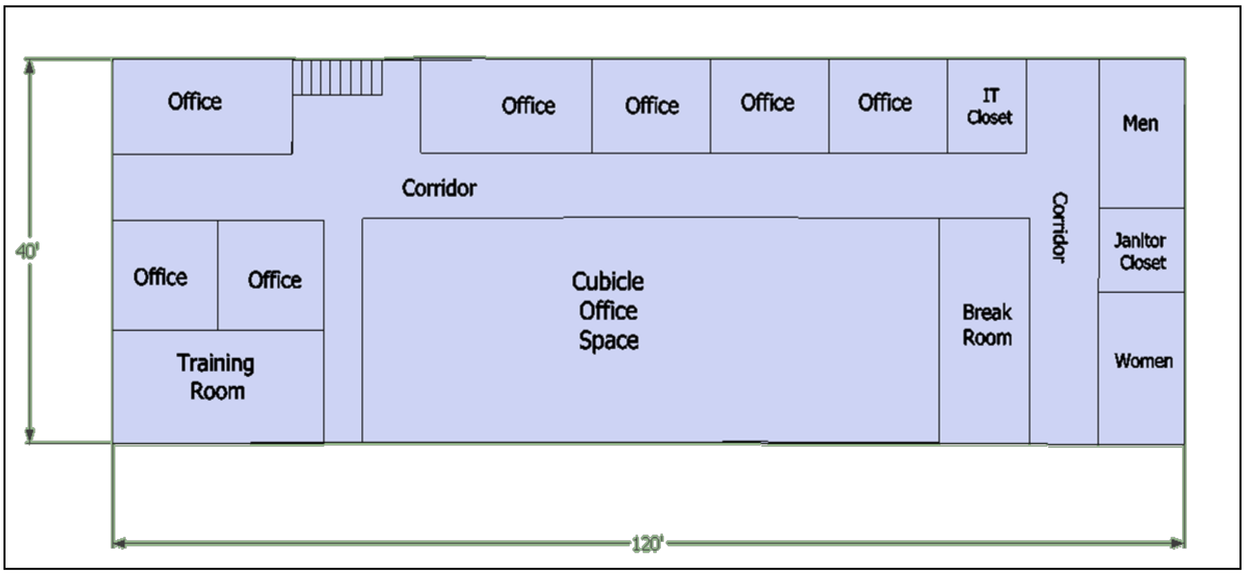

Figure S10: 2nd floor office space layout

The second floor slab was designed as a continuous one-way slab. The concrete is to have a compressive strength of 3,000 psi at 28 days with grade 60 reinforcing steel bars. The slab is to be a minimum of 4 in. in thickness. The ACI moment coefficients for integral end, more than two spans were used to determine the positive and negative moments to size the reinforcing steel. The reinforcing steel is presented in Figure S11. It was determined #3 bars spaced at 11 in. center of center for negative moment at the end of the slab. The first positive moment located at the first clear span from the end requires #4 bars spaced at 8 in. center of center. The second negative moment from the end of the slab over the support requires #4 bars spaced 11 in. center of center. The remaining negative moments over supports require #4 bars spaced 10 in. center of center and the remaining positive moments between the supports require #4 bars spaced 7 in. center of center. The slab will not require reinforcement for shear because the factored shear strength (Vu) of 991 psi is less than the reduced shear strength (ϕVn) of 3,285 psi. The reinforcing steel required controlling the effects of temperature and shrinkage is #3 bars spaced 12 in. center of center transversely of the moment resisting steel. The development lengths were determined and shown in Figure S12 to be 18.5 in. to the right of each support and 12.5 in. to the left of each support. For simplicity to the contractor, the reinforcing steel to resist the negative moments will extent the entire length of the slab with #4 bars spaced 10 in. center of center and a 1.5 in. clear cover on each side of slab. Also, the reinforcing steel to resist the positive moments will extent the entire length of the slab with #4 bars spaced 7 in. center of center and a 1.5 in. clear cover on each side of the slab. The slab designed gives the contractor the option use stay in place form deck 2 in. deep, 18 gage or another material in equivalent.

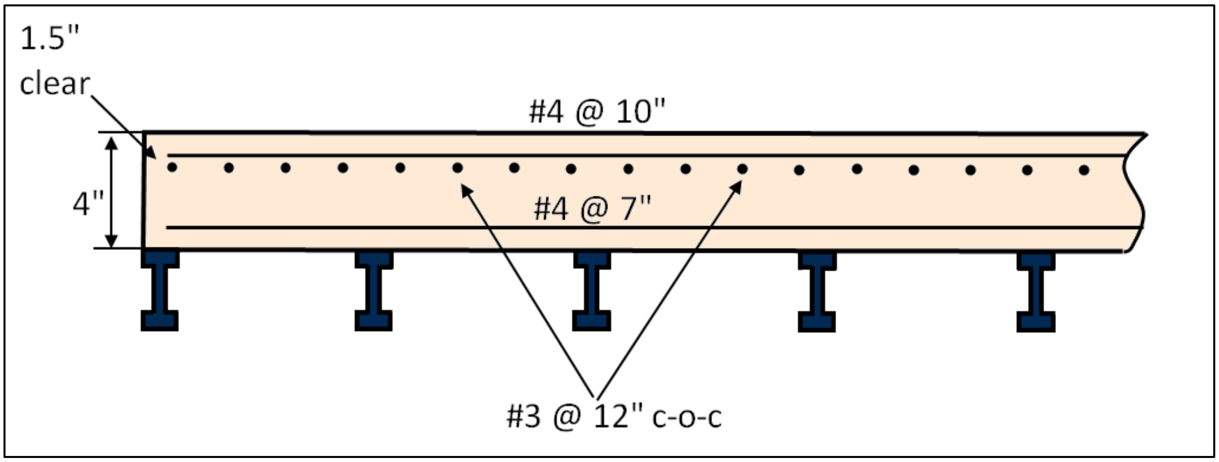

Figure S8: 2nd floor concrete floor slab

After the 2nd floor slab was designed, the structural framing was determined. Structural analysis was conducted on each structural member with the aid of a structural analysis program, *Enercalc*, with a spacing of 10 ft. The office floor framing system was designed to support loads of 50 psf for office areas and 80 psf for hallways/corridors. All members were limited to a depth of 12 in. to minimize the space between floors. Each tributary area distributed to each member was analyzed as a simply supported beam. The maximum moments, maximum shear, and the support reactions were determined for each loading situation on each floor beam. This information was used to select an appropriate sized member allowed to support each loading case including the self-weight of the member from the AISC Steel Construction Manual. In order to simplify logistics in ordering and erecting the steel, all beams were selected as W12x120. The reactions determined from each loading case were used as point loads and analyzed to determine the size of the girder in the same procedure as stated previously. The girders selected are to be W18x192. The design layout of the 2nd floor framing is shown in Figure S12. The girders then distribute the loads to the columns used in the support of warehouse structure.

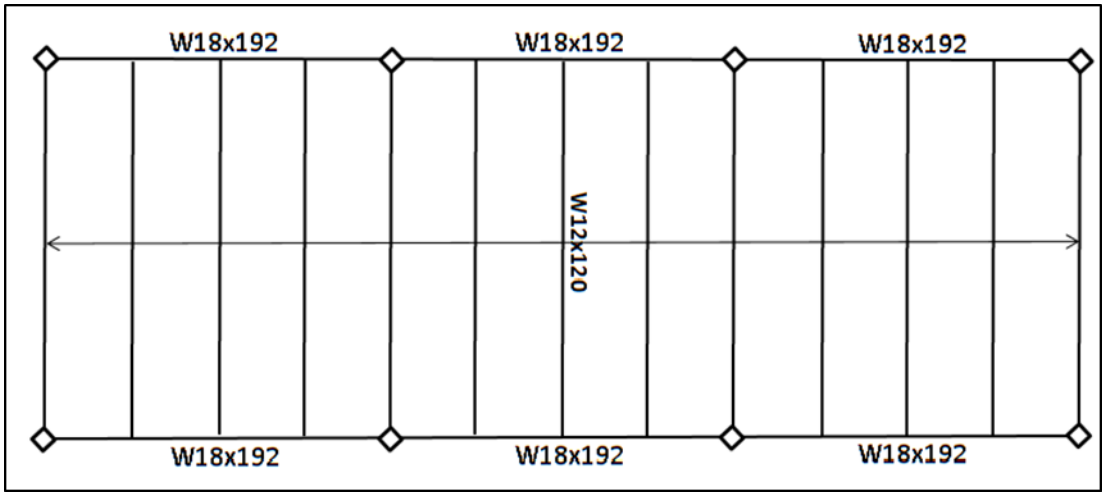

Figure S12: 2nd floor framing

The connections for the 2nd floor framing chosen are single plate framing connections. These connections offer an economical type of flexible connection, as well as simplicity for steel erectors. The bolt holes are prepunched in the plate and web of the beam and the plate is shop-welded to the supporting beam or column.

The single plate connection connecting the W12X120 beams to the W18X192 girder beams is illustrated in Figure S13. A required shear force of 30.1 kips was obtained from the end reaction from the structural analysis of the W12X120 beam. The available strength of the connection must equal or exceed the required shear force in order to satisfy all design requirements of the AISC Construction Manual. For a required strength of 30.1 kips, AISC Table 10-9a was referenced to obtain a plate thickness of ¼ in., A36 steel with 3 - ¾ in. A325 N bolts and 3/16 in. E70 fillet welds. The vertical edge distance, L_{ev}, is 1-1/4 in., which satisfies AISC Specification Table J3.4 requirements. The horizontal edge distance, L_{eh}, is 1-1/2 in., which satisfies the design requirement of twice the size of the bolt diameter. The clear distance between bolts is to be 3 in. The available strength of the supported beam web was obtained from Table 10-1 in the AISC Construction Manual for top flange coped and multiplied by the web thickness. This value was 133.4 kips, which exceeds the required strength of 30.1 kips. The next design check was the allowable strength of the bolts resisting shear. The allowable bolt shear was determined from the following equation:

$\phi R_n = \phi F_n A_b n$ (AISC Equation J3-1)

$\phi = 0.75$

F_n= nominal tensile stress F_{nt} (AISC Table J3.2)

A_b= nominal area of bolt

n= number of bolts

The allowable strength for bolt shear was determined to be 59.7 kips which exceeds the required strength of 30.1 kips. The available block shear strength was checked for block shear rupture along the shear failure path and the perpendicular tension failure path. This was determined from the following equation:

$\phi R_n = \phi[0.6F_uA_{nv}+U_{bs}F_uA_{nt}] \leq \phi[0.6F_yA_{gv}+U_{bs}F_uA_{nt}]$ (AISC Equation J4-5)

$\phi = 0.75$

U_{bs}= 1.0, tension stress is uniform

A_{gv}= gross are subjected to shear, in^2

A_{nt}= net area subject to tension, in^2

A_{nv}= net area subject to shear, in^2

The allowable strength for block shear rupture was determined to be 54.55 kips, which exceeds the required strength of 30.1 kips. The bearing strength at the bolt holes is based on the strength of the parts being connected and the arrangement of the bolts. The nominal bearing strength at the bolt holes were determined by the following equation:

$\phi R_n = \phi[1.2L_ctF_u]n \leq \phi[2.4dtF_u]n$ (AISC Equation J3-6a)

$\phi = 0.75$

L_c= clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material, in.

d= nominal bolt diameter, in.

F_u= minimum tensile strength of the connected material, ksi

t= thickness of connected material, in.

n= number of bolts

The nominal bearing strength at the bolt holes were determined to be 41.6 kips, which also exceeds the required strength of 30.1 kips. The last design check for the connection was for weld shear. The allowable strength of the welds was design for minimum weld strength of 70 ksi and a 3/16 in. fillet weld. The following equation was used to determine the weld strength:

$\phi R_n = \phi F_w A_w$ (AISC Equation J2-3)

$\phi = 0.75$

F_w= nominal strength of the weld metal per unit area, ksi (AISC Table J2.5)

A_w= effective are of the weld, in^2

The allowable strength of the welds was determined to be 35.5 kips, which exceeds the required strength of 30.1 kips.

All design checks were conducted on the single plate connection determined from AISC Table 10-9a. The plate chosen to be used will be PL ¼ X 3-1/2 X 8-1/2" A36 steel with 3 - ¾ in. A325 N bolts and 3/16 in. E70 fillet welds shown in Figure S8. The supported beam is to be coped at the top flange only a vertical distance of 2-1/4 in. and a horizontal distance of 4-3/4 in.

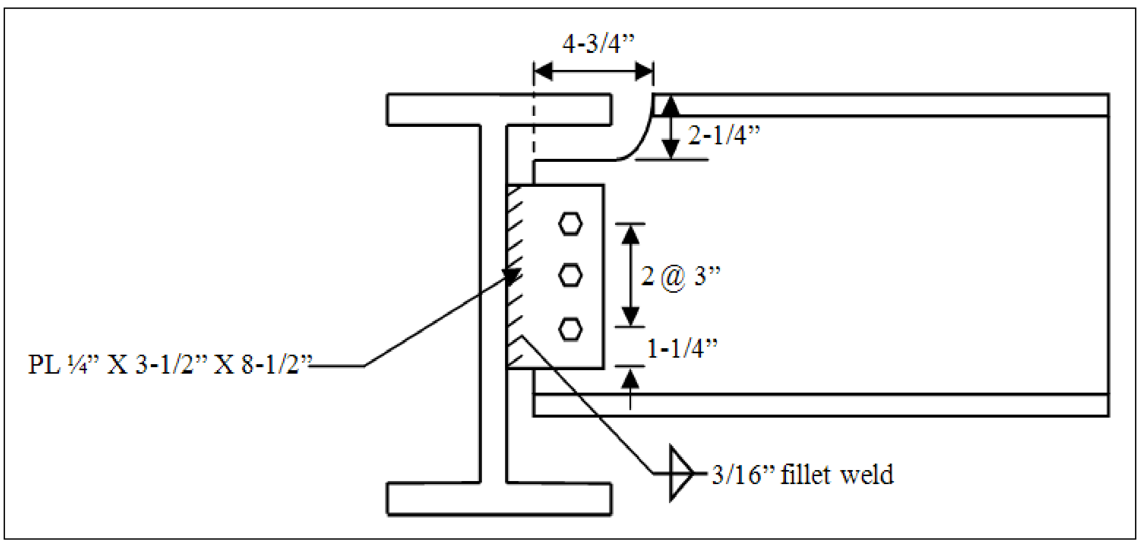

Figure S13: Beam to girder-beam connection

The single plate connection connecting the W18X192 girder beam to the HSS 6X6X3/8 column is illustrated in Figure S14. This connection design was performed similar to the beam to girder beam design. The required shear strength determined from end reaction of the structural analysis of the W18X192 was 43.02 kips. The vertical edge distance, L_{ev}, is 1-1/4 in., which satisfies AISC Specification Table J3.4 requirements. The horizontal edge distance, L_{eh}, is 1-3/4 in., which satisfies the design requirement of being greater than twice the size of the bolt diameter. The clear distance between bolts is to be 3 in. By utilizing AISC Table 10-9a, a connection using a plate thickness of ¼ in., a height of 11-1/2 in., 4 - ¾ in. A325 N bolts, and 3/16 in. fillet weld has an allowable shear strength of 52.2 kips, which exceeds the required shear strength of 43.02 kips.

The design connection was then checked for the supported beam web, bolt shear, block shear rupture, bolt bearing, weld shear, and shear yielding of the plate. For standard holes, eccentricity can be ignored because less than 9 bolts are used. All design checks for this connection used the same equations as mentioned for the previous connection. All allowable shear strengths exceeded the required shear strength of 43.02 kips. The final plate design consists of a PL ¼ X 5 X 11-1/2" A36 steel with 4 - ¾ in. A325 N bolts and 3/16 in. E70 fillet welds.

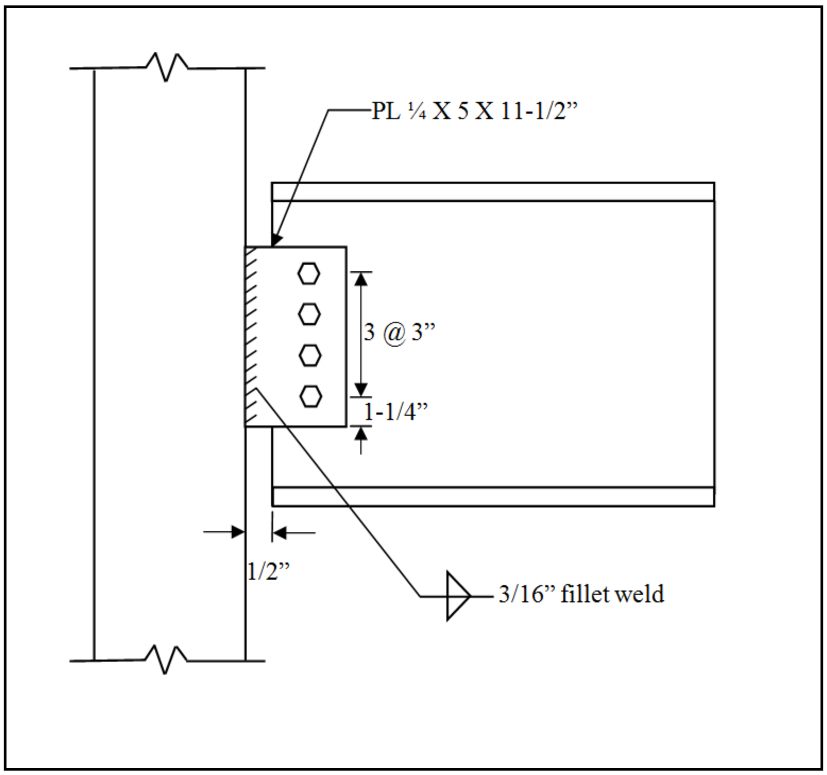

Figure S14: Girder-beam to column connection

Concrete Tilt-Up Panel Design

The material chosen for the exterior walls of the warehouse are concrete tilt-up panels. Tilt-up panels were chosen because of construction cost savings and speed of construction. Concrete tilt-up panels are cast in place on site, which reduces transportation cost. The speed of construction also reduces construction cost because concrete panels take less time to construct than ordinary masonry walls and can be done while other aspects of the project are taking place.

The tilt-up wall panels were designed with a height of 28 ft, including the minimum of 3 ft as the parapet required by ASCE 7-10. Since the column spacing of the warehouse structure is 40 ft, the tilt-up panels were designed with a length of 20 ft. Three different panels were designed for the exterior walls: solid panel with no openings, panel having an opening with a height of 15 ft. and a length of 10 ft for the loading dock, and a panel having an opening with a height of 9 ft and a length of 6 ft for entry doors. The panels were also designed for final in-place condition. The tilt-up wall panels were designed as a non-load bearing wall, which means the only vertical load applied to the wall is the self weight of the panels. The lateral design load used was the components and cladding wind load of 25.8 psf.

Seismic lateral load was considered before designing the tilt-up panels. According to ACI 551.2, a load design check was utilized by the following equation:

$F_p = 0.4IS_{DS}W_p$

I= seismic importance factor

S_{DS}= spectral response acceleration parameter at short periods

W_p= weight of the panel

If the value of F_p is less than the wind load, then the wind load controls and the wall panels are to be designed using the wind loads for components and cladding. The value of F_p was determined to be 7.5 psf, which is less than the wind load and in this event; the wind load of 25.8 psf was used in the design of the wall panels.

The compressive strength of the concrete is to be 4,000 psi and the reinforcing steel is Grade 60. The wall panels were designed in accordance with ACI 551.2 with the following load cases:

1) 1.2D + 0.8W

2) 1.2D + 1.6W

3) 0.9D + 1.6W

The thickness of the panels is to be 6.25 in. with a single layer of vertical reinforcement. The depth of the reinforcing steel is in the middle of the panel at a distance of 3.13 in.

The solid panel was designed as simply supported with the maximum moment occurring at mid-height of the panel per ACI 551. Load Case 2 controlled the design of the panel. The design required 31 - #6 reinforcing bars with an area of 13.84 in^2 be used for vertical reinforcing. The first design check was to check the vertical stress at the mid-height section of the panel by dividing half the factored panel weight by the area of the panel. The vertical stress was calculated to be 17.5 psi, which is less than 0.06f'c of 240 psi in accordance to ACI 551. The tensile strain was determined to be 0.0047, which was greater than the strain in the concrete of 0.0021. Per ACI 318 Section 14.8.2.4, the reinforcement in the panel should provide design strength be greater than the cracking moment. The cracking moment was obtained using the modulus of rupture. The cracking moment was calculated to be 61.7 k-ft. and the design moment strength was determined to be 165.2 k-ft, which is satisfactory for the design. The minimum ratio of vertical reinforcement to gross area of 0.0015 is less than the actual ratio of vertical reinforcement of 0.00909. The applied moment was determined per ACI 318 Section 14.8.3 to be 152.8 k-ft., which is less than the design moment strength. The maximum deflection due to service loads must not exceed the height of the panel divided by 150. The maximum allowable deflection is 2.24". The deflection of the wall design is 0.24", which is less than the maximum allowable deflection of 2.24". The horizontal reinforcement was determined per ACI 318 Section 14.3.3 to be 0.002 times the gross area. This was determined to be 4.2 in^2, which 20 - #4 horizontal reinforcing bars are to be used. The total design of the solid concrete wall panel is illustrated in Figure S15.

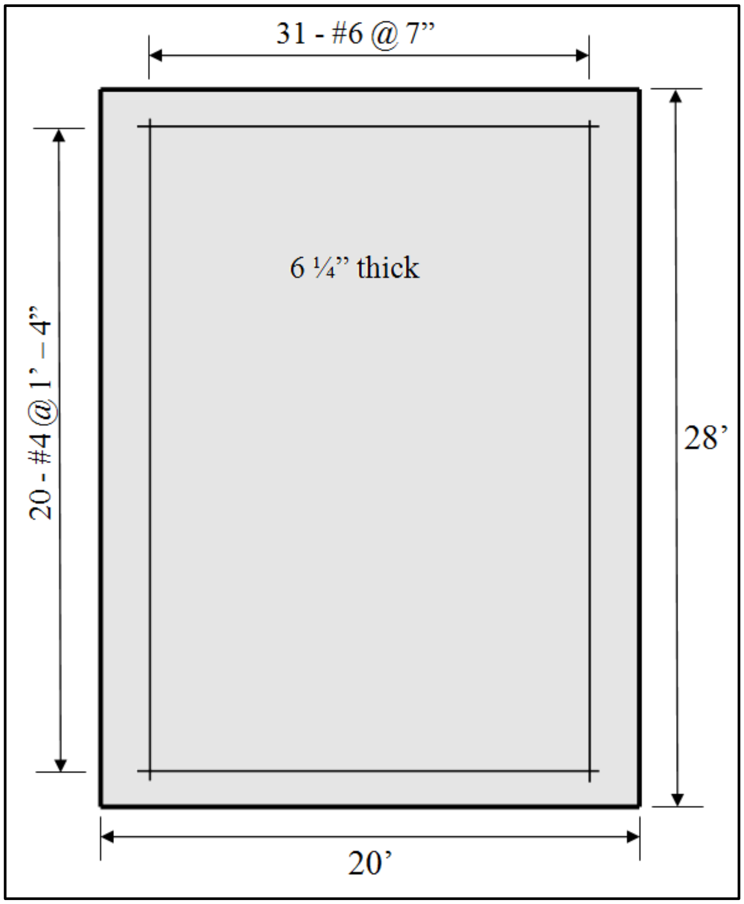

Figure S15: Solid concrete wall panel

The design for the wall panels with openings are designed the same way as the solid wall panels, but only the supporting legs on each side of the openings are analyzed. The design for the wall panels with openings for the truck loading docks and a personnel door are presented in Figures S16 and S17, respectively. The reinforcing steel for both panels are centered in the middle of panel at a distance of 3.13 in. The truck loading dock wall panel will require 11 - #6 vertical reinforcing bars in each supporting leg and 16 - #6 vertical reinforcing bars directly above the opening. It was also determined that 11 - #4 horizontal reinforcing bars are required for each leg and 10 - #4 horizontal reinforcing bars are required above the opening in the panel.

The personnel door wall panel was designed with an opening 9 ft high and 6 ft long. The panel will require 9 - #6 vertical reinforcing bars in each supporting leg and 10 - #6 vertical reinforcing bars directly above the opening. The horizontal reinforcement required is 7 - #4 bars in each supporting leg and 14 - #4 bars above the opening in the panel.

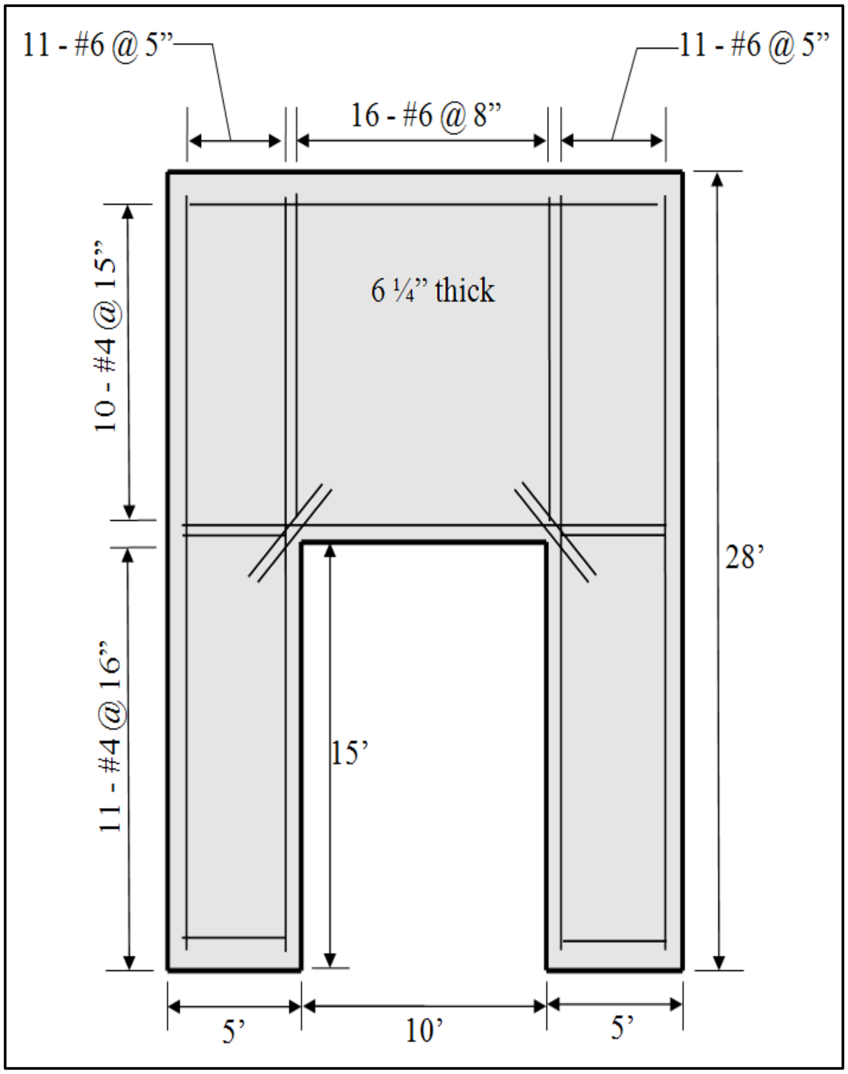

Figure S16: Truck loading dock concrete panel

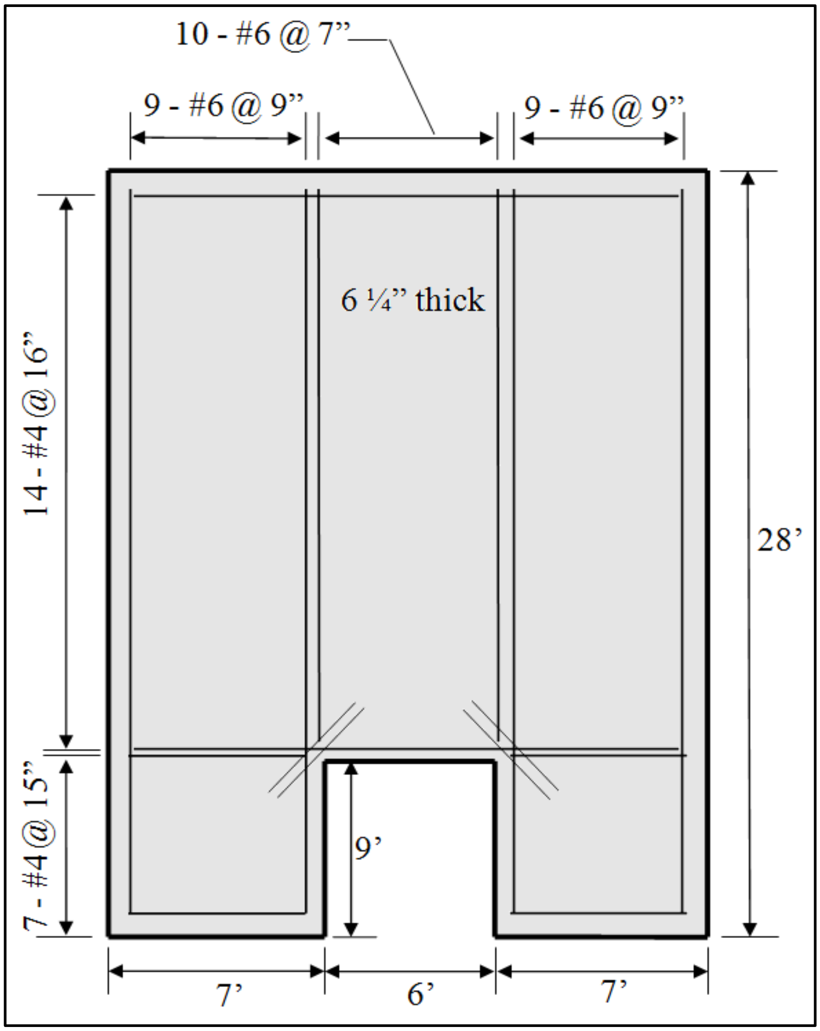

Figure S17: Personnel door concrete panel

The bottoms of tilt-up panels will be secured to the slab and the tops secured to the tops of joists and joist girders. Connections were designed per the fifth edition of PCI Design Handbook to resist horizontal tension and shear (PCI 1999). Exterior footings will carry the bearing load of the self-weight of wall panels. Top connections of panels along the 400 ft span of the warehouse will be welded to the top chords of the K-series joists, and to the top chords of joist girders along the 640 ft span of the warehouse. Tilt –up panels will be offset from the edges of the structural steel and slab of the structure by 3 in. Each of these connections will consist of 6 x 6 x 5/16 in. connection angles with the vertical leg bolted to concrete inserts in the panels with one 1½ in. ASTM A36 bolt and the horizontal leg welded to top chords with 6 inches of ¼ in. fillet welds. From Figure 6.15.9, design tension of an A36 bolt is equal to 57.6 kips, exceeding the required strength of 52.9 kips, determined using Equation 6.5.18 (PCI 1999). Using equation J2-3, the design strength of the ¼ in. fillet welds is equal to 34 kips, exceeding the required shear strength of 15.6 kips (AISC 2011). The location of the 1 9/16 x 1 7/8 in. short slot in the angle was determined using Figure 6.15.14 (PCI 1999). A detail of this connection is shown in Figure S18.

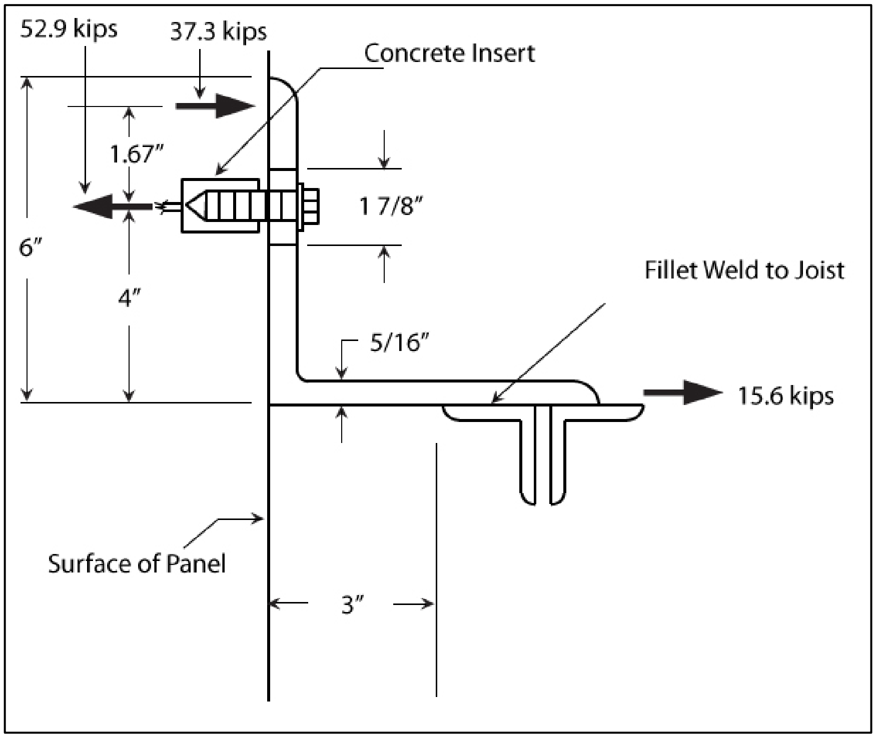

Figure S18: Panel-support detail

The bases of all panels will sit on shims transferring the weight of the panel to the exterior footing of the structure. The bases will be restrained horizontally by welding a 6 x 6 x ¼ in. plate to a 6 x 6 x ¼ in. plate embedded 4 in. deep in the panel with two ½ in. diameter studs, and a 11 x 6 x ¼ in. plate embedded 4 in. deep in the slab with two ½ in. diameter studs, as shown in Figure S16. The limiting design force of the concrete around the studs is 16.1 kips as determined by Figure 6.15.7A, exceeding the required strength of 15.64 kips (PCI 1999). The limiting design force in the studs at the connection as determined by Figure 6.15.8, Table B is 16 kips, exceeding the required strength of 15.64 kips (PCI 1999). Using equation J2-3, the design strength of the ¼ in. fillet welds used in the connection is equal to 34 kips, exceeding the required shear strength of 15.6 kips (AISC 2011). Detailing of the tilt-up panel to slab connection is shown in Figure S19.

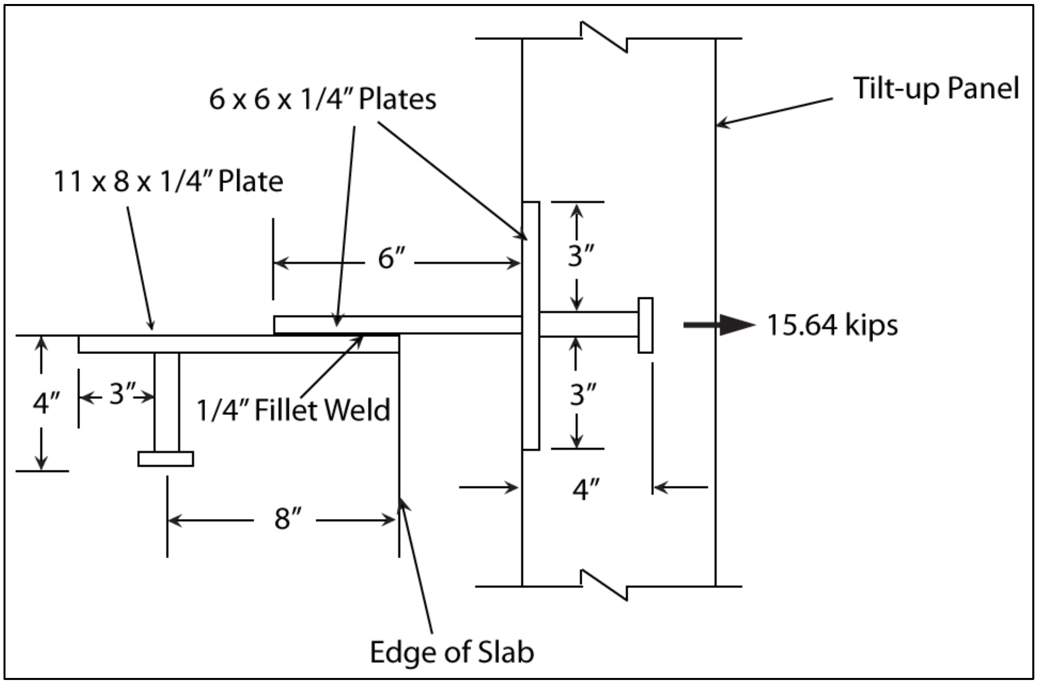

Figure S19: Panel-slab detail

Concrete panels will be secured to each other with slotted plates bolted to concrete inserts on the interior of the panels at mid span of the panel height. ½ in. ASTM A307 bolts will connect a 10 x 4 ½ x ¼ in. plate with 9/16 x 1 ¼ in. long slots to the ends of adjacent collinear panels. Bolts will be located 2 ½ in. from the edge of each panel. Panels at corners of the structure will be connected with ¼ in. thick angles bolted to panels using the same bolts and long slots as those used in collinear panels. Detailing of these connections is shown in Figure S20.

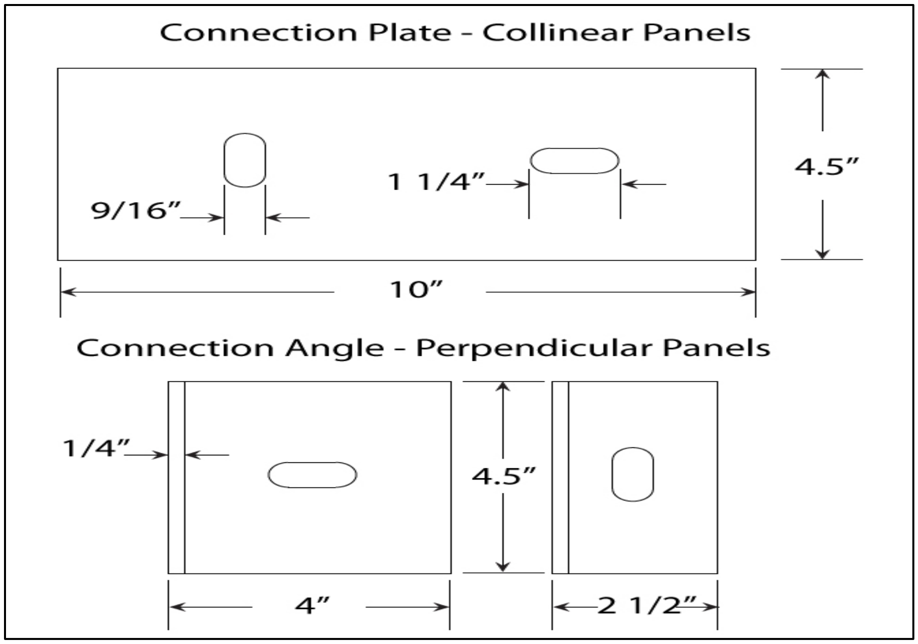

Figure S20: Panel-Panel Detail

Structural Conclusion

The structural design team of Optimal Civil Designs has developed what we believe to be the most cost effective and structurally sound design of the warehouse of interest on Coalburg Road. The design provides a gross floor area of 251,200 s.f. for storage and 9,600 s.f. of available office space. The design also provides the warehouse interior with a clear height of 21 ft. A cost analysis comparing total steel weights associated with varying bay sizes and joist spacings was utilized in providing the most cost-effective structural layout. By designing the steel structure as a braced frame with steel decking acting as a roof diaphragm, we ensured the safety of the structure and concurrently cut costs associated with alternative lateral force resisting systems. We made a conscious effort throughout the design process to select the most ideal members available, such as HSS columns, steel joist girders, deep-rib steel decking, and cable bracing. Design codes were followed closely in the design of each component of the structure, for both concrete and steel. The concrete slab was designed not only for the minimum design load of 250 psf, but it was also designed to support lift trucks and storage racks that typical storage warehouses require. The exterior walls were chosen to be concrete tilt-up panels because tilt-up panels offer a cost-effective material for construction cost savings and speed of construction. Based on code requirements that our design meets and structural analysis of the frame, we are confident that our warehouse design is sufficient for the many purposes that it may serve.

7. Environmental

The environmental portion of the Coalburg Road warehouse project includes the design of the storm water system, the sanitary sewer system, and the potable water system. Also addressed, are considerations for sustainability, hazardous waste, air pollution, noise pollution, and light pollution.

Storm Water

The existing storm water conveyance path ways will need to be redirected due to the construction of the warehouse and introduction of impervious surfaces to reduce erosion of these pathways. The current conditions of the storm water drainage system is naturally formed open channel ditches due to the loose fill placed onto the site, which is primarily made up of mine spoils. The construction of the warehouse will include a new buried storm water system for the management of runoff. The primary goal of designing the new storm water system is to provide underground infrastructure to convey and manage the storm water runoff on the project site to the retention pond. This will minimize property damage through the prevention of flooding, erosion, and improper management of the water shed. The storm water collection starts along the property line adjacent to Coalburg Road and continues East until it reaches the retention pond. The topography of the land will be used to minimize the burial depths of the pipes in the system.

a. Determining Watershed Area

The drainage area calculations were performed by finding the watershed that will contribute storm water to the onsite retention pond. This was completed by using a topographic map of the area to determine the direction the storm water runoff will flow. The watershed area was determined using Google Earth Pro’s polygon tool and aerial photography as shown in the following map, Figure E.1. The light blue area shows the watershed area contributing to the dark blue retention pond.

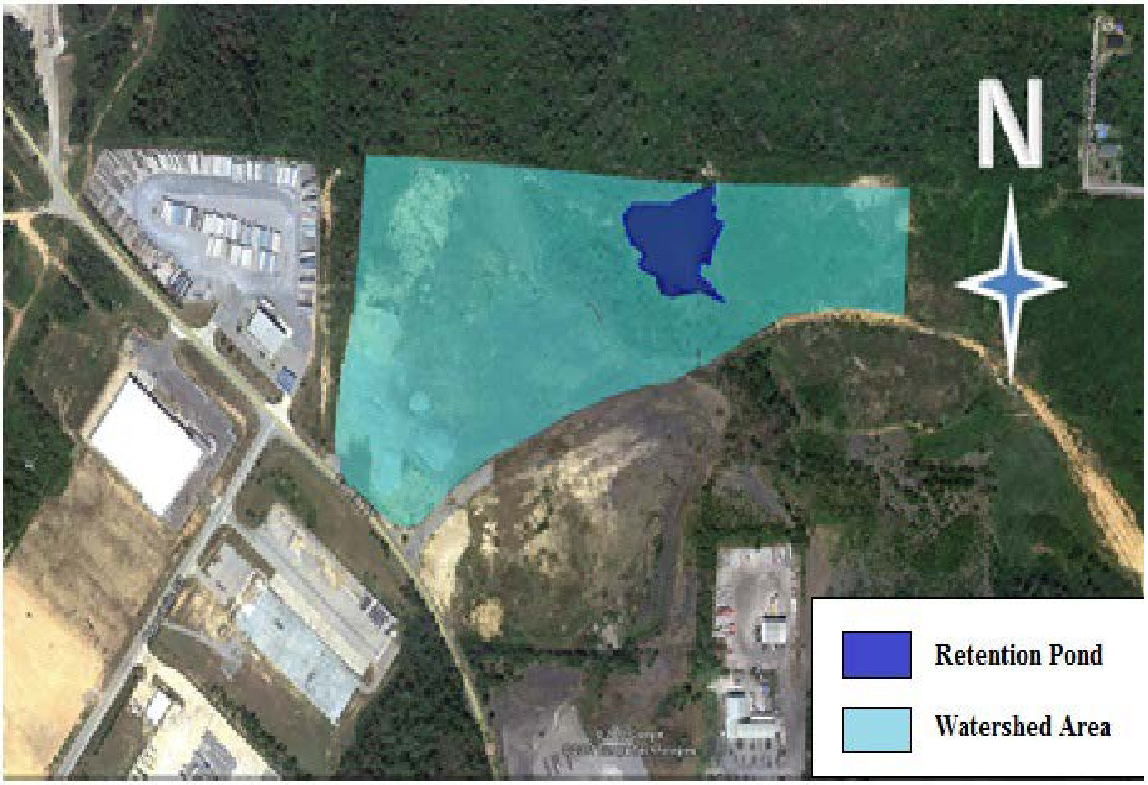

Figure E1: Total watershed area

The total watershed area draining into the onsite retention pond was found to be approximately 24.9 acres for the preconstruction topography. This area was then broken into subsections based on the preconstruction hydrological soil conditions which were observed using aerial photography and estimating the vegetation density of the land as well as using the provided geotechnical report to understand the properties of the soil. As shown in figure E.2, the green area shows where hydrological soil conditions were good, meaning a high vegetation density (more than 67% cover), and high infiltration rate of storm water into the soil. The light blue areas indicate where there are fair soil conditions, a moderate vegetation density (between 33% and 67% cover), and a moderate infiltration rate. The yellow areas indicate poor soil conditions, low vegetation (less than 33% cover), and low infiltration rates.

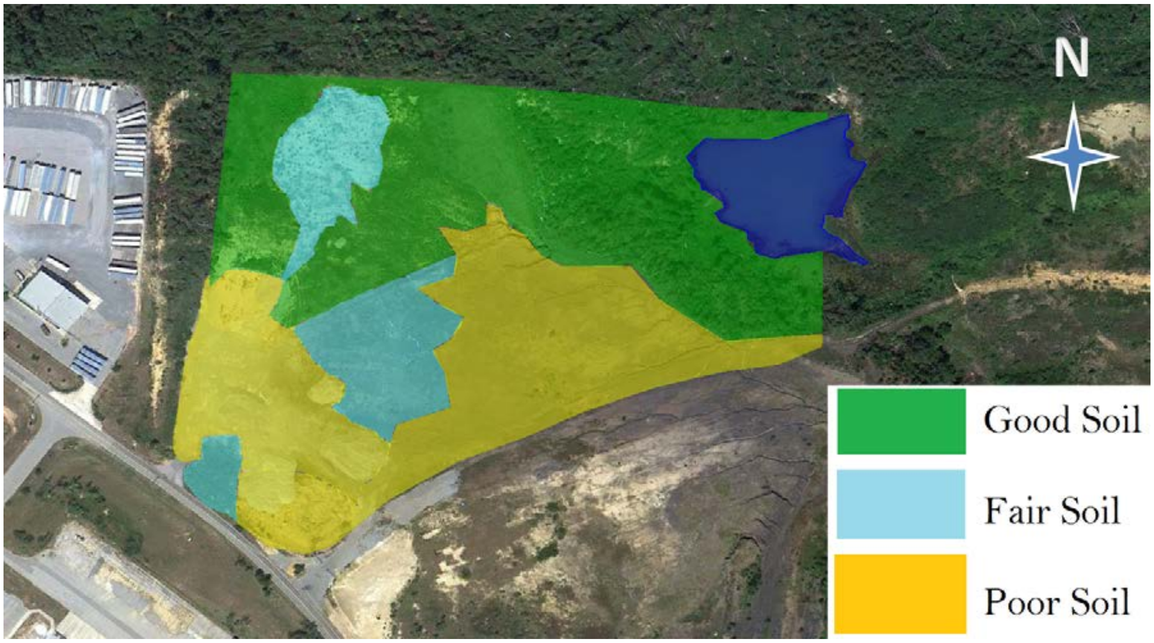

Figure E2: Preconstruction hydrological soil conditions

The preconstruction hydrological soil conditions were used along with rainfall data for Jefferson County from the Southeast Regional Climate Center to anticipate the storm water runoff volume by using the TR-55 NRCS curve number runoff method. The volume of storm water runoff was estimated for varying storm intensities as shown in the following table.

Table E1: Preconstruction storm water runoff volume for varying storm intensities

Storm Intensity (Years)	1 yr.	2 yr.	5 yr.	10 yr.	25 yr.	50 yr.	100 yr.
Jefferson County Max Precipitation (Inches)	3.5	4.1	5.3	6.1	6.9	7.6	8.4
Effective Precipitation (Inches)	1.6	2.1	3.1	3.8	4.5	5.2	5.9
Total Volume of Runoff (Acre-Feet)	3.2	4.2	6.3	7.7	9.2	10.5	12.1

In order to determine the post construction storm water runoff volume, the layout of the warehouse and parking lot design was integrated into our aerial photography hydrological soil condition map as shown in Figure E.2 below.

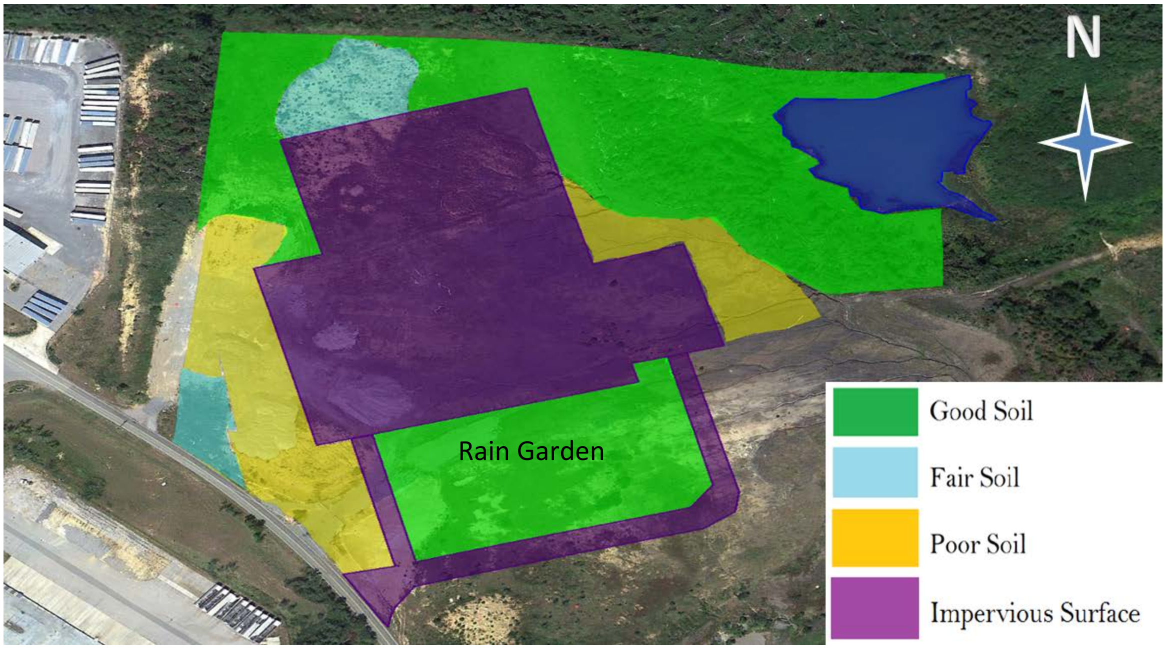

Figure E3: Post construction hydrological soil conditions

This new impervious surface area extended onto an area of the project site that was not draining into the onsite retention pond in preconstruction conditions, but any storm water from this area will be captured by the storm water system implemented into the construction project. To help counteract the additional storm water runoff from the impervious surface area, a large rain garden will be implemented in between the south side of the warehouse and the entrance road as labeled in Figure E.2. The post construction drainage area totals 27.4 acres versus the preconstruction drainage area 24.9 acres. The volume of the storm water runoff for varying storm intensities is summarized in the following table.

Table E2: Post construction storm water runoff volume for varying storm intensities

Storm Intensity	1 yr.	2 yr.	5 yr.	10 yr.	25 yr.	50 yr.	100 yr.
Jefferson County Max Precipitation (Inches)	3.5	4.1	5.3	6.1	6.9	7.6	8.4
Effective Precipitation (Inches)	2.0	2.6	3.7	4.4	5.2	5.8	6.7
Total Volume of Runoff (Acre-Feet)	4.2	5.4	7.8	9.5	11.2	12.7	14.5

b. Retention pond

The height of the onsite retention pond was determined by a field test of measuring the depth at the center of the pond. This depth was found to be 7.5 feet in the lowest rainfall conditions during the month of August. In order to determine the maximum depth of the pond at any given time, the spillway elevation on the topographical map was used with the known height of 7.5 feet for low flow conditions, the maximum depth was found to be 8 feet. In order to determine the surface area of the pond, Google Earth Pro's polygon tool was used to approximate the area. Using the volumetric formula for a trapezoid and by assuming a slope of 4:1 for the pond, the maximum volume of the onsite retention pond was determined to be 14.19 acre-feet. The change in storm water runoff volume in a 50 year storm between the pre and post construction conditions of the site draining into the retention pond was found to be 1.89 acre-feet. This change in runoff volume was added to the maximum retention pond volume to get a volume representative of the required design volume of the retention pond to retain the storm water runoff for a 50 year storm. The additional volume required for the retention pond to be designed for a 50 year storm was determined to be 16.08 acre-feet, corresponding to a volume increase of 1.89 acre-feet to the retention pond, equivalent to the additional storm water runoff volume due to the construction of the project. In order for the retention pond to increase to the required design volume, the height of the pond will need to be increased by 1.08 feet while maintaining the assumed 4:1 slope of the pond. A hydraulic dredging mechanism will be used as a cost effective way of increasing the height of the pond without having to pump all the water out for the pond prior to dredging. The cost associated with hydraulic dredging was determined to be approximately $10.05 per cubic yard, such that the cost of dredging the pond to the required volume of 16.08 acre-feet from 14.19 acre-feet will cost approximately $265,000. This cost is justified by meeting all the requirements of the NPDES permit associated with discharging into the Five Mile Creek watershed including the protection of endangered species in this watershed as required by the Environmental Protection Agency's Endangered Species Act by reducing particulate loads of the effluent from the property entering Five Mile Creek. The precise amount of particulate pollutant load reduction cannot be determined without extensive water sampling, but by designing for a 50 year storm an expected reduction load for any storm up to a 50 year storm can be expected to be at least 40%.

c. Storm Water Infrastructure

In order for the storm water runoff on the project site to be managed properly there will be infrastructure including underground pipes, drainage inlets, curb gutters, lined swales, and grass swales implemented on the project site. Underground pipes will follow the drainage inlets, and the curb gutters will lead to the inlets. The pipes will lead to lined swales near the retention pond in order to minimize erosion around the steep topography in that region of the site.

The storm water system was designed to direct the flow of storm water to the existing retention pond. Bentley's CivilStorm software was used to model and design the storm water system. The storm water system was designed to handle the peak intensity of a 25 year storm occurring at the duration of 5 minutes and representing an intensity of 8.4 inches per hour as shown below on the intensity duration frequency curve developed from NRCS Hydro-35 rainfall data for Jefferson County.

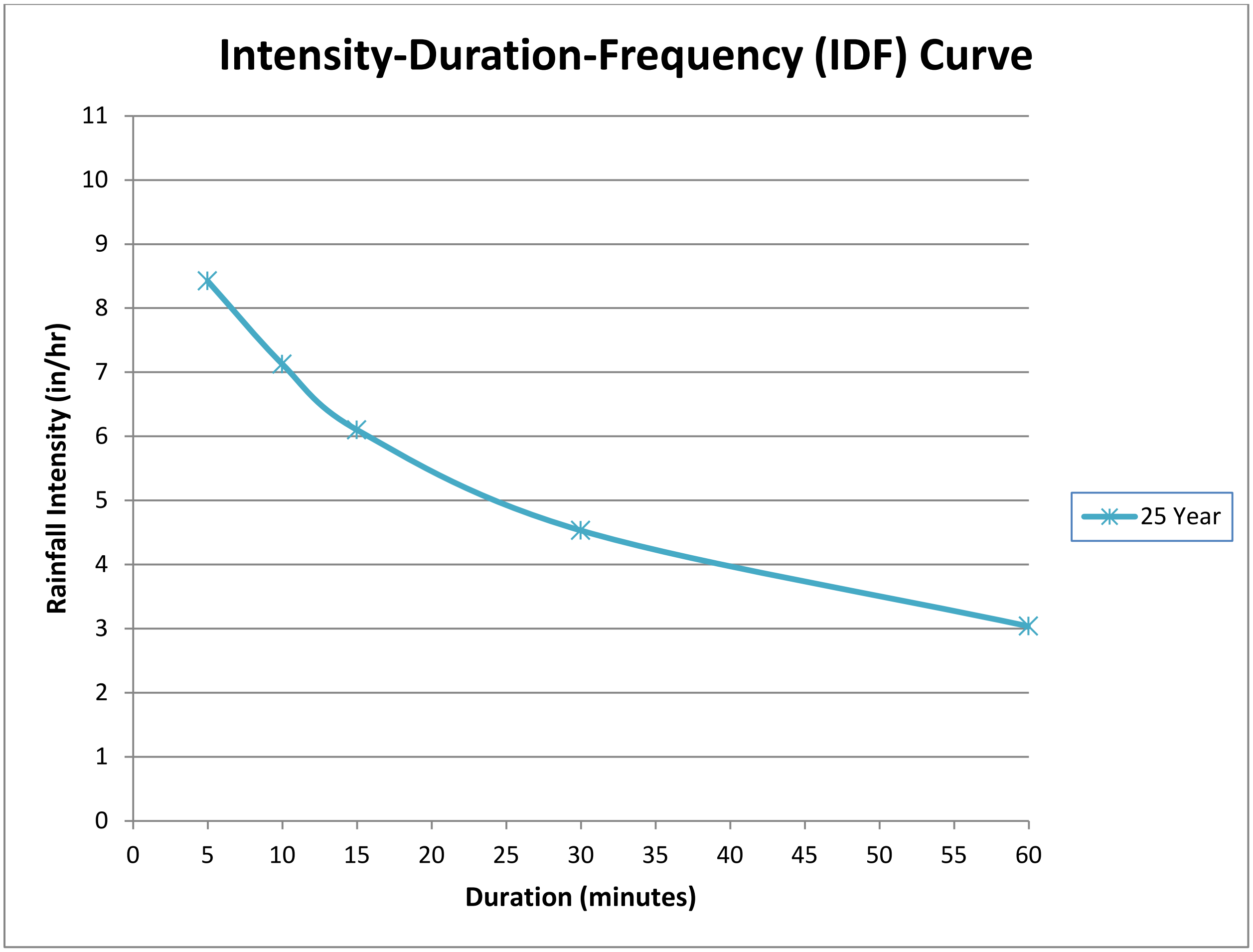

Figure E4: Intensity duration frequency curve

The storm water system for the warehouse building and surrounding area was designed as seen in figure E4. The direction of flow is indicated by black arrows, inlets are squares, outlets are triangles, the parking lot is peach colored, the roadways are cyan, the warehouse is purple, the retention pond is a greenish blue, grassed swales are denoted by green lines, lined swales are blue lines, and the storm water pipes are represented by red lines.

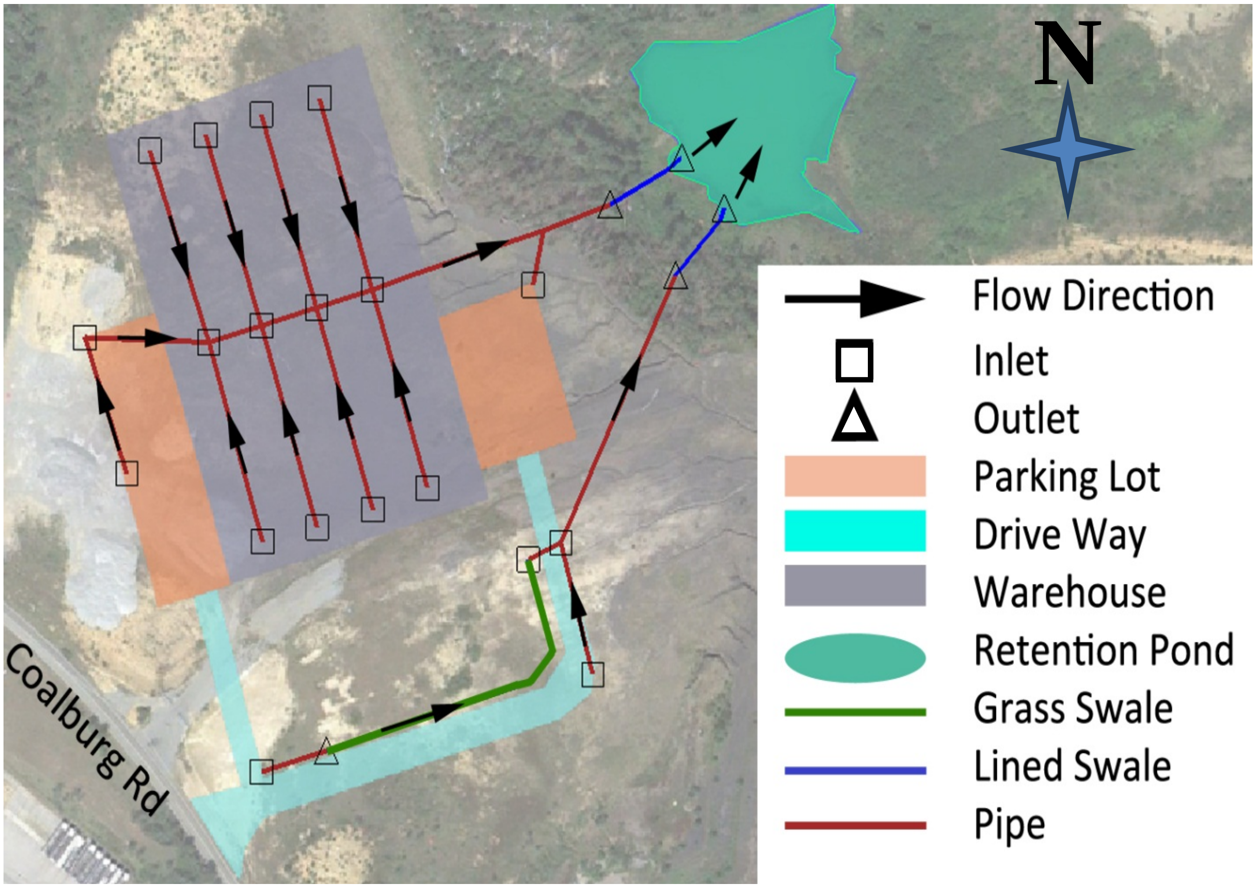

Figure E5: Storm water sewer layout

To the south of the building is an unused area of land that was utilized in this proposed design as a means of providing an area for an 80,000 square foot expansion space to the warehouse without affecting the infrastructure. An open channel grass swale, represented by the green line, will drain the storm water from this part of the site into a culvert under the road on the east side and then to a pipe leading to the existing retention pond.

The storm water from the roof will be drained into pipes running down the columns that are then connected to pipes underneath the warehouse slab. This system consists of one main pipe, running from west to east below the middle of the warehouse slab, and 8 auxiliary pipes branching off of the main pipe. The auxiliary pipelines, as shown labeled in figure E5 representing the underground piping system, carrying the water from the roof catchments will each measure 12 inches in diameter and be 267 feet long. The main pipeline will measure 15 inches in diameter and a total of 327 feet long. The direction of flow for the pipes is represented by the direction of the red arrows.

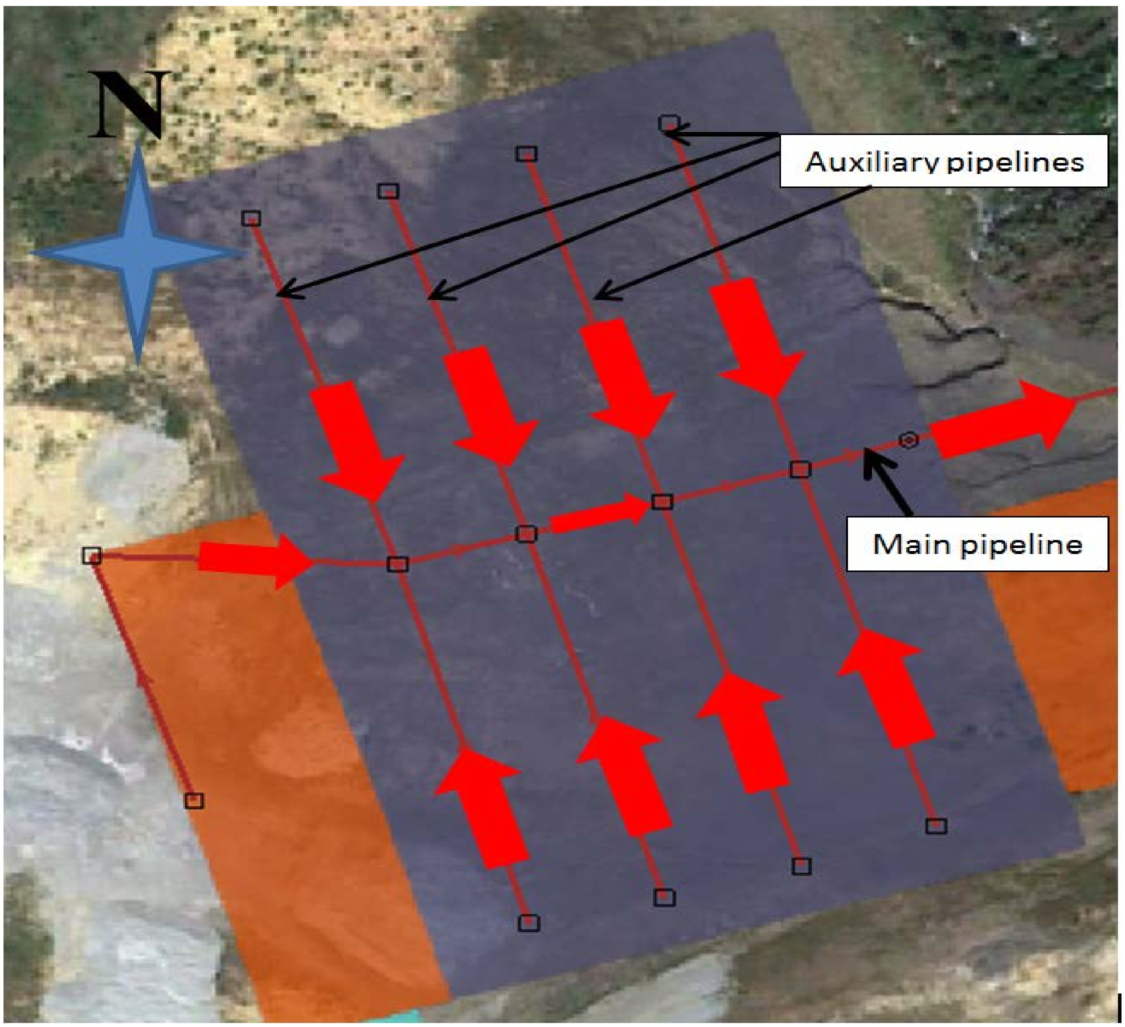

Figure E6: Storm sewer pipe layout under warehouse floor slab

To collect the storm water, the roof was sectioned into 12 80’x80’ square catchment areas. Each of these areas were designed to drain into a pipe in the center of the catchment, and then into the auxiliary pipe directly below it. From there, the water will travel to the main pipeline. This main pipe line has been designed to outfall into an open lined swale channel which will carry the storm water to the existing retention pond. The main employee parking lot, shown in figure E6 has been designed to have a downward slope to the northwest of 0.5%. This slope will cause the storm water from the employee parking lot to flow towards the northwest corner where there will be a catch basin that will guide the storm water into an underground pipe which will then guide the storm water to the main pipe line running below the warehouse slab and then finally to the existing retention pond.

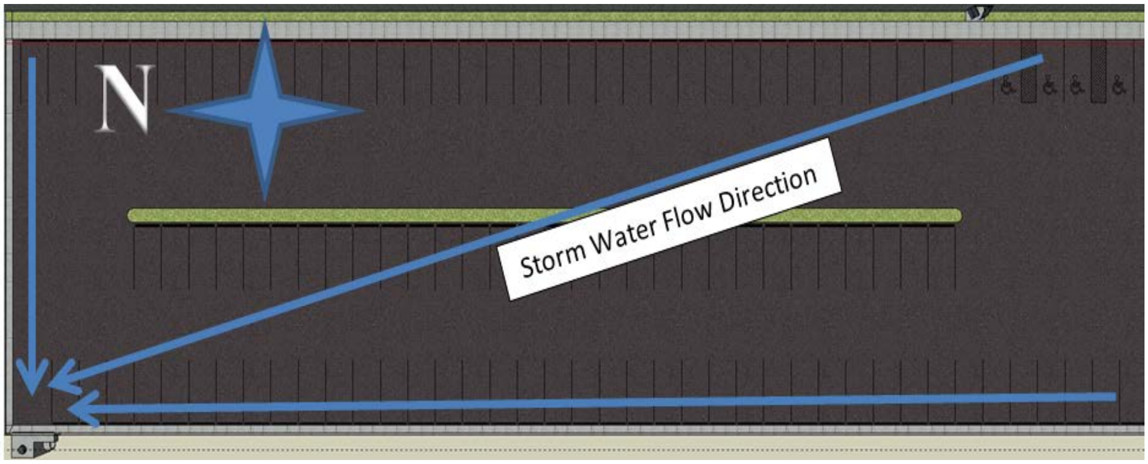

Figure E7: Employee parking lot storm water flow path

The loading bay truck pad, labeled in figure E7 was designed with a slope of 0.5% to the northwest. This slope will cause the storm water from this area to flow to the northwest corner of the truck pad. Here, a catchment basin was designed to capture the storm water from this area and carry it to a main pipe line that will guide the storm water to the retention pond.

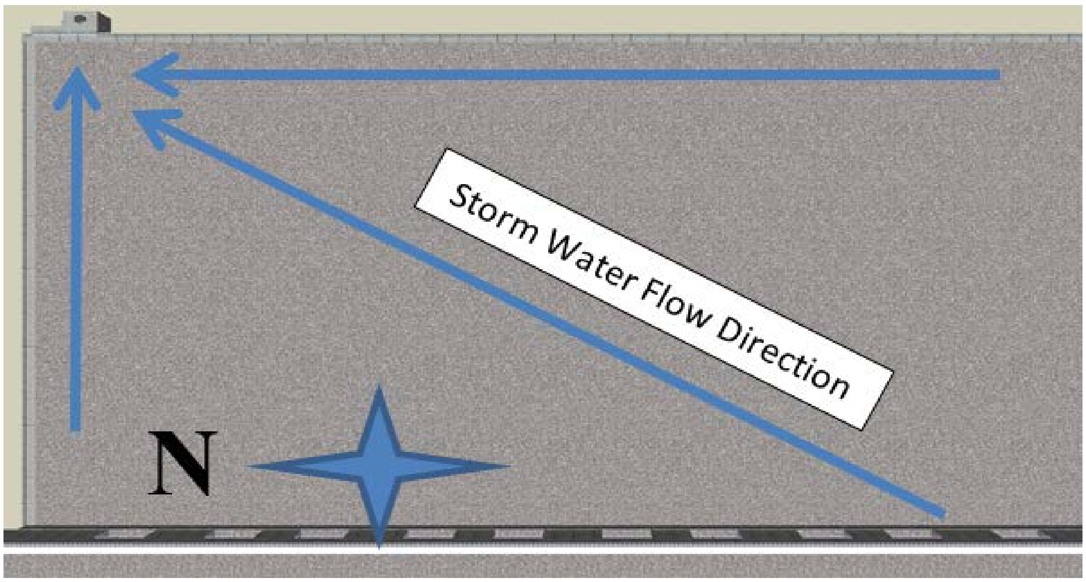

Figure E8: Truck docking bay storm water flow path

The main roadway for the trucks entering the facility was split into two main sections, seen in figure E8, these sections are labeled as “Southern Road” and “Eastern Road,” each draining into its own catch basin on the east side, as dictated by the topography of the site. For the eastern road, the catchment basin was placed at the center point of the road against the eastern curb. This catch basin will capture the water from the eastern road area north of the catchment until reaching a high point at the beginning of the truck loading parking lot, and then the stormwater will be conveyed through a pipe leading to an open channel concrete lined swale. For the southern part of the road, the water will flow towards the eastern side, where a catchment will then pipe the water to the catchment directly north of this catchment. The area of land designated for future development, will drain storm water into an open channel grass swale, outlined in green, that will guide the storm water through a culvert under the eastern road, into a pipe leading to a lined swale, and ultimately into the existing retention pond. The direction of water flow for this channel is represented by the black arrows This open channel grass swale has been designed to be 2 feet deep with a bottom width of 3 feet and a side slope (H:V) of 1.5. The roadway that will be subject to only passenger car traffic, labeled “Western Road” will drain to a catchment located at the south eastern corner of the western road, this catchment area begins at ta high point located where the employee parking lot begins. From here, the storm water will run through a 12 inch diameter pipe for 50 feet until it empties into the grass swale.

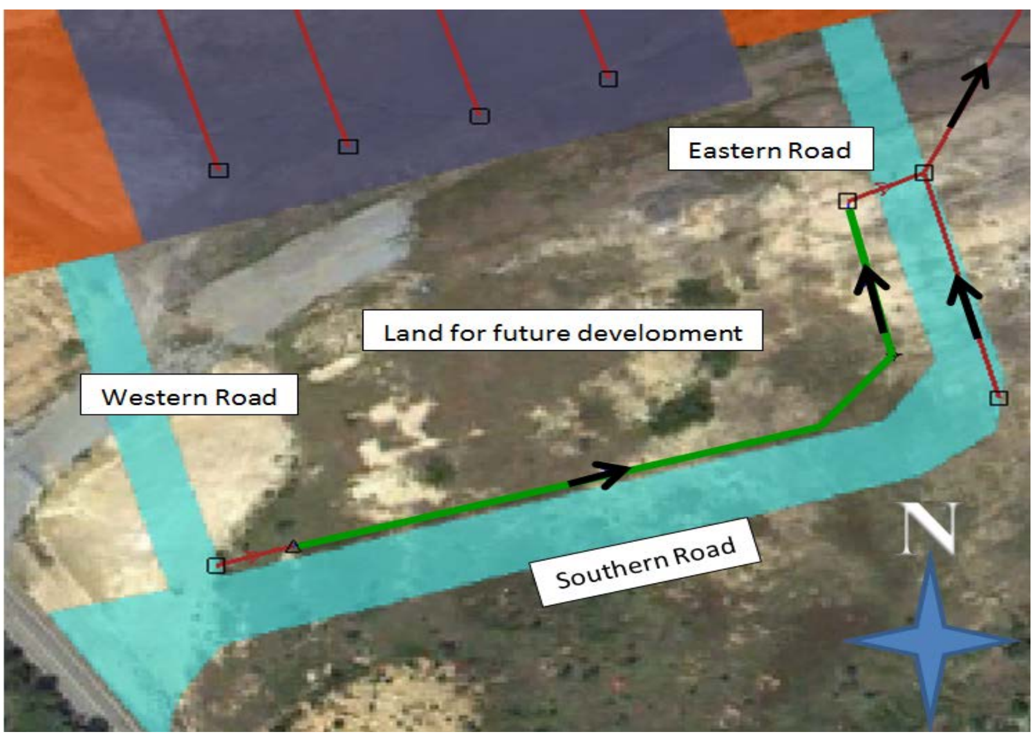

Figure E9: Driveway and expansion area storm water layout

The catchment areas for the storm sewer are broken up into 7 main types: the employee parking lot (broken into 2 equal catchment areas) on the west side of the warehouse, the roof (broken into 12 equal catchment areas), the expansion area land for future development (broken into 3 equal catchment areas), and the truck loading parking lot, east driveway, south driveway, and west driveway (each with one catchment area). The employee parking lot, roof, and expansion area are broken into multiple catchment areas, and for each of these equally subdivided areas there is a corresponding storm drain inlet, just as there is an inlet for the other catchment areas that are not subdivided (the driveways and truck loading parking lot). The NRCS TR-55 curve number tabular hydrograph method was used to determine the peak flow for each catchment area. Table E3 below, shows the data used to determine the flow for each catchment area.

Table E3: Storm water system main catchment area's peak flow rates and variables

	Employee Parking	**Truck Parking**	**Roof**	**Expansion Area**	**East Road**	**South Road**	**West Road**
Length to inlet (ft)	195	260	100	490	140	500	300
Length of grass swale (ft)	0	0	0	527	0	0	527
Length of concrete channel (ft)	120	120	120	120	120	120	120
Length of total pipe (ft)	841	193	768	450	584	584	500
Manning's n	0.011	0.011	0.011	0.17	0.011	0.011	0.011
Precipitation (in) (24hr 25 yr)	4.1	4.1	4.1	4.1	4.1	4.1	4.1
Slope (ft/ft)	0.005	0.005	0.005	0.005	0.005	0.005	0.005
V surface (ft/s)	1.22	1.22	1.22	1.18	1.22	1.22	1.22
Tc to inlet (hr)	0.053	0.067	0.031	0.115	0.041	0.114	0.075
Tt pipe(hr)	0.191	0.044	0.175	0.106	0.133	0.133	0.114
Tt concrete channel (hr)	0.005	0.005	0.005	0.040	0.005	0.005	0.005
Tt grass swale (hr)	0	0	0	0.124	0	0	0.124
ΣTt (hr)	0.249	0.115	0.210	0.386	0.178	0.251	0.317
Area (sq mi)	0.001	0.001	0.001	0.002	0	0.001	0
Curve number	98.0	98.0	98.0	74.0	98.0	98.0	98.0
Max retention (in)	0.204	0.204	0.204	3.514	0.204	0.204	0.204
Initial abstraction (in)	0.041	0.041	0.041	0.703	0.041	0.041	0.041
Runoff (in)	3.865	3.865	3.865	1.670	3.865	3.865	3.865
Area*Runoff (sq mi*in)	0.003	0.004	0.003	0.003	0.001	0.004	0.001
Initial abstraction/ Precipitation (in/in)	0.010	0.010	0.010	0.171	0.010	0.010	0.010
Peak Hydrograph times (hr)	12.45	12.32	12.41	12.56	12.38	12.45	12.52
Peak flow (cfs)	0.036	0.054	0.038	0.033	0.014	0.044	0.018

From the above table, the length to inlet is the furthest distance for each catchment area to its respective inlet. The time of concentration was determined for each catchment area using the following equations:

For a length to inlet less than or equal to 300 feet:

$T_c = \frac{0.007(nL)^{0.8}}{{P_2}^{0.5} S^{0.4}}$

Where,
T_c = travel time (hr)
n = Manning's roughness coefficient, determined from TR-55 table 3-1
L = length to inlet (ft)
P_2 = 25-year, 24-hour rainfall (in)
S = slope of hydraulic grade line (land slope, ft/ft)

For a length to inlet more than 300 feet:

$T_c = \frac{L}{3600V}$

Where,
T_c = travel time (hr)
V = Velocity (ft/s), velocity was determined from TR-55 table 3-2
L = length to inlet (ft)

The total length from the inlet of each catchment area to the outfall at the retention pond, was determined by summing the lengths of all pipes, grass swale open channels, or concrete open channels that the storm water was conveyed through to reach the retention pond for each catchment area. The travel time, Tt, for each catchment area through all pipes and open channels was also determined by using the same two Tc equations above. The sum of all the individual travel times was summed to get ΣTt.

The maximum retention for each catchment area was found using the following equation:

$S = \frac{1000}{CN} - 10$

Where,
S = maximum retention (in)
CN = curve number

The initial abstraction for each catchment area was found by multiplying the max retention by 0.2.

The runoff was found using the following equation:

$$Q = \frac{(P - I_a)^2}{(P - I_a) + S}$$

where

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

Then, using TR-55 Table Exhibit 5-III, the peak hydrograph times were found for each catchment area. Finally, the peak flow (cubic feet per second) for each catchment area was found by multiplying the area of each catchment (square miles) by the runoff (inches) and by the peak hydrograph time (hours).

Imputing the peak flow rate and time of concentration for each catchment area into CivilStorm allowed for an accurate simulation of the model, where then, the pipe sizes, slopes, and velocities could be optimized for the most cost effective storm water sewer system. This storm water system has been designed to maximize the use of the existing topography of the site area while optimizing pervious surfaces by implementing grass swales and landscaped areas in the employee parking lot with a 3 foot wide grassed area along the face of the warehouse and a 4 foot wide grassed area in the middle of the parking lot. All pipes are gravity fed and have a subcritical flow. The velocities in pipes and channels range from 2 ft/s to 10 ft/s as this range will prevent buildup or scouring or sediment in the pipes and channels.

d. NPDES Permitting

The project will require a general National Pollutant Discharge Elimination System (NPDES) permit to be issued by the Alabama Department of Environmental Management (ADEM) in order for the site to legally discharge storm water into the surrounding watershed and be in compliance with state laws and the Environmental Protection Agency’s Clean Water Act. In order to obtain the NPDES permit, a Notice of Intent (NOI) must be submitted. A Construction Best Management Practices Plan (CNMPP) is not required with the NOI because the project site is not considered a priority construction site due to the fact that the surrounding watershed, Five Mile Creek, currently is not on the state’s 303(d) for turbidity, siltation, or sedimentation, nor does it have a published Total Maximum Daily Load for any water pollution criteria. The NOI will include a general description of the construction activity, the latitude and longitude to the nearest second of the entrance to the construction site and each point of discharge, the water bodies receiving discharges from the project site, a portion or copy of a U.S. Geological Survey map showing the site location, and a contact person, address, and phone number for the site to be covered under the general NPDES permit.

e. CBMPP

The construction best management practice plan will ensure effective erosion and sediment control appropriate for the project site conditions. The CBMPP will be designed to control storm water at peak flow rate, volume, and velocity to minimize soil erosion with the use of erosion control blankets where there is high erosion potential such as steep slopes, in order to stabilize the ground, slow erosion, and facilitate vegetative growth. An exit pad made up of a 6” layer of course aggregate will be placed at the construction site entrance in order to remove mud from the tires of any vehicles leaving the construction site and prevent increased sediment runoff into the municipal storm drain system. The pad will be 50’ long and 50’ wide, as suggested by the Field Guide for Erosion and Sediment Control on Construction Sites in Alabama. Sediment barriers fences will be placed along the toes of slopes and in the drainage ways to reduce sediment runoff during the construction phase of the project and will total 1,100 feet of silt fencing. The excavation area, silt fencing, and gravel exit pad placement are shown in the following figure.

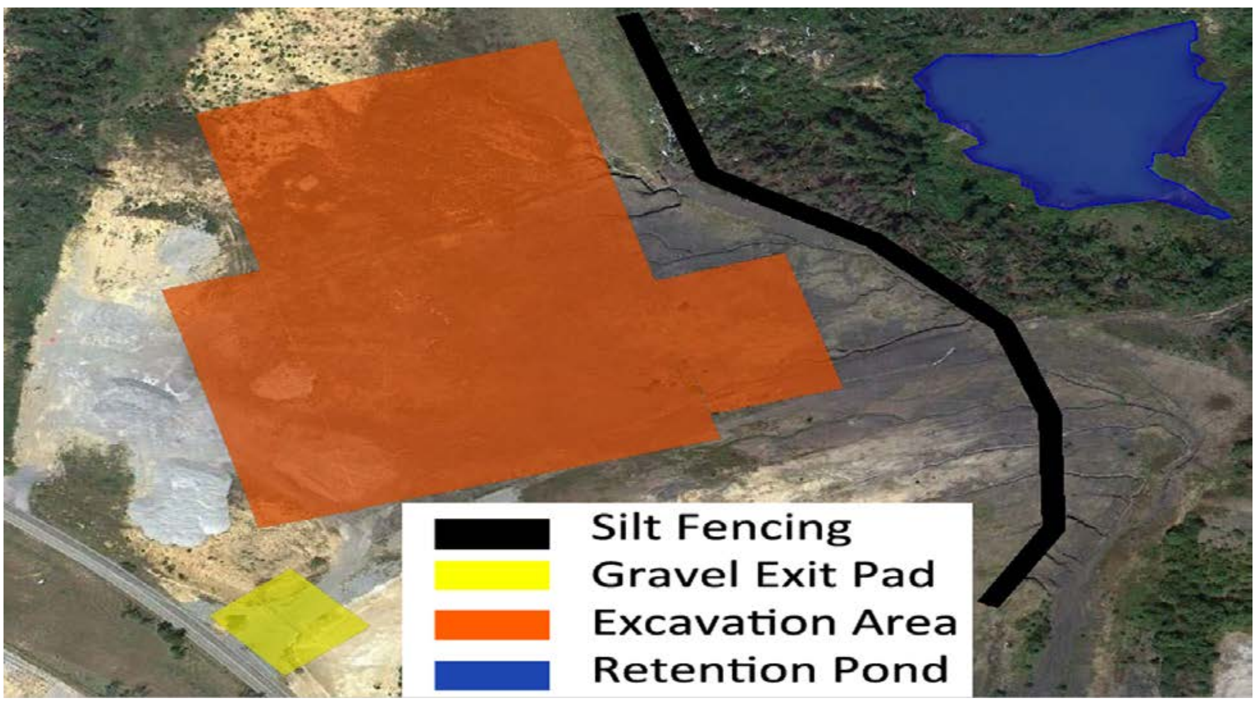

Figure E10: Best management practices placement

Sanitary Sewer

The potable water and sanitary sewer system will be laid out as shown in Figure E11.

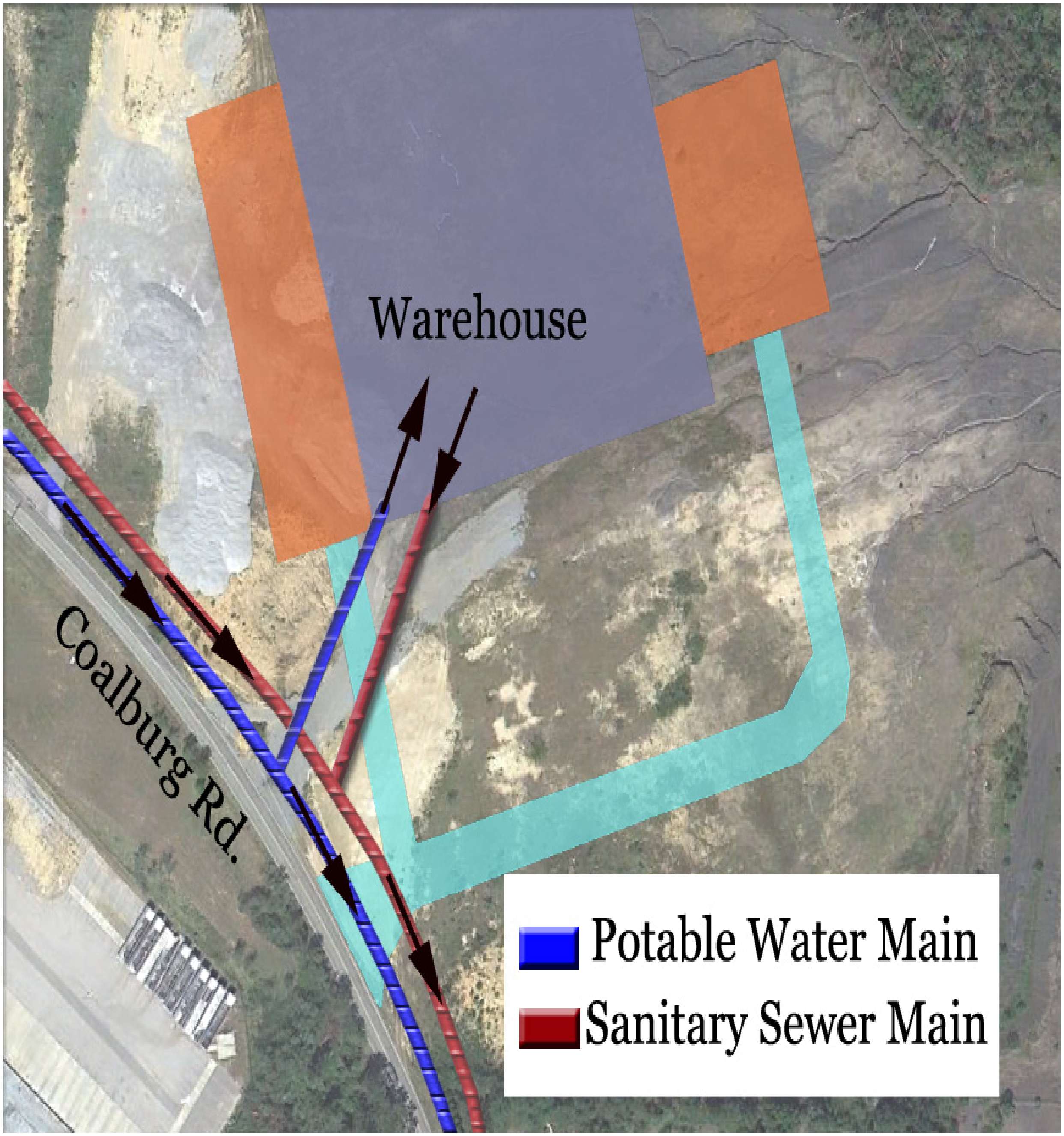

Figure E11: Potable Water and Sanitary Sewer System

For the sanitary sewer system, Bentley SewerGEMS Sanitary V8i was used for the primary design. The sanitary sewer system will run at a downwards slope from the proposed warehouse to an existing main running alongside Coalburg Road. The existing main consists of a 24” diameter ductile iron sewer pipe located approximately 18’ below the ground surface elevation of Coalburg Road, according to the Birmingham Water Works Board. The proposed building main will run from the south-west corner of the building to this existing main and will consist of a 260’ long, 6” PVC pipe. As shown below, Figure E11 shows the layout and conduits used in the analysis of this proposed sewer system. In this figure, conduits are abbreviated with CO-xx and manholes are abbreviated with MH-xx.

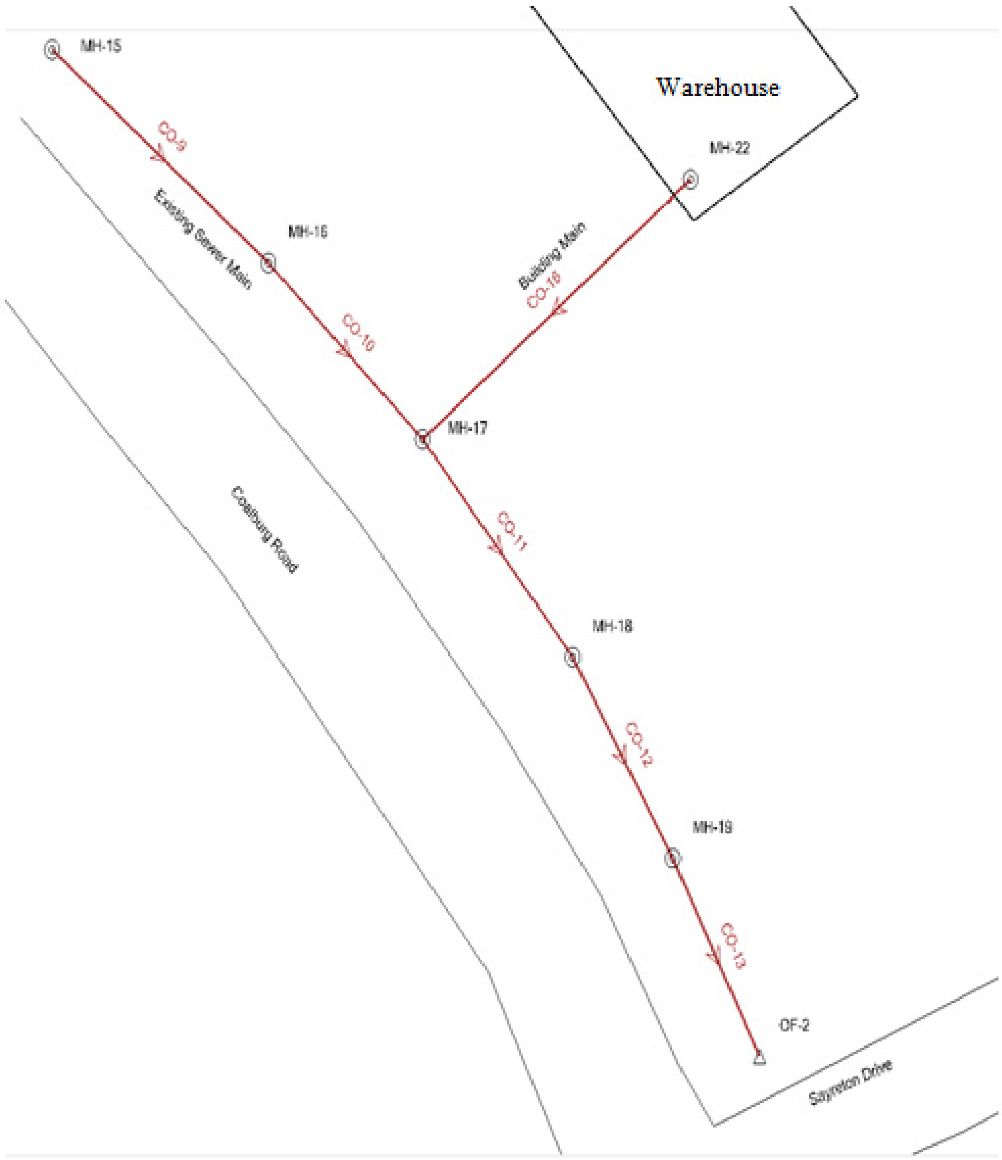

Figure E12: Sanitary Sewer Conduits and Manholes

The sanitary sewer system is gravity fed and is designed according to the ADPH specifications for a warehouse/office area with a flow rate of 15 gpd/person/8 hr shift and because the building will have 110 employees, the design flow rate is 1650 gpd. The sewer system is designed with a 6” pipe to meet velocity specifications between 2-10 ft/s under all perceivable discharge loading conditions. According to the design scenario run in SewerGEMS, assuming these flows, the system pipes will have a flow and velocity as shown in Table E4.

Table E4: System Flow and Velocities

Label	Flow (maximum, cfs)	Velocity (maximum calculated, ft/s)
CO-16	0.02	3.14
CO-9	1.00	4.71
CO-10	1.00	6.00
CO-11	1.02	6.98
CO-12	1.02	5.42
C0-13	1.02	3.58

Potable Water System

The potable water system was designed using Bentley WaterGEMs. This system will be designed with a layout similar to the sanitary sewer system. However, the potable water system will run parallel to the sanitary sewer system with a horizontal separation of 5 feet and vertical separation of 4 feet above the sanitary sewer system. This will ensure that in the case of a rupture in the sanitary sewer system, the potable water will not become contaminated. The figure below illustrates the potable water system.

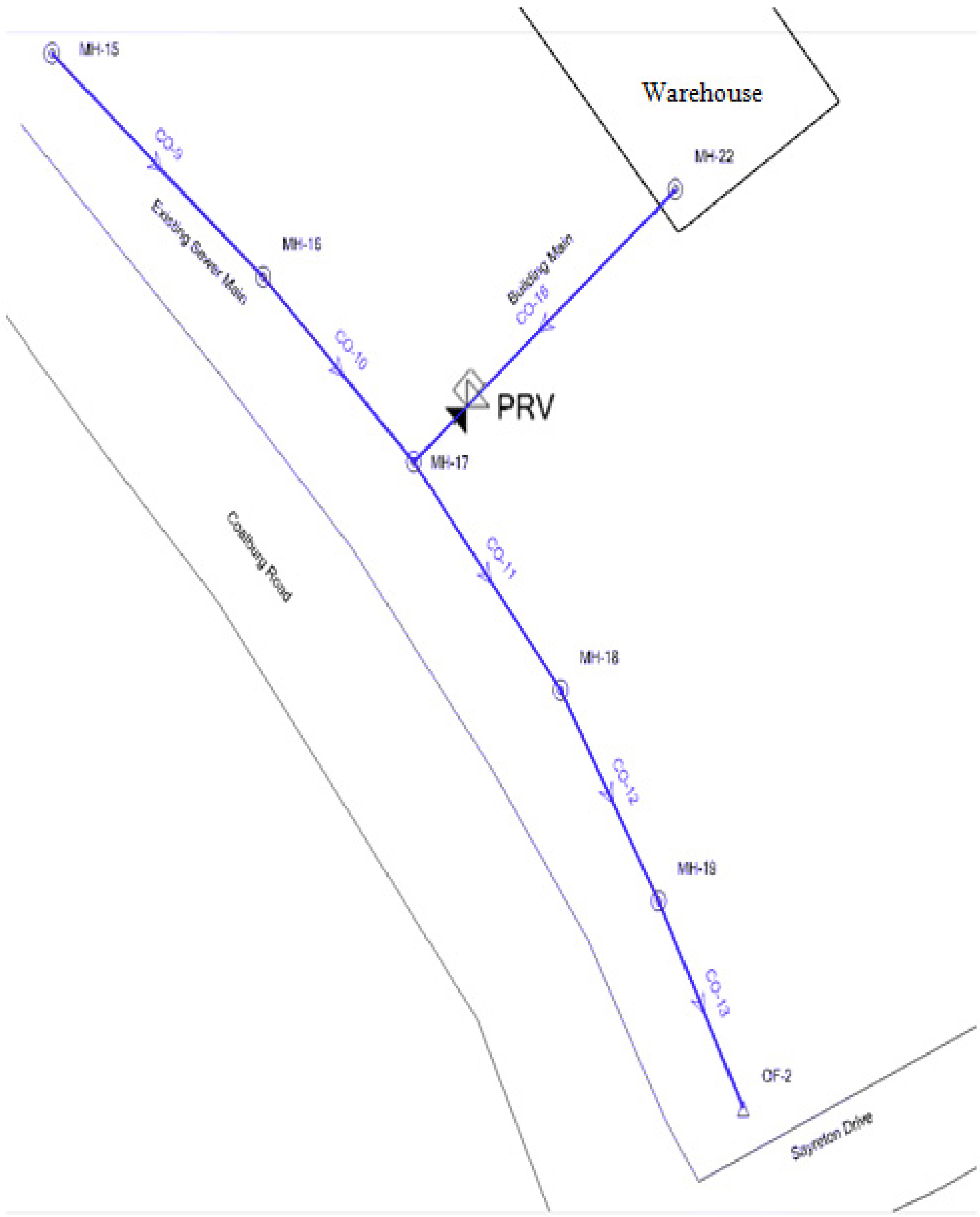

Figure E13: Potable water system

The table below shows the designed parameters of the potable water system.

Table E5: Potable water system properties

Label	Length (Scaled) (ft)	Diameter (in)	Material	Flow (gpm)	Velocity (ft/s)	Headloss Gradient (ft/ft)	Pressure (psi)
Building Main	260	6	PVC	1,750	19.86	0.16	26.5

The potable water system will be designed to maintain 20 psi of pressure with a flow 1750 gallons per minute such that it meets the fire safety pressure requirements for a construction type 1B with an occupancy type S-2 low hazard storage warehouse with a sprinkler system, as required by the International Fire Code. The fire safety pressure and flow requirements are greater than what will be required by the warehouse personnel. The existing water main along Coalburg Road, that the potable water system will be attached to, has a pressure of about 76-80 psi. The potable water system consists of a 260’ length, 6” diameter, PVC pipe; and it maintains a pressure of 26.5 psi and a velocity of 19.8 feet per second. In order to ensure the pressure does not get too high in the building main pipe, a 6” pressure reduction valve will be installed on the building main pipe at a location near the existing water main on Coalburg Road.

Sustainability

Sustainability will be considered in the design of the warehouse. This will be done to ensure the most sustainable practices are implemented to protect the environment and enable the most cost effective design. One of the sustainability aspects of the project that will be considered is maximizing the areas of pervious surfaces. This will be done to reduce storm water runoff and ultimately reduce the amount of water runoff entering the retention pond in the north east corner of the site. Rain gardens will also be implemented into the design of the parking lots and pavement areas, which will further reduce the amount of storm water runoff. By reducing the amount of storm water runoff, the overall cost of the storm water system and retention pond design will be reduced as well.

Storm water reuse will be implemented around the building pad. This will work by collecting the water from the roof top storm water runoff using cisterns. The collected water will then be recycled and used in irrigation at the site. By collecting and reusing the storm water in this manner, the overall cost of irrigating the vegetated areas at the site will be reduced. The collected storm water can also be used to help wash trucks, the exterior of the building, or any impervious surfaces on site.

The building materials used in the construction of the facility will be bought from local vendors as much as feasibly possible. By doing this, transportation costs for materials, greenhouse gas emissions, and the overall total carbon footprint of the construction process will be reduced. Renewable energy sources will be considered in the operation of the warehouse. One of the major renewable energy sources this warehouse will attempt to utilize will be day lighting. By the use of passive lighting through windows in the upper portions of the walls, energy costs can be reduced. This will also provide natural lighting to the interior of the warehouse, providing a better, more comfortable work environment for the employees that will be working there.

Hazardous Waste

According to the Phase I Site Assessment prepared for the project site, there are no hazardous waste concerns at the site. Therefore no remediation procedures will be required for the construction process of the site. However, in the case of a truck fire or spill an emergency contingency plan should be in place and ready to utilize in case of such event occurring onsite.

Air, Noise, and Light Pollution

Airborne particulate matter from the site during the construction phase will be minimized using dust control management practices. These practices will include maintaining a moist ground surface, optimizing vegetative cover, and minimizing the amount of disturbed area at any one time. According to EPA National Ambient Air Quality Standards (NAAQS), the state of Alabama is in nonattainment for particulate matter; therefore, reducing the air pollutant is of priority and imposes a negligible cost. Noise pollution from the site can be reduced if necessary by delaying early morning truck traffic until 8 or 9 a.m. to reduce nearby residential areas from being disturbed by noise pollution. Light pollution from the warehouse can be reduced if necessary by implementing timers and/or motion sensors on sections of lighting systems during non-operational night-time hours.

Conclusions

In concluding the environmental portion of this report, designs were made for the storm water sewer, sanitary sewer, and potable water systems. The optimization of impervious surfaces, as well as other sustainable practices, was implemented into the project to give it a low impact design characteristic while also saving money on the project costs. Pollution concerns including water, air, noise, and light pollution were addressed and contingency plans implemented in the event of a worst case scenario by implementing best management practices into the construction and operation phases of the warehouse project. The cost of all environmental designs is discussed in full detail in the construction management portion of the project.

8. Construction Management

Overview

The two primary objectives of the construction management team were cost and project scheduling. In analyzing these two objectives our team has constructed a detailed project cost, identifying daily output and the necessary crews to complete each task. Along with the project cost, a detailed project schedule was created in order to provide the general contractor a visual sequence blueprint to the construction process. The final cost of the project was calculated at $11.9 million with an approximate construction time of 17 months. For the project schedule the start date was set at January 1, although this date could be changed to allow for circumstances such as better weather conditions for the concrete portion of the project or other issues that may arise.

Project Scheduling

The scope of the project has been broken down into six major phases allowing the contractor to set benchmarks for the project. Each phase has a start date along with the duration of each phase. As shown below in table CM1, phases two and three of the project have the longest duration of the project accounting for 69% of the construction time. Figure CM1 below compares the duration of each phase with the overall project schedule.

Table CM1: Phase Duration

Phase	Description	Start Date	Duration (days)
1	Pre-Construction	1/1/2011	35
2	Earthworks	2/6/2012	155
3	Structural	9/10/2012	120
4	Exterior Improvements	2/5/2013	13
5	Openings/Finishes	12/17/2012	72
6	Finalize Project/Occupancy	4/26/2013	13

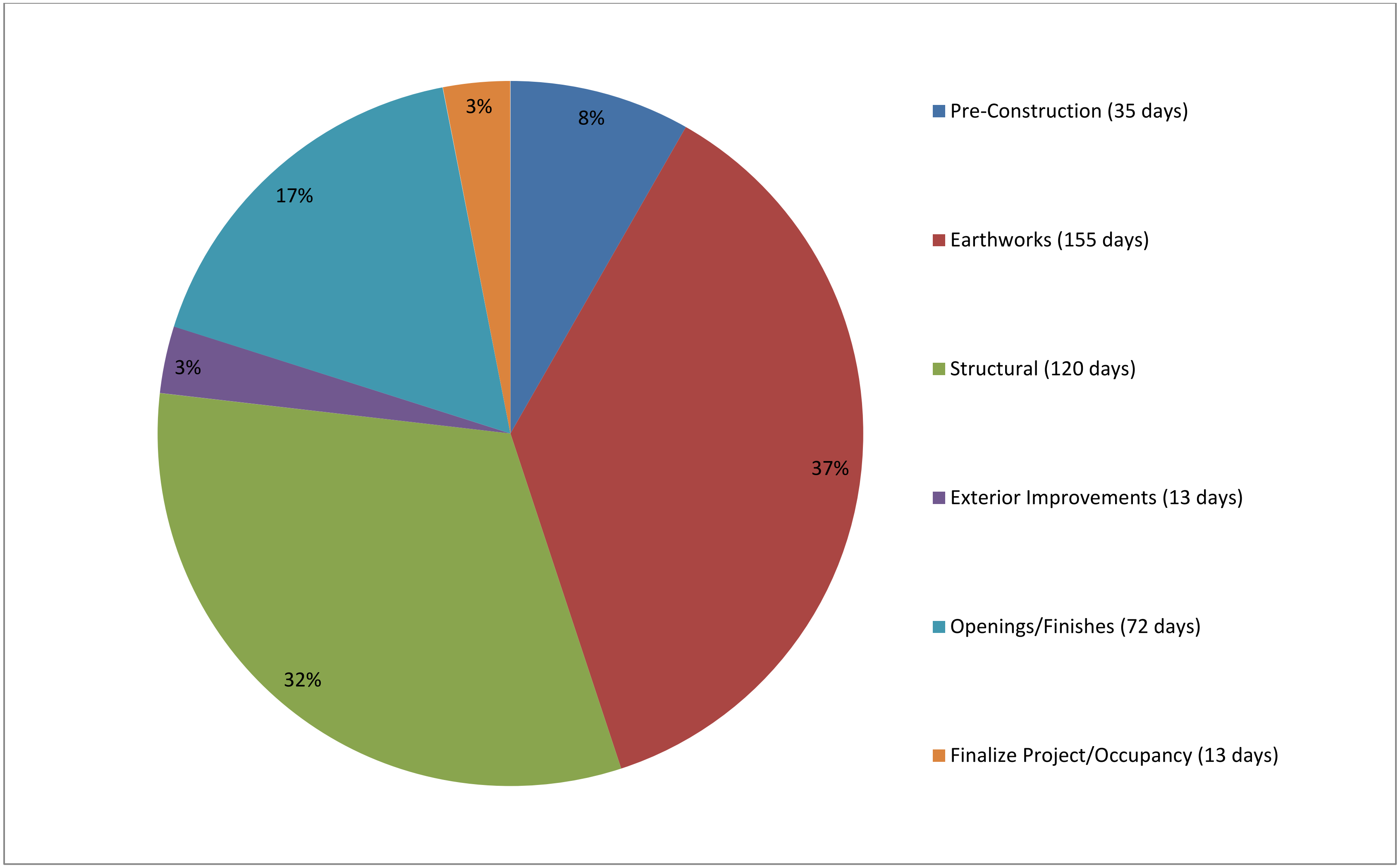

Figure CM1: Project Time Analysis

Figure CM2: Gantt Project Schedule

a. Phase 1- Preconstruction

The preconstruction phase of the project can be considered one of the most important parts of the construction process. After the owner agrees to a conceptual plan, all necessary permits must be obtained before the construction can begin. The critical permit to obtain is the National Pollutant Discharge Elimination System (NPDES), governed by the Alabama Department of Environmental Management (ADEM). This permit takes approximately 30 days to obtain with a $645.00 fee administered. The remaining permits are administered by the City of Birmingham Department of Planning, Engineering and Permitting. Also required by other government entities are fees required for water and sewer. All permits and regulations are shown in table CM2 shown below.

Table CM2: Permits & Fees

Administrator	Permit Name	Time (days)	Requirements	Fee structure	Cost
Alabama Department of Environmental Management (ADEM)	National Pollutant Discharge Elimination System(NPDES)	30 days	BMPP	$645/5 yr.	$645
City of Birmingham Dept. of Planning, Engineering & Permitting	Civil Construction Permit	10	2 sets of plans	$100.00	$100
	Soil Erosion & Sediment Control	10	2 sets of plans	$400	$400
	Driveway or Sidewalk Construction Permit	10	2 sets of plans	$100	$100
	Building Permit	10	4 sets of plans	$8.5/1000 of construction cost	$89,947
Jefferson County Office of Sewer Service	Sewer Impact Fee	2		150 tap/35 connect (225/fixture)	$3,560
Birmingham Water Works Board	Water Tap	3		Development charge, Main Fee ($16/ ft. of property frontage), Tapping Fee	$11,690
Total Permits					$106,442

While waiting on permits to be issued, the general contractor should be procuring all necessary labor, material and equipment. Obtaining all necessary resources at the correct times will allow the project to have a smooth flow, minimizing any delays that could be incurred. Table CM3, included in the appendix illustrates the crews, materials and equipment needed in each phase of the project. The total duration for phase 1 is approximately 35 days.

b. Phase 2: Earthworks

The Earthwork phase has the longest duration of the project schedule. This can be attributed to the soil conditions of the construction site. The project site contains mine deposits which require compaction in order to be suitable for a stable footprint. The geo-technical report for the site, provided by the owner, had disclosed unsuitable soil for a structure to be built upon without treatment. The report provided an option of two treatment processes for the soil: (1) excavate the buildings footprint at a depth of 14 foot, bring the soil back in one foot increments and compact it using moisture and a sheepsfoot roller or (2) dynamic compaction, which is completed by dropping a weight with a crane on the building site. After the weight is dropped over the entire area needing to be treated, four foot of surcharge soil is brought in to further compact the soil for a period of two months. After the surcharge is removed, the soil is then tested again for stability and deemed ready for construction. After analyzing the two methods, it was determined to use dynamic compaction, saving the customer approximately $640,000. The calculations of these two methods are shown in tables CM3 and CM4.

Table CM3: Soil Treatment with Cut & Fill

Description	Volume	Cost/BCY	Column1
Excavate &Haul Away	132741	$3.89	$516,361
Excavate & Return Haul	132741	$3.89	$516,361
Compaction of Return Soil	132741	$3.20	$424,770
Total			$1,457,493

Table CM4: Soil Treatment with Dynamic Compaction

Description	Volume	Cost/Unit	Cost
Dynamic Compaction(S.F.)	256000	1.75	$448,000
Surcharge	39481	3.89	$153,642
Grade Surcharge	39481	1.59	$62,776
Remove Surcharge	39481	3.89	$153,642
Total Cost			$818,060

After the soil is treated, excavation of the footings, storm water drainage and underground utilities can begin. Using the blueprint provided by the environmental team, trenches are excavated, followed by placing of all class 5 concrete pipe and junction boxes for the drainage of the warehouse building. After the storm water system and utility lines are filled and compacted, all continuous strip footings and column footings are excavated. Upon completion of all slab excavation, exterior earthworks such as storm water swales and retention pond dredging can begin. The total duration for phase two is 155 working days, with 50 days being attributed to the soil surcharge remaining on the building pad.

c. Phase 3: Structural

The construction management team has broken down phase three into two segments, concrete and steel. This phase is primarily the erection of the warehouse and office space. Information provided by the structural team was used to determine the sequence of this phase. The remainder of this phase will describe how this information was used in the scheduling of the structural phase.

The concrete segment of phase three consists of forming, reinforcing and placement of the concrete during the project. After the footings are excavated (Earthworks), reinforcement is placed in the footings per specified by Geo-Tech team. After reinforcement is completed, concrete is placed in the continuous and spread footings. Upon completion of the footings, column area must be formed for an isolator joint, preventing cracking of the slab caused by stresses on the column and footing below the slab.

After the footings are completed, work can begin on the building slab. One example of the coordination with the structural team is the construction of the building slab. The structural team specified 80 foot construction joints in the north-south direction. Using the *RSMeans Building Construction Cost Data* as a reference, two crews could construct 370 yd^3 of a slab on grade per day. Using this estimate it was determined to construct the slab in 80’ x 128’ (348 yd^3) sections per day. This would allow one 80’ wide N-S strip to be completed per week, and the slab to be constructed in five weeks.

Upon the slab curing for approximately three weeks, construction of the tilt walls can be started. The surface area of the slab is utilized as the bottom surface of the wall panel forms. The 104 wall panels will be constructed at a rate of ten per day, allowing the panels to be completed in eleven days. After the walls cure for seven days, they can be erected at a rate of 32 panels per day, allowing the walls to be placed on the footing in 4 days. The second floor, elevated slab, of the office should be constructed before the panels are erected, allowing the concrete to be placed from outside the building with a pump. The steel segment includes the erection of the columns, joist girders, joists and the steel roof decking system. The columns must be placed and secured prior to concrete being placed inside the isolator joint due to the anchor bolts being below slab grade. After the concrete cures, girders and joists can be erected. Due to the lengthy duration of the roof system, the construction management team suggests having two decking and EPDM crews along with four insulation crews allowing phase three to be completed in the allotted 120 days.

d. Phase 4: Exterior Improvements

Phase 4 includes paving, curbing and landscaping. Flexible paving was chosen for the employee parking and driveways due to the economic advantage. Rigid paving, or concrete, was chosen for the truck staging area as a result of a concentrated point load on the surface for an extended time. The duration of phase four is approximately thirteen days. Tables CM5 and CM6, shown below illustrate cost and time durations of the flexible paving portion of phase four, followed by table CM7 illustrating the rigid paving of the staging area. The construction management team has chosen to complete phase four after the building has been erected, minimizing damage to the paving surfaces due to heavy construction traffic.

Table CM5: Flexible Paving-Parking

		Cars Parking			**Truck Parking-Staging**	
	Dense Grade	Binder Course	Wearing Course	Dense Grade	Binder Course	Wearing Course
Length	374	374	374	140	140	140
Width	105	105	374	255	255	255
Surface Area(yd^2)	4363	4363	4363	3967	3967	3967
Thickness(inch)	6	3	1	10	4.0	2
Output/day(yd^2/day)	5000	4905	10575	4600	4140	6345
Time(day)	1	1	1	1	1	1
Cost/(yd^2)	9.4	13	5	15.0	17	10
Cost	$41,015	$57,160	$20,115	$59540	$68028	$39270
Total Cost						$285,128

Table CM6: Flexible Paving-Roadways

Trucks(Driveways)		
Dense Grade	Binder Course	Wearing Course
45	25	
840	256	
4911	4911	4911
10	4.0	2
4600	4140	6345
1.1	1.2	0.8
15.0	17	10
$73,716	$84,226	$48,620
Total Cost		$206,561

Table CM7: Rigid Paving –Truck Staging

Truck Dock-Rigid Parking	
Length (ft.)	250
Width (ft.)	50
Surface Area (yd^2)	1389
Thickness(inch)	8
Output(yd^2/day)	1375
Time(days)	1
Cost/(yd^2)	38
Cost	$52,778

e. Phase 5: Openings and Interior Finishes

Phase five includes all enclosures including doors and windows. Interior finishes includes all plumbing, mechanical and electrical (PME) components as well as the infrastructure of the office. Because the PME components were outside the scope of the project, an estimated duration was used in the schedule. Six inch metal studs as well as ½” fire resistant Gypsum board was selected in the office structure, providing protection for the building at a small difference of only fifty cents per square foot. Phase 5 was estimated to be completed in 72 days but may have to adjusted, due to PME being outside the scope of OCD.

f. Phase 6: Finalize Project

Although the project should have a routine cleanup schedule, a thorough construction site cleanup should be performed upon completion of the first five phases. After the construction site has been cleaned, the owner and architect should make an inspection of the project. After the inspection, the owner and architect should provide a punch list to be completed before occupancy occurs. The contractor then will have approximately two weeks to resolve any issues on the punch list. After the owner makes a final inspection, the contractor will turn the keys over to the owner for occupancy.

Project Cost

The construction management team has taken into consideration cost effective measures to be implemented in this project. Prices were obtained from many vendors as a comparison to RSMeans values. One such case is the price of concrete; prices were obtained from Sherman Concrete as well as Ready Mix USA. Andy Blake at Sherman quoted a price of $67.40 per cubic yard (3000psi), while USA quoted a price of $73 per cubic yard. RSMeans has a published price of $99.00 per cubic yard. These savings may be attributed to the sluggish local economy, causing local companies to reduce prices in order to compete for projects. It is recommended by this team to check with all local vendors for competitive bid prices before purchasing materials for the project.

The project cost has been broken down into divisions using the CSI Masterformat method. As one could expect, due to their lengthy duration of phases two and three of the project schedule, divisions three, five and thirty one are some of the most expensive. The total project cost is calculated at approximately 11.9 million dollars. These prices are based on current values provided by RSMeans. The expected project date is in the year 2016; taking this into account, inflation must be considered for the future project. Using an inflation rate of 5%, the projected cost will be approximately 14.5 million dollars. The project cost, divided into the primary 13 divisions is shown in table CM7 below. Division one includes approximately 1 million dollars in contingencies due to some of the project being outside OCD’s scope of project work.

Table CM7: Division Project Cost

Division	Description	Cost
1	General Requirements	$1,227,000
2	Existing Conditions	$63,000
3	Concrete	$1,859,000
5	Metals	$1,681,000

6	Wood, Plastics, & Composites	$10,000
7	Thermal & Moisture	$666,000
8	Openings	$80,000
9	Finishes	$107,000
10-12	Specialties, Equip. & Furnishings	$8,000
10	Specialties	$0
12	Furnishings	$0
13	Special Construction	$0
21-26	Plumbing & Mechanical	$3,692,000
31	Earthwork	$1,367,000
32	Exterior Improvements	$972,000
33	Utilities	$163,000
34	Transportation	$0
	Total	$11,900,000

Construction Management Conclusion

The construction management team has created a detailed project schedule along with an itemized detailed cost summary. Total materials, labor, and equipment charges have been calculated from divisions within OCD's scope of the project. The total for materials was calculated at $4.3 million, labor at $1.5 million, and equipment at $1.6 million. The remainder of the project cost is incurred in contingencies, mechanical and electrical, along with regulation permits and fees. To further assist in the project, the construction management team has included a detailed crew list in appendix CM3 of the report.

Considerations were taken to ensure the most cost economical construction project plan was used in this project, such as the soil treatment of the building slab. The schedule proposed by our team began in January for convenience only. The contractor may want to start the project at a different time, allowing the structural portion to begin in April. Starting phase 3 in April would allow better conditions for the concrete to cure, minimizing the need for admixtures preventing freezing. Our team has worked together to ensure this project can be constructed with as few complications as possible.

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