Final Report

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ROADRUNNER
EXECUTIVE TOWER

Table of Contents

List of Figures

Project Background

Roadrunner Executive Tower (The Tower) will be a 180’ x 220’ x 38’ two-story productivity space designed to foster the relationship between academia, local professionals, and the natural environment. The first story, inspired by the UTSA makerspace, will be open to the public and outfitted to attract anyone from individuals up to large groups of people who would otherwise go to a coffee shop, library, or similar setting. It will be a relatively open space with a surplus of seating arrangements, tables, couches, and even private study rooms that can be reserved. The second story will provide office space for local businesses and professional firms that tenants can rent out on a yearly basis. RAACC has accounted for movable partitions in the design of the second floor to accommodate customization and optimization of the space per the tenants’ desire. The second story will be offset 20 feet in the x and y direction from the first story to create a wraparound balcony on the north and east elevation and consequently an overhang on the south and west elevations. Low impact development features will be implemented throughout the site as the primary means of stormwater management and will serve to improve the existing conditions of the site, all while creating a peaceful and nature-centric aesthetic.

Property Background

The site for the development of Roadrunner Executive Tower is located at the intersection of UTSA Boulevard and University pass. The property is currently zoned as Master Plan Community District (MPCD) which is intended to “encourage the development of areas of mixed uses that are internally compatible in an effort to achieve well designed development and provide a more efficient arrangement of land uses, building and circulation systems” per the City of San Antonio’s (CoSA) Unified Development Code. RAACC finds the mission of MPCD developments to be inline with the purpose of Roadrunner Executive Tower, and as such, does not elect to rezone the property. The codes corresponding to MPCD development that most affected the design of The Tower are the maximum building height requirement, parking requirement, and the parkland/open space requirement. The maximum building height permitted for MPCD development is 40’ (The Tower is projected to have a 38’ slab to roof height) which gives context to the choice to build horizontally as opposed to the more economic vertical construction (economical at least in the perspective to land acquisition costs). There is a requirement of 60% parkland/open space, which RAACC has met by including a green space/walking trail and through the implementation of several LID features.

Site Information

Environmental due diligence of the site revealed that the property is outside the mandatory detention zone (figure 1) however a sand-filter detention pond will be implemented regardless as a sort of redundancy to ensure the site will manage stormwater runoff generated in even the largest of precipitation events. The site lies outside of the FEMA flood plain (figure 2) and is not within any known golden checked warbler habitat or over any known karst features (figure 3).

The property is within the Edward's Aquifer Contributing Zone within the Transition Zone (figure 4) which has implications mainly involving the LID design, which are discussed in the LID section of this report.

Land Development

RAACC will be responsible for platting the 8.35-acre property, which is a part of a larger property that is recorded in an original 116-acre survey out of the Anselmo Pru Survey 20, abstract 574, county block 4766, which remains unplatted. Per the City of San Antonio Development Services Department (DSD), the plat will qualify as a major plat due to the proposed improvement/extension of a public utility, which in this case is water and sewer. The plat will serve to also identify any easements that lie within the site, which requires extensive research of public recorded land records and adjacent plats, which is provided to the public via the Bexar County Clerk Official Public Records Search. Standard development notes from SAWS, CPS, CoSA are also included in a plat and are provided by the City of San Antonio. Additionally, our plat will need to include a standard LID/NCDP note as The Tower will implement several low impact development features. For a plat to be approved, it must be signed and sealed by a professional land surveyor, signed and sealed by a professional engineer, signed by the owner or the owner's representative, signed by a public notary who was present for the signing of the owner, reviewed, approved and signed by the Director of the DSD, and finally stamped by the Bexar County Clerk, Ms. Lucy Adame-Clark.

Low Impact Development (LID)

Construction of Road Runner Executive Tower (RRET) will convert an undeveloped 8-acre lot of 100% pervious cover to approximately 60% impervious cover, which will result in an increase in stormwater runoff**.** Thus, RAACC is responsible, per development requirements set in place by the City of San Antonio, to manage the anticipated increase in runoff in a manner that would not result in any adverse impact downstream from our site. In addition to adverse impact requirements, our site is over the Edward's Aquifer Contributing Zone within the Transition Zone**,** meaning that the stormwater runoff is expected to outfall into a stream or tributary that will *contribute* the runoff into the Edward's Aquifer Recharge Zone, of which there are stricter development requirements treatment standards to ensure pollutants do not enter into the aquifer. Considering adverse impact and the potential of the runoff to reach the Recharge Zone, RAACC has accepted the duty to provide volume reduction and treatment to the runoff. RAACC has decided to implement several LID features standardized and championed by the San Antonio River Authority (SARA) in their 3rd Edition of the San Antonio River Basin Low Impact Development Technical Design Guidance Manual (SARB LID TDGM) to satisfy these requirements. Additionally, there are also numerous development credits that will apply to the zoning requirements for MPCD developments and fee reductions regarding stormwater management. Between the requirement to responsibly manage the increase in stormwater runoff, the site's vicinity to the Edwards Aquifer, the economic benefits, the environmental benefits, and the consequential benefits to the users of Road Runner Executive Tower and the immediate community in the area, the implementation of LID features presented itself as an opportunity to elegantly solve the problems associated with the increase in stormwater runoff generated by the development.

The following paragraphs will discuss the justification for implementation of each LID feature, give a brief description of how they will work and the technical requirements to be met, and explain sizing information supported by calculations that will be referenced as appendices. All requirements being referenced and satisfied are per the SARB LID TDGM.

Bioretention

Justification

Per the drainage plan, all stormwater runoff will drain north-east to south-west across the site. The majority of stormwater runoff will sheet flow into the bioswale natural channels (discussed later) at the north and south extents of the site, however the proposed grading of the site presented the risk of stormwater runoff pooling at the middle parking island and at the parking spaces bordering the green space/walking path. To address this, bioretention (figure 5) will be implemented to accept the stormwater runoff that would otherwise pond in these areas if a traditional parking island and curb system were to be used.

Modus Operandi

Runoff accepted into the bioretention areas will be in sheet flow where sawtooth curbs will allow the runoff to enter the bioretention areas. Sawtooth curbs have been selected to provide a decrease in the velocity of the runoff for the purpose of limiting erosion. Additionally, a river stone fringe will be utilized at the perimeter of the bioretention areas to provide further energy dissipation of the runoff and additional erosion control for the soil media. To accommodate precipitation events that occur in high frequency and/or events where the rate of precipitation exceeds the design precipitation rate (1.1 inches/hour), overflow outlets have been implemented at the south extent of each bioretention feature to redirect excessive runoff back onto the parking lot. From here, the excessive runoff will re-enter sheet flow, drain southwest across the parking lot, and enter channel flow once it enters the natural channel at the southern perimeter of the site. From here, the water can either infiltrate into the channel media or will be directed into the sand filter detention pond at the southwest corner of the site. To prevent lateral flow from infiltrated runoff under the parking lot, which could cause problems given expansive clays in the area, an impermeable membrane will line the bioretention areas. The impermeable liner is also required as our site lies within the Edwards Aquifer Contributing Zone within the Transition Zone (SARB LID TDGM, 2023) and infiltration is not allowed. A cross-sectional view illustrating the components and intent of the bioretention design can be seen in the construction documents.

Requirements

Captured runoff is required to completely infiltrate into the soil media at an infiltration rate of greater than 0.5 inches per hour. This is to both avoid the need of an underdrain and to meet the infiltration requirements for bioretention areas. The infiltration requirements are to allow for surface infiltration into the soil media in less than 24 hours, and complete dewatering of the soil media in less than 48 hours. To treat all pollutants of concern, which include sediment, nitrogen, phosphorous, pathogens, metals, grease/oil, and temperature, the required depth of the soil media will be 4 feet (table B-3, SARB LID TDGM). To accommodate the two extreme conditions associated with bioretention, which are complete saturation of the soil media and drought, plants will need to be selected from appendix E of the SARB LID TDGM.

Sizing

Per the drainage plan, the volume of runoff that can be expected to contribute to these bioretention areas is found using a precipitation rate of 1.1 inches per hour, which is standardized in the Texas Center for Environmental Quality's (TCEQ) Edwards Aquifer Compliance Design Manual and accepted for design in the SRB LID TDGM**.** To the desired level of treatment, the guide directs the use of a 4-foot soil media layer with a mix design of. 92% sand, 3% fines, and 5% organic matter. According to the SRB LID TDGM, a soil media depth of 4 feet will treat infiltrated stormwater in terms of suspended solids, oil/grease, lead, phosphorous, zinc, nitrogen, and bacteria (figure 6). Additionally, the mix design has been shown to meet the desired infiltration rate of 6 inches per hour. Supporting calculations can be seen in appendix A.

Bioswale Natural Channels

Justification

Per the grading plan, runoff is designed to be in sheet flow throughout the parking lot. In areas where runoff is not directed into the bioretention at the middle portion of the site (as discussed above) the runoff is directed by the proposed grade to drain into a bioswale natural channel at the north and south extents of the site.

Modus Operandi

As opposed to the sawtooth curbs allowing the infiltration into the LID feature, the runoff for the bioswale is permitted to enter the feature via a ribbon pavement and wheel stop system. RAACC found this option, which would use dowels to anchor the precast wheel stops at each parking stall, more economical than implementing saw-tooth curbs along the length of the bioswale natural channels. Additionally, RAACC finds the aesthetics of this system more appealing and in line with the overall vision of Road Runner Executive Tower. The bioswale (figure 7) will be completed with a filter strip at the side adjacent to the parking lot, larger river stones at the middle of the feature, and be sloped throughout its length. When the rate of precipitation is greater than the infiltration rate of the system, the slope and porosity differential between the large river stone and the less porous soil media will encourage stormwater to run downslope, where it will eventually outfall into the detention pond at the southwest corner of the site. Because the bioswale is prioritizing conveyance over infiltration, the landscaping within them will feature flora requiring less water and larger river stones, which is colloquially referred to as "zero-scaping" (figure 8). The river stones will also be the primary means of erosion control within the bioswale natural channels. Considering the slope from east to west within the bioswale, check dams can be implemented to satisfy velocity and slope requirements. Consistent with bioretention, an impervious liner will be implemented to prevent subsurface lateral flow that could otherwise cause damage to the adjacent right of ways and the parking lot. A cross sectional view illustrating the components and intent of the bioswale design can be seen in the construction documents.

Requirements

The requirements for bioswales are consistent with those for bioretention in terms of infiltration (greater than 0.5” per hour), maximum ponding depth (18”) and required soil media depth (4') to provide the intended level of treatment of the runoff. Bioswales have additional requirements regarding the slope throughout their length, which consequentially invoke velocity requirements. According to convention, bioswales are typically between 2-8 feet in width. In using the river stone in the middle of the channels, the velocity requirement to be satisfied is in accordance with the reinforced turf value of no more than 14 feet per second. To limit erosion, bed slope is limited to 2% across the length, but the implementation of check dams allows the length to be broken up into segments that can have slopes up to 5%. Regardless, the average bed slope of the segments should still be 2%. Additionally, check dams have a maximum recommended height of 5 feet and are to include a gravel splash pad, at least 4” thick, underlain by a geotextile and should extend 2 feet from the base of the check dam to again limit erosion. It is recommended that the stone splash pad be No. 57 stone, which could also be mortared to prevent the risk of removal. The bioswale natural channels will not be designed to overflow at grade as the bioretention has but will instead include steeper and higher banks to facilitate channel flow down grade from west to east across the site.

Sizing

The sizing of the bioswale natural channels follows the same methods as described in the bioretention section and they will include the same soil media profile and design. RAACC will not be implementing check dams as the minimal slope (3%) in conjunction with the head loss and energy dissipation from the use of larger river stones is not expected to exceed, or come close, to 14 ft/sec. Supporting calculations can be seen in Appendix B.

Rooftop Rain Capture

Justification

Road Runner Executive Tower has a 180’ by 220’ (39,600 SF) building footprint. Whereas the bioretention features and bioswale natural channels serve to treat and provide the required flow reduction for runoff attributed to the parking lot, RAACC will be implementing cisterns to store rainfall captured by the building footprint. By locating the cisterns on the high point of the site, the rooftop rain capture system will provide irrigation to the landscaping throughout the site without the use of a pump. Therefore, the temporary storage provided by the cisterns will mitigate the adverse impact attributed to the building footprint and will replace a traditional irrigation line/sprinkler system.

Modus Operandi

The Tower will feature a standing seam metal roof at a single pitch (west to east) sloping 2% length. This slope will direct water into an industrial gutter system, which will then redirect water into the two downspouts that lead directly into the cisterns (figure 9). The cisterns will sit atop a reinforced concrete pad surrounded by smaller bioretention areas to accommodate instances of overflow or low flow discharge. The cisterns will feature two safeguards to eliminate the risk of backing up through the gutters and into the roof; an air release valve at the top of the cistern to allow the air that becomes displaced by the water to escape from within the cisterns, and an overflow outlet that is designed to discharge large quantities of water simultaneously as the cistern reaches design capacity. The cisterns are located at the highest point of our site (994 feet above sea level) where the gravitational potential energy from the height differential will be used to provide irrigation to the landscaping throughout the site without the use of a pump.

Structural

The Roadrunner Executive Tower will consist of a main two-story structure that will house conference rooms and study spaces to be rented out to businesses or students. Multiple design alternatives were considered for this project, and the following alternative was chosen based on its ability to best fit the owner's needs:

Building Codes

The building codes and fire codes specified by the city of San Antonio regarding commercial buildings will be used for the design of the structure. The design of the building materials used in construction will also follow their respective code requirements.

The following building codes will be used for the design of the Roadrunner Executive Tower:

- 2021 International Building Code (IBC)
- 2021 International Conservation Code (ICC)
- Building Code Requirements for Structural Concrete (ACI 318-19)
- AISC Steel Construction Manual, 15th Edition
- 2018 National Design Specification for Wood Construction
- Minimum Design Loads and Criteria for Buildings or Structures (ASCE 7-22)

Building Dimensions

The area of each floor will be 180ft x 220ft, however the second floor will be shifted over by 20ft in both directions to allow for a 20ft wide balcony on the roof of the first story. This will also mean there is a 20ft overhang along two sides of the second story. The dimensions of the suspended foundation will be 200ft x 240ft to accommodate this. The first story will be 18ft in height, with 5ft being included for mechanical space and the height of the structural elements (13ft floor to ceiling), and the second story will be 15ft (10ft floor to ceiling). The final building height will be 33ft on one end and 37.2ft on another end to achieve a ¼" per ft slope and allow rain to drain. 2ft parapets will be added along the sides of the roof.

Proposed Design

The Roadrunner Executive Tower will be a two-story steel framed structure with a suspended reinforced concrete foundation using a pan-joist system. The second story will cover the same area as the first story with the addition of a cantilevered overhang on the second floor at the northwest corner of the building. The lateral system for the structure will consist of three steel braces span from the foundation to the roof located within the walls of the elevator shaft, as well as additional braces at the corners of each story. The exterior walls of the structure will consist of glass that will follow structural and environmental specifications. Sections of the exterior not spanned by glass will have insulative coverings specified by the architect and will be supported by metal studs. Minor partitions will be made of timber.

A concrete-and-steel composite floor system will serve as the deck for the second floor and will also accommodate a seating area at the roof of the first story (second story balcony). Piers will have to be poured at corners and column locations. Pans will then be used as forms for the suspended concrete foundation, then steel columns and beams will be erected for the framing.

Design Loads

The following loadings and pressures are retrieved from the ATC Hazards site along with the ASCE 7-22.

Live Loads

Table 4.3-1 of the ASCE

Offices:	(65)psf
Lobbies:	100psf
Corridors above first floor:	80psf
Stairs/Exits:	100psf
Dining/Restaurant:	100psf
General Assembly:	100psf
Mechanical Room:	150psf
Flat Roof:	20psf

4.5.1.1 of the ASCE also requires a 50 lb/ft (live) load be placed at any guardrails, such as along the stairs or the balcony on the second story.

Dead Loads

Table C3.1-1a of the ASCE

Gypsum and Mechanical:	13psf
Tile Flooring:	10psf
VCT/Carpet Flooring:	5psf
Roofing and Rigid Insulation:	8psf

Stud Walls w/ Metal Panels: 15psf

Windows, frame, and sash: 8psf

4" Slab on Composite Deck: 52.5psf

5" Foundation Slab: 75psf

Roof Joists: 2.5psf

Other structural members will include their self weight in addition to their carried dead loads when calculating their capacity throughout the design process.

Wind Loads

The method used to attain the wind pressures placed laterally along the building was the Components and Cladding Method in accordance with ASCE 7-22.

Nominal Design Wind Speed: 107mph

Risk Category: II

Exposure: B

Internal Pressure Coefficient: +/- 0.18

Final Wind Pressures used in design can be found in ***Appendix A****.*

Seismic Loads

The full list of coefficients and seismic values attained by the ATC can be found in Appendix D.

Occupancy Category: II

Site Class: C

Fa: 1.3

Fv: 1.5

SDS: 0.043

SD1: 0.023

Seismic Design Category: A

According to the ASCE, a structure in an SDC of A is not required to design for any seismic forces beyond a lateral force at each floor of 1% the total weight of the structure, for stiffness and integrity purposes.

Misc. Loads

Snow loads within San Antonio range from 0psf to 5psf according to ASCE 7-22, and neither of those values will exceed the roof live load in LRFD combinations. Similarly, the roof will be designed to drain water as to prohibit any Rain loads from exceeding roof live *load.*

*An example schematic for an elevator was provided to estimate the loading within the elevator shaft. A hoist beam is required at the top center of the shaft that needs to withstand a **7.5kip** load. The bottom slab of the shaft will withstand point loads ranging from **4kips** to **21kips**.*

Foundation

The final foundation framing is as shown in (figure 10)**.** The spacing of the columns and piers have been changed over the course of this project to fit two different needs. First, we did not want the ceiling beams to be too tall and take away from the ceiling space. And second, if our pier reactions were too high, the final pier lengths would require us to drill very deep into the limestone, which is both expensive and time consuming. The loads calculated above were inputted along with the framing into a structural design program called RAM Structural Systems, offered by Bentley CONNECT. The 20” depth pan for the suspended foundation as well as the joist spacing (6’-2”), and member widths were all taken from a standard form-sizing manual provided by form manufacturers for contractors and engineers to standardize. By using these dimensions, the need for custom formwork by the contractor is greatly reduced. RAM was used to identify the highest-loaded member from each of the three foundation member types, and was used to size the final longitudinal reinforcement that was used on the final plan sheet. The shear reinforcement, however, was calculated manually on an Excel sheet made by the structural engineer of this project. The shear loads were calculated by RAM, but the minimum stirrups to meet those demands were taken from the calculations done in **Appendix G**. The pier reactions were also provided by the software as shown in (figure 11), but the capcities of the piers based on depth were manually calculated based on values provided by the geotech, and can be found in **Appendix H**. The depth of the suspended slab itself was originally based off of projects of similar use and size, however the capacity of the slab was tested for this plan and the reinforcement was determined based on hand calculations in **Appendix D**.

Second Floor Framing

The framing of the second floor can be seen in (figure 12). As can be inferred from the foundation plan, all the columns are located above a pier to reduce the shear demand on the foundation members. The composite deck type selected was based on a list provided by VULCRAFT that uses standard dimensions and values similar to other deck manufacturers. The demand on the deck in this project was calculated, and those values were compared to those in the manual before the deck was selected. This entire process can be found in **Appendix C**. Based on the weight of the deck as well as the loads previously defined, the program was able to size the beam members as well as the columns, which the final sizes used can be found in the construction plan sheets. A randomly selected column as well as a randomly selected beam was spot-checked to compare the accuracy of the values displayed by the software. The hand-calculation versus the program’s can be found in **Appendices E & F**.

Roof Framing

The framing of the roof can be seen in (Figure 13). All columns were continued to the top with the exception of the columns supporting the stairs surrounding the elevators. This is because they were not needed beyond the second floor to help support the roof deck, and we believed this created a more open space for tenants to walk through when entering the second story through the stairs. The beam sizes (non-composite) were taken from RAM as well, however the joist sizes were taken from an Economic Joist Guide provided by VULCRAFT that uses sizes and capacities standardized amongst joist manufacturers. The roof dead load and live load were added together and multiplied against the 6' spacing of the joists, and that value was compared to the ASD tables provided before finalizing the sizes shown on the construction plan sheet. Similar to the composite deck sizing for the second floor framing, the metal roof deck was also sized from the VULCRAFT manual while using the demand I calculated by hand. This process can be found in **Appendix B**.

Braces

Braces were placed along the walls of the elevator shafts. There were also two braces placed at a corner of the structure, placed perpendicular to one another, and this configuration was also mirrored on the corner opposite to the first. The wind loads calculated from **Appendix A** were placed as point loads along the frame of the structure, tested against all four faces, and the final axial forces in the braces were displayed by the software. The brace configurations can be seen in figures 14, 15, 16, and the final sizes for all the braces were rectangular 6x6x1/4 HSS tubes. The largest tensile or compressive load on either of the brace configurations was 68kips on a single member.

Geotechnical

The site is located on the corner of UTSA Boulevard and Univ Pass. Geological the site on the border of the Del Rio and Buda limestone formation. For an accurate understanding of the soil type three borings were drilled at the depths of 10 and 30 feet, one 30-foot boring in the building and two in the parking lot. The boring logs revealed the following soil types: fat clay (CH), lean clay (CL), and limestone. During and after the process of drilling no water was observed. As results of boring one, which was drilled at the location of the building, two plasticity index tests were conducted. The first plasticity index provided a plasticity index of fourth-seven which indicated fat clay and twenty-three indicating lean clay. The potential vertical rise (PVR) was calculated to be 3.26, due to the fat clay. As a result of a high PVR we are lime treating the parking lot. Lime treating the fat clay will stabilize the subgrade, increase the load bearing capacity, and lower the plasticity index. Lime treatment and PVR calculations can be observed in appendix I. For the boring logs reference, they can be found in appendix H.

Utilities

Water Utilities

The water conveyance that will be utilized for the proposed building includes a 12" water line located directly north of the building on UTSA Boulevard. Per the SAWS Infrastructure Planning EDU Calculation Sheet, an office building has an average capacity of 0.035 gal/sf for one day. The square footage of the building is approximately 79,200 sf, producing a total capacity of 2772 gal/day for this building. size for the proposed water meter is 2" with a 2" service line. Static pressure at the meter will be 79.33 psi which means this line will not require a pressure-reducing valve.

Wastewater Utilities

The wastewater lateral will connect to a 12” sewer main south of the building located on a public easement. The EDU measurement for wastewater is 200 gpd. Giving us a total EDU of 13.86 for the building. Peak Flow was calculated to be 11,940 gpd. This includes inflow and infiltration as required by SAWS. Using the Manning Equation, we were able to find the average flow and velocity of a 6” diameter PVC pipe (1,128,742 gpd). This calculation is for a 6” diameter pipe running at 100% capacity. SAWS requires a minimum 6” sewer lateral, giving us more than enough flow for the building output.

The elevation between the existing wastewater main and the building caused an 8% slope that could possibly wear down the pipe at an accelerated rate. To deter this, we are proposing a drop manhole that will allow for a 2% slope to be maintained throughout the line.

Fire Protection Utilities

Fire protection for our site will provide a 2” DI fire line directly to the building for a NFPA 13 sprinkler system. This line will include a backflow prevention device to keep stagnant water from going back into the main SAWS line. A designated fire lane will wrap around the entirety of the building with a width of 26 FT. With the square footage that the building has and the material being used for the building, a designated fire flow line with 1 fire hydrant will be required per the international fire code. There is an existing fire hydrant located directly north of the building that can be used as one of the required hydrants. The fire flow line will have to produce at least 1,000 gpm with a minimum pressure of 20 psi. This line will have to be 6” of DI with a mechanical joint gate valve.

Drainage

According to COSA Storm Water Design Manual and the TxDOT Hydrology manual, the rational method can be used for areas under 200 acres. The tables are values to be used in the calculations of the runoff rate for a 25-year and 100 year storm (Q in cfs) for our lot in existing conditions. Top of elevation of our existing property is 994 ft above sea level that will drain into a 966 ft above sea level culvert exiting the property. See attached map and tables. Even though we are outside of the mandatory detention center, we have designed an optional detention pond located at the lowest point of the property (SW corner). Directly in front of the walking trail area and in-between the parking island are areas for three optional bio-retention ponds to water and maintain the property landscaping. The Proposed drainage plan will follow the same flow path of the existing drainage with a minimum 2% slope. The proposed drainage paln will utilize sawtooth curb outlets to drain parking lot runoff into the existing channels that will be modified to carry runoff to the detention pond. The Proposed drainage will also flow through sheet/shallow flow through the parking lot and open green space (see appendix J).

Site Plan

The site consists of 8.44 acres of undeveloped land off UTSA Blvd, located in the north east side Northwest side of San Antonio Tx. This Property is zoned as a Master Plan Community District (MPCD), to develop areas of mixed uses that are internally compatible in an effort to achieve well designed development and provide a more efficient arrangement of land uses. Based on the GFA this site will need minimum 260 parking spaces, that is 1 per 300 sqft GFA.

Minimum 25 ft clearance was left between 90º parking spaces. Per the Unified Development Code 20% of open space has been reserved as Impervious Cover. Two 12’ x 50’ loading zones were added as well as a dumpster area to the site plan.

Figures

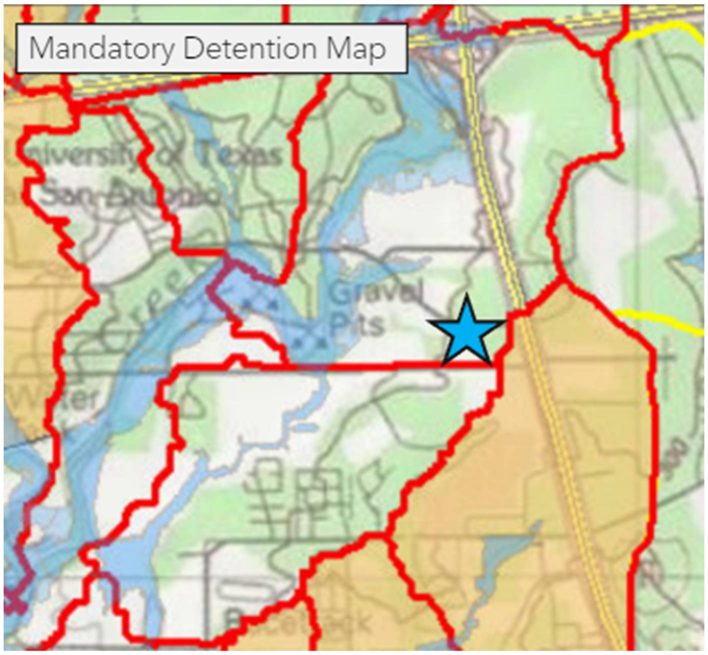


Figure 1 - Site in relation to Mandatory Detention Zones.

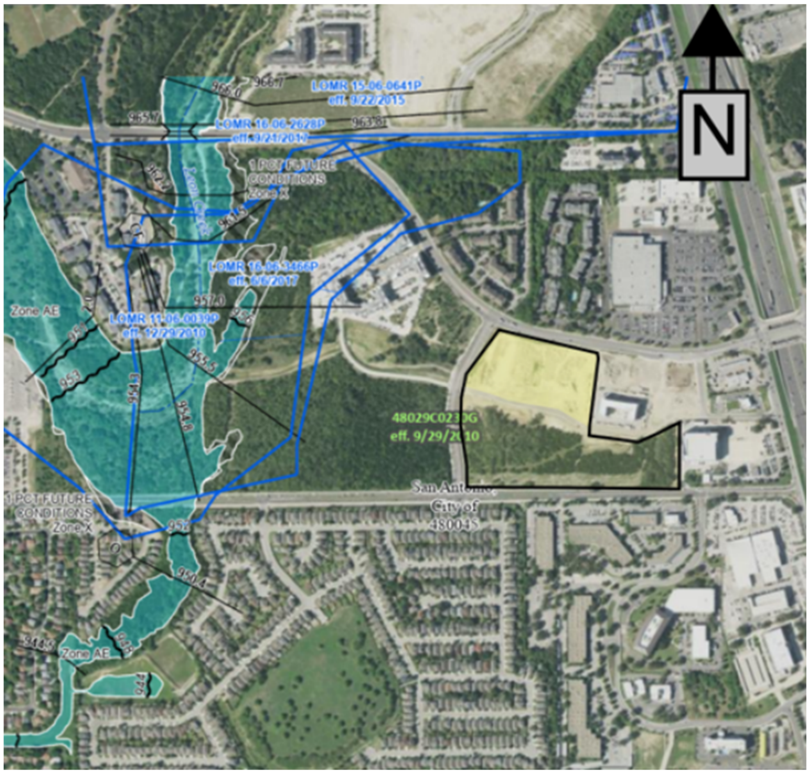


Figure 2 - Site in relation to FEMA Floodplain.

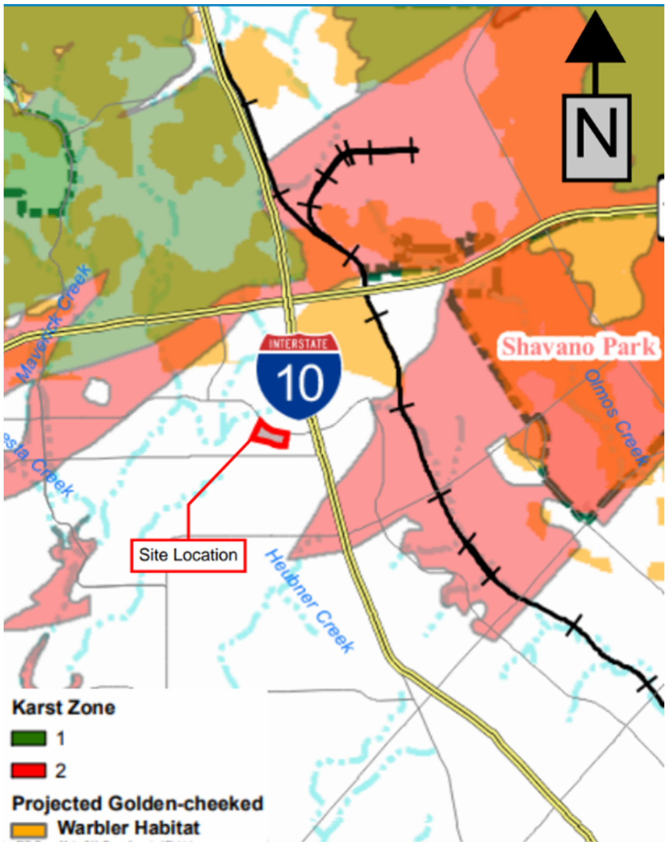


Figure 3 - Site in relation to Karst Zones and Golden Cheeked Warbler Habitat.

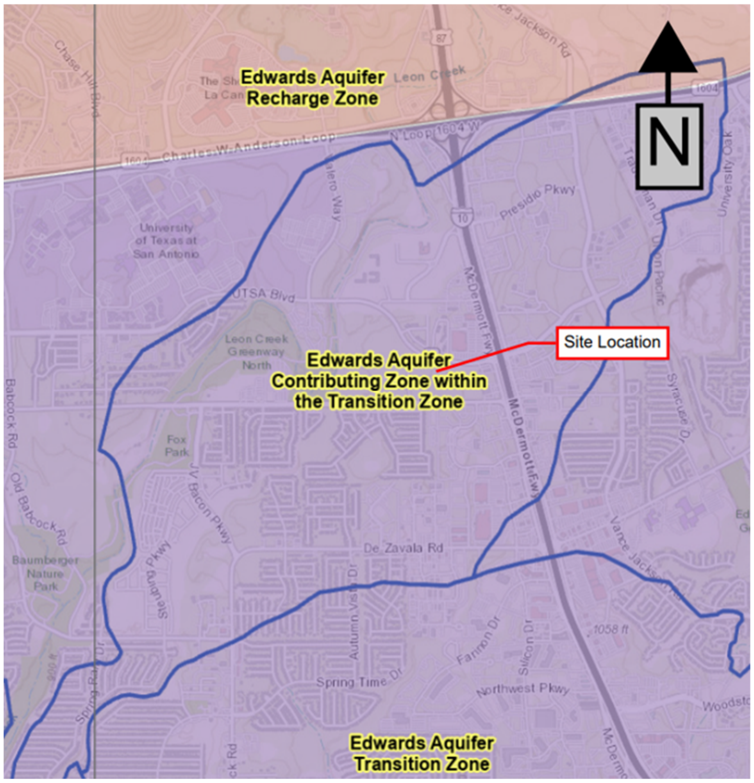


Figure 4 - Site in relation to Edwards Aquifer.

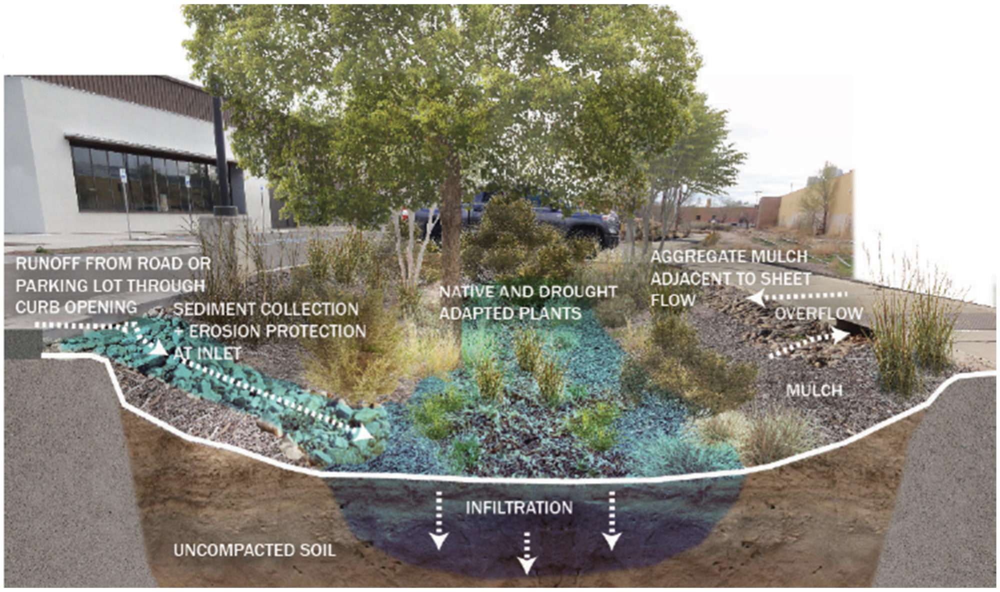


Figure 5 - Typical bioretention cross section.

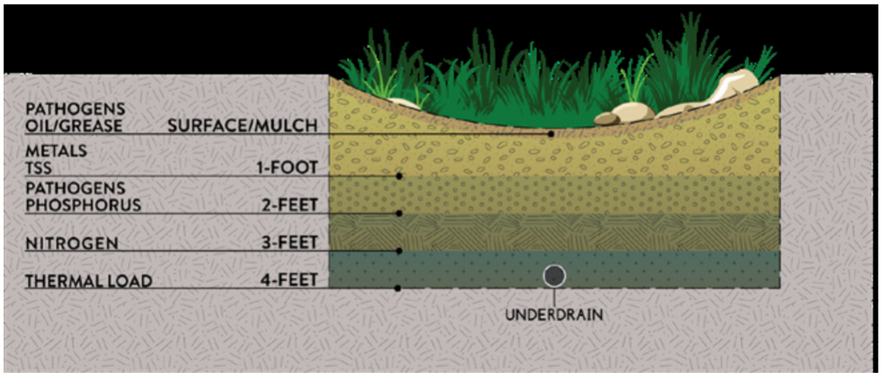


Figure 6 - Depth of soil media required to remove pollutants.

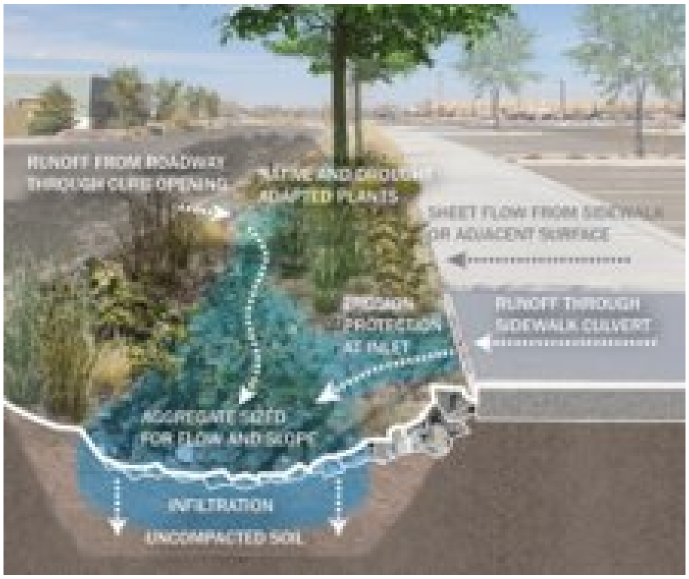


Figure 7 - Typical Bioswale cross section.

Figure 8 - Example of zero-scaping.

Figure 9 - An example of a roof-gutter-downspout-cistern system.

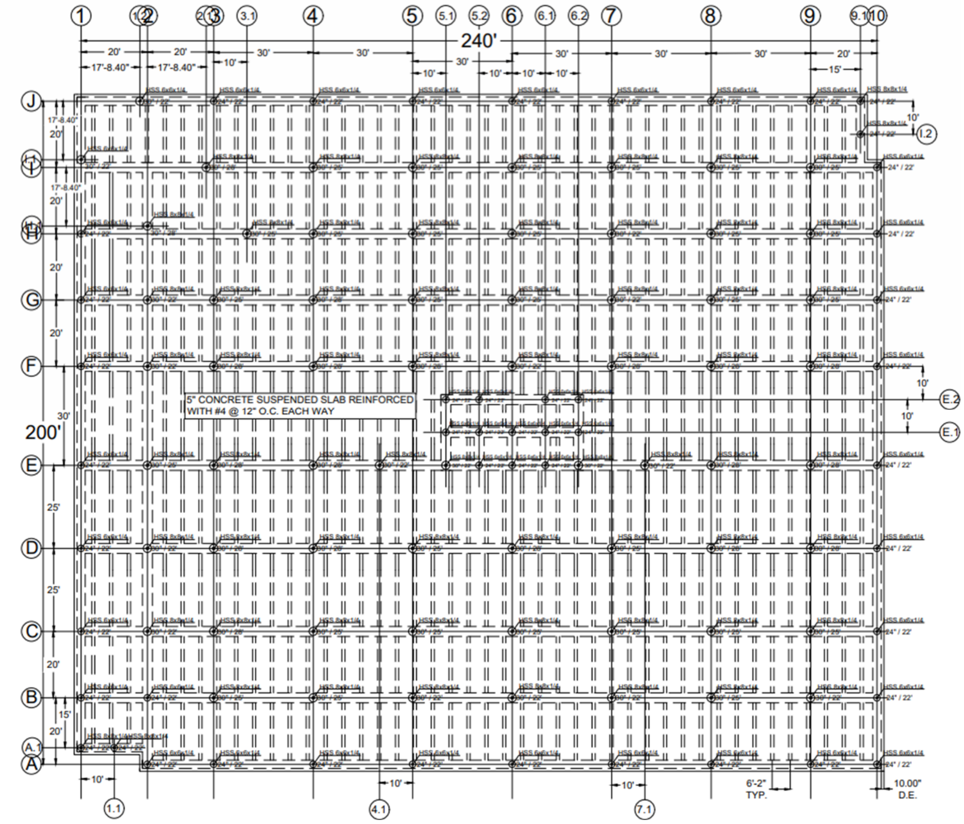


Figure 10 - Foundation Plan

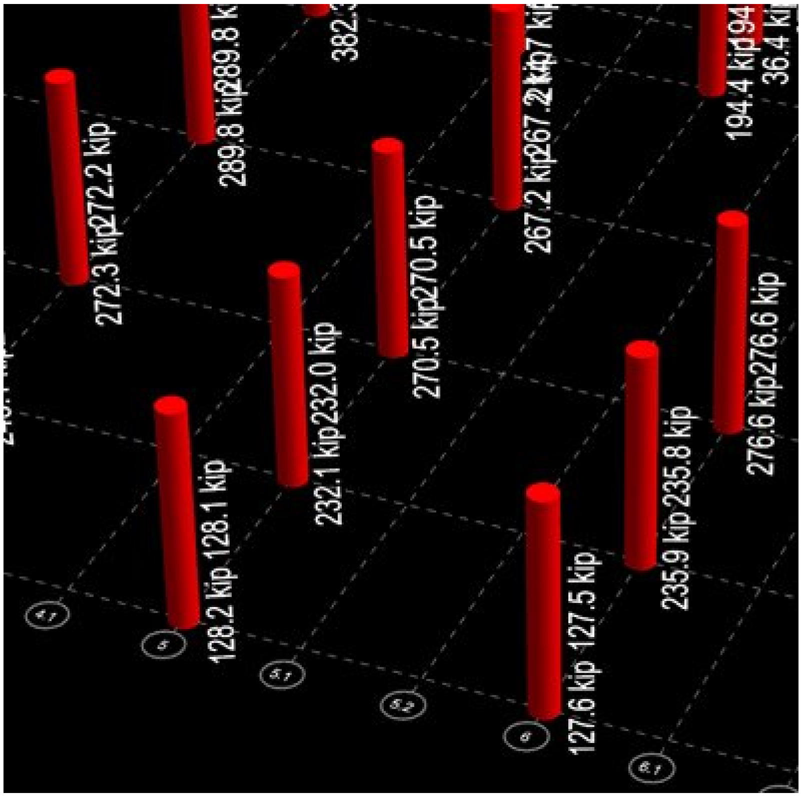


Figure 11 - pier reactions

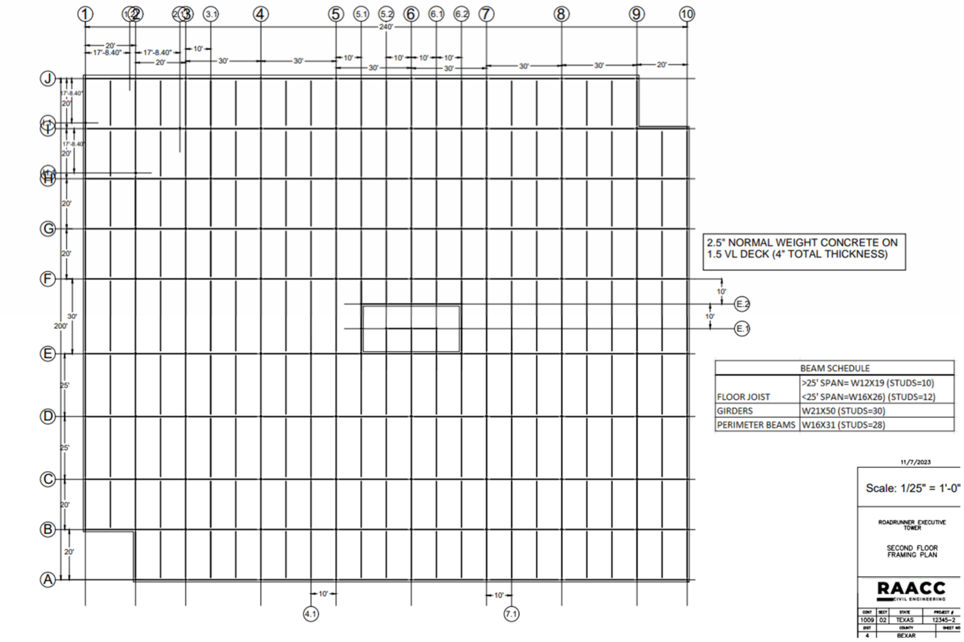


Figure 12 - second story framing.

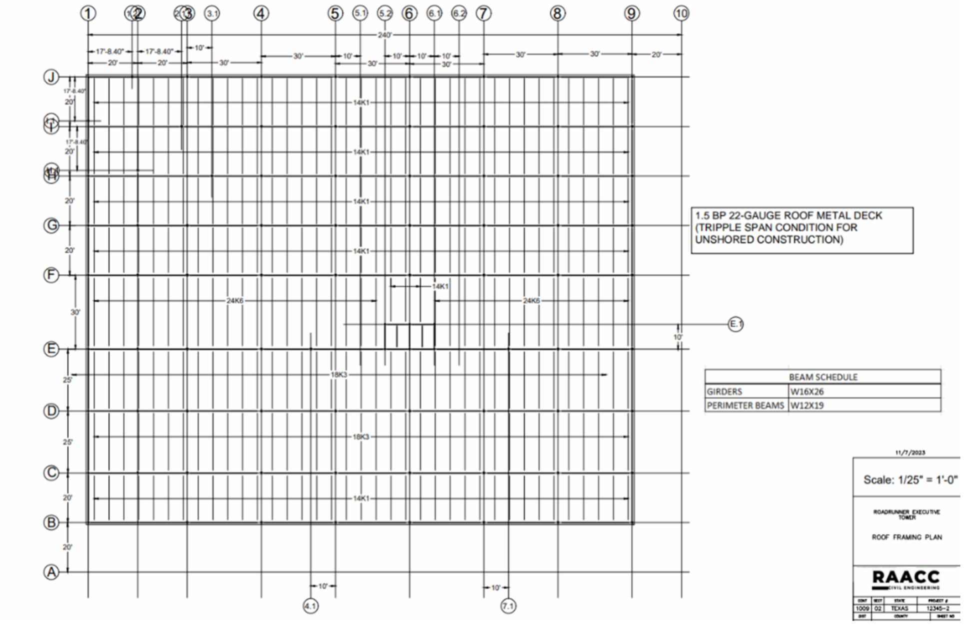


Figure 13 - Roof Framing Plan

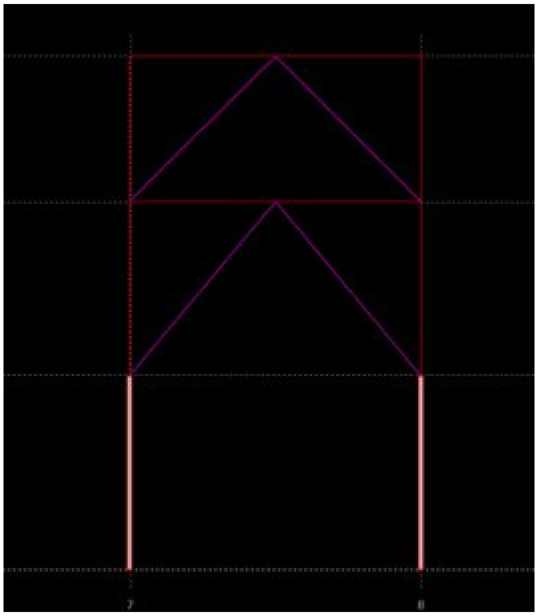


Figure 14 - Brace Elevations

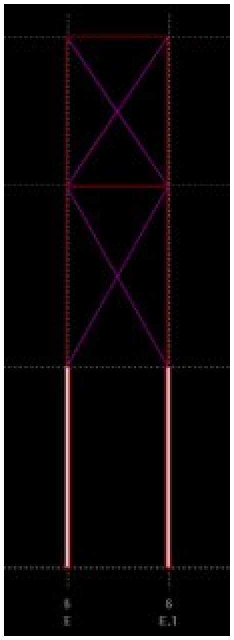


Figure 15 - Brace Elevations

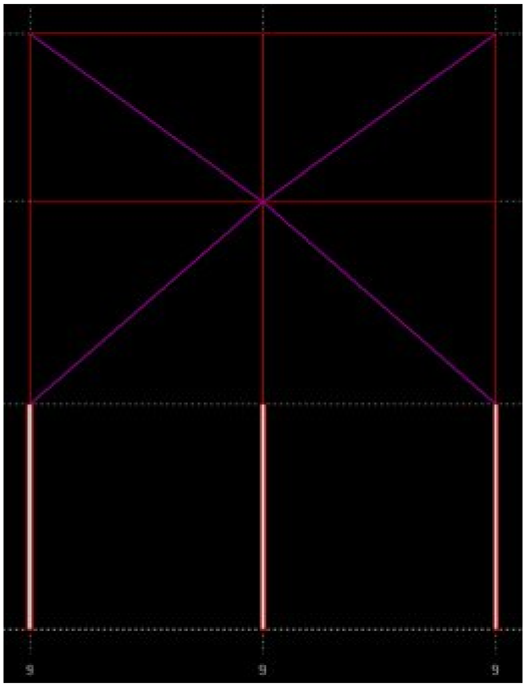


Figure 16 - Brace Elevation

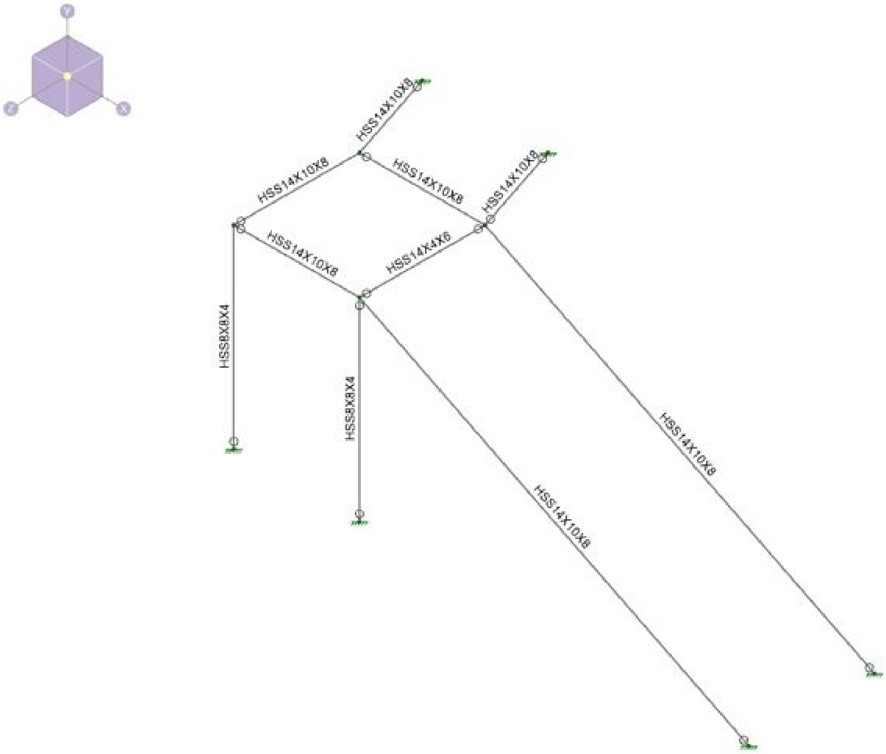


Figure 17 - Stair Design 1

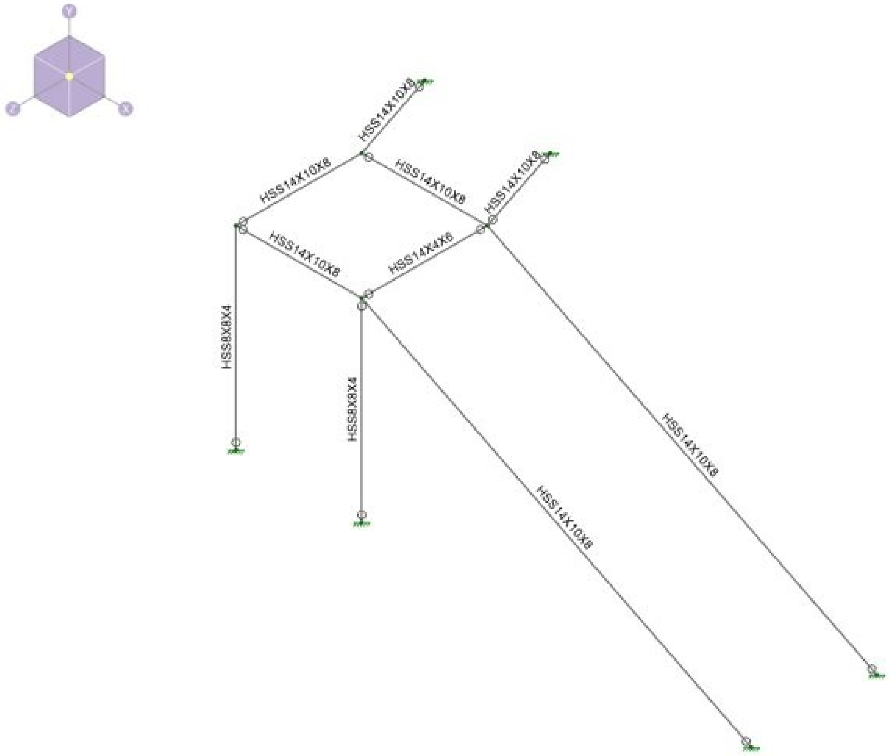


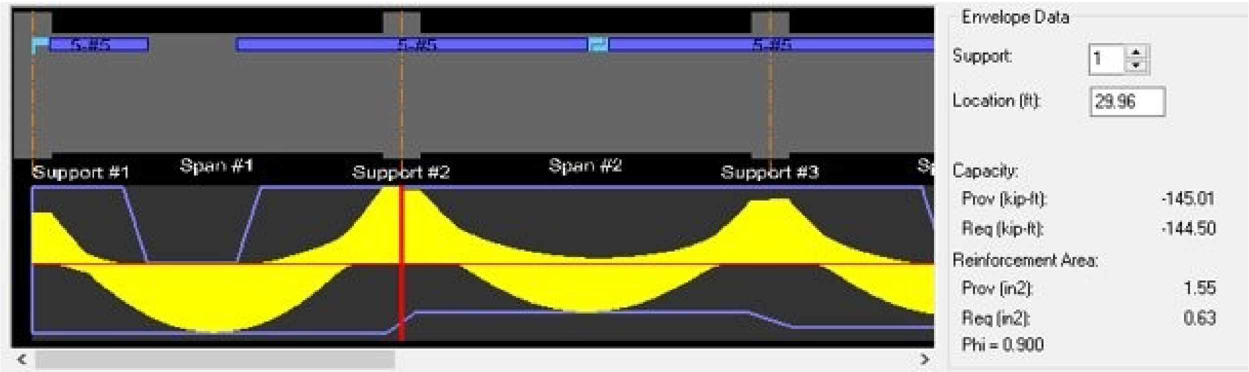
Figure 18 - Stair design 2

LC 2: 1.2D+1.6L

AISC 15th (360-16): LRFD Code Check

Limit State	Required	Available	Unity Check	Result
Applied Loading - Bending/Axial	-	-	-	-
Applied Loading - Shear + Torsion	-	-	-	-
Axial Tension Analysis	0 k	940.5 k	-	-
Axial Compression Analysis	6.22 k	856.584 k	-	-
Flexural Analysis (Strong Axis)	71.277 k-ft	370.5 k-ft	-	-
Flexural Analysis (Weak Axis)	155.927 k-ft	293.668 k-ft	-	-
Shear Analysis (Major Axis y)	88.278 k	316.512 k	0.279	PASS
Shear Analysis (Minor Axis z)	77.283 k	216.072 k	0.358	PASS
Bending & Axial Interaction Check (UC Bending Max)	-	-	0.882	PASS
Torsional Analysis	61.728 k-ft	269.176 k-ft	0.229	PASS

Figure 19 - Stair loads



Envelope Data

Support: 1

Location (ft): 29.96

Capacity:

Prov (kip-ft): -145.01

Req (kip-ft): -144.50

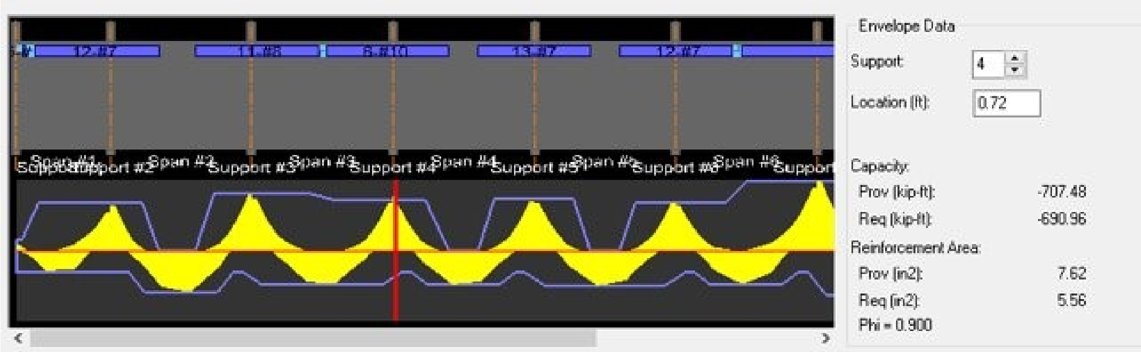
Reinforcement Area:

Prov (in2): 1.55

Req (in2): 0.63

Phi = 0.900

Figure 20 - Girder Moments Diagram



Envelope Data

Support: 4

Location (ft): 0.72

Capacity:

Prov (kip-ft): -707.48

Req (kip-ft): -690.96

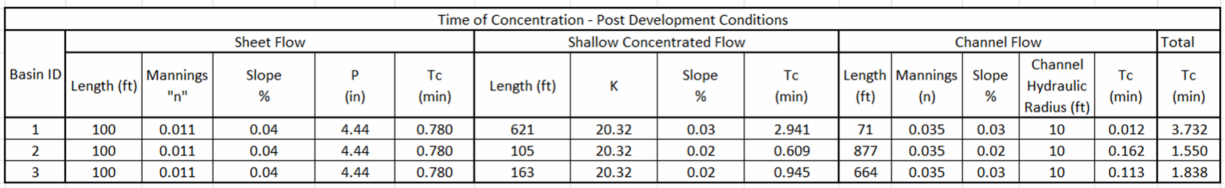
Reinforcement Area:

Prov (in2): 7.62

Req (in2): 5.56

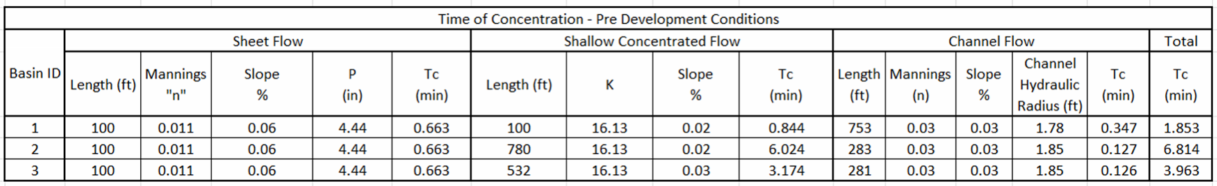
Phi = 0.900

Figure 21 - Joist moment diagram



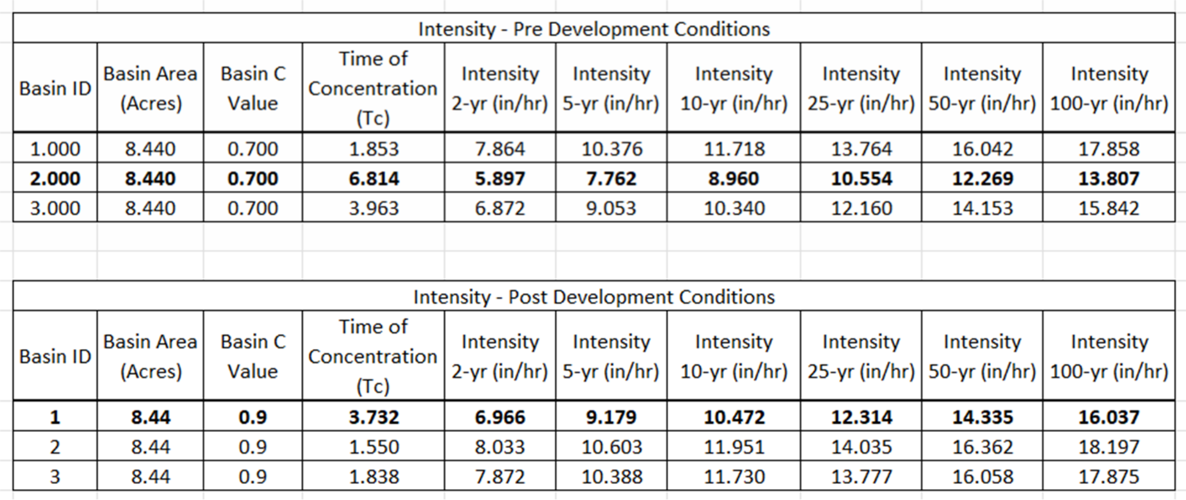
Time of Concentration - Post Development Conditions															
	Sheet Flow					Shallow Concentrated Flow				Channel Flow					Total
Basin ID	Length (ft)	Mannings "n"	Slope %	P (in)	Tc (min)	Length (ft)	K	Slope %	Tc (min)	Length (ft)	Mannings (n)	Slope %	Channel Hydraulic Radius (ft)	Tc (min)	Tc (min)
1	100	0.011	0.04	4.44	0.780	621	20.32	0.03	2.941	71	0.035	0.03	10	0.012	3.732
2	100	0.011	0.04	4.44	0.780	105	20.32	0.02	0.609	877	0.035	0.02	10	0.162	1.550
3	100	0.011	0.04	4.44	0.780	163	20.32	0.02	0.945	664	0.035	0.03	10	0.113	1.838

Figure 22 – Time of Concentration calculations



Time of Concentration - Pre Development Conditions															
	Sheet Flow					Shallow Concentrated Flow				Channel Flow					Total
Basin ID	Length (ft)	Mannings "n"	Slope %	P (in)	Tc (min)	Length (ft)	K	Slope %	Tc (min)	Length (ft)	Mannings (n)	Slope %	Channel Hydraulic Radius (ft)	Tc (min)	Tc (min)
1	100	0.011	0.06	4.44	0.663	100	16.13	0.02	0.844	753	0.03	0.03	1.78	0.347	1.853
2	100	0.011	0.06	4.44	0.663	780	16.13	0.02	6.024	283	0.03	0.03	1.85	0.127	6.814
3	100	0.011	0.06	4.44	0.663	532	16.13	0.03	3.174	281	0.03	0.03	1.85	0.126	3.963

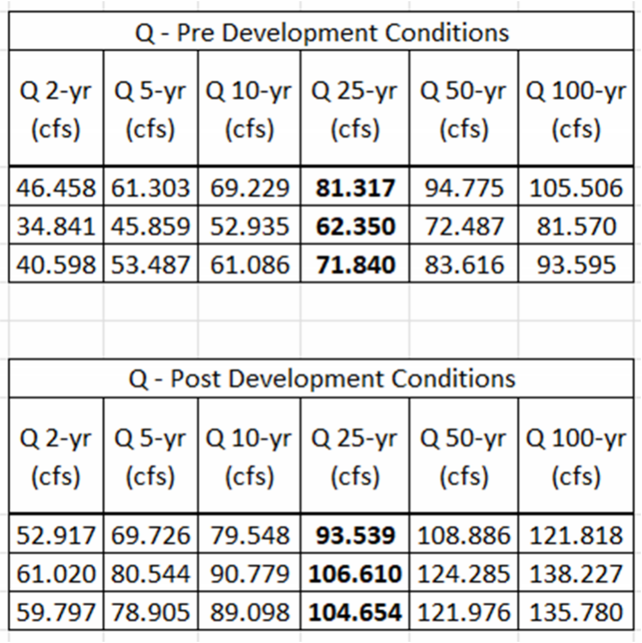
Figure 23 – time of concentration calculations



Intensity - Pre Development Conditions									
Basin ID	Basin Area (Acres)	Basin C Value	Time of Concentration (Tc)	Intensity 2-yr (in/hr)	Intensity 5-yr (in/hr)	Intensity 10-yr (in/hr)	Intensity 25-yr (in/hr)	Intensity 50-yr (in/hr)	Intensity 100-yr (in/hr)
1.000	8.440	0.700	1.853	7.864	10.376	11.718	13.764	16.042	17.858
2.000	**8.440**	**0.700**	**6.814**	**5.897**	**7.762**	**8.960**	**10.554**	**12.269**	**13.807**
3.000	8.440	0.700	3.963	6.872	9.053	10.340	12.160	14.153	15.842

Intensity - Post Development Conditions									
Basin ID	Basin Area (Acres)	Basin C Value	Time of Concentration (Tc)	Intensity 2-yr (in/hr)	Intensity 5-yr (in/hr)	Intensity 10-yr (in/hr)	Intensity 25-yr (in/hr)	Intensity 50-yr (in/hr)	Intensity 100-yr (in/hr)
1	**8.44**	**0.9**	**3.732**	**6.966**	**9.179**	**10.472**	**12.314**	**14.335**	**16.037**
2	8.44	0.9	1.550	8.033	10.603	11.951	14.035	16.362	18.197
3	8.44	0.9	1.838	7.872	10.388	11.730	13.777	16.058	17.875

Figure 24 - Rainfall intensity calculations



Q - Pre Development Conditions					
Q 2-yr (cfs)	Q 5-yr (cfs)	Q 10-yr (cfs)	Q 25-yr (cfs)	Q 50-yr (cfs)	Q 100-yr (cfs)
46.458	61.303	69.229	**81.317**	94.775	105.506
34.841	45.859	52.935	**62.350**	72.487	81.570
40.598	53.487	61.086	**71.840**	83.616	93.595

Q - Post Development Conditions					
Q 2-yr (cfs)	Q 5-yr (cfs)	Q 10-yr (cfs)	Q 25-yr (cfs)	Q 50-yr (cfs)	Q 100-yr (cfs)
52.917	69.726	79.548	**93.539**	108.886	121.818
61.020	80.544	90.779	**106.610**	124.285	138.227
59.797	78.905	89.098	**104.654**	121.976	135.780

Figure 25 - flow calculations

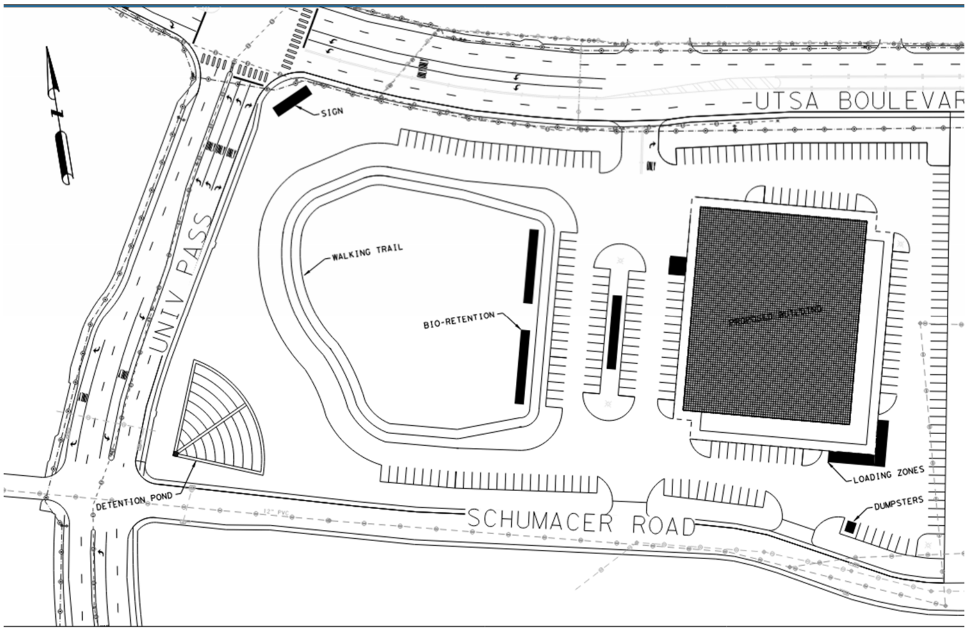
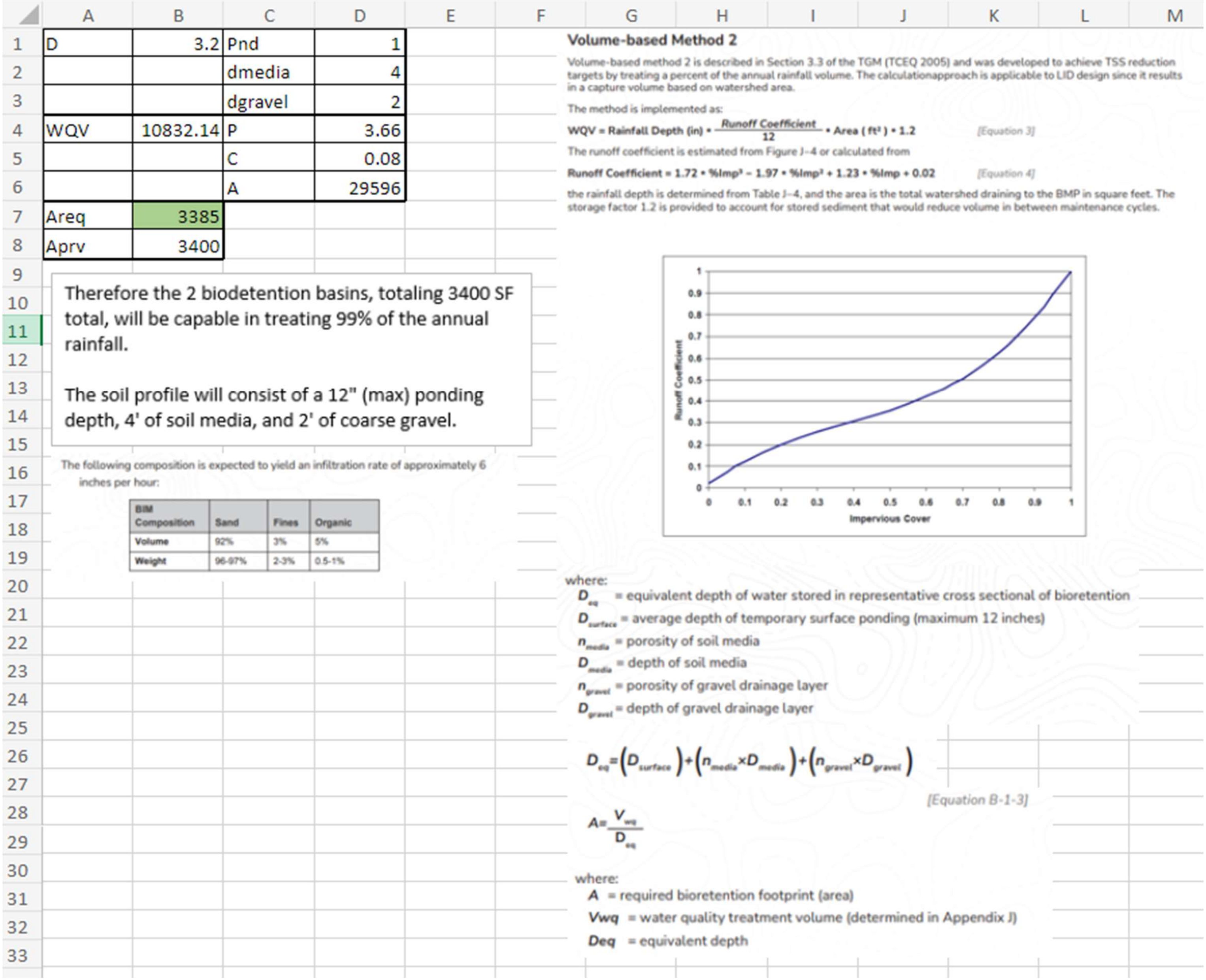


Figure 26 - site plan

Appendix A



	A	B	C	D
1	D	3.2	Pnd	1
2			dmedia	4
3			dgravel	2
4	WQV	10832.14	P	3.66
5			C	0.08
6			A	29596
7	Areq	3385		
8	Aprv	3400		

Therefore the 2 biodetention basins, totaling 3400 SF total, will be capable in treating 99% of the annual rainfall.

The soil profile will consist of a 12" (max) ponding depth, 4' of soil media, and 2' of coarse gravel.

The following composition is expected to yield an infiltration rate of approximately 6 inches per hour:

BIM Composition	Sand	Fines	Organic
Volume	92%	3%	5%
Weight	96-97%	2-3%	0.5-1%

Volume-based Method 2

Volume-based method 2 is described in Section 3.3 of the TGM (TCEQ 2005) and was developed to achieve TSS reduction targets by treating a percent of the annual rainfall volume. The calculationapproach is applicable to LID design since it results in a capture volume based on watershed area.

The method is implemented as:

$$\textbf{WQV} = \textbf{Rainfall Depth (in)} \cdot \frac{\textit{Runoff Coefficient}}{12} \cdot \textbf{Area (ft}^2\textbf{)} \cdot 1.2 \qquad \text{[Equation 3]}$$

The runoff coefficient is estimated from Figure J-4 or calculated from

$$\textbf{Runoff Coefficient} = 1.72 \cdot \%Imp^3 - 1.97 \cdot \%Imp^2 + 1.23 \cdot \%Imp + 0.02 \qquad \text{[Equation 4]}$$

the rainfall depth is determined from Table J-4, and the area is the total watershed draining to the BMP in square feet. The storage factor 1.2 is provided to account for stored sediment that would reduce volume in between maintenance cycles.

where:

D_{eq} = equivalent depth of water stored in representative cross sectional of bioretention
$D_{surface}$ = average depth of temporary surface ponding (maximum 12 inches)
n_{media} = porosity of soil media
D_{media} = depth of soil media
n_{gravel} = porosity of gravel drainage layer
D_{gravel} = depth of gravel drainage layer

$$D_{eq} = \left(D_{surface}\right) + \left(n_{media} \times D_{media}\right) + \left(n_{gravel} \times D_{gravel}\right)$$

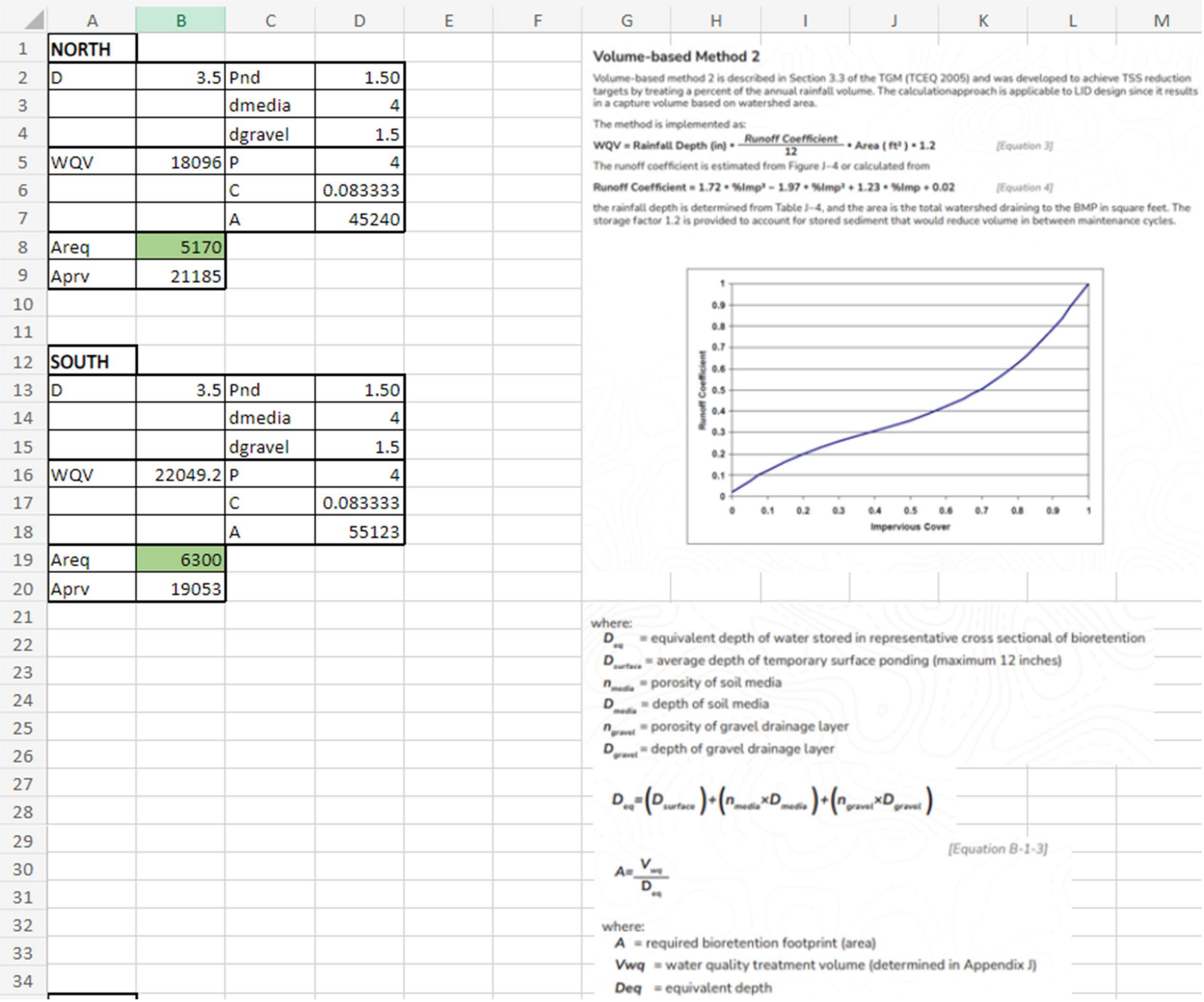
[Equation B-1-3]

$$A = \frac{V_{wq}}{D_{eq}}$$

where:

A = required bioretention footprint (area)
Vwq = water quality treatment volume (determined in Appendix J)
Deq = equivalent depth

Appendix B



	A	B	C	D
1	**NORTH**			
2	D	3.5	Pnd	1.50
3			dmedia	4
4			dgravel	1.5
5	WQV	18096	P	4
6			C	0.083333
7			A	45240
8	Areq	5170		
9	Aprv	21185		
10				
11				
12	**SOUTH**			
13	D	3.5	Pnd	1.50
14			dmedia	4
15			dgravel	1.5
16	WQV	22049.2	P	4
17			C	0.083333
18			A	55123
19	Areq	6300		
20	Aprv	19053		

Volume-based Method 2

Volume-based method 2 is described in Section 3.3 of the TGM (TCEQ 2005) and was developed to achieve TSS reduction targets by treating a percent of the annual rainfall volume. The calculationapproach is applicable to LID design since it results in a capture volume based on watershed area.

The method is implemented as:

$$\text{WQV} = \text{Rainfall Depth (in)} \cdot \frac{\text{Runoff Coefficient}}{12} \cdot \text{Area}(\text{ft}^2) \cdot 1.2 \qquad \text{[Equation 3]}$$

The runoff coefficient is estimated from Figure J-4 or calculated from

$$\text{Runoff Coefficient} = 1.72 \cdot \%\text{Imp}^3 - 1.97 \cdot \%\text{Imp}^2 + 1.23 \cdot \%\text{Imp} + 0.02 \qquad \text{[Equation 4]}$$

the rainfall depth is determined from Table J-4, and the area is the total watershed draining to the BMP in square feet. The storage factor 1.2 is provided to account for stored sediment that would reduce volume in between maintenance cycles.

where:

D_{eq} = equivalent depth of water stored in representative cross sectional of bioretention
$D_{surface}$ = average depth of temporary surface ponding (maximum 12 inches)
n_{media} = porosity of soil media
D_{media} = depth of soil media
n_{gravel} = porosity of gravel drainage layer
D_{gravel} = depth of gravel drainage layer

$$D_{eq} = \left(D_{surface}\right) + \left(n_{media} \times D_{media}\right) + \left(n_{gravel} \times D_{gravel}\right)$$

[Equation B-1-3]

$$A = \frac{V_{wq}}{D_{eq}}$$

where:

A = required bioretention footprint (area)
Vwq = water quality treatment volume (determined in Appendix J)
Deq = equivalent depth

Appendix C

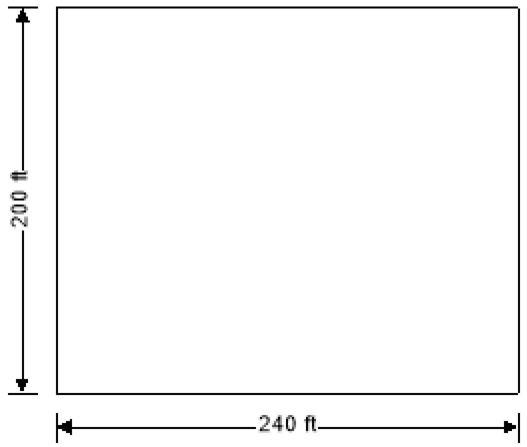
Tekla Tedds Alpha Consulting Engineers	Project				Job Ref.	
	Section				Sheet no./rev. 1	
	Calc. by A	Date 9/20/2023	Chk'd by	Date	App'd by	Date

WIND LOADING

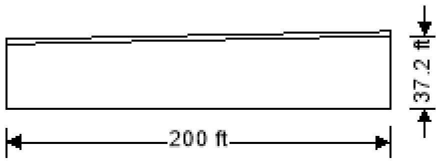
In accordance with ASCE7-16

Using the components and cladding design method

Tedds calculation version 2.1.14



Plan



Elevation

Building data

Type of roof	Monoslope
Length of building	b = **240.00** ft
Width of building	d = **200.00** ft
Height to eaves	H = **33.00** ft
Pitch of roof	α_0 = **1.2** deg
Height of parapet	h_p = **2.50** ft
Mean height	h = **33.00** ft
End zone width	$a = \max(\min(0.1\times\min(b, d), 0.4\times h), 0.04\times\min(b, d), 3ft)$ = **13.20** ft

General wind load requirements

Basic wind speed	V = **107.0** mph
Risk category	II
Velocity pressure exponent coef (Table 26.6-1)	K_d = **0.85**
Ground elevation above sea level	z_{gl} = **0** ft
Ground elevation factor	$K_e = \exp(-0.0000362 \times z_{gl}/1ft)$ = **1.00**
Exposure category (cl 26.7.3)	B
Enclosure classification (cl.26.12)	Enclosed buildings
Internal pressure coef +ve (Table 26.13-1)	GC_{pi_p} = **0.18**
Internal pressure coef –ve (Table 26.13-1)	GC_{pi_n} = **-0.18**
Parapet internal pressure coef +ve (Table 26.11-1)	GC_{pi_pp} = **0.18**
Parapet internal pressure coef –ve (Table 26.11-1)	GC_{pi_np} = **-0.18**
Gust effect factor	G_f = **0.85**

Topography

Topography factor not significant	K_{zt} = 1.0

Velocity pressure

Velocity pressure coefficient (Table 26.10-1) K_z = **0.72**

Velocity pressure $q_h = 0.00256 \times K_z \times K_{zt} \times K_d \times K_e \times V^2 \times 1\text{psf/mph}^2$ = **17.9** psf

Velocity pressure at parapet

Velocity pressure coefficient (Table 26.10-1) K_z = **0.73**

Velocity pressure $q_p = 0.00256 \times K_z \times K_{zt} \times K_d \times K_e \times V^2 \times 1\text{psf/mph}^2$ = **18.3** psf

Peak velocity pressure for internal pressure

Peak velocity pressure – internal (as roof press.) q_i = **17.89** psf

Equations used in tables

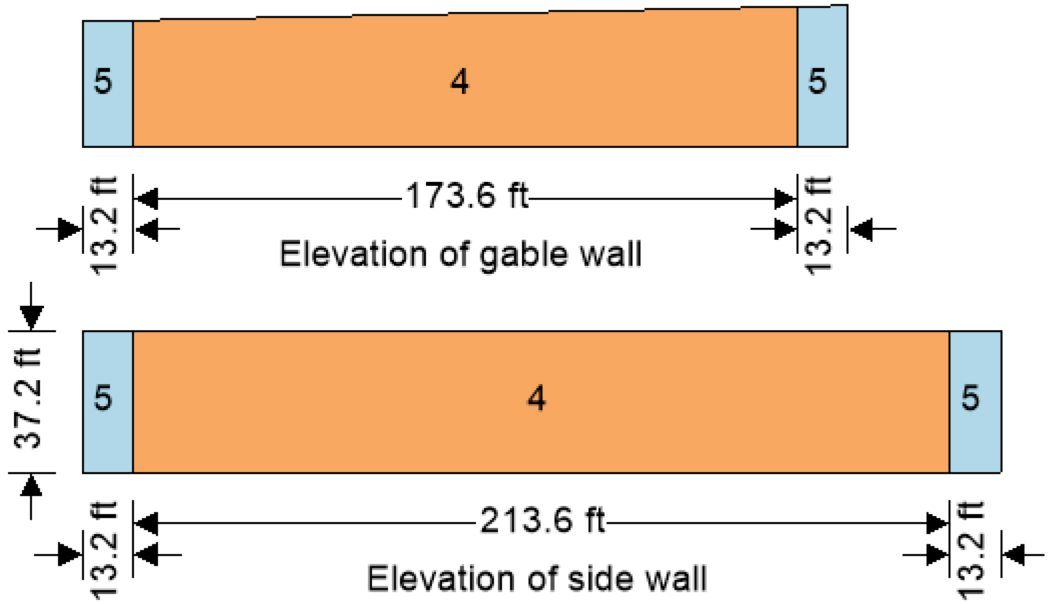
Net pressure $p = q_h \times [GC_p - GC_{pi}]$

Parapet net pressure $p = q_p$ x $[GC_p - GC_{pi\_p}]$

Components and cladding pressures - Wall (Table 30.3-1 and Figure 30.3-2A)

Component	Zone	Height (ft)	V press. (psf)	Length (ft)	Width (ft)	Effect Area (ft²)	+GC$_p$	-GC$_p$	Pres (+ve) (psf)	Pres (-ve) (psf)
<=10 sf	4	33.0	17.9	-	-	10.0	0.90	-0.99	19.3	-20.9
50 sf	4	33.0	17.9	-	-	50.0	0.79	-0.88	17.3	-18.9
200 sf	4	33.0	17.9	-	-	200.0	0.69	-0.78	15.6 #	-17.2
>500 sf	4	33.0	17.9	-	-	500.1	0.63	-0.72	14.5 #	-16.1
<=10 sf	5	33.0	17.9	-	-	10.0	0.90	-1.26	19.3	-25.8
50 sf	5	33.0	17.9	-	-	50.0	0.79	-1.04	17.3	-21.8
200 sf	5	33.0	17.9	-	-	200.0	0.69	-0.85	15.6 #	-18.4
>500 sf	5	33.0	17.9	-	-	500.1	0.63	-0.72	14.5 #	-16.1

The final net design wind pressure, including all permitted reductions, used in the design shall not be less than 16psf acting in either direction

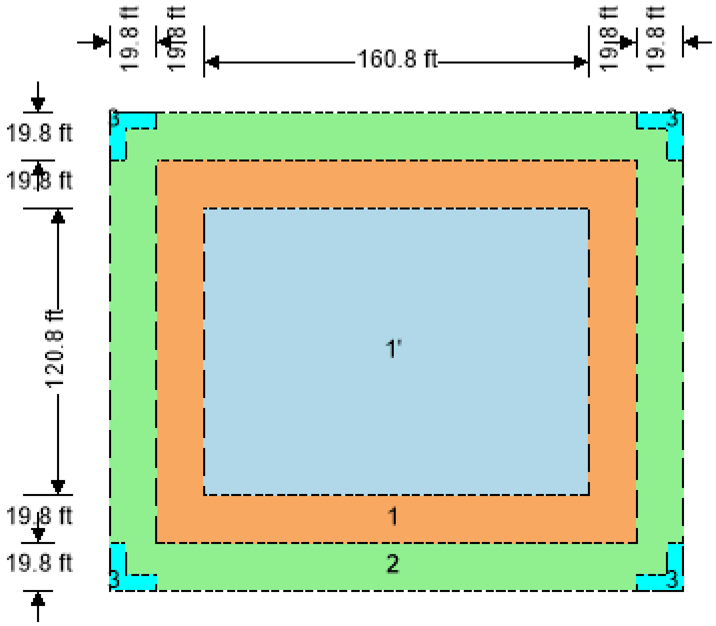


Components and cladding pressures - Roof (Figure 30.3-2A)

Component	Zone	Length (ft)	Width (ft)	Eff. area (ft^2)	+GC$_p$	-GC$_p$	Pres (+ve) (psf)	Pres (-ve) (psf)
<=10 sf	1	-	-	10.0	0.30	-1.70	8.6 #	-33.6
100 sf	1	-	-	100.0	0.20	-1.29	6.8 #	-26.3
200 sf	1	-	-	200.0	0.20	-1.16	6.8 #	-24.0
>500 sf	1	-	-	500.1	0.20	-1.00	6.8 #	-21.1
<=10 sf	1'	-	-	10.0	0.30	0.00	8.6 #	-3.2 #
100 sf	1'	-	-	100.0	0.20	0.00	6.8 #	-3.2 #
500 sf	1'	-	-	500.0	0.20	0.00	6.8 #	-3.2 #
>1000 sf	1'	-	-	1000.1	0.20	0.00	6.8 #	-3.2 #
<=10 sf	2	-	-	10.0	0.30	-2.30	8.6 #	-44.4
100 sf	2	-	-	100.0	0.20	-1.77	6.8 #	-34.9
200 sf	2	-	-	200.0	0.20	-1.61	6.8 #	-32.0
>500 sf	2	-	-	500.1	0.20	-1.40	6.8 #	-28.3
<=10 sf	3	-	-	10.0	0.30	-3.20	8.6 #	-60.5
100 sf	3	-	-	100.0	0.20	-2.14	6.8 #	-41.5
200 sf	3	-	-	200.0	0.20	-1.82	6.8 #	-35.8
>500 sf	3	-	-	500.1	0.20	-1.40	6.8 #	-28.3

The final net design wind pressure, including all permitted reductions, used in the design shall not be less than 16psf acting in either direction

Tekla Tedds Alpha Consulting Engineers	Project			Job Ref.	
	Section			Sheet no./rev. 4	
	Calc. by A	Date 9/20/2023	Chk'd by	Date	App'd by	Date



Plan on roof

Appendix D

• Metal Roof Deck Sizing

[Deck will be spanning 6'-0" between joists]

Roof Live Load = 20 psf
Roof Dead Load ≈ 25 psf

(
Metal Deck 3 psf
Joists 2 psf
Beams 2 psf
MEP 10 psf
Cieling/Insulation 8 psf
)

Technically, only the live load and deck weight applies

Assuming Fixed Ends:

$W = 1.2(25) + 1.6(20) = 62$ psf

$M_u^- = \frac{Wl^2}{12} = \frac{62(6)^2}{12} = 186$ lb-ft/ft

$M_u^+ = \frac{Wl^2}{24} = \frac{62(6)^2}{24} = 93$ lb-ft/ft

$R_u = V_u = Wl/2 = 62(6)/2 = 186$ lb/ft

Assuming Simply Supported:

$M_u^+ = Wl^2/8 = 62(6)^2/8 = 279$ lb-ft/ft

Using 1.5B-36 Gr50 22-Gage Deck:

$\phi V_n = 4035$ lb/ft $> V_u = 186$ lb/ft ✓

$\phi M_n^+ = 634$ lb-ft/ft > 279 lb-ft/ft $= M_u^+$ ✓
$\phi M_n^- = 671$ lb-ft/ft > 186 lb-ft/ft $= M_u^-$ ✓

$\phi W_n = 141$ psf $> W_u = 62$ psf ✓

(6'-0", single span)

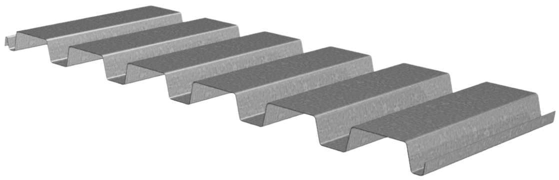
$L/240 = 47$ psf max live > 20 psf Roof Live ✓

1.5B-36/1.5BI-36/1.5PLB-36 ROOF DECKS
GRADE 50 STEEL

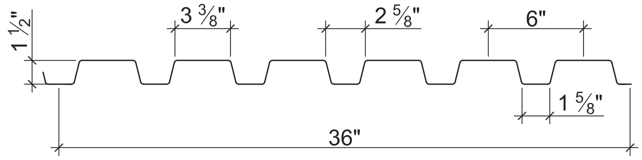
LRFD

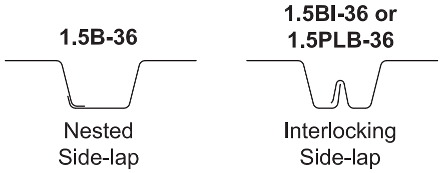
1.5B ROOF DECKS

- **1.5B-36 Deck used with Side-lap Screws**
- **1.5BI-36 Deck used with TSWs or BPs**
- **1.5PLB-36 Deck used with PunchLok® II System**



Nominal Dimensions





Section Properties

Deck Gage	Deck Weight w_{dd} (psf)	Base Metal Thickness t (in.)	Yield Strength F_y (ksi)	Effective Moment of Inertia at Service Load $I_d = (2I_e+I_g)/3$ I_d+ (in⁴/ft)	I_d- (in⁴/ft)	Effective Section Modulus at F_y = 50 ksi S_e+ (in³/ft)	S_e- (in³/ft)	Design Moment øM$_n$+ (lb-ft/ft)	øM$_n$- (lb-ft/ft)	Vertical Web Shear øV$_n$ (lb/ft)
22	1.6	0.0295	50	0.155	0.178	0.169	0.179	634	671	4035
20	2.0	0.0358	50	0.197	0.217	0.224	0.229	840	859	4874
19	2.3	0.0418	50	0.239	0.257	0.266	0.278	997	1042	5666
18	2.6	0.0474	50	0.277	0.290	0.306	0.318	1148	1193	6398
16	3.3	0.0598	50	0.364	0.367	0.393	0.402	1474	1508	7996

Design Reactions at Supports Based on Web Crippling, øR$_n$ (lb/ft)

Bearing Length of Webs

Deck Gage	One-Flange Loading						Two-Flange Loading					
	End Bearing				Interior Bearing		End Bearing				Interior Bearing	
	1½"	2"	3"	4"	3"	4"	1½"	2"	3"	4"	3"	4"
22	1235	1357	1563	1706	2204	2383	1289	1389	1556	1672	2728	2966
20	1763	1932	2215	2408	3164	3406	1949	2093	2333	2497	3960	4286
19	2344	2562	2927	3169	4222	4527	2702	2893	3213	3426	5324	5740
18	2954	3221	3669	3959	5334	5699	3515	3754	4156	4417	6762	7265
16	4525	4915	5568	5967	8206	8709	5681	6043	6651	7023	10487	11191

Standard Features

- ASTM A653 SS GR50 Min., with G60 or G90, white or gray primer optional
- ASTM A1008 SS GR50 Min. with gray primer
- Standard lengths – 6’-0” to 42’-0”
- IAPMO UES ER-0652, UL, and FM Listed
- Tables conform to ANSI/SDI RD-2017

Optional Features

- Inquire regarding cost and lead times for:
  - -Short cuts < 6’-0”
  - -Sheet Lengths > 42’-0”
  - -Alternative metallic and painted finishes
- Web Perforated Acoustical Versions

1.5B-36/1.5BI-36/1.5PLB-36 ROOF DECKS
GRADE 50 STEEL

Inward Uniform Design Loads, LRFD (psf)

Deck Gage	Spans	Criteria	Span (ft-in.)										
			2'-0"	3'-0"	4'-0"	5'-0"	6'-0"	7'-0"	8'-0"	9'-0"	10'-0"	11'-0"	12'-0"
22	Single	øW_n	1267	563	317	203	141	103	79	63	51	42	35
		L/240	1270	376	159	81	47	30	20	14	10	8	6
	Double	øW_n	1240	575	329	212	148	109	83	66	54	44	37
		L/240	3514	1041	439	225	130	82	55	39	28	21	16
	Triple	øW_n	1502	708	407	263	184	136	104	82	67	55	46
		L/240	2754	816	344	176	102	64	43	30	22	17	13
20	Single	øW_n	1679	746	420	269	187	137	105	83	67	56	47
		L/240	1614	478	202	103	60	38	25	18	13	10	7
	Double	øW_n	1572	732	419	271	189	139	107	84	68	57	48
		L/240	4283	1269	535	274	159	100	67	47	34	26	20
	Triple	øW_n	1898	900	519	336	235	173	133	105	85	71	59
		L/240	3357	995	420	215	124	78	52	37	27	20	16
19	Single	øW_n	1994	886	499	319	222	163	125	98	80	66	55
		L/240	1958	580	245	125	73	46	31	21	16	12	9
	Double	øW_n	1894	886	508	328	229	169	129	102	83	69	58
		L/240	5073	1503	634	325	188	118	79	56	41	30	23
	Triple	øW_n	2281	1087	628	407	285	210	161	128	104	86	72
		L/240	3976	1178	497	254	147	93	62	44	32	24	18
18	Single	øW_n	2295	1020	574	367	255	187	143	113	92	76	64
		L/240	2270	673	284	145	84	53	35	25	18	14	11
	Double	øW_n	2162	1012	581	375	262	193	148	117	95	79	66
		L/240	5724	1696	716	366	212	134	89	63	46	34	27
	Triple	øW_n	2602	1242	718	465	326	240	185	146	119	98	82
		L/240	4487	1329	561	287	166	105	70	49	36	27	21
16	Single	øW_n	2948	1310	737	472	328	241	184	146	118	97	82
		L/240	2983	884	373	191	110	70	47	33	24	18	14
	Double	øW_n	2727	1278	734	474	331	244	187	148	120	99	83
		L/240	7244	2146	906	464	268	169	113	79	58	44	34
	Triple	øW_n	3280	1567	907	588	412	304	233	185	150	124	104
		L/240	5678	1682	710	363	210	132	89	62	45	34	26

Note:

1. Table does not account for web crippling. Required bearing should be determined based on specific span conditions.

Appendix E

COMPOSITE DECK-SLAB SUPERIMPOSED LOAD

Keep up with Vulcraft/Verco by following us at https://www.linkedin.com/company/vulcraft--division-of-nucor-corp-/

Composite Deck-Slab Strength

Input Design Criteria

NUCOR VULCRAFT
Print

Design of Composite Deck-Slab Strength

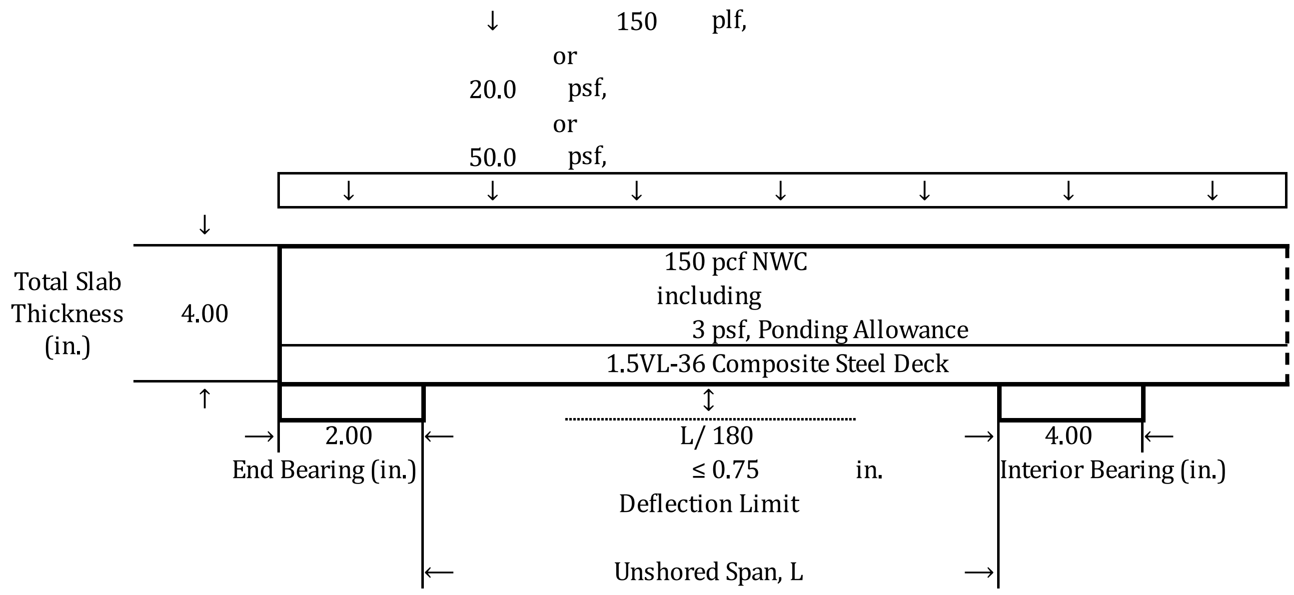
Unit System		Imperial	
Design Method		LRFD	
Deck Option		Composite	
Deck Type		1.5VL-36	
Total Slab Thickness (in.)	3.5 ≤	4	≤ 7.5
Structural Concrete Unit Weight (pcf)		150	≥ 90
Structural Concrete Strength (psi)	2500 psi ≤	4000	≤ 6000 psi
Deflection Limit	L /	360	

Design for Maximum Unshored Span of Composite Steel Deck

Construction Deflection Limit	L /	180	
Const. Deflection not to exceed (in.)		0.75	≤ 0.75
End Bearing (in.)		2.00	≥ 0.75
Interior Bearing (in.)		4.00	≥ 0.75
Concrete Ponding Allowance (psf)		3.00	
Construction Concentrated Load (plf)		150.00	≥ 150 plf
Construction Live Load with Concrete (psf)		20.00	≥ 20 psf
Construction Live Load without Concrete (psf)		50	≥ 50 psf

Superimposed Live Load Table Range

Start Table at Span of (ft)	7.00
Spans Increment at (ft)	1.00



1.5VL-36 Composite Steel Deck-Slab (LRFD)

with 4 in. 150 pcf 4000 psi NWC

NUCOR VULCRAFT

Maximum Unshored Span

Gage	1 Span	2 Span	3 Span
22	6'-1"	7'-2"	7'-3"
20	7'-4"	8'-5"	8'-8"
19	7'-10"	9'-3"	9'-7"
18	8'-3"	9'-11"	10'-3"
16	9'-0"	11'-1"	11'-1"

Maximum Unshored Span based on:

Construction Live Load w/ Concrete	20.00	psf			
Construction	50.00	psf	Minimum End Bearing	2.00	in.
Concentrated Construction Load	150.00	plf	Minimum Interior Bearing	4.00	in.
Concrete Ponding Allowance	3.00	psf	Maximum Deflection L/	180	≤ 0.75 in.
Concrete Volume	0.94	yd³ / 100 ft² (Note: Does not include allowance for ponding)			

Composite Steel Deck Properties (steel deck only)

Gage	F_y ksi	w_{dd} psf	S_e^+ in ^{3}/ft	S_e^- in ^{3}/ft	I_d^+ in ^{4}/ft	I_d^- in ^{4}/ft	ϕVn kip/ft

18	50	2.60	0.306	0.318	0.277	0.290	6.398
16	50	3.30	0.393	0.402	0.364	0.367	7.996

Superimposed Design Load, ϕWn, / Deflection at L/360, psf[1]

Gage	7'-0"	8'-0"	9'-0"	10'-0"	11'-0"	12'-0"	13'-0"	14'-0"	15'-0"
22	510/565	380/379	290/266	226/194	178/145	142/112	114/88	92/70	74/57
20	613/606	458/406	352/285	276/208	220/156	177/120	143/94	117/75	96/61
19	710/643	532/431	410/302	323/220	258/165	209/127	171/100	141/80	116/65
18	796/675	598/452	462/317	365/231	293/174	238/134	196/105	162/84	135/68
16	979/741	738/496	572/348	454/254	367/191	300/147	248/115	207/92	174/75

Notes: [1] For high loads, long term concrete creep should be considered.

Composite Steel Deck-Slab Properties							Min. Temperature & Shrinkage	
Gage	w_1 psf	Ic in.^{4}/ft	Iu in.^{4}/ft	Id [1] in.^{4}/ft	ϕMno kip-ft/ft	ϕVno kip/ft	A_s min [2] in.^{2}/ft	or Dramix® Steel Fiber 4D 65/60BG, lbs/cy
22	39.5	2.79	6.09	4.44	3.42	4.18	0.028	18
20	39.9	3.21	6.31	4.76	4.05	4.18	0.028	18
19	40.2	3.59	6.51	5.05	4.65	4.18	0.028	18
18	40.5	3.91	6.69	5.3	5.17	4.18	0.028	18
16	41.2	4.56	7.08	5.82	6.3	4.18	0.028	18

Notes: [1] $I_d = (I_c + I_u)/2$
[2] Minimum area of steel for temperature and shrinkage

Composite Deck-Slab V4.0 is based on: Date: 11/16/2023
ANSI/SDI C-2017, IAPMO UES ER-0652, and IAPMO UES ER-0423

• Second Floor Composite Deck Sizing

Second Floor Live Load : 100 psf Roof Balcony/Assembly (Controls)

Dead Load on Deck : $150(3''/12) = 37.5$ psf (Concrete)
$+10$ psf (Tile)

Assuming Fixed Ends:

$W = 1.2(37.5+10) + 1.6(100) = 217$ psf

$M_u^- = \frac{Wl^2}{12} = \frac{(217)(10)^2}{12} = 1.81$ k-ft/ft

$M_u^+ = \frac{Wl^2}{24} = \frac{(217)(10)^2}{24} = 0.91$ k-ft/ft

$R_u = V_u = Wl/2 = (217)(10)/2 = 1.09$ kips/ft

Assuming Simply Supported:

$M_u^+ = Wl^2/8 = (217)(10)^2/8 = 2.71$ k-ft/ft

Using 1.5VL-36 4" Deck (18 Gage):

[Triple-Span condition needed for 10'-0" unshored]

$\phi W_n = 365$ psf $> W_u = 217$ psf ✓

$\phi M_n = 5.17$ k-ft/ft $> M_u^+ = 2.71$ k-ft/ft $> M_u^- = 1.81$ k-ft/ft ✓

$\phi V_n = 4.18$ kips/ft $> V_u = 1.09$ kips/ft ✓

Appendix F

• Second Floor Composite Deck Sizing

Second Floor Live Load : 100 psf Roof Balcony/Assembly
(Controls)

Dead Load on Deck : $150(3''/12) = 37.5$ psf (Concrete)
+10 psf (Tile)

Assuming Fixed Ends:

$W = 1.2(37.5+10) + 1.6(100) = 217$ psf

$M_u^- = \frac{Wl^2}{12} = \frac{(217)(10)^2}{12} = 1.81$ k-ft/ft

$M_u^+ = \frac{Wl^2}{24} = \frac{(217)(10)^2}{24} = 0.91$ k-ft/ft

$R_u = V_u = Wl/2 = (217)(10)/2 = 1.09$ kips/ft

Assuming Simply Supported:

$M_u^+ = Wl^2/8 = (217)(10)^2/8 = 2.71$ k-ft/ft

Using 1.5VL-36 4" Deck (18 Gage):

[Triple-Span condition needed for 10'-0" unshored]

$\phi W_n = 365$ psf $> W_u = 217$ psf ✓

$\phi M_n = 5.17$ k-ft/ft $> M_u^+ = 2.71$ k-ft/ft $> M_u^- = 1.81$ k-ft/ft ✓

$\phi V_n = 4.18$ kips/ft $> V_u = 1.09$ kips/ft ✓

Appendix G

Gravity Column Design

RAM Steel 23.00.00.92
DataBase: Roadrunner Executive Tower 11/17/23 17:00:14
Building Code: IBC Steel Code: AISC360-16 LRFD

Story level Roof, Column Line 6-D, Column # 111

Fy (ksi)	=	50.00	Column Size	=	HSS8X8X1/4
Orientation (deg.)	=	0.0			

INPUT DESIGN PARAMETERS:

		X-Axis	Y-Axis
Lu (ft)		15.00	15.00
K		1	1
Braced Against Joint Translation		Yes	Yes
Column Eccentricity (in)	Top	6.50	6.50
	Bottom	6.50	6.50

CONTROLLING AXIAL COLUMN LOADS - Skip-Load Case 1:

	Dead	Live	Roof
Axial (kip)	18.10	0.00	9.00

DEMAND CAPACITY RATIO: (1.2DL + 1.6RF)

Pu (kip)	=	36.11	0.90Pnx (kip)	=	251.85	Pu/0.90Pnx	=	0.143
			0.90Pny (kip)	=	251.85	Pu/0.90Pny	=	0.143
			0.90Pn (kip)	=	251.85	Pu/0.90Pn	=	0.143

DEMAND/CAPACITY LIMIT FOR STRENGTH : 1.000

CONTROLLING COMBINED COLUMN LOADS - Skip-Load Case 2:

			Dead	Live	Roof
Axial (kip)			18.10	0.00	9.00
Moments	Top	Mx (kip-ft)	0.61	0.00	0.32
		My (kip-ft)	0.00	0.00	0.00
	Bot	Mx (kip-ft)	0.00	5.91	0.00
		My (kip-ft)	0.00	2.95	0.00

Reverse curvature about X-Axis
Single curvature about Y-Axis

CALCULATED PARAMETERS: (1.2DL + 1.6LL + 0.5RF)

Pu (kip)	=	26.21	0.90*Pn (kip)	=	251.85
Mux (kip-ft)	=	9.45	0.90*Mnx (kip-ft)	=	70.10
Muy (kip-ft)	=	4.73	0.90*Mny (kip-ft)	=	70.10
Rm	=	1.00			
Cbx	=	1.78	Cby	=	1.67
Cmx	=	0.56	Cmy	=	0.60
Pex (kip)	=	624.56	Pey (kip)	=	624.56
B1x	=	1.00	B1y	=	1.00

INTERACTION EQUATION

Pu/0.90*Pn = 0.104

Eq H1-1b: 0.052 + 0.135 + 0.067 = 0.254

Story level Second, Column Line 6-D, Column # 111

Fy (ksi)	= 50.00	Column Size	= HSS8X8X1/4
Orientation (deg.)	= 0.0		

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu (ft)	18.00	18.00
K	1	1
Braced Against Joint Translation	Yes	Yes
Column Eccentricity (in) Top	6.50	6.50
Bottom	6.50	6.50

CONTROLLING AXIAL COLUMN LOADS - Skip-Load Case 1:

	Dead	Live	Roof
Axial (kip)	70.57	60.00	9.00

DEMAND CAPACITY RATIO: (1.2DL + 1.6LL + 0.5RF)

Pu (kip) =	185.18	0.90Pnx (kip) =	226.82	Pu/0.90Pnx	=	0.816
		0.90Pny (kip) =	226.82	Pu/0.90Pny	=	0.816
		0.90Pn (kip) =	226.82	Pu/0.90Pn	=	0.816

DEMAND/CAPACITY LIMIT FOR STRENGTH : 1.000

CONTROLLING COMBINED COLUMN LOADS - Skip-Load Case 6:

	Dead	Live	Roof
Axial (kip)	70.57	50.00	9.00
Moments Top Mx (kip-ft)	0.00	0.00	0.00
My (kip-ft)	0.00	2.46	0.00
Bot Mx (kip-ft)	0.00	0.00	0.00
My (kip-ft)	0.00	0.00	0.00

Single curvature about X-Axis
Single curvature about Y-Axis

CALCULATED PARAMETERS: (1.2DL + 1.6LL + 0.5RF)

Pu (kip)	=	169.18	0.90*Pn (kip)	=	226.82
Mux (kip-ft)	=	0.00	0.90*Mnx (kip-ft)	=	70.10
Muy (kip-ft)	=	3.94	0.90*Mny (kip-ft)	=	70.10
Rm	=	1.00			
Cbx	=	1.00	Cby	=	1.67
Cmx	=	0.60	Cmy	=	0.60
Pex (kip)	=	433.72	Pey (kip)	=	433.72
B1x	=	1.00	B1y	=	1.00

INTERACTION EQUATION

Pu/0.90*Pn = 0.746

Eq H1-1a: 0.746 + 8/9(0.000 + 0.056) = 0.796

• Column Spot-Check

Checking Column D6 (Gridlines D and 6):

Trib width along X = $\frac{30' + 30'}{2} = 30 \text{ ft}$

Trib width along Y = $\frac{25' + 25'}{2} = 25 \text{ ft}$

Total tributary area = $30' \times 25' = 750 \text{ ft}^2$

Dead Load = 25 (Roof)
+15 (Second Floor)
+ 150(4/12) (Concrete on composite deck)
= 90 psf

Live Load = 80 psf (Second Floor/Corridor)

Roof Live Load = 20 psf

$W = 1.2(90) + 1.6(80) + 0.5(20) = 246$ psf ✓

$W \times A = 246 \text{ psf} \times 750 \text{ ft}^2 = 184500 \text{ lbs} = 184.5 \text{ kips} \approx 185 \text{ kips} = P_u$

Calculated axial load = 184.5 kips
Computer axial load = 185.2 kips

Table 4-4 of AISC: (Foundation to 2nd Floor)

Assuming HSS 8×8×1/4 with $L_c = 18' \rightarrow \phi P_n = 227$ kips

Interaction: $P_u / \phi P_n = 184.5/227 = 0.813 \approx 0.816$ ✓

Calculated Computer

Appendix H

Gravity Beam Design

RAM Steel 23.00.00.92
DataBase: Roadrunner Executive Tower 11/17/23 02:40:28
Building Code: IBC Steel Code: AISC 360-16 LRFD

Floor Type: Second Floor **Beam Number = 302**

SPAN INFORMATION (ft): I-End (200.00,20.00) J-End (200.00,40.00)

Beam Size (Optimum) = W12X19 Fy = 50.0 ksi
Total Beam Length (ft) = 20.00

COMPOSITE PROPERTIES (Not Shored):

	Left	**Right**
Deck Label	Composite Deck	Composite Deck
Concrete thickness (in)	2.50	2.50
Unit weight concrete (pcf)	150.00	150.00
f'c (ksi)	4.00	4.00
Decking Orientation	perpendicular	perpendicular
Decking type	VULCRAFT 1.5VL	VULCRAFT 1.5VL

beff (in) = 60.00 Y bar(in) = 13.01
Mnf (kip-ft) = 218.56 Mn (kip-ft) = 161.00
C (kips) = 86.15 PNA (in) = 9.77
Ieff (in4) = 325.18 Itr (in4) = 480.93
Stud length (in) = 3.00 Stud diam (in) = 0.75
Stud Capacity (kips) Qn = 17.2 Rg = 1.00 Rp = 0.60
# of studs: Full = 34 Partial = 10 Actual = 10
Number of Stud Rows = 1 Percent of Full Composite Action = 30.93

LINE LOADS (k/ft):

Load	Dist	DL	CDL	LL	Red%	Type	PartL	CLL
1	0.000	0.405	0.405	0.000	---	NonR	0.000	0.000
	20.000	0.405	0.405	0.000			0.000	0.000
2	0.000	0.250	0.250	0.800	---	NonR	0.000	0.200
	20.000	0.250	0.250	0.800			0.000	0.200
3	0.000	0.019	0.019	0.000	---	NonR	0.000	0.000
	20.000	0.019	0.019	0.000			0.000	0.000

SHEAR (Ultimate): Max Vu (1.2DL+1.6LL) = 20.89 kips 1.00Vn = 86.01 kips

MOMENTS (Ultimate):

Span	Cond	LoadCombo	Mu kip-ft	@ ft	Lb ft	Cb	Phi	Phi*Mn kip-ft
Center	PreCmp+	1.2DL+1.6LL	56.4	10.0	0.0	1.00	0.90	92.62
	Init DL	1.4DL	47.2	10.0	---	---		
	Max +	1.2DL+1.6LL	104.4	10.0	---	---	0.90	144.90
Controlling		1.2DL+1.6LL	104.4	10.0	---	---	0.90	144.90

REACTIONS (kips):

	Left	**Right**
Initial reaction	8.74	8.74
DL reaction	6.74	6.74
Max +LL reaction	8.00	8.00
Max +total reaction (factored)	20.89	20.89

DEFLECTIONS: **Ratio**

Initial load (in)	at	10.00 ft	=	-0.643	L/D	=	373			
Live load (in)	at	10.00 ft	=	-0.305	L/D	=	786	>	360	0.46
Post Comp load (in)	at	10.00 ft	=	-0.305	L/D	=	786	>	240	0.31
Net Total load (in)	at	10.00 ft	=	-0.949	L/D	=	253	>	240	0.95

• Beam Spot Check

Checking example 20ft joist-beam on second floor:

Dead Load = 25 psf (Second Floor tile + MEP)
+ 40.5 psf (Weight from composite deck)

[Can be found in Appendix C]

Live Load = 80 psf

$W = 1.2(65.5) + 1.6(80) = 206.6$ psf

$W \times T_w = 206.6 \times 10$ ft $= 2066$ plf

10 ft spacing

✓

$M_u = \frac{wl^2}{8} = (2066)(20)^2/8 = 103.3$ k-ft ≈ 104.4 k-ft

(Simply Supported)

Calculated vs. Computer

From table 3-10 of AISC:

Computer

ϕM_n of W12×19 = 92.6 k-ft = ϕM_n = 92.62 k-ft ✓

The above capacity is for the W12×19 before it becomes composite.

I don't know how to calulate the capacity of a composite beam... :(

Appendix I

PROJECT DETAILS

INPUTS

f_c'	4000 psi
f_{yt}	60000 psi
h	25 in.
λ	1
N_u	0 lbs
Clear Cover	1.5 in.
d	23 in.
b_w	36 in.
#6 Bars =	6 bars
A_s =	2.64 in.^{2}

Positive for compression, negative for tension

Assuming (b_w/6) bars are being used

Assuming #6 bars are used

RESULTS

$\phi V_{n,max}$	ϕV_n @ $d/4$ "	Number of Legs		Max Spacing	$\phi V_{n,max}$ When $V_s \le 4\sqrt{f_c'}b_w d$	ϕV_n @ $d/2$"	Number of Legs		Max Spacing	Stop stirrups when $V_u \le$
		No. 3	No. 4				No. 3	No. 4		
314.28 *kips*	#3: 192.4 *kips* #4: 285.55 *kips*	5	5	*@5in.*	235.65 *kips*	#3: 109.6 *kips* #4: 135 *kips*	3	3	*@11in.*	35.01 *kips*

$s_{max} =$ Along length	$\frac{d}{2} \le 24in.$	[When: $V_s \le 4\sqrt{f_c'}b_w d$]	$s_{max} =$ 11 *in.*
	$\frac{d}{4} \le 12in.$	[When: $V_s > 4\sqrt{f_c'}b_w d$]	$s_{max} =$ 5 *in.*

-Reference code ACI 318-19 [9.7.6.2.2]

$\frac{A_{v,min}}{s} =$	Greater of:	$0.75\sqrt{f_c'}\frac{b_w}{f_{yt}} =$ 0.0285 *in.*	$\frac{A_{v,min}}{s} =$ 0.0300 *in.*
		$50\frac{b_w}{f_{yt}} =$ 0.0300 *in.*	

-Reference code ACI 318-19 [9.6.3.4]

# of stirrup legs based on $A_{v,min}$	[When: $V_s \le 4\sqrt{f_c'}b_w d$]	$\frac{A_{v,min}}{s} \times \frac{s_{max}}{A_{bar}} =$	No. 3 bar: 3 *legs* No. 4 bar: 2 *legs*
	[When: $V_s > 4\sqrt{f_c'}b_w d$]	$\frac{A_{v,min}}{s} \times \frac{s_{max}}{A_{bar}} =$	No. 3 bar: 2 *legs* No. 4 bar: 1 *legs*

# of stirrup legs based on s_{max} along width	[When: $V_s \le 4\sqrt{f_c'}b_w d$]	$d \le 24in.$	$s_{max} =$ 23 *in.*	$\#Legs = \frac{b_w}{s_{max}} + 1 =$ 3 *legs*
	[When: $V_s > 4\sqrt{f_c'}b_w d$]	$\frac{d}{2} \le 12in.$	$s_{max} =$ 11.5 *in.*	$\#Legs = \frac{b_w}{s_{max}} + 1 =$ 5 *legs*

-Reference code ACI 318-19 [9.7.6.2.2]

Final # of stirrup legs	[When: $V_s \le 4\sqrt{f_c'}b_w d$]	No. 3 bar: 3 *legs* No. 4 bar: 3 *legs*
	[When: $V_s > 4\sqrt{f_c'}b_w d$]	No. 3 bar: 5 *legs* No. 4 bar: 5 *legs*

$\phi V_n = \phi V_c + \phi V_s$ *-Reference code ACI 318-19 [22.5.1.1]*

$\phi =$ 0.75 *-Reference code ACI 318-19 [21.2.1]*

$\sqrt{f_c'} \le 100psi$	For strength calculations:	$\sqrt{f_c'} =$ 63.246 *psi*
$\frac{N_u}{6A_g} \le 0.05f_c'$	For strength calculations:	$\frac{N_u}{6A_g} =$ 0.000 *psi*

-Reference code ACI 318-19 [22.5.3.1]

-Reference code ACI 318-19 [22.5.5.1.2]

$\rho_w = \frac{A_s}{A_g} =$ 0.002933

$\lambda_s = \sqrt{\frac{2}{1+\frac{d}{10}}} \le 1$	$\lambda_s =$ 0.7785

-Reference code ACI 318-19 [22.5.5.1.3]

$V_c \le 5\lambda\sqrt{f_c'}b_w d =$ 261.84 *kips* *-Reference code ACI 318-19 [22.5.5.1.1]*

V_c	$A_v \ge A_{v,min}$	$\left[2\lambda\sqrt{f_c'} + \frac{N_u}{6A_g}\right]b_w d =$ 104.73 *kips*	$\phi V_c =$ 78.55 *kips*
	$A_v < A_{v,min}$	$\left[8\lambda_s\lambda(\rho_w)^{\frac{1}{3}}\sqrt{f_c'} + \frac{N_u}{6A_g}\right]b_w d =$ 46.69 *kips*	$\phi V_c =$ 35.02 *kips*

-Reference code ACI 318-19 [Table 22.5.5.1(a)]

-Reference code ACI 318-19 [Table 22.5.5.1(c)]

Max allowable strength from this cross-section	$\phi V_{n,max} = \phi V_c + \phi 8\sqrt{f_c'}b_w d =$ 314.28 *kips*

-Reference code ACI 318-19 [22.5.1.2]

[When: $V_s = 4\sqrt{f_c'}b_w d$]	$\phi V_n =$ 235.65 *kips*

-When demand exceeds this value, tighter spacing is required as per code ACI 318-19 [Table 9.7.6.2.2]

When $A_{v,min}$ is not required	$V_u \le \phi\lambda\sqrt{f_c'}b_w d =$ 39.28 *kips*

-Reference code ACI 318-19 [9.6.3.1]

Stop stirrups when: $V_u \le$ 35.01 *kips*

When $d/2$ " spacing is used	No. 3	$V_s = \frac{A_v f_{yt} d}{s} =$ 41.40 *kips*	$\phi V_n = \phi V_c + \phi V_s =$ 109.60 *kips*
	No. 4	$V_s = \frac{A_v f_{yt} d}{s} =$ 75.27 *kips*	$\phi V_n = \phi V_c + \phi V_s =$ 135.00 *kips*

-Reference code ACI 318-19 [22.5.8.5.3]

When *d/4* " spacing is used	No. 3	$V_s = \frac{A_v f_{yt} d}{s} =$	151.80 *kips*	$\phi V_n = \phi V_c + \phi V_s =$	192.40 *kips*
	No. 4	$V_s = \frac{A_v f_{yt} d}{s} =$	276.00 *kips*	$\phi V_n = \phi V_c + \phi V_s =$	285.55 *kips*

-Reference code ACI 318-19 [22.5.8.5.3]

For a concrete beam with 25" of height [$d=23"$; $f_c'=4ksi$; $f_{yt}=60ksi$]

b_w	$\phi V_{n,max}$ of section		ϕV_n @ $d/4"$		Number of Legs		Max Spacing	$\phi V_{n,max}$ When $V_s \leq 4\sqrt{f_c'}b_w d$		ϕV_n @ $d/2"$		Number of Legs		Max Spacing	Stop stirrups when $V_u \leq$	Min. number of #6 bars:	Min. A_s
			No.3	No.4	No. 3	No. 4				No.3	No.4	No. 3	No. 4				
8	69.8 *kips*	*Calculation required if V_u is between these two columns* ←————————→	62.9 *kips*		2		@5in.	52.3 *kips*	*Calculation required if V_u is between these two columns* ←————————→	38.1 *kips*		2		@11in.	8.7 *kips*	*2 Bars*	*0.88 in.^{2}*
21	183.3 *kips*		114.1 *kips*		3			137.4 *kips*		66.5 *kips*		2			21.3 *kips*	*4 Bars*	*1.76 in.^{2}*
24	209.5 *kips*		143.4 *kips*	209.5 *kips*	4	4		157.1 *kips*		83.4 *kips*	108.8 *kips*	3	3		23.3 *kips*	*4 Bars*	*1.76 in$^{\cdot 2}$*
30	261.9 *kips*			231.0 *kips*		4		196.3 *kips*			121.9 *kips*		3		29.1 *kips*	*5 Bars*	*2.20 in.^{2}*
36	314.2 *kips*			285.5 *kips*		5		235.6 *kips*			135.0 *kips*		3		35.0 *kips*	*6 Bars*	*2.64 in$^{\cdot 2}$*
42	366.6 *kips*			298.6 *kips*		5		274.9 *kips*			148.0 *kips*		3		40.8 *kips*	*7 Bars*	*3.08 in.^{2}*
48	419 *kips*			353.1 *kips*		6		314.2 *kips*			180.0 *kips*		4		46.6 *kips*	*8 Bars*	*3.52 in$^{\cdot 2}$*
54	471.4 *kips*			366.2 *kips*		6		353.4 *kips*			193.0 *kips*		4		52.5 *kips*	*9 Bars*	*3.96 in.^{2}*
60	523.8 *kips*			420.7 *kips*		7		392.7 *kips*			206.1 *kips*		4		58.3 *kips*	*10 Bars*	*4.4 in.^{2}*
					Use if V_u is between these two columns ←——— ———→							Use if V_u is between these two columns ←——— ———→					

PROJECT DETAILS

INPUTS

f_c'	4000 psi
f_{yt}	60000 psi
h	25 in.
λ	1
N_u	0 lbs
Clear Cover	1.5 in.
d	23 in.
b_w	8 in.
#6 Bars =	2 bars
A_s =	0.88 in.^{2}

*Positive for compression, negative for tension

*Assuming (b_w/6) bars are being used

*Assuming #6 bars are used

RESULTS

$\phi V_{n,max}$	ϕV_n @ $d/4$ "	Number of Legs		Max Spacing	$\phi V_{n,max}$ When $V_s \le 4\sqrt{f_c'}b_w d$	ϕV_n @ $d/2$"	Number of Legs		Max Spacing	Stop stirrups when $V_u \le$
		No. 3	No. 4				No. 3	No. 4		
69.84 *kips*	#3: 62.99 *kips*	2	2	*@5in.*	52.37 *kips*	#3: 38.15 *kips*	2	2	*@11in.*	8.72 *kips*
	#4: 69.84 *kips*					#4: 55.09 *kips*				

$s_{max} =$ Along length	$\frac{d}{2} \le 24in.$	[When: $V_s \le 4\sqrt{f_c'}b_w d$]	$s_{max} =$ 11 *in.*
	$\frac{d}{4} \le 12in.$	[When: $V_s > 4\sqrt{f_c'}b_w d$]	$s_{max} =$ 5 *in.*

-Reference code ACI 318-19 [9.7.6.2.2]

$\frac{A_{v\ min}}{s} =$	Greater of:	$0.75\sqrt{f_c'}\frac{b_w}{f_{yt}} =$ 0.0063 *in.*	$\frac{A_{v\ min}}{s} =$ 0.0067 *in.*
		$50\frac{b_w}{f_{vt}} =$ 0.0067 *in.*	

-Reference code ACI 318-19 [9.6.3.4]

# of stirrup legs based on $A_{v,min}$	[When: $V_s \le 4\sqrt{f_c'}b_w d$]	$\frac{A_{v\ min}}{s} \times \frac{s_{max}}{A_{bar}} =$	No. 3 bar: 1 *legs*
			No. 4 bar: 1 *legs*
	[When: $V_s > 4\sqrt{f_c'}b_w d$]	$\frac{A_{v\ min}}{s} \times \frac{s_{max}}{A_{bar}} =$	No. 3 bar: 1 *legs*
			No. 4 bar: 1 *legs*

# of stirrup legs based on s_{max} along width	[When: $V_s \le 4\sqrt{f_c'}b_w d$]	$d \le 24in.$	$s_{max} =$ 23 *in.*	$\#Legs = \frac{b_w}{s_{max}} + 1 =$ 2 *legs*
	[When: $V_s > 4\sqrt{f_c'}b_w d$]	$\frac{d}{2} \le 12in.$	$s_{max} =$ 11.5 *in.*	$\#Legs = \frac{b_w}{s_{max}} + 1 =$ 2 *legs*

-Reference code ACI 318-19 [9.7.6.2.2]

Final # of stirrup legs	[When: $V_s \le 4\sqrt{f_c'}b_w d$]	No. 3 bar: 2 *legs*
		No. 4 bar: 2 *legs*
	[When: $V_s > 4\sqrt{f_c'}b_w d$]	No. 3 bar: 2 *legs*
		No. 4 bar: 2 *legs*

$\phi V_n = \phi V_c + \phi V_s$ *-Reference code ACI 318-19 [22.5.1.1]*

$\phi =$ 0.75 *-Reference code ACI 318-19 [21.2.1]*

$\sqrt{f_c'} \le 100psi$	For strength calculations:	$\sqrt{f_c'} =$ 63.246 *psi*	*-Reference code ACI 318-19 [22.5.3.1]*
$\frac{N_u}{6A_g} \le 0.05f_c'$	For strength calculations:	$\frac{N_u}{6A_g} =$ 0.000 *psi*	*-Reference code ACI 318-19 [22.5.5.1.2]*

$\rho_w = \frac{A_s}{A_g} =$ 0.0044

$\lambda_s = \sqrt{\frac{2}{1+\frac{d}{10}}} \le 1$ | $\lambda_s =$ 0.7785 *-Reference code ACI 318-19 [22.5.5.1.3]*

$V_c \le 5\lambda\sqrt{f_c'}b_w d =$ 58.19 *kips* *-Reference code ACI 318-19 [22.5.5.1.1]*

V_c	$A_v \ge A_{v,\ min}$	$\left[2\lambda\sqrt{f_c'} + \frac{N_u}{6A_g}\right]b_w d =$ 23.27 *kips*	$\phi V_c =$ 17.46 *kips*	*-Reference code ACI 318-19 [Table 22.5.5.1(a)]*
	$A_v < A_{v,\ min}$	$\left[8\lambda_s\lambda(\rho_w)^{\frac{1}{3}}\sqrt{f_c'} + \frac{N_u}{6A_g}\right]b_w d =$ 11.88 *kips*	$\phi V_c =$ 8.91 *kips*	*-Reference code ACI 318-19 [Table 22.5.5.1(c)]*

Max allowable strength from this cross-section	$\phi V_{n,max} = \phi V_c + \phi 8\sqrt{f_c'}b_w d =$ 69.84 *kips*

-Reference code ACI 318-19 [22.5.1.2]

[When: $V_s = 4\sqrt{f_c'}b_w d$]	$\phi V_n =$ 52.37 *kips*

-When demand exceeds this value, tighter spacing is required as per code ACI 318-19 [Table 9.7.6.2.2]

When $A_{v,min}$ is not required	$V_u \le \phi\lambda\sqrt{f_c'}b_w d =$ 8.73 *kips*

-Reference code ACI 318-19 [9.6.3.1]

Stop stirrups when: $V_u \le$ 8.72 *kips*

When *d/2* " spacing is used	No. 3	$V_s = \frac{A_v f_{yt} d}{s} =$ 27.60 *kips*	$\phi V_n = \phi V_c + \phi V_s =$ 38.15 *kips*
	No. 4	$V_s = \frac{A_v f_{yt} d}{s} =$ 50.18 *kips*	$\phi V_n = \phi V_c + \phi V_s =$ 55.09 *kips*

-Reference code ACI 318-19 [22.5.8.5.3]

When *d/4* " spacing is used	No. 3	$V_s = \frac{A_v f_{yt} d}{s} =$ 60.72 *kips*	$\phi V_n = \phi V_c + \phi V_s =$ 62.99 *kips*
	No. 4	$V_s = \frac{A_v f_{yt} d}{s} =$ 110.40 *kips*	$\phi V_n = \phi V_c + \phi V_s =$ 100.25 *kips*

-Reference code ACI 318-19 [22.5.8.5.3]

PROJECT DETAILS

INPUTS

$f_c{}'$	4000 psi
f_{yt}	60000 psi
h	25 in.
λ	1
N_u	0 lbs
Clear Cover	1.5 in.
d	23 in.
b_w	30 in.
#6 Bars =	5 bars
A_s =	2.2 in.^{2}

*Positive for compression, negative for tension

*Assuming (b_w/6) bars are being used

*Assuming #6 bars are used

RESULTS

$\phi V_{n,max}$	ϕV_n @ $d/4$ "	Number of Legs		Max Spacing	$\phi V_{n,max}$ *When* $V_s \leq 4\sqrt{f_c{}'}b_w d$	ϕV_n @ $d/2$"	Number of Legs		Max Spacing	Stop stirrups when $V_u \leq$
		No. 3	No. 4				No. 3	No. 4		
261.90 *kips*	#3: 156.53 *kips*	4	4	*@5in.*	196.38 *kips*	#3: 96.5 *kips*	3	3	*@11in.*	29.17 *kips*
	#4: 231.05 *kips*					#4: 121.91 *kips*				

s_{max} = Along length	$\frac{d}{2} \leq 24 in.$	[When: $V_s \leq 4\sqrt{f_c{}'}b_w d$] $s_{max} =$ 11 *in.*
	$\frac{d}{4} \leq 12 in.$	[When: $V_s > 4\sqrt{f_c{}'}b_w d$] $s_{max} =$ 5 *in.*

-Reference code ACI 318-19 [9.7.6.2.2]

$\frac{A_{v\ min}}{s} =$	Greater of:	$0.75\sqrt{f_c{}'}\frac{b_w}{f_{yt}} =$ 0.0237 *in.*	$\frac{A_{v\ min}}{s} =$ 0.0250 *in.*
		$50\frac{b_w}{f_{yt}} =$ 0.0250 *in.*	

-Reference code ACI 318-19 [9.6.3.4]

# of stirrup legs based on $A_{v,min}$	[When: $V_s \leq 4\sqrt{f_c{}'}b_w d$]	$\frac{A_{v\ min}}{s} \times \frac{s_{max}}{A_{bar}} =$	No. 3 bar: 3 *legs*
			No. 4 bar: 2 *legs*
	[When: $V_s > 4\sqrt{f_c{}'}b_w d$]	$\frac{A_{v\ min}}{s} \times \frac{s_{max}}{A_{bar}} =$	No. 3 bar: 2 *legs*
			No. 4 bar: 1 *legs*

# of stirrup legs based on s_{max} along width	[When: $V_s \leq 4\sqrt{f_c{}'}b_w d$]	$d \leq 24 in.$	$s_{max} =$ 23 *in.*	$\#Legs = \frac{b_w}{s_{max}} + 1 =$ 3 *legs*
	[When: $V_s > 4\sqrt{f_c{}'}b_w d$]	$\frac{d}{2} \leq 12 in.$	$s_{max} =$ 11.5 *in.*	$\#Legs = \frac{b_w}{s_{max}} + 1 =$ 4 *legs*

-Reference code ACI 318-19 [9.7.6.2.2]

Final # of stirrup legs	[When: $V_s \leq 4\sqrt{f_c{}'}b_w d$]	No. 3 bar: 3 *legs*
		No. 4 bar: 3 *legs*
	[When: $V_s > 4\sqrt{f_c{}'}b_w d$]	No. 3 bar: 4 *legs*
		No. 4 bar: 4 *legs*

$\phi V_n = \phi V_c + \phi V_s$ *-Reference code ACI 318-19 [22.5.1.1]*

$\phi =$ 0.75 *-Reference code ACI 318-19 [21.2.1]*

$\sqrt{f_c{}'} \leq 100psi$	For strength calculations:	$\sqrt{f_c{}'} =$ 63.246 *psi*	*-Reference code ACI 318-19 [22.5.3.1]*
$\frac{N_u}{6A_g} \leq 0.05f_c{}'$	For strength calculations:	$\frac{N_u}{6A_g} =$ 0.000 *psi*	*-Reference code ACI 318-19 [22.5.5.1.2]*

$\rho_w = \frac{A_s}{A_g} =$ 0.002933

$\lambda_s = \sqrt{\frac{2}{1+\frac{d}{10}}} \leq 1$ | $\lambda_s =$ 0.7785 *-Reference code ACI 318-19 [22.5.5.1.3]*

$V_c \leq 5\lambda\sqrt{f_c{}'}b_w d =$ 218.20 *kips* *-Reference code ACI 318-19 [22.5.5.1.1]*

V_c	$A_v \geq A_{v,\ min}$	$\left[2\lambda\sqrt{f_c{}'} + \frac{N_u}{6A_g}\right]b_w d =$ 87.28 *kips*	$\phi V_c =$ 65.46 *kips*	*-Reference code ACI 318-19 [Table 22.5.5.1(a)]*
	$A_v < A_{v,\ min}$	$\left[8\lambda_s\lambda(\rho_w)^{\frac{1}{3}}\sqrt{f_c'} + \frac{N_u}{6A_g}\right]b_w d =$ 38.91 *kips*	$\phi V_c =$ 29.18 *kips*	*-Reference code ACI 318-19 [Table 22.5.5.1(c)]*

Max allowable strength from this cross-section	$\phi V_{n,max} = \phi V_c + \phi 8\sqrt{f_c{}'}b_w d =$ 261.90 *kips*	*-Reference code ACI 318-19 [22.5.1.2]*
[When: $V_s = 4\sqrt{f_c{}'}b_w d$]	$\phi V_n =$ 196.38 *kips*	*-When demand exceeds this value, tighter spacing is required as per code ACI 318-19 [Table 9.7.6.2.2]*
When $A_{v,min}$ is not required	$V_u \leq \phi\lambda\sqrt{f_c{}'}b_w d =$ 32.73 *kips*	*-Reference code ACI 318-19 [9.6.3.1]*

Stop stirrups when: $V_u \leq$ 29.17 *kips*

When *d/2* " spacing is used	No. 3	$V_s = \frac{A_v f_{yt} d}{s} =$ 41.40 *kips*	$\phi V_n = \phi V_c + \phi V_s =$ 96.50 *kips*
	No. 4	$V_s = \frac{A_v f_{yt} d}{s} =$ 75.27 *kips*	$\phi V_n = \phi V_c + \phi V_s =$ 121.91 *kips*

-Reference code ACI 318-19 [22.5.8.5.3]

When *d/4* " spacing is used	No. 3	$V_s = \frac{A_v f_{yt} d}{s} =$	121.44 *kips*	$\phi V_n = \phi V_c + \phi V_s =$	156.53 *kips*
	No. 4	$V_s = \frac{A_v f_{yt} d}{s} =$	220.80 *kips*	$\phi V_n = \phi V_c + \phi V_s =$	231.05 *kips*

-Reference code ACI 318-19 [22.5.8.5.3]

Appendix J

- **Pier Depths and Capacities**

From Geotech:

	Bearing	Soil Friction
First 10'	0 psf	0 psf
Next 10'	10,000 psf	750 psf
Limestone	30,000 psf	2,000 psf

24" dia @ 22 ft

$$= (2 \times \pi \times 10 \times 750) + (2 \times \pi \times 2 \times 2000) + (2 \times 2 \times 1/4 \times \pi \times 30000)$$

$$= 166.5 \text{ kips}$$

30" dia @ 22 ft

$$= (2.5 \times \pi \times 10 \times 750) + (2.5 \times \pi \times 2 \times 2000) + (2.5^2 \times 1/4 \times \pi \times 30000)$$

$$= 237.6 \text{ kips}$$

30" dia @ 25 ft

$$= (2.5 \times \pi \times 10 \times 750) + (2.5 \times \pi \times 5 \times 2000) + (2.5^2 \times 1/4 \times \pi \times 30000)$$

$$= 284.7 \text{ kips}$$

30" dia @ 28 ft

$$= (2.5 \times \pi \times 10 \times 750) + (2.5 \times \pi \times 8 \times 2000) + (2.5^2 \times 1/4 \times \pi \times 30000)$$

$$= 331.8 \text{ kips}$$

BORING LOG NO. B-1

Page 1 of 1

PROJECT: Roadrunner Executive Tower

CLIENT: RoadRunner Development LLC
San Antonio, TX

SITE: 5644 UTSA Blvd, San Antonio, Texas 78249
San Antonio, Texas

LOCATION See Exploration Plan

Latitude: 29.5746° Longitude: -98.5994°

Surface Elev.: 994 (Ft.)

MODEL LAYER	GRAPHIC LOG	DEPTH	ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	FIELD TEST RESULTS	WATER CONTENT (%)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
1	🟩	**FAT CLAY (CH)**, dark brown, very stiff				☒	5-7-8 N=15	17.9		
						☒	9-11-12 N=23	16.8	70-23-47	79
				5		☒	9-10-11 N=21	19.3		
						☒	7-9-12 N=21	15.9		
		8.0	986							
2	▨	**LEAN CLAY (CL)**, brown, very stiff to hard, with calcareous deposits				☒	12-17-27 N=44	10.1	40-17-23	68
				10						
						☒	50/5"	6.9		
				15						
		20.0	974							
4	▦	**LIMESTONE**, gray, hard, (rock-like)		20		☒	50/3"	5.6		
							50/0"			
				25						
							50/0"			
		30.0	964	30						
		Boring Terminated at 30 Feet								

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Advancement Method:
Air Rotary

Abandonment Method:
Boring backfilled with auger cuttings upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Notes:

WATER LEVEL OBSERVATIONS

No free water observed

Roadrunner Development LLC
BY STUDENTS, FOR STUDENTS

Boring Started: 10-16-2023	Boring Completed: 10-16-2023
Drill Rig: CME 75	Driller: Ramco
Project No.: 902301	

BORING LOG NO. B-2

Page 1 of 1

PROJECT: Roadrunner Executive Tower

CLIENT: RoadRunner Development LLC
San Antonio, TX

SITE: 5644 UTSA Blvd, San Antonio, Texas 78249
San Antonio, Texas

LOCATION See Exploration Plan

Latitude: 29.5744° Longitude: -98.5999°

Surface Elev.: 976 (Ft.)

MODEL LAYER	GRAPHIC LOG	DEPTH	ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	FIELD TEST RESULTS	WATER CONTENT (%)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
1	▨	**FAT CLAY (CH)**, dark brown, hard				■	4.5 (HP)	23.2	73-32-41	
		2.0	974							
2	▨	**LEAN CLAY (CL)**, brown, hard				■	4.5 (HP)	23.0		
				5		☒	32-50/4"	22.0	47-23-24	
						☒	36-50/4"	22.0		
							50/1"			
		10.0	966	10						
		Boring Terminated at 10 Feet								

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Advancement Method:
Air Rotary

Abandonment Method:
Boring backfilled with auger cuttings upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Notes:

WATER LEVEL OBSERVATIONS

No free water observed

Roadrunner Development LLC
BY STUDENTS, FOR STUDENTS

Boring Started: 10-16-2023	Boring Completed: 10-16-2023
Drill Rig: CME 75	Driller: Ramco
Project No.: 902301	

BORING LOG NO. B-3

Page 1 of 1

PROJECT: Roadrunner Executive Tower

CLIENT: RoadRunner Development LLC
San Antonio, TX

SITE: 5644 UTSA Blvd, San Antonio, Texas 78249
San Antonio, Texas

MODEL LAYER	GRAPHIC LOG	LOCATION See Exploration Plan Latitude: 29.5753° Longitude: -98.6007° Surface Elev.: 963 (Ft.) DEPTH ELEVATION (Ft.)	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	FIELD TEST RESULTS	WATER CONTENT (%)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
2	▨	**LEAN CLAY (CL)**, brown, medium stiff, with calcareous deposits 2.0 961			☒	3-5-6 N=11	27.2	43-18-25	71
4	▦	**LIMESTONE**, gray, hard, (rock-like) 10.0 953	5 10			50/0" 50/0" 50/0" 50/0"			
		Boring Terminated at 10 Feet							

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic

Advancement Method:
Air Rotary

Abandonment Method:
Boring backfilled with auger cuttings upon completion.

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Notes:

WATER LEVEL OBSERVATIONS

No free water observed

Roadrunner Development LLC
BY STUDENTS. FOR STUDENTS

Boring Started: 10-16-2023	Boring Completed: 10-16-2023
Drill Rig: CME 75	Driller: Ramco
Project No.: 902301	

Appendix K

PVR Input Sheet

Project Name: RoadRunner Executive Tower
Project Number:
Boring No.: All

Surcharge Pressure: 1.00 psi **Climatic Rating, C_w:**

Stratum Number	Plasticity Index	Bottom Depth (feet)	Moisture Condition		
			Dry	Average	Optimum
I	47	2	x		
II	47	4	x		
III	47	6	x		
IV	47	8	x		
V	23	10	x		
VI	23	12		x	
VII	23	15		x	

PVR
3.26
inches

Effective PI		Support Index	
BRAB	PCI	BRAB	PCI
47.0	39.8	0.66	0.74

RULES

1. *No greater* than 15 feet in depth.
2. *One and only one* Moisture Condition per strata. Mark with X.
3. *Even and one-half foot* intervals only.
4. Use PI = 8 for non-expansive layers. *NOT PI = 0*.
5. Error checking is limited and erroneous results may occur.

Lime Percent Calculation

$$\overset{\substack{\text{Lime}\\\text{percent}\\\downarrow}}{\frac{5}{100}} \times \overset{\substack{\text{Dry unit wt.}\\\downarrow\text{ of soil}}}{105\text{ pcf}} \times \overset{\substack{\swarrow\text{ Thickness zone}\\\text{of treatment}}}{\frac{6}{12}} \times \overset{\swarrow\text{ sq ft}}{9}$$

25 lbs/sq. yd.

Appendix L

Wastewater Calculations and Specs

Waste water flow for our building:		13.86	EDU	2,772	gpd
Lot size:	8.35	acres			
Peak dry weather flow:	6930	gpd			
Inflow and Infiltration:	5010	gpd			
Wet weather Flow:	11940	gpd	Min flow required for our building: Q		
Average flow velocity: V	0.0529	ft/s			

n =	0.013	
R =	0.125	ft
S =	10%	
A =	0.1964	ft

Maximum flow provided by an 6" pipe			1128743.29	gpd		
Average Flow Velocity: V	8.8944666	ft/s				
Average Flow : Q	1.7464285	ft/s^2	783.85	gpm	1128743.29	gpd

Determination of Wastewater Flows

1. For the purpose of pipe sizing, an equivalent dwelling unit (EDU) is assumed to produce an average wastewater flow of 200 gallons per day.
2. SAWS will evaluate commercial and industrial wastewater flows on a case-by-case basis. Use of SAWS Infrastructure Planning EDU calculation sheet is recommended.

3. Strict attention must be given to minimizing inflow and infiltration. In sizing wastewater mains, external contributions must be accounted for by including 600 gallons per acre served for inflow and infiltration. Wastewater mains in the Edwards Aquifer Recharge Zone must meet the requirements of the Texas Commission on Environmental Quality.
4. The peak dry weather flow is 2.5 times the average flow. In designing for an existing facility, flows must be measured in lieu of calculations for the preexisting developed area.
5. The peak wet weather flow is obtained by adding inflow and infiltration to the peak dry weather flow.
6. Determination of peak dry and wet-weather flow on an existing pipe segment will be required if by-pass pumping is involved. It is the responsibility of the developer customer to monitor and control existing flows during construction to prevent overflows from occurring. Flow measuring equipment shall be utilized as required. Reference section 11.3.3 below.

Determination of Pipe Size

1. All gravity wastewater mains must have a minimum diameter of eight inches.
2. For wastewater mains 15 inches in diameter or smaller, the main must be designed so that the peak wet weather flow will not exceed 90% of the capacity of the pipe flowing full. For wastewater mains 18 inches in diameter or larger, the main must be designed so that the peak wet weather flow will not exceed 95% of the capacity of the pipe flowing full.
3. The maximum design velocity calculated using the peak wet weather flow may not exceed 10 feet per second unless special conditions make no other option available. In such cases, proper consideration must be given to pipe material, abrasive characteristics of the wastewater flows, turbulence and displacement by erosion or shock.
4. Design of wastewater mains must employ the Manning’s Equation with a minimum “n” factor of 0.013 or as required by TCEQ.
5. The Manning Formula is: $V = \frac{1.49}{n} \times R_h^{0.67} \times \sqrt{s}$

11.1 WASTEWATER LATERALS

1. An individual wastewater lateral from the wastewater main to the property line must be installed to serve each lot or tract within a proposed development, in a location approved by SAWS.
2. Wastewater laterals from single-family lots should normally discharge into a wastewater main. At the end of a dead-end line, SAWS may allow up to two wastewater laterals from single-family lots to be connected to a manhole, except on the Edwards Recharge Zone. Wastewater laterals from commercial developments with flows of more than 20,000 gallons per day must discharge into a proposed or existing manhole. Where the flow line of any service lead is 24 inches or more above the flow line of the manhole, a standard drop manhole must be installed per 30 TAC 217.55 (k)(2)(G)- (H) and current SAWS standard construction specifications.
3. Wastewater laterals must be a minimum of six inches in diameter and must minimize the use of bends. The use of 90-degree bends is prohibited.
4. Wastewater laterals with a diameter of six inches must use full body fittings, extruded or factory-fabricated, for connection to a proposed SAWS wastewater main or an approved saddle-type connector for connection to an existing SAWS wastewater main.
5. Wastewater laterals must be a minimum of five feet below the finished grade at the property line, exceptions may be approved by SAWS Director.
6. Wastewater laterals shall not be connected to wastewater mains greater than twenty feet deep, exceptions may be approved by SAWS Director.
7. Wastewater laterals should have a standard 2.0 percent slope but may have a minimum 1.0 percent slope if approved by SAWS.
8. Wastewater laterals may not be connected to mains larger than 21 inches in diameter unless approved by SAWS Director. Any connection to larger mains must have a private wastewater flapper valve inside the property line and adequate on-site venting of wastewater gases at or near the building site.
9. Wastewater laterals shall not exceed 86 feet from the wastewater main to the property line. Wastewater laterals that will exceed 86 feet will be required to extend an 8-inch sewer main and manhole from the wastewater main to the property line.

Water Calculations and Specs

9.5	EDU	2" PVC		
Average daily flow for our building:	1.9	gpm	2736	gpd
Peak Daily Flow:	3.8	gpm	5472	gpd
Peak Hourly Flow:	14.25	gpm	20520	gpd

Velocity PHF:	1.455	ft/s	<5 ft/s	
Static Pressure @ Meter:	79.3305	psi	>80 psi	PRV NOT Required
Operating Pressure @ Meter:	53.42	psi	>40 psi	

Friction Loss coe for PVC C =	120	
constant k =	1.318	
Hydrualic radius R =	0.04167	FT
Surface area of pipe A =	0.0218	FT
Length of run L =	94.65	LF
Head loss hL =	0.286	
Elevation at meter h1 =	987	FT
Elevation at building connection h2 =	994	FT
Static Ground Presure of existing line P2 =	80	PSI
Density of water p =	5.202	lb/ft^2
gravity constant g =	9.81	
Hydraulic Grade Line for our area HGL =	1170	FT

8.6 LOCATION OF WATER METERS

Water meters must be located outside of the fence line and accessible at all times with protection from traffic. Meters must be within or adjacent to public rights-of-way whenever possible. Meters may not be located in areas enclosed by fences. Meters two inches and smaller must be located in a public right-of-way, a water line easement, or a minimum five-foot by five-foot separate water meter easement. Meters three inches and larger must be located at least one foot, but not more than 50 feet, outside of the public right-of-way, in a water line easement or a minimum ten-foot by twelve-foot water meter easement and is subject to approval by SAWS.

9.1 DETERMINATION OF WATER REQUIREMENTS

All water system infrastructures must be designed according to the following assumptions and requirements.

1. The San Antonio Water System employs the factor "Equivalent Dwelling Unit" (EDU) to determine the water demands for its water mains. An EDU, for purposes of water system design, is 290 gallons average daily flow (or .2 gpm).
2. Hazen Williams Friction Coefficient C=120 for PVC and HDPE pipe and C=100 for ductile iron pipe. A higher C factor may be used for new mains only upon approval by SAWS with sufficient documentation to show the effects of long-term use.
3. Average daily flow = .2 gpm per EDU
4. Peak daily flow = .4 gpm per EDU
5. Peak hourly flow = 1.5 gpm per EDU
6. Pressure zones are established to provide static pressures of 56 psi to 150 psi, depending on area geography and elevations.
7. If maximum static pressure exceeds 80 psi at the proposed meter location, a Pressure Reducing Valve (PRV) rated for a maximum working pressure of no less than 300 psi must be installed on the customer side of the meter, in conformance with the current plumbing code with local amendments adopted by the City of San Antonio, prior to a SAWS meter being installed. The PRV(s) must have the ability to reduce the operating pressure to no greater than 80 psi. The PRV's proper settings must be performed and confirmed by the contractor.
8. Minimum operating pressure shall be 40 psi at the highest elevation meter location using peak hourly flow.
9. The velocity in a distribution main may not exceed 5 feet per second during peak hourly flow.
10. The velocity in transmission mains as designated by SAWS may not exceed 3 feet per second during peak daily flow.

9.10 VALVE REQUIREMENTS

1. All valves in the potable water system must open "right (clockwise)." For recycled water and pump stations, valves will open "left (counterclockwise)".
2. Valves must be located at the intersection of two or more mains and must be spaced so that no more than 30 customers will be without water during a shutdown.
3. On mains less than 36 inches in diameter, valves may be no more than 1000 feet apart. For mains 36 inches and larger, the location and frequency of required valves may vary depending on SAWS' engineering design considerations.
4. The number of valves at each intersection shall be the same as the number of pipe extensions, or reduced by one as approved by SAWS to minimize the number of customers out-of-service during a "shut-down".
5. At dead ends, gate valves must be located one pipe length or a minimum of 10 feet from the end points of the main. The customer's engineer must provide drawings showing complete restraint for all such valves, pipe extensions and end caps.
6. Branch piping for both new and future branches must be separated from the water main by gate valves. Future branch valves must have proper restraints and caps.

7. Valves at intersections must be placed at the point of curvature of the curb line.
8. On water mains 16 inches and smaller, valves must be resilient seated gate valves.
9. On water mains 16 inches in diameter and larger, automatic combination air/vacuum valves must be placed at all high points.
10. On water mains greater than 16 inches in diameter, butterfly valves must be used.
11. All butterfly valves must have actuators enclosed in a valve box.
12. Valves separating pressure zones, (Division valves, or pressure zone boundaries) must be equipped with a locking type debris cap. The valve box lid must state Division Valve.
13. Fire hydrant valves must be resilient seated gate valves and must be restrained to the main.
14. All valves shall be mechanically restrained.
15. Valves (minimum Pressure Class 200 psi rated) shall be class 250 lb., with 150 lb. bolt pattern (class 'E' flanges). The 250 lb. valve with the 150 lb. bolt pattern provides the 200 psi.

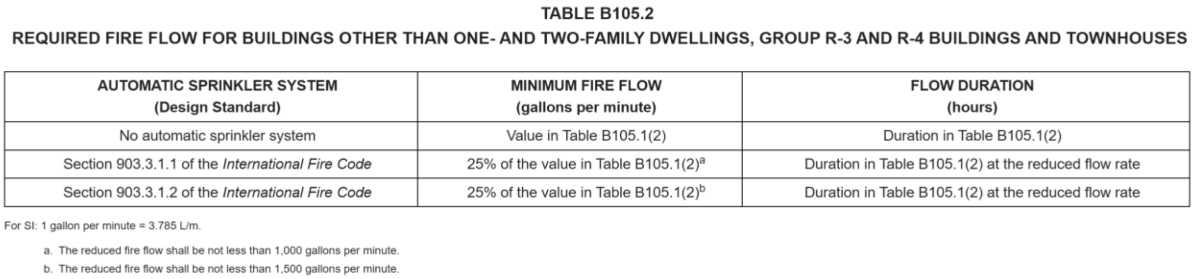


TABLE B105.2
REQUIRED FIRE FLOW FOR BUILDINGS OTHER THAN ONE- AND TWO-FAMILY DWELLINGS, GROUP R-3 AND R-4 BUILDINGS AND TOWNHOUSES

AUTOMATIC SPRINKLER SYSTEM (Design Standard)	MINIMUM FIRE FLOW (gallons per minute)	FLOW DURATION (hours)
No automatic sprinkler system	Value in Table B105.1(2)	Duration in Table B105.1(2)
Section 903.3.1.1 of the *International Fire Code*	25% of the value in Table B105.1(2)[a]	Duration in Table B105.1(2) at the reduced flow rate
Section 903.3.1.2 of the *International Fire Code*	25% of the value in Table B105.1(2)[b]	Duration in Table B105.1(2) at the reduced flow rate

For SI: 1 gallon per minute = 3.785 L/m.

a. The reduced fire flow shall be not less than 1,000 gallons per minute.

b. The reduced fire flow shall be not less than 1,500 gallons per minute.

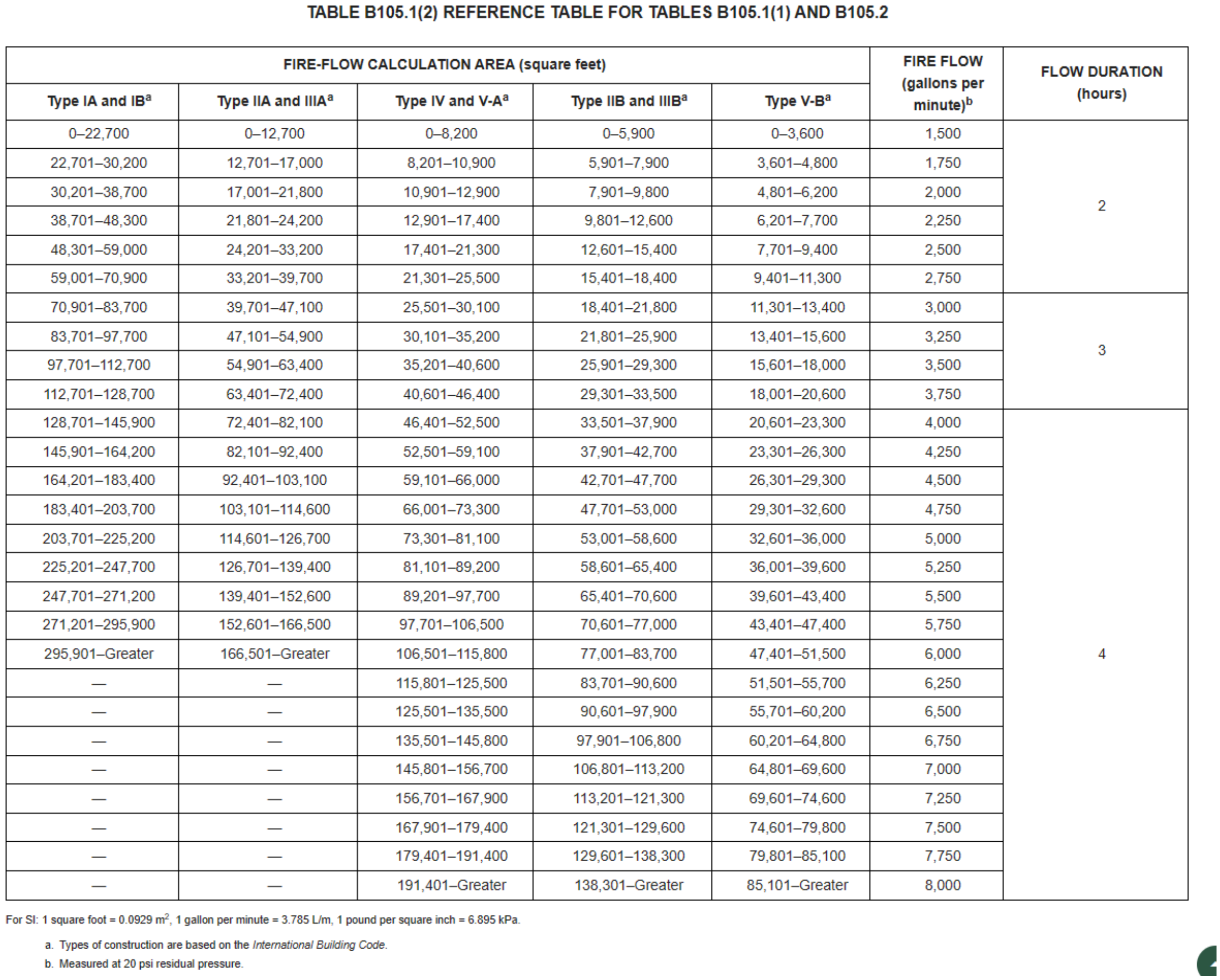


TABLE B105.1(2) REFERENCE TABLE FOR TABLES B105.1(1) AND B105.2

FIRE-FLOW CALCULATION AREA (square feet)					FIRE FLOW (gallons per minute)[b]	FLOW DURATION (hours)
Type IA and IB[a]	Type IIA and IIIA[a]	Type IV and V-A[a]	Type IIB and IIIB[a]	Type V-B[a]		
0–22,700	0–12,700	0–8,200	0–5,900	0–3,600	1,500	2
22,701–30,200	12,701–17,000	8,201–10,900	5,901–7,900	3,601–4,800	1,750	
30,201–38,700	17,001–21,800	10,901–12,900	7,901–9,800	4,801–6,200	2,000	
38,701–48,300	21,801–24,200	12,901–17,400	9,801–12,600	6,201–7,700	2,250	
48,301–59,000	24,201–33,200	17,401–21,300	12,601–15,400	7,701–9,400	2,500	
59,001–70,900	33,201–39,700	21,301–25,500	15,401–18,400	9,401–11,300	2,750	
70,901–83,700	39,701–47,100	25,501–30,100	18,401–21,800	11,301–13,400	3,000	3
83,701–97,700	47,101–54,900	30,101–35,200	21,801–25,900	13,401–15,600	3,250	
97,701–112,700	54,901–63,400	35,201–40,600	25,901–29,300	15,601–18,000	3,500	
112,701–128,700	63,401–72,400	40,601–46,400	29,301–33,500	18,001–20,600	3,750	
128,701–145,900	72,401–82,100	46,401–52,500	33,501–37,900	20,601–23,300	4,000	4
145,901–164,200	82,101–92,400	52,501–59,100	37,901–42,700	23,301–26,300	4,250	
164,201–183,400	92,401–103,100	59,101–66,000	42,701–47,700	26,301–29,300	4,500	
183,401–203,700	103,101–114,600	66,001–73,300	47,701–53,000	29,301–32,600	4,750	
203,701–225,200	114,601–126,700	73,301–81,100	53,001–58,600	32,601–36,000	5,000	
225,201–247,700	126,701–139,400	81,101–89,200	58,601–65,400	36,001–39,600	5,250	
247,701–271,200	139,401–152,600	89,201–97,700	65,401–70,600	39,601–43,400	5,500	
271,201–295,900	152,601–166,500	97,701–106,500	70,601–77,000	43,401–47,400	5,750	
295,901–Greater	166,501–Greater	106,501–115,800	77,001–83,700	47,401–51,500	6,000	
—	—	115,801–125,500	83,701–90,600	51,501–55,700	6,250	
—	—	125,501–135,500	90,601–97,900	55,701–60,200	6,500	
—	—	135,501–145,800	97,901–106,800	60,201–64,800	6,750	
—	—	145,801–156,700	106,801–113,200	64,801–69,600	7,000	
—	—	156,701–167,900	113,201–121,300	69,601–74,600	7,250	
—	—	167,901–179,400	121,301–129,600	74,601–79,800	7,500	
—	—	179,401–191,400	129,601–138,300	79,801–85,100	7,750	
—	—	191,401–Greater	138,301–Greater	85,101–Greater	8,000	

For SI: 1 square foot = 0.0929 m^2, 1 gallon per minute = 3.785 L/m, 1 pound per square inch = 6.895 kPa.

a. Types of construction are based on the *International Building Code*.

b. Measured at 20 psi residual pressure.

Fire Protection Specs

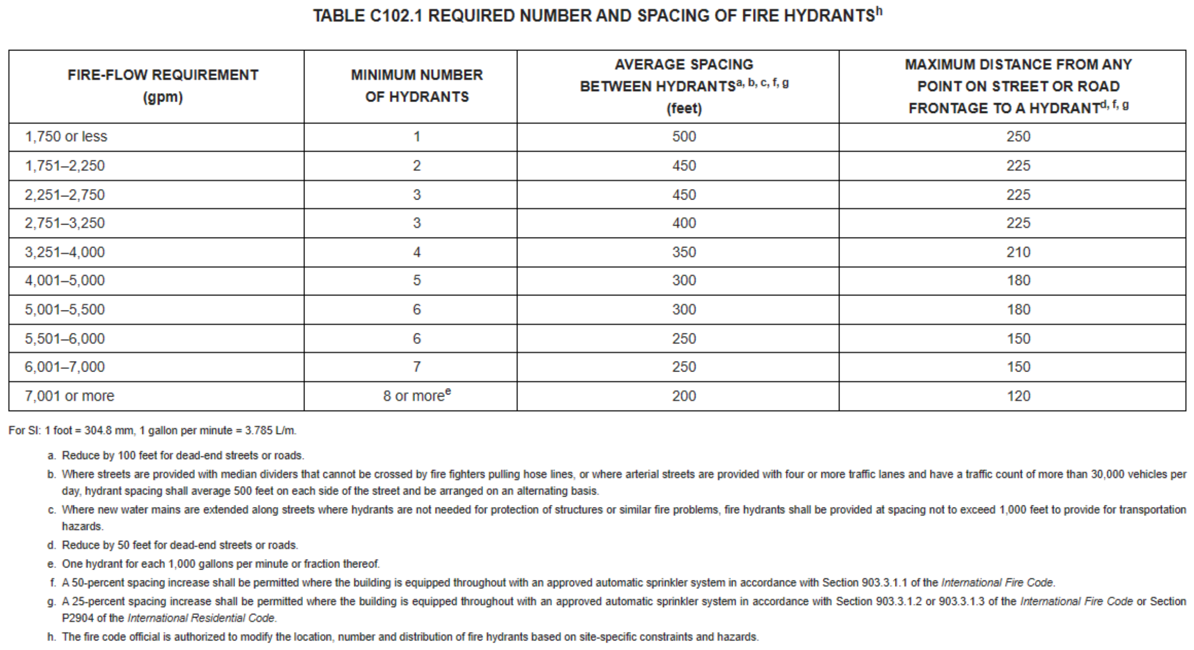


TABLE C102.1 REQUIRED NUMBER AND SPACING OF FIRE HYDRANTS[h]

FIRE-FLOW REQUIREMENT (gpm)	MINIMUM NUMBER OF HYDRANTS	AVERAGE SPACING BETWEEN HYDRANTS[a, b, c, f, g] (feet)	MAXIMUM DISTANCE FROM ANY POINT ON STREET OR ROAD FRONTAGE TO A HYDRANT[d, f, g]
1,750 or less	1	500	250
1,751–2,250	2	450	225
2,251–2,750	3	450	225
2,751–3,250	3	400	225
3,251–4,000	4	350	210
4,001–5,000	5	300	180
5,001–5,500	6	300	180
5,501–6,000	6	250	150
6,001–7,000	7	250	150
7,001 or more	8 or more[e]	200	120

For SI: 1 foot = 304.8 mm, 1 gallon per minute = 3.785 L/m.

a. Reduce by 100 feet for dead-end streets or roads.

b. Where streets are provided with median dividers that cannot be crossed by fire fighters pulling hose lines, or where arterial streets are provided with four or more traffic lanes and have a traffic count of more than 30,000 vehicles per day, hydrant spacing shall average 500 feet on each side of the street and be arranged on an alternating basis.

c. Where new water mains are extended along streets where hydrants are not needed for protection of structures or similar fire problems, fire hydrants shall be provided at spacing not to exceed 1,000 feet to provide for transportation hazards.

d. Reduce by 50 feet for dead-end streets or roads.

e. One hydrant for each 1,000 gallons per minute or fraction thereof.

f. A 50-percent spacing increase shall be permitted where the building is equipped throughout with an approved automatic sprinkler system in accordance with Section 903.3.1.1 of the *International Fire Code*.

g. A 25-percent spacing increase shall be permitted where the building is equipped throughout with an approved automatic sprinkler system in accordance with Section 903.3.1.2 or 903.3.1.3 of the *International Fire Code* or Section P2904 of the *International Residential Code*.

h. The fire code official is authorized to modify the location, number and distribution of fire hydrants based on site-specific constraints and hazards.