DESIGN SUMMARY REPORT OF SITE DEVELOPMENT LOCATED AT LINDA VISTA BLVD AND ORACLE RD

By

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With Honors in

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- Responsible for:
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    - Making room reservations
    - Printing attendance sheets and timesheets
    - Compiling the report
        - Revising and editing the report
            - Structural, Geotechnical, Hydrology
        - Printing and binding the report
    - Putting the PowerPoint presentation together
    - Keeping members on task
- Part of the structural department
    - Research on Bracing
    - Load Resisting System
        - Lateral loads
    - Shear Wall Design
    - Architectural Floor Plan – Second story
    - Wind Load Analysis
    - LEED credits

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- Responsible for:
    - Help with the completion of the report and presentation
    - Hydrology Lead
        - Detention Basin Design
        - LEED credits
    - Grading of the site
    - AutoCAD drawing edits
    - Provide assistance to transportation section when needed

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- Structural Lead
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  - Gravity Loads
  - Seismic Load
  - Composite Floor Design
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Brad Simmons

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  - Parking layout
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- Part of the transportation department
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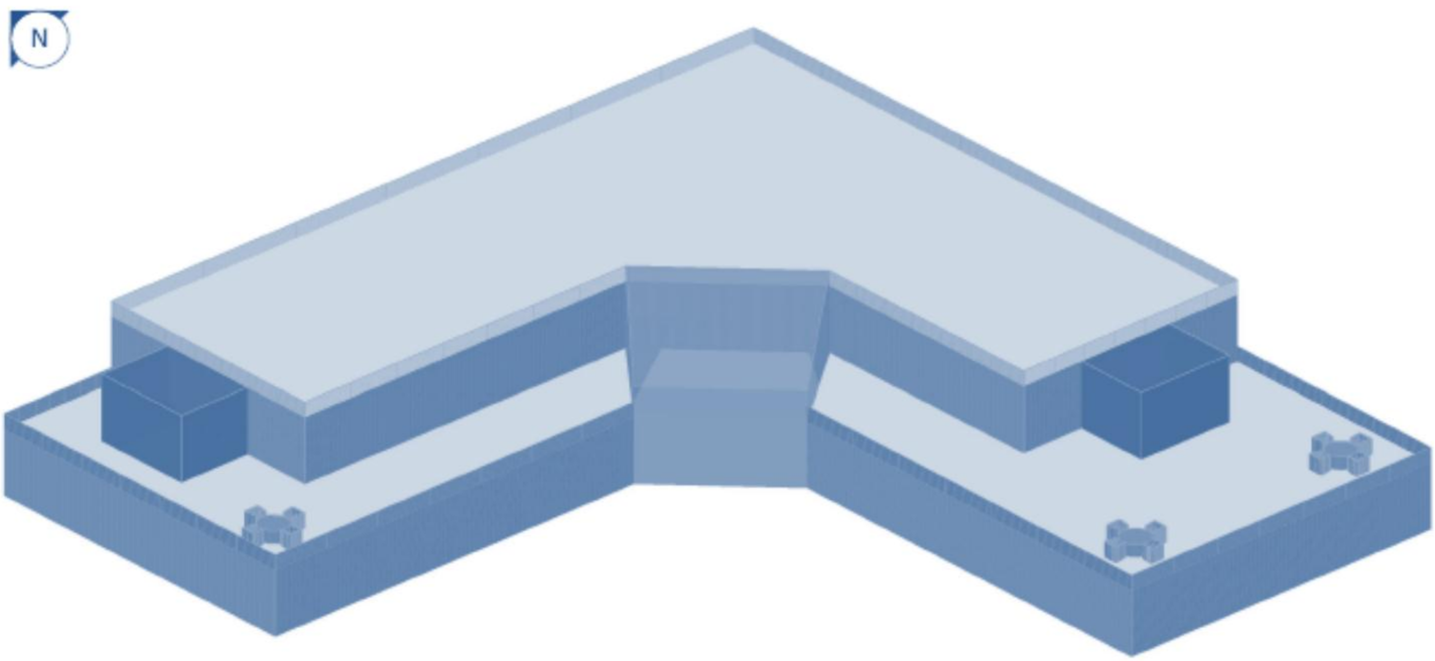

CE 408 Issues in Professional Engineering Practice

Design Summary Report

May 3, 2011

ARIZONA

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Introduction

The proposed general office building is to be located at the northeastern corner of the intersection of Oracle Road and Linda Vista Boulevard in the town of Oro Valley. This area was divided into four different parcels, one of which was designed in the following sections. The client requested a general office building in the Southwestern parcel of about 5 acres, which is labeled as Parcel 3 in Figure 1.

Figure 1: Parcel Layout

Since the construction site is part of Oro Valley, the different zoning code requirements had to be accounted in the design process. In the subsequent paragraphs, the restrictions and the suggestions for each main design focus area are summarized.

Zoning

The regulations for grading are intended to reduce erosion effects of storm water discharge, to preserve the flood carrying capacity of natural or to construct waterways by limiting soil loss, to create stability, and to reduce scarring by having graded areas blend with the natural contours of the surrounding land. The town zoning code also provided limitations on cut and fill slopes. Different cut and fill slopes require stabilization methods; such as, vegetating, rock riprap, and grouted riprap. More importantly, these regulations ensure the community who adverse impacts will be avoided when new developments are designed.

For the hydrology focus area, the Oro Valley Drainage Criteria Manual specified that all of Oro Valley lies within a critical basin. As Oro Valley code states, only a detention basin may be used as opposed to a retention basin. If the water level in the basin is less than 3 feet deep, the maximum slope should be a 3:1 horizontal to vertical ratio. A barrier must be placed around the basin at a minimum height of 42 inches. Other restriction include continuous uniform slopes not exceeding 20% of the basin perimeter and not allowing new detention basins to take more than 12 hours to drain free from standing water. Also, adjacent to all washes with a base flood flow rate of 50 cfs, a setback from the wash is required for all new development to create a safety buffer. The buffer may be used for suitable open space purposes, such as for outdoor recreation and wildlife habitat areas, and for other activities using temporary and portable structures only.

In regard to the Parking and Transportation Zoning Codes, the Oro Valley Zoning regulations in Chapter 27 were researched. For the parking aspect of the project, Oro Valley requires that the number of allowed parking spaces for a general office building is 3 spaces for every 1000 square feet of the building. Each parking space is to have at least a width of 8 feet and a length of 20 feet while disabled spaces need to have a width of 8 feet, a length of 20 feet and an adjacent aisle of 5 feet to allow for ease of vehicle access. Parking aisles are to include a maximum of 9 parking spaces per landscaped island. These spots can be mirrored and should include an island on both edges to end the rows.

These island dimensions are to be 10 feet in width and 20 feet in length for a single island and a width of 10 feet and length of 40 feet for double-parking rows. Each island will include 1 canopy tree. For bicycle parking, Oro Valley zoning codes state that for every 20 required vehicular parking spaces, there shall be 1 bicycle parking space.

The zoning codes also include regulations for the entrances and the exits of parking areas. These state that a minimum of 150 feet, measured at the centerline shall separate any entrances or exits from the nearest intersecting street centerline. Also, adequate turning radii shall be included for all entrances. AASHTO states that a 24 foot radius should be used for passenger cars. Also, needed to be included in the layout is both a minimum 2-way traffic aisle that is 24 feet wide and a minimum of 1-way fire lane access aisle with a width of 20 feet.

According to the zoning code provided by the town of Oro Valley, the new building is classified as C-1, Commercial District, because it is a large office building and it located near a major arterial. Based on the requirements of the Commercial District, the building can only be 25 feet high, excluding architectural elements that may be no higher than 35 feet, needs to have a minimum of 20% open space, and a setback of 50 feet. However, an amendment has been added so that buildings can have a height of 30 feet. The town of Oro Valley highly suggests that the design of the building should complement the existing contours and the surrounding environment. The design of the building should try to keep as much of the current landscape undisturbed to preserve the natural quality of the area.

Grading

The Southwestern parcel has a natural high point at the southeastern corner of the property. In order to take advantage of this, the grading allows water to drain to the north and west, across the parking lot, into the two proposed basins. Contours, spot grades and flow slope percentages are shown in Figure 2. The finished floor elevation and elevations inside the detention basins are also shown. A grading plan containing flow arrows can also be seen in the Appendix.

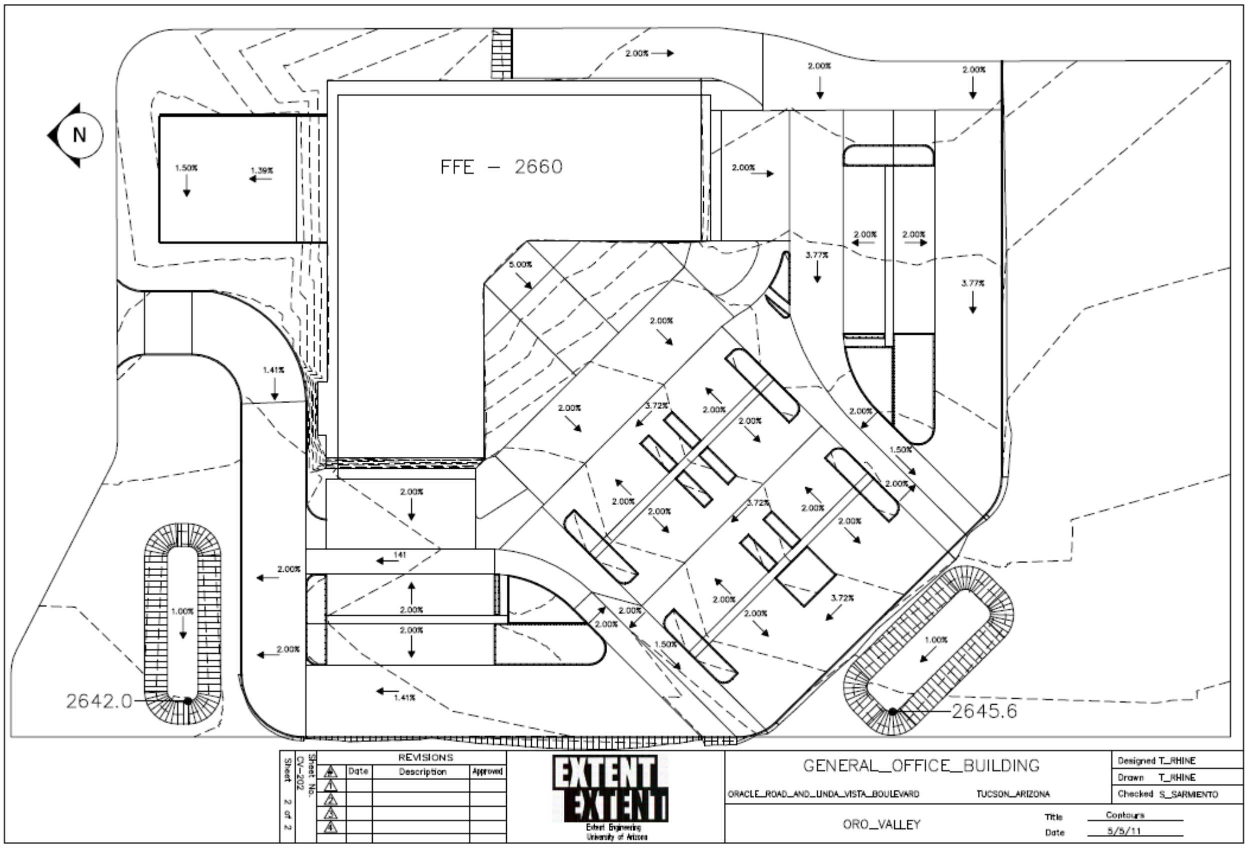

Figure2: Grading Plan

The Engineered fill requirements and specifications were found by use of AutoCAD Civil 3D 2011. The cut and fill volumes were calculated and a compaction factor of 10% was given to the soil being reused. The calculations for these volumes are shown below, and the cost is detailed in the hydrology section.

Table 1: Cut/Fill Calculations

	Cut (yd^3)	Fill(yd^3)	Net(yd^3)
Comparison	5732.47	5267.94	464.53 Cut
Adjusted	5159.23	5269.94	108.71 Fill

1. Geotechnical Analysis & Recommendation

1.1 Introduction

This portion of the report discusses the scope of work that was required of the geotechnical team. Calculations, design procedures and details of the foundations are available in the Appendix. Other details such as an estimated cost of the foundation system and the recommended procedures for the excavations for the footings will be discussed in the subsequent sections.

1.2 Foundation Selection

Recalling on the pre-design soil investigation, shallow foundations were recommended by the geotechnical team. To briefly summarize, this decision was based on factors that included current soil conditions, soil classification, structural size and economics. Deep foundations were investigated but due to the expensive construction process, shallow foundations will provide an optimum serviceability. The analysis of the soil conditions were based on the available Standard Penetration Test (STP) data, which was provided by the client. From that data, it was determined that the upper soil stratum will have the bearing capacity to support the proposed structure with minimum soil improvement, which will be discussed in the later sections.

1.3 Soil Parameters

Each of the parameters used for the design of the structural foundation were extracted from information interpolated between boring logs in proximity to the structure's footprint. This was necessary since the SPT testing was not coordinated with the site layout thus soil samples were not available at specific points around the location of the structural columns. The standard penetration test is known for giving weak empirical correlations for many coarse gravels and clay. The medium to dense sand containing a small amount of silt will give reliable SPT results. Based on the observation of the boring logs, the existing soil is homogenous throughout the site. This leaves a small amount of uncertainty for the design of the structural foundation based on the selected parameters. The parameters include the friction angle, modulus of elasticity, and the unit weight of the soil. These parameters are summarized in the Appendix.

The bearing capacity calculations were based on effective stress analysis (ESA). Effective stress analysis is used for short-termed conditions which is also known as drained conditions when referring to a coarse grained soil. The soil report provided by the client classifies the soil as a coarse grained from a laboratory sieve analysis, therefore it is unnecessary to calculate the bearing capacity for only long-term conditions, total stress analysis (TSA), due to the soil's ability to allow water to pass quickly compared to a fined-grained soil.

Most of the parameters were determined from industry accepted correlation tables. From the soil profiles, worst-case parameters were extracted from the upper layer of the soil. This will provide a conservative design because in general soil conditions usually improve with depth. Various loose spots were identified in the profiles and a consolidation lab test showed the soil having some consolidation potential. Thus, critical construction specifications must be considered. The parameters used for bearing capacity and settlement are shown below in Table 2.

Table 2 Design Parameters

'	γ (pcf)	E (ksf)	υ
33	115	1254	0.3

In order to ensure that the footings are placed on a firm base and reduce the consolidation potential, over-excavation is recommended. This process will require the contractor to excavate approximately two feet around the perimeter and below the embedment depth of the footing. The excavated soil will then be saturated and used as backfilled compacted with a hand tamper to 95% or greater. The footing design is based off of this process, thus it is required that the contractor arranges field test to be performed by a qualified engineer or technician. Another detail includes a layer of well-graded gravel below the base of every footing. This will ensure the rough footing factor is taken into account with respect to the calculation of the bearing capacity.

1.4 Foundation Design

In order to increase the ease of construction and reduce the construction cost, two square spread footing sizes were designed for all the internal columns. A strip foundation was designed for the outer perimeter masonry load bearing wall. Using the limiting equilibrium method developed by Terzaghi, the theoretical bearing capacity was determined based on the soil parameters described earlier. A preliminary design using the AASHTO (2004) method for coarse grained soil was used but compared to the theoretical bearing capacity the footing geometry was significantly smaller. Based on engineering judgment the conservative design calculated from Terzaghi was used. Both calculations of the two methods may be observed in the Appendix.

The two footing dimensions that were designed for the interior columns are designed from the loads of columns E-5 and F-4. A moment due to siesmic loads created some eccentricity on the footings in which tension develops at the base. Soil does not have any tensile strength and the footing will not effectively transmit the load to the soil. In order to avoid such a situation, the footings were designed in a way such that the eccentricity is less than a sixth of the width. The large siesmic moments resulted in larger foundations.

Throughout the foundation design process, Allowable Strength Design (ASD) was used due to the designers familiarity of the concepts and the popularity in the industry of foundation design. A target factor of safety of about three or more was achieved for every column load to ensure a structurally safe and economical foundation system. To summarize the foundation dimensions, a seven foot square footing is recommended for the larger loaded columns and six foot square footing for the ligher columns.

A conservative footing size of three feet will be used for the support of the outer load bearing CMU wall. The strip footing dimensions are smaller than the typical interior footings due to the structural load being uniformily distributed throughout the whole perimeter of the building. The embedment depth and thickness was reduced to 2.5' and 1.5' respectively. More specific details on the footings can be found in the construction drawings.

The column to footing connection system is designed to distribute the axial force to a column base plate. The steel base plate will be made up of grade 50 steel. The distributed load from the base plate will be transferred to the concrete footings uniformly. The specific design shown in the construction details also has the capability of resisting uplift forces resulting from the axial loads and moments. The column base plate will be attached to the concrete footing using four hooked anchor rods per footing with a diameter between 1" to 1.25". The columns will be welded to steel angles that are anchored to the base plate with the anchor rods.

The International Building Code, IBC (2006), requires a minimum embedment depth of 12 inches. Based on the loads transferred by the critical columns, a depth of 3.5 feet is recommended to ensure the proper bearing capacity and provide the necessary cover for the increase of the thickness of the footing designed for punching shear. From the punching shear calculations, the amount of required reinforcing steel was determined. ACI code recommends the reinforcing steel have a minimum of three inch concrete cover. The calculations for punching shear are found in the Appendix.

Primary settlement calculations were performed for each footing. This was necessary in order to determine if differential settlement would be an issue. Differential settlement may result in high moments that may have not been anticipated during the structural design. Having knowledge of the soil classification resulted in the safe assumption that settlement would not govern the design. A summary of the differential settlement of each footing with respect to an adjacent footing is shown in the Appendix. A tolerable differential settlement is approximately half an inch or less. As shown in the differential settlement summary table, every footing combination stays within the tolerable range.

1.5 Estimated Foundation Cost

The overall cost of the whole foundation system will be based on two major components such as excavation and concrete. All unit cost were taken from RS Means construction pricing database. Table 3 represents an engineering estimate of the basic components of the foundation system.

Table 3 Foundation Cost

Description	**Material Unit Cost ($/CY)**	**7x7' ($/ftg.)**	**6x6' ($/ftg.)**	**Strip Ftg. ($/LF)**	**Footing Cost ($)**
Excavation	$23.50	$387.75	$306.37	$176.25	$20,028.96
Concrete (4000 psi)	$103.00	$373.85	$274.67	$154.50	$18,013.56
	Total	**$761.60**	**$581.04**	**$330.75**	**$38,042.52**

As shown above, the unit cost of the material is provided based on current 2011 data. The total cost is based on the twelve 7x7’, eight 6x6’ footing and approximately 660 linear feet of the strip footing to support the CMU wall. The excavation cost includes the dimensions required for over-excavation for the procedures to ensure soil improvement.

1.6 LEED

No specific LEED points are given for the foundation design. Though, it is recommended that the contractor to the best of his/her ability be required take action to reduce construction material waste.

2. Hydrology Analysis & Recommendation

2.1 Improvements

In the initial proposal, the methodology used consisted of designing one basin to collect the entirety of the runoff on this site. This idea was propagated by many different factors. The foremost was propagated by the need for a parking layout with better circulation. If smaller basins were designed, there would be more room to shift around parking spots to allow for changes. The new goal was to try and localize water various areas of the parking into two different basins on the site. The original basin was reduced in size, while a second one was added along to direction of flow of the site. Basin A was placed in the southwestern part of the parcel, while Basin B was placed at the initial proposed basin, in the northwestern part. This new design would decrease the overall runoff on the site and improve the lifespan and safety of the parking lot.

Surface water flows from the southeast to the northwest within the proposed site. Because the site is in a FEMA floodplain zone, the lowest floor elevation must be at least one foot above the 100 yr floodwater elevation, which for the parcel is general office building is at 2660 feet. Linda Vista Blvd will be the southern boundary for the drainage area, while the eastern, western and southern boundaries will follow the perimeter of the parcel due to the location of roads and streams. With these boundaries, the drainage area that determines the capacity of the detention basin is five acres or 218,120 square feet. Due to the location chosen for the detention basin, a "Type 1: 'online' detention basin without retention" was used. It should be noted that this type of basin was approved for the proposed site by Pima County.

2.2 Detention Basin Design

In order to design adequate detention basins for the site, the flow that they are required to carry and to distribute was calculated. The use of the program PC Hydro was deemed necessary by the Oro Valley Drainage Manual and Pima County Stormwater Detention/Retention Manual to calculate this flow. This program is designed to calculate the peak flow for different peak storms. There are 3 different areas of data that must be inputted into PC Hydro to calculate the peak flow. The first is the watershed data, in which information about the watershed is inputted. The watershed area was inputted in this section of PC Hydro, and was found by tracing the watershed map in AutoCAD and hatching the space to find the area in square miles. The watershed type is listed as undeveloped foothills, because of the widely undeveloped area in the watershed. The length of longest watercourse, the length to center of gravity, and the slope breaks were all gathered from ArcGIS. The basin factor is 0.035 as specified by the Oro Valley Drainage Manual. The second area is the soil/vegetation data input form. The vegetation cover type was considered to be desert brush, while the cover density was estimated as 30%. This cover density value was estimated as such due to field visits. When different values were tested, very little significance was placed on any changes made to the inputted estimate. By using some of the built in tools, the soil type percentages were found from ArcGIS as well. The Oro Valley Drainage Manual provided the curve numbers that were needed to put into the program. The rainfall data input was the final area that was put into the system. The latitude and longitude of the area, given as 32.38178 and -110.9623, and the return period were used to get the rainfall data for the precipitation depths for 5, 10, 15, 30, 60, 120, 180, 360, 720, and 1440 minutes. All the criterion needed was organized in an Excel file and entered into PC Hydro. The peak discharges of pre-developed and post-developed land as well as hydrographs were found to determine the maximum runoff volume. All the inputted values for the land are listed in Table 4.

Table 4: PC Hydro Calculations

Calculations Summary			
	Pre Development Entire Site	**Post Development (Basin A)**	**Post Development (Basin B)**
Watershed Area (square miles)	0.007824 sq mi	0.007824 sq mi	0.007824 sq mi
Watershed Type	Undeveloped Foothills	Low-Density/urbanized	Low-Density/urbanized
Length of Watercourse (feet)	671 ft	671 ft	671 ft
Vegetation Cover Type	Desert Brush	Desert Brush	Desert Brush
Impervious Cover	3%	30%	20%
Peak Discharge (cubic feet per second)	32.3 cfs	38.6 cfs	36.6 cfs
Minimum Basin Storage (cubic feet)		6863 cf	4724 cf

Calculations and Hydrograph can be found in the Appendix.

2.3 Detention Basin Details

As previously stated, two different detention basins were designed for the difference in runoff volume between the pre-developed and post-developed land. The hydrograph of the difference between these runoffs can be found in the Appendix. Using the program PC-Hydro, the maximum outflow using the 15% reduction was determined to be 27.20 cubic feet per second.. The detention basins were designed using a rectangular base with side slopes of 3:1, where the ratio is horizontal to vertical distance. This preliminary design had a storage capacity of 25,715 cubic feet, while the final design of the two basins have a total storage capacity of 17,967. The change in design allowed for 7,748 less cubic feet to be cut from the lot, thus saving the project about $1,005. According to the Stormwater Detention/Retention Manual, a minimum of 900 cubic feet must be added to the volume of the basin to account for sedimentation. Both basins have sufficient volume for both stormwater storage and sedimentation accumulation. Therefore the volume of the proposed design basin is adequate.

The calculations of these dimensions are listed in the Appendix, and Figure 3 and Figure 4 illustrate different views of the basin with dimensions.

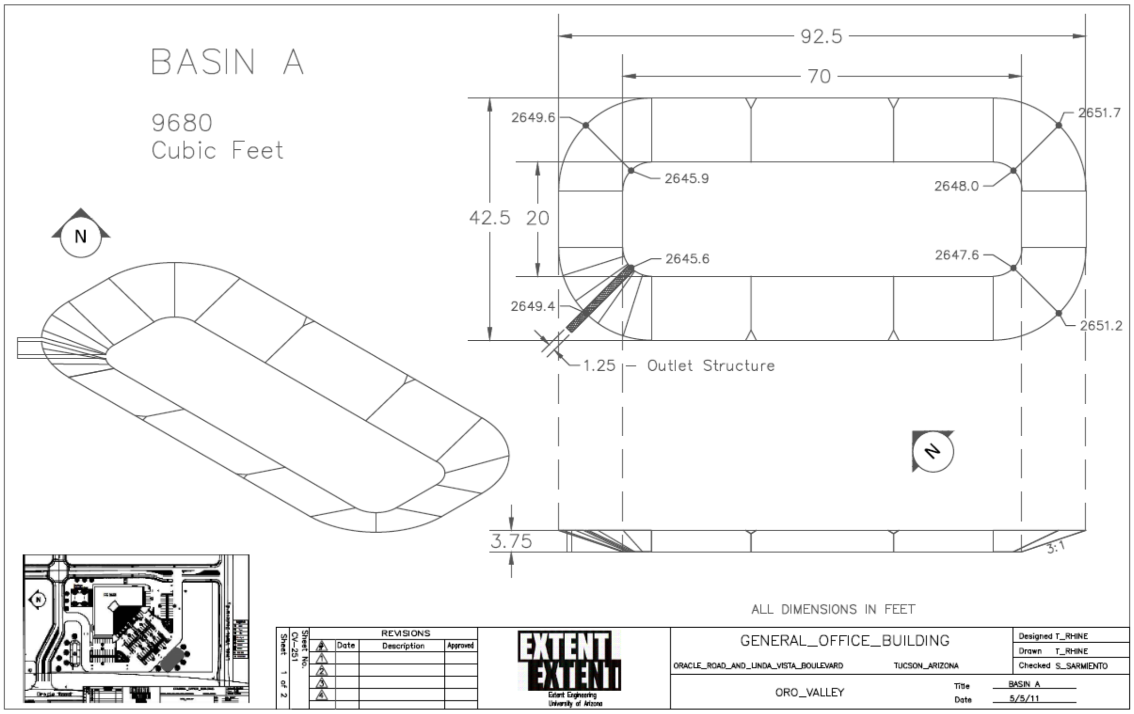

Figure 3: Dimensional Detention Basin A Design

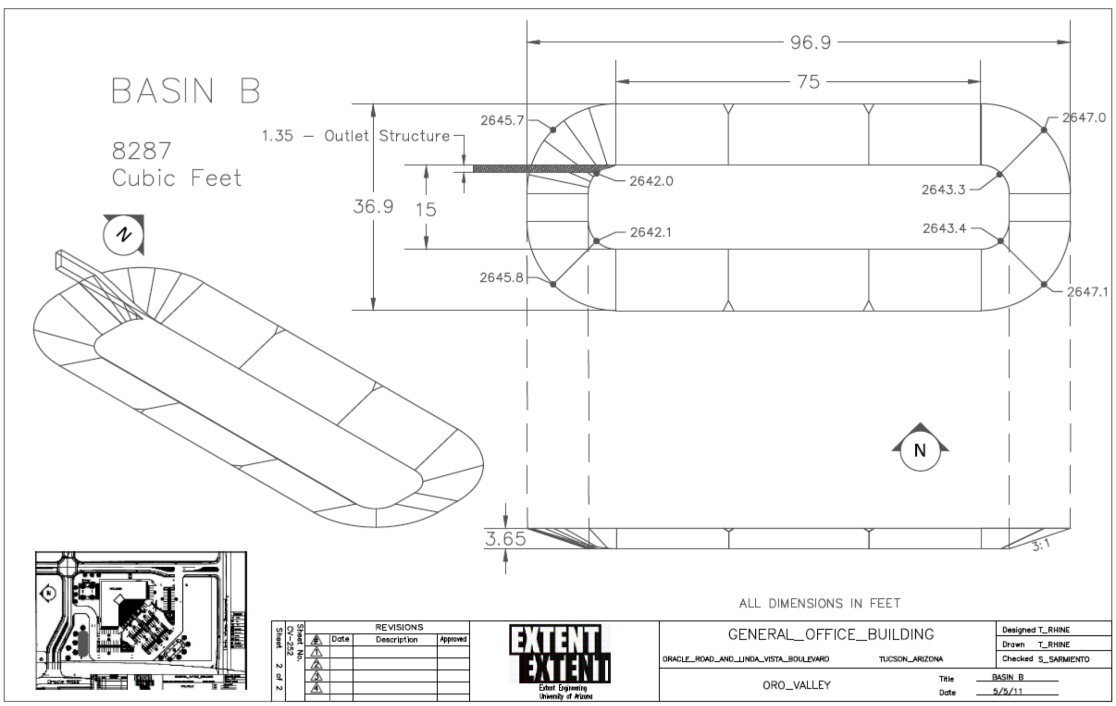

Figure4: Dimensional Detention Basin B Design

2.4 Outlet Structure

The outlet structures for both basins are composed of a weir and rip rapped side slopes. The weir dimensions are 1.25 feet by 3.75 feet and 1.3 feet by 3.65 feet for Basins A and B respectively. During a 100 year storm event, the flow through the weir is reduced to 27.20 cubic feet per second at a depth of 3.75 and 3.65 feet. Using the equation Q = AV, the velocity of the water flowing from the weir is 5.80 feet per second for Basin A and 5.73 feet per second for Basin B. Using the following equation obtained from Appendix D of Hydraulic Engineering Circular Number 14, 3rd Edition, $D_{50} = \alpha V^2$, the minimum average diameter of the rip rap was determined to be 5 inches. To account for any errors or factors of safety, a D_{50} of 6 inches will be used in grouted rip rap along the basin walls. This velocity through the outlet structure may cause some erosion problems, and the exposed sides are a major concern and require stabilization to ensure that seepage forces and hydrostatic pressure do not cause an embankment failure. To prevent any failures, steps to ensure the structural integrity of the basin embankments must be taken. This includes lining the inner and outer sides with grouted rip rap as well as compacting the embankment soil just enough so as not to limit permeability. Because of this, the thickness of the rip rap layer will be 12 inches with 6 inches of granular fill to protect the underlying soils.

2.5 Water Harvesting

Water harvesting structures will be utilized in the landscape bufferyards. Water harvesting structures must fit within the overall site plan and abide all local regulations and codes. Land use may restrict the property area available, as well as the size and location for these water harvesting practices. The decision making regarding the implementation of water harvesting principles often lies within the cost and the commitment level of the financier. The principles of water harvesting can be beneficial to any site at varying cost, and the effectiveness and applicability of water harvesting techniques should be analyzed for each site that is to be developed. Water harvesting techniques and a cistern will be used on EXTENTS site design, and are described below.

A cistern will be implemented to store the runoff of the site in a closed container for irrigation. This system will operate with the harvested rainwater when there is a specified depth in the cistern. The onsite cistern will provide water to a small irrigation system via gravity feed, if it is located efficiently, or with the help of a small pump. Water within a cistern should be stored so it is not accessible to mosquitoes, insects, other animals, or sunlight. The cistern is designed for the 5-year storm, which corresponds to a volume of 1320 cubic feet. A cylindrical tank of 13 foot diameter and 1 0 feet in height will be used to store this volume. Overflow must be accounted for and routed so as not to cause drainage issues. When there is not adequate volume the irrigation system will revert to potable water through that of a drip system.

All of the water harvesting and drainage structures should be maintained to ensure that they function as intended during storms. It is recommended that overflow pipes be inspected and cleaned of any debris prior to the summer monsoon and winter rainy seasons. Failure to keep maintenance could result in undesirably high water levels in the detention basins and damage to the site. Any large amounts of organic material or sedimentation should be cleaned regularly from the basin bottoms. Any build up will impede drainage and result in possible overflows and mosquito growth.

2.6 Native Vegetation and Barrier Wall

Preserving native vegetation necessary when land development projects arise and especially relevant in Oro Valley. Pima County released the Native Plant Preservation Manual, which serves as a guideline for the relocation and planting of native vegetation. This manual helps determine the issues of erosion and deterioration, which are prevalent in this site. The plants chosen for water harvesting areas include mesquite trees, ironwood trees, palo verde trees, ocotillo cacti, bottle brush and bird of paradise bushes. While these plants were chosen for their drought resistance, as previously discussed a drip system will be installed.

The Fairy Duster (Calliandra Eriophylla) will be used in order to combat the problem of high velocity runoff. This is one of the more common plants in the Sonoran Desert and is an excellent choice because plant's roots are a combination of a taproot and a network of shallow roots. These roots give it the ability to decrease the velocity of the incoming water without the risk of being washed out. This usage will allow the Fairy Duster to remain as a buffer for the runoff on the site. Beargrass (Nolina Microcarpa) will be used to combat erosion on the site. It is ideal because it is a small, native grass that has tough roots which hold soil into place. Since it is a small grass, it can be planted in any extra space available for landscaping and will decrease erosion around and in the basins. The Velvet Mesquite (Prosopis Velutina) and Foothill Paloverde (*Parkinsonia microphylla*) will be used for screening the basin. These native trees will be used because their large structure allows them to hide unsightly design elements. Their 15 to 20 foot height and 30 foot spread allow them to be placed around a small basin area and combat erosion along the basin area.

Due to the basin design consisting of side slopes of 3:1 and a depth of 3.65 and 3.75 feet, security barriers are required around the detention basin in accordance with Oro Valley Drainage Manual. A 42 minimum tall fence and a vegetative screening no less than 42 inches wide will be placed around the outside of the basin.

2.7 Grading

The southwest parcel was graded in order to ensure that all water on the parcel flowed into both of the detention basins located at the western side of the parcel. The primary flow paths are located along the centerline of the roadways. The fixed floor elevations were first established for the building at an elevation of 2660 ft, which is over a foot above the 100-year water surface elevation. Beginning at the fixed floor elevation of the building, the lot was then graded using various slopes ranging from 1.5% up to 3.8%. The basin is graded to the northwest at about 1.0% to ensure all the runoff can exit through the weir. Basin A and Basin B have outlet elevations of 2645.9 and 2642 feet respectively.

2.8 LEED

- **SS 5.1 - Site Development - Protect or Restore Habitat <u>1 Points</u>** - LEED requirements ask protection of 50% of the site, with native vegetation. This is done through water harvesting as previously discussed.
- **SS 5.2 - Site Development - Maximize Open Space <u>1 Point</u>** - LEED credit is given for projects that exceed local zoning open space requirements by 25%, which is accomplished.
- **SS 6.1 - Stormwater Design - Quantity Control <u>1 Point</u>** - LEED credits are received when stormwater plans prevent the peak discharge rate from exceeding pre development peak discharge rates for given storms. This is accomplished by reducing the runoff 15% from pre-existing conditions.
- **WE 1 – Water Efficient Landscaping <u>2 Points</u>** - LEED credits are received when the potable water consumption is reduced by 50% from a midsummer baseline. This is accomplished through desert landscaping, a cistern, and a drip system.

Total LEED Credit – <u>5 Points</u>

2.9 Cost Analysis

The final cost analysis was calculated assuming the cost of $2.50 per cubic yard to reuse the fill at 10% compaction, and $2.00 per cubic yard to import new fill. 5,733 cubic yards are being reused from cut on the site, which leaves 109 cubic yards left to be imported to finish grading the site. The cost analysis for hydrology was $14,551, which included basin excavation and the grading for the entire lot.

3. Transportation

3.1 Site Report

In order to design the lot, the first task was to designate the region. To “designate a region” means to classify it based on the definitions given by the American Association of State Highways and Transportation Officials (AASHTO) in their reference manual *A Policy on Geometric Design of Highways and Streets,* also known as “AASHTO Green Book.” This manual defines an urban region as, “an area that has a higher population density than other areas in the same region.” Cities as well as some large towns are usually referred to as urban regions. Quantitatively, an urban region is defined in the AASHTO manual as having 50,000 or more 0people. The assigned lot, including the parcel 3, fell into the region defined as suburban, defined by AASHTO as, “a residential area, with a lower population density than other areas in the same region.” Suburban areas are also classified as areas outside urban areas with a large amount of residential developments. Much of the area is still being developed, which is causing the lot and the surrounding areas to amass small amounts of buildings or attractions.

The general office building is located at the Northeastern parcel of Oracle Road and Linda Vista Boulevard intersection, which is a 4-directional signalized intersection. Along Oracle Road and North of the Linda Vista Boulevard intersection, there is an entrance in between the Northern and Southern parcels. In addition to this entrance, there is another entrance located on Linda Vista Boulevard.

The existing arterial, Oracle Road, has 6 lanes, 3 lanes traveling in both the North Bound and South Bound directions along with a median. The median allows traffic traveling South Bound to use the entrance located in between the Northern and Southern parcels. Linda Vista Boulevard is a 2 lane road, one of which is a lane for East Bound traffic while the other lane is for West Bound traffic. The site's exiting traffic, traveling South bound on Oracle Road, will have to make a right turn out of the lot due to the inability of making a direct left turn because it is blocked by the allocated curb. Thus, the traffic that wishes to travel South bound on Oracle will have to take a right and use the next intersection, Desert Sky and Oracle Road, to make a U-turn. However, they may choose to use the exit on Linda Vista Boulevard, going West Bound and make a left turn at the Oracle Road and Linda Vista Boulevard intersection, which gives the exiting traffic different options depending on their preference. A picture of the entire layout is shown in figure 5.

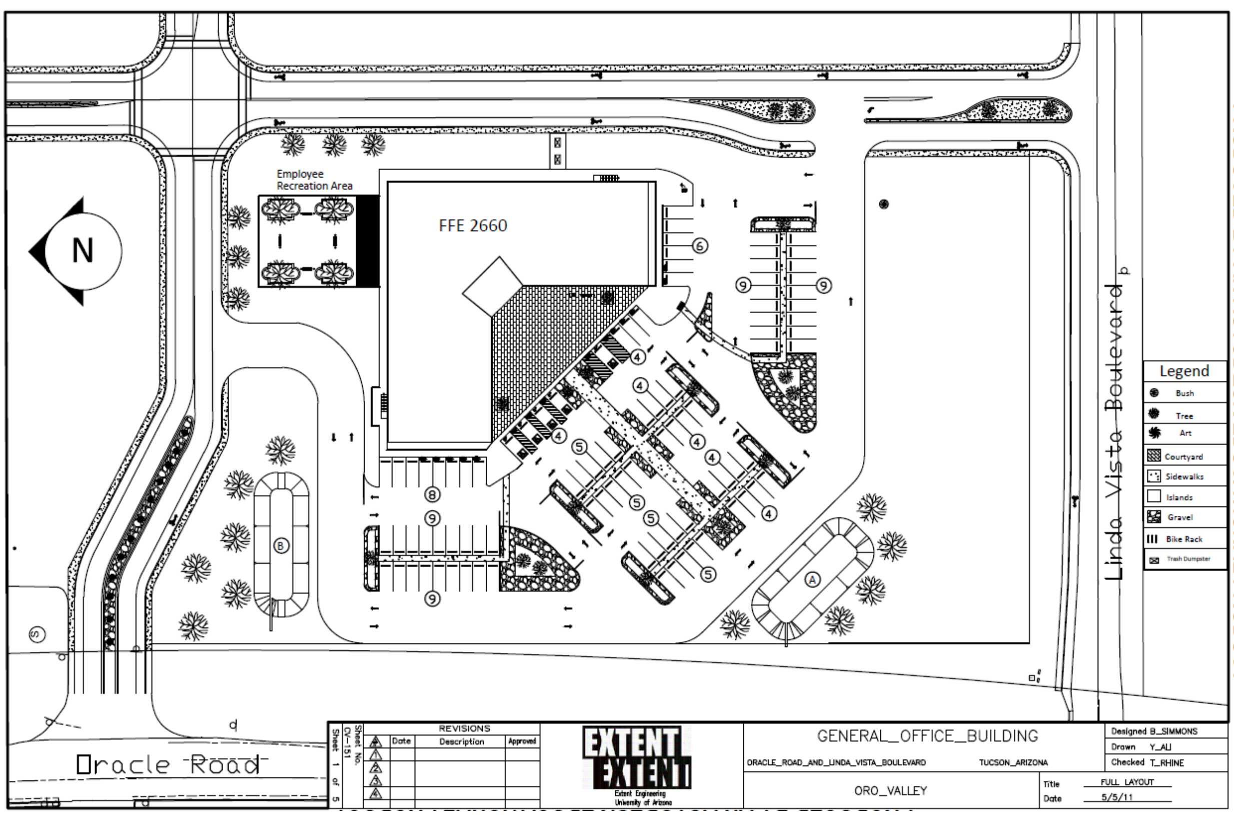

Figure 5: Final Layout

3.2 Layout Changes From the Previous Semester

The layout from last semester has been given a major overhaul. Almost every design aspect in the parking layout has been changed. The most major being the 45 degree angled straight parking design which was originally straight north south parking. This change allowed for vehicle circulation, pedestrian safety, amount of open space, and overall appearance to be improved. Parking islands were increased, in size, in order for them to be utilized as a traffic flow improvement, allowing for gentler curves and less hard turns needed to be taken by drivers. Increased parking space around the building allowed for closer entrance parking for the disabled. LEED credits were also gained by designating spaces for 5 alternative vehicles and 5 carpool/vanpool vehicles. The loading zone was made into a simpler design, allowing trucks to back-up next to the building and a larger outside roadway was created in place of the confusing emergency vehicle roadway, which ran behind the building, in the original. Also, a newly created employee recreation area was placed into the design. A layout of the landscaped parking area and disabled parking is shown in figure 6 and figure 7.

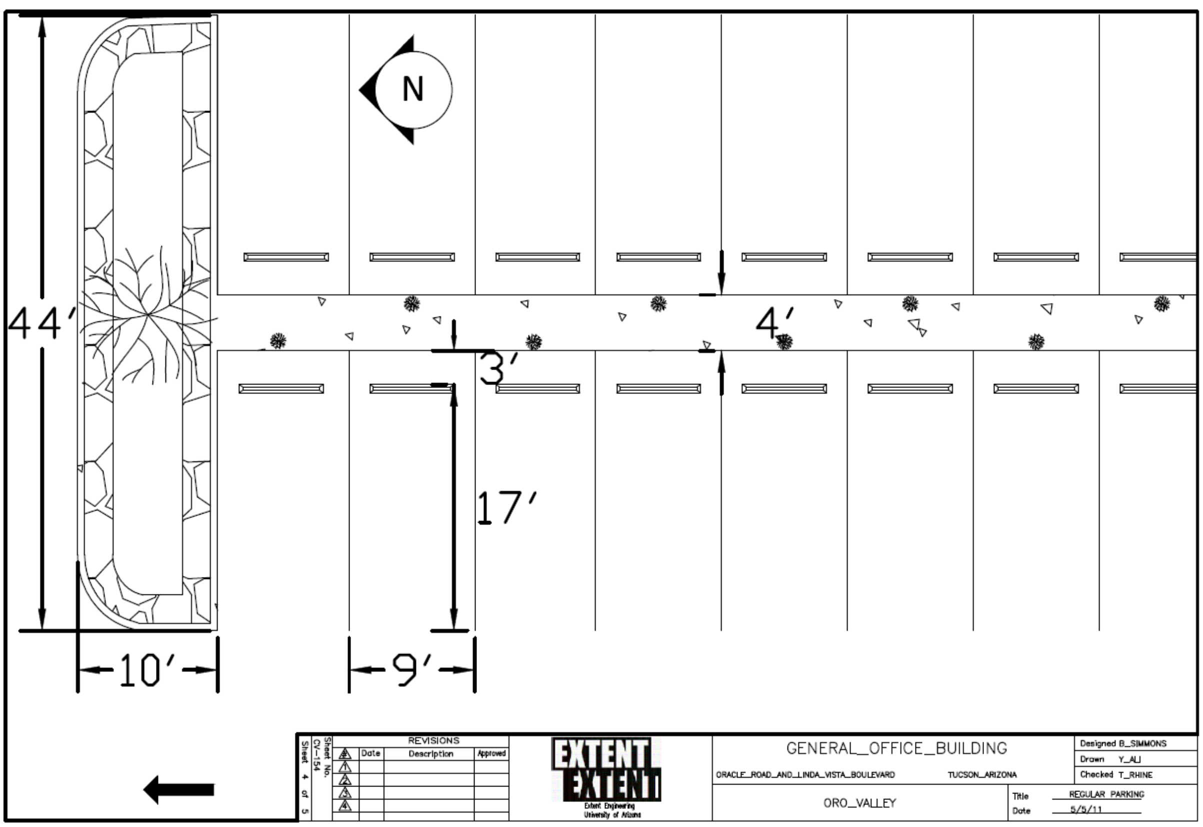

Figure 6: Landscaped Parking Area

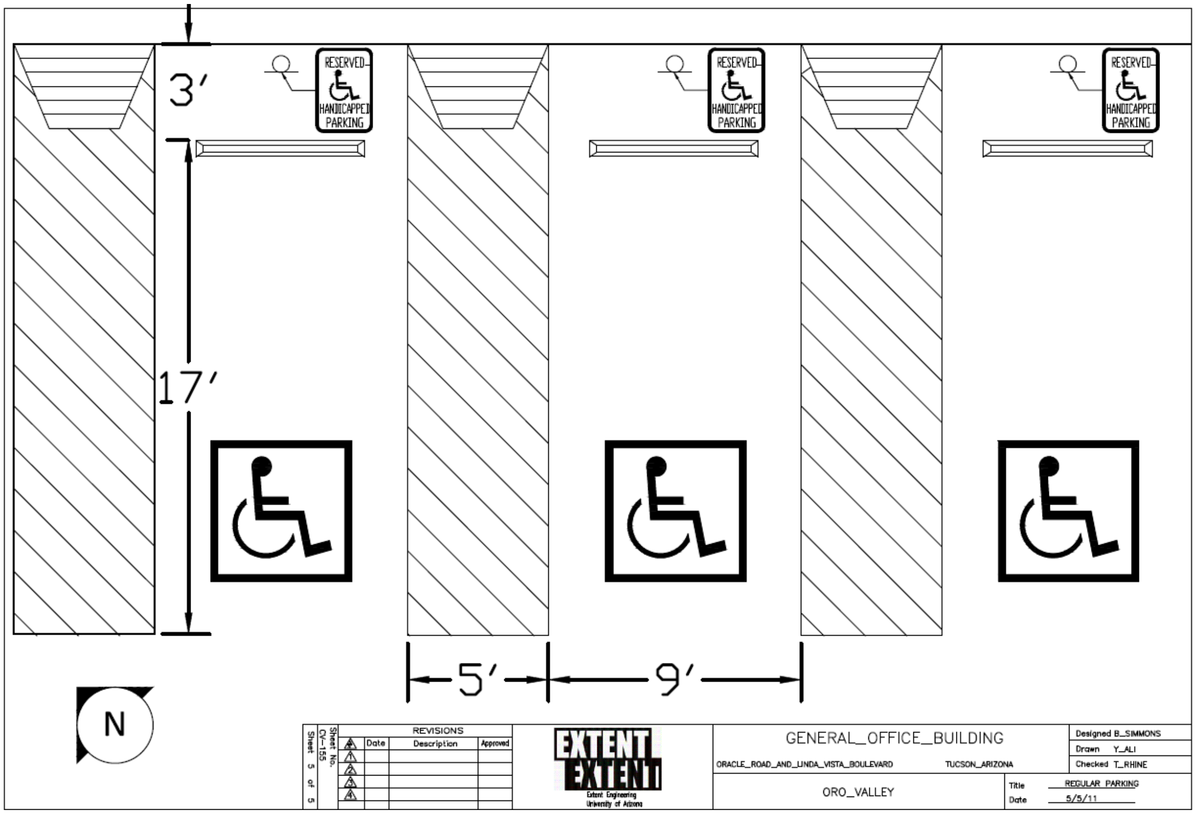

Figure 7: Disabled Parking

3.3 Parking

Using the Oro Valley zoning code, which takes into account the square footage and type of the proposed building, the amount of parking spaces needed to the accommodate a 31,000 square foot general office building was determined. The zoning code states that for every 1,000 square feet, 3 parking spaces need to be allocated. Therefore, the parking lot was found to require 93 total spaces, in this regards. The zoning code required that 4 out of the 93 spaces would need to be designated as disabled parking. The parking layout must also include a Van-Accessible space, increasing the total number of parking to 94 spaces. In regard to the dimensions, a regular parking space has a minimum width of 9 feet and a length of 20 feet. Mobility-Impaired Accessible Spaces or Disabled parking spaces must have a minimum width of 9 feet, a length of 20 feet and an adjacent 5-foot aisle for ease of vehicle entering and exiting. Van-Accessible Parking spaces have a width of 9 feet, length of 20 feet, and 8 foot adjacent aisle.

When dealing with transportation, differing modes of transit must also be taken into account. The main alternate mode of transit that was accounted for was bicycles. The number of required bicycle parking, as stated by the Oro Valley zoning code, is 1 bicycle parking space per 20 required vehicular parking spaces. Since the required parking for the lot is 93 spaces, 4 Class II bicycle spaces are required. Class II is defined as a space which provides a stationary and permanent object, such as fixed to the ground or attached to a structure, where the owner can lock their bicycle. Class II spaces are to be highly visible and close to the primary entrance of the building, without blocking pedestrian access. The bike paths on the roads were also developed with the lane widths being 6 feet, allowing and encouraging for safer and more comfortable travel for bicyclists.

Other parking layout design improvements were also deemed necessary in order to accommodate the proposed building. The loading zone was redesigned, allowing for better circulation of any and all loading vehicles as well as the ability for emergency vehicles to enter and exit this particular point if needed. Loading zone dimensions include a 60 foot long, 24 foot wide space which allows enough room for larger vehicles to enter by backing up.

3.4 Layout Development

While considering this particular design, main aspects and features were taken into account, including the type of access facility to be used, the placement of parking aisles, and the landscape islands. A 4-way stop intersection was chosen for the connection of the East bound collector road off Oracle Road and the North bound collector road off Linda Vista Boulevard. These two collectors were made to meet at a 4-way stop intersection, presented in Figure 8. Through the use of trip generation, the amount of trips calculated was not enough to justify a reason to design a traffic circle rather than a 4-way stop. The 4-way stop intersection's simple design makes travel easier for the everyday driver whereas a traffic circle, though used for traffic calming, is unnecessary and can possibly be more confusing to those on the road. The 4-way stop is shown in Figure 8.

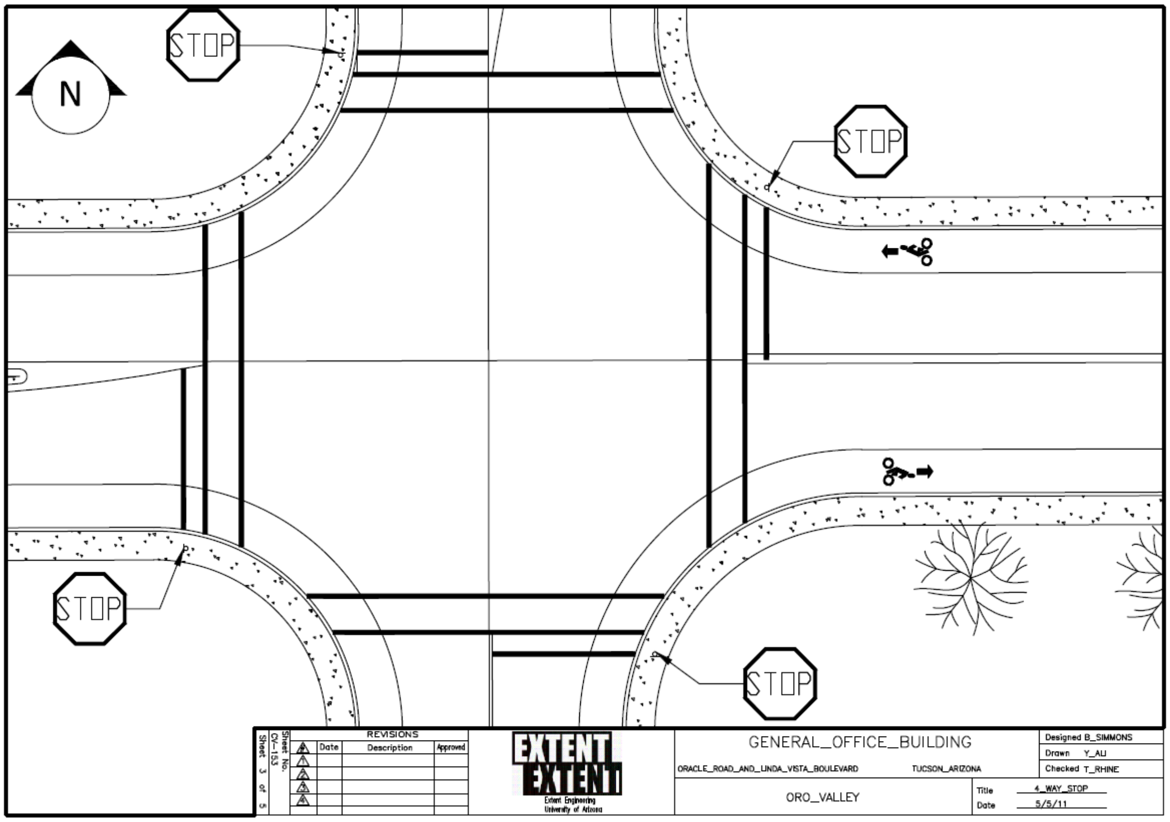

Figure 8: 4-way Stop

The road encompassing the proposed building was given a width of 24 feet or 12 feet per lane in each direction. The exceptions to this was the southeast side of the property, which was set at 40 feet because it was designed to be the roadway leading to the loading zone and also the outermost roadway was given an average width of 35 feet, allowing for larger emergency vehicles to enter and exit. The turning radii were based off AASHTO standards of 24 feet for the average vehicle. The loading zone was given a turning radius of 46 feet, allowing for the larger vehicles to enter and exit without hindrance. The North entrance and exit was given a width of 30 feet, allowing for emergency vehicles to safely enter and exit. The South entrance and exit was given a width of 36 feet.

Straight parking, at an angle, was concluded to be the best alternative for the site compared to diagonal parking. Straight parking allowed for a symmetrical layout, while keeping the design of the layout simple and user friendly. This layout causes very few issues in circulation, which provides fewer points of conflict in the Parking Area Access Lanes (PAAL). Single row slanted parking tends to need more width in one direction, which reduces the number of allowable space from East to West in the parking layout. The one way angled parking layout also causes more complicated circulation, and therefore was not chosen as the design method. Two row diagonal parking was not chosen because it uses too much space that could otherwise be used as open space in the parcel. Back-in diagonal parking was also eliminated as an option so that motorists would not need to stop in the travel lane before parking. Some of the signage used in the layout is shown below in Figure 9.

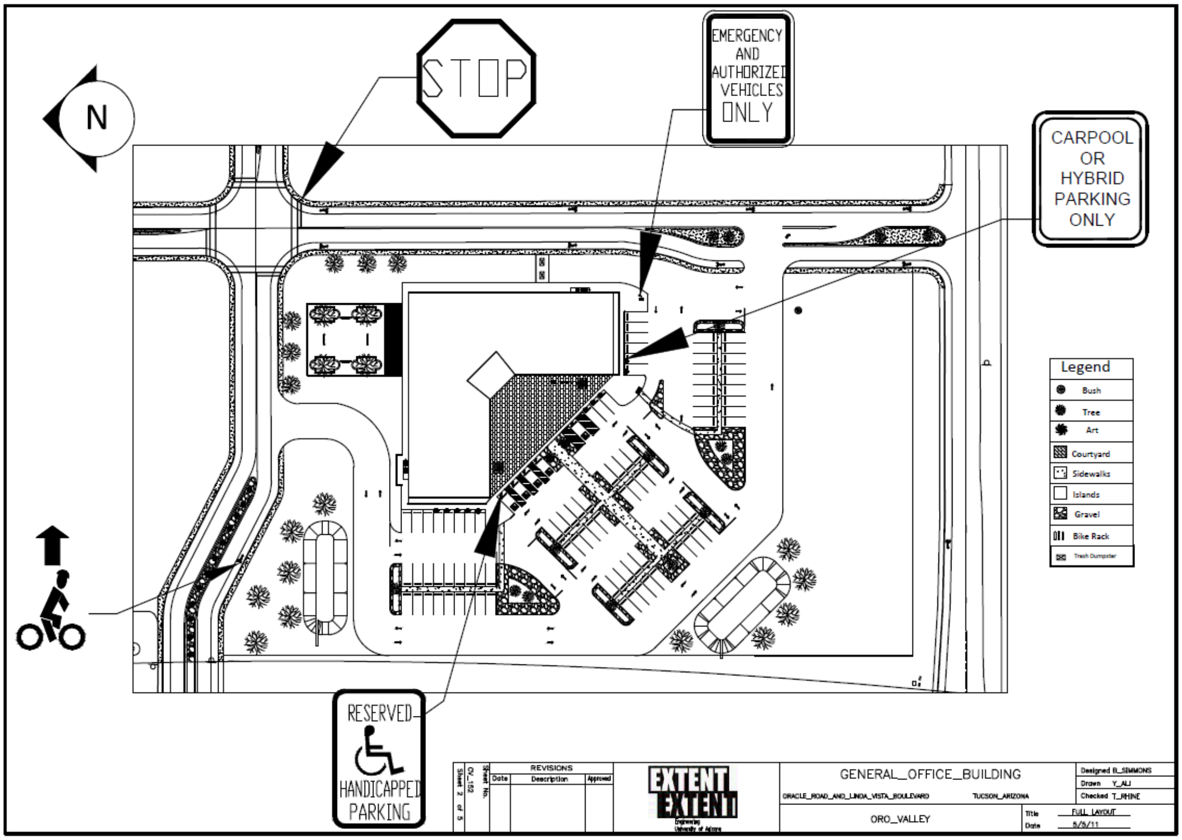

Figure 9: Layout with Signage

3.5 Landscaping

The placement of parking aisles was configured in order to occupy the least amount of space, while also keeping the layout aesthetically pleasing, simple and convenient to drivers. Parking aisle dimensions were 4 feet wide, leading to and from vehicles, connected to a pedestrian walkway 10 foot wide allowing access to the building's main entrance. Landscaped islands were placed at both ends of the 9 space rows. Small trees and bushes have been planted throughout the site for both aesthetic benefits as well helping to limit high heat effects. Curb widths included in the layout were set at 6 inches. Wheel stops with widths 7 inches and lengths of 6 feet were also included within the parking layout. Decorative artwork and a brick courtyard have also been implemented into the design. An employee recreation area with benches has also been placed on the northeast side of the property.

3.6 Trip Generation & HCS Analysis

Another top priority to the site design was the amount of future incoming and outgoing traffic of the lot. This helps for the future planning of the site's infrastructure. The Trip Generations for the parcel were calculated as follows:

- Average Vehicle Trip Ends vs. 1000 GSF/Day: 342 Average Vehicle Trips

(171 Vehicles Entering & 171 Vehicles Exiting)

- Range of Rates (Average Vehicle Trip Ends vs. 1000 Sq. Ft Gross Floor Area):
  111– 893 Average Vehicle Trips

- A.M. Peak Hour Average Rate: 48
  (42 Vehicles Entering & 6 Vehicles Exiting)
  A.M. Peak Hour Range of Rates: 19 – 185.

- P.M. Peak Hour Average Rate: 46
  (8 Vehicles Entering & 38 Vehicles Exiting)
  P.M. Peak Hour Range of Rates: 15 – 198

Live traffic counts were also performed in order to get an idea of the actual number of vehicles along Oracle Road and Linda Vista Boulevard. Times were taken in two count periods, one in the AM peak hour period (7:00 AM to 9:00 AM) and the other in the PM peak hour period (4:00 PM to 6:00 PM). Table 5 shows the total amounts taken in both time periods and a pictorial representation in Figure 10.

Table 5: Traffic Count Totals

TOTALS					TIMES	
NB Oracle	SB Oracle	EB Linda Vista	WB Linda Vista	Total	Start Time	End Time
2488	4591	160	101	7340	7:00 AM	9:00 AM
4580	3365	122	118	8185	4:00 PM	6:00 PM

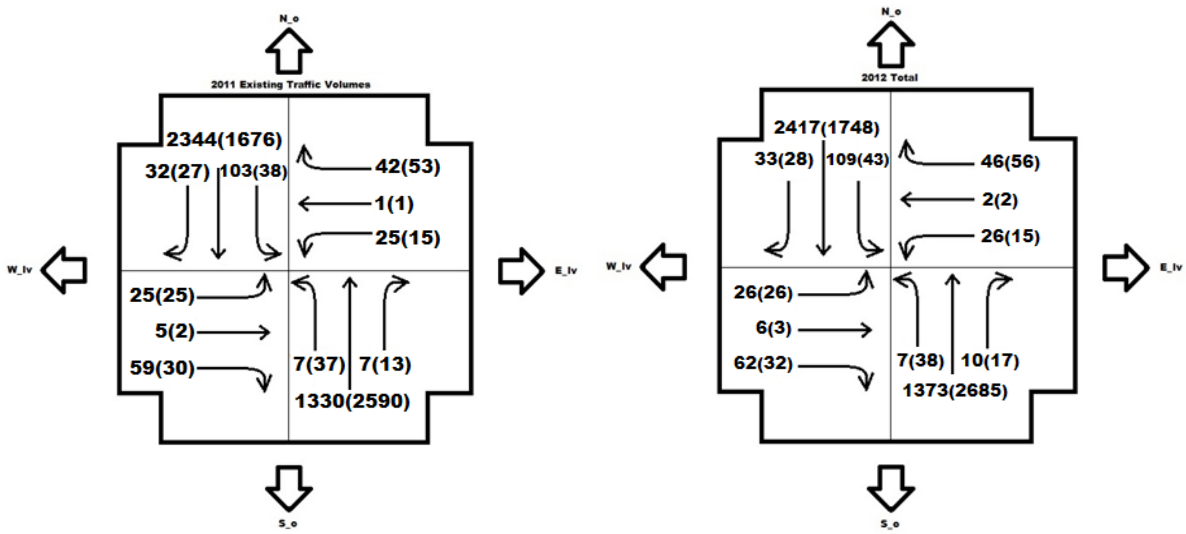

Figure 10: Traffic Counts

From both the Trip Generation Values and Traffic Count totals, HCS (Highway Capacity Software) was able to be implemented in order to find the needed signal timing which would help determine the best results for LOS (Level of Service) at the intersection of Linda Vista Boulevard and Oracle Road.

The results from the HCS+ are as follows:

Table 6: HCS+ Results, AM Peak Hour

AM Peak Hour Result	2011 Existing Conditions	2012 Total Conditions
Northbound Approach	LOS B	LOS B
Southbound Approach	LOS C	LOS C
Eastbound Approach	LOS E	LOS E
Westbound Approach	LOS D	LOS E
Intersection	LOS C	LOS C
Cycle Length	115 sec.	120 sec.
Intersection Delay	25 sec/veh	25.4 sec/veh

Table7: HCS+ Results, PM Peak Hour

PM Peak Hour Result	2011 Existing Conditions	2012 Total Conditions
Northbound Approach	LOS C	LOS C
Southbound Approach	LOS B	LOS B
Eastbound Approach	LOS E	LOS E
Westbound Approach	LOS E	LOS E
Intersection	LOS C	LOS C
Cycle Length	124 sec.	130 sec.
Total Delay	25.8 sec/veh	26.1 sec/veh

The maximum cycle length was set to equal 130 seconds, and the sequences chosen were set to actuate operation. The same phasing sequence was used for AM and PM peak hour traffic counts because there were no significant deviations between the two counts, of the existing traffic data, in terms of volume. However, the project is not expected to be completed in the near future. Therefore, a 3% inflation for the current data was included in the calculations. There are many approaches that introduce this inflation into the current data's count. One option chosen was increasing the peak hour volume that was extracted from the 120 minute counting periods, broken into 15 minute sections, of both the AM and PM counts. After increasing the total peak hour volume, the turn movement's numbers are entered into their slots according to the specific turn movement percentage of the total counts. The inflation process did not affect the level of service for the intersection and it remained at LOS C due to number of lanes at the existing intersection, which can easily handle these increased traffic volumes.

3.7 Cost Analysis

In regard to the amount of costs contributed by transportation design, the two largest total expenditures was ABC (Aggregate Base Course) and AC (Asphalt Concrete), which would be used in order to create the parking lot and the roads within the site. The total cost came to a total of $205,000.

3.8 LEED

- **SS 4.1 - Future Bus Stop Location(s) - <u>6 Points</u>** - LEED requirements ask for, at most, a ¼-mile walking distance to one or more bus stops. Oracle would be a favorable option for inclusion of a bus stop, but sidewalks/walkways would need to be provided if outside the site. Sun Tran routes 312X and the 401 will be negotiated to drive by the parcel and contain a bus stop there.

- **SS 4.2 – Bicycles - <u>1 Point</u>** - LEED credit is given for facilities including secure bicycle racks and/or storage, within 200 yards of the building entrances, for 5% or more of all office users. Credit is also given to the presence of showers or changing facilities for 0.5% of FTE (full time equivalent) occupants.
- **SS 4.3 and SS 4.4 - Low-emitting/Fuel Efficient Vehicles & Carpools/Vanpools - <u>3 Points & 2 Points</u>** - LEED credits are received when 5% of the total parking capacity is given to Low-emitting/Fuel Efficient Vehicles. Also, additional credit is received when 5% of the total parking is allocated for carpools/vanpools.

Total LEED Credit – <u>12 Points</u>

4. Structural

4.1 Architectural Floor Plan

Our building consists of two tier levels with a total area of 31,600 square feet. The first level has an area of 19,820.5 square feet while the second level has an area of 11,756.3 square feet. Our building features a trapezoidal atrium that has a glass panel slant down from the second floor to the top of the first floor. The architectural floor plan was created around a centralized area, which was chosen as the atrium. The space of the atrium was extended to the back of the building, so it could be a continuous space and a meeting point for the rest of the building. All the walls were drawn to be six inches thick so that our HSS columns could fit inside. Accommodations for ADA were considered in our floor plan, such as the size of the door, the hallway space, restroom space, and the elevator. There is one staircase in the center of our building for inside usage, and two fire escape stairs for the North and South sides of the building. Another feature that is unique to our building is the deck. Extent Engineering decided to use the excess roof area from the larger bottom tier as a deck. The deck also includes part of two conference rooms that extend onto the deck from the second tier. The exterior parts of the conference rooms will be glass like the atrium.

The International Building Code, IBC (2006), was consulted to obtain the occupancy load of the building. The occupancy load determines the number of restroom stalls needed per gender, the number of water fountains, and many other features of the building. The occupancy load for our building was 316, although separate occupancy loads were calculated for each floor. It was assumed that the building would have 50% male employees and 50% female employees, which was a necessary assumption to compute the number of restroom stalls needed for each gender. The number of stalls and sinks is directly related to the occupancy load for each floor. For the first floor, which has an occupancy load of 198, the women's restroom should have three stalls with three sinks. For the men's restroom, a total of three stalls and three sinks were required. In the IBC, [P] section 419.2, it states that 2/3 of the stalls can be replaced as urinals. Therefore the men's restroom for the first floor should have two urinals, one stall, and three sinks. For the second floor, the occupancy load is lower at 118 since it is a smaller area. The upstairs women's restroom requires three stalls and 2 sinks. For the men's restroom, two urinals, one stall, and two sinks are necessary to meet code standards. The number of water fountains was determined in a similar way and 2 water fountains were required per floor, one of which has to be ADA approved. The service sink that is also necessary to have in the building will be placed in the janitor's closet near the restrooms on the first floor.

4.2 Gravity Loads

The gravity loads for the building were required for the derivation of the lateral seismic loads as well as the design of the composite floor and the beams, columns, and foundation making up the gravity load resisting system. The gravity loads were computed for dead and live loads and computed separately for each floor as well as the roof. Loading conditions ultimately fell into two ranges which provided and economic option for downsizing structural elements under the lighter loading conditions. Foundations, therefore, were designed in two sizes. Columns and beams, on the other hand, were designed only for the highest loading condition due to the added difficulty in constructability variable sizes of members would have introduced.

4.2.1 Dead Loads

The dead loads were computed by summing the structural self weight of each floor and the superimposed dead loads comprised of loads such as mechanical equipment, floor finishing (carpet), and so on to produce a dead load based on the area. The weight of walls was taken as the per [vertical] square foot weight multiplied by the height and perimeter of the wall. The areal weight of the composite floor system and its constituent parts was taken from the United Steel Deck Inc. design manual.

Column tributary areas were taken as one half the distance between the column of interest and the nearest load bearing structural element, either the masonry wall or another column. This resulted in tributary areas centered on the column and roughly the size of one column bay (25' by 25'). The tributary area was then multiplied by the areal dead load of the ground and second floors, and roof when applicable, and these tabulated for future use. The roof and second floor dead loads were summed to produce the column loads and the maximum column load was used as the column design load. The roof, second floor, and ground floor dead load were summed and used as the design dead loads for column footing design. The dead load for the masonry wall was taken as the sum of the dead load for the area one-half-bay's distance from the perimeter of the building to the adjacent columns and then divided by the length of the perimeter to produce a dead load per unit length for use in the design of the masonry wall as well as the strip footing supporting the wall.

4.2.2 Live Loads

Live loads, unlike dead loads, vary greatly from one part of the structure to another. The magnitude of the dead load for an area is dependant on the designated usage of the area. The area usage and boundaries were taken from the architectural floor plan of the building and superimposed on the column tributary area. The total live load for a columns tributary area was computed by summing the live loads of its constituent areas, which themselves were assigned live loads equal to the product of their areal live load magnitude (taken from the design guidelines published in ASCE 7-98 table 4-1) and the area within the tributary boundary. Alternatively, the total live load of a column's tributary area could be described as the product of the tributary area and the area-weighted average of the live load magnitudes of its component areas. The live load for the masonry wall was taken as the highest loading condition for the area along the wall multiplied by one-half-bay's distance from the perimeter of the building to the adjacent columns to produce a live load per unit length.

4.3 Lateral Loads

The lateral loads were very important to the design of our building because the governing loads were the lateral loads, specifically seismic load. The lateral loads determined the design of the shear walls. Although seismic load governed, wind load still had to be calculated to make sure that it did not have to be taken into consideration.

4.3.1 Wind Loads

The wind loads were analyzed using a nodal approach, meaning the equivalent point load in kips was found for the building. Since our building consists of two floors, only two nodal forces were obtained. The forces were determined using the Main Wind Force Resisting System – Method 2 from ASCE 7-05. The three different wind loads that were calculated for the different sides of the building included the windward side, leeward side, and uplift. Although the main formula is consistent for all three sides, the main change is the wall pressure coefficient, C_p, for each side. The formula used was $P = q_z\ x\ G\ x\ C_p$, which ignores the effects of internal pressure. The windward side is assumed to go into the building, while both the leeward and uplift sides assume that the forces are leaving the building. In order to use the appropriate values for the variables, the type of exposure and case had to be determined. For example, our building was assumed to be Exposure B, Case 1 because there are limited buildings in the surrounding area and it is a low rise building that does not reach 60 feet in height. Also, the building is categorized as Category II since it is not for over 300 people. Other coefficients include the directionality factor, basic wind speed for Oro Valley, importance factor, and gust factor. All of these coefficients were found in the ASCE 7-05. Once all the important variables were identified, iterations for the windward side were calculated based on the height and the tributary width. Our column layout consists of 25 foot bays, which is why our tributary width was 25 feet. The results of the total wind loads that were calculated were

8.39 psf for windward side, 5.24 psf for the leeward side, and 9.44 psf for uplift. It is important to mention that the calculations give negative values for the leeward side and uplift because these loads are acting away from the building

4.3.2 Seismic Loads

To find the lateral load on the building due to seismic earth movement the process detailed in ASCE 7-98 was followed. Several regional and building related variables were determined before the seismic load derivation could begin:

- S_S and S_1, the spectral response acceleration for short and 1 second periods respectively, and taken as .289 and .082 respectively, were taken from the USGS Seismic Hazard Mapping Project for the region in which the building is to be located
- R, the response modification coefficient, was taken as 4 for a bearing wall system consisting of special reinforced masonry shear walls.
- I, the occupancy importance factor, was taken as 1 for a seismic use group of II, for high occupancy buildings with non-critical function.
- Site Class, based on soil conditions. Our site class is conservatively taken as D

Next, it was important to derive the value of C_s, the seismic response coefficient, as well as its upper and lower bounds. The equation is $C_s = S_{DS} / (R/I)$.

Where S_{DS} is the design spectral response acceleration for short periods, taken as two-thirds of maximum spectral response for short periods, S_{MS}, which itself is defined as: $S_{MS} = F_aS_S$

Where F_a, the site coefficient for S_S, is a function of site class and S_S. F_a is taken from table 4.1.2.4a in ASCE 7-98 through linear interpolation as 1.57. With this information we can determine C_s: $C_S = 0.076$

The upper and lower bounds of C_S are functions of S_1, the spectral response acceleration for 1 second periods: $C_{S,min} = 0.1S_{D1}I$ $C_{S,max} = S_{D1} / [T(R/I)]$

Where T is the fundamental period of the structure and S_{D1} is the design spectral response acceleration for 1 second periods. S_{D1} is taken as two-thirds of maximum spectral response for 1 second periods, S_{M1}, which itself is defined as: $S_{M1} = F_vS_1$

Where F_v, the site coefficient for S_1, is a function of site class and S_1. F_v is taken from table 4.1.2.4b in ASCE 7-98 through linear interpolation as 2.4. The fundamental period of the building, T, may be taken as no greater than the product of C_U and the recommended initial approximation of T, T_a. T_a is to be taken as $0.1N$, where N is the number of stories in the building, not to exceed 12. For the building being evaluated, N is equal to 2, $T_a = 0.1N = 0.2$

And C_U is defined in table 5.3.3 in ASCE 7-98 as a direct function of S_1. For an S_1 value of 0.082:

$$C_U = 1.2,\ T_{max} = 0.3$$

For the seismic analysis here, T was taken as T_{max}. The upper and lower bounds of C_S can now be determined: $C_{S,min} = 0.013$ $C_{S,max} = .109$

The value of C_S previously calculated is within the upper and lower bounds and was therefore taken as the value of C_S for the purposes of determine the lateral seismic load. Once CS has been determined, the total base shear due to seismic ground movement, V, can be determined. V is the product of the building weight, W, and the seismic response coefficient, C_S, so $V = C_S W$.

F_X, The distribution of V on each story was determined by multiplying V by the vertical distribution factor, C_{VX}, which itself is the weighted average of the product of the floor weight W_X and the respective floor height, h_X: $C_{VX} = W_X h_X / \Sigma(W_i h_i)$ $F_X = C_{VX} W_X$

To determine F_X for each floor, a table was made containing the weights and heights of both floors in order to dynamically calculate F_X given the input of C_S and the floor dead loads.

4.4 Shear Walls and Steel Stud

The main purpose of designing masonry shear walls was for all of our lateral loads to be resisted by the walls. This design allowed for columns to be designed for axial loads only. In order to start the design of the masonry, only the governing lateral load was taken into consideration. For our building, the seismic loads were the governing lateral load and it was assumed that if our shear walls could withstand the larger lateral load than the smaller lateral load would not cause any problems.

Allowable Strength Design (ASD) was used for the masonry shear walls because it is the typical design procedure used in practice. ASD allows for a 0.7 reduction in seismic because of the load combination of 0.7E. Also, the allowable shear stresses can be increased by 1.33 for duration increase. The initial steps to determine the how to analyze the building was to decide on the types of diaphragms that would be used. Due to the shape of our building, it was obvious that there would be considerable torsion, which is why it was decided to analyze the first floor as a rigid floor diaphragm. A rigid floor diaphragm assumes that there is a shear force as well as a torsional force placed on the building. However, not the entire building had to be analyzed as a rigid diaphragm; the roof was able to be analyzed as a flexible roof diaphragm. The flexible diaphragm does not translate any torsional forces, which makes a simpler design process.

The flexible roof diaphragm was designed for the governing lateral load, seismic, of 38.50 kips. In order to determine the shear force that had to be resisted by each wall, the tributary area for the load. The lateral load was divided by the tributary height and then multiplied by the tributary length of the wall. Once the distribution became apparent because of the tributary length of the wall, the shear force was multiplied by the reduction factor of 0.7. The final value of the actual shear force was multiplied by 1000 lbs and divided by the actual length of the wall. Since all of the walls were analyzed, only the wall with the largest shear force was designed. The flexible roof diaphragm requires anchor bolts of 1" which has an allowable shear of 2050 lbs. The spacing of the bolts was calculated manipulating the allowable shear, the 1.33 increase, and the actual shear force. The final calculation of spacing yields 11" max, yet 8" was chosen for masonry modular.

For the rigid floor diaphragm, the architectural floor plan of the building had to be considered. The architectural features that were most important were the openings in the walls, such as windows and doors. In the analysis, it was assumed that the window height and the door height are the same. Also, the area under the windows was not considered for simplicity. Each shear wall in the building was separated into piers. Piers are the sections in between the openings of the wall. For example, if there are two windows in a wall, it would have three piers, A, B, and C. This same procedure was repeated for each shear wall in order to determine the shear force distribution in each pier. The design was made for the most rigid wall, which would carry the most shear force and have the most overturning moment. The rigidity of all the walls was compared and Wall 4 was the most rigid. It was designed to carry the largest portion of the governing 165.3 kips from the seismic load, which was a total of 96.68 kips, which was reduced to 67.67 kips. When compared to the allowable stress, the shear stress of every pier is under the acceptable values. This means that there is no additional shear reinforcing without considering torsion.

However, it is important to consider torsion in the building. In order to consider the torsion, the center of mass and center of rigidity had to be calculated for the whole building. For the center of mass, the weight of the walls and the geometry had to be taken into account. The results for the center of mass were 84.17 feet in the x-axis and 82.02 feet in the y-axis. For the center of rigidity, the relative rigidities of each wall were calculated and the results were 112.30 feet in both the x and y direction. Other considerations for torsion include torsional eccentricity, to which 5% minimum accidental eccentricity was added, seismic base shear, and the torsional moment for North to South seismic force and East to West seismic force. The torsional eccentricity had the values of -19.38 feet for the x direction and -21.53 for the y direction. The seismic base shear was determined by the weight of the walls multiplied by the C_s coefficient. The torsional moment due to the North to South seismic force was -11384.43 kip -feet while the East to West seismic force was -12646.26 kip – feet. The total resisting force was the combination of the force due to shear and the force due to torsion. Although both the seismic forces gave the largest total force of 293.04 kips for Wall 3, the East to West seismic force governed the design. Therefore, the piers for Wall 3 were distributed with the new total force. The updated actual shear stress proved to be lower than the allowable shear stress, which means that there is no additional shear reinforcing required. To resist any overturning moment, the anchor bolts will help relieve this stress in the building. Finally, for the axial reinforcing, the total steel in the wall should be 1% of the cross-sectional area. After the calculations, a result of 8" CMU with #5 bars at 40" on center will be a sufficient design to meet this requirement.

For our building, steel stud wall for the second floor was the best material since a continuation of masonry would be too heavy and it is a cheaper alternative to steel framing.

The steel stud selected was 600S162-43. This steel stud will be sufficient for the forces that need to be resisted by the second floor. The steel stud will be shear walled meaning that it will be expected to carrying a portion of the shear force, similar to any other masonry wall in the building.

4.5 Composite Deck Design

A composite deck design was chosen over a simple concrete slab on joist design in order to reduce the size of joist required to support the floor system on the second floor. By fixing the concrete deck to the joist the slab and beam are allowed to act as a composite member to resist moment and increase stiffness. The arrangement places the concrete and compression and the steel in tension, similar to the way in which steel reinforcement is utilized in concrete beams.

Moment capacity and deflection were calculated by deriving the transformed moment of inertia of composite member. Several assumptions were made for the sake of simplicity; it was assumed that additional moment in the member not due to live or dead load of the floor was negligible, that stud spacing was sufficient to develop fully composite action. Furthermore, to simplify the iterative design process, only composite deck arrangements in which the neutral axis was located in the slab were considered. Since the size of the required to satisfy this condition was within the minimum thickness required for fire proofing, this condition was non-restrictive.

Several composite deck designs were sufficient to adequately support the live and dead loads of the floor system, so a cost analysis was done in order to determine the economical option. The primary variable in the cost of the composite floor system was the distance between floor joists which the deck is required to span:

- A longer span requires
  - A thicker gage and greater depth of steel deck in order to support the concrete without significant pooling during construction phases
  - A larger beam size to resist the relatively larger load to the increased tributary area of the joist.
- A shorter beam requires
  - A greater number of joists per bay
  - A greater number of joist-on-girder shear connections per bay

Several design options were calculated using a spreadsheet, ranging from the maximum allowable span of 12.99" (12' 6" for the purposes of our 25' bay size) to the minimum span which produced savings in beam weight (after the minimum slab depth had already been reached). The design option with the longest span was found to be the most economical. All design values were dictated by the United Steel Deck Inc. design manual.

4.5.1 Prefabricated Truss Members

For the second story roof, which supports relatively light loads, a prefabricated cold formed steel roof truss was found to be the most economical design option. Similarly, an exposed cold formed steel truss system was found to be the most economical option for the support system integrated into the glass roof and walls of the atrium.

The roof trusses will be incorporated in the beam system in place of a a concrete slab and will bear on joists. The roof trusses will be placed every two feet, closely enough to allow the joist loading to be approximated by a distributed load equal to the roof load (taking into account the weight of the trusses themselves).

4.5.2 Girder and Column Design

The rigid envelope provided by the masonry shear wall allowed the building design to forgo the use of moment resisting frames and instead make use of only bolted shear connections. Under these connection conditions a simply supported beam model was appropriate for the design of the girders. The composite deck design called for a joist spacing of 12’ 6”, which translates into a centered point loading of the girders. The assumption was also made the girder would be fully braced by the composite floor system. A Steel W selection was selected from the AISC Steel Design Manual to resist the maximum moment according to LRFD design criteria.

The column was designed under the assumption that the base of the column was a fixed connection at the column footing and a pin connection at the roof, and that the column was braced at the center of its span, where it connects to the second floor composite floor system. For ease of utility and constructability the design was to be limited to HSS 6x6 in order to fit within standard interior wall thicknesses. Due to the geometry of HSS members the beam bearing on the column has an innate eccentricity caused by the distance between the connection and the central work point of the beam-column connection (unlike W sections, which when bolted to the web are effectively connected directly to the work point of the beam-column connection. A sample calculation, however, showed that the moment due to eccentric loading was negligible for our loading conditions and an HSS 6x6 section. A thickness was selected using AISC Steel Design Manual Table 4-4.

4.6 LEED

- **Water Efficiency 2 and 3 - <u>6 Points</u>** - LEED requirements ask for reusing potable water for building sewage, such as toilets or urinals and irrigation.
- **Material Resources 2 – 6 - <u>8 Points</u>** - LEED credit is given for facilities that control their construction waste management. The reuse of materials and use recycled content as well as regional materials. One point will be given for rapidly renewable materials as well.
- **Indoor Environmental Quality 3.1 – 8.2 - <u>13 Points</u>** - LEED credits are received when low-emitting materials throughout the building, such as adhesives, paints, flooring systems, etc. Other areas include construction IAQ Management Plan, controllability of systems, thermal comfort, and daylight and views.

Total LEED Credit – <u>29 Points</u>

4.7 Cost Analysis

The cost analysis for structural was $3.95 million, which included floors, columns, CMU, ground concrete, roof joist, roof girder and roof truss.

Conclusion

The Final Site Layout can be found on the following page, which shows all of the previous discussed decisions. All of the different aspects in the layout that were mentioned are shown within the following figure. The detention basin, parking layout, and column layout were carefully placed and thought out to create the optimal design for the parcel. The total cost of our project is about $4,209,378.

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Appendix

Geotechnical Appendix

AASHTO (2004)
Bearing Capacity (SPT Data)

$$q_{ult} = 200N_1 B(C_{w1} + C_{w2}\frac{D_f}{B}) \text{ (psf)}$$

$$N_1 = N\left(\frac{2000}{\sigma'_z}\right)^{\frac{1}{2}};\ C_N \leq 2$$

BH: B-10 **FS:** 3

Depth (ft)	N	γ (pcf)	σ'$_z$ (psf)	C_N	N_1
2	21	114	228	2.0	42
5.5	13	108	606	1.8	24
11	20	114	1233	1.3	25

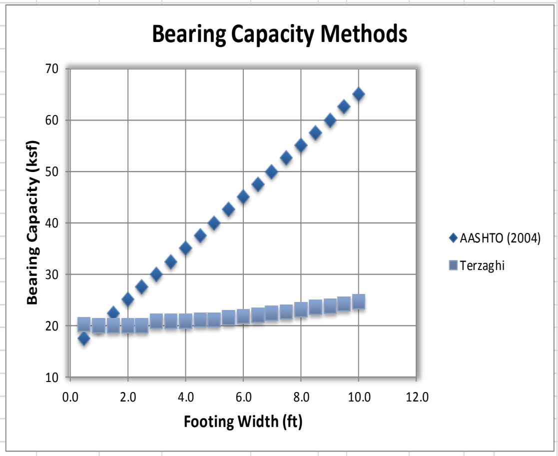
Cw1 = 1.0 N1,ave = 25

Cw2 = 1.0 Df = 3 (ft)

q_{ult}	q_a	B
18	6.20	0.5
20	7.03	1.0
23	7.86	1.5
25	8.70	2.0
28	9.53	2.5
30	10.36	3.0
33	11.20	3.5
35	12.03	4.0
38	12.86	4.5
40	13.70	5.0
43	14.53	5.5
45	15.36	6.0
48	16.20	6.5
50	17.03	7.0
53	17.86	7.5
55	18.70	8.0
58	19.53	8.5
60	20.36	9.0
63	21.20	9.5
65	22.03	10.0

Table A.11 Correlation for Coarse-Grained Soils

N	N60	Compactness	g		(kN/m3)	g		(PCF)
0 - 4	0 - 3	Very Loose	11	-	13	70	-	83
4 - 10	3 - 9	Loose	14	-	16	89	-	102
10 - 30	9 - 25	Medium	17	-	19	108	-	121
30 - 50	25 - 45	Dense	20	-	21	127	-	134
>50	>45	Very Dense	>21	-		>134	-	



Theoretical Bearing Capacity

E-5

Ultimate Net Bearing Capacity (ESA):

$$ESA:\ q_u = \gamma D_f(N_q - 1)s_q d_q r_q w_q + \frac{1}{2}\gamma B N_\gamma s_\gamma d_\gamma r_\gamma w_\gamma$$

Design Parameters:

D_f	3.5	(ft)	t	2	(ft)	D_r	80	(%)
B	7	(ft)	L	7	(ft)	B'	7	(ft)
γ	115	(pcf)	ϕ'	33		L'	7	(ft)

Bearing Capacity Factors:

N_q	26.1	N_g	26.6
s_q	1.6	s_g	0.6
d_q	1.1	d_γ	1.0
r_q	1.0	r_g	1.0
w_q	1.0	w_γ	1.0

Structural Load	258.06	(k)		
Weight of Concrete	14.7	(k)		
Design Load	272.76	(k)		
Design Moment	119.61	(k -ft)	e =	0.4385

q_u =	25.31	(ksf)
$(\sigma_a)_{min}$ =	7.66	(ksf)
FS =	3.31	

Elastic Settlement		
Quantity		
Soil Properties	**Inputs**	**Units**
Allowable Vertical Load Q_a	272760	lb
Elastic Modulus E_u	1254000	psf
Poisson's ratio, ν_u	0.3	
Foundation Factors		
Shape factor, μ_s	0.36	
Neglect embedment factor	NO	
Neglect side wall factor	YES	
Embedment Factor, μ_{emb}	0.91	
Side wall Factor, μ_{wall}	1.00	
Foundation Parameters		
Footing type	Non-circular	
Length of circumscribed rectangle, L_1	7	ft
Width of circumscribed rectangle, B_1	7	ft
Embedment Depth, D_f	3.5	ft
Footing thickness	2	ft
Perimeter of Footing	28	ft
Outputs		
Actual area of wall in contact	56	ft^2
Elastic Settlement, r_e	**0.224**	in

Theoretical Bearing Capacity

F-4

Ultimate Net Bearing Capacity (ESA):

$$ESA:\ q_u = \gamma D_f(N_q - 1)s_q d_q r_q w_q + \frac{1}{2}\gamma B N_\gamma s_\gamma d_\gamma r_\gamma w_\gamma$$

Design Parameters:

D_f	3.5	(ft)	t	2	(ft)	D_r	80	(%)
B	6	(ft)	L	6	(ft)	B'	6	(ft)
γ	115	(pcf)	ø'	33		L'	6	(ft)

Bearing Capacity Factors:

N_q	26.1		N_g	26.6
s_q	1.6		s_g	0.6
d_q	1.2		d_γ	1.0
r_q	1.0		r_g	1.0
w_q	1.0		w_γ	1.0

Structural Load	167.38	(k)		
Weight of Concrete	10.8	(k)		
Design Load	178.18	(k)		
Design Moment	119.61	(k -ft)	e =	0.6713

q_u =	24.77	(ksf)
$(\sigma_a)_{min}$ =	8.27	(ksf)
FS =	2.99	

Elastic Settlement		
Quantity		
Soil Properties	**Inputs**	**Units**
Allowable Vertical Load Q_a	178180	lb
Elastic Modulus E_u	1254000	psf
Poisson's ratio, ν_u	0.3	
Foundation Factors		
Shape factor, μ_s	0.39	
Neglect embedment factor	NO	
Neglect side wall factor	YES	
Embedment Factor, μ_{emb}	0.89	
Side wall Factor, μ_{wall}	1.00	
Foundation Parameters		
Footing type	Non-circular	
Length of circumscribed rectangle, L_1	6	ft
Width of circumscribed rectangle, B_1	6	ft
Embedment Depth, D_f	3.5	ft
Footing thickness	2	ft
Perimeter of Footing	24	ft
Outputs		
Actual area of wall in contact	48	ft^2
Elastic Settlement, r_e	**0.178**	in

Theoretical Bearing Capacity

Ultimate Net Bearing Capacity (ESA):

$$ESA:\ q_u = \gamma D_f (N_q - 1) s_q d_q r_q w_q + \frac{1}{2}\gamma B N_\gamma s_\gamma d_\gamma r_\gamma w_\gamma$$

Design Parameters:

D_f	2.5	(ft)	t	1.5	(ft)	D_r	80	(%)
B	3	(ft)	L	1	(ft)	B'	3	(ft)
γ	115	(pcf)	ϕ'	33		L'	1	(ft)

Bearing Capacity Factors:

N_q	26.1		N_g	26.6
s_q	1.0		s_g	1.0
d_q	1.2		d_γ	1.0
r_q	1.0		r_g	1.0
w_q	1.0		w_γ	1.0

Structural Load =	5.7	(k/LF)		
Weight of Concrete =	0.675	(k)		
Design Load =	6.38	(k)		
Design Moment =	0	(k -ft)	e =	0.0000

q_u =	13.41	(ksf)
$(\sigma_a)_{min}$ =	2.13	(ksf)
FS =	6.31	

Elastic Settlement		
Quantity		
Soil Properties	**Inputs**	**Units**
Allowable Vertical Load Q_a	6375	lb
Elastic Modulus E_u	1254000	psf
Poisson's ratio, ν_u	0.3	
Foundation Factors		
Shape factor, μ_s	0.65	
Neglect embedment factor	NO	
Neglect side wall factor	YES	
Embedment Factor, μ_{emb}	0.67	
Side wall Factor, μ_{wall}	1.00	
Foundation Parameters		
Footing type	Non-circular	
Length of circumscribed rectangle, L_1	1	ft
Width of circumscribed rectangle, B_1	3	ft
Embedment Depth, D_f	2.5	ft
Footing thickness	1.5	ft
Perimeter of Footing	8	ft
Outputs		
Actual area of wall in contact	12	ft^2
Elastic Settlement, r_e	0.048	in

Differential Settlement Summary (in):

D. Settlement	Column	Settlement	Column	Settlement	D. Settlement	Column	Settlement	Column	Settlement
0.001	A-1	0.206	B-2	0.205	0.023	E-1	0.195	F-1	0.172
0.01	A-1	0.206	A-2	0.196	0.009	E-2	0.179	E-3	0.188
0.009	A-2	0.196	B-2	0.205	0.013	E-2	0.179	F-2	0.192
0.058	B-1	0.147	B-2	0.205	0.02	F-1	0.172	F-2	0.192
0	B-1	0.147	C-1	0.147	0.019	F-2	0.192	F-3	0.173
0.001	B-2	0.205	C-2	0.206	0.015	E-3	0.188	F-3	0.173
0.059	C-1	0.147	C-2	0.206	0.031	E-4	0.208	F-4	0.177
0.001	C-1	0.147	D-1	0.146	0.004	F-3	0.173	F-4	0.177
0.018	C-2	0.206	D-2	0.188	0.053	E-5	0.224	E-6	0.171
0.042	D-1	0.146	D-2	0.188	0.034	E-5	0.224	F-5	0.19
0.049	D-1	0.146	E-1	0.195	0.01	F-5	0.19	F-6	0.18
0.009	D-2	0.188	E-2	0.179	0.009	E-6	0.171	F-6	0.18
0.016	E-1	0.195	E-2	0.179					

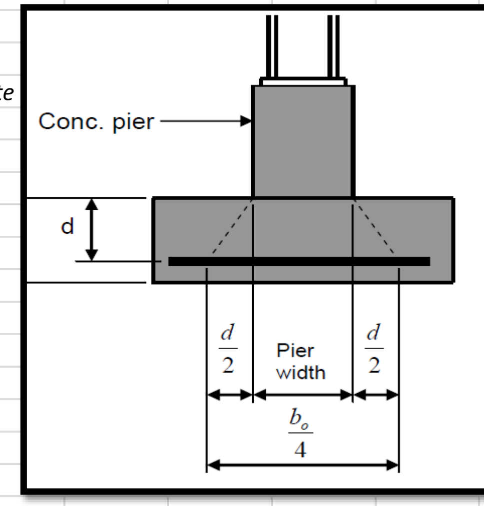
Two-Way Shear (Punching Shear), ACI:

Designed based on #7 rebar & 4000 psi concrete

$$\frac{b_0}{4} = \text{Column Width} + 2\left(\frac{d}{2}\right)$$

$$V_{u2} = q_u(Footing\ Area - \left(\frac{b_0}{4}\right)^2)$$

$$d_{req} = \frac{V_{u2}}{\emptyset 4\sqrt{4000}b_0}$$



Column Width	=	0.5	(ft)
Footing Area	=	49	(ft)^2
d (assumed)	=	1.25	(ft)
Load (LRFD)	=	324	(k)
q_u	=	6.61	(ksf)
b_0	=	9	(ft)
V_{u2}	=	290.43	(k)

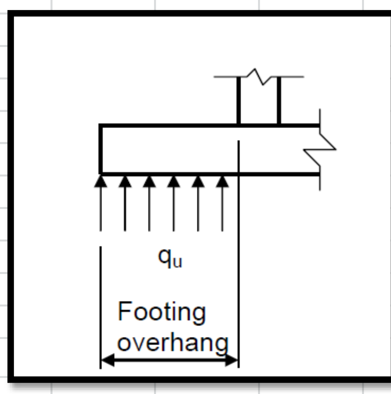
d_{req} = 1.04 (ft) ***footing thickness of 2 feet is ok***

Amount of Tensile Steel:

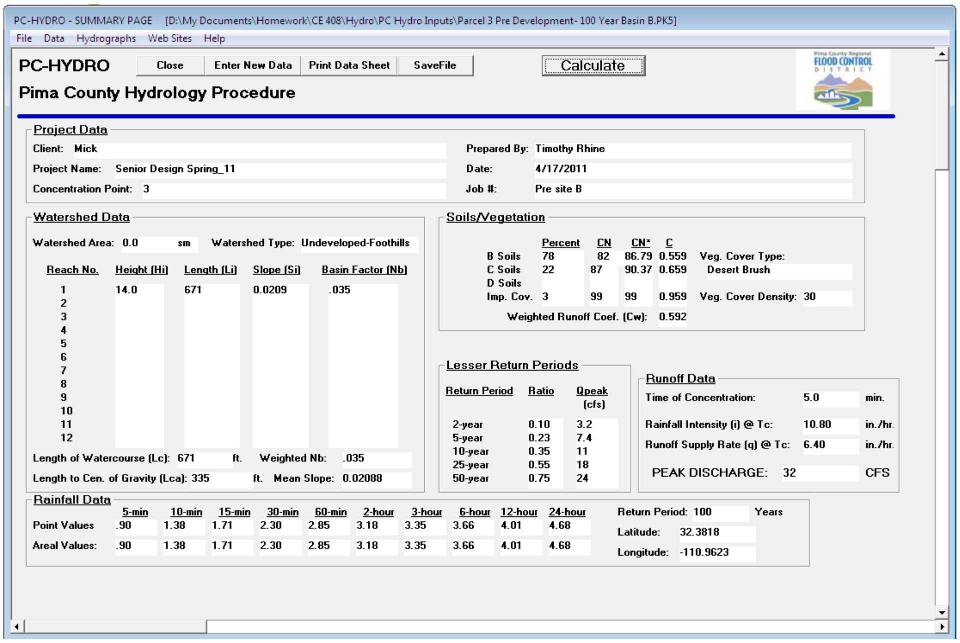
$$M_u = Footing\ Width * q_u * \left(\frac{(footing\ overhang)^2}{2}\right)$$

$$\frac{M_u}{\emptyset b d^2}$$

$$A_s = \rho b d$$



M_u	208.21	(k-ft)
ρ_{min}	0.0033	
A_s	3.08	
# of rebar	8	



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Pima County Hydrology Procedure

Project Data

Client: Mick | Prepared By: Timothy Rhine

Project Name: Senior Design Spring_11 | Date: 4/17/2011

Concentration Point: 3 | Job #: Pre site B

Watershed Data

Watershed Area: 0.0 sm Watershed Type: Undeveloped-Foothills

Reach No.	Height (Hi)	Length (Li)	Slope (Si)	Basin Factor (Nb)
1	14.0	671	0.0209	.035
2				
3				
4				
5				
6				
7				
8				
9				
10				
11				
12				

Length of Watercourse (Lc): 671 ft. Weighted Nb: .035

Length to Cen. of Gravity (Lca): 335 ft. Mean Slope: 0.02088

Soils/Vegetation

	Percent	CN	CN*	C
B Soils	78	82	86.79	0.559
C Soils	22	87	90.37	0.659
D Soils				
Imp. Cov.	3	99	99	0.959

Veg. Cover Type: Desert Brush

Veg. Cover Density: 30

Weighted Runoff Coef. (Cw): 0.592

Lesser Return Periods

Return Period	Ratio	Qpeak (cfs)
2-year	0.10	3.2
5-year	0.23	7.4
10-year	0.35	11
25-year	0.55	18
50-year	0.75	24

Runoff Data

Time of Concentration: 5.0 min.

Rainfall Intensity (i) @ Tc: 10.80 in./hr.

Runoff Supply Rate (q) @ Tc: 6.40 in./hr.

PEAK DISCHARGE: 32 CFS

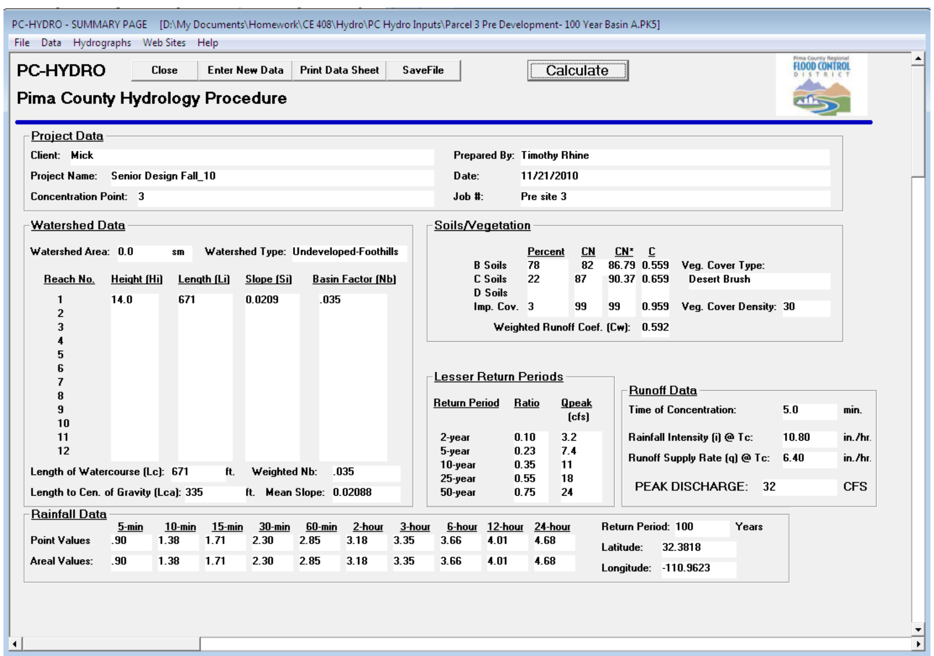
Rainfall Data

	5-min	10-min	15-min	30-min	60-min	2-hour	3-hour	6-hour	12-hour	24-hour
Point Values	.90	1.38	1.71	2.30	2.85	3.18	3.35	3.66	4.01	4.68
Areal Values:	.90	1.38	1.71	2.30	2.85	3.18	3.35	3.66	4.01	4.68

Return Period: 100 Years

Latitude: 32.3818

Longitude: -110.9623



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Pima County Hydrology Procedure

Project Data

Client: Mick | Prepared By: Timothy Rhine

Project Name: Senior Design Fall_10 | Date: 11/21/2010

Concentration Point: 3 | Job #: Pre site 3

Watershed Data

Watershed Area: 0.0 sm Watershed Type: Undeveloped-Foothills

Reach No.	Height (Hi)	Length (Li)	Slope (Si)	Basin Factor (Nb)
1	14.0	671	0.0209	.035
2				
3				
4				
5				
6				
7				
8				
9				
10				
11				
12				

Length of Watercourse (Lc): 671 ft. Weighted Nb: .035

Length to Cen. of Gravity (Lca): 335 ft. Mean Slope: 0.02088

Soils/Vegetation

	Percent	CN	CN*	C
B Soils	78	82	86.79	0.559
C Soils	22	87	90.37	0.659
D Soils				
Imp. Cov.	3	99	99	0.959

Veg. Cover Type: Desert Brush

Veg. Cover Density: 30

Weighted Runoff Coef. (Cw): 0.592

Lesser Return Periods

Return Period	Ratio	Qpeak (cfs)
2-year	0.10	3.2
5-year	0.23	7.4
10-year	0.35	11
25-year	0.55	18
50-year	0.75	24

Runoff Data

Time of Concentration: 5.0 min.

Rainfall Intensity (i) @ Tc: 10.80 in./hr.

Runoff Supply Rate (q) @ Tc: 6.40 in./hr.

PEAK DISCHARGE: 32 CFS

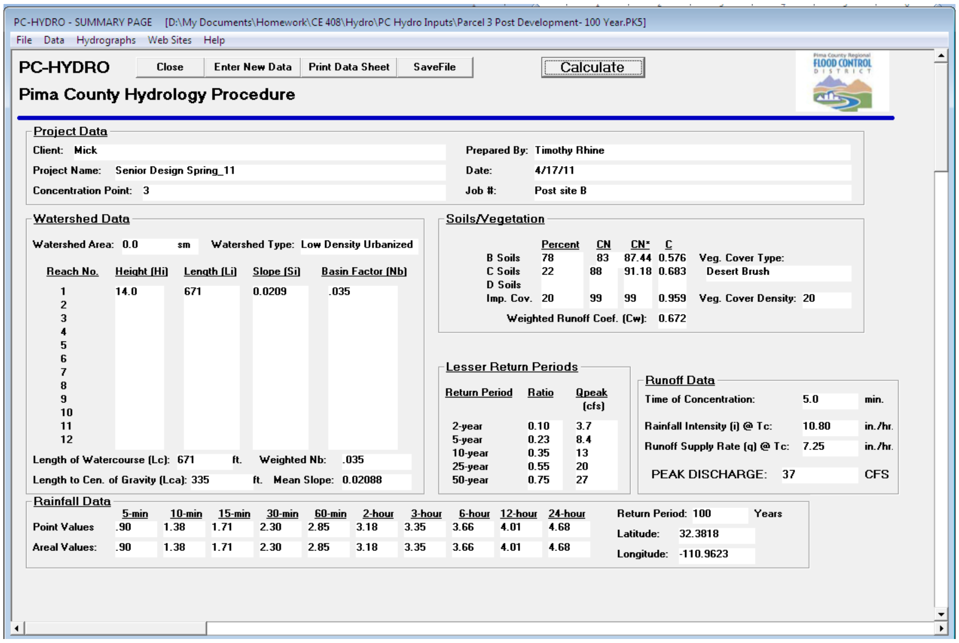
Rainfall Data

	5-min	10-min	15-min	30-min	60-min	2-hour	3-hour	6-hour	12-hour	24-hour
Point Values	.90	1.38	1.71	2.30	2.85	3.18	3.35	3.66	4.01	4.68
Areal Values:	.90	1.38	1.71	2.30	2.85	3.18	3.35	3.66	4.01	4.68

Return Period: 100 Years

Latitude: 32.3818

Longitude: -110.9623



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Pima County Hydrology Procedure

Project Data

Client:	Mick	Prepared By:	Timothy Rhine
Project Name:	Senior Design Spring_11	Date:	4/17/11
Concentration Point:	3	Job #:	Post site B

Watershed Data

Watershed Area: 0.0 sm Watershed Type: Low Density Urbanized

Reach No.	Height (Hi)	Length (Li)	Slope (Si)	Basin Factor (Nb)
1	14.0	671	0.0209	.035
2				
3				
4				
5				
6				
7				
8				
9				
10				
11				
12				

Length of Watercourse (Lc): 671 ft. Weighted Nb: .035

Length to Cen. of Gravity (Lca): 335 ft. Mean Slope: 0.02088

Soils/Vegetation

	Percent	CN	CN*	C
B Soils	78	83	87.44	0.576
C Soils	22	88	91.18	0.683
D Soils				
Imp. Cov.	20	99	99	0.959

Veg. Cover Type: Desert Brush

Veg. Cover Density: 20

Weighted Runoff Coef. (Cw): 0.672

Lesser Return Periods

Return Period	Ratio	Qpeak (cfs)
2-year	0.10	3.7
5-year	0.23	8.4
10-year	0.35	13
25-year	0.55	20
50-year	0.75	27

Runoff Data

Time of Concentration:	5.0	min.
Rainfall Intensity (i) @ Tc:	10.80	in./hr.
Runoff Supply Rate (q) @ Tc:	7.25	in./hr.
PEAK DISCHARGE:	37	CFS

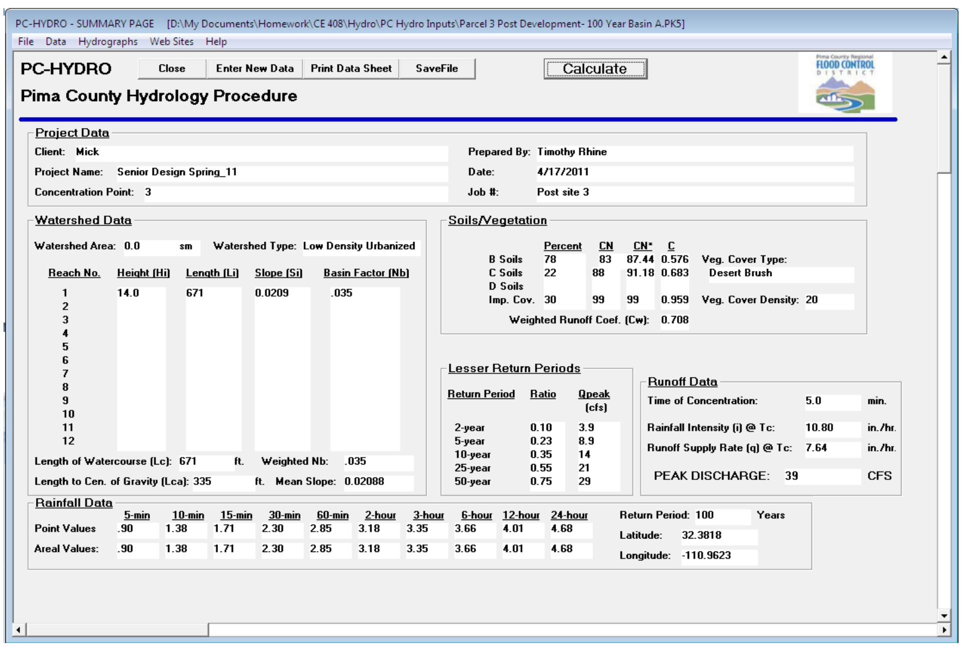
Rainfall Data

	5-min	10-min	15-min	30-min	60-min	2-hour	3-hour	6-hour	12-hour	24-hour
Point Values	.90	1.38	1.71	2.30	2.85	3.18	3.35	3.66	4.01	4.68
Areal Values:	.90	1.38	1.71	2.30	2.85	3.18	3.35	3.66	4.01	4.68

Return Period: 100 Years

Latitude: 32.3818

Longitude: -110.9623



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File Data Hydrographs Web Sites Help

PC-HYDRO | Close | Enter New Data | Print Data Sheet | SaveFile | Calculate

Pima County Hydrology Procedure

Project Data

Client:	Mick	Prepared By:	Timothy Rhine
Project Name:	Senior Design Spring_11	Date:	4/17/2011
Concentration Point:	3	Job #:	Post site 3

Watershed Data

Watershed Area: 0.0 sm Watershed Type: Low Density Urbanized

Reach No.	Height (Hi)	Length (Li)	Slope (Si)	Basin Factor (Nb)
1	14.0	671	0.0209	.035
2				
3				
4				
5				
6				
7				
8				
9				
10				
11				
12				

Length of Watercourse (Lc): 671 ft. Weighted Nb: .035

Length to Cen. of Gravity (Lca): 335 ft. Mean Slope: 0.02088

Soils/Vegetation

	Percent	CN	CN*	C
B Soils	78	83	87.44	0.576
C Soils	22	88	91.18	0.683
D Soils				
Imp. Cov.	30	99	99	0.959

Veg. Cover Type: Desert Brush

Veg. Cover Density: 20

Weighted Runoff Coef. (Cw): 0.708

Lesser Return Periods

Return Period	Ratio	Qpeak (cfs)
2-year	0.10	3.9
5-year	0.23	8.9
10-year	0.35	14
25-year	0.55	21
50-year	0.75	29

Runoff Data

Time of Concentration:	5.0	min.
Rainfall Intensity (i) @ Tc:	10.80	in./hr.
Runoff Supply Rate (q) @ Tc:	7.64	in./hr.
PEAK DISCHARGE:	39	CFS

Rainfall Data

	5-min	10-min	15-min	30-min	60-min	2-hour	3-hour	6-hour	12-hour	24-hour
Point Values	.90	1.38	1.71	2.30	2.85	3.18	3.35	3.66	4.01	4.68
Areal Values:	.90	1.38	1.71	2.30	2.85	3.18	3.35	3.66	4.01	4.68

Return Period: 100 Years

Latitude: 32.3818

Longitude: -110.9623

Basin A

Pre Development			Post Development						
Pima County Hydrology Procedures			Pima County Hydrology Procedures						
Synthetic Hydrograph			Synthetic Hydrograph						
Concentration Point:		3	Concentration Point:		3				
Return Period:		100 Years	Return Period:		100 Years				
Peak Discharge:		32.3 cfs	Peak Discharge:		38.6 cfs				
Time of Concentration:		5.0 min	Time of Concentration:		5.0 min				
Rise Time:		13.6 min	Rise Time:		13.6 min				
Runoff Volume:	0.7045 af,	30,692.8 cf	Runoff Volume:	0.8415 af,	36662.1 cf	Differences			
Time	Discharge	Runoff Volume	Time	Discharge	Runoff Volume	Time	Discharge	Storage	Storage
(min)	(cfs)	(af)	(min)	(cfs)	(af)	(min)	(cfs)	(af)	(cf)
0	0	0	0	0	0	0	0	0	0
1	0.6	0.001	1	0.7	0.0012	1	0.1	0.0002	8.712
2	1.7	0.0031	2	2.1	0.0037	2	0.4	0.0006	26.136
3	3.3	0.0068	3	3.9	0.0081	3	0.6	0.0013	56.628
4	5	0.0136	4	6	0.0162	4	1	0.0026	113.256
5	7	0.0217	5	8.3	0.0259	5	1.3	0.0042	182.952
6	9.2	0.0332	6	11	0.0396	6	1.8	0.0064	278.784
7	11.7	0.0478	7	13.9	0.0571	7	2.2	0.0093	405.108
8	14.2	0.0649	8	16.9	0.0775	8	2.7	0.0126	548.856
9	17.1	0.085	9	20.4	0.1016	9	3.3	0.0166	723.096
10	20.5	0.1067	10	24.4	0.1275	10	3.9	0.0208	906.048
11	24.3	0.1313	11	29	0.1569	11	4.7	0.0256	1115.136
12	27.8	0.1692	12	33.2	0.2021	12	5.4	0.0329	1433.124
13	30.7	0.2066	13	36.7	0.2467	13	6	0.0401	1746.756
14	31.6	0.2445	14	37.7	0.292	14	6.1	0.0475	2069.1
15	29.8	0.2831	15	35.6	0.3382	15	5.8	0.0551	2400.156
16	27.7	0.3163	16	33.1	0.3778	16	5.4	0.0615	2678.94
17	25.8	0.3477	17	30.8	0.4153	17	5	0.0676	2944.656
18	23.8	0.3776	18	28.5	0.451	18	4.7	0.0734	3197.304
19	22	0.406	19	26.2	0.485	19	4.2	0.079	3441.24
20	20.2	0.4315	20	24.1	0.5155	20	3.9	0.084	3659.04
21	18.7	0.4554	21	22.3	0.5439	21	3.6	0.0885	3855.06
22	17.2	0.4772	22	20.6	0.57	22	3.4	0.0928	4042.368
23	15.7	0.4958	23	18.8	0.5922	23	3.1	0.0964	4199.184
24	14.4	0.514	24	17.1	0.614	24	2.7	0.1	4356
25	13.1	0.5311	25	15.6	0.6343	25	2.5	0.1032	4495.392
26	11.8	0.5467	26	14.1	0.653	26	2.3	0.1063	4630.428

27	10.7	0.5602	27	12.7	0.6691	27	2	0.1089	4743.684
28	9.7	0.5716	28	11.5	0.6827	28	1.8	0.1111	4839.516
29	8.7	0.5824	29	10.4	0.6957	29	1.7	0.1133	4935.348
30	7.7	0.5932	30	9.2	0.7085	30	1.5	0.1153	5022.468
31	7	0.602	31	8.3	0.719	31	1.3	0.117	5096.52
32	6.3	0.6108	32	7.5	0.7296	32	1.2	0.1188	5174.928
33	5.6	0.6187	33	6.7	0.7391	33	1.1	0.1204	5244.624
34	5.1	0.6252	34	6.1	0.7468	34	1	0.1216	5296.896
35	4.3	0.6317	35	5.2	0.7545	35	0.9	0.1228	5349.168
36	4	0.6368	36	4.7	0.7607	36	0.7	0.1239	5397.084
37	3.7	0.6412	37	4.5	0.766	37	0.8	0.1248	5436.288
38	3.3	0.6456	38	4	0.7712	38	0.7	0.1256	5471.136
39	3	0.6496	39	3.6	0.7759	39	0.6	0.1263	5501.628
40	2.7	0.6535	40	3.3	0.7805	40	0.6	0.127	5532.12
41	2.5	0.6571	41	3	0.7849	41	0.5	0.1278	5566.968
42	2.3	0.6599	42	2.8	0.7882	42	0.5	0.1283	5588.748
43	2.1	0.6626	43	2.6	0.7914	43	0.5	0.1288	5610.528
44	2	0.6653	44	2.4	0.7947	44	0.4	0.1294	5636.664
45	1.8	0.668	45	2.1	0.7979	45	0.3	0.1299	5658.444
46	1.6	0.6707	46	1.9	0.8012	46	0.3	0.1305	5684.58
47	1.5	0.6725	47	1.8	0.8033	47	0.3	0.1308	5697.648
48	1.4	0.6741	48	1.7	0.8052	48	0.3	0.1311	5710.716
49	1.3	0.6756	49	1.5	0.807	49	0.2	0.1314	5723.784
50	1.2	0.6772	50	1.4	0.8089	50	0.2	0.1317	5736.852
51	1	0.6788	51	1.3	0.8108	51	0.3	0.132	5749.92
52	1	0.6801	52	1.1	0.8124	52	0.1	0.1323	5762.988
53	0.9	0.6812	53	1.1	0.8137	53	0.2	0.1325	5771.7
54	0.8	0.6822	54	1	0.8149	54	0.2	0.1327	5780.412
55	0.8	0.6832	55	0.9	0.8161	55	0.1	0.1329	5789.124
56	0.7	0.6843	56	0.9	0.8174	56	0.2	0.1331	5797.836
57	0.7	0.6853	57	0.8	0.8186	57	0.1	0.1333	5806.548
58	0.6	0.6861	58	0.7	0.8196	58	0.1	0.1335	5815.26
59	0.6	0.6869	59	0.7	0.8205	59	0.1	0.1336	5819.616
60	0.5	0.6877	60	0.6	0.8214	60	0.1	0.1337	5823.972
61	0.5	0.6885	61	0.6	0.8224	61	0.1	0.1339	5832.684
62	0.4	0.6892	62	0.5	0.8233	62	0.1	0.1341	5841.396
63	0.4	0.6899	63	0.5	0.8241	63	0.1	0.1342	5845.752
64	0.4	0.6904	64	0.4	0.8247	64	0	0.1343	5850.108
65	0.3	0.6909	65	0.4	0.8253	65	0.1	0.1344	5854.464
66	0.3	0.6915	66	0.4	0.8259	66	0.1	0.1344	5854.464
67	0.3	0.692	67	0.3	0.8265	67	0	0.1345	5858.82

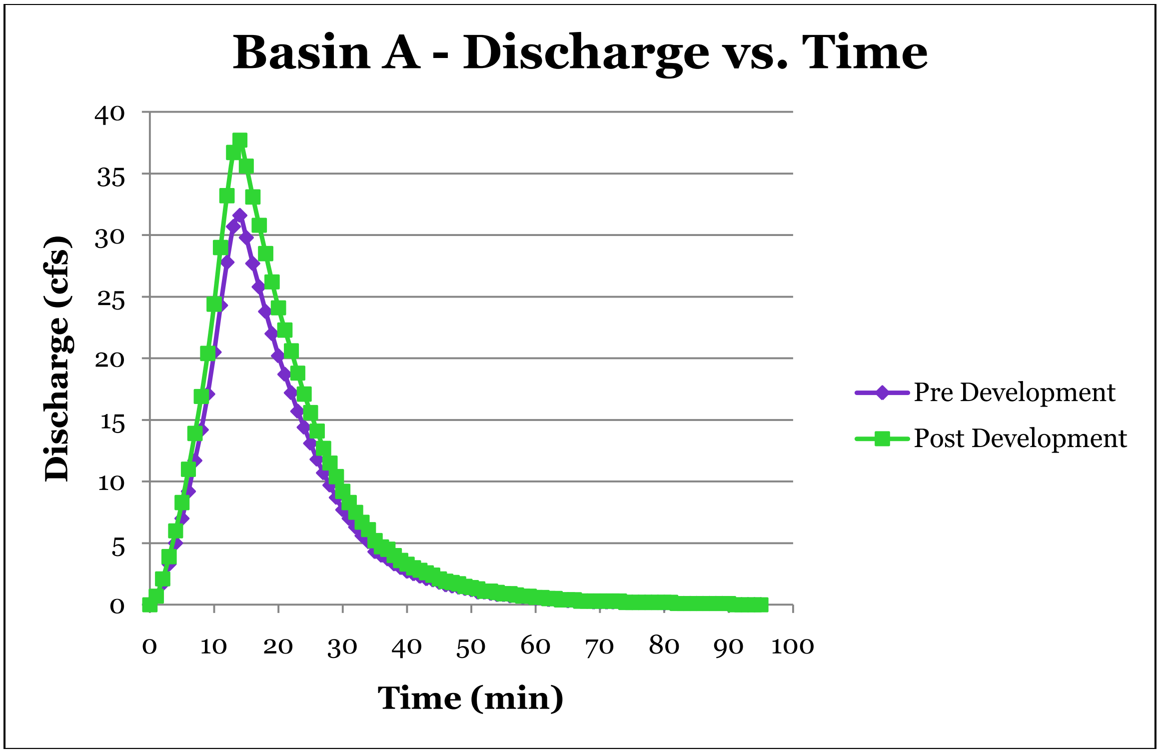
68	0.3	0.6925	68	0.3	0.8272	68	0	0.1347	5867.532
69	0.2	0.693	69	0.3	0.8277	69	0.1	0.1347	5867.532
70	0.2	0.6934	70	0.3	0.8282	70	0.1	0.1348	5871.888
71	0.2	0.6938	71	0.3	0.8288	71	0.1	0.135	5880.6
72	0.2	0.6943	72	0.3	0.8293	72	0.1	0.135	5880.6
73	0.2	0.6947	73	0.3	0.8298	73	0.1	0.1351	5884.956
74	0.2	0.6951	74	0.2	0.8303	74	0	0.1352	5889.312
75	0.2	0.6956	75	0.2	0.8309	75	0	0.1353	5893.668
76	0.2	0.696	76	0.2	0.8314	76	0	0.1354	5898.024
77	0.2	0.6965	77	0.2	0.8319	77	0	0.1354	5898.024
78	0.2	0.6969	78	0.2	0.8325	78	0	0.1356	5906.736
79	0.2	0.6973	79	0.2	0.833	79	0	0.1357	5911.092
80	0.1	0.6978	80	0.2	0.8335	80	0.1	0.1357	5911.092
81	0.1	0.6982	81	0.2	0.834	81	0.1	0.1358	5915.448
82	0.1	0.6987	82	0.1	0.8345	82	0	0.1358	5915.448
83	0.1	0.6991	83	0.1	0.8351	83	0	0.136	5924.16
84	0.1	0.6995	84	0.1	0.8356	84	0	0.1361	5928.516
85	0.1	0.7	85	0.1	0.8361	85	0	0.1361	5928.516
86	0.1	0.7004	86	0.1	0.8367	86	0	0.1363	5937.228
87	0.1	0.7009	87	0.1	0.8372	87	0	0.1363	5937.228
88	0.1	0.7013	88	0.1	0.8377	88	0	0.1364	5941.584
89	0.1	0.7018	89	0.1	0.8382	89	0	0.1364	5941.584
90	0	0.7022	90	0.1	0.8387	90	0.1	0.1365	5945.94
91	0	0.7026	91	0	0.8393	91	0	0.1367	5954.652
92	0	0.7031	92	0	0.8398	92	0	0.1367	5954.652
93	0	0.7035	93	0	0.8403	93	0	0.1368	5959.008
94	0	0.704	94	0	0.8409	94	0	0.1369	5963.364
95	0	0.7044	95	0	0.8414	95	0	0.137	5967.72
								Total Storage Volume =	**5968 cf**
					Total Design Volmume			**1.15x5968=**	**6862.88**

Basin B

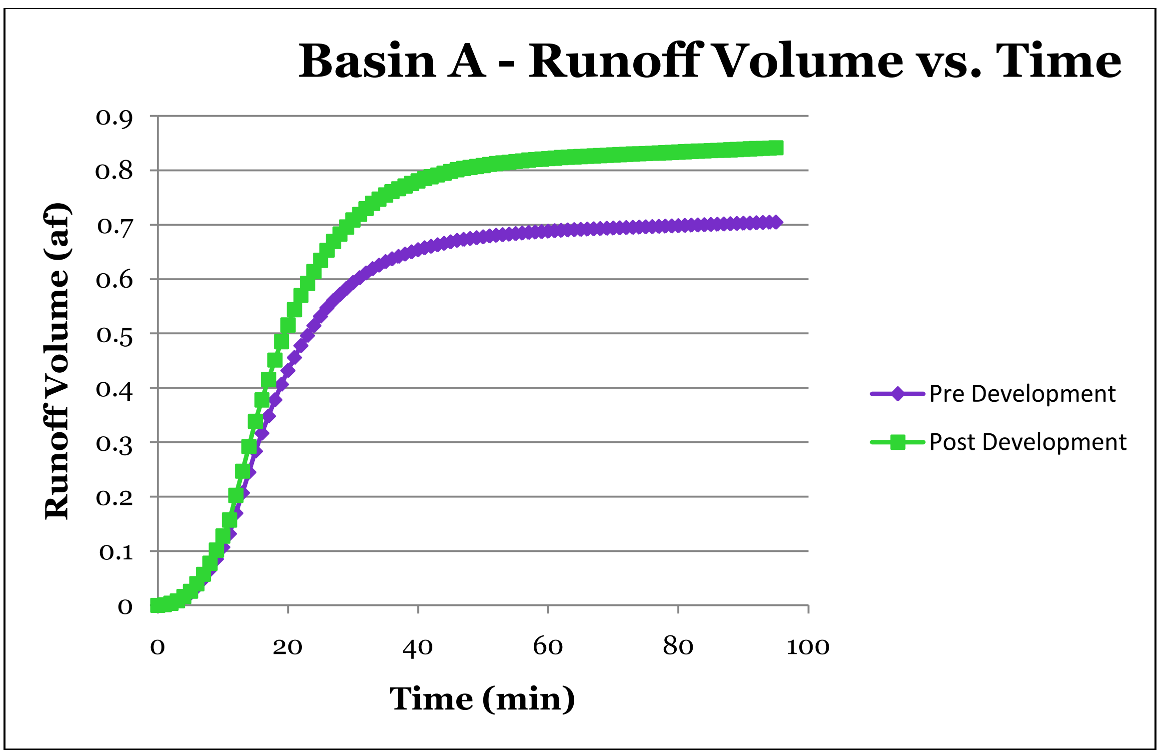
Pre Development			Post Development						
Pima County Hydrology Procedures			Pima County Hydrology Procedures						
Synthetic Hydrograph			Synthetic Hydrograph						
Concentration Point:		3	Concentration Point:		3				
Return Period:		100 Years	Return Period:		100 Years				
Peak Discharge:		32.3 cfs	Peak Discharge:		36.6 cfs				
Time of Concentration:		5.0 min	Time of Concentration:		5.0 min				
Rise Time:		13.6 min	Rise Time:		13.6 min				
Runoff Volume:	0.7045 af,	30,692.8 cf	Runoff Volume:	0.7988 af,	34803.3 cf	Differences			
Time	Discharge	Runoff Volume	Time	Discharge	Runoff Volume	Time	Discharge	Storage	Storage
(min)	(cfs)	(af)	(min)	(cfs)	(af)	(min)	(cfs)	(af)	(cf)
0	0	0	0	0	0	0	0	0	0
1	0.6	0.001	1	0.7	0.0012	1	0.1	0.0002	8.712
2	1.7	0.0031	2	2	0.0035	2	0.3	0.0004	17.424
3	3.3	0.0068	3	3.7	0.0077	3	0.4	0.0009	39.204
4	5	0.0136	4	5.7	0.0154	4	0.7	0.0018	78.408
5	7	0.0217	5	7.9	0.0246	5	0.9	0.0029	126.324
6	9.2	0.0332	6	10.5	0.0376	6	1.3	0.0044	191.664
7	11.7	0.0478	7	13.2	0.0542	7	1.5	0.0064	278.784
8	14.2	0.0649	8	16.1	0.0736	8	1.9	0.0087	378.972
9	17.1	0.085	9	19.3	0.0964	9	2.2	0.0114	496.584
10	20.5	0.1067	10	23.2	0.121	10	2.7	0.0143	622.908
11	24.3	0.1313	11	27.5	0.1489	11	3.2	0.0176	766.656
12	27.8	0.1692	12	31.5	0.1918	12	3.7	0.0226	984.456
13	30.7	0.2066	13	34.8	0.2342	13	4.1	0.0276	1202.256
14	31.6	0.2445	14	35.8	0.2772	14	4.2	0.0327	1424.412
15	29.8	0.2831	15	33.8	0.321	15	4	0.0379	1650.924
16	27.7	0.3163	16	31.5	0.3586	16	3.8	0.0423	1842.588
17	25.8	0.3477	17	29.2	0.3942	17	3.4	0.0465	2025.54
18	23.8	0.3776	18	27	0.4281	18	3.2	0.0505	2199.78
19	22	0.406	19	24.9	0.4604	19	2.9	0.0544	2369.664
20	20.2	0.4315	20	22.9	0.4893	20	2.7	0.0578	2517.768
21	18.7	0.4554	21	21.2	0.5164	21	2.5	0.061	2657.16
22	17.2	0.4772	22	19.6	0.5411	22	2.4	0.0639	2783.484
23	15.7	0.4958	23	17.9	0.5622	23	2.2	0.0664	2892.384
24	14.4	0.514	24	16.3	0.5828	24	1.9	0.0688	2996.928
25	13.1	0.5311	25	14.8	0.6022	25	1.7	0.0711	3097.116
26	11.8	0.5467	26	13.4	0.6199	26	1.6	0.0732	3188.592

27	10.7	0.5602	27	12.1	0.6352	27	1.4	0.075	3267
28	9.7	0.5716	28	10.9	0.6481	28	1.2	0.0765	3332.34
29	8.7	0.5824	29	9.8	0.6605	29	1.1	0.0781	3402.036
30	7.7	0.5932	30	8.8	0.6726	30	1.1	0.0794	3458.664
31	7	0.602	31	7.9	0.6826	31	0.9	0.0806	3510.936
32	6.3	0.6108	32	7.1	0.6926	32	0.8	0.0818	3563.208
33	5.6	0.6187	33	6.3	0.7016	33	0.7	0.0829	3611.124
34	5.1	0.6252	34	5.8	0.7089	34	0.7	0.0837	3645.972
35	4.3	0.6317	35	4.9	0.7163	35	0.6	0.0846	3685.176
36	4	0.6368	36	4.5	0.7221	36	0.5	0.0853	3715.668
37	3.7	0.6412	37	4.2	0.7271	37	0.5	0.0859	3741.804
38	3.3	0.6456	38	3.8	0.7321	38	0.5	0.0865	3767.94
39	3	0.6496	39	3.4	0.7366	39	0.4	0.087	3789.72
40	2.7	0.6535	40	3.1	0.741	40	0.4	0.0875	3811.5
41	2.5	0.6571	41	2.8	0.7451	41	0.3	0.088	3833.28
42	2.3	0.6599	42	2.6	0.7482	42	0.3	0.0883	3846.348
43	2.1	0.6626	43	2.4	0.7513	43	0.3	0.0887	3863.772
44	2	0.6653	44	2.2	0.7544	44	0.2	0.0891	3881.196
45	1.8	0.668	45	2	0.7575	45	0.2	0.0895	3898.62
46	1.6	0.6707	46	1.8	0.7605	46	0.2	0.0898	3911.688
47	1.5	0.6725	47	1.7	0.7626	47	0.2	0.0901	3924.756
48	1.4	0.6741	48	1.6	0.7644	48	0.2	0.0903	3933.468
49	1.3	0.6756	49	1.4	0.7661	49	0.1	0.0905	3942.18
50	1.2	0.6772	50	1.3	0.7679	50	0.1	0.0907	3950.892
51	1	0.6788	51	1.2	0.7697	51	0.2	0.0909	3959.604
52	1	0.6801	52	1.1	0.7712	52	0.1	0.0911	3968.316
53	0.9	0.6812	53	1	0.7724	53	0.1	0.0912	3972.672
54	0.8	0.6822	54	0.9	0.7736	54	0.1	0.0914	3981.384
55	0.8	0.6832	55	0.9	0.7748	55	0.1	0.0916	3990.096
56	0.7	0.6843	56	0.8	0.7759	56	0.1	0.0916	3990.096
57	0.7	0.6853	57	0.7	0.7771	57	0	0.0918	3998.808
58	0.6	0.6861	58	0.7	0.778	58	0.1	0.0919	4003.164
59	0.6	0.6869	59	0.6	0.7789	59	0	0.092	4007.52
60	0.5	0.6877	60	0.6	0.7798	60	0.1	0.0921	4011.876
61	0.5	0.6885	61	0.5	0.7807	61	0	0.0922	4016.232
62	0.4	0.6892	62	0.5	0.7815	62	0.1	0.0923	4020.588
63	0.4	0.6899	63	0.4	0.7823	63	0	0.0924	4024.944
64	0.4	0.6904	64	0.4	0.7829	64	0	0.0925	4029.3
65	0.3	0.6909	65	0.4	0.7835	65	0.1	0.0926	4033.656
66	0.3	0.6915	66	0.3	0.7841	66	0	0.0926	4033.656
67	0.3	0.692	67	0.3	0.7846	67	0	0.0926	4033.656

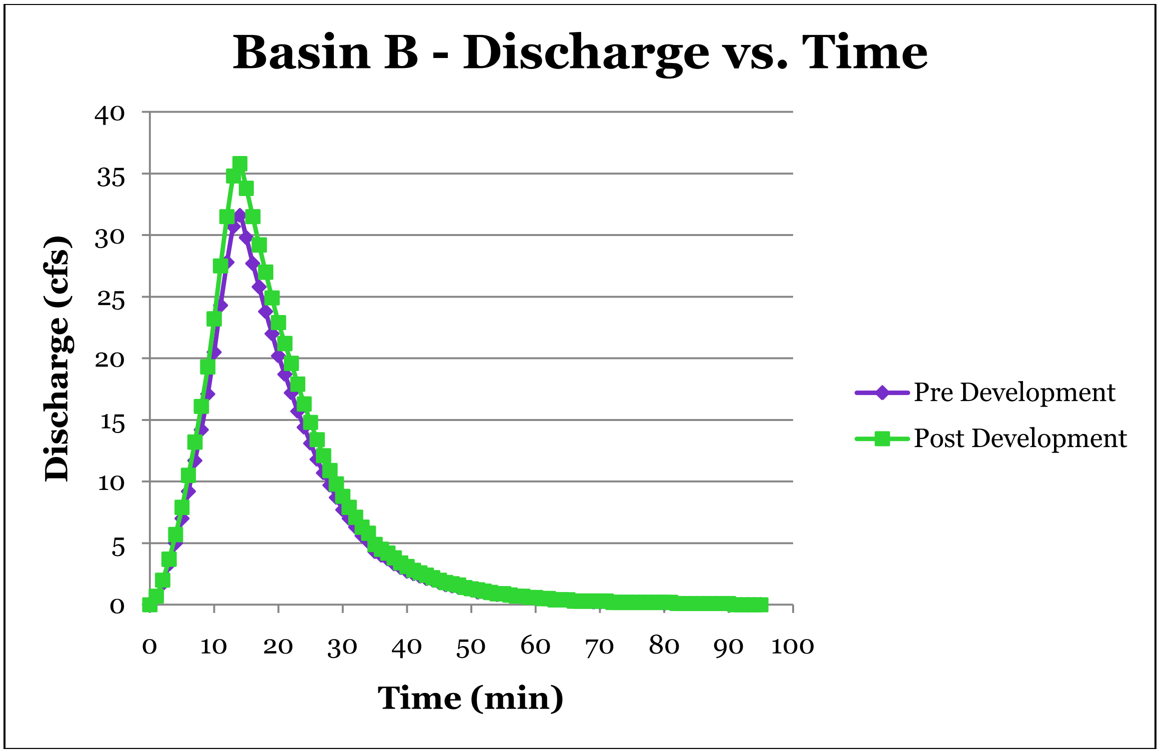
68	0.3	0.6925	68	0.3	0.7852	68	0	0.0927	4038.012
69	0.2	0.693	69	0.3	0.7858	69	0.1	0.0928	4042.368
70	0.2	0.6934	70	0.3	0.7862	70	0.1	0.0928	4042.368
71	0.2	0.6938	71	0.3	0.7867	71	0.1	0.0929	4046.724
72	0.2	0.6943	72	0.2	0.7873	72	0	0.093	4051.08
73	0.2	0.6947	73	0.2	0.7877	73	0	0.093	4051.08
74	0.2	0.6951	74	0.2	0.7882	74	0	0.0931	4055.436
75	0.2	0.6956	75	0.2	0.7888	75	0	0.0932	4059.792
76	0.2	0.696	76	0.2	0.7892	76	0	0.0932	4059.792
77	0.2	0.6965	77	0.2	0.7897	77	0	0.0932	4059.792
78	0.2	0.6969	78	0.2	0.7902	78	0	0.0933	4064.148
79	0.2	0.6973	79	0.2	0.7907	79	0	0.0934	4068.504
80	0.1	0.6978	80	0.2	0.7913	80	0.1	0.0935	4072.86
81	0.1	0.6982	81	0.2	0.7917	81	0.1	0.0935	4072.86
82	0.1	0.6987	82	0.1	0.7922	82	0	0.0935	4072.86
83	0.1	0.6991	83	0.1	0.7928	83	0	0.0937	4081.572
84	0.1	0.6995	84	0.1	0.7932	84	0	0.0937	4081.572
85	0.1	0.7	85	0.1	0.7937	85	0	0.0937	4081.572
86	0.1	0.7004	86	0.1	0.7942	86	0	0.0938	4085.928
87	0.1	0.7009	87	0.1	0.7947	87	0	0.0938	4085.928
88	0.1	0.7013	88	0.1	0.7952	88	0	0.0939	4090.284
89	0.1	0.7018	89	0.1	0.7957	89	0	0.0939	4090.284
90	0	0.7022	90	0.1	0.7962	90	0.1	0.094	4094.64
91	0	0.7026	91	0	0.7967	91	0	0.0941	4098.996
92	0	0.7031	92	0	0.7972	92	0	0.0941	4098.996
93	0	0.7035	93	0	0.7977	93	0	0.0942	4103.352
94	0	0.704	94	0	0.7982	94	0	0.0942	4103.352
95	0	0.7044	95	0	0.7987	95	0	0.0943	4107.708
					Total Storage Volume =				**4108**
					Total Design Volmume			**1.15x4108=**	**4723.86**

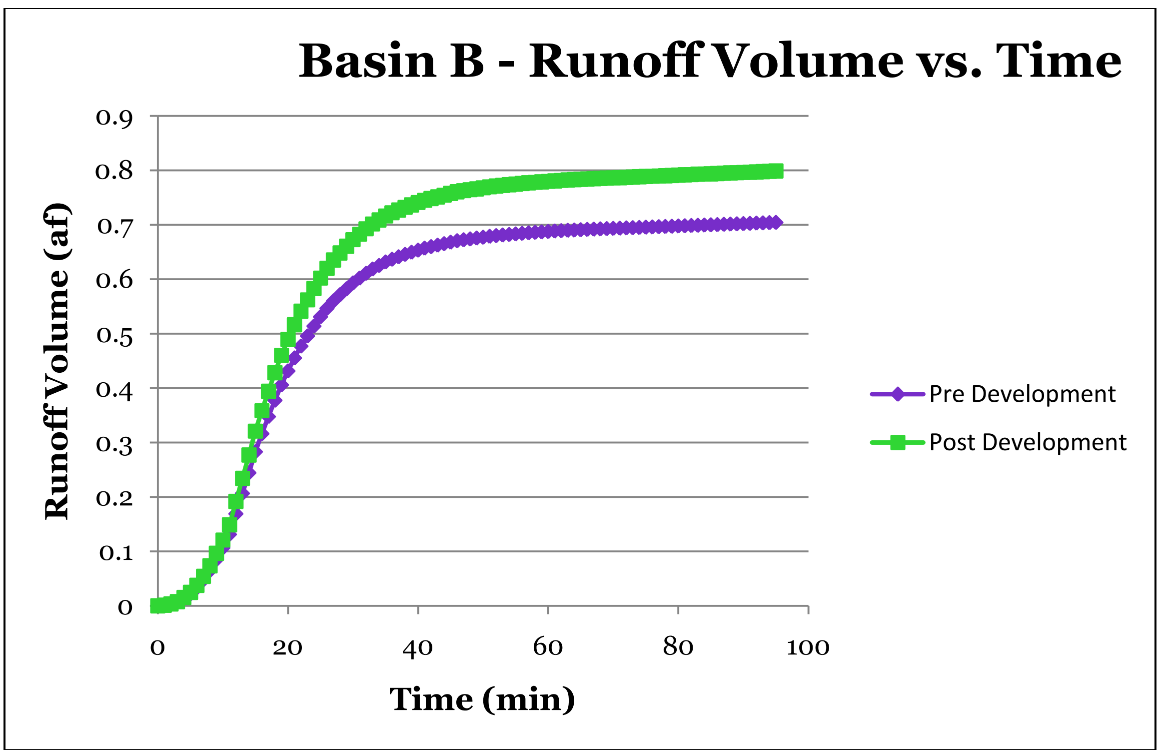


Basin A - Discharge vs. Time
Discharge (cfs)
40
35
30
25
20
15
10
5
0
0
10
20
30
40
50
60
70
80
90
100
Time (min)
Pre Development
Post Development



Basin A - Runoff Volume vs. Time
Runoff Volume (af)
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0
20
40
60
80
100
Time (min)
Pre Development
Post Development


Basin B - Discharge vs. Time
Discharge (cfs)
40
35
30
25
20
15
10
5
0
0 10 20 30 40 50 60 70 80 90 100
Time (min)
Pre Development
Post Development


Basin B - Runoff Volume vs. Time
Runoff Volume (af)
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0 20 40 60 80 100
Time (min)
Pre Development
Post Development

Transportation Appendix

The Oro Valley Zoning Code has specific requirements depending on the building type and size. As stated in the Oro Valley Zoning Code in Table 27-14, a general office building has to provide 3 parking spaces for every 1000 square feet of floor space.

TABLE 27-14: ALLOWED PARKING SPACES

Use	Required Parking Sq. Ft.
Restaurants	
a. Convenience Use	15/1000
b. Standard	10/1000
Bars, Taverns and Nightclubs	10/1000
Community Parks	5/acre
Commercial Recreational	
a. Limited Indoor Recreation	6/1000
b. Outdoor	.3/person
c. Bowling Alley	5/1000
Theaters	1/3 seats
General Retail	4/1000
Service Shop	4/1000
Shopping Center	5/1000
Medical Office	4.5/1000
Financial Services	3.5/1000
Grocery Store, Supermarket	6/1000
General Office	3/1000
Vehicle Servicing and Maintenance	5/1000
Repair Service	2/1000
Lodging Establishments	1/unit
Employees	1 per 2
Health Facilities	
a. Hospitals	1/bed
b. Long Term Care Facilities	.33/bed, 1 per employee based on maximum shift
Schools, Places of Worship or Assembly	1 per 4 seats and 2 per 3 employees
Child Care Centers	1/1000 and 2 per 3 employees
Industrial Employee Parking	.75/employee

As for taking the size of the building, the general office building has a size of 31000 square foot. Therefore, (3/1000)*31000=93 parking spaces (including disabled parking).

TABLE 27-15: NUMBER OF MOBILITY IMPAIRED PARKING SPACES

Total Parking Spaces in Lot	Minimum Required Number of Accessible Spaces
1-25	1
26-50	2
51-75	3
76-100	4
101-150	5
151-200	6
201-300	7
301-400	8
401-500	9
501-1000	2% of total space
Over 1000	20 spaces plus 1 space for every 100 spaces or fraction thereof over 1000

ITE Trip Generation Manual and Excel File

31000	**Avg. Vehicle Trip Ends VS 1000 GSF/Day**
General Office Building	341.31

Table 1: Trip Generation (Avg. Vehicle Trip Ends)

			Input	**Range of Rates**	
710	**General Office Building**	1000 Sq.Ft. Gross Floor Area	31000	110.98	892.8

Table 2: Trip Generation (Range of Rates)

Avg. Vehicle Trip Ends vs:	**Input**	**A.M. Peak Hr. Avg. Rate**	**A.M. Peak Hr. Range of Rates**	
1000 Sq.Ft. Gross Floor Area	**31000**	48.05	18.6	185.38

Table 3: Trip Generation (A.M. Peak Hour Avg. and Range)

		P.M. Peak Hr. Avg. Rate	**P.M. Peak Hr. Range of Rates**	
1000 Sq.Ft. Gross Floor Area	**31000**	46.19	15.19	198.09

Table 4: Trip Generation (P.M. Peak Hour Avg. and Range)

Cost Analysis Calculations:

ABC (Aggregate Base Concrete):

100,776 ft^2*(1/3)ft = 33,592 ft^3

33,592 ft^3*$(1yd/3ft)^3$ = 1,244.15yd^3

1,244.15yd^3*$40 = <u>$49,765.90</u>

AC (Asphalt Concrete):

1553.63TN*$100/TN = <u>$155,363</u>

Structural Appendix

Live Loads

Column		Total	Corridor	Corridor (2nd Fl)	Deck	Lobby	Cubicles	Office
		Live Load	100	80	100	100	50	50
		(kips)						
A1	1st	**44.3**	197.375	0	0	98.8125	159.375	128.6875
Total (Kip)	2nd	**54.5**	0	141	56.25	348.5	51.25	0
98.9	R	**6.8**	0	0	0	0	0	0
A2	1st	**42.4**	209.1875	0	0	60.0625	98.8125	201.8125
Total (Kip)	2nd	**55.9**	0	40.625	468.75	0	0	115.625
98.3	R	**1.9**	0	0	0	0	0	0
B1	1st	**37.0**	148.4375	0	0	0	312.375	125.0625
Total (Kip)	2nd	**32.6**	0	85.375	0	0	511.875	0
69.7	R	**7.5**	0	0	0	0	0	0
B2	1st	**40.6**	195.1875	0	0	0	189.875	178.9375
Total	2nd	**49.6**	0	117.5	312.5	0	0	175.75
90.2	R	**3.8**	0	0	0	0	0	0
C1	1st	**36.7**	135.1875	0	0	0	312.375	146.8125
Total (Kip)	2nd	**32.6**	0	87.25	0	0	508.25	0
69.4	R	**7.5**	0	0	0	0	0	0
C2	1st	**47.7**	103.985	0	0	150.0625	189.14	27.5625
Total (Kip)	2nd	**49.1**	0	107.75	312.5	0	0	180.375
96.8	R	**3.8**	0	0	0	0	0	0
D1	1st	**36.1**	131	0	0	0	191.25	261.375
Total (Kip)	2nd	**32.5**	0	85.75	0	0	508.25	0
68.6	R	**7.5**	0	0	0	0	0	0
D2	1st	**53.1**	449.5313	0	0	0	159.8438	0
Total (Kip)	2nd	**43.3**	0	44	155	172.145	0	138.375
96.3	R	**8.3**	0	0	0	0	0	0
E1	1st	**44.0**	227.2188	0	0	105.0313	0	84.625
Total (Kip)	2nd	**45.3**	0	183.965	0	0	36.285	0
89.3	R	**7.5**	0	0	0	0	0	0
E2	1st	**51.6**	456.4219	0	0	0	0	0
Total (Kip)	2nd	**28.2**	0	20	0	264	0	0
79.8	R	**7.5**	0	0	0	0	0	0
E3	1st (D2)	**53.1**	449.5313	0	0	0	159.8438	0
Total (Kip)	2nd	**43.3**	0	44	155	172.145	0	138.375

96.3	R	**8.3**	0	0	0	0	0	0
E4	1st (C2)	**47.7**	103.985	0	0	150.0625	189.14	27.5625
Total (Kip)	2nd	**49.1**	0	107.75	312.5	0	0	180.375
96.8	R	**3.8**	0	0	0	0	0	0
E5	1st (B2)	**40.6**	195.1875	0	0	0	189.875	178.9375
Total (Kip)	2nd	**54.5**	0	141	56.25	348.5	51.25	0
95.1	R	**6.8**	0	0	0	0	0	0
E6	1st (A2)	**42.4**	209.1875	0	0	60.0625	98.8125	201.8125
Total (Kip)	2nd	**62.5**	0	0	625	0	0	0
104.9	R	**0.0**	0	0	0	0	0	0
F1	1st	**34.8**	184.3438	0	0	0	0	0
Total (Kip)	2nd	**32.2**	0	172.15	0	0	0	0
67.0	R	**7.5**	0	0	0	0	0	0
F2	1st (E1)	**43.9**	227.2188	0	0	105.0313	0	84.625
Total (Kip)	2nd	**43.2**	0	183.75	0	0	36.25	0
87.2	R	**7.5**	0	0	0	0	0	0
F3	1st (D1)	**36.1**	131	0	0	0	191.25	261.375
Total (Kip)	2nd	**32.5**	0	85.75	0	0	508.25	0
68.6	R	**7.5**	0	0	0	0	0	0
F4	1st (C1)	**36.7**	135.1875	0	0	0	312.375	146.8125
Total (Kip)	2nd	**36.5**	0	29.5	87.25	0	0	508.25
73.2	R	**6.5**	0	0	0	0	0	0
F5	1st (B1)	**37.0**	148.4375	0	0	0	312.375	125.0625
Total (Kip)	2nd	**42.0**	0	28	141	56.25	348.5	51.25
79.0	R	**5.8**	0	0	0	0	0	0
F6	1st (A1)	**44.3**	197.375	0	0	98.8125	159.375	128.6875
Total (Kip)	2nd	**62.5**	0	0	573.75	51.25	0	0
106.8	R	**0.6**	0	0	0	0	0	0
		KLF	Ground	1st Floor	Roof			
CMU	1st	**1.200**	100	100	12			
KLF	2nd	**1.200**						
2.400	R	**0.144**						

Dead Loads

Column	Area		Stud Wall	Curt. Wall	Par. Stud	CMU	
A1	625	1st	0	0	0	0	31458.33
Total (Kip)	582	2nd	15.5	25	0	0	90496.25
	42.875	2nd R	0	0	0	0	857.5
140.5	569	R	0	0	15.5	0	17652.5
A2	625	1st	0	0	0	0	31458.33
Total (Kip)	163	2nd	24.75	0	0	0	18057.5
	463	2nd R	0	0	0	0	9250
71.9	156	R	0	0	24.75	0	13148.75
B1	625	1st	0	0	0	0	31458.33
Total (Kip)	625	2nd	0	0	0	0	43750
	0	2nd R	0	0	0	0	0
87.7	625	R	0	0	0	0	12500
B2	625	1st	0	0	0	0	31458.33
Total (Kip)	319	2nd	0	0	0	0	22312.5
	306	2nd R	25	0	0	0	12875
79.1	625	R	0	0	0	0	12500
C1	625	1st	0	0	0	0	31458.33
Total (Kip)	625	2nd	0	0	0	0	43750
	0	2nd R	0	0	0	0	0
87.7	625	R	0	0	0	0	12500
C2	625	1st	0	0	0	0	31458.33
Total (Kip)	319	2nd	25	0	0	0	29062.5
	306	2nd R	0	0	0	0	6125
83.0	**313**	R	0	0	25	0	16375
D1	625	1st	0	0	0	0	31458.33
Total (Kip)	625	2nd	0	0	0	0	43750
	0	2nd R	0	0	0	0	0
87.7	625	R	0	0	0	0	12500
D2	625	1st	0	0	0	0	31458.33
Total (Kip)	398	2nd	12.75	0	0	0	31300.31
	91	2nd R	0	0	0	0	1815
82.2	625	R	0	0	12.75	0	17663.75
E1	625	1st	0	0	0	0	31458.33
Total (Kip)	625	2nd	0	0	0	0	43750
	0	2nd R	0	0	0	0	0

87.7	625	R	0	0	0	0	12500
E2	625	1st	0	0	0	0	31458.33
Total (Kip)	625	2nd	0	0	0	0	43750
	0	2nd R	0	0	0	0	0
81.5	313	R	0	0	0	0	6250
E3	625	1st	0	0	0	0	31458.33
Total (Kip)	398	2nd	12.75	0	0	0	31300.31
	91	2nd R	0	0	0	0	1815
82.2	625	R	0	0	12.75	0	17663.75
E4	625	1st	0	0	0	0	31458.33
Total (Kip)	319	2nd	25	0	0	0	29062.5
	306	2nd R	0	0	0	0	6125
86.2	470	R	0	0	25	0	19525
E5	625	1st	0	0	0	0	31458.33
Total (Kip)	625	2nd	24.75	0	0	0	50432.5
	0	2nd R	0	0	0	0	0
104.4	625	R	0	0	24.75	0	22523.75
E6	625	1st	0	0	0	0	31458.33
Total (Kip)	0	2nd	0	0	0	0	0
	625	2nd R	0	0	0	0	12500
56.5	625	R	0	0	0	0	12500
F1	625	1st	0	0	0	0	31458.33
Total (Kip)	625	2nd	0	0	0	0	43750
	0	2nd R	0	0	0	0	0
87.7	625	R	0	0	0	0	12500
F2	625	1st	0	0	0	0	31458.33
Total (Kip)	625	2nd	0	0	0	0	43750
	0	2nd R	0	0	0	0	0
87.7	625	R	0	0	0	0	12500
F3	625	1st	0	0	0	0	31458.33
Total (Kip)	625	2nd	0	0	0	0	43750
	0	2nd R	0	0	0	0	0
87.7	625	R	0	0	0	0	12500
F4	625	1st	0	0	0	0	31458.33
Total (Kip)	625	2nd	0	0	0	0	43750
	0	2nd R	0	0	0	0	0
87.7	625	R	0	0	0	0	12500
F5	625	1st	0	0	0	0	31458.33
Total (Kip)	582	2nd	15.5	25	0	0	90496.25
	43	2nd R	0	0	0	0	857.5
141.6	625	R	0	0	15.5	0	18777.5

F6	625	1st	0	0	0	0	31458.33
Total (Kip)	54	2nd	0	25.5	0	0	50236.25
	571	2nd R	0	0	0	0	11425
105.6	625	R	0	0	0	0	12500
			KLF				
CMU		1st	0.63				
Total (KLF)		2nd	0.875				
		R	0.25				
4.0		CMU	2.22				

Wind Loads

z (ft)	k_z	P (psf)	Px_w (lb)	$(Px_w)_{avg}$ (lb)	Node Load (k)
1	0.70	12.34	2159.5	2159.5	
2	0.70	12.34	2159.5	2159.5	
3	0.70	12.34	2159.5	2159.5	
4	0.70	12.34	2159.5	2159.5	
5	0.70	12.34	2159.5	2159.5	
6	0.70	12.34	2159.5	2159.5	
7	0.70	12.34	2159.5	2159.5	
8	0.70	12.34	2159.5	2159.5	
9	0.70	12.34	2159.5	2159.5	
10	0.70	12.34	2159.5	2159.5	
11	0.70	12.34	2159.5	2159.5	
12	0.70	12.34	2159.5	2159.5	
13	0.70	12.34`	2159.5	2159.5	
13.5	0.70	12.34	2159.5	2159.5	14.58
14	0.70	12.34	2159.5	2159.5	
15	0.70	12.34	2159.5	2159.5	
16	0.70	12.34	2159.5	2159.5	
17	0.70	12.34	2159.5	2159.5	
18	0.70	12.34	2159.5	2159.5	
19	0.70	12.34	2159.5	2159.5	
20	0.70	12.34	2159.5	2159.5	
21	0.70	12.34	2159.5	2159.5	
22	0.70	12.34	2159.5	2159.5	
23	0.70	12.34	2159.5	2159.5	
24	0.70	12.34	2159.5	2159.5	
25	0.70	12.34	2159.5	2159.5	
26	0.70	12.34	2159.5	2159.5	
27	0.70	12.34	2159.5	2159.5	21.06

28	0.70	12.34	2159.5	2159.5	
29	0.70	12.34	2159.5	2159.5	
30	0.70	12.34	2159.5	2159.5	

Seismic

Roof Weight			k =	1	T =	0.3
CMU Perimeter	275	ft				
CMU Weight	198412.5	lbs	C_u =	1.5	T_a =	0.2
Stud Perimeter	175	ft				
Stud Weight	3500	lbs	S_{D1} =	0.1312	S_{M1} =	0.1968
Roof Area	4144	ft^2				
Roof Weight	82876.59	lbs	F_v =	2.4	S_1 =	0.082
W_x	281	kips	C_S =	0.076		
			$C_{s,Max}$ =	0.109	$C_{s,Min}$ =	0.013
2nd Story Weight			R =	4	I =	1
CMU Perimeter	700	ft				
CMU Weight	699300	lbs	S_{DS} =	0.302	S_{MS} =	0.453383
Floor Area	24520	ft^2				
Floor Weight	1716409	lbs	F_a =	1.5688	S_S =	0.289

Level	W_x (kips)	h_x (ft)	h_x^k	$W_x h_x^k$	C_{vx}	V	F_x (kips)
R	281	27	27	7595	0.19	21	38.5
2	2416	13.5	13.5	32612	0.81	183	165.3

Shear walls

Wall	Shear force wall must resist (k)	Shear force (k)	V_{Actual} (k)	Lateral Load per foot (plf)
7	246.79	246.79	172.76	2467.95
8	197.44	197.44	138.21	1974.36
9	98.72	98.72	69.10	987.18
10	98.72	98.72	69.10	987.18
11	148.08	148.08	103.65	1480.77
12	296.15	296.15	207.31	2961.54

Wall 4					
Pier	Height (ft)	Length (ft)	Height/Length Ratio	Relative Rigidity, R_f	V distribution (k)
A	13.5	6.625	2.04	0.476	1.49
B	13.5	5	2.70	0.360	1.12
C	13.5	15	0.90	2.916	9.10
D	13.5	5	2.70	0.360	1.12
E	13.5	14.75	0.92	2.826	8.82
F	13.5	14.75	0.92	2.826	8.82
G	13.5	14.75	0.92	2.826	8.82
H	13.5	14.75	0.92	2.826	8.82
I	13.5	14.75	0.92	2.826	8.82
J	13.5	14.75	0.92	2.826	8.82
K	13.5	5.375	2.51	0.610	1.90
				21.678	no shear reinforcing required without considering torsion

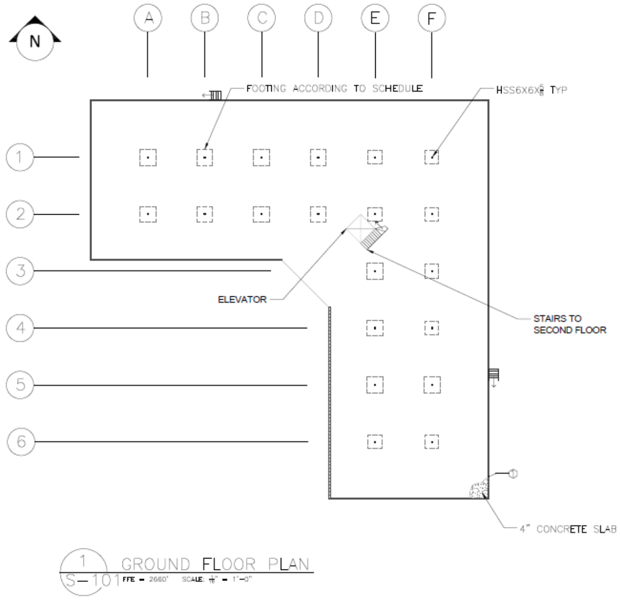
Torsional Eccentricity			
	Eccentricity between center of mass and center of rigidity		
	e_y (ft)	-30.28	
	e_x (ft)	-28.13	
	Add minimum 5% accidental eccentricity		
	e_y (ft)	-21.53	
	e_x (ft)	-19.38	
Seismic Base Shear			
	Accounts for total weight of the walls		
	V(kips)	587.50	represents total seismic shear
Torsional Moment			
	Due to N-S seismic force rotating about CR, using e_x		
	T (k-ft)	-11384.43	
	Due to E-W seismic force rotating about CR, using e_y		
	T (k-ft)	-12646.26	

Composite Floor

Composite Floor Design: 2 Beams NW Concrete								
				Girder Length:		25	ft	
Beam Spacing:		12.5	ft	**Beam Length:**		25	ft	
				# of Beams:		2		
Half Effective Width ((1/2)b_{eff}):				**Effective Width (b_{eff}):**				
1) (1/8) L	3.125	ft		6.25	ft			
2) (1/2) S	6.25	ft						
Step 1) Metal Deck Selection:								
Depth:		3.0	in		Allowable Span:		12.99	ft
Gage:		18	gage		ΦM_{nf}:		80.96	kip-in
Slab Thickness:		5.5	in		ΦV_{nt}:		7000.00	lbs
f'_c:		4,000	psi		I_{av}:		11.9	in^4
N (E_s/E_c):		9			NW Conc W		48.00	psf
NW Concrete		145	pcf		LW Conc W		0.00	psf
LW Concrete		115	pcf		Deck W:		2.80	
Shoring		NO						
Live Loads:			**Dead Loads:**					
Live Load:	63.00	psf	Deck:		2.80	psf		
S.I.D.L:	12.00	psf	Concretre Slab:		48.00	psf		
			Ceiling/Other:		5.00	psf		
			Steel (Beam Only):		0.00	psf		
Deck Moment and Shear Calculation (Factored Loads):								
Load Combination		**W Load**		**Moment ($WL^2/8$)**		**Shear (WL/2)**		
1) 1.4D		78.12	psf	18.31	kip-in/ft	488.25	lb/ft	
2) 1.2D +1.6L		186.96	psf	43.82	kip-in/ft	1168.50	lb/ft	
			Moment: Deck Manual Check OK			80.96	>	43.82
			Shear: Deck Manual OK			7,000.00	>	1,168.50
Deck Deflection Calculation (Unfactored Loads):								
W(psf) Load:			**Deflection ($5WL^4$)/(384EI):**			**Allowable Deflection (L/240):**		
75.00	psf		0.119382	in	<	0.416667	in	OK
Step 2) Check Beam for Construction Loading								
Construction Loads:								
	Service Load		Factored Load (1.4D)		Factored Load (1.2D+1.6L)			
Steel:	0.00	psf	**71.12**	**psf**		**80.16**	**psf**	
Wet Slab:	48.00	psf						

Deck:	2.80	psf						
Live Load:	12.00	psf						
Total Load:	**62.80**	**psf**						
Full Structure Loads:								
	Service Load		Factored Load (1.4D)		Factored Load (1.2D+1.6L)			
Steel:	0.00	psf	**94.92**	**psf**		**182.16**	**psf**	
Slab:	48.00	psf						
Deck:	2.80	psf						
Ceiling/Other:	5.00	psf						
S.I.D.L	12.00	psf						
Live Load:	63.00	psf						
Total Load:	**130.80**	**psf**						
W Section Selection								
Strength Design								
Select Beam Based on Construction Stage:								
W_u:		M_u $(W_uL^2)/8$:		Try	W	10	19	
1.002	kip/ft	78.28125	kip-ft	ΦM_{px}	81	k-ft		
				A_s	5.62	in^2	78.28125	OK
Include Self Weight				d	10.2	in		
Weight	19	plf		I_{xx}	96.3	in^4		
M_u	79.765625	k-ft	<	81	k-ft	OK		
Check Strength in Full Structure:								
b_e	6.25	ft						
	75	in						
A_sf_y	281	k						
a	1.10196078	in	<	2.5	in	OK		
d'	10.0490196	in						
ΦM_n	235.314542	kip-ft						
W_u	2.277	k/ft						
Including self weight:								
M_u	179.375	k-ft	<	235.3145	k-ft	OK		
Deflection Control								
Try	W	16	26					
ΦM_{px}	166	k-ft						
A_s	7.68	in^2						
d	15.7	in						
I_{xx}	301	in^4						
Deflection limit	0.5	in						

E_c	3644.14743	ksi						
n	7.95796563							
b_{tr}	9.42451922	in						
A_{tr}	23.561298	in^2						
Y bar	4.22452429	in						
I_{tr}	1153.92081	in^4						
W_s (incl. self wt.)	1.661	klf						
Deflection	0.43625275	in	<	0.5	in	OK		
Select Shear Studs								
Full shear	281	k	A_sf_y					
Stud Strength	18.3	k	AISC Table 3-21					
# of Stud/side	15.36		A_sf_y / Stud Strength					
# of Stud pairs	16							
Cost Estimates								
Steel	$ 0.75	/lb						
Shear Connection	$ 200.00	/Connection						
Cope	$ 100.00	/Cope						
NW Concrete	$ 85.00	$/yd^3$						
LW Concrete	$ 100.00	$/yd^3$						
Deck	$ 1.00	$/ft^2$						
Shear Stud	$ 5.00	/shear stud						
Cost Penalty 1	$ 1,000.00	/in above 6"						
Cost Penalty 2	$ 1,000.00	/in above standard						
Cost Penalty 3	$ 2,000.00	/bay for shored contruction						
Cost								
Steel	$ 975.00							
Shear Connection	$1,600.00							
Cope	$400.00							
NW Concrete	$ 651.34							
LW Concrete	$ -							
Deck	$1,750.00							
Shear Stud	$160.00							
Cost Penalty 1	$0.00	(Slab Thickness > 6")						
Cost Penalty 2	$0.00	(Nom. d > 16")						
Cost Penalty 3	$0.00	(Shoring)						
Total	**$5,536.34**							
Weight								
	/bay	psf						
Joist	950	1.52						
Girder	2100	3.36	W	24	84			
Deck	1750	2.80						
Concrete	30000	48.00						



NOTES
1. CONCRETE DECK OMITTED FOR CLARITY

Footing Schedule			
Footing Number	B (ft)	L (ft)	Df (ft)
A1	7	7	3.5
A2	7	7	3.5
B1	7	7	3.5
B2	7	7	3.5
C1	7	7	3.5
C2	7	7	3.5
D1	7	7	3.5
D2	7	7	3.5
E1	6	6	3.5
E2	6	6	3.5
E3	7	7	3.5
E4	7	7	3.5
E5	7	7	3.5
E6	6	6	3.5
F1	6	6	3.5
F2	6	6	3.5
F3	6	6	3.5
F4	6	6	3.5
F5	7	7	3.5
F6	6	6	3.5

Sheet No. S-101 Sheet 1 of 1
REVISIONS
Date | Description | Approval

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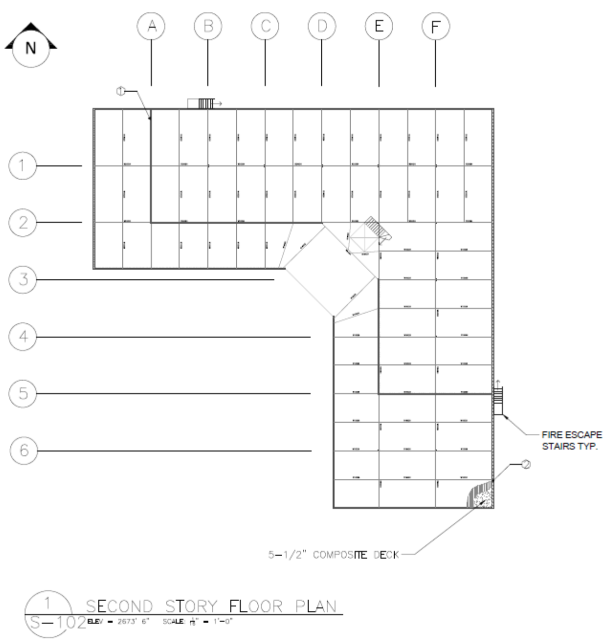
Extent Engineering
University of Arizona

GENERAL_OFFICE_BUILDING
PRACTICE
ORACLE_ROAD_AND_LINDA_VISTA_BOULEVARD TUCSON_ARIZONA
ORO_VALLEY

Designed T_BENNETT-GOLD
Drawn T_BENNETT-GOLD
Checked S_SARMIENTO
Title GROUND FLOOR
Date 4/26/11



NOTES
1. STEEL STUD WALL BEARS ON FLOOR BEAM
2. COMPOSITE DECK OMITTED FOR CLARITY

Sheet No. S-102 Sheet 1 of 1
REVISIONS
Date | Description | Approval

EXTENT
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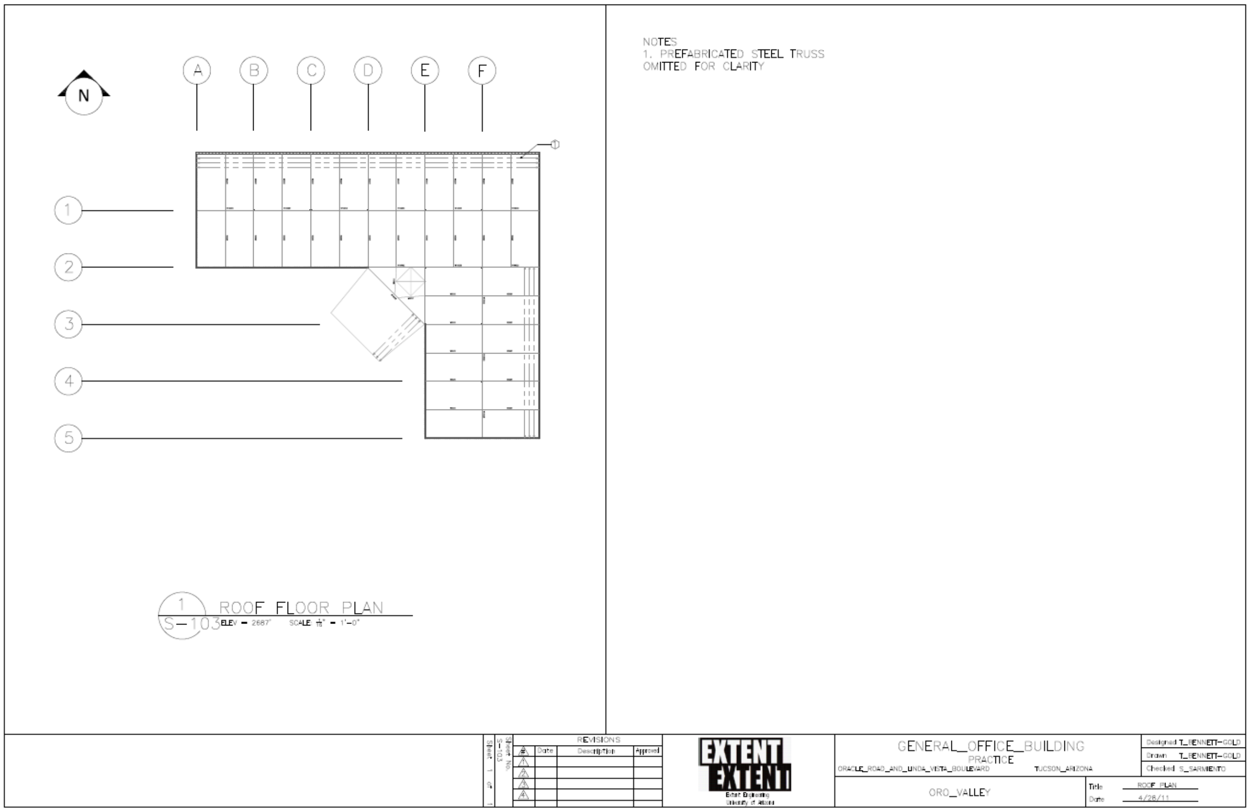
University of Arizona

GENERAL_OFFICE_BUILDING
PRACTICE
ORACLE_ROAD_AND_LINDA_VISTA_BOULEVARD TUCSON_ARIZONA
ORO_VALLEY

Designed T_BENNETT-GOLD
Drawn T_BENNETT-GOLD
Checked S_SARMIENTO
Title SECOND FLOOR
Date 5/5/11


N
A
B
C
D
E
F
1
2
3
4
5
1 ROOF FLOOR PLAN
S-103 ELEV = 2687' SCALE 1/8" = 1'-0"
NOTES
1. PREFABRICATED STEEL TRUSS
OMITTED FOR CLARITY
Sheet 1 of 1
Sheet No. S-103
REVISIONS
Date Description Approved
EXTENT
EXTENT
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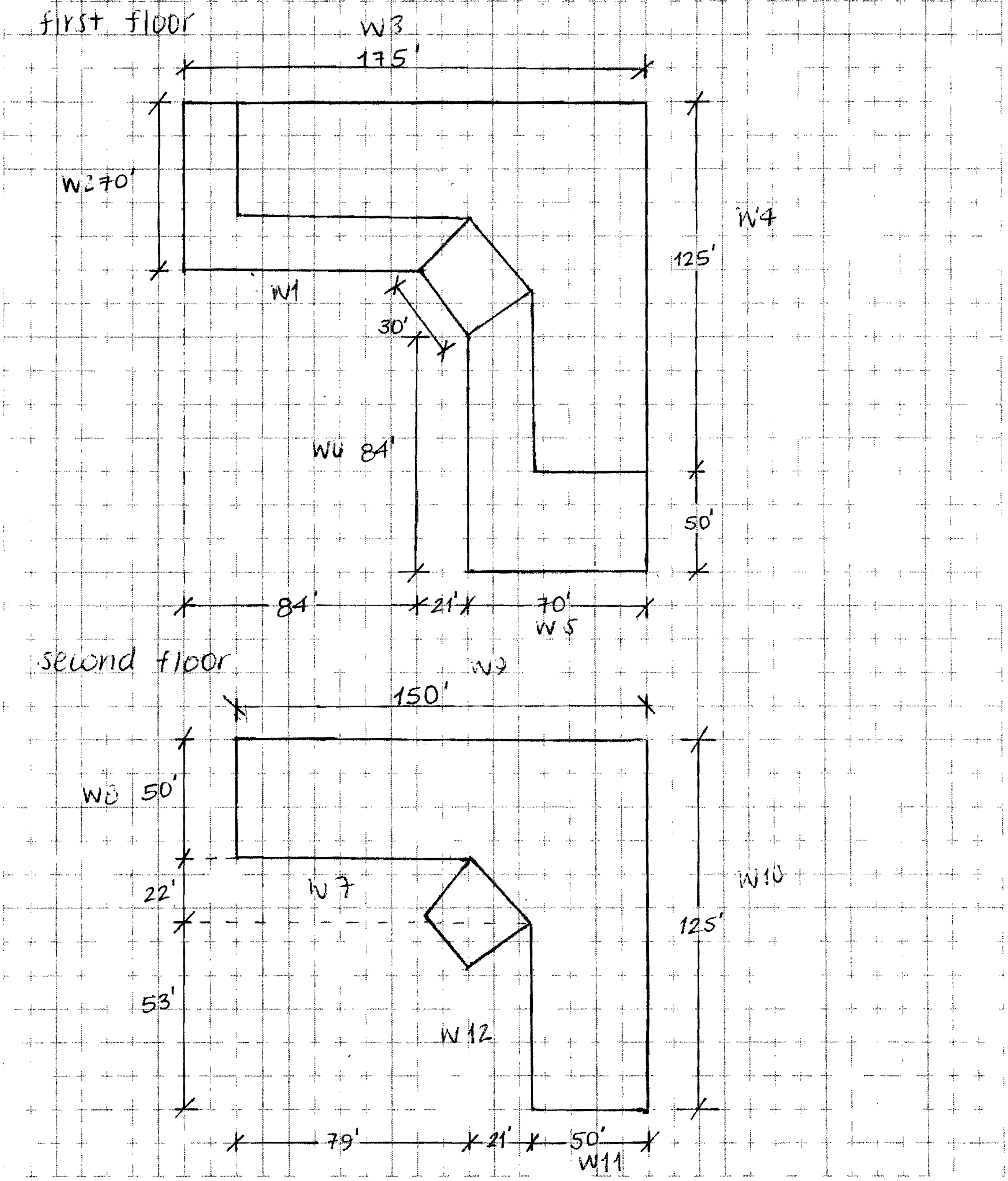
GENERAL_OFFICE_BUILDING
PRACTICE
ORACLE_ROAD_AND_LINDA_VISTA_BOULEVARD
TUCSON_ARIZONA
ORO_VALLEY
Designed T_BENNETT-GOLD
Drawn T_BENNETT-GOLD
Checked S_SARMIENTO
Title ROOF PLAN
Date 4/28/11


first floor
W3
175'
W2 70'
W4
125'
W1
30'
W6 84'
50'
84'
21'
70'
W5
second floor
W9
150'
W8 50'
W7
22'
W10
125'
53'
W12
79'
21'
50'
W11

Occupancy Load:

First floor:

$$\frac{175^2 - 105^2 + (1/2)21^2}{100} = 198.205 \longrightarrow 198$$

Second floor:

$$\frac{150 \times 125 - 75 \times 100 + 22.5^2}{100} = 117.563 \rightarrow 118$$

Occupancy Load for building = 198 + 118 = 316

Bathrooms for building:

- First floor occupancy: 198 assuming 50% F & 50% M
  99 F & 99 M

  n° of bathroom stalls: $1 + \frac{x}{50}$, where x equals occupancy load per gender
  [P] Table 2902.1 - Business sections

  women bathroom:

  $1 + \frac{99}{50} = 2.98 \rightarrow 3$ stalls

  $\frac{99}{40} = 2.475 \rightarrow 3$ sinks

  men bathroom:

  $1 + \frac{99}{50} = 2.98 \rightarrow 3$ stalls, which includes urinals

  $\frac{2}{3} \times 3 = 2$ urinals, see [P] section 419.2

  $\frac{99}{40} = 2.475 \rightarrow 3$ sinks

Total: women: 3 stalls, 3 sinks — men: 2 urinals & 1 stall, 3 sinks

Bathrooms for building - cont.:

- Second floor occupancy: 118 assuming 50% F & 50% M
  59 F & 59 M

women bathroom:

$1 + \frac{59}{50} = 2.18 \rightarrow 3$ stalls

$\frac{59}{40} = 1.475 \rightarrow 2$ sinks

men bathroom:

$1 + \frac{59}{50} = 2.18 \rightarrow 3$ stalls

$\frac{2}{3} \times 3 = 2$ urinals

$\frac{59}{40} = 1.475 \rightarrow 2$ sinks

Total women 3 stalls men 2 urinals + 1 stall
2 sinks 2 sinks

* at least 1 stall must be ADA approved in all bathrooms

Drinking Fountains:

2 fountains per floor, one of which is ADA approved
Total of 4 fountains in the building

Other:

1 service sink is required in the building

Wind Load

windward side

$P = q_z G C_p - q_i(\cancel{GC_{pi}})^{0}$ ignoring internal pressure

- $q_z = 0.00256\, K_z K_{zt} K_d V^2 I$

$K_z = 2.01\left(\frac{z}{z_g}\right)^{2/\alpha}$

assume Exposure B, case 1 : low rise building
$\alpha = 7.0$ & $z_g = 1200'$ $(h \le 60\,ft)$
ASCE 7-05 Table 6-2

$K_z = 2.01\left(\frac{30}{1200}\right)^{2/7} = 0.7$ for $z \le 30'$

$K_{zt} = 1$

$K_d = 0.85$ directionality factor, building
ASCE 7-05, Table 6-6

$V = 90$ mph basic wind speed for Oro Valley, AZ
ASCE 7-05, Figure 6-1

$I = 1.00$ importance factor, category II

$q_z = 0.00256 \times 0.7 \times 1 \times 0.85 \times 90^2 \times 1.00 = 12.34$ psf

- $G = 0.85$ gust factor, rigid structure

- $C_p = 0.8$ wall pressure coefficient, windward wall

$P = 12.34 \times 0.85 \times 0.8 = 8.39$ psf for $z \le 30'$

Wind load

leeward side

$P = q_h G C_p$ h = height of building

• $q_{30} = 0.00256\, K_h\, K_{zt}\, K_d\, V^2\, I$

$K_h = 0.7$ ASCE 7-05, Table 6-3
exposure B, case 1

* see windward side for other variables

• $G = 0.85$ gust factor

• $C_p = -0.5$ pressure coefficient, leeward wall
L/B (0-1)

$P = (0.00256 \times 0.7 \times 1 \times 0.85 \times 90^2 \times 1.00) \times 0.85 \times -0.5$

$P = -5.24$ psf

$P = 5.24$ psf away from the building

Wind loads

uplift

$P = q_h G C_p$

- $q_{30} = 12.37$
- $G = 0.85$ gust factor
- $C_p = -0.9$ roof pressure coefficient
  ASCE 7-05, Figure 6-6 (con't)

$P = 12.37 \times 0.85 \times -0.9$

$P = -9.44$ psf

$P = 9.44$ psf away from the building

Extent Engineering

Job rigidity Subject

Designed by SARMIENTO

Sheet No. 1 of 2

Date 4/23/11

Ignore atrium for simplicity

rigid floor diaphragm

V @ ctr of mass

N-S [illegible]

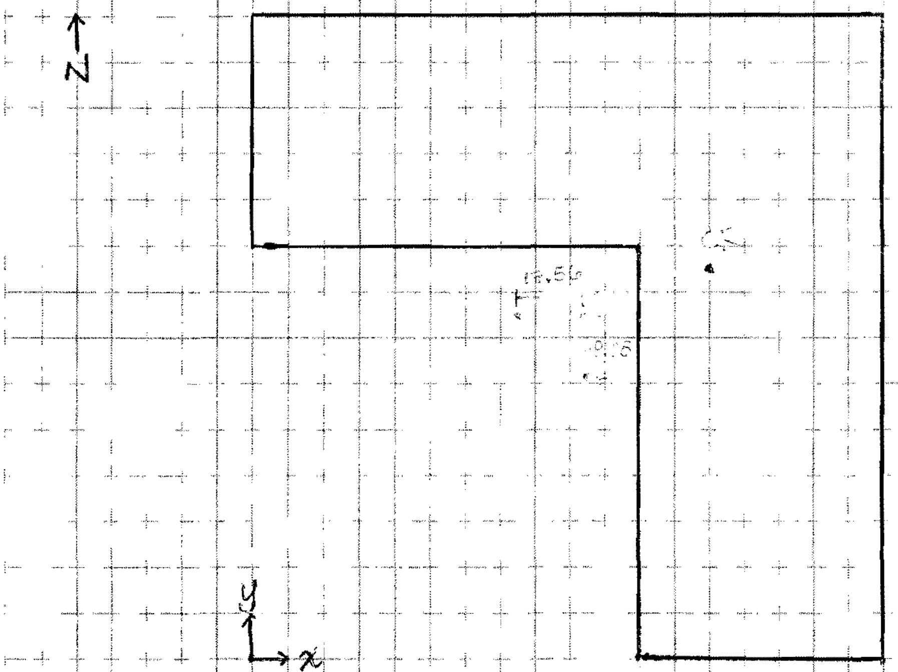
[illegible]

resisting moment: [illegible]

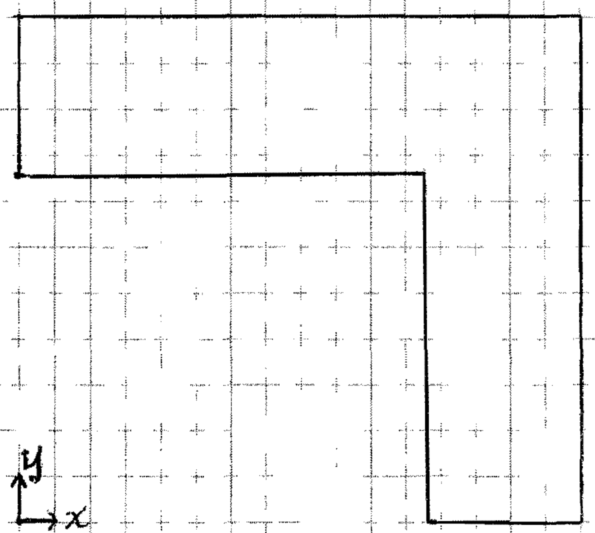
E-W [illegible]

[illegible]

resisting moment: [illegible]



flexible roof diaphragm



Center of mass

• rigid floor diaphragm

Region 1: $x = \frac{105}{2}$, $y = 105 + \frac{70}{2}$, Area $= 105 \times 70 = 7350\ ft^2$

Region 2: $x = 105 + \frac{70}{2}$, $y = \frac{175}{2}$, Area $= 175 \times 70 = 12{,}250\ ft^2$

$$x_{cm} = \frac{52.5 \times 7350 + 140 \times 12{,}250}{7350 + 12{,}250} = 107.1875\ ft$$

$$y_{cm} = \frac{140 \times 7350 + 87.5 \times 12{,}250}{7350 + 12{,}250} = 107.1875\ ft$$

center of mass: 107.1875', 107.1875'

• flexible roof diaphragm

Region 1: $x = \frac{100}{2}$, $y = 75 + \frac{50}{2}$, Area $= 100 \times 50 = 5{,}000\ ft^2$

Region 2: $x = 100 + \frac{50}{2}$, $y = \frac{125}{2}$, Area $= 125 \times 50 = 6250\ ft^2$

$$x_{cm} = \frac{50 \times 5{,}000 + 125 \times 6{,}250}{5{,}000 + 6{,}250} = 91.67\ ft$$

$$y_{cm} = \frac{100 \times 5{,}000 + 62.5 \times 6{,}250}{5{,}000 + 6{,}250} = 79.167\ ft$$

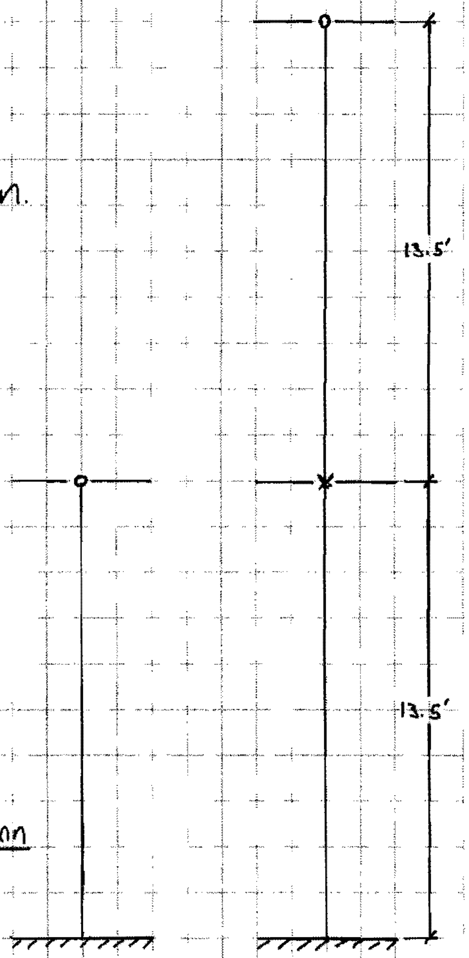
center of mass: 91.67', 79.167'

* calculation only necessary for rigid diaphragms

Column Design

27' or 13.5' HSS 6x6 column
Grade 50 steel
Assume pin connection at top of column
and fixed connection at ~~■■~~ ~~■~~ foundation.
Frame is sidesway inhibited.

Assume negligible moment due to
unbalanced or eccentric loading.



Loading Conditions

Dead Load: 156^k

Live Load: 107^k

Roof Live Load: 8^k

Critical Load Combination

$$P_u = 1.2\,DL + 1.6\,LL + .5\,L_R$$
$$= 1.2 \cdot 156 + 1.6 \cdot 107 + .5 \cdot 8$$

$$P_u = 363^k$$

Select Section

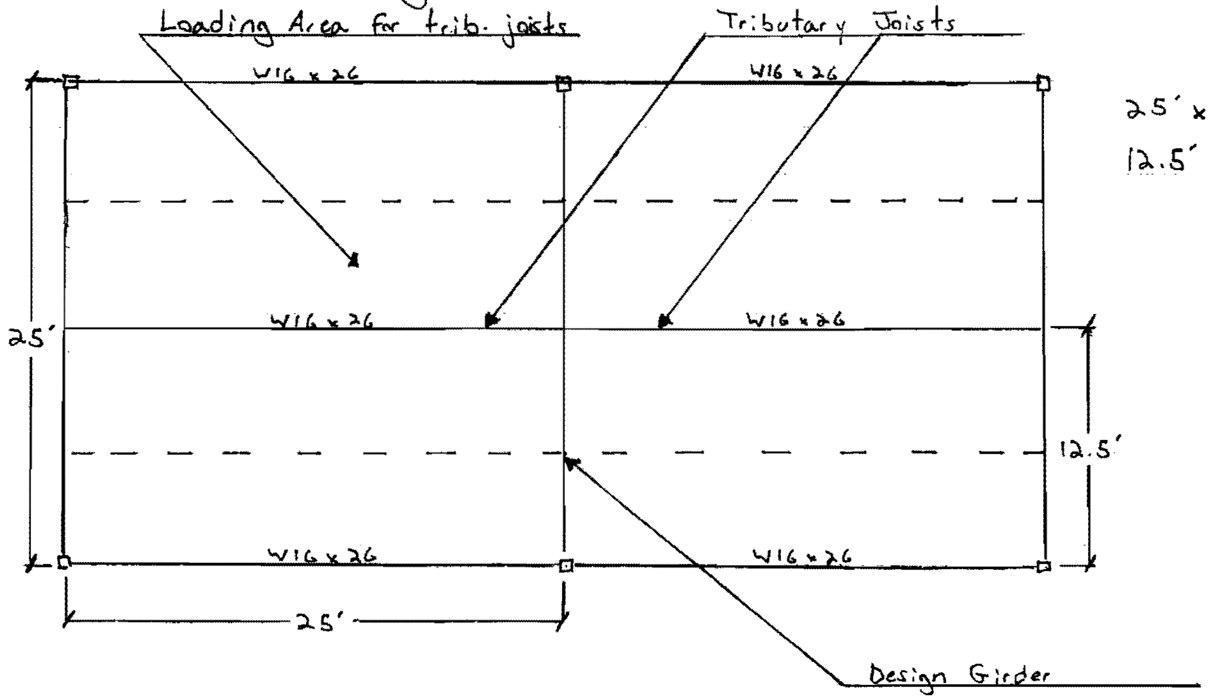
$k = 0.8$ $L_b = 13.5$ $\Rightarrow$ $kL = 11$

Use AISC Table 4-4 Try HSS $6 \times 6 \times 5/8$

$$\phi_c P_n = 378^k > P_u = 363^k$$

Use HSS $6 \times 6 \times 5/8$

Girder Design



25' x 25' bay
12.5' joist spacing

Loading Conditions

Dead Load

5.5" concrete slab	48 psf
3" 18 gage deck	2.8 psf
ceiling	5 psf

SIDL

Mech	10 psf
Finish	1 psf
Misc	3.2 psf
	70 psf

Live Load 63 psf

Joist Load for serviceability

$12.5' \times 25' \times (70 + 63) = 42^k$

Load Combinations

$1.4\,DL = 98$ psf

$1.2\,DL + 1.6\,LL = \underline{185 \text{ psf}}$ critical

Joist Load

$12.5' \times 25' \times 185$ psf

$= \underline{58^k}$

Girder point load is one half the load carried by adjacent beams loading on design girder

$\therefore\ P_u = 58^k$

$P = 42^k$ (serviceability)

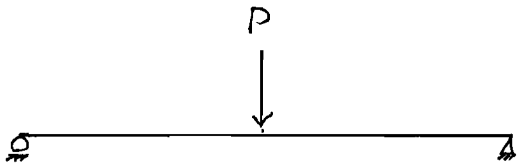
Check Serviceability

Maximum allowable deflection for exposed concrete: $\Delta_{max} = L/600$

$$\Delta_{max} = \frac{25 \times 12}{600} = \underline{.50''}$$

Girder Loading

Assume simply supported beam conditions. Assume Grade 50 steel.
Joist is continuously braced by composite deck
$L_b = 0$



$$M_u = \frac{PL}{4} = \frac{58 \cdot 25}{4} = \underline{363\ k\text{-}ft}$$

$$V = \frac{P}{2} = \frac{58}{2} = \underline{29^k}$$

$$\Delta = \frac{PL^3}{48EI} = \frac{42(25 \cdot 12)^3}{48 \cdot 29{,}000 \cdot I} = \frac{815}{I}$$

$$\Delta_{sw} = \frac{5wL^4}{384EI} = \frac{5w(25 \cdot 12)^4}{384 \cdot 29000\ I} = 3637 \frac{w}{I}$$

Select Section

Find I_{req} for serviceability limit

$$\Delta = \Delta_{max} \Rightarrow \frac{815}{I_{req}} = .50'' \Rightarrow I_{req} = 1630$$

Trial Section

Use AISC Table 3-2

Try W24x68 $I = 1830$ $\Delta = \frac{815}{1830} = .445''$ $\Delta_{sw} = 3637 \frac{.068}{1830} = .135''$

$\Sigma\Delta = .445 + .135 = .581''$ X NG

Try W24x76 $I = 2100$ $\Delta = .388''$ $\Delta_{sw} = .132''$

$\Sigma\Delta = .388 + .132 = .520$ X NG

Try W24x84 $I = 2370$ $\Delta = .344''$ $\Delta_{sw} = .129$ $\Sigma\Delta = .473''$ ✓

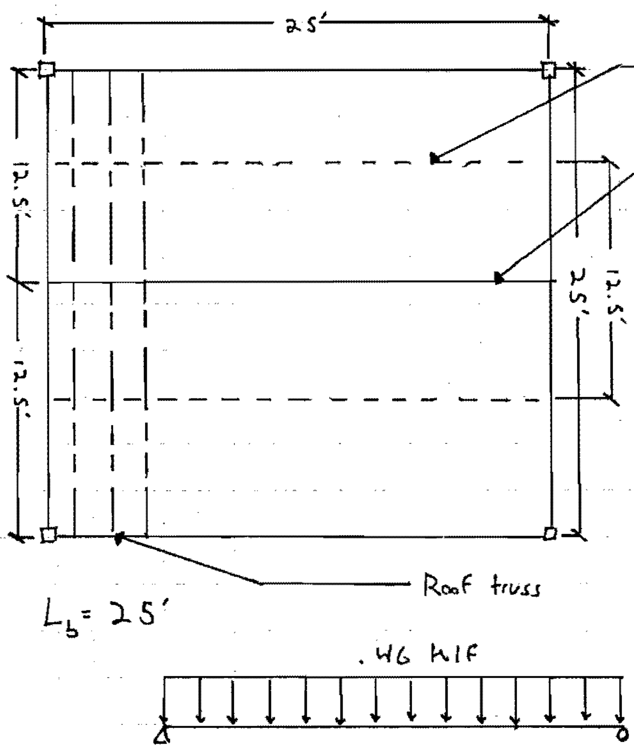
Additional moment from self weight: $\frac{(1.2 \cdot .084) \cdot 25^2}{8} = 8\ k\text{-}ft \Rightarrow M_u = 363 + 8 = 371k$

$\phi_b M_{px} = 840\ k\text{-}ft > M_u = 371\ k\text{-}ft$

$\therefore$ Use $\underline{W24 \times 84}$

Roof Joist Design

Bolted connections ⇒ assume simple support conditions
Roof trusses @ 2' o.c. ⇒ assume distributed load
Top bearing trusses ⇒ assume unbraced for full length



Loading Conditions

$DL = 20$ psf $\qquad LL_R = 8$ psf

Critical Load Combination:

$1.2DL + 1.6LL_R = \underline{36.8 \text{ psf}}$

$\omega = 36.8 \times 12.5 = 460 \text{ plf} = \underline{.46 \text{ klf}}$

Serviceability : $20 + 8 = \underline{28}$ psf

$\omega_s = 28 \times 12.5 = 350 = \underline{.35 \text{ klf}}$

$M_u = \dfrac{.46 \cdot 25^2}{8} = \underline{35.9 \text{ k-ft}}$

Serviceability

Unplastered roof construction
$\Rightarrow \Delta_{max} = L/180 = 1.67''$

Use grade 50 steel
$E = 29{,}000$ ksi

Find I_{req}

$$1.67 = \frac{5 \cdot \frac{.35}{12}(25 \cdot 12)^4}{384 \cdot 29{,}000 \cdot I_{req}} \Rightarrow I_{req} = \underline{63.7 \text{ in}^4}$$

Select Section

Use AISC Table 3-3

Try W12 x 14 $I = 88.6$

$w_{sw} = .014$ klf $\Delta = \dfrac{5\frac{(.35+.014)}{12}(25\cdot 12)^4}{384\cdot 29000\cdot 88.6} = 1.25'' < 1.67''$ ✓ ok

Check Lateral torsional buckling

Distributed load, no bracing ⇒ $C_b = 1.14$

$\phi_b M_n = C_b \phi M_n \leq \phi_b M_p$

$\dfrac{M_u}{C_b} = 31.5$ use AISC table 3-11

W12 x 14 X NG

Try W8 x 31 $\phi M_n = 71$ k-ft ⇒ $\phi_b M_n = 71 \cdot 1.14 = 80.94$

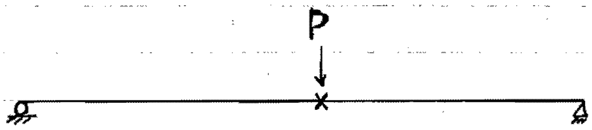
Check deflection

$I = 110$ $\Delta = \dfrac{5\frac{(.031+.35)}{12}(25\cdot 12)^4}{384\cdot 29000\cdot 110} = 1.05''$ ✓

<u>Use W8 x 31</u>

Roof Girder Design

Bolted connections ⇒ assume simple support conditions
Braced at center span by roof joist connection



$L_b = 12.5'$

Loading Conditions

~~$P_u = (.46\text{ klf} + .031\text{ klf}) \cdot 25$~~ $P_u = (.46 + 1.2 \cdot .031) \cdot 25$
$= \underline{12.43\text{ k}}$

$M_u = \dfrac{12.43 \cdot 25}{4}$
$= \underline{77.69}\text{ k-ft}$

Serviceability: $P_s = (.35 + .031) \cdot 25$
$= \underline{9.525^{k}}$

Serviceability

$\Delta_{max} = L/180 = \underline{1.67''}$

use grad 50 steel
$E = 29000$ ksi

Find I_{req}

$$1.67 = \frac{9.525 \cdot (25 \cdot 12)^3}{48 \cdot 29000 \cdot I_{req}} \Rightarrow \underline{I_{req} = 110}$$

Select Section

Use AISC table 3-3

try $\underline{W12 \times 19}$ $\underline{I = 130}$ $\Delta = \Delta_p + \Delta_{sw}$

$$\Delta_p = \frac{9.525\,(25 \cdot 12)^3}{48 \cdot 29000 \cdot 130} = 1.42 \quad \Delta_{sw} = \frac{5 \cdot \frac{.019}{12}(25 \cdot 12)^4}{384 \cdot 29000 \cdot 130} = .0443$$

$\underline{\Delta = 1.47''}$ ✓ ok

Lateral Torsional Buckling

Lateral bracing at load point $\Rightarrow \underline{C_b = 1.67}$

$\frac{M_u}{C_b} = \underline{46.52}$ Use AISC Table 3-11

$L_r > L_b$ X NG

Try ~~W10~~ x ~~W8 x 21~~ W10 x 22 $I = 118$

$\phi_b M_n = 66 \Rightarrow C_b \phi_b M_n = 110$ k-ft

$\Delta_{sw} = \frac{5 \cdot \frac{.022}{12} (25 \cdot 12)^4}{384 \cdot 29000 \cdot 118} = .057''$

$\Delta_p = \frac{9.325 (25 \cdot 12)^3}{48 \cdot 29000 \cdot 118} = 1.566$

$\Delta = 1.62'' < 1.67''$ ✓ ok

<u>Use W10 x 22</u>