California State University, Fresno

Department of Civil and Geomatics Engineering

Final Design Report

CE 180B: Senior Project

Instructor: Dr. Fariborz Tehrani

By:

STATIC ENGINEERING

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ABBREVIATIONS

CBC: California Building Code

ACI: American Concrete Institute

ASTM: American Society of Testing and Materials

SPT: Standard Penetration Test

ASCE: American Society of Civil Engineers

AWWA: American Water Works Association

LRFD: Load and Resistance Factor Design

NOTATIONS

psf	pounds per square foot
lbs	pounds
kips	thousand pounds
ft	feet
pcf	pounds per cubic foot
N_{60}	standard penetration number
q_{all}	allowable bearing capacity
q_{ult}	ultimate bearing capacity
FS	factor of safety
Q_{all}	allowable load
D_f	depth of footing below ground
LS	lump sum
CY	cubic yards
LF	linear foot
SQYD	square yards
CFT	cubic foot
gpd	gallons per day
gpm	gallons per minute
cfs	cubic feet per second
in/hr	inches per hour

ABSTRACT

The current valley children's hospital located in Madera is reaching its capacity to serve the high number of patients in the Central Valley. Families in Fresno and other nearby cities must travel long distances to obtain proper treatment for their children, which can be problematic for patients who are in need of immediate medical care. A new children's hospital in Tulare will be able to solve this issue and improve the quality of medical services received by families in the Central Valley.

Jaskaran

The report presents the design process of 3-story steel building Children's Hospital Wing. The building is located near Tulare, Ca of off Hwy 99 and on Paige Rd. The building floor plan dimension are 100 ft. by 240 ft. The columns are spaced 30-40-30 ft. along the short dimension and 30 ft. in the longer dimension. Each floor has the height of 15 ft., and the height to the top of parapet is 49 ft. The building consists of moment frames in the longer direction and braces in the shorter direction. The brace system was chosen in the shorter direction in order to compare the difference between wall sizes compared to moment frames in the shorter direction. The design of the wing included the design of roof joists, roof girders, floor joists, floor girders, columns for all three levels, X-bracing, simple connections, moment frame connections, brace frame connections, footing to column connections, and the isolated square footing. The flooring system is a concrete slab with subflooring and terrazzo flooring.

Regina

The associated parking structure will be a 200-stall, two-story structure subject to universal design codes of ASCE 7-10 and the California Building Code, in addition to the local jurisdiction of Tulare County. The structure will be constructed from precast, prestressed concrete. The

prestressing will be achieved through the use of pre-tensioning. The design of the structure was broken down into five different beams, four different girders, five different columns, and shear walls. Each member was analyzed for all applicable loads through LRFD critical load combinations. After the critical load was found, each member was analyzed for shear, moment, and axial force as appropriate. ACI was then used to design each member to resist the critical loads. In addition to member design, a 42-inch high barrier must be designed in order to protect motor vehicles from crashes.

Lamia

The design of foundations for the parking structure and a site grading plan are essential components of the design of this project. The overall objective of designing the foundations is to ensure that the subsoil can safely support the column loads applied by the structure without undergoing shear failure or excessive settlement. The purpose of the grading plan is to ensure that the ground is properly sloped to allow the storm water to flow to the drainage inlets, and to determine cut and fill volumes of soil required for the site. The outcome of the design phase will result in a project that is consistent with the proper standards and regulations while meeting the appropriate requirements for quality and sustainability.

Mario

The design of a sanitary sewer system is another essential component for designing and constructing a hospital. The main objective of the sewer design is to safely carry the peak discharge to the main industrial sewer line at the same time maintaining a minimum velocity of 2-ft/s at depth to diameter (d/D) ratio of 0.5. The design should accommodate for minimum and

maximum flow rates of the waste water that will be generated. An inadequate design will result in sewage backup or solids settling at the bottom of the pipes which in return can cause clogging.

Nadun

For any given project, the stormwater drainage system is an utmost important element. The stormwater drainage system has to be able to drain the same amount of runoff in the post-developed site when compared to the pre-developed site. If there is no proper stormwater drainage, there is a risk of flooding ultimately causing the hospital to shut down. This stormwater design consists of designing the collection system, conveyance facility and the collection facility. The details of the design and the results will be presented further along this report.

Deep

Potable water distribution system is an essential part of this project. Water distribution system will consist of pipes, fittings, valves, pump, storage tank etc. This report gives water demands and pipe layout of the hospital. Preliminary pipe sizes are also included in this report. Water distribution system will receive water from city of Tulare, CA and storage tank will store water for fire flow. Water distribution will be designed to operate under peak flows.

Antonio

The purpose of this Schematic Design Report is to provide to the reader the process of the analysis and design of the Water Distribution System to the New Valley Children's Hospital and how this new hospital will impact the existing water distribution system for the City of Tulare. Improvements will have to be made to the existing system to maintain an adequate distribution network throughout the City of Tulare at all times. In this Schematic Design Report, you will

find the analysis of the existing water system, the design process, regulations that had to be followed per the City of Tulare, Plans of the Design, and Specifications to certain types of pipes that may be used in this design.

Bagazi

The purpose of this report is to submit a schematic design to build an adequate parking lot plan for a children hospital located west from Tulare, CA and south from Fresno, CA; south of Paige Avenue. There will be design of two parking lot. There will faculties parking lot and patients parking garage. The main reason of this design is to help the community of this area and around to have a fast and good medical care for children. The design of the parking lot structure not only serves to create parking space, but also considers accessibility to surrounding building structures. During the planning phase of designing a parking lot structure for the children's hospital, we referenced and analyzed different design standards and options available to us. The student will start the design of the transportation parking lot design based on the specification of either the city or standards for hospitals as well as to the *Parking Generations* book. These references helped in the estimation and revealed the overall demand of the parking lot design. Furthermore, the Caltrans method was utilized to conduct flexible pavement calculation of the parking lot area based on the maximum capacity with respect to the two buildings, clubhouse, and hotel that were already in its fixed place. The report includes the calculation for the parking lot design using AutoCAD drawing as the process contributing to the project, and includes the parking orientation and geometry research.

CHAPTER 1: DESIGN SCOPE OF WORK

The scope of work for structural design includes the design of the hospital wing for lateral and gravity loading and includes the design of the foundation design of the hospital wing.

Lamia

The scope of work for the geotechnical design of this project include the foundations for the parking structure and the site grading plan. Necessary soil data for the foundation design was obtained from geotechnical reports, including boring logs and direct shear test reports, which were provided by Salem Engineering. The data was interpreted and the necessary soil properties were determined. Using the relevant soil data, design calculations were completed to determine the allowable load bearing capacity and settlement, and the calculations were used to determine the appropriate depth and size of the spread footings. The footings were designed to be able to support the column loads with the proper factor of safety and settlement was minimized to a suitable level. Additionally, reinforcement bars of the appropriate size and spacing were designed for the footings. The grading plan was created using AutoCAD 2014 and the cut and fill quantities of soil for the site were determined. The grading plan was designed to ensure that the finished ground surface has the appropriate elevations and slopes to allow for proper drainage of storm water to the drainage inlets. Construction drawings and plans were prepared, including foundation plans, footing cross-sections and reinforcement details. Additionally, a quantity take-off list and detailed cost estimates were prepared for the foundations and grading.

Mario

The scope of designing the sanitary sewer system, the designer must conduct preliminary investigation, review design specifications required by the county, design the pipe size and gradient, and prepare drawings and cost estimations. The design should accommodate for

minimum and maximum flow rates of the waste water that will be generated. An inadequate design will result in sewage backup or solids settling at the bottom of the pipes which in return can cause clogging. The first task is to obtain data such as annual occupancy, average daily consumption, water demand, and existing pipe location for the main sewer line. Once the data is obtained a hydrograph will be produced to determine peaking factors that will help design for maximum and minimum flow rates. The hydraulic design will accommodate minimum and maximum flowrates by designing pipe size and gradient of the pipe. Manhole spacing will also be following Tulare public work standards. Software such as SewerGEMS and AutoCAD will be used to help design the sanitary sewer system. Once the design is complete cost estimations can be calculated using the quantity takeoff. A set of construction plans such as site plan, plan and profile sheets, and detail sheet will be created for this design. Throughout the whole design phase construction specifications will be followed according to Tulare county standards.

Nadun

The methodology used for this design in similar to any other stormwater design. First, the rainfall data was collected from the city of Tulare stormwater masterplan. After analyzing rainfall data, an Intensity-Duration-Frequency (IDF) curve was developed for the site location. With the help of the grading plan the locations of the inlets were determined. Using the rational method, the flow for the inlets through on land flow and gutter flow was calculated. Using the rational method, the pipe size and pipe flow was determined. Using the volumetric method, the dimensions of the detention pond was determined. The expected results of the design would be an adequate design that can withstand a 2-hour event with return period of 10-years.

Deep

The scope of work for pressurized distribution system includes demand calculations for potable water and fire flow to size main pipes, select appropriate booster pump and to determine volume of tank necessary to store water for fire flow. Design must meet required flow and pressure demands set by the city. Required pipe type and placement of pipe must follow standards set by city of Tulare. Preliminary research was done to find applicable codes and come to come up with design layout. Demand for potable water, fire flow, and length and diameter of pipe was calculated. Water distribution system layout was optimized for efficiency. PVC C-900 pipe material was recommended to be used.

CHAPTER 2: DESIGN METHODOLOGY AND CALCULATIONS

Jaskaran

For the structural design of the hospital wing in the schematic design process the beams, and isolated spread footing was designed for gravity loading and the braces were design for lateral loading. The methodology used to design the members was LRFD.

Loading

Gravity Loads

ASCE 7-10 and ASCE 7-16 are used to calculate the dead, live, and roof live loads. Rain, Snow, and Ice loads were not considered in the design due to the location of the building in California.

Dead load

Dead Loads are calculated using ASCE 7-16 and AISC design example. The minimal design dead load is obtained using ASCE 7-16, Chapter C3, Table C3.1-1a. Total dead load for the roof were calculated to be 100 psf. The total dead load for the floors was calculated to be 125 psf. Calculations and material used are shown below in the Appendix 1.

Live Load

ASCE 7-10 is used to obtain the minimal design loads for the roof live and floor live from Chapter 4, Table 4.3-1. Calculations shown below in the Appendix.

- For the operating room and laboratories 60 psf is used.
- For patient's room 40 psf is used.
- For the corridor above first floor 80 psf is used.

Lateral Load

Wind Load

For the calculation for the wind loads ASCE 7-10 chapter 28 part 2, the envelope method is used. The envelope method part 2 is for enclosed simple diaphragm low-rise buildings. The risk

category for the hospital was determined to be 4 using Table 1.5-1. Risk category 4 is for buildings designated as essential facilities. For the wind importance Table 1.5-2 was used, for risk category 4 the wind importance factor is 1. The basic wind speed is obtained using Figure 26.5-1B in ASCE 7-10, it was found to be 115 miles per hour for risk category 4. For the wind directional factor Table 26.6.1 is used to, for building main wind force resisting system the directionality factor is .85. Section 26.7.2 was used to obtain the exposure category C. Topographic factor was found to be one because the location of the building is not on an escarpment, ridge, or hill. The gust factor is obtained for Section 26.9.1. The adjustment factor was obtained from Figure 28.6-1 in the ASCE 7-10, using the mean roof height of 49 feet. The horizontal design wind pressure is determined using ASCE 7-10 equation 28.5-1.

- Risk Category is 4
- Importance Factor is 1
- Basic Wind Speed is 115 mph.
- Directionality Factor is .85
- Exposure Categories is C
- Topographic Factor is 1
- Gust Factor is .85
- Adjustment Factor is 1.572
- The horizontal design wind pressures for zones A, B, C, and D are 33.02, -17.14, 21.85, and -10.22 respectively.

Seismic Load (Coauthor: Nicole Mahoney)

For the calculation of seismic loading the equivalent lateral force method is used. The importance factor was obtained using ASCE 7-10 Table 1.5-1 for risk category of 4. The design spectral response acceleration is obtained using the USGS Seismic Deign Map. To obtain the approximate fundamental -period ASCE 7-10 equation 12.8-7 was used and C_t and x were found using the ASCE 7-10 Table 12.8-1. The response modification coefficient R was obtained using ASCE 7-10 Table 12.2-1. In order to obtain the seismic base shear ASCE 7-10 equation 12.8-1 is used and the seismic response coefficient was determined using the ASCE 7-10 equation 12.8-2. The calculations for base shear and forces at each level is shown below in the appendix A.

Table 1: Seismic Loading

Risk Category:	IV	[ASCE 7-10 Table 1.5-1]
I_E:	1.5	[ASCE 7-10 Table 1.5-2]
S_{ds}	0.535	From USGS Seismic Design Report
S_{d1}	0.326	
T_L	12	
C_t	0.028	[ASCE 7-10 Table 12.8-2]
x	0.8	[ASCE 7-10 Table 12.8-2]
h_n	45	
T	0.588	[ASCE 7-10 Eq. 12.8-7]
R	8	[ASCE 7-10 Table 12.2-1]
C_s	0.100	[ASCE 7-10 Eq. 12.8-2]
$C_{s\ max}$	0.104	[ASCE 7-10 Eq. 12.8-3]
$C_{s\ min}$	0.035	[ASCE 7-10 Eq. 12.8-5]
$C_{s\ actual}$	0.100	

Beam Analysis and Design

Building Frame Beam

For the analysis of the beam hand calculations and SkyCiv structural software is used. The beam was analyzed using LRFD load case 2: 1.2D + 1.6L. For the analyses, the beam is assumed to be simply supported and have pin-pin connection condition. The max moment in the beam was obtained from Equation 1 shown below. The live deflection of the beam was limited to L/480. The moment of inertia was calculated using the Equation 2 shown below. Table 3-3 from AISC Steel Manual was used to obtain the size of the beam using the max moment and unbraced length. The beams are analyzed using the SkyCiv software in order to check the accuracy of the hand calculations. Beam were checked for moment, shear, slenderness, and deflection. The sample calculation and design shown below in Appendix 1.

Equation 1: $M_u = \frac{wL^2}{8}$

Equation 2: $I = \frac{wL^4}{129}$

Moment Frame Beam

For the analysis and design of the moment frame beam, the moment frame was analyzed using SkyCiv software. The frame was analyzed separately for dead, live and seismic loading. Using the LRFD load case 6: 1.2D + 1E + 1L the beam was designed for moment, shear and deflection. For the moment frame, beam and column connection was considered to be fixed-fixed connection, and the beam was designed for max moment at the end. The beam size was determined by using ASCE 7-10 Table 3-10. The beam was limited to live deflection of L/480. Using Equation 4 shown below moment of inertia was obtained and compared to the moment of inertia of the beam.

Equation 3: $I = \frac{wL^4}{384\Delta}$

Columns Analysis and Design

Simple Columns

The interior building frame column were considered to be simple pin-pin connected columns. For the analysis of the columns LRFD load case 2: 1.2D + 1.6L governed for gravity loading. The first floor column was designed for the most critical load. Once the max compression load was determined on the column, ASCE 7-10 Table 4-1a was used to obtain he size of the column. The column was designed for an unbraced length of 15 feet, because at each floor the column would be braced by beams. Calculations shown below in Appendix A.

Moment Frame Columns

For the analysis of the moment frame column SkyCiv is used. The column was analyzed for moment, shear and axial loading separately. The LRFD load case 6: 1.2D + 1.0E + 1.0L governed for the design of the column. The column was designed using ASCE 7-10 Table 6-2 for the combined loading. The column is designed with the unbraced length of 15 ft.

Brace

For the analysis of the X-brace system the seismic loading is used over wind loading because the seismic loading governed over wind. The X-bracing is placed on the outer bays in the shorter direction of the building. Braces for each floor were analyzed for seismic loading and sized using the AISC Manuel. For the design, the rupture force governed the size. The sample calculation and design is shown below in the Appendix 1.

Foundation Analysis and Design

Footing Size

For the analysis of the foundation, the size of the footing was determined using gravity loading. LRFD load case 2: 1.2D + 1.6L is used to obtain the compression load on the footing. Using the load case, the total compression load on the footing was determined to be 531 kips. The properties of the soils were obtained from the geotechnical team using the boring logs. The Meyerhof's Equation shown below in the Appendix A. was used to determine the size of the footing. The corner exterior column and the exterior column were also checked for compression load, the compression load on the other column were less than the interior column. For practicality the largest footing size are used.

Rebar Size and Placement

For the analysis of the reinforcement in the concrete footing, the footing is design for interior column and moment frame column. The footing was checked for two-way shear, using ACI 318 Section 22.6.1.2, assuming the footing required no shear reinforcement. The footing was also analyzed for one-way shear, using ACI 318 Section 22.5.5.1. The bearing capacity of the footing was also determined, using ACI 318 Section 21.2.1(a). The rebar was design checking the moment capacity using ACI 318 Section 22.2.2.4.

Connections Analysis and Design

Base Plate and Anchor Bolt

The analysis and the design of the base plate and anchor bolt was determined using the AISC design guide. The base plate is designed for larger moment using AISC design guide second edition Section 3.4. First, the axial load and moment were determined on the plate. A trial plate size was determined keeping the column in mind. The equivalent eccentricity was determined. The inequality was checked using AISC design guild second edition equation 3.4.4. Then the yield length and tensile force in the anchor rod was determined. The base plate thickness is

determined per AISC Equations 3.3.14a, 3.3.14b, or 3.3.15a and b. Then the anchor rod size and length was determined checking for concrete breakout strength.

Simple Beam to Column Connection

For the simple connection, the connection design for shear. An A36 steel is used for the double angle member, and the weld strength is 70ksi. The simple connection is checked for beam web strength, bolt shear, bolt bearing on beam web, bolt tearout on beam web, and available strength at the column flange.

Moment Frame Connection

For the moment frame, the connection is designed welded flange-plated for moment connection (beam-to-column flange). For the web plate was checked for strength of the bolted for web plate and beam web per AISC Specification Section J3.6 and J3.10, shear strength of the web plate per AISC Specification Section J4.2(a), block shear rupture of the web plate per AISC Specification Section J4.3, Table 9-3a, 9-3b and 9-3c, and Equation J4-5. The web plate was also checked for weld shear strength of the web plate to column flange per AISC Manual Equations 8-2a or 8-2b, and column flange rupture strength at welds per AISC Specification Section J4.2(b). The top flange plate was checked for tension yielding per AISC Specification Section J4.1, fillet weld strength for top flange plate to beam flange was checked per AISC Manual Equation 12-1a or 12-1b, connection elements rupture strength at top flange welds using AISC Manual Equation 9-2, and fillet weld at top flange plate was determined using AISC Specification Equation J2-5 and Equation 8-2a or 8-2b The bottom flange plate was designed for compression and checked for flange plate compressive strength, fillet weld strength for per AISC Manual Equation 8-2a or 8-2b, beam bottom flange rupture strength at weld and fillet weld at bottom flange per AISC Specification Equation J2-5.

Regina

The Project was designed to include 200 parking stalls. The typical stall is nine feet wide and twenty feet long, in accordance with the Tulare County standards. Twelve of these stalls meet ADA requirements by being marked as such and incorporating five-foot-wide access aisles next to the stall. Additionally, two of these eleven stalls meet ADA van requirement, as they are eleven feet wide instead of nine. This is more than the minimum of seven ADA stalls for a typical lot of 200 to 300 stalls because this parking structure serves a hospital, so there is an additional ADA requirement. There are two sets of stairs and elevators on the sides of the structure closest to the hospital so that pedestrian transport to and from is most convenient. A dimensioned first floor plan is shown below in Figure 1, and the second floor is shown in Figure 2.

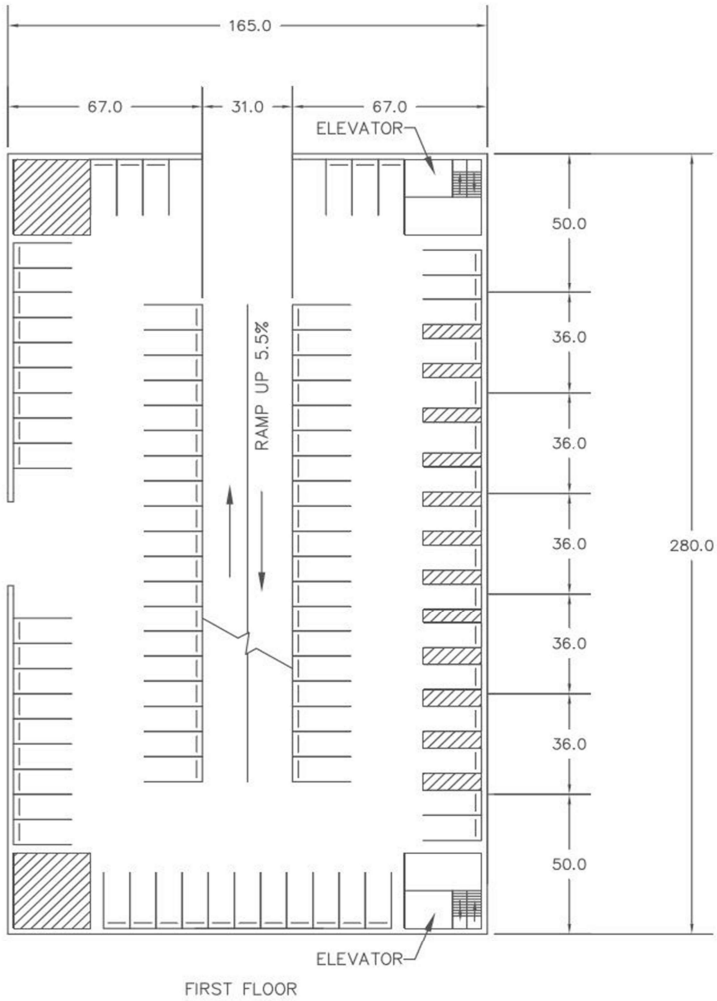


Figure 1: Dimensioned First Floor Plan

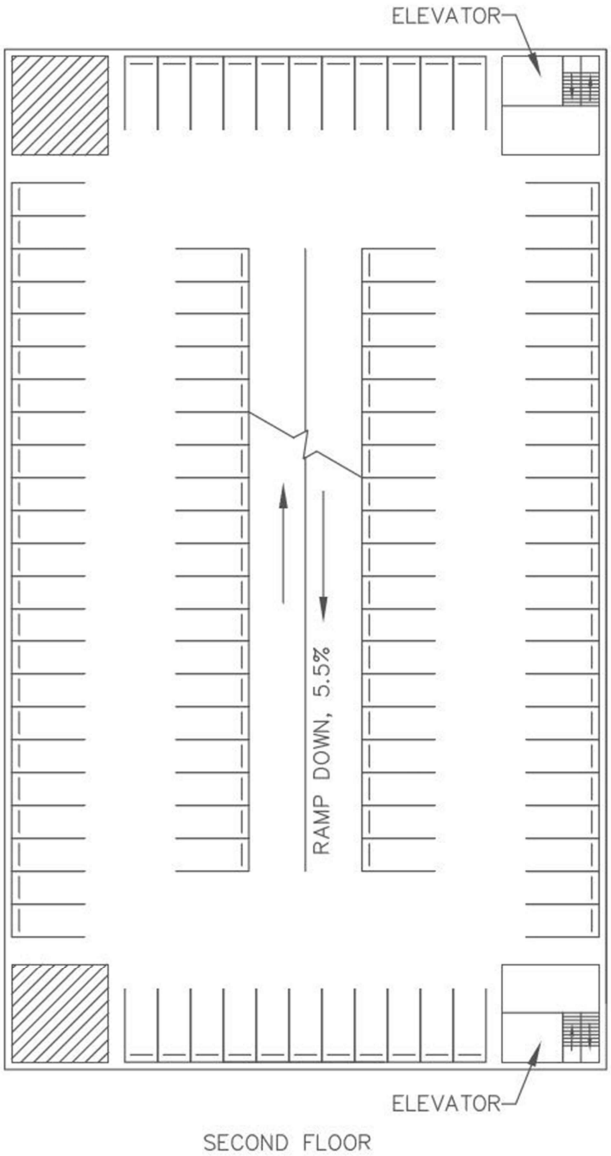


Figure 2: Dimensioned Second Floor Plan

The dimensions shown on the first floor indicate the location and size of the bays within the structure. They are eliminated on the second-floor plan in order to avoid redundancy. An elevation view of the structure is shown below in Figure 2. The vertical scale is exaggerated by a factor of 5 in order to more clearly show the elevation changes

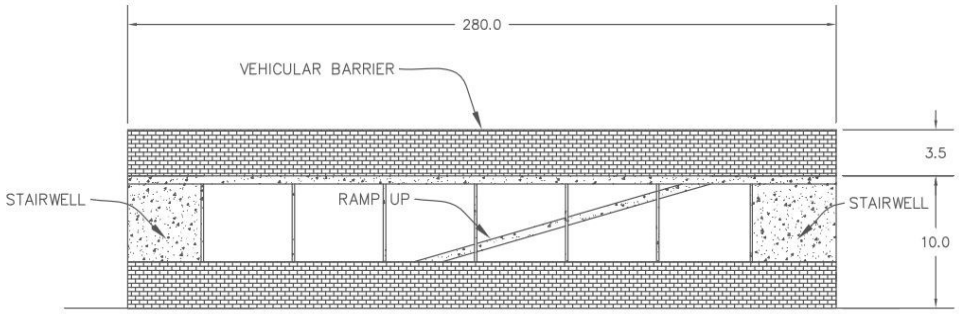


Figure 3: Elevation View

Type Selection

The vertical load resisting system was divided into various types of beams, girders, and columns. Pretopped double tee beams were selected for the beams. A pretopped beam includes a driving wearing surface, eliminating the need for a cast-in-place surface after erection of the structure, while the double tee is an efficient choice in that it serves to act as both a beam and a slab. A combination of L-beams and inverted tee beams were selected for the girders, depending on whether they were supporting a single bay or two bays. The columns were chosen to have a rectangular cross section with the addition of corbels near the top for additional load-bearing capacity.

In total, the structure was designed for five different beams, four different girders, and five different columns.

The lateral force resisting system (LFRS) was selected as special moment-resisting frames (MRF). Shear walls are more typical for the industry in terms of precast structures, but shear walls have a poor track record in terms of seismic resistance, historically speaking. Therefore,

frames were chosen, with the assumption that connections would be designed to allow frame action analysis at a later date.

Design Loads

The loading for each member was determined in accordance with ASCE 7-10. The live load was found to be 40 psf. The only dead load was found to be the self-weight of the members, which varies from member to member. The snow and rain load were both found to be 0. The appropriate load factors were then applied in order to find the critical load combination. The resulting design load for each member is shown below in Table 1. A detailed table of each combination for each member is included in Appendix B

Table 2: Vertical Design Loads

Member ID	Load
B1	166.0 PSF
B2	161.2 PSF
B3	161.2 PSF
B4	161.2 PSF
B5	161.2 PSF
G1	6574 PLF
G2	6411 PLF
G3	12435 PLF
G4	9672 PLF
C1	320.4 K
C2	236.7 K
C3	278.6 K
C4	310.9 K
C5	580.3 K

The lateral forces were calculated separately. It was found that seismic was the controlling lateral force, with a design base shear of 616.5 psf and associated uplift of 9.1 psf. The design forces for the lateral system are shown below in Table 2. Detailed calculations showing wind and seismic

forces are included in Appendix C. USGS design data that was used to find the geologic hazard data, including the design response spectrum, is included as well.

Once the design load was found, shear and moment diagrams were developed for the beams and girders. A selection of representative shear and moment diagrams are included in Appendix D. The design shear, moment, and axial load as appropriate are shown in Table 3 below.

Table 3: Design Shear, Moment, and Axial Load

Member ID	Shear (K)	Moment (K-ft)	Axial Load (K)
B1	67.7	1154	-
B2	54.8	931.7	-
B3	65.5	1114	-
B4	54.8	931.7	-
B5	48.4	725.4	-
G1	118.3	1065	-
G2	160.3	2003	-
G3	310.9	3886	-
G4	290.2	4352	-
C1	-	-	320.4
C2	-	-	236.7
C3	-	-	278.6
C4	-	-	310.9
C5	-	-	580.3

In addition to longitudinal moment, the beams were also designed for transverse moment. This transverse moment comes from an alternative to the 40 psf live load specified in ASCE 7-10. This alternative is a concentrated load of 3000 pounds distributed over an area of 4.5 inches by 4.5 inches; this is the theoretical live load of a tire jack that would be used to change a flat. The worst case location of this concentrated load is split between two double tee beams, because although the load is halved per each beam, the greater moment arm still results in the most moment. The moment diagram of this case is included in Appendix D. The transverse moments are shown in Table 4 below.

Table 4: Transverse Moment

Member ID	Transverse Moment (K-in/ft)
B1	15.4
B2	15.2
B3	15.2
B4	15.2
B5	15.2

After all design loads were determined, the various members were designed.

Beam Design

The beams were designed first, so that the appropriate dead load could be determined and transferred to the girders. A standard cross section chosen by consulting the safe load tables in the PCI design handbook. The sections were then checked for shear, flexure, shrinkage, temperature, and long-term losses, and adjusted accordingly for the design results. All design equations were taken from the ACI code. A spreadsheet of all design calculations, including appropriate equations, is included in Appendix F. A hand calculation is included as well, to verify the accuracy of the spreadsheet.

In order for the beams to connect to the supporting girders and make an efficient use of vertical space, the end of each beam will be dapped. The additional reinforcement used to support the dapping is calculated in Appendix F.

Girder Design

The girders were designed in a theoretically similar fashion to the beams. Differences arose in that the girders are L-beams and inverted tee beams, as opposed to double tee beams. There is also no dapping in the girders. A spreadsheet of all design calculations, including appropriate

equations, is included in Appendix D. A hand calculation was not supplied for the girders, as the design process was theoretically similar to the beam design process.

Column Design

The columns were designed last, so that all appropriate dead load could be considered. The typical cross section was generated by a simple compression check, using the axial load and the specified strength of concrete. In the event of a perfectly concentric load, this simple check would enough to completely design the column. However, because of the eccentric nature of the load, appropriate prestress is needed to counter balance the resulting tensile forces. A spreadsheet of all design calculations, including appropriate equations, is included in Appendix D. A hand calculation is included as well, to verify the accuracy of the spreadsheet.

Lamia

The boring logs and direct shear test reports provided by Salem Engineering were used to obtain the relevant properties of the soil, such as dry density, standard penetration number, moisture content, depth of the soil layers, and soil description. The direct shear test reports were used to determine the cohesion, angle of friction, and effective normal stress of the soil. The data used for the final design include an average dry density of 105 pcf, an angle of friction of 33 degrees, and cohesion of 30 psf. The soil on the site is mostly silty sand with traces of clay. Since the depth of the water table is more than 50 feet below the ground and not provided in the borehole logs, liquefaction did not have to be considered for the design. The borehole data was also used to determine the blow count values from the standard penetration test (SPT). All relevant design parameters are shown in Table 5 below:

Table 5: Soil Properties and Design Parameters

Soil Properties			
	Dry Density (γ_d)	105	pcf
	Friction angle (Φ')	33	degrees
	Friction angle (Φ')	0.5760	radians
	Cohesion (c')	30	psf
Design Parameters			
US Safety hammer	η_H =	60	%
Borehole Diameter = 8 inches	η_B =	1.15	
standard sampler	η_S =	1.0	
Rod length <12 ft	η_R =	0.75	
	$\gamma_{concrete}$ =	150	pcf
	time for settlement =	20	years
	Alpha (α) =	5	
	Pa =	2000	psf

The soil data was used to carry out the calculations for allowable bearing capacity and settlement. The ultimate bearing capacity of the footings were calculated using Meyerhof's bearing capacity equation, obtained from (Das 2016).

$$q_u = c' N_c F_{cs} F_{cd} F_{ci} + q N_q F_{qs} F_{qd} F_{qi} + \frac{1}{2} \gamma B N_\gamma F_{\gamma s} F_{\gamma d} F_{\gamma i} \qquad \textit{Equation 4}$$

The angle of friction (Φ') obtained from the direct shear test report was used to determine the general bearing capacity factors N_c, N_q, and N_γ. These values were used to find the shape, depth, and load inclination factors. The Meyerhof's equation was then used to calculate the ultimate bearing capacity of the footings. The allowable bearing capacity was found by dividing the ultimate bearing capacity by a factor of safety of 3. The allowable load that the footings can safely support was determined by multiplying the allowable bearing capacity in kips per square

foot by the area of the footing. The footing has sufficient bearing capacity if the applied load on the column does not exceed the allowable load that the footing can carry.

The settlement values for each footing were calculated using Schmertmann's method for sandy soil. The N_{60} values for the soil layers were calculated using the field penetration numbers and information from the bore logs were used to determine correction factors for hammer efficiency, borehole diameter, sampler method, and rod length (η_H, η_B, η_S, η_R). The settlement calculations were carried out to determine the settlement of the structure over a span of 20 years.

Schmertmann's strain influence factor method for rectangular footings was used to calculate the settlement. The variation of the strain influence factor with depth was plotted and the values for soil modulus of elasticity (E_s) were determined from the SPT correlation with N_{60}. The standard penetration numbers and modulus of elasticity values are shown in Table 2 under Results and Analysis. The Schmertmann equation for settlement was obtained from (Das 2016) and is reflected below:

$$S_e = C_1 C_2 (\overline{q} - q) \sum_{0}^{z_2} \frac{I_z}{E_s} \Delta z \qquad \textit{Equation 5}$$

Rebars were designed for the footings by applying the standards of ACI 318-14. A concrete compressive strength (*f'c*) of 4000 psi was used and all steel reinforcement will be Grade 60 (f_y = 60 ksi). The factored soil pressure was determined using the column loads provided by the structural engineer. The footing's nominal shear capacity was checked for both two-way shear and one-way shear. The thickness of the base was assumed and using a concrete cover requirement of 3 inches, the effective depth (d) was calculated. Using the applicable equations from ACI 318, the shear strength of concrete was determined and the nominal shear capacity was checked. It was confirmed from the calculations that no shear reinforcement will be required.

Next, the bearing strength of the footing was determined. Reinforcement dowels were designed in order to transfer the load from the base of the column to the footing. The required area of the dowels within the concrete stem was found and the size of the dowels was selected accordingly. The development length of the dowels was determined using ACI 318 codes. The thickness of the footing was checked against the development length, and it was confirmed that the thickness is adequate. The bending moment capacity was determined in both the long and short directions, with the critical section at the face of the column. The moment arm was defined for both directions and the moment capacity equations were used to calculate the required area of reinforcement. The minimum required rebar area specified by ACI 318 was also determined. The number and sizes of rebars were selected for the footing base and the center-to-center spacing between rebars was determined. In the longitudinal direction, the rebars were distributed uniformly across the entire width of the footing. In the lateral (short) direction, the rebars were distributed based on a distribution ratio, so that a certain percentage of the total rebar area was distributed across the center band while the remaining portion was distributed equally on each side of the central band. Lastly, the development length of the rebars in the long direction was checked per ACI codes to determine whether to use hooked or straight bars. Finally, the detailing and cross-sectional plans for the footings were completed using computer software.

The grading plan was created to determine the volume of soil to be cut and the volume of soil to be used as fill in order to design the finished ground surface for the site. Topographic data was obtained from Google Earth to create the existing ground surface and a complete site plan was also developed. To create the grading design, suitable slopes and elevations for the finished ground surface were provided in order to allow for storm water drainage. Tulare County and other typical design standards were consulted in selecting the proper slopes for the ground.

Finished ground elevations were calculated based on these slopes and the existing topographic data. Cut and fill volumes of soil were calculated from the elevations in order to determine the quantity of soil that needs to be imported to the site.

Mario

The hydraulic design of the sanitary sewer system begins with data collection. The data will not only be used to design the pipe size and gradient but also to produce drawings. The hydraulic design of a sanitary sewer involves estimation of waste flow rates for the design data. Sewer pipe capacities are dependent on many factors such as roughness of pipe, maximum allowable depth of flow, and minimum pipe velocities and slope.

The continuity equation and the manning's equation seen below will be used for steady state flow for gravity sewers. Peak hourly flow for the entire service are will be used for the design of new sanitary sewer lines. The hydraulic equation that will be used will be manning equation as seen in equation below. Manning's coefficient "n" will be the friction coefficient and will vary depending on pipe material, size of pipe, and depth of flow. According to Sewer System Master Plan (SSMP) of Tulare County manning's coefficient usually ranges between 0.011-0.017. The pipe type will be designed for concrete with an n-value of 0.013. Flow depth for new trunk sewers the design (d/D) ratio usually ranges from 0.5 to 0.92. Depending on diameter of pipe for sewer lines the diameter will govern the minimum slope of pipe. Next, design velocities and minimum slopes so that the gravity sewer will have a minimum velocity of 2 feet per second (fps) when pipeline is half-full (SSMP Tulare County, 2018). For self-cleaning a velocity of 2 fps will be required. This design will safely carry the design peak discharge so that the suspended material will travel to the main sewer line. Manhole spacing should be where pipes connect and ever 300 feet in length.

Continuity Equation: $Q = V A$

Where: Q = peak flow, cfs

V = velocity, fps

A = cross-sectional area of pipe, sq. ft.

Manning Equation: $$V = \frac{1.486 R^{2/3} S^{1/2}}{n}$$

Where: V = velocity, fps

n = Manning's coefficient of friction

R = hydraulic radius (area divided by wetted perimeter), ft

S = slope of pipe, feet per foot

Deep

Design methodology describes the process to calculate, potable water demand, fire flow demand, pipe size, pump selection, tank volume etc. Following sections give methodology behind design of distribution system. Source for water is City of Tulare, another team member of Static Engineering is responsible for the design of city infrastructure such that minimum flow and pressure requirements are met. In case city infrastructure fails to deliver the required demand in case of equipment failure or emergency, on site storage tank will also be built to hold enough water to meet the demand.

Tables and Figures used for each demand calculation method can be found in Appendix. These tables are used to find maximum base demand. First method is Unit loading method. Value from tables is multiplied by number of patients and staff it will be serving to get demand in gallon/day. Demand is divided by 24*60 to convert the units to gallons per minute. Value for this demand can be found in results section. Second method to calculate demand is by land use method. This

method does not give accurate results since hospital is not directly listed under land use type. Values for Multifamily residential are used because those are the highest values on the table. This method is used to compare the demand obtained from other methods. Third method is calculation of demand based on number of fixtures. Minimum number of fixtures required is obtained from California plumbing code. Unit water supply fixture load for each fixture is multiplied by number of fixtures to get total water supply fixture unit load. Approximate values of demand based on number of fixtures is obtained from **Error! Reference source not found.**. WSFU is used to calculate demand based on total WSFU load.

Fourth method is demand based on number of beds and area of the building in square meter. Total number of beds is multiplied to calculate demand. Area of the building is also multiplied by values to calculate demand. Results from each demand calculation method are summarized in results secotion. Most conservative demand is used for design of pipes as this will account for any uncertainties.

Fire flow is calculated using two different methods. Firstly, California Fire Code (California Fire code, 2016) is used obtain fire flow and duration that flow must be provided. Fire flow depends upon type and square footage of the building. Secondly, fire flow is calculated based on occupancy area, construction class, occupancy, exposure etc. (Walski, et al., 2007) **Error! Reference source not found.**below is used to calculated fire flow demand

Equation 6

$$NFF = 18 * F * A^{0.5} * O * (1 + (X + P))$$

Where,

NFF = needed fire flow (gpm)
F = class of construction coefficient
A = effective are (ft^2)

O = occupancy factor

X = exposure factor

P = communication factor

Following is the methodology used for the selection of components for this system. Methodology for pipe sizing and pump selection is summarized below. Preliminary size of the pipes is calculated by hand and modeling software is then used to further improve the system. Preliminary pipe sizes are calculated using hand calculations. Flow is a product of area and velocity. Flow is known from maxim demand and velocity of 5 feet per second is assumed for initial calculations. Dividing flow with velocity gives required area from which diameter of the pipe is calculated.

Energy is lost in pipes due to the friction. Energy lost is of two types; major loss and minor loss. Major loss is caused by friction between water and surface of the pipe. Minor loss is caused by change in direction of flow or in interruption in flow. Elbows, valves, contractions etc. cause major loss. Total loss in a system is sum of both major loss and minor loss.

Equation 7

$$h_L = \sum h_f + \sum h_m$$

Where,

h_L = Total head loss (ft)

h_f = major loss (ft)

h_m = minor loss (ft)

Major loss is calculated using the following below. The equation is called Hazen-Williams equation and it uses pipe loss C factor. Pipes with higher C factor represents pipes with less friction. (Walski, et al., 2007)

Equation 8

$$h_f = \left(\frac{C_f * L}{C^{1.852} * D^{4.87}}\right) * Q^{1.852}$$

Where,

h_f = major loss (ft)

L = length of pipe (ft)

C = Hazen-Williams C-factor

D = diameter of pipe (ft)

Q = flow rate (cfs)

C_f =unit conversion factor (4.73)

Energy equation is given below. Energy difference between two points is energy input from pump or energy lost from head loss or turbine. This equation is used to calculate energy input by pump or to calculate how much energy is lost.

Equation 9

$$\frac{P_1}{\gamma} + \frac{V_1^2}{2g} + Z_1 + \sum h_p = \frac{P_2}{\gamma} + \frac{V_2^2}{2g} + Z_2 + \sum h_t + \sum h_L$$

Where,

P = pressure (psf)

V = velocity (ft/s)

Z = elevation (ft)

γ = specific weight of water (lb./ ft^3)

g = acceleration (ft/s^2)

h_L = total head loss (ft)

h_p = pump input (ft)

h_t = turbine loss (ft)

Pump is required to pump water from the storage tank in case of fire emergency. Pump must meet certain criteria to be appropriate for this project. Required suction head specified for the pump must be less than available net positive suction head to prevent cavitation. Impeller and casing of the pump must be of stainless steel or other allow that does not rust. Pump must be able to meet the head and demand requirements for this project. Pump shall have minimal maintenance requirements and be able to run continually at peak performance in case of fire event. Engine or motor used to power the pump shall be energy efficient and compliant with air resource board standards to make this system sustainable. Net positive suction head for the pump gives minimum pressure requirements on the suction port of the pump so that water does not vaporize. (Mays, 2000) Vaporization of water on suction side of pump results in cavitation. Cavitation results in noisy operation of the pump and reduced pump performance. Cavitation

also causes premature wear on the impeller of the pump, resulting in shortening the life of the pump. (Burdis) There are two different net positive suction head values for the system; available suction head and required suction head. Available suction head depends upon setup of the system and can be calculated using equation below. Required suction head depends on type of the pump used and it is obtained from pump manufactured. Available suction head the intake port must be higher than required head to prevent cavitation. Available suction head is calculated as shown below. (Mays, 2000)

Equation 10

$$NPSH = h_{atm} + h_s - h_{vp} - h_L$$

Where,

NPSH = Net positive suction head(ft)

h_{atm} = atmospheric pressure(ft)

h_s = static head of water on suction side (ft)

h_{vp} = vapor pressure of water (ft)

h_L = friction losses in suction pipe (ft)

Energy grade line and Hydraulic grade line represent energy of the system. Energy grade line gives pressure head, velocity head, and elevation head at any point in the system. Hydraulic grade line gives pressure head and elevation head only. (Walski, et al., 2007) Velocity head is not part of the hydraulic grade line. Energy grade lines and velocity grade lines can be used to visually represent losses over the length segment of the system. Following equations are used to calculate EGL and HGL. The equations are obtained from Water Distribution Systems Handbook by Larry W. Mays (Mays, 2000)

Equation 11

$$EGL = \frac{P}{\gamma} + \frac{V^2}{2g} + \mathrm{Z}, \qquad HGL = \frac{\mathrm{P}}{\gamma} + \mathrm{Z}$$

Where,

EGL = Energy grade (ft)

HGL = Hydraulic grade (ft)

P = Pressure (psf)

V = Velocity (ft/s)

Z = Elevation (ft)

γ = Specific weight of water (lb./ ft^3)

g = Acceleration (ft/s^2)

Hand calculations are used to get preliminary size of the pipes. To further enhance the system and to understand how the system behaves as demand changes throughout the day, a hydraulic model is created. Hydraulic model simulates the complete behavior of system such as response of each element of the system in case of fire at one specific node. EPANET is the software used for modeling. EPANET is capable of modeling extended period simulation of water distribution system. (EPANET, 2014) EPANET can model a system with pipes, pumps, tanks etc. EPANET works based on principles of conservation of mass and energy. Conservation of mass states that amount of water that enters the pipe equals amount of water that leaves the pipe. This means that water is only used at nodes. Conservation of energy means difference in energy between two points must be same regardless of the path taken. (Walski, et al., 2007) This phenomenon is described by energy equation. To use EPANET modeling, AUTOCAD layout is transferred in the software. Hazen-Williams coefficient is used in the software to calculate head loss. Information such as head loss coefficient, length of each pipe, preliminary size of each pipe etc. The layout for this site consists of 15 pipes, 7 nodes, 8 junctions, 1 pump ,1 storage tank, and 1

reservoir. To model the system, connection to the city is treated as a reservoir with elevation head equal to the required pressure head. Elevation information and water demand is entered at each node. Length, diameter, and headless coefficient are entered for each pipe. Minimum and maximum water levels, diameter of the thank, elevation tank information is added for the storage tank. Three-point pump information is added for the pump curve. Firstly, the model is run for potable water demand to capture behavior of model on daily basis as the demand fluctuates. Secondly, the model is run with fire flow to capture behavior under extreme conditions. Running this model ensures that selected pipe sizes are of accurate diameter to serve the purpose. Layout used in the model is shown below

Antonio

The present water system for the City of Tulare runs on 22 active wells. According to the Tim Doyle, Public Works Water Collection Utility Manager, 8 of the 22 active wells have Variable-Frequency Drives(VFD) to maintain pressure between 50-54 psi. Although this might not always be possible, especially during summer when water demands are much greater. Pressure may drop to around 33 psi.

Currently, the maximum water demand occurs in the summer months, which is not a surprise here in the Central Valley. The Current Average Peak Hour Demand Flow is 17,239 gallons per minute(gpm) and the Max Day Peak Hour Demand is 19,280 gpm. According to the Urban Water Management Plan 2015 for the City of Tulare, the current capacity for the city is 21,119 gpm.

The City of Tulare is currently in construction of two new wells and two new 2MG tanks that will add to the city of capacity and storage for City growth. For this analysis and design, the new wells and tanks are not taken into consideration.

Tulare's Water Distribution system is so large, it would be impossible to analyze a system that massive in the time limited. To reduce the time, it would take to analyze, skeletonizing the system was recommended. Skeletonizing is the process through which only the main service lines of the water distribution system are considered. In my analysis, the only main service lines considered were those pipes lines whose diameters were 10" or larger. This made analyzing the system and making a model simpler.

After the Tulare Water System was skeletonized, it made running a model to run my analysis simpler, but creating the model was still difficult. Using EPANET, a software used to analyze pressurized piping systems, it can help analyze a system and make adjustments to meet the necessary objectives. EPANET requires information about the water system that can obtained from the City of Tulare and its master plans.

EPANET models require physical properties of pipes, water sources, pumps, and water demand loads at the nodes of the system. Without these requirements, the EPANET model cannot be created. For a City's water system to run at an efficient rate, even during the times when the most water is used, this has to be analyzed on the model as well. Needed Fire Flow (NFF) will have to be applied at different areas of the Tulare Water system to provide adequate pressure to areas of need, which have to maintain their water pressures if a fire occurs in the city.

Two EPANET models will be created; One model will contain the current Tulare Water distribution system with all of its components, while the other will be the future water system,

which includes the New Valley Children's Hospital. Once the existing water distribution model is established, the hospital can be implemented and future analysis can begin. With future analysis, comes the design phase of the project.

Water Demands are an essential portion to creating a well-developed water distribution system in the EPANET Model. Water demands coordinate how much loading an area will have. Loading in this case refers to the amount of water people will use in the areas surrounding the points of interest called nodes in EPANET. Different Land uses for the City of Tulare can be seen below in Figure 4.

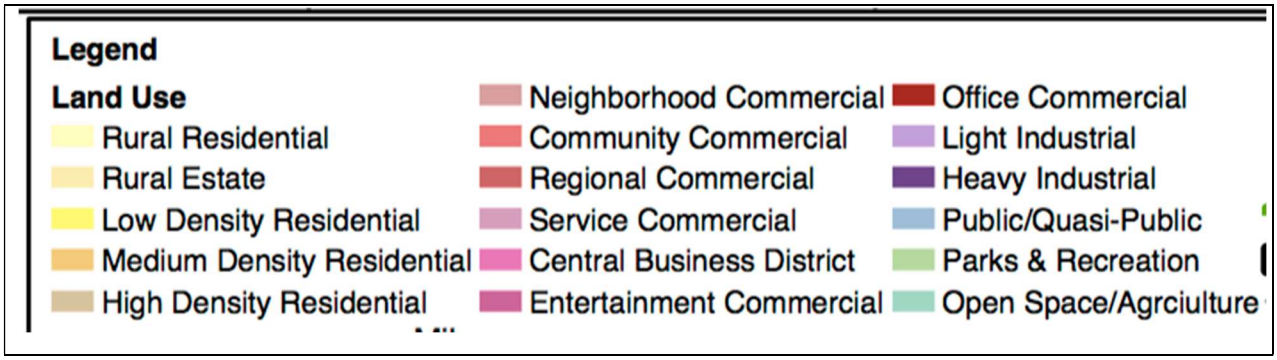


Figure 4: Different Types of Land Use for the City of Tulare

These different land uses different amounts of water per acre. For example, according to the City of Tulare Water Master Plan of 2009, the Rural Residential land areas use about 1,200 gpd/acre, while the High Density Residential land areas use about 4,000 gpd/acre in their respective areas. For the EPANET model, the areas that the nodes include will need to be calculated.

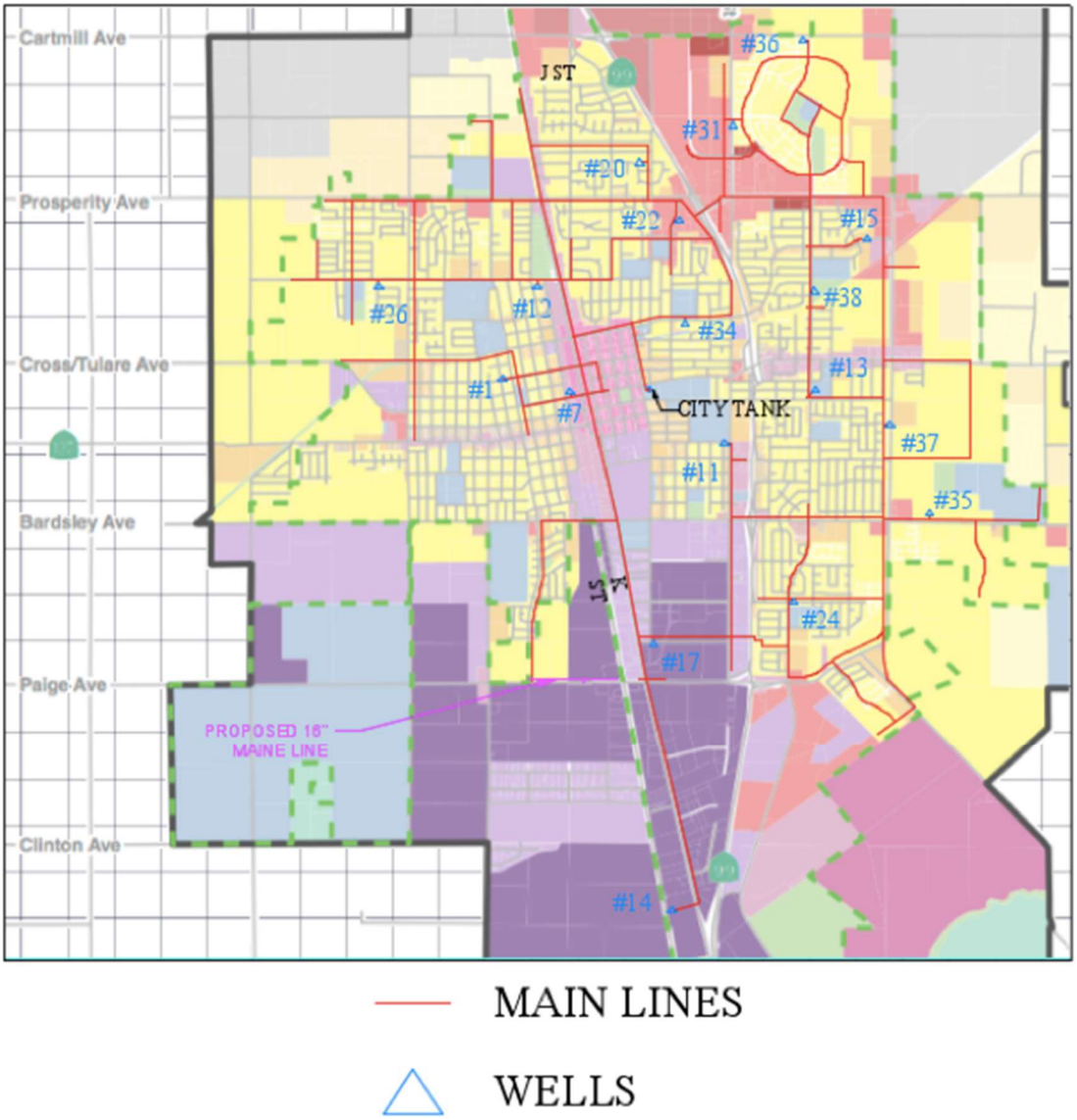


Figure 5: Land Use Map for Tulare

In terms of needed fire flow for the New Valley Children's Hospital, it resides in a commercial area. For the City of Tulare, the commercial areas receive 2,500 gpm for two hours for water distribution analysis. Neighborhood and Industrial areas receive 1,500 and 3,500 gpm, respectively, for the duration of two hours as well. When fire flows are active, the pressure of any point in the system is not to drop below 20 psi.

After the models are created and analysis of the future water distribution system has been complete, design of the main water service lines outside of the hospital begin. This includes any of the existing pipes that have to replaced and improved. The hospital may affect the pressure and velocity around the area, so the correct pipe material and size will have to be adequate for this system. Pipe material, which dictates the roughness of the pipe, pipe velocity, pipe pressure, node pressure, and headloss throughout the pipe are the key factors that contribute to this design.

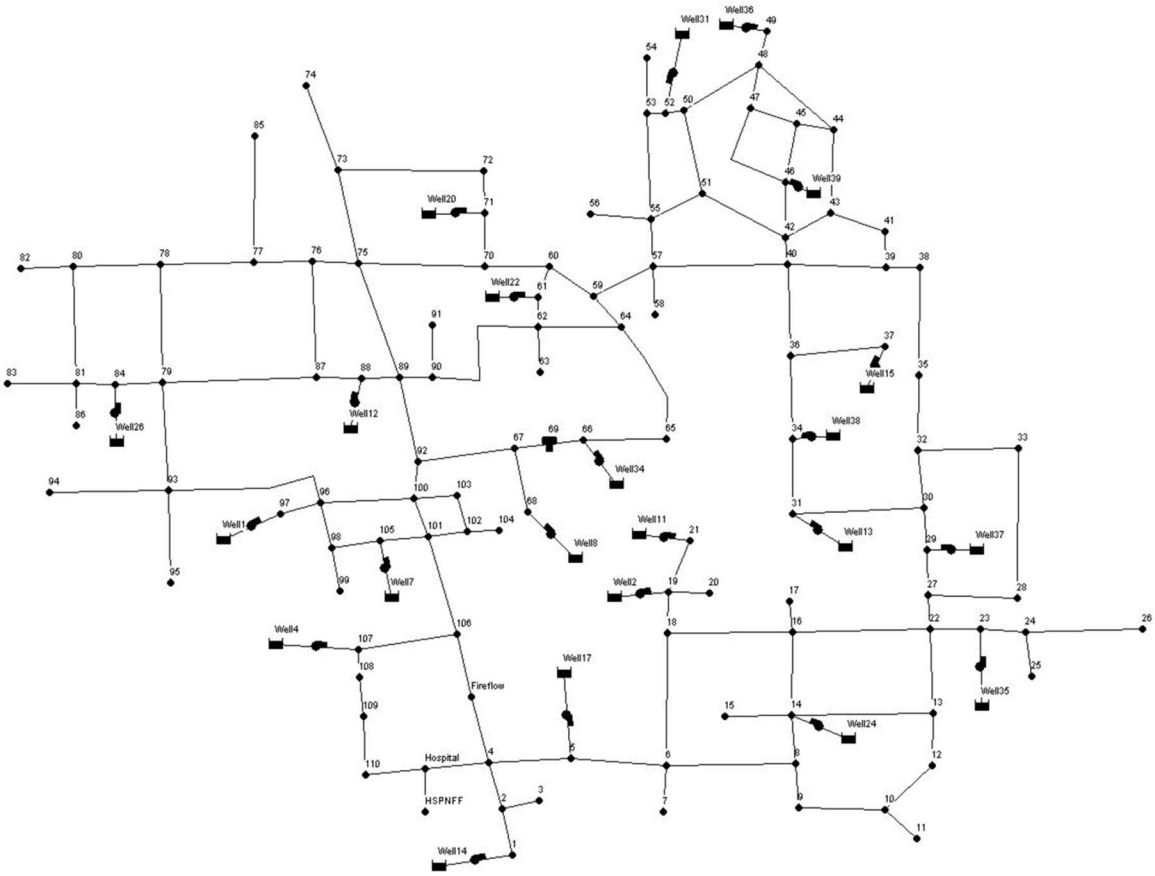


Figure 6: EPANET Model

The design phase will consider the connection of the existing water distribution system to the New Valley Children's Hospital and other improvements that system may need to have an adequate water distribution system throughout the city. There are two nearby connections to the east and west of the project site, both about a quarter-mile from the site. Both service lines are

12” pipelines, so the proposed connection will be a 12” mainline going across Paige Avenue, north of the site. This will connect the two existing 12” mainlines together with the hospital connection in between as shown in Figure # below.

Figure 7: Proposed Connection from Hospital to Tulare

Regulations for the design of the water service lines were based on City standards. Many of the standards for the City of Tulare refer to the American Water Works Association Regulations (AWWA). For main service lines, according to AWWA M-32, the recommended pressure in the

pipes is between 40-80 psi, for Maximum Day Demand(MDD) and Peak Hour Demand(PHD). Also, pressure will not fall below 20 psi for any node of interest during fire flow times. Pressure zones have to be taken into consideration in most cities due to the difference in elevation. Because the City of Tulare has no significant difference in elevation, Tulare does not have multiple pressure zones.

Velocities for the main lines in the City of Tulare should not exceed 8 ft/s in accordance to AWWA M-32. While 8 ft/s the standard, usually 7 ft/s is recommended due to velocities impacting the amount of headloss throughout the system. In terms of headloss, if velocity and pressure meet the specified requirements, headloss should not be a major factor due to the low impact. But if the headloss exceeds 10 ft per 1,000 ft, changes will have to be made to the service pipe.

Nadun

The design of the stormwater system had to follow the city of Tulare standards and specifications. The catchment area was divided into six sub basins.

The rainfall data was collected through the city of Tulare masterplan. The following table 6 provides the reference for the rainfall data. The pre and post development runoffs will be compared in the report. The pre development analysis is the analysis done before the project completion and post development analysis is the analysis after completion of the project. The detention pond has to be able to retain the excess runoff from post development runoff.

Table 6: Depth-Frequency-Intensity data

	5-min	10-min	15-min	30-min	1-hr	2-hr	3-hr	6-hr	12-hr	24-hr	48-hr
2-year	0.16	0.21	0.25	0.34	0.45	0.55	0.62	0.76	0.92	1.13	1.38
5-year	0.23	0.31	0.36	0.48	0.64	0.79	0.89	1.09	1.34	1.65	2.03
10-year	0.31	0.4	0.46	0.6	0.77	0.95	1.07	1.32	1.63	2.01	2.48
25-year	0.39	0.5	0.57	0.73	0.93	1.15	1.3	1.61	1.99	2.47	3.05
50-year	0.46	0.58	0.66	0.83	1.05	1.3	1.47	1.83	2.26	2.8	3.47
100-year	0.53	0.66	0.75	0.93	1.16	1.44	1.63	2.03	2.52	3.13	3.88

By using the data from Table 6, the IDF curves for 10-year storm event were developed as follows in figure 14.

Pre Development Analysis

The rainfall data for analysis was collected from Tulare county masterplan. The IDF curve was developed using those data provided in table 1. The return period was selected to be 10 years. The pre development runoff coefficient was determined to be 0.3 according to the city of Tulare master plan. The total area of the planned site was 39.06 acres. The site had silty sand soil. The land had an effective percent imperviousness of 95% and non-effective percent imperviousness of 0%.

The rational method was used to determine the predevelopment runoff. The rational equation that was used to calculate the runoff is as follows,

$Q = CIA$; Equation 12

Q = peak flow (cfs)

I = rainfall intensity (in/hr)

A = area (acres)

Post development analysis

The post development analysis was completed with the runoff coefficients provided by the city of Tulare masterplan. The total area was divided into six sub basins. The rational method was used to estimate the runoff from each sub basin. Then the overall runoff was calculated. The following table shows the areas of the sub basins. The figure 8 below shows the sketch of the divided sub basins. The following Table 7 provides the sub-basin number and its total area. For a

design return period of 10-years, the city of Tulare master plan did not require using a weighted runoff coefficient for the design.

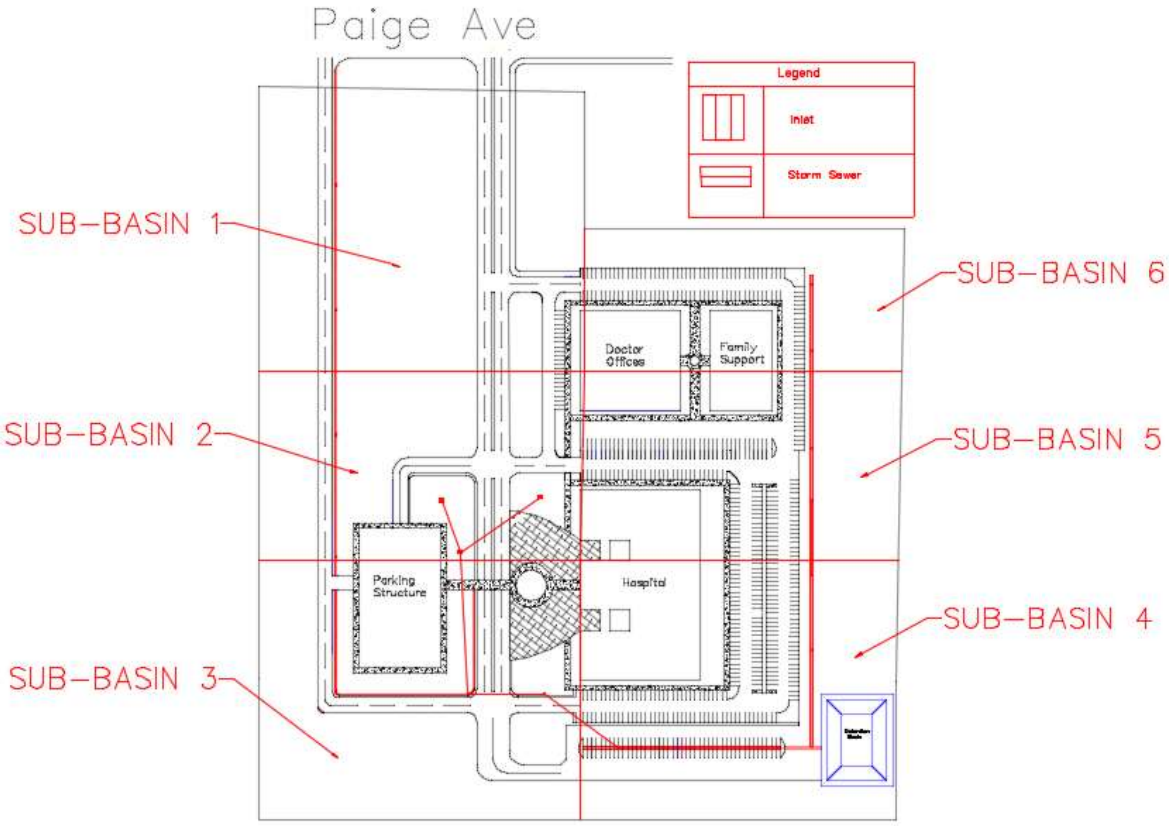


Figure 8: Sub-Basin Locations

Table 7: Sub-Basin Data

Sub basin number	Area (acre)
Sub basin 1	8.48
Sub basin 2	5.6
Sub basin 3	7.68
Sub basin 4	7.57
Sub basin 5	5.54
Sub basin 6	4.2

Since the site area is considered a community commercial area, a runoff coefficient value of 0.8 was selected according to the city of Tulare master plan.

The same equation 12 mentioned above was used to determine the post development runoff of the site. But for the post development analysis the time of concentration was considered for each sub-basin and at the overall outflow location.

Stormwater collection system

The locations of inlets were determined using the grading plan and there are 33 inlets divided among the six sub basins. The stormwater collection system design includes the calculation of flow to the inlets. The airport equation method was used to calculate the time of concentration of stormwater to the inlets through gutter and overland flow. Using the time of concentration from the airport method, the flow to the inlets was calculated. The flow to the inlets was calculated using the rational method.

Airport formula that was used to find the time of concentration of overland flow. The formula is as follows,

$T_0 = \frac{0.395(1.1-C)*\sqrt{L_0}}{S^{0.33}{}_0}$; Equation 13

T_0 = on land flow time (min)

L_0 = on land flow length (ft)

S_0 = slope ft/ft

C = runoff coefficient

The other variant that was used to find the gutter flow time of concentration is as follows,

$T_S = \frac{L_S}{60*V_S}$;Equation 14

Where,

L_S = flow length (ft)

V_S = conveyance coefficient multiplied by the square root of slope of channel

The Figure 9 below shows the locations of the planned inlets that are spread out through the site.

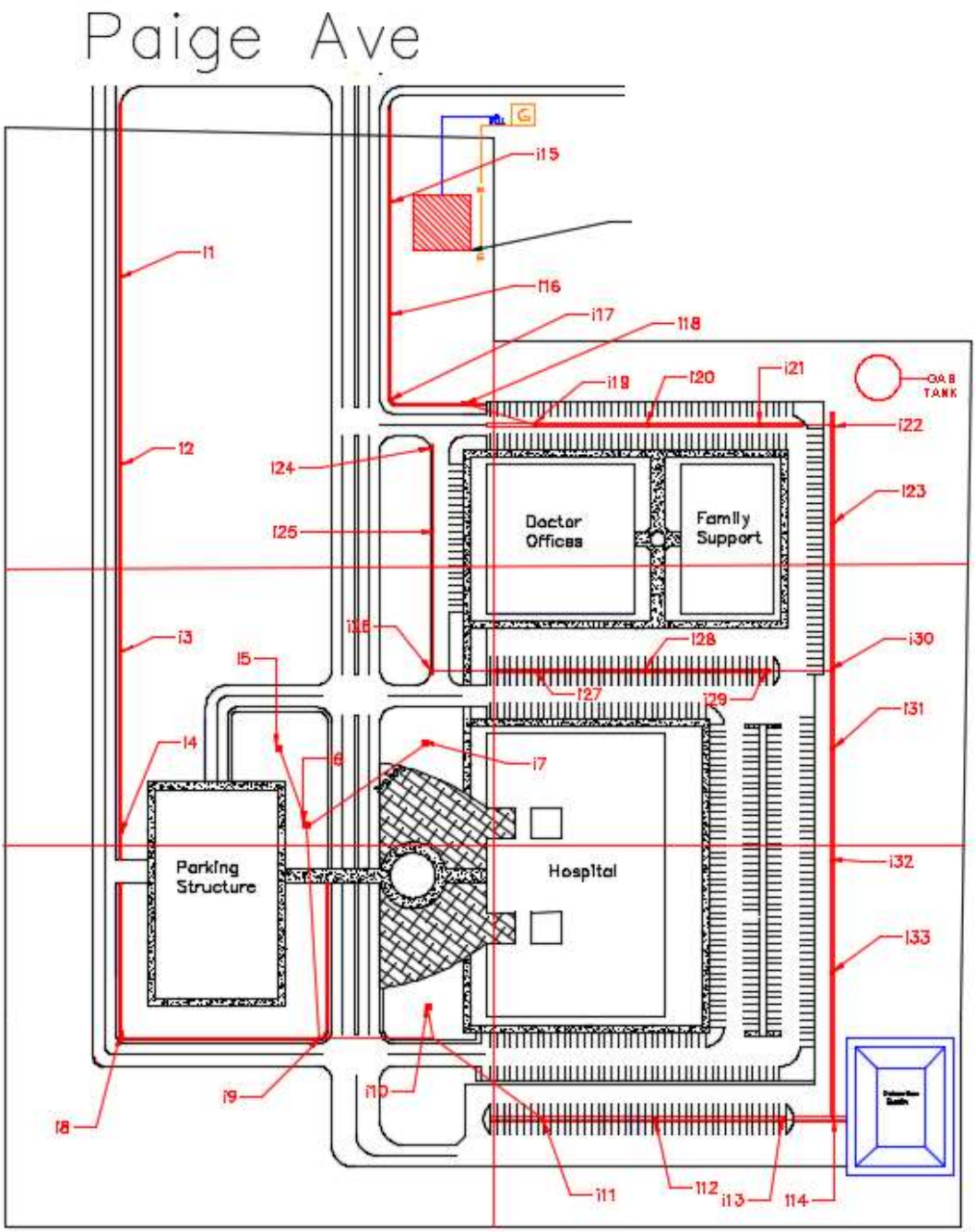


Figure 9: Locations of Inlets

Stormwater conveyance facility

The third design component of the stormwater design is the conveyance facility. The design of the stormwater conveyance facility consists of designing the storm sewer. The adequate selection of pipe size, material and the slope is vital for the function of the stormwater system. The following figure shows the plan for the stormwater sewer. The pipe cover will be based on the size of the pipe.

The following Figure 10 shows an example of a size of the pipe and the ground cover that was required for it.

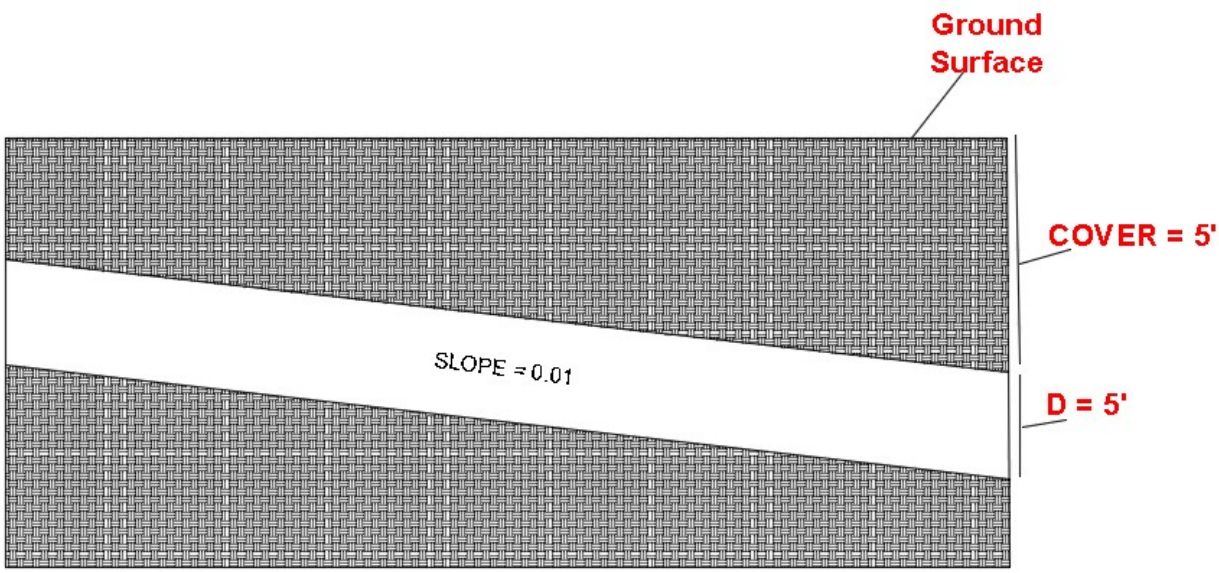


Figure 10: Pipe Layout Plan

$Q_p = \frac{k}{n} R^{\frac{2}{3}} S_0^{\frac{1}{2}} A;$ Equation 15

Where,

Q_p = the peak flow in pipe (cfs)

k = conversion factor (1.49 for U.S units)

n = manning's roughness coefficient

S_0 = channel slope (foot/foot)

R = hydraulic radius (feet)

A = Cross sectional area (square feet)

Just full flow conditions were assumed for the storm sewer. The following figure 11 shows the circular channel ratio graph that was used to calculate the A/A_{full} value that was required to determine the flow velocity and time of concentration in pipe.

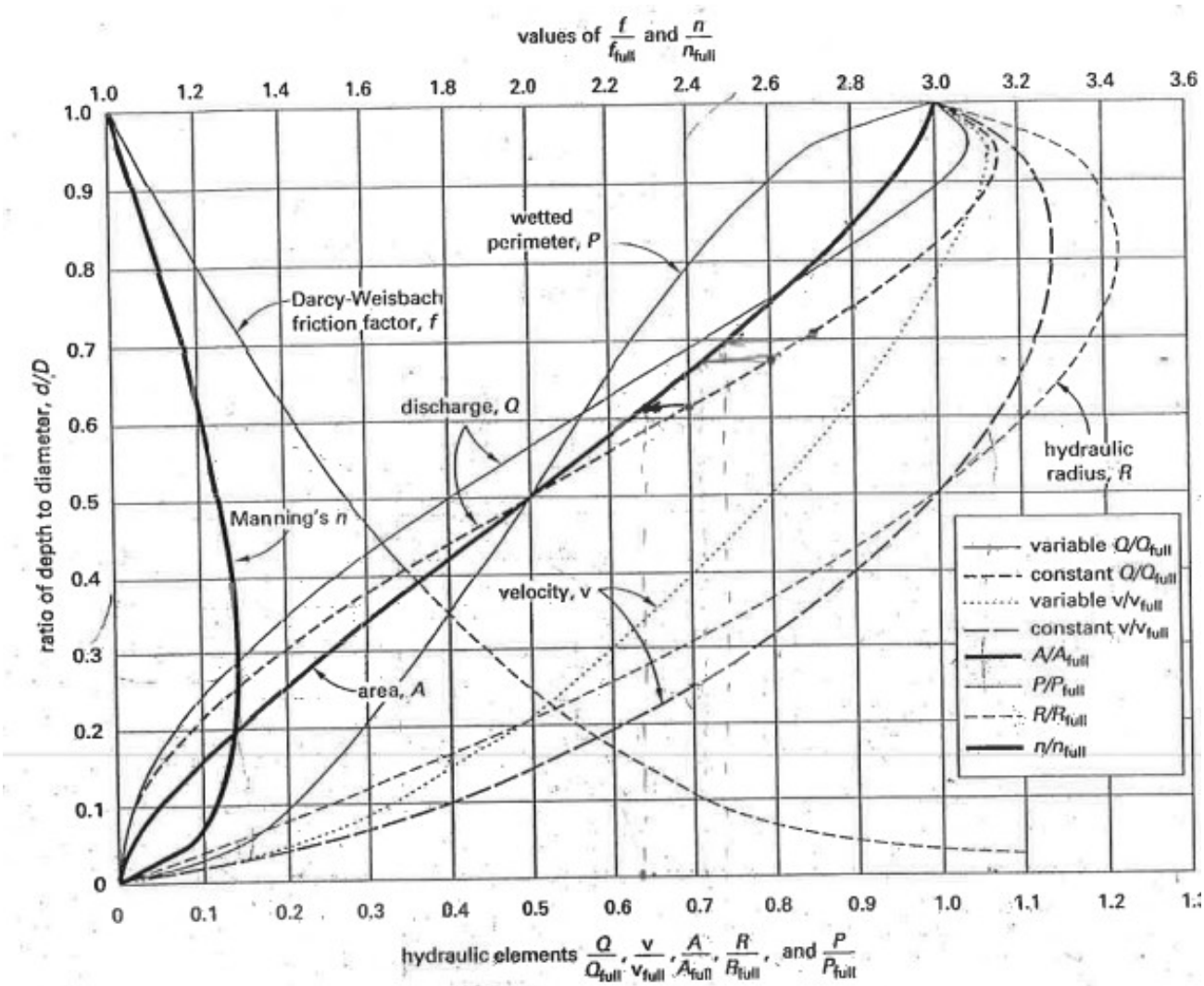


Figure 11: Circular Channel Ratios

An energy analysis (EGL check) was conducted using the elevation changes throughout the pipes until detention pond. The elevation data that was used to determine the potential energy and the kinetic energy is provided in the Table 8 below.

Table 8: Elevation Data used for Calculating the EGL

Pipes	Length (ft)	D (in)	Slope (ft/ft)	GSE U/S (ft)	GSE D/S (ft)	Elevation Crown U/S (ft)	Elevation Crown D/S (ft)	Elevation Invert U/S (ft)	Elevation Invert D/S (ft)
1-2	250	21	0.01	4973.8	4971.3	4967.8	4965.3	4966.05	4963.55
2-3	246	24	0.01	4973.25	4970.79	4967.25	4964.79	4965.25	4962.79
3-4	240	27	0.01	4972.75	4970.35	4966.75	4964.35	4964.5	4962.1
4-8	270	30	0.01	4970.8	4968.1	4964.8	4962.1	4962.3	4959.6
8-9	257	36	0.01	4974.5	4971.93	4968.5	4965.93	4965.5	4962.93
5-6	108	21	0.01	4943.8	4942.72	4937.8	4936.72	4936.05	4934.97
7-6	190	21	0.01	4973.7	4971.8	4967.7	4965.8	4965.95	4964.05
6-9	287	30	0.01	4972.8	4969.93	4966.8	4963.93	4964.3	4961.43
9-10	145	48	0.01	4972.3	4970.85	4966.3	4964.85	4962.3	4960.85
10-11	182	48	0.01	4969.9	4968.08	4963.9	4962.08	4959.9	4958.08
11-12	150	54	0.01	4968.5	4967	4962.5	4961	4958	4956.5
12-13	170	54	0.01	4972.85	4971.15	4966.85	4965.15	4962.35	4960.65
13-14	171	54	0.01	4970.8	4969.09	4964.8	4963.09	4960.3	4958.59
14-Pond	17	60	0.01	4971.2	4971.03	4965.2	4965.03	4960.2	4960.03

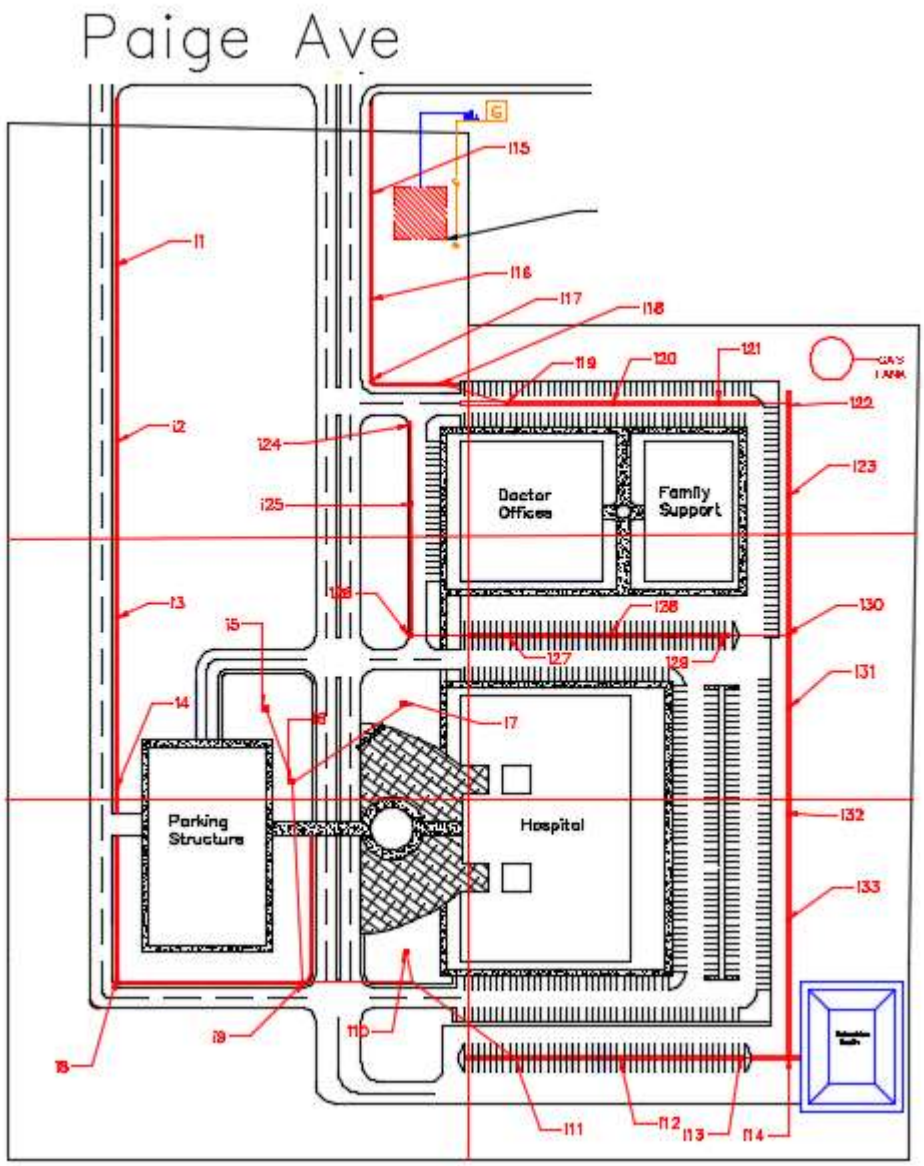


Figure 12: Location of Planned Storm Sewer Pipes and Detention Pond

Stormwater storage facility

The detention pond falls under the storage facility. Since area of the site is less than 150 acres, the volumetric method will be used to determine the dimensions of the pond. The volumetric method is based on rational method. The maximum storage is the main objective to be found when designing the storage facility. The storage is based on inflow and outflow to the pond. The detention pond should be able to hold the runoff from a 10-year storm without flooding the premises. The planned dimensions for the detention pond is provided on Figure 13 below.

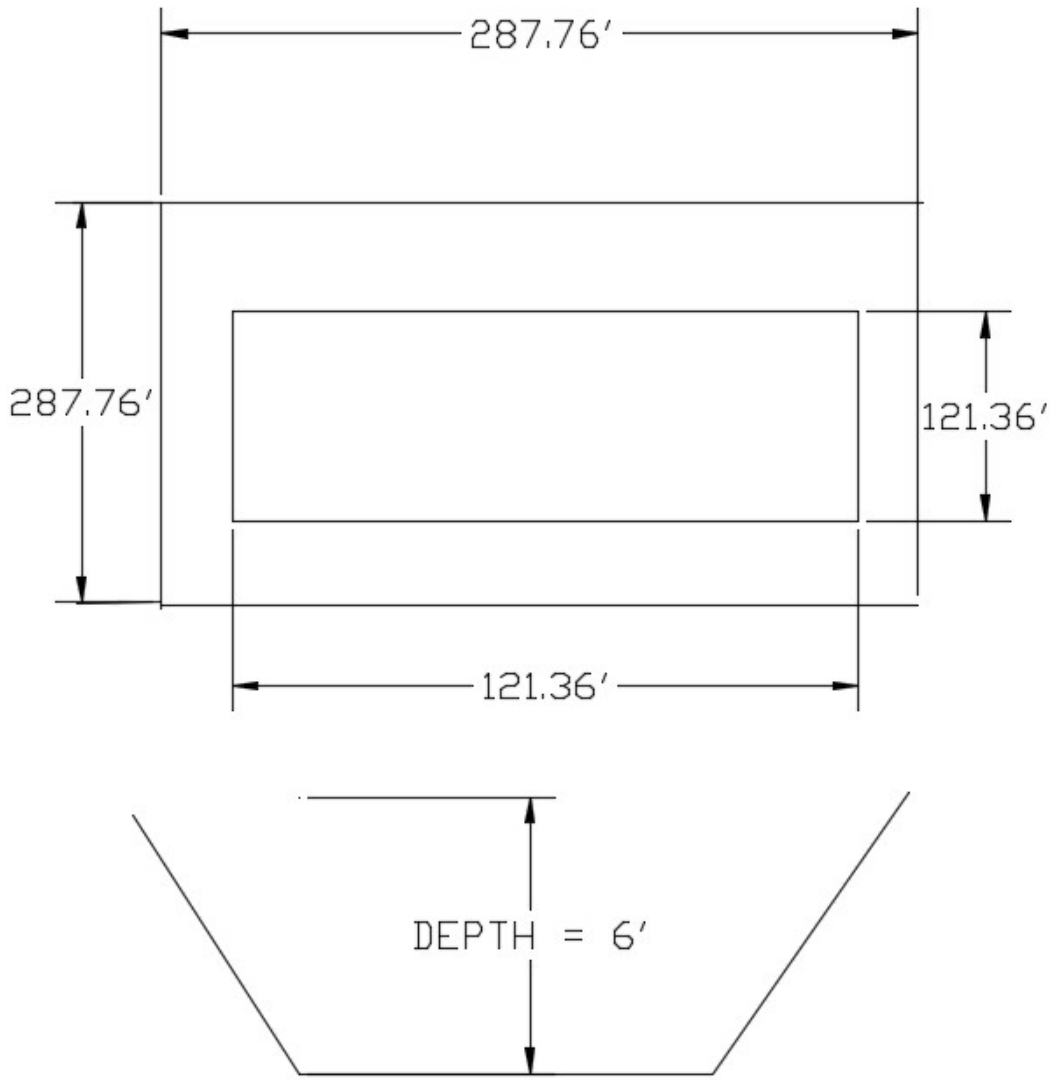


Figure 13: Plan for the Detention Pond

The Id value for determining the maximum storage was determined from the following equation. The total area of 39.07 acres, runoff coefficient of 0.8, calculated concentration time of 9 minutes and an average flow of 89.58 cfs were used for the calculations.

$I_d = \frac{74.1}{(10+T_d)^{0.786}}$; Equation 16

Where,

Td = detention time (min)

Id = intensity (in/hr)

Bagazi

The student will start the design of the transportation parking lot design based on the specification of either the city or standards for hospitals. Prior to designing any transportation structure, a demand estimation is made which helps in designing the parking lot and on estimating the increase in volume that will occur on the roads. The parking space capacity of garage will be calculated based of the number of beds for the patients. According to the city of Tulare "For hospitals and institutions at least one (1) parking space for every two (2) beds provided in said buildings." For the faculty there will be one space per 3 employees. So, based on that the student will create the equations to find out the number of spaces that needed to be install.

For the patients there will be about 190 beds.

Parking space for patients= # of beds /2 Equation 17

Parking space for faculty= # of employees/3 Equation 18

Based on that equations the total number of the parking garage that need to be installed is about 95 parking space and that is consider to be the minimum of parking space the design can have. The number of faculties parking space is calculated to be around at least 630 parking space. The

site can carry way more than the number calculated. Also, the accessible parking space is considered in this project to be calculated. The engineer has decided to use 10% of the total number of the parking space that has been calculated.

ADA parking space= 2% * Total number of parking spaces Equation 19

The total number of ADA parking space is calculated to be 15 parking space for the entire hospital with consideration of the dimensions of the ADA parking standards. Also, there will be a 10% of solar-powered electric vehicle charging station(SCS) from the total parking space installed in different locations in the parking lot and parking garage.

SCS = 5% * Total number of parking spaces Equation 20

The total number of the solar-powered electric vehicle charging station parking is calculated to be 36 parking space.

Table 9: Minimum Number of Accessible Parking Spaces (2010 Standards (208.2)

Total Number of Parking Spaces Provided in Parking Facility (per facility)	(Column A) Minimum Number of Accessible Parking Spaces (car and van)	Minimum Number of Van-Accessible Parking Spaces (1 of six accessible spaces)
1 to 25	1	1
26 to 50	2	1
51 to 75	3	1
76 to 100	4	1
101 to 150	5	1
151 to 200	6	1
201 to 300	7	2
301 to 400	8	2
401 to 500	9	2
500 to 1000	2% of total parking provided	1/6 of Column A*

	in each lot or structure	
1001 and over	20 plus 1 for each 100 over 1000	1/6 of Column A*
***One out of every 6 accessible spaces**		

CHAPTER 3: RESULTS AND ANALYSIS

Jaskaran

Beam

Table 10: Roof Beam Results

Roof Beam	Size
J1	W8x48
J2	W8x28
G1	W12x120
G2	W12x96
G3	W18x192

Table 11: Floor Beam Results

Floor Beams	Size
J1	W10x60
J2	W10x33
G1	W14x176
G2	W14x145
G3	W18x192

Column

Table 12: Column Results

Column	Size
C1	W12x65
C2	W12x50
C3	W24x162

Brace

Table 13: Brace Size and Location

Brace Size and Location	Demand (kip)	Capacity(kip)
First Floor HSS10x6x1/2	419	439
Second Floor HSS8x4x5/8	304	382
Third Floor HSS3x2x5/16	65.5	76.6

Foundation

Table 14: Foundation Size and Column

Foundation Size	Demand (kips)	Bearing Capacity(kips)
9x9 Interior Column C1	531	694

Rebar

Table 15: Rebar Size and Location

Rebar Location	# of Rebar	Size of Rebar
Moment Frame Column at the Bottom, Each Way	15	#10
Simple Column At the Bottom, Each Way	6	#9

Connections

Moment Frame Connection

Table 16: Moment Frame Connection

	Type of Steel	Size	Connection to beam	Size	Connection to Column	Size
Web Plate	A36	3/8”x8”x15”	Bolt	1(1/8)” dia. bolt	Weld	¼”
Top Flange Plate	A36	3”x10”x24”	Weld	1”	Weld	1.25”
Bottom Flange Plate	A36	2”14”x24”	Weld	1”	Weld	14/16”

Simple Connection

Table 17: Simple Connection

	Type of Steel	Size	Connection to Beam	Size	Connection to Column	Size
Web Plate	A36	(2) L4x3½”x 1/2”	Weld	Weld A, 5/16”, 70Ksi	Bolt	(3) 7/8”

Base Plate

Table 18: Base Plate and Anchor Rod

	Type Of steel	Size	Thickness
Base Plate	A36	32”x20”	4”
Anchor Rod	A36	3” dia.	18” embedment

<u>Regina</u>

After design was completed, the type selection of each member was either confirmed or adjusted and redesigned. The final results for each member are shown below in Table 4.

Table 19: Final Member Design

Member ID	Type	Width (LF)	Length (LF)	Strands
B1	12DT34	12	68	168-S
B2	10DT34	10	68	148-S
B3	10DT34	10	68	188-D1
B4	10DT34	10	68	128-S
B5	10DT34	10	60	128-S
G1	26LB36	1.5	36	168-S
G2	26LB36	1.5	50	248-S
G3	26IT44	3	50	168-S
G4	41IT32	3	60	308-S
C1	Square	1.5	10	128-S
C2	Square	2	10	128-S
C3	Square	2.5	10	168-S
C4	Square	2	10	148-S
C5	Square	1	7	168-S

Plans were developed for the Project. The plans can be found in Appendix E.

Specifications were developed for the Project. The specs were based off of the PCI Guide Specifications for structural precast concrete and adjusted as appropriate for the project. These

specs are preliminary and may change as the project moves forward and design continues, particularly that of connections. The specs can be found in Appendix E.

A cost estimate was developed based on the sections chosen, and the cost of the structure was found to be approximately $1,507,000. The cost estimate is preliminary and may change as the project moves forward. It can be found in Appendix F.

Table 20: Roof Beam Results

Roof Beam	Size
J1	W8x48
J2	W8x28
G1	W12x120
G2	W12x96

Table 21: Floor Beam Results

Floor Beams	Size
J1	W10x60
J2	W10x33
G1	W14x176
G2	W14x145

Table 22: Brace Size and Location

Brace Size and Location	Demand (kip)	Capacity(kip)
First Floor HSS10x6x1/2	419	439
Second Floor HSS8x4x5/8	304	382
Third Floor HSS3x2x5/16	65.5	76.6

Table 23: Foundation Size and Column

Foundation Size	Demand (kips)	Bearing Capacity(kips)
9x9 Interior Column C1	531	694
7x7 Exterior Column C2	319	391
6X6 Exterior Column C3	159	188

Lamia

Table 24 below shows the data obtained from the boring log and the calculated values of N_{60} and soil modulus of elasticity (Es) for each layer of soil.

Table 24: Interpreted Borehole Data for Soil Layers

Layer No.	Depth (ft)	Dry Density (pcf)	Blow Count (N)	N_{60}	Alpha	Pa (psf)	Es (psf)
1	8	105	28	24.15	5	2000	241,500
2	12.5	101.2	35	30.1875	5	2000	301,875
3	17.5		16	13.8	5	2000	138,000
4	23		7	6.0375	5	2000	60,375
5	33		9	7.7625	5	2000	77,625
6	38		18	15.525	5	2000	155,250
7	47		18	15.525	5	2000	155,250
8	50		30	25.875	5	2000	258,750

Three sets of rectangular spread footings were designed to support the appropriate column loads. The footing sizes and the column loads applied to each are shown in the table below. All footings will be placed at a depth of 5 ft below the ground surface.

Table 25: Spread Footing Sizes

Width (ft)	Length (ft)	$Q_{applied}$ (kips)
12	24	435.6
6	10	181
8	12	244.5

Table 26 shows the calculated bearing capacities and allowable loads for each of the footings. The allowable bearing capacity values were obtained using a factor of safety of 3. Since the applied column loads are much less than the allowable loads that the footings can carry, all three sets of footings meet the requirement for bearing capacity.

Table 26: Bearing Capacity Results

Footing: 12' X 24'		
$Q_{applied}$	435.6	kips
B	12	ft
L	24	ft
Area	288	ft^2
Depth (Df)	5	ft
q	525	psf
c'	30	psf
γ	105	pcf
Nc	38.64	
Nq	26.09	
Nγ	35.19	
Fcs	1.34	
Fqs	1.32	
Fγs	0.8	
Fcd	1.117	
Fqd	1.112	
Fγd	1	
Fci	1	
Fqi	1	
Fγi	1	
qu	39648.14	psf
$q_{(all)}$	13216.05	psf
F.S.	3	
Q_{all}	3806.22	kips

Footing: 6' X 10'		
$Q_{applied}$	181	kips
B	6	ft
L	10	ft
Area	60	ft^2
Depth (Df)	5	ft
q	525	psf
c'	30	psf
γ	105	pcf
Nc	38.64	
Nq	26.09	
Nγ	35.19	
Fcs	1.405	
Fqs	1.39	
Fγs	0.76	
Fcd	1.23	
Fqd	1.22	
Fγd	1	
Fci	1	
Fqi	1	
Fγi	1	
qu	33739.57	psf
$q_{(all)}$	11246.52	psf
F.S.	3.0	
Q_{all}	674.79	kips

Footing: 8' X 12'		
$Q_{applied}$	244.5	kips
B	8	ft
L	12	ft
Area	96	ft^2
Depth (Df)	5	ft
q	525	psf
c'	30	psf
γ	105	pcf
Nc	38.64	
Nq	26.09	
Nγ	35.19	
Fcs	1.45	
Fqs	1.433	
Fγs	0.733	
Fcd	1.175	
Fqd	1.168	
Fγd	1	
Fci	1	
Fqi	1	
Fγi	1	
qu	35744.75	psf
$q_{(all)}$	11914.92	psf
F.S.	3.0	
Q_{all}	1143.83	kips

Table 27 shows the settlement results obtained using the Schmertmann method for all footing sizes. The individual settlements of the footings over a 20-year period were less than 2 inches and the differential settlements between adjacent footings were less than 1 inch. Therefore, the settlement will be fairly uniform throughout the structure, and the footings were concluded to be safe from excessive settlement.

Table 27: Settlement Results

Footing Size	6' x 10'	8' x 12'	12' x 24'	
P (applied load)	181	244.5	435.6	kips
Width	6	8	12	ft
Length	10	12	24	ft
Depth (D_f)	5	5	5	ft
Area	60	96	288	ft^2
W_f	45	72	216	kips
q	525	525	525	psf
q bar	3767	3297	2263	psf
C_1	0.919	0.905	0.849	
C_2	1.46	1.46	1.46	
Settlement (S_e)	1.08	1.36	1.71	inch

Microsoft Excel was used to calculate the settlement and bearing capacity values, and sample hand calculations are provided in the appendix. AutoCAD was used to create a foundation plan showing the layout of the footings for the parking structure. Cross-sectional plans and reinforcement details were created using MicroStation. All plans and construction drawings can be found in Appendix A.

The rebar design calculations were carried out for each set of footings. For the 6 ft x 10 ft footings, four number 5 dowel bars will be used for the concrete stem. Six number 6 steel rebars will be placed in the longitudinal direction in the footing. In the lateral direction, six number 7 bars will be placed across the center 5 ft band, and two number 5 rebars will be placed on each side of the central band. The 8 ft x 12 ft footings will consist of four number 5 dowels in the stem and seven number 7 bars in the footing's long direction. Seven number 7 bars will also be used across the center 6 ft band in the lateral direction, with two number 5 rebars on each side. The 12 ft x 24 ft footings will consist of four number 6 dowels within the stem and thirteen number 9 rebars in the longitudinal direction. It will also consist of eleven number 8 rebars across the center 12 ft band and four number 7 bars on each side of the central band in the short direction.

The clear cover of the rebars in the footings will be 3 inches. The 6x10 and 8x12 footings will be 20 inches thick while the 12x24 footing will be 24 inches thick. Since the provided development length is sufficient for all the footings, the rebars will not need to be hooked.

The grading plan is provided in Appendix A and shows the cross-slopes and spot elevations that will make up the finished ground surface. From the topographic data, it was found that the existing ground at the location of the site is relatively flat. To determine the cut and fill volumes, the site was divided into rectangular sections with the designations shown in Table 28 below. Using the average end area method, the cut and fill volumes were calculated from the area of each section and the distances between each area, as shown in the table. The total volume of cut was found to be 269,716 cubic yards and the total volume of fill was found to be 333,679 cubic yards, which is a difference of 63,963 cubic yards. 31,706 cubic yards of soil can be obtained from the excavated soil for the detention pond, which results in 32,257 cubic yards of soil that needs to be imported from off-site.

Table 28: Cut and Fill Calculations

FILL				
Designation	**Area**	**Distance**	**Volume**	
	SFT	**FT**	**CFT**	
F1	4,322.6			
F2	4,221.0	100.0	427,180	
F3	4,127.1	100.0	417,405	
F4	8,370.3	140.7	879,192	
F5	7,071.4	112.7	870,140	
F6	7,482.9	32.5	236,507	
F7	7,447.7	114.1	851,791	
F8	6,348.6	125.9	868,477	
F9	6,440.5	56.0	358,095	
F10	6,933.6	64.0	427,971	
F11	6,798.5	85.0	583,614	
F12	6,607.4	150.0	1,005,443	
F13	6,430.6	150.0	977,850	
F14	6,458.9	35.0	225,566	
F15	4,035.1	116.0	608,652	
F16	4,356.0	64.7	271,452	
		TOTAL	9,009,335	CFT
			333,679	**CY**

CUT				
Designation	**Area**	**Distance**	**Volume**	
	SFT	**FT**	**CFT**	
C1	3,959.9			
C2	3,913.2	100.0	393,655	
C3	3,892.1	100.0	390,265	
C4	7,829.0	140.7	824,581	
C5	7,191.8	112.7	846,423	
C6	6,956.6	32.5	229,912	
C7	5,907.8	114.1	733,914	
C8	5,597.8	125.9	724,278	
C9	5,507.4	56.0	310,946	
C10	5,332.7	64.0	346,883	
C11	5,100.8	85.0	443,424	
C12	4,290.8	150.0	704,370	
C13	3,576.1	150.0	590,018	
C14	3,560.0	35.0	124,882	
C15	3,436.7	116.0	405,809	
C16	3,146.8	64.7	212,976	
		TOTAL	7,282,334	CFT
			269,716	**CY**

Mario

The discharge was calculated by taking 80% of the potable water demand. An excel spreadsheet was set up using manning's equation. The diameter of the pipe and its corresponding slope was adjusted until the desired discharge was met. Minimum slope and maximum pipe to depth ratios were taken into consideration. The figure below shows pipe size, pipe length, and the slope for the sanitary sewer system.

Table 29: Sanitary Sewer Pipe Characteristics

Sanitary Sewer Design							
Pipe	Pipe Size (in) "D"	Flow Depth (in) "d"	Pipe Length (ft) "L"	Flow Rate (cfs) "Q"	Slope "S"	n-value	Velocity (ft^2/s) "v"
DO-MH7	4	2	30	0.087	0.0084	0.013	2.00
FS-MH7	4	2	30	0.087	0.0084	0.013	2.00
MH7-MH6	5	2.5	210	0.174	0.0102	0.013	2.56
MH6-MH5	5	2.5	75	0.174	0.0102	0.013	2.56
HP-MH5	15	7.5	50	1.39	0.00185	0.013	2.26
MH5-MH4	15	7.5	310	1.56	0.00185	0.013	2.54
MH4-MH3	10	5	350	1.56	0.00185	0.013	2.54
MH3-MH2	10	5	240	1.56	0.00185	0.013	2.54
MH2-MH1	10	5	243	1.56	0.00185	0.013	2.54

Nadun

Intensity data and IDF curves

The following table 30 provides the calculated intensity. The rainfall data was collected from the city of Tulare masterplan. The following figure 8 provides the IDF curves for 5,10 and 100 years developed from the calculated intensities.

Table 30: Intensity data for 10-year event

Time (min)	Time (hr)	Rainfall (in)	Intensity (in/hr)
5	0.08	0.31	3.72
10	0.17	0.4	2.4
15	0.25	0.46	1.84
30	0.5	0.6	1.2
60	1	0.77	0.77
120	2	0.95	0.475
180	3	1.07	0.36
360	6	1.32	0.22
720	12	1.63	0.14
1440	24	2.01	0.08
2880	48	2.48	0.05

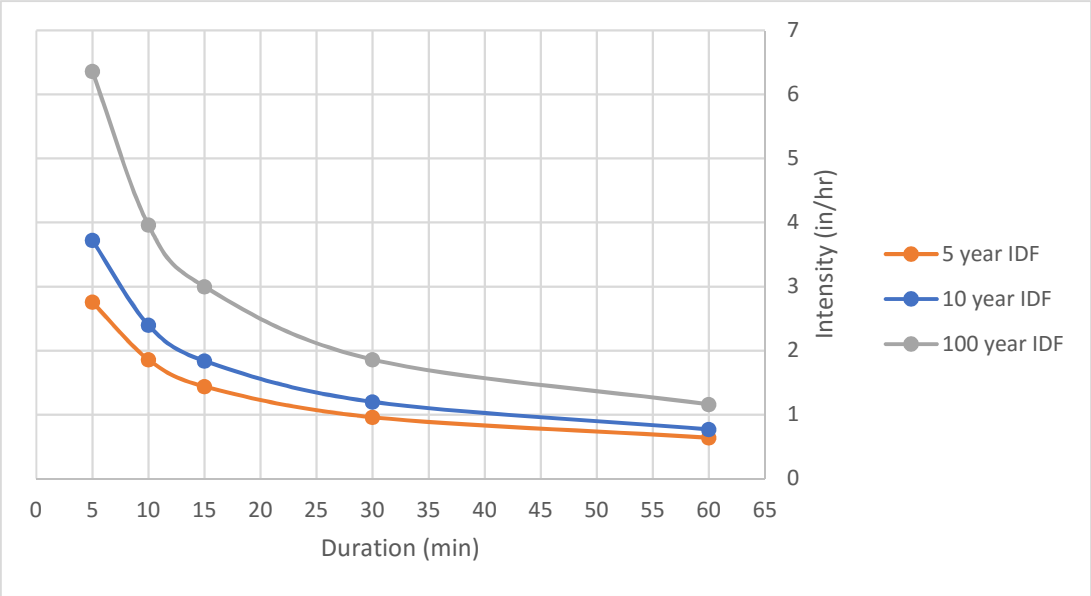


Figure 14: IDF curves for 5, 10 & 100 year return periods

Comparison between pre and post development analysis

The following table 31 provides the comparison between runoff volumes, runoff coefficients and peak runoff values for pre and post development analysis.

Table 31: Comparison between pre and post development

	Runoff coefficient (%)	Peak overland runoff (cfs)
Pre development	30%	5.56
Post development	80%	41.26

Pipe sizing and EGL

The pipe sizes that is required to carry the stormwater was in diameters between 21 inches and 60 inches. The most economical option was to use concrete pipes. A manning's N value of 0.013

was used for the design. A slope of 0.01 was chosen for the storm sewer and it met the design standards to carry the full flow without interruptions. A total of 34 pipes were required to carry the stormwater form inlets to the detention pond.

The following table 32 provides the calculations for determining the pipe size and the table 33 provides the energy analysis of the pipes.

Table 32: Pipe sizes chosen

Pipe #	Length (ft)	Slope	n	Qp (cfs)	Td (min)	d (ft)	Pick (')	Qfull	Q/Qfull	A/Afull	A	Vs (ft/s)	ts (min)
1-2	250	0.01	0.013	11.19	19.40	1.54	1.75	15.84	0.71	0.66	1.59	7.05	0.59
2-3	246	0.01	0.013	22.12	20.5	1.98	2	22.63	0.98	0.95	2.98	7.41	0.55
3-4	240	0.01	0.013	29.02	22.1	2.20	2.25	30.99	0.94	0.88	3.50	8.29	0.48
4-8	270	0.01	0.013	40.80	10.8	2.50	2.5	41.06	0.99	1	4.91	8.31	0.54
8-9	257	0.01	0.013	54.50	11.5	2.78	3	66.81	0.82	0.77	5.44	10.01	0.43
5-6	108	0.01	0.013	12.28	8.7	1.59	1.75	15.84	0.77	0.72	1.73	7.09	0.25
7-6	190	0.01	0.013	12.54	8.5	1.60	1.75	15.84	0.79	0.65	1.56	8.02	0.39
6-9	287	0.01	0.013	34.77	7.8	2.35	2.5	41.06	0.85	0.78	3.83	9.08	0.53
9-10	145	0.01	0.013	105.49	20.1	3.56	4	144.03	0.73	0.69	8.67	12.17	0.20
10-11	182	0.01	0.013	128.02	9.1	3.83	4	144.03	0.89	0.83	10.43	12.27	0.25
11-12	150	0.01	0.013	150.55	6.0	4.07	4.5	197.26	0.76	0.71	11.29	13.33	0.19
12-13	170	0.01	0.013	164.84	7.7	4.21	4.5	197.26	0.84	0.76	12.09	13.64	0.21
13-14	171	0.01	0.013	184.82	7.3	4.40	4.5	197.26	0.94	0.88	14.00	13.21	0.22
14-Pond	17	0.01	0.013	207.35	5.0	4.59	5	261.34	0.79	0.65	12.76	16.25	0.02
15-16	150	0.01	0.013	12.28	15.8	1.59	1.75	15.84	0.77	0.72	1.73	7.09	0.35
16-17	121	0.01	0.013	24.29	17.0	2.05	2.25	30.99	0.78	0.73	2.90	8.37	0.24
17-18	90	0.01	0.013	36.23	17.0	2.39	2.5	41.06	0.88	0.82	4.03	9.00	0.17
18-19	103	0.01	0.013	50.20	13.1	2.70	3	66.81	0.75	0.7	4.95	10.15	0.17
19-20	147	0.01	0.013	62.70	5.2	2.93	3	66.81	0.94	0.88	6.22	10.08	0.24
20-21	150	0.01	0.013	74.66	5.83	3.13	3.5	100.84	0.74	0.7	6.73	11.09	0.23
21-22	94	0.01	0.013	86.59	5.88	3.31	3.5	100.84	0.86	0.81	7.79	11.11	0.14
22-23	130	0.01	0.013	98.35	5.98	3.47	3.5	100.84	0.98	0.95	9.14	10.76	0.20
23-30	197	0.01	0.013	110.85	7.54	3.63	4	144.03	0.77	0.72	9.05	12.25	0.27
24-25	111	0.01	0.013	17.16	9.55	1.80	2	22.63	0.76	0.71	2.23	7.69	0.24
25-26	190	0.01	0.013	31.95	12.11	2.28	2.5	41.06	0.78	0.73	3.58	8.92	0.36
26-27	138	0.01	0.013	41.50	12.80	2.51	3	66.81	0.62	0.6	4.24	9.78	0.24
27-28	150	0.01	0.013	57.98	5.24	2.85	3	66.81	0.87	0.84	5.94	9.77	0.26
28-29	158	0.01	0.013	72.70	6.78	3.10	3.5	100.84	0.72	0.65	6.25	11.62	0.23
29-30	83	0.01	0.013	83.24	10.30	3.26	3.5	100.84	0.83	0.76	7.31	11.38	0.12
30-31	103	0.01	0.013	209.87	9.86	4.61	5	261.34	0.80	0.66	12.96	16.19	0.11
31-32	150	0.01	0.013	225.87	7.62	4.74	5	261.34	0.86	0.81	15.90	14.20	0.18
32-33	150	0.01	0.013	242.34	8.95	4.87	5	261.34	0.93	0.89	17.48	13.87	0.18
33-Pond	200	0.01	0.013	258.94	8.88	4.99	5	261.34	0.99	1	19.63	13.19	0.25

The results of the EGL check are provided in the table 33 below. It provides the calculated potential and kinetic energy that and the total energy change in the pipes.

Table 33: Energy change in pipes

Pipes	ΔPE (ft)	ΔKE (ft)	ΔE (ft)
1-2	2.5	0.018	2.518
2-3	2.46	0.288	2.748
3-4	2.4	0.071	2.471
4-8	2.7	0.081	2.781
8-9	2.57	0.213	2.783
5-6	1.08	0.180	1.260
7-6	1.9	0.181	2.081
6-9	2.87	0.163	3.033
9-10	1.45	0.049	1.499
10-11	1.82	0.304	2.124
11-12	1.5	0.383	1.883
12-13	1.7	0.243	1.943
13-14	1.71	0.772	2.482
14-Pond	0.17	0.721	0.891

Detention pond parameters

The following table 34 provides the dimensions of the detention pond. The detention pond needs to carry the total runoff that is created by the development.

Table 34: Detention Pond Dimensions

Design Storm	Si (ft3)	L1 (ft)	L2 (ft)	B1(ft)	B2(ft)
100-year	127876.1	121.36	287.76	121.36	287.76

Deep

Demand/Load

Water demand is calculated using 6 different methods. These methods are based on no. of people served, area served, no. of beds, WSFU, and method outlined in the California pluming code.

Table 35 summarizes the demands obtained using each method. Lowest value obtained is with area method and highest value obtained is with California Plumbing code. Each hospital room has one toilet and shower. In California Plumbing code method calculation, it is assumed that both toilet and shower are used simultaneously which gives the highest demand. Area method is not accurate at all in this case as Hospital is not an option on the values provided. Most conservative value is used to obtain the results for calculations purposes. Demand calculated using no. of people served gives results pertaining to this project and this method is commonly used in the industry. Base demand of 400 gpm for potable water flow is used to design the water distribution system for the hospital. Fire demand is calculated using **Error! Reference source not found.**and the values obtained are summarized in Table 36 below. Calculated fire flow is 2267 gpm. **Error! Reference source not found.** gives minimum fire flow requirements set by California Fire code. California Fire code and city of Tulare requires fire flow of 2500 gpm for 2 hours. Storage tank volume and distribution mains are designed for fire flow of 2500 gpm for the duration of 2 hours. Irrigation demand is 84 gpm and calculations in Appendix B shows calculations for irrigation demand. Required water for irrigation will be delivered in 8 hours at night as peak demand and stress on distribution system is low at that time. One main distribution line is designed for irrigation system. Irrigation efficiency of 80 % and

ET of 0.26 is used to calculate irrigation demands. It is recommended to use drought tolerant plants to make the hospital sustainable.

Table 35: Potable Water Demand

	No. of people served	Area method	Design Load	Approximation Method	Claifornia Plumbing Code	No. of beds
Building	(gpm)	(gpm)	(gpm)	(gpm)	(gpm)	(gpm)
Hospital Wing 1	110.1	7.6	260.0	360.0	345.0	57.0
Hospital Wing 2	110.1	7.6	110.0	150.0	125.0	25.4
Hospital Wing 3	110.1	7.6	185.0	290.0	260.0	57.0
Doctors office	33.0	4.1	98.0	50.0	65.0	194.7
Family support	33.0	2.6	110.0	50.0	110.0	97.4
	396.2	29.4	763.0	900.0	905.0	431.5

Table 36: Fire flow and Irrigation Demand

Fire demand		
City Requirement	2500	gpm
Calculated	2267	gpm
Calculated (rounded)	2500	gpm
Irrigation demand		
Grass/Trees	84	gpm

Grading

Grading plan for the site is shown in **Error! Reference source not found.**. Lowest elevation for the site is 258 ft. and highest elevation on the site is 260 ft. City of Tulare requires minimum clear cover of 4 ft. below fine grade. Minimum cover of 5 ft. is used to lower the impact of loading on pipelines and to be below freezing zone as a safety measure. Elevation of the pipes range from 253 to 255 ft. below ground.

Hydraulic Results

Initial velocity of 5 ft. per second is used is to get preliminary pipe design. System is optimized using EPANET to reduce friction loss. Hydraulics results are summarized in

Appendix C. System is modeled. Velocity is under 5 ft./s during 24-hour simulation and pressure is above minimum residual requirements set by the city. Sections below further discuss the design and hydraulics of the system.

Pipe Sizing

Pipes are sized to keep the flow under 5 ft/s. Pipe sizes range from 3-inch to 10-inch max. Main pipeline that runs from city point of connecting to the storage tank is 10-inch.

Table 37 gives the length and sizes of required main pipes. These sizes are obtained upon optimization. Flow and headless through each pipe during daily use and fire flow can be obtained in Appendix C. Pipes connecting to Family Support and Doctor's office are 4-inch. Pipe going to parking structure for fire suppression is 6-inch. Pipe material is C-900 PVC.

Table 37: Pipe Size

Pipes				
	Length	Start Elevation	End Elevation	Diameter
Pipe	ft	ft	ft	inch
P-1	100	258	256.5	10
P-2	45	257.5	256.5	10
P-3	45	257.5	256.5	10
P-4	640	257.5	254.35	10
P-5	140	255.35	254.25	10
P-6	75	255.25	254	10
P-7	155	255	253.5	10
P-8	130	254.5	253.35	10
P-9	230	254.35	254.25	10
P-10	380	255.25	253.5	10
P-11	210	255.35	254.15	8
P-12	200	255.25	254	6
P-13	200	254.5	254	6
P-14	200	254.35	254	6
P-15	75	255	253.75	6
P-16	30	255.15	254	4
P-17	260	255.15	253.75	8
P-18	30	254.75	254.26	4
P-19	130	254.75	255.75	8
P-20	380	256.75	255.15	8
P-21	130	255	255	8
P-22	700	255	255	8
P-23	30	255	255	6
P-24	30	255	255	6
P-25	30	255	255	6
P-26	20	255.15	255.15	10
P-27	200	255	255	3

Pump Selection

Pump used to boost pressure is Berkley B5Z9.5BH. Pump curve for this pump is shown in **Error! Reference source not found.** below. Pump used is closed coupled mount drive. Maximum RPM for this pump is 1800 rpm. Maximum working pressure for this pump is 175 psi. Figure 10 shows the pump curve obtained from Berkley pumps and Figure 11 shows the pump curve used in EPANET model. Suction side of pump is 5 inch and discharge size is 6”. Diameter of the impeller is 9.5”. Pump produces no flow at 95 feet of head and 1700 gpm at 45 ft. of head.

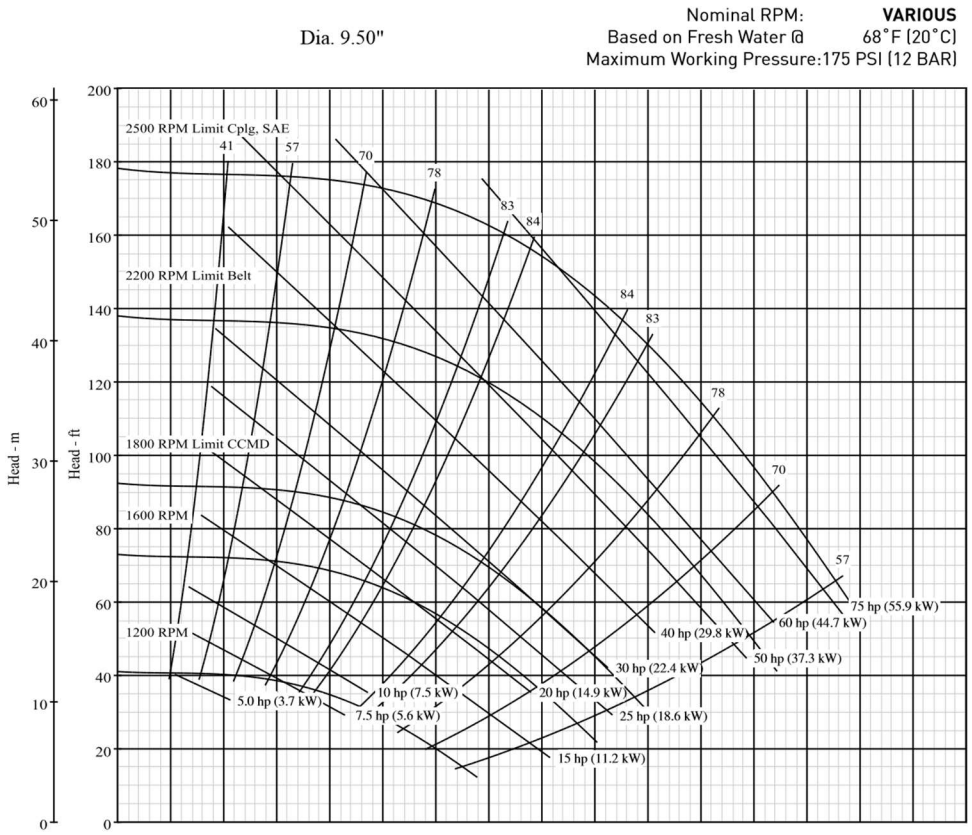


Figure 15: Pump Curve

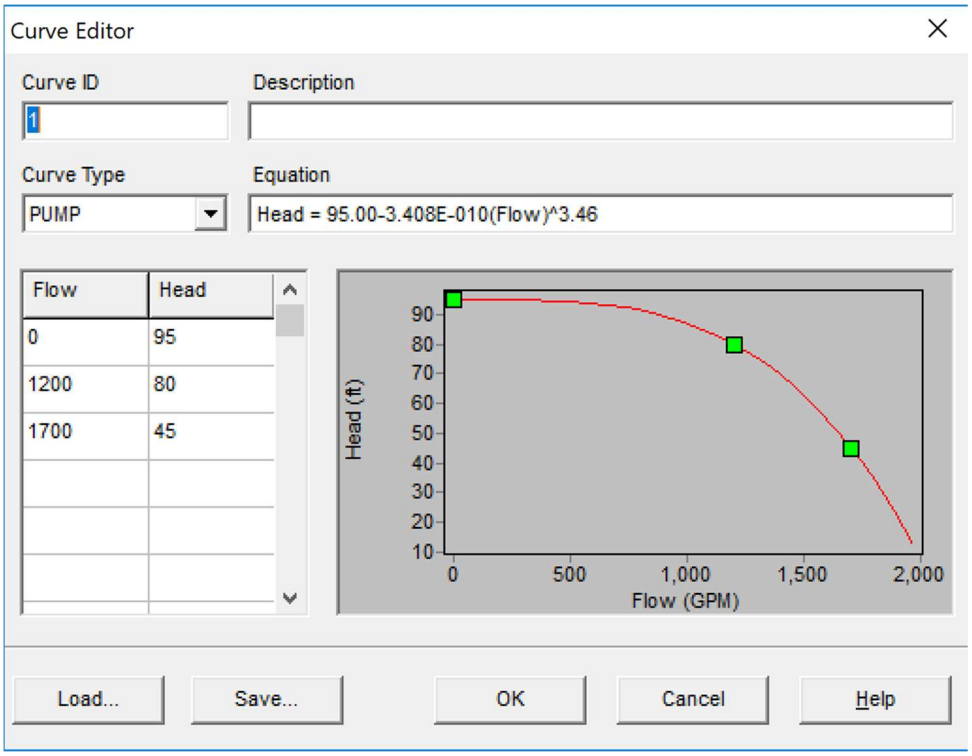


Figure 16: Pump curve- EPANET

NPSH

Figure 17 gives required net positive suction head for this pump. For flow of 1800 rpm at max speed the required head is 10 feet. Available suction head is 140 ft which makes this pump acceptable for this application. This is true as this pump will not be lifting water from the ground. The pump is used to boost water pressure

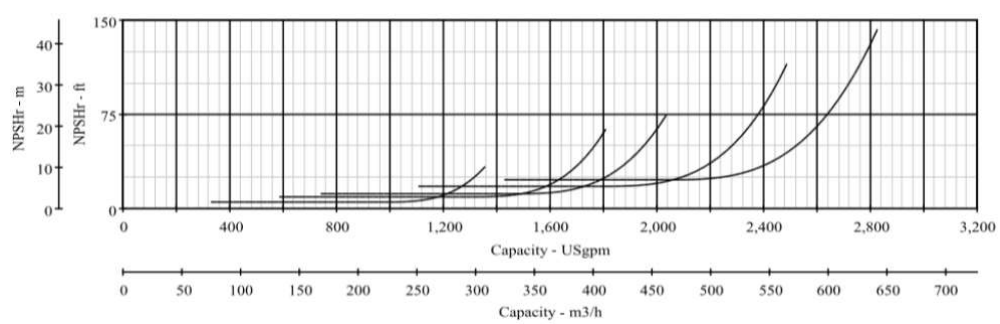


Figure 17: Net Positive Suction Head

EGL/HGL

Energy grade lines represent total energy available which includes elevation head, pressure head, and velocity head. Hydraulic grade lines only include elevation head and pressure head. From the beginning to end, total energy decreases as some of the energy is lost due to friction. Figure 13 below shows Energy grade line and hydraulic grade line of pipe 10 during fire flow. Pipe 10 connects water storage tank to distribution system. Beginning of the pipe has total head of 392 ft. and hydraulic head of 389 ft. at the end of the pipeline, the total head is 378ft and hydraulic head is 376. Approximately 12 ft. of head is lost in 374 ft. of pipeline. This is because velocity is high in pipeline due to high flow. Such diagrams can be drawn for each pipeline to visually see how much energy is lost or to obtain head lost per feet of pipeline. Appendix B includes calculations for energy grade line and hydraulic grade line. Energy equation is used for the calculations.

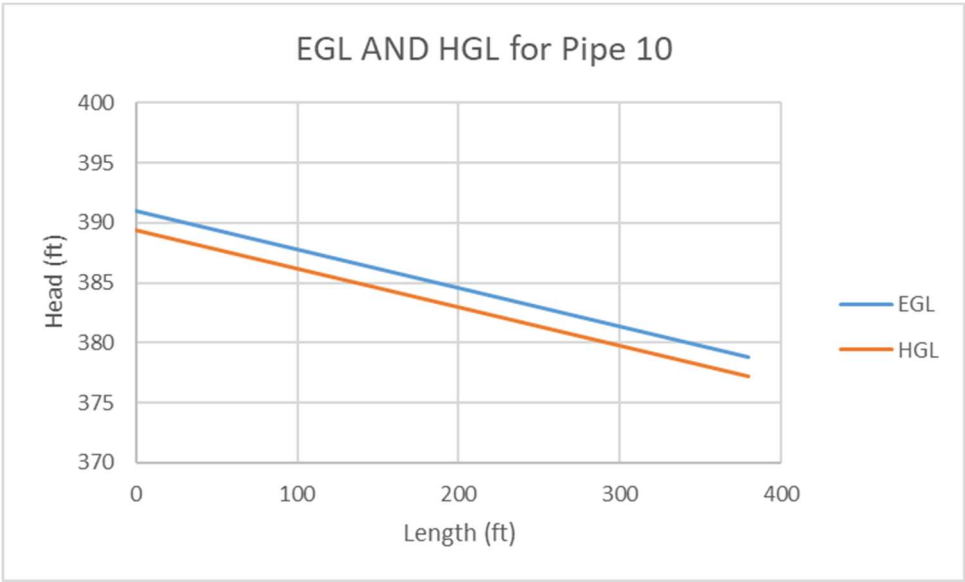


Figure 18: EGL and HGL for Pipe 4 during Fire Flow

System Model

EPANET modeling is used to analyze the behavior of water disturbing system over period of 24 hours and to make necessary changes so that overall efficiency of the system can be improved. Required pipe sizes for potable water is only 4 to 6 inches but pipes are designed with diameter of 8 and 10 inches because fire demand of 2500 gpm governs the system. For redundancy, fire flow volume required is stored in on site elevated tank. Required tank diameter is 50 feet elevation is 115 ft above ground. Tank itself is 25 feet high. The system is analyzed for two scenarios: potable water demand and fire flow demand. Figure 14 shows layout of potable water distribution system and Figure 18 gives layout for fire flow. For potable water, system is designed for available pressure of 40 psi at point of connection. Booster pump is used to increase pressure up to 75psi. The model uses two diurnal curves. Curve one shown in Figure 15 is obtained from city of Tulare master plan and it models the fluctuation of demand for 24-hour period. Diurnal curve 2 shown in Appendix C is used for irrigation control. Irrigation will be done at night from 19:00PM to 4:00AM as there will be less evaporation at that time. Maximum peaking factor in demand is 1.40 at 6:00AM. Figure 21 shows how the demand changes over 24-hour period because of peaking factors. Blue curve in the figure is demand for irrigation and pink curve is demand for one hospital wing. Other two wings for the hospital have identical demand pattern. There is no irrigation demand from 4:00 AM to 19:00 PM. Figure 22 shows how the pressure at output of pump changes as the demand fluctuates. Pressure stays between 77psi and 78 psi. Values for flow in each pipe and pressure at each node can be found in **Error! Reference source not found.** and **Error! Reference source not found.** respectively. During no fire flow, pressure at each node and flow through each pipe meets the criteria set by city of Tulare. This model does not incorporate the usage of variable frequency drive (VFD) at booster pump. Usage

of VFD will further improve the efficiency of the system and there will be no fluctuations and pressure output at pump can be further reduced based on the behavior of the system.

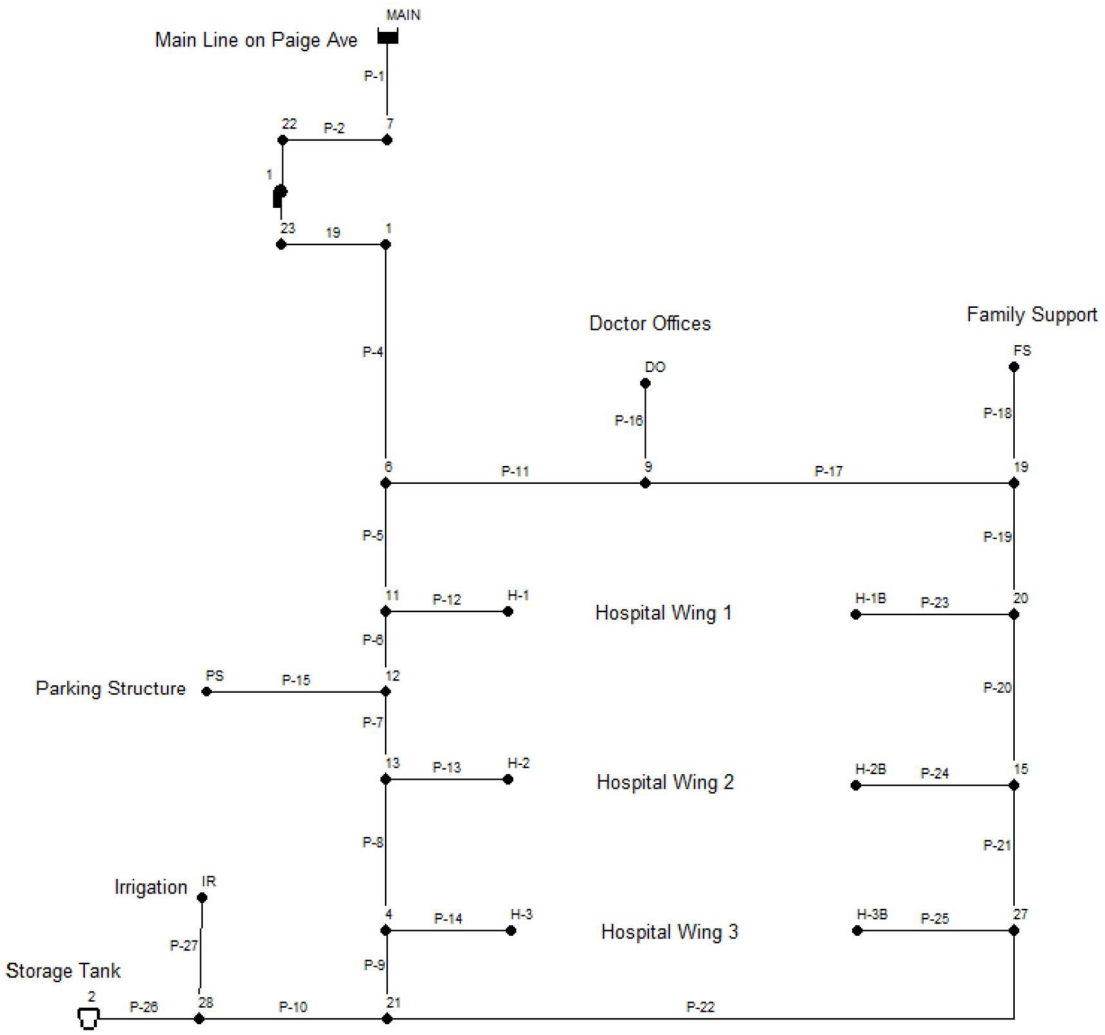


Figure 19: System Layout potable water

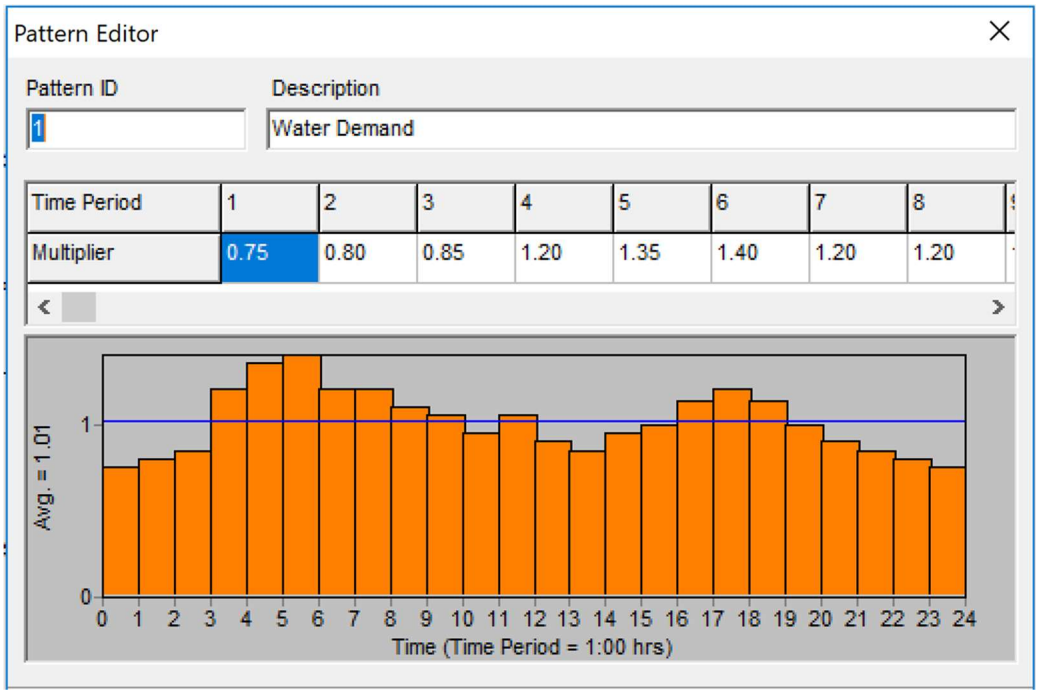


Figure 20: Diurnal Pattern

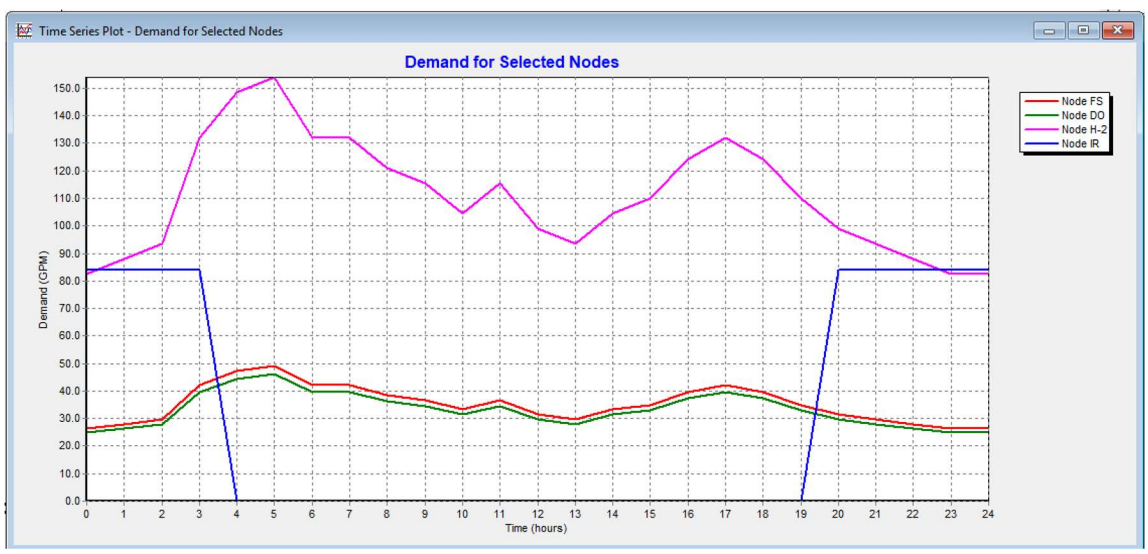


Figure 21: 24 Hour demand

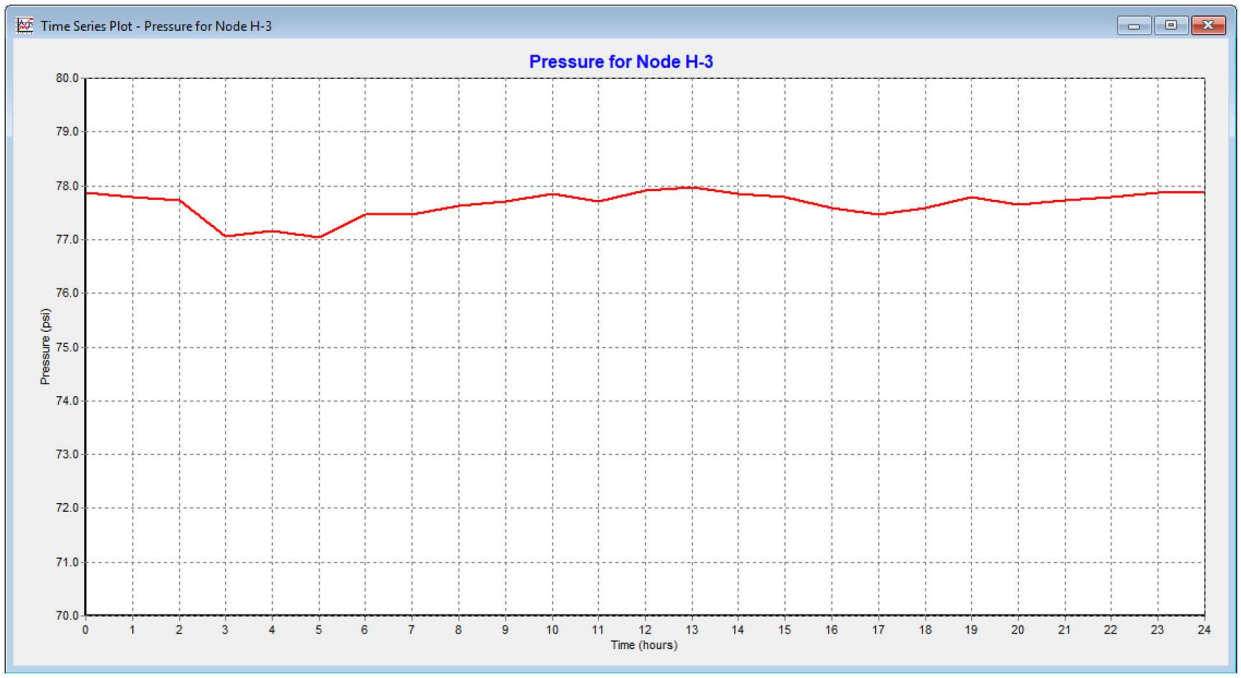


Figure 22: 24 Hour Pressure variation at Pump

Figure 18 shows the layout for fire flow. For fire flow, it is designed in a manner where all water for fire flow will be provided from onsite storage tank. In event of fire, flow thorough pipe P-10 is 2500 gpm. This is a conservative model and illustrates worst case scenario. In reality, some of the flow will be provided by city of Tulare. On site storage is designed for redundancy if city infrastructure fails to deliver the required fire flow. Hydraulics for the pipes and nodes during fire flow can be found in APPENDIX C. In an event of fire, system meets minimum pressure requirements set by city of Tulare. Figure 20 shows how elevation of storage tank changes during fire flow. Each hospital has two points of connection for fire flow. This is for redundancy so that fire personnel have multiple points of connection which gives them broad area of coverage. This approach makes maintenance of system easer as one half of the loop can be turned off if any maintenance needs to be performed

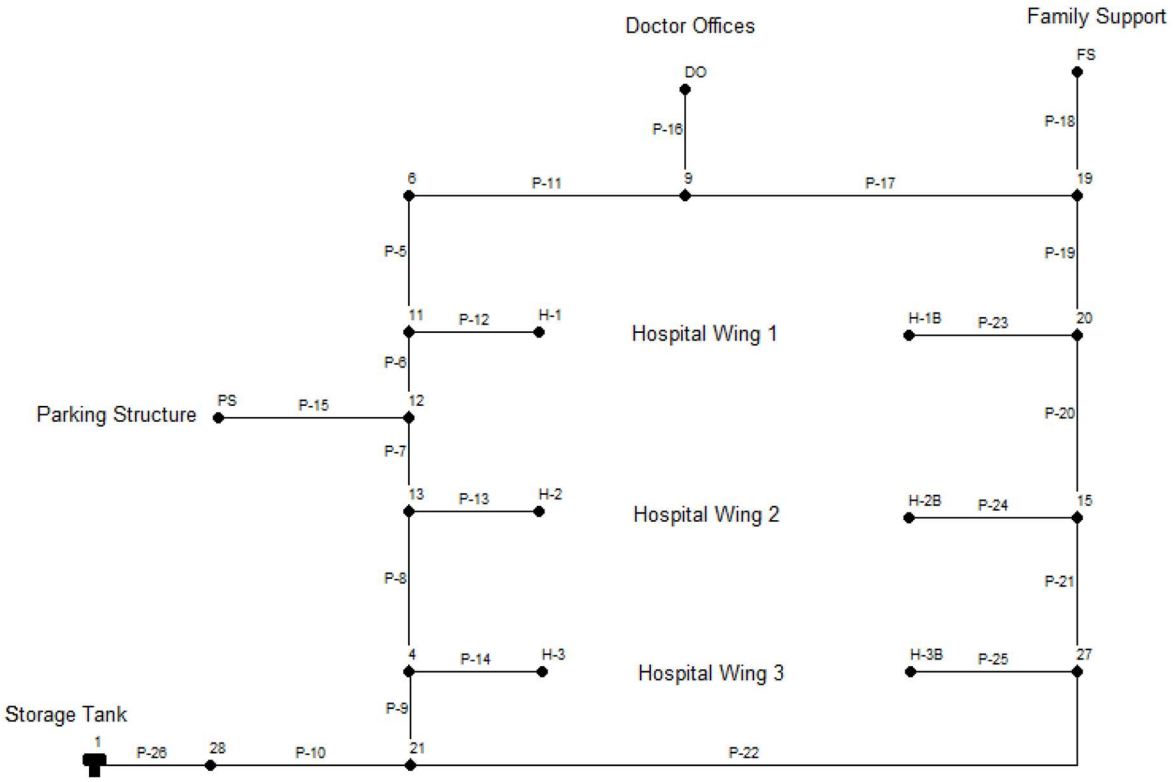


Figure 23: Fire Flow Layout

<u>Antonio</u>

Shown below is the data taken from EPANET showing different critical scenarios for the needed fire flow. According to the Tulare General Master Plan, there are eight fire test locations around the city. Using EPANET, these eight fire testing locations were used for testing in the software model to find the most critical scenario.

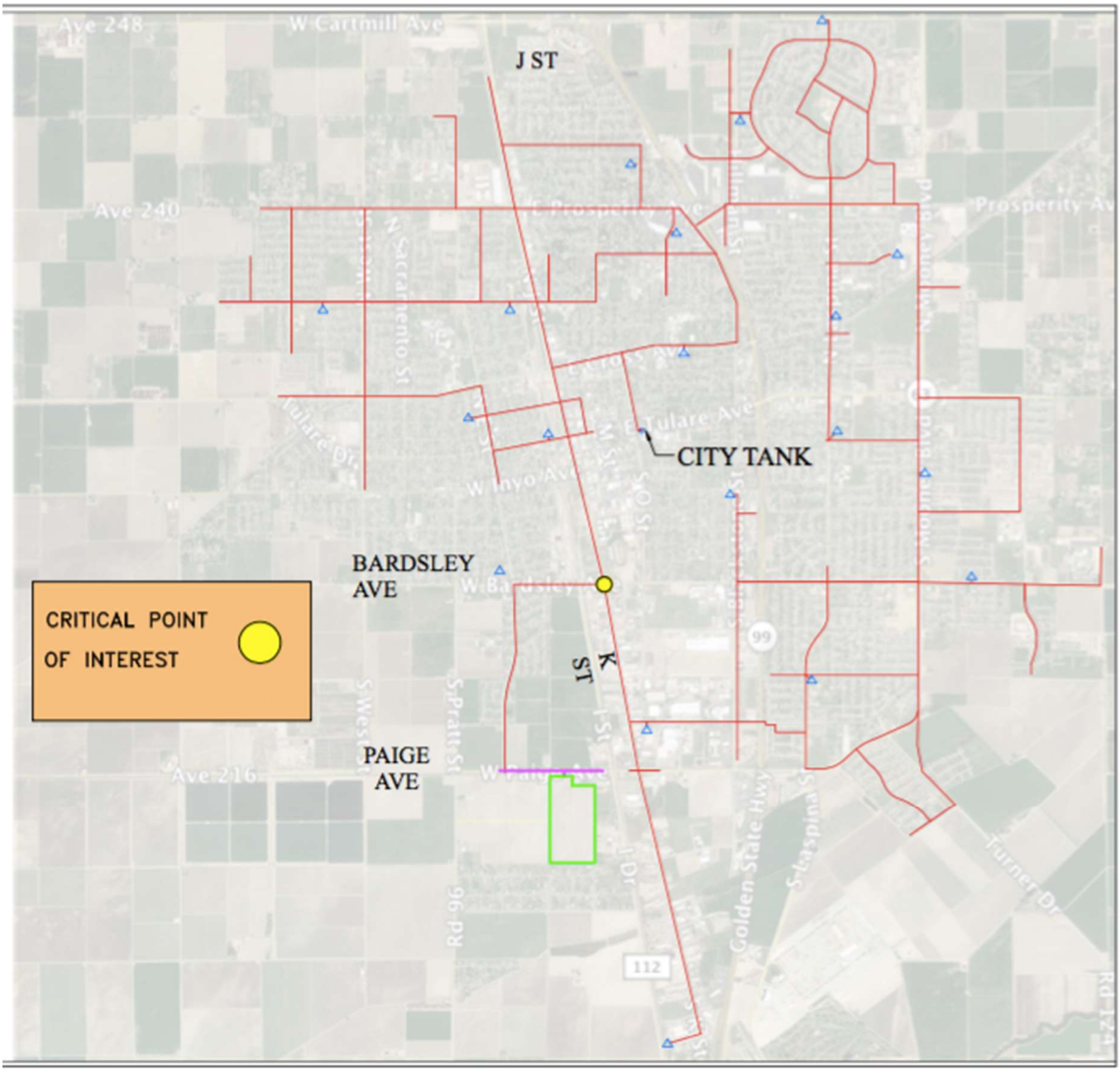


Figure 24: Off-Site Fire Flow

Table 38: Off-Site Fire Flow Hospital Results

	PRESSURE (PSI)	DEMAND (GPM)
HOSPITAL	33	750
	VELOCITY (FT/S)	UNIT HEADLOSS (FT/KFT)
PIPE (EAST)	1.2	0.4
PIPE (WEST)	1	0.24

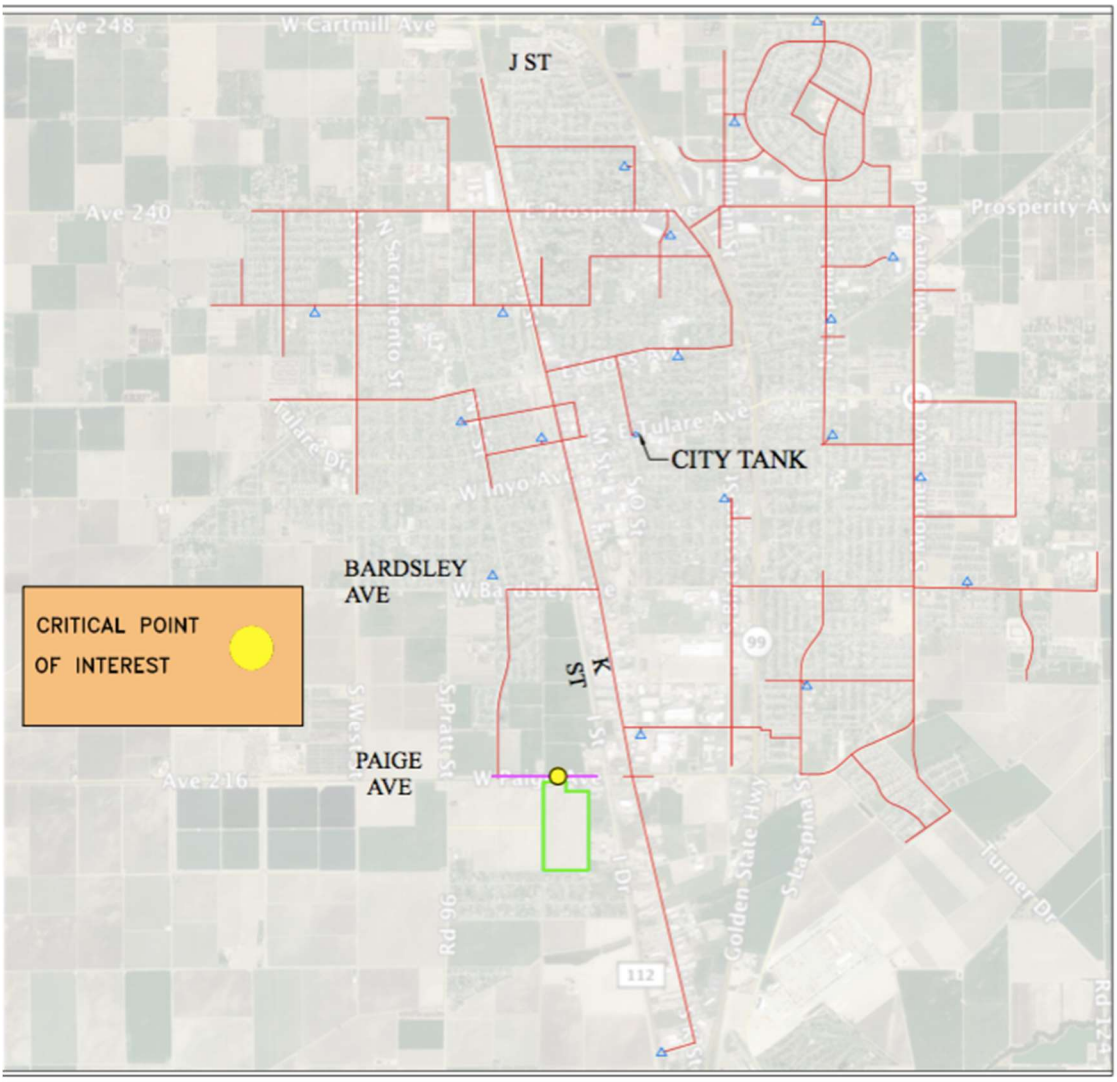


Figure 25: Hospital Fire Flow

Table 39: On-Site Fire Flow Hospital Results

	PRESSURE (PSI)	DEMAND (GPM)
HOSPITAL	33	3250
	VELOCITY (FT/S)	UNIT HEADLOSS (FT/KFT)
PIPE (EAST)	7.0	9.4
PIPE (WEST)	2.2	1.2

Bagazi

Parking Geometrics

Parking stall designs are influenced by the size of automobiles presently in production; the finances involved in construction; and the users. Thus, the hospital's parking lot will function for two different groups of users i.e. the visitors and the patients and the employees.

The main purpose of the design is to make the most of the total number of parking spaces in the total available space i.e. 125,000 square feet. The parking lot design should be made by keeping in view the subsequent considerations:

- It should provide uninterrupted movement or traffic circulation through the parking lot.
- It should also allow secure movement of pedestrians from the parking space to the hospital's building.
- It should allow for suitable landscaping of the parking space without disturbing the lighting of the site.

The parking lot is designed with the intention that both the patients and the employees can easily find a parking stall. The hospital's parking space capacity will be deigned based on the number of beds that will be exists in the building. The parking design will be designed while keeping in view the standards of the Tulare city in California State.

Parking capacity analysis

Dimensions

The main features of parking lot dimensions are the stall's width to motor vehicle width, and the easiness of moving into and out of the parking space. Given the typical hospital structure having patients and visitors, it is suggested that a 9" wide and 20" long parking stall be used throughout the facilities. The total parking area was about 125,000 square feet. All parking spaces, exclusive of access drives or aisles, shall consist of a rectangular area not less than 8feet wide by eighteen

(18) feet in length except that parallel parking stalls shall be 9 feet by twenty (20) feet. The dimension of the parking lot is to be based on the city's standard specification and measurements which is 20'x9' ('=ft.). There will be paved parking around the buildings of hospitals, Doctor offices, and Family support. In the parking area of about 125,000 square feet, more than 600 parking stalls have been design. Also, there are about more than 216 concrete parking stalls for the parking garage. The parking garage will be following the 90-degree parking design inside the building. These parking are including both ADA and Solar power-electric vehicles station and have their own space.

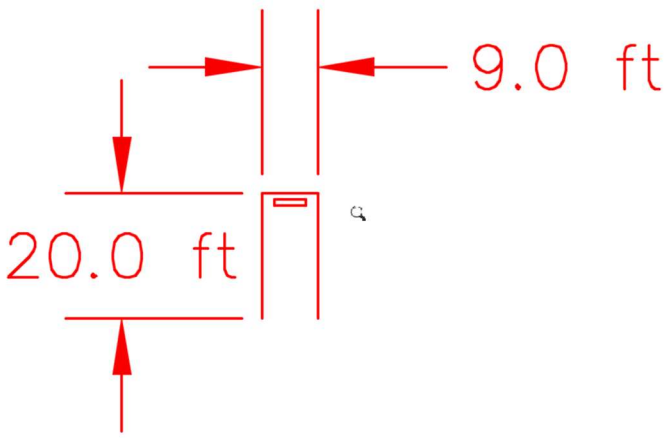


Figure 26: Typical Section of the Parking Drawing

Sidewalks

The driveways and sidewalks were designed according to the standards of the Tulare city. The sidewalks width should be 48" with no handrails. The slope should be 1 in 20 max and the cross slope of the sidewalks is 1/4" per foot max. The parking lot has two driveways measuring 36 feet each which fulfills the requirement of the city of Tulare.

Aisles

The aisle length between each parking stall is 12 feet. Moreover, according to ADA National Network, the number of accessible parking designed is 8 stalls, and 2 of them are van accessible

(ADA, 2014). Access aisle of the van-accessible stall is designed to be 5 feet with a 19 feet length.

Location

The parking is divided into two main lots to enhance the effectiveness and accessibility. The hospital's parking lot area A is located in front of the main building having handicapped stalls in compliance with ADA standards of providing convenience to incapacitated drivers. These stalls are attached directly to the main hospital building allowing stress-free accessibility to ramps and building. Additionally, the hospital's parking space has a designated space for a loading zone in the back for heavy vehicles.

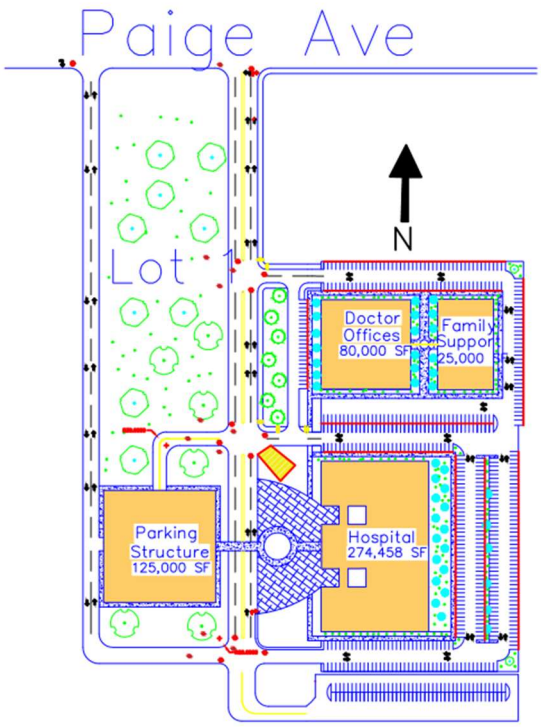


Figure 27: Site view

HANDICAPPED ACCESSIBLE PARKING

General Accessible parking for disabled persons must conform to the standards of the Tulare city in California State. The accessible parking for disabled persons should be suitably positioned near or adjacent to the main accessible hospital's entrance, through the shortest accessible route. Moreover, a nearby or an accessible route must always be provided from the accessible parking to the accessible entrance. A nearby or an accessible route does not have curbs or stairs, and it should be at least three (3) feet wide, with a firm, steady, and non-slippery surface. The slope beside the accessible route should not exceed 1:12 in the travelling course.

The measurements of accessible parking spaces must be at least 8' wide with a 5' wide adjacent access aisle. In addition, it is preferable to have the first space and 1 in every 6 additional spaces to be van accessible. The dimensions of this space should be at least 8-11' wide space with a 5' wide adjacent access aisle.

PARKING SPACE SIGNS

Following the Americans with Disabilities Act Accessibility Guidelines (ADAAG), the accessible parking space should have marking and signs which should be displayed on secure mountings in a clearly visible area. Moreover, pavement marking codes must be used to enhance the signs. The spaces for van parking should also be marked accordingly.

- Every parking space to have a fix and stable (70 sq. in.) reflecting porcelain painted steel sign at the inner end of parking space. This sign is to be mounted to least 36" above the ground.
- The van accessible parking space should have a mounted sign of "VAN ACCESSIBLE" adjacent to accessible parking spaces.

- A handicapped symbol of 36" x 36" measurement and white on blue background shall be evident while parking a vehicle.

Number of Spaces

The parking stalls in each parking lot will have specific space for disabled persons

Table 40: Minimum Number of Accessible Parking space

Minimum Number of Accessible Parking Spaces

2010 Standards (208.2)

Total Number of Parking Spaces Provided in Parking Facility (per facility)	(Column A) Minimum Number of Accessible Parking Spaces (car and van)	Mininum Number of Van-Accessible Parking Spaces (1 of six accessible spaces)
1 to 25	1	1
26 to 50	2	1
51 to 75	3	1
76 to 100	4	1
101 to 150	5	1
151 to 200	6	1
201 to 300	7	2
301 to 400	8	2
401 to 500	9	2
500 to 1000	2% of total parking provided in each lot or structure	1/6 of Column A*
1001 and over	20 plus 1 for each 100 over 1000	1/6 of Column A*
*one out of every 6 accessible spaces		

Parking Lot Access Ramps

Any path of travel will be considered a ramp if its slope is greater than 1:20 (5 percent) of horizontal run. Ramps or raised barriers should be made at curbs to ease the access to the accessible routes leading from the parking lot. Ramps must be 5feet wide with a slope of 2% and ramp grades cannot exceed **1:12.** Handrails should also be in accordance with the recent

requirements listed in the Architectural Barriers Act and by the city standards and specifications. The hand rail height should be between 34" to 38" whereas the diameter of grip of the hand rail should be 1 1/2" Two different ramps conforming to ADA specifications were designed from parking area to allow ease in accessibility from parking lot to hospital's building. Ramps shall be constructed according to the following criteria.

Parallel Curb Ramps are adapted to parallel sidewalks. This design offers an option in a situation where the limited space prevents the top landing. A preferable level of landing measuring minimum 48" long accommodates maneuvering among runs and right-angle turns to linking directions, like parking access aisles and crossings

Landing aspects:

- It should be at minimum same as the width of the ramp run leading to it.
- The landing length shall be at least of 60".

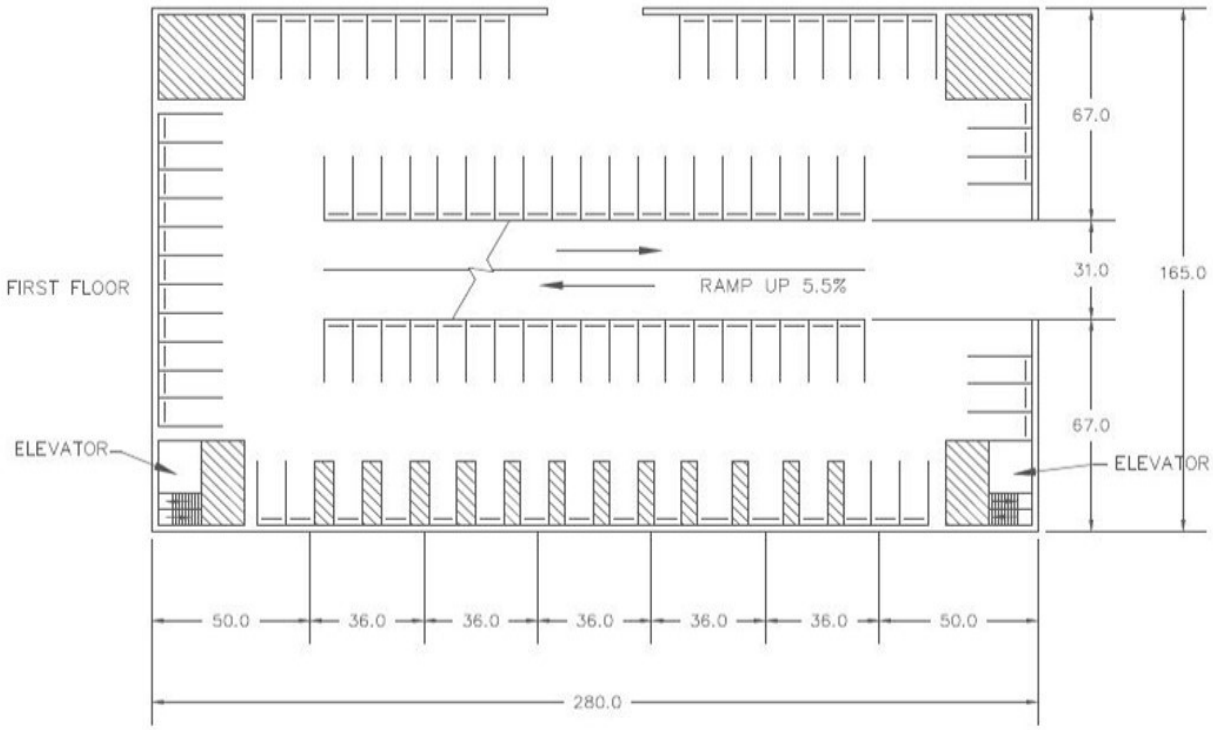


Figure 28: Typical Drawing of Parking Garage (Regina Barton)

Pavement Material Selection and Structure Design

A flexible type of pavement is opted to be designed by using the standards of the Caltrans method. Pavement Design Caltrans method is used to create flexible pavement which is RHMA-G. Thus, the decided material is a blend of Hot Mixture Asphalt (HMA) and Rubber Hot Mixture Asphalt (RHMA). Before choosing the pavement, the grading was made with 2% slopes in between-cross parking slots. Prior to design the pavement, the R-Value of the soil was required. The soil investigation was made by the geotechnical team to determine the R-Value for Highway Design Manual (HDM) then the co-relation was used to find RHMA- G, R-value based on the geotechnical report provided, standards and specification of the city of Tulare. After calculating the G and R value, the value for the thickness of the aggregate base was taken from Table 633.1 in the Highway Design Manual of Caltrans.

Rubberized Hot Mixed Asphalt (RHMA) and HMA was selected for pavement design. It is more efficient and cost effective as it is made of recycled rubber tires and does not need to replaced when cracked but only requires resurfacing and repairs in case of damage. According to Caltrans HDM, the life span of RHMA is 20-years, which is effective and preferable in the selected location having extreme weathers. it supports the expansion and contraction of concrete in the extreme weather. Moreover, in addition to being more time and cost effective, it is also environment friendly. The pavement design will have a 20% of the RHMA with 2" thickness and 4" of HMA below to RHMA. Then there will be an 7'' of Class 2 base.

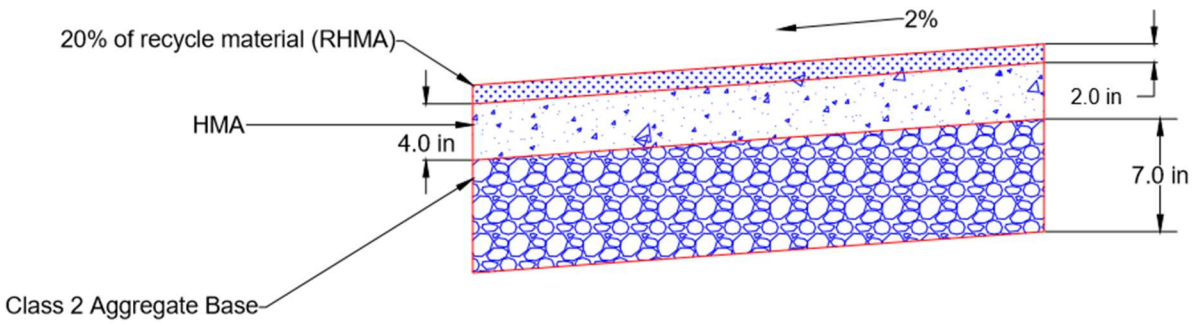


Figure 29:Typical-section of Pavement

Interior traffic control

Traffic control warranty analysis

One of the significant feature of designing a parking lot is to take into consideration its interior circulation. The parking lot include the stop signs at the end of aisles and drive ways. A stop sign supports the interior circulation that was designed by the calculation of the stop-side distance in each lane that is located in the center of the site plan. The speed limit of the parking lot is 10mph

and the aisle width of 27 feet between the stall enables the vehicles to park easily. The parking lot is designed keeping in view the entry of large vehicles on the site, thus, the parking lot has a turning radius of 46 feet and in accordance to AASHTO, the response time of stopping is 2.5 seconds. By means of this information, the minimum stopping side distance came out to be 46.3 feet. In this case, the provided distance is 43 feet which will not allow the vehicles to stop without harm.

$$d = 1.47si\ t\ +\ (si2 -\ sf2)/30(0.348 + -0.01G)$$

where d= total stopping distance, feet

$s_{i=}$ initial speed, mi/h

$s_{f=}$ final speed, mi/h

t= reaction time, s

G= grade, %

Consequently, a traffic control plan was required. The students have to design the intersection and install a traffic control to it. The intersection will be on Pagie Avenue and the entrance and exist of the site. The student has studied the warranties and found out that it meets the warranties to install a Stop sign in the intersections and at the end of each aisle, to ensure that the vehicles stop safely.

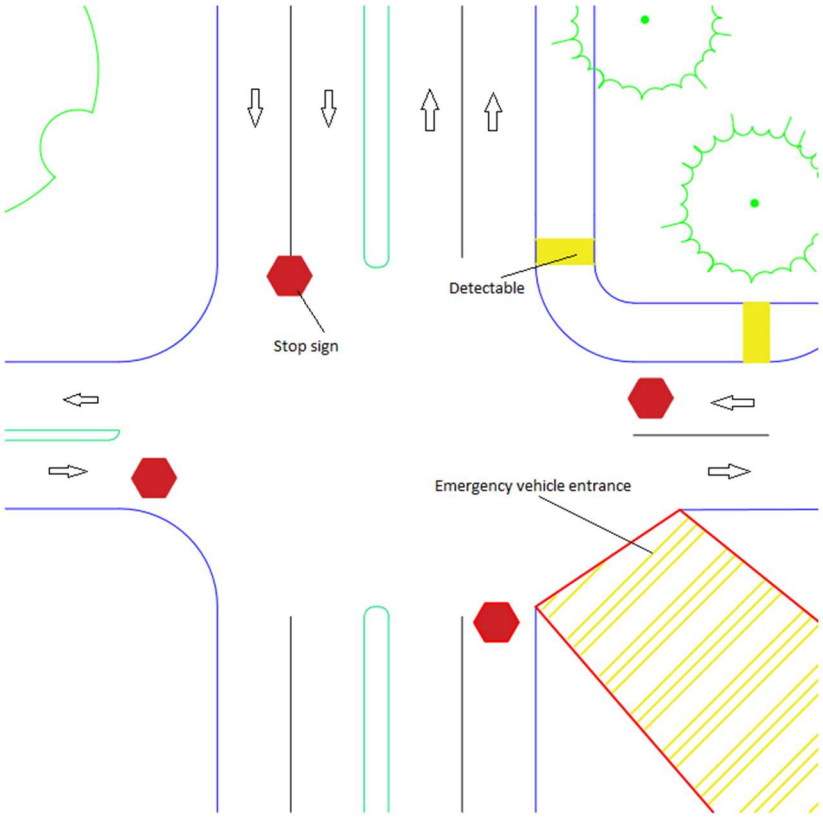


Figure 30: Intersection with Stop Sign by AutoCAD

Loading zone design

The aim is to provide a designated or labelled easy to get to passenger loading zone positioned away from other traffic patterns. The requirements of loading zone design are:

- It should be nearby to accessible entrance
- It should have a curb ramp to a level of sidewalk
- It should have a canopy for shelter
- It should have communication system for support
- It should have access aisles which should be at minimum 5 feet wide by 23feet and 1inch long, and parallel and level with the vehicle pull-up space

Intersection

Designing an intersection

Due to the increase in traffic volume after the completion of the project, the design of the intersection was required. The intersection will be on Pagie Avenue and the entrance and exist of the site. The factors including the dimensions, maneuverability of bicycles and other means of transportation like automobiles, transit vehicles, trucks, etc. affect the design of an intersection. Alternative intersections offer enhanced safety, efficiency, and reduce the delays with minimum expenses and with less impacts than traditional solutions. A left turn will be designed and this left turn will be a median crossover left turn bay from Pagie avenue to the hospital. The anticipated intersection was drawn by using CAD and can be seen in Figure 6

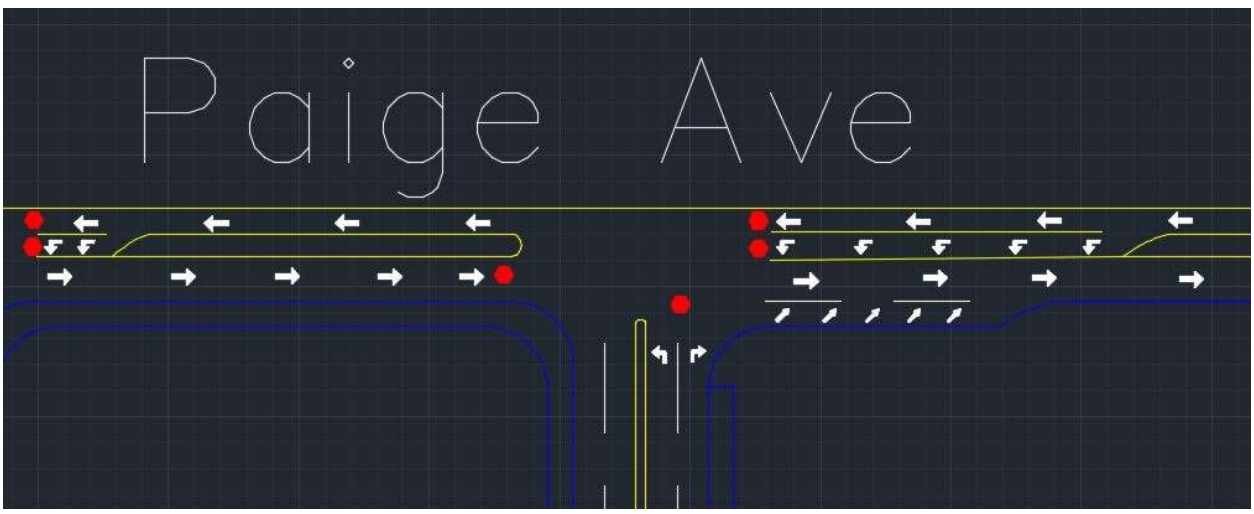


Figure 31**: Median crossover**

To design the intersection, calculations were made from ITE (Trip Generation Manual 9th Edition* Volume 3). This matrix shows the expected number of vehicles will be on the coming to the site and use Paige Ave. the calculations are based on traffic count from 4 to 5 P.M. The east bound has 149 cars per hour and the west bound has 155 cars per hour. It was found that the

directional distribution measured on the basis of average vehicle trip on weekday (A.M. Peak hour of Generator) was the highest i.e. 69% entering, 31% existing.

Angle of Intersection

The intersection is designed as an alternative intersection design having a displace left turn. This intersection is designed at a right angle (90°) which provides the most advantageous settings for intersecting and turning traffic movements. This right angle dimension of the intersection will provide:

- The undeviating and shortest possible crossing distance for the pedestrians, motor vehicles, and bike riders.
- Sight lines which adjusts and elevates corner sight distance and allows the motorists to assess the relative position and the speed of approaching traffic flow.
- reduction in congestion
- Intersection geometry that lessens the vehicle speed while taking a turn and this helps in avoiding accidents and also helps in reducing the severity of such accidents.
- an alert to the motorists that while taking a turn they should drive appropriately through the traffic on the road they are exiting, to traffic they are entering, and to pedestrians passing the intersection.

Deviating much from right angle intersections will decrease the visibility and will escalate the size of the intersection which will result in high speed turns and may reduce yielding by turning traffic. Thus, such intersections are more effective, cost-effective and better solutions as compared to the traditional designs. Such intersections cater to the mobility needs and safety of the drivers of different type of vehicles, bicyclists, transit riders, and pedestrians.

Left-turn Channelization

The main purpose of a left-turn lane is to accelerate the flow of traffic by controlling the movement of turning traffic, to increase the size of the intersection, and to improve the safety features.

Speed Limit

Speed changing areas for vehicles entering or exiting the main flow of traffic are favorable to the efficacy and safety of an intersection. The incoming or the entering traffic combines with ongoing traffic flow most efficiently when the integration angle is less than 15 degrees or at minimum speed differentials in our case 10mph.

CHAPTER 4: SUSTAINABILITY

4.1. Design Implementation

To reduce the depletion of materials, recycled steel will be utilized for this project. Using recycled steel is beneficial to the project budget as it is cheaper to reuse steel than having to make brand new members. With this aspect of recycled steel, it also reduces greenhouse gas emissions and takes less energy to produce the required members. This is because the material has already been processed before.

In order to make the structure more sustainable Fly Ash concrete will replace Portland Cement concrete, which will help reduce the cost and help reduce the CO_2 emission in the air.

GEOTECHNICAL

In order to determine if the foundation design for the parking structure is sustainable, various factors will be taken into consideration, such as the availability of local materials, energy efficient construction methods, and a cost-effective approach. Materials used in the construction process should contain low embodied energy and minimize the amount of greenhouse gas emissions (Riversong, 2011). Reinforced concrete is the most common material used in the construction of foundations, but concrete is known to contain a large amount of embodied energy, and its production can contribute significantly to carbon emissions and global warming (Riversong, 2011). The environmental impact of concrete in construction can be minimized by substituting cement with alternative, low-energy materials such as pulverized fuel ash or ground granulated blast furnace slag, without compromising the foundation's strength or bearing capacity (Livesey & Mace, 2016). Recycled or secondary aggregates could also be employed to reduce the amount of concrete used (Service, n.d.). The foundations for the parking structure will be designed to reuse and conserve materials and resources. The use of recycled materials, such as

recycled steel for the reinforcing bars, will be implemented in the construction of the footings. The soil distributed on site will also be restored in order to reduce the amount of soil to be brought from off-site. This will reduce the CO_2 emissions from trucks used to transport soil. The grading plan will be implemented with the goal of conserving existing soil and resources and minimizing the amount of cut and fill required.

4.2. Design Assessment

STRUCTURAL

In order to measure the effect of the fly ash concrete instead of Portland cement two different metrics were used. Cost and measuring the different between CO_2 emission being produced by the product. Shown below in the tables the difference between cost and the emissions produced.

Table 41: CO2 Produced per wing

	Pounds of Material	Co_2 Produced
Fly Ash	864,000	-
Portland Cement	1,026,000	1,026,000

Table 2: Cost Difference per wing

	Price of Material	Total Cost
Fly Ash	$40/ton	$17,280
Portland Cement	$75/ton	$38,475

A sustainability analysis was conducted on the parking structure using ASCE's Envision checklist to assess the structure's environmental effect. The full checklist was completed, and the summary scorecard is included in Appendix A. However, a detailed explanation of the background and rationale of each selection in the checklist is beyond the scope of this report. Instead, this report serves to briefly discuss which sustainability features were selected to be included in the design, as well as a general explanation on why others were ignored; how the selected features will be implemented; and how they will be measured.

There are many other criteria from Envision that could possibly be met and would be met in a normal project. However, the confines of the scope of work, and the fact that the work will be completed by a single designer within five months, make many criteria infeasible. For example, under normal circumstances, a whole host of environmental studies and impact reports would be complete to conform to the National Environmental Protection Act and the California Environmental Quality Act. The preparation of these and the associated mitigation studies would fulfill many of the Envision criteria, especially in the Natural World section. However, hiring environmental consultants is simply impractical. Also impractical are other standard practices, such as stakeholder involvement and coordination. Key stakeholders were previously identified and contacted, but their continual involvement for a theoretical project is unrealistic. Therefore, sustainable practices for the parking structure were limited to three different Envision criteria.

The parking structure will incorporate Envision's quality of life criterion of improving community mobility and access (QL 2.4). The intent is to "locate, design and construct the project in a way that eases traffic congestion, improves mobility and access, does not promote urban sprawl, and otherwise improves community livability." The resource allocation criterion of using recycled materials will also be included (RA 1.3), as well as the use of regional materials (RA 1.4). The intent of RA 1.3 is to "reduce the use of virgin materials and avoid sending useful materials to landfills by specifying reused materials, including structures, and material with recycled content," and the intent of RA 1.4 is to "minimize transportation costs and impacts and retain regional benefits through specifying local sources."

QL 2.4 has three parts. Part one requires that the project in question provide access to facilities. This is a fundamental feature of the parking structure, as it will be just a short walk of just a few seconds from the parking structure to the hospital. Furthermore, the parking structure utilizes a

traditional design with generally open walls, which encourages both the feeling and reality of public safety. Part two requires that expected traffic flows and volumes be considered. The parking structure will incorporate this by including enough stalls for the projected amount of traffic generated by the hospital, as dictated by the City of Tulare. Part three requires that traffic congestion be reduced. This will be met by utilizing a two-way traffic design throughout the parking structure.

RA 1.3 has two parts, namely that existing materials are reused as appropriate, and that a significant amount of recycled materials be incorporated. The first part is not applicable to the parking structure, as the current project site is an empty lot, and contains only a single shed and no reusable materials. However, the second part can be met by this project. Envision defines the threshold of "significant" recycled content as 5%. Therefore, a provision will be included in the specs to incorporate the use of fly ash, which is a recycled material and a byproduct of other industrial processes, into the concrete ("Sustainability."). Furthermore, according to a phone conversation with Jose Mendoza, the manager of the western region of the Concrete Reinforcing Steel Institute, 100% of reinforcing steel is from recycled sources. Consequently, the combination of recycled rebar and recycled concrete components will be enough to meet the 5% requirement.

Lastly, R 1.4 contains two parts as well. The project team must identify local sources of materials, and use a significant amount of these materials. "Local" is defined by Envision as a 100-mile range for concrete, and a 250-mile range for plants. In the case of the parking structure, the 250-mile range would govern, as "concrete" refers to fresh concrete designed to be cast in place, as opposed to the precast concrete manufactured at a plant that the parking structure will require. There are many precast concrete plants within the specified range: Oldcastle Precast is

just a little over 2 miles away; Mid State Precast is about 18 miles away; and Jensen Precast is around 50 miles away. While a sole supplier will not be designated, as that is beyond the scope of work of the parking structure, in addition to being a decision that cannot be made without the consent of the owner, it is reasonable to understand that these will be the primary bidders for the project. Greater transportation distances will jack up prices enough that competitors will no longer be competitive (PCI). To ensure that this one of these plants will be used, a specification will be added limiting the distance from precast plant to site to under 100 miles.

GEOTECHNICAL

The sustainability of the foundation design and grading plan can be evaluated by applying the Envision rating system. Sustainability analysis of the geotechnical design will focus primarily on the categories of resource allocation, natural world, and climate and risk. To evaluate whether the design meets the resource allocation requirements, the foundation and grading design can be assessed to determine if local materials are used and if any wastes generated from the construction process are properly handled (Bertera, 2012). The use of recycled or reused materials, such as reinforcing steel, will be considered and the energy consumption of all materials used will need to be evaluated. To reduce the amount of excavated materials brought off-site and to conserve resources, it will be determined whether the grading design minimized the volume of cut and fill and therefore minimized the amount of soil to be exported or imported. A sustainable use of resources and material conservation will be required throughout the design and construction phases. To meet the requirements of the natural world rating category, the grading plan will need to be evaluated for its potential impact on the natural environment of the on-site soil (Bertera, 2012). Soils disturbed during the grading process and construction should be properly managed and restored so that the on-site biotic environment is not affected. The

environmental impact of designing and building the foundations can be assessed by estimating the amount of greenhouse gas or air pollutant emissions that might be discharged, such as the CO_2 emitted from the production of cement for the concrete.

WATER RESOURCES- SANITARY SEWER

MARIO

Envision will improve sustainability for infrastructures. Its expected for Envision 2.0 rating system to improve community quality of life and stimulate sustainable growth and development when designing the sanitary sewer system. Sustainability will be achieved when using recycled material for the pipes, this will save money and will add less cutback on unnecessary waste of material. The gradient of these pipes will also implement sustainability, having a proper gradient will allow the waste to travel to the main sewer line with little to no settlement. Sustainability can also be implemented when choosing material, the use of recycled material will increase sustainability. Choosing the right material can improve the life on the system. Picking a material that is resilient will cut back on maintenance cost as well as the life expectancy which all increase sustainability within the project

ANTONIO

Being an engineer, designing and fixing the problems of today is not as simple as it may seem. Our project has to serve the future and whatever the future brings with it. As I develop the future water distribution system for the City of Tulare, that includes the New Valley Children's hospital, this network will have to perform the necessary tasks for future conditions as well.

Pumps around the City of Tulare use a lot of energy to provide water from the groundwater aquifers to the water system. In order to reduce that energy usage over a long period of time, solar panels can be implemented to the pumps at the wells. The solar panels work well for saving

energy if the energy usage occurs over a long period of time. These pumps work well, especially if they are being ran at their optimal efficiency. Running at maximum efficiency provides the most work without wasting any excess energy.

Since the pumps will continue to pump water as long as there is water in those aquifers, this is an implementation will pay for itself in the long run and help reduce the amount of energy being used. According to Energy Sage, the 20-yr savings for houses that use electricity is around $29,000. Because the city water pumps each use about the same energy to serve the water distribution system, this could lead to saving money for the future. Solar panels don't just save energy, but also reduce carbon emission into the air. Depending on the size of the pump and how many Kilowatts its uses, that is how much carbon emission will be saved. The table below shows the amount saved for different Kilowatts used.

Table 42**: $C0_2$ Reduction by Solar Panels**

CO_2 Reductions by Solar System Size Table

SYSTEM SIZE (KW)	ANNUAL SOLAR ENERGY PRODUCTION (KWH)	CARBON EMISSION REDUCTIONS PER YEAR (METRIC TONS)
2kW	2,840	2.1
3kW	4,260	3.2
4kW	5,680	4.2
5kW	7,100	5.3
6kW	**8,520**	**6.3**
7kW	9,940	7.4
8kW	11,360	8.5
10kW	14,200	10.6
12kW	17,040	12.7
15kW	21,300	15.8

Nadun

Sustainable development is an important factor in civil engineering design. This statement is much more valid for civil engineering designs in the modern era. The new children's hospital designed in Tulare, CA is planned to be a sustainable infrastructure. Sustainable development is not only limited to buildings. It can be implemented to transportation engineering, geotechnical engineering, structural engineering, water resource and environmental engineering as well. This sustainability report will discuss the implementation of sustainable development for storm water design. Storm water design usually consists of collection of hydrologic data, design of collection facility, design of conveyance facility and design of storm water storage system. The sustainability implementations can be usually added on all factors of the design expect for collection of hydrologic data.

In the process of designing for storm water system several implementations can be done. When designing the storm water collection facility, the inlets and gutters can be designed form sustainable materials. For example, the gutters can be made out from recycled concrete and the inlets could be designed from recycled steel. For the conveyance facility the swells can be made out of either grass channel or reinforced grass channel. The better option can be selected after calculating the flows through the swells in further design. Apart from being sustainable, using grass for a channel will provide better natural looking conditions as well. For the sewers as mentioned before; recycled material can be used. The storm water that end up in the storm water collection facility can be used for several things apart from just retaining in the retention pond. After sedimentation and filtration of storm water it can be used for irrigation, ground water recharge and treatment for human consumption. These are the ways that the sustainability development could be implemented for a storm water system.

The expected results would be to build and maintain a sustainable infrastructure. The best way to measure the results would to use a rating tool. Envision is one of the leading ways to measure sustainability in infrastructure. Apart from envision, the guidelines provided by environmental protection agency (EPA) called low impact development (LID) also can be used prior to design to design sustainable storm water system.

DEEP

Sustainable infrastructure does not impact the environment in a negative manner. Past construction has been done in a manner which impacts the environment in a negative way. To alleviate the impact that has been done and to save resources on earth, it is very important to develop sustainable infrastructure. (Royal Roads University, n.d.) There are multiple ways to measure if the infrastructure is sustainable or not. Two most commonly used measurement tools in the industry are LEED and ENVISION 2.0. LEED is specific to energy consumption etc. whereas ENVISION is very broad when it comes to rating. Envision has five chapters that give a rating based on the quality of life, leadership, resource allocation, the natural world, and climate and risk. (Institute For Sustainable Infrastructure, 2018) It is very important that the new children's hospital being built is sustainable. Only one part of the sustainability evaluation is analyzed.

Resource allocation from ENVISION 2.0 is analyzed in this part. The project will be designed to reduce energy consumption and reduce potable water consumption. To reduce energy consumption, variable frequency drive for pumps will be used and to reduce potable water consumption, a pressure regulator for fixtures will be used. Pressure regulating valve will be

used at each fixture to reduce pressure when pressure is high. The amount of electricity and potable water saved is analyzed below.

The hospital building is a three-story building. High pressure is required to pump water to the very top node of the building. 1 psi of pressure is equivalent to 2.31 feet of head. Each floor of the building is 16 ft high. The difference between the lower faucet to top of the ceiling is approximately 48 ft which translates to approximately 20.7 psi of pressure difference between two nodes without friction loss. Also, the faucet closer to the mains will have higher pressure compared to the one far away due to energy losses due to friction. For instance, water sense faucet has a flow rate of 1.6 gpm at 60 psi and 0.8 gpm at 20 psi.**Invalid source specified.** Such faucet will have a flow rate of 1.20 gpm at 40 psi. Flow rate changes by 0.4 gpm with a pressure difference of 20 psi. Each wing with approximately 100 faucets will translate into 40 gallons of water saved per minute of use. To save this water, pressure regulating valve will be installed before every faucet and dialed in at minimum pressure requirements. With this device, the pressure will be equal at each node and the same amount of water will be used per faucet. The exact amount of savings will be determined as the project proceeds and exact pressure and no of fixture requirements are determined.

Variable frequency drives regulate the voltage and frequency delivered to the motor. This action regulates the rotations per minute the motor spins at. Adding variable frequency drive to pump will adjust the rotation of the pump such that the flow will match the required demand. **Invalid source specified.** According to the study by Energy Department of California, energy consumption of pump to be significantly reduced by using these drives. Energy savings increase as much as 50 percent**Invalid source specified.**. According to their research, a 25 horsepower pump running most of the day with speed varying from 100% to 67% as demand fluctuates can

results in energy consumption by 45%.**Invalid source specified.** Calculations for energy savings will be performed as pump is sized based on the requirement of this project

Finally, two elements from Envision rating system are analyzed in this part. To make this Hospital sustainable, pressure regulating valves will be used to decrease potable demand consumption and variable frequency drive will be used to decrease the power consumption of the pump

Bagazi

The objective of designing a parking lot is to maximize the available space by observing, bearing period of the material and cost as well. The layout of the park should allow safe movements of pedestrians and allow continuous flow of traffic. Circulation patterns for pedestrians and vehicles should be very simple. Pedestrian routes should be considered to minimize the use of "short cuts" that often damage landscape areas. Circulation systems will be designed in to reduce conflicts between vehicular by focusing on the main entries as well as exits. Access to parking areas should be through a pavement with no backing into the public right-of-way. Thus, the parking structure would be having. The pavement will be made using rubberized mixes of asphalt to minimize traffic noise (Zhou, Holikatti, & Vacura, 2014). The choice of asphalt rubber in making the pavements is based on the fact that this area is characterized by high agriculture climate and was suggested by Eng. Koko from Caltrans. Equally, it is a preferred material for constructing surface layer. However, the student will use recycled tire together with asphalt to mitigate reflection cracking.

The parking design of the parking lot will include a solar-powered electric vehicle charging station. The station will charge approximately 4 electric vehicles at a time. Similarly, the

charging station will be equipped with onboard battery storage to facilitate charging of EVs day and night as well as during the times of blackout (EV ARC^{TM}, 2017). The level of power in the charging station will be a single 120-volt together with a 240-volt outlet. This equipment is the easiest type of charging and is cheap in terms of cost for installation. The two levels, 120-volt and 240-volt, will be placed opposite to each other to allow different users to use them effectively. However, with time, the parking lot will include more advanced charging equipment.

CHAPTER 5: DESIGN SPECIFICATIONS

Steel Notes

A. Structural Steel Wide Flange and Tee Shapes: Shall be new and shall conform to the requirements of ASTM A992.

B. Structural Steel Channels and Angles: Shall be new and shall conform to the requirements of ASTM A36.

C. Structural Steel Plate: Shall be new and shall conform to the requirements of ASTM A36.

D. Arc-welding Electrodes: Arc-welding electrodes shall be E70 series electrodes for A36, A572 and A992 material, E80 Series for A706 reinforcing steel and E90 series for A615 reinforcing steel. Electrodes shall be as recommended by their manufacturers for the positions and conditions of actual use. All welds used in members and connections in the seismic Force Resisting System shall be made with filler metals meeting the requirements specified in AWS D1.8 clauses 6.3.

E. High Strength Bolts: High strength bolts (HSB) shall conform to ASTM A325 or A490 Type 1, 2, or 3

F. Anchor Bolts: Anchor bolts shall conform to ASTM F1554 grade 36.

G. Welding: Welding shall be by operators who are qualified by test as per AWS "Standard Qualification Procedure" to perform type of work required.

H. High Strength Bolting: All high strength bolted connections shall be bearing type connections unless otherwise noted on the plans. Where noted on the plans, high strength bolted connections shall be slip critical type connections.

I. Bolts, rods, washers and nuts exposed to weather shall be hot dipped galvanized steel in compliance with ASTM A153.

Water System Notes

A. Interior diameter of the blue brute (c900) PVC pipe shall be 12 inches

B. Blue brute (c900) shall be DR 25, pressure class 165 psi

C. Excavation for pipe laying shall be a cut into a t-shape with at 12-13 inches flange length starting from outside edge of trench width

D. Trench width shall not exceed the 18 inches greater than the outside diameter of pipe

E. Bedding below proposed pipe shall be 6 inches

F. Initial backfill shall be at a height of 1 foot greater than the outside diameter of the pipe @ 90% relative compaction in increments of 6 inches

G. Finishing backfill shall be 2 feet in height at 95% relative compaction

H. C900 PVC pipe shall be at least 10 feet horizontal distance from nearest edges of sewer lines and shall be at least 12 inches above sewer line when the pipes must cross.

I. Fittings will have 1/16-inch cement mortar lining conforming to AWWA C104

Stormwater notes

A. All pipes greater than 24-inch diameter shall be reinforced concrete pipe. Pipe smaller than 12-inch diameter shall not be allowed.

B. Pipes shall have cover depth of at least 5 feet.

C. Class of pipe shall not change between manholes and shall be the highest class required between manholes.

D. The contractor shall perform all excavation necessary or required to construct all pipelines and structures covered by these Plans and Specifications.

Transportation notes

1. Movement signs which are to be blanked out might be secured by the

Following techniques:

A. Plate signs might be secured utilizing a 1 in thick sheet is perfect with the material in the sign.

B. (b) Plate signs that are to be blanked out for a period not surpassing one year may on the other hand be secured utilizing a self-glue plastic film.

C. (c) Other signs should be secured utilizing a free cover sheet of material affirmed by the Engineer.

   a) Self-cement plastic film should be good with the material in the substance of the sign and might be settled and expelled as per the maker's suggestions.

   b) Loose covers might be safely attached to the back of the sign. Tape or other sticky material might not be connected to the characteristics of signs.

   c) Coverings to movement signs might be adequately obscure to avoid reflection from the secured sign and might not be evacuated until the Designer so educates.

   d) Unless generally allowed by the Engineer, the characteristics of activity signs which have been raised and which don't relate either completely or to a limited extent to the activity circumstance which applies around then might be blanked out as expressed in this Clause.

General Notes

A. Codes and Standards: Comply with all Federal, State, and Local codes and safety regulations. In addition, the fabrication, priming, and erection of structural steel shall comply with all the applicable provisions of the following codes, specifications, and standards, except where more stringent requirements are shown or specified:

1. "Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings" by the American Institute of Steel Construction, current edition.
2. "Codes of Standard Practice for Steel Buildings and Bridges" by said AISC, current edition.
3. A.W.S. "Structural Welding Code – Steel," D1.1, current edition.
4. A.W.S. "Structural Welding Code – Seismic Supplement," D1.8, current edition.
5. "Specifications for Structural Joints using ASTM A325 or A490 bolts," current edition as approved by the Research Council on Riveted and Bolted Structural Joints of the Engineering Foundation, and endorsed by the AISC.
6. All materials and work shall be subject to inspection at the mill, the fabricating shop, and at the building site. Material or workmanship not complying fully with the drawings, and/or specifications will be rejected.
7. If the inspector, through oversight or otherwise, has accepted material or work which is defective or contrary to specifications, this material or work, regardless of state of completion, may be rejected.
8. Shop drawings for steel fabrications shall be submitted for review.
9. Submittals shall include anchor bolt setting plans, erection drawings and fabrication drawings. Information shown on the shop drawings shall include, but not be limited to, the following:
    a. Anchor bolt setting plans shall show layout, anchor bolts sizes and grades, embedment, and template construction.
    b. Erection Drawings shall show layout, marking and position of each member, and field connections.

c. Fabrication Drawings shall show details of members, including sizes, grades, connections, spacing of bolts and welds, designation of Architecturally Exposed Structural Steel, and the limits of paint applications.

10. Partial submittals shall be clearly identified by the contractor.
11. The omissions from the shop and installation drawings of any materials shown on the Specifications shall not relieve the contractor of the responsibility of furnishing and installing such materials, even though such drawings may have been returned and reviewed.
12. Shop drawings and calculations for temporary shoring and bracing shall be submitted for review. The shop drawings shall show layout, size of members and connection details. Calculations shall show all stresses in members and connections, from dead, live, and lateral loads in accordance with the requirements of the C.B.C. current governing edition. Shop drawings and calculations for temporary shoring and bracing shall be stamped and signed by a civil engineer registered in the State of California.
13. Contract drawings shall not be reproduced in whole or in part. Contract drawings modified into shop drawings will be returned without review.
14. Revised submittals shall have clear indications of revised or new information. Clouding is an acceptable form of identification.
15. The compressive strength of the concrete used for all footings shall be 4000 psi.
16. The clear cover of the reinforcement bars shall be 3 inches from the sides and bottom of the footing base in accordance with CBC Section 1808A and ACI 318-14 Section 20.6.
17. Grade 60 steel shall be used for all footing reinforcement.
18. Concrete for the stem of the footing shall be 4000 psi.

19. Concrete for the footings shall be mixed in accordance with ASTM C94: Standard Specification for Ready-Mixed Concrete
20. Soil below the footings shall have a relative compaction of at least 95%
21. Concrete mixture proportions shall satisfy the durability requirements of ACI 318-14.
22. The design of the spread footings shall meet the 2016 California Building Code standards for shallow foundations.
23. All reinforcement for the footings shall be designed according to the standards of ACI 318-14 (American Concrete Institute).
24. Max pipe velocity is 8ft/s and max head loss is 10ft per 1000 ft of pipe according to standards set by City of Tulare
25. Minimum fire flow is 2500 gpm for 2 hrs. according to city of Tulare Standards
26. Max pressure is 80 psi, Minimum pressure is 35 psi and minimum residual pressure is 20 psi in event of fire according to California Plumbing Code and City of Tulare standards.
27. Pipe separation between sewer and main pipe is 10 ft horizontal and 1 ft vertical according to City of Tulare standards
28. The cover for the storm sewer will be done according to the city of Tulare masterplan.

CONCLUSION/ RECOMMENDATIONS

This report covered the final design of a New Children's Hospital for the Central Valley. Important aspects include the steel hospital building, concrete parking structure, foundations for the parking structure, a grading plan, a storm water system including a detention pond, a sewer and water system, and a transportation plan for the entire lot.

REFERENCES

1. ACI Committee 318. 2014. *Building Code Requirements for Structural Concrete (ACI 318-14): An ACI Standard: Commentary on Building Code Requirements for Structural Concrete (ACI 318R-14).* Farmington Hills, MI: American Concrete Institute.

2. Bertera, Bill. 2012. "The Envision Rating System." *ACWI.* Accessed February 18, 2018. https://acwi.gov/acwi-minutes/acwi2012/slide.lib/09_Bertera_Presentation_Harvard_06_2012_4D.pdf.

3. Das, Braja M. 2016. *Principles of Foundation Engineering (8th Edition).* Boston, MA: Cengage Learning.

4. Livesey, Katie, and Elodie Mace. 2016. "Briefing Paper: The Challenge of Measuring and Mitigating the Environmental Performance of Foundations and Substructures." *BRE Trust.* Accessed February 19, 2018. http://www.bre.co.uk/filelibrary/Briefing%20papers/98666-Foundations--Substructures-Briefing-Paper.pdf.

5. Nawy, Edward G. 2008. *Reinforced Concrete: A Fundamental Approach (6th Edition).* Pearson Prentice Hall.

6. Riversong, Robert. 2011. *Choosing Foundation Materials: A Subconscious Decision?* May 30. Accessed February 18, 2018. https://www.buildinggreen.com/blog/choosing-foundation-materials-subconscious-decision.

7. Service, Center for Alternative Technology: CAT Information. n.d. *When laying foundations, how can I minimize environmental impact?* Accessed February 18, 2018. http://info.cat.org.uk/questions/ecobuild/when-laying-foundations-how-can-i-minimise-environmental-impact/.

8. UpCodes. 2016. "California Building Code 2016 (Vol 2): Chapter 18A Soils and Foundations." *UpCodes.* Accessed February 15, 2018. https://up.codes/viewer/california/ca-building-code-2016-v2/chapter/18A/soils-and-foundations#18A.

9. Bouchart, F., Chase, D. V., Goldman, F. E., Goulter, lan, Lingireddy, S., Male, J. W., Mays, L. W., Ormsbee, L. E., Sakarya, A. B. A., and Uber, J. G. (2000). *Water distribution systems handbook*. McGraw-Hill, New York.

10. Dagostino, F. R., and Wujek, J. B. (2005). *Mechanical and electrical systems in construction and architecture*. Pearson Prentice Hall, Upper Saddle River, NJ.

11. Walski, T. M., Chase, D. V., Savic, D. A., Grayman, W., Beckwith, S., and Koelle, E. (2007). *Advanced Water Distribution Modeling and Management*.

12. *2016 California fire code: California code of regulations, Title 24, Part 9*. (n.d.). *2016 California fire code: California code of regulations, Title 24, Part 9*.

13. *2016 California plumbing code: California code of regulations, Title 24, Part 5*. (n.d.). *2016 California plumbing code: California code of regulations, Title 24, Part 5*.

14. Mays, L. W. (2001). *Water resources engineering*. Wiley, New York.

15. Mahoney, N., Tahsin, L., Singh, J., Singh, D., Makewita, N., Solario, A., Durun, G., Bagazi, A., Guttierez, M., and Barton, R. (n.d.). *Final Engineering Report by Static Engineering Group*. *Final Engineering Report by Static Engineering Group*, rep. Fall 2017

16. "Utility Master Plans." (n.d.). *Utility Master Plans | City of Tulare, CA*, <http://www.tulare.ca.gov/departments/community-development/development-services/engineering/city-standards-development-library/utility-master-plans> (Feb. 27, 2018).

17. "EPANET." (2017). *EPA*, Environmental Protection Agency, <https://www.epa.gov/water- research/epanet> (Feb. 27, 2018).

18. "Impact of Cavitation, Air on Centrifugal Pump Performance." (n.d.). *waterworld.com*, <http://www.waterworld.com/articles/print/volume-25/issue-6/departments/pump-tips-techniques/impact-of-cavitation-air-on-centrifugal-pump-performance.html> (Feb. 27, 2018).

19. "Operation and Maintenance of Water Treatment and Supply Systems." (n.d.). *Study Session 6 Operation and Maintenance of Water Treatment and Supply Systems: View as single page*, <http://www.open.edu/openlearncreate/mod/oucontent/view.php?id=80190&printable=1> (Feb. 27, 2018).

20. "Save Water With Pressure Reducing Valves - billhowe.com." (n.d.). <https://www.bing.com/cr?IG=FF856087B3D34240BFFC1AB73AF0A49D&CID=02CDA936F1C167FC04B2A295F06E665B&rd=1&h=KxEebp7NMnYjmQibBaqvHiI7k0ZVKbpVu8Yn2xjgXbY&v=1&r=https%3a%2f%2fwww.billhowe.com%2fhow-pressure-reducing-valves-save-water&p=DevEx,5066.1> (Feb. 27, 2018).

21. "What is a Variable Frequency Drive?" (n.d.). *VFD, Inverters, & AC Drives*, <https://www.vfds.com/blog/what-is-a-vfd> (Feb. 27, 2018).

22. "Variable Frequency Drive." (2000). *Energy.ca.gov*, <http://www.energy.ca.gov/process/pubs/vfds.pdf>

23. "Why It Matters." (n.d.). *Institute For Sustainable Infrastructure*, <http://sustainableinfrastructure.org/envision/why-it-matters/> (Feb. 27, 2018).

24. “Sustainable Infrastructure.” (n.d.). *Sustainable Infrastructure | CRC Research*, <https://crcresearch.org/sustainable-infrastructure/sustainable-infrastructure> (Feb. 27, 2018).

25. “Envision.” (n.d.). *Go to Institute For Sustainable Infrastructure.*, <https://sustainableinfrastructure.org/envision/> (Feb. 27, 2018).

26. "Accessible Parking Spaces." ADA Compliance Brief: Restriping Parking Spaces - 2010 Standards. Accessed February 27, 2018. https://www.ada.gov/restriping_parking/restriping2015.html.

27. "City Improvement Standards." *City Improvement Standards | City of Tulare, CA*. N.p., n.d. Web. 10 Feb. 2018.

28. Cheng, DIngXin, and Gary Hicks. "CalRecycle." *Life Cycle Cost Comparison of Rubberized and Conventional HMA in California.* May 2012. Accessed February 27, 2018. http://www.calrecycle.ca.gov/publications/Documents/1460/20131460.pdf.

29. "High-Quality, High-Value Solar Powered Products." Envision Solar. Accessed February 20, 2018. http://www.envisionsolar.com/ev-charging/.

30. Tulare, City of. 2009 *City of Tulare Sewer System Master Plan.* Technical Report, Tulare: City of Tulare

31. Tulare, City of. 2009 *City of Tulare Sewer System Specifications.* Specifications, Tulare: City of Tulare

32. City of Tulare Urban Water Management Plan. (2015). Retrieved February 27, 2018, from http://www.tulare.ca.gov/home/showdocument?id=2287

33. Cooper, J. P., & Robinson, L. (2018). *M32 computer modeling of water distribution systems*. Denver, CO: American Water Works Association

34. Cheng, DIngXin, and Gary Hicks. "CalRecycle." Life Cycle Cost Comparison of Rubberized and Conventional HMA in California. May 2012. Accessed February 27, 2018.

35. City Improvements.” *City Improvement Standards | City of Tulare, CA*. N.p., n.d. Web. 10 Feb. 2018

36. Envision. (2015). Rating System for Sustainability Infrastructure. Retrieved from http://research.gsd.harvard.edu/zofnass/files/2015/06/Envision-Manual_2015_red.pdf

37. EV ARCTM. (2017). Electric Vehicle Autonomous Renewable Charger. Retrieved from http://www.envisionsolar.com/ev-arc/

38. U.S Department of Transportation Federal Highway Administration. "Manual on Uniform Traffic Control Devices for Streets and Highways (MUTCD)." 2010. https://mutcd.fhwa.dot.gov/

39. "Why Use Asphalt Instead of Concrete." Dykes Paving. (n.d.). Accessed March 15, 2017. http://www.dykespaving.com/blog/why-use-asphalt-instead-of-concrete/

40. Zhou, H., Holikatti, S., & Vacura, P. (2014). Caltrans Use of Scrap Tires in Asphalt Rubber Products: a Comprehensive Review. *Journal of Traffic and Transportation Engineering*.

Bertera, B. (2012). *The Envision Rating System.* Retrieved February 18, 2018, from ACWI: https://acwi.gov/acwi-minutes/acwi2012/slide.lib/09_Bertera_Presentation_Harvard_06_2012_4D.pdf

Burdis, A. R. (n.d.). *Impact of Cavitation, Air on Centrifugal Pump Performance.* Retrieved from Water World: http://www.waterworld.com/articles/print/volume-25/issue-6/departments/pump-tips-techniques/impact-of-cavitation-air-on-centrifugal-pump-performance.html

California Fire code (2016). [Motion Picture].

City of Tulare. (n.d.). *Storm Drains.* Tulare : City of Tulare.

ELECTRIC VEHICLE AUTONOMOUS RENEWABLE CHARGER. (n.d.). Retrieved 02 20, 2018, from http://www.envisionsolar.com/ev-charging/

Engineers Edge. (n.d.). *Full pipe flow calcualator and equations*. Retrieved April 17, 2018, from https://www.engineersedge.com/fluid_flow/partially_full_pipe_flow_calculation/partiallyfullpipeflow_calculation.htm

EPANET. (2014, 12 14). Retrieved from EPA: https://www.epa.gov/water-research/epanet

Institute For Sustainable Infrastructure. (2018, February 21). Retrieved from http://sustainableinfrastructure.org/envision/why-it-matters/

Livesey, K., & Mace, E. (2016). *Briefing Paper: The Challenge of Measuring and Mitigating the Environmental Performance of Foundations and Substructures.* Retrieved February 19, 2018, from BRE Trust : http://www.bre.co.uk/filelibrary/Briefing%20papers/98666-Foundations--Substructures-Briefing-Paper.pdf

Mahoney, N., Singh , J., Singh, D., Tahsim, L., Makewita, N., Gutierrez, M., . . . Solario, A. (2017). *Engineering Report, Static Engineering Group.*

Mays, L. W. (2000). *Water Distribuion Systems Handbook.* McGraw-Hill.

Pasha, D. F. (2016).

Riversong, R. (2011, May 30). *Choosing Foundation Materials: A Subconscious Decision?* Retrieved February 18, 2018, from https://www.buildinggreen.com/blog/choosing-foundation-materials-subconscious-decision

Royal Roads University. (n.d.). *Sustainable Infrastructure*. Retrieved from Community reserach connections : https://crcresearch.org/sustainable-infrastructure/sustainable-infrastructure

Service, C. f. (n.d.). *When laying foundations, how can I minimize environmental impact?* Retrieved February 18, 2018, from http://info.cat.org.uk/questions/ecobuild/when-laying-foundations-how-can-i-minimise-environmental-impact/

Tulare, C. o. (2009). *2009 city of tulare storm drain.* Tulare: City of Tulare.

Walski, T. M., Chase, V. D., Savic, A. D., Grayman, W., Beckwith, S., & Koelle, E. (2007). Advanced Water Distribution Modeling And Management. In *Advanced Water Distribution Modeling And Management* (pp. 49-52).

Bertera, B. (2012). *The Envision Rating System.* Retrieved February 18, 2018, from ACWI: https://acwi.gov/acwi-minutes/acwi2012/slide.lib/09_Bertera_Presentation_Harvard_06_2012_4D.pdf

Burdis, A. R. (n.d.). *Impact of Cavitation, Air on Centrifugal Pump Performance.* Retrieved from Water World: http://www.waterworld.com/articles/print/volume-25/issue-6/departments/pump-tips-techniques/impact-of-cavitation-air-on-centrifugal-pump-performance.html

California Fire code (2016). [Motion Picture].

City of Tulare. (n.d.). *Storm Drains.* Tulare : City of Tulare.

ELECTRIC VEHICLE AUTONOMOUS RENEWABLE CHARGER. (n.d.). Retrieved 02 20, 2018, from http://www.envisionsolar.com/ev-charging/

Engineers Edge. (n.d.). *Full pipe flow calcualator and equations*. Retrieved April 17, 2018, from https://www.engineersedge.com/fluid_flow/partially_full_pipe_flow_calculation/partiallyfullpipeflow_calculation.htm

EPANET. (2014, 12 14). Retrieved from EPA: https://www.epa.gov/water-research/epanet

Institute For Sustainable Infrastructure. (2018, February 21). Retrieved from http://sustainableinfrastructure.org/envision/why-it-matters/

Livesey, K., & Mace, E. (2016). *Briefing Paper: The Challenge of Measuring and Mitigating the Environmental Performance of Foundations and Substructures.* Retrieved February 19, 2018, from BRE Trust : http://www.bre.co.uk/filelibrary/Briefing%20papers/98666-Foundations--Substructures-Briefing-Paper.pdf

Mahoney, N., Singh , J., Singh, D., Tahsim, L., Makewita, N., Gutierrez, M., . . . Solario, A. (2017). *Engineering Report, Static Engineering Group.*

Mays, L. W. (2000). *Water Distribuion Systems Handbook.* McGraw-Hill.

Pasha, D. F. (2016).

Riversong, R. (2011, May 30). *Choosing Foundation Materials: A Subconscious Decision?* Retrieved February 18, 2018, from https://www.buildinggreen.com/blog/choosing-foundation-materials-subconscious-decision

Royal Roads University. (n.d.). *Sustainable Infrastructure*. Retrieved from Community reserach connections : https://crcresearch.org/sustainable-infrastructure/sustainable-infrastructure

Service, C. f. (n.d.). *When laying foundations, how can I minimize environmental impact?* Retrieved February 18, 2018, from http://info.cat.org.uk/questions/ecobuild/when-laying-foundations-how-can-i-minimise-environmental-impact/

Tulare, C. o. (2009). *2009 city of tulare storm drain.* Tulare: City of Tulare.

Walski, T. M., Chase, V. D., Savic, A. D., Grayman, W., Beckwith, S., & Koelle, E. (2007). Advanced Water Distribution Modeling And Management. In *Advanced Water Distribution Modeling And Management* (pp. 49-52).

APPENDICES

APPENDIX A. CONSTRUCTION PLANS AND DRAWINGS

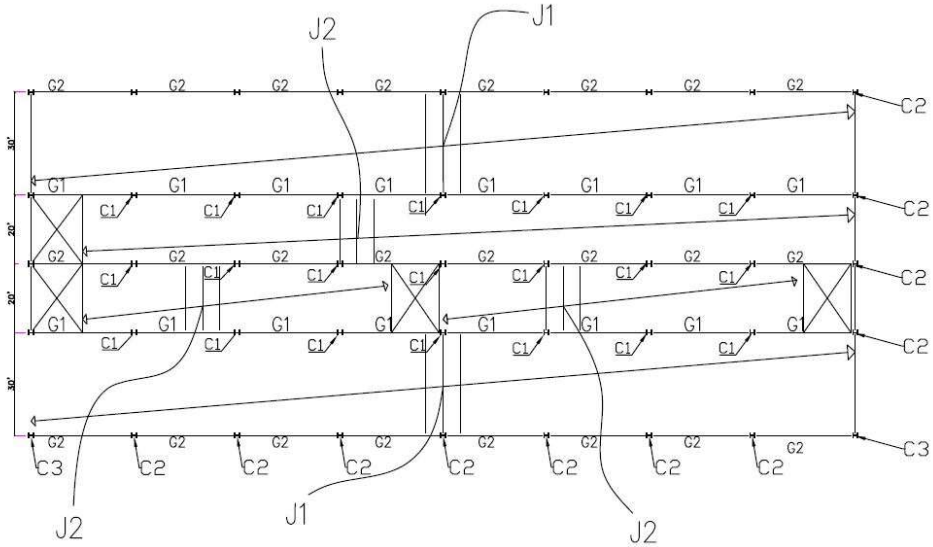


Figure 32: Hospital wing framing plan.

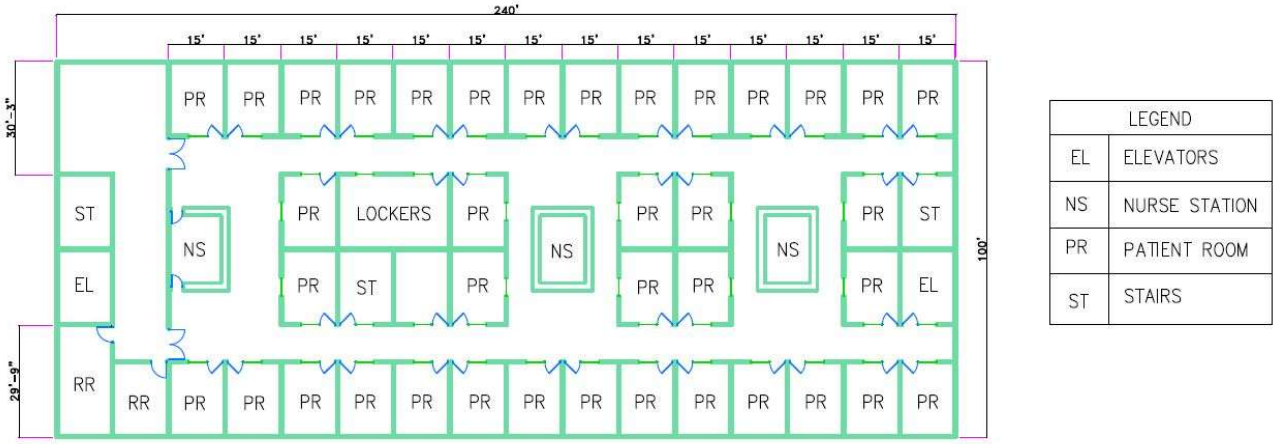


Figure 33: Floor plan for each floor in the hospital wing.

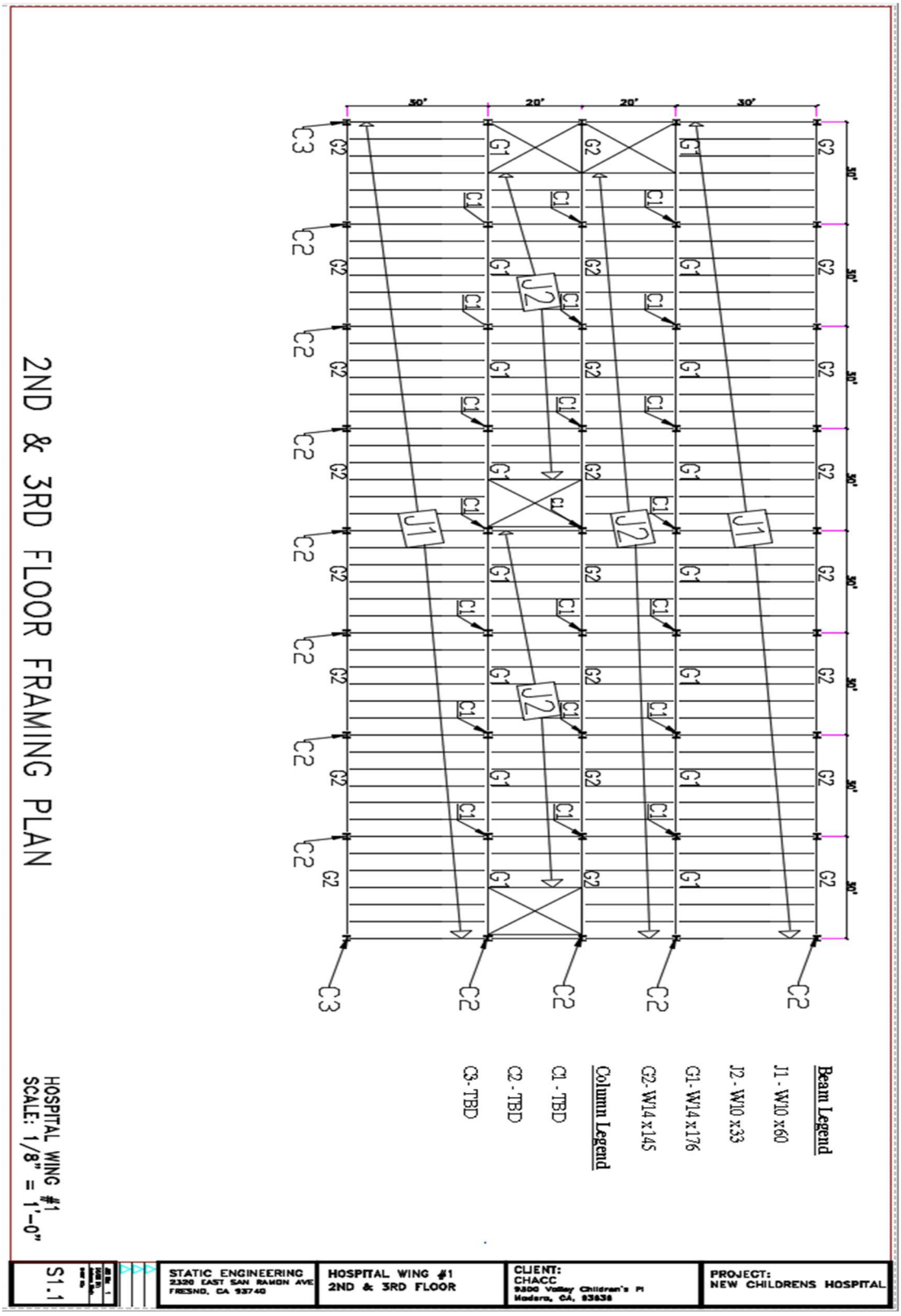


Figure 34: Hospital wing Floor Framing Plan.

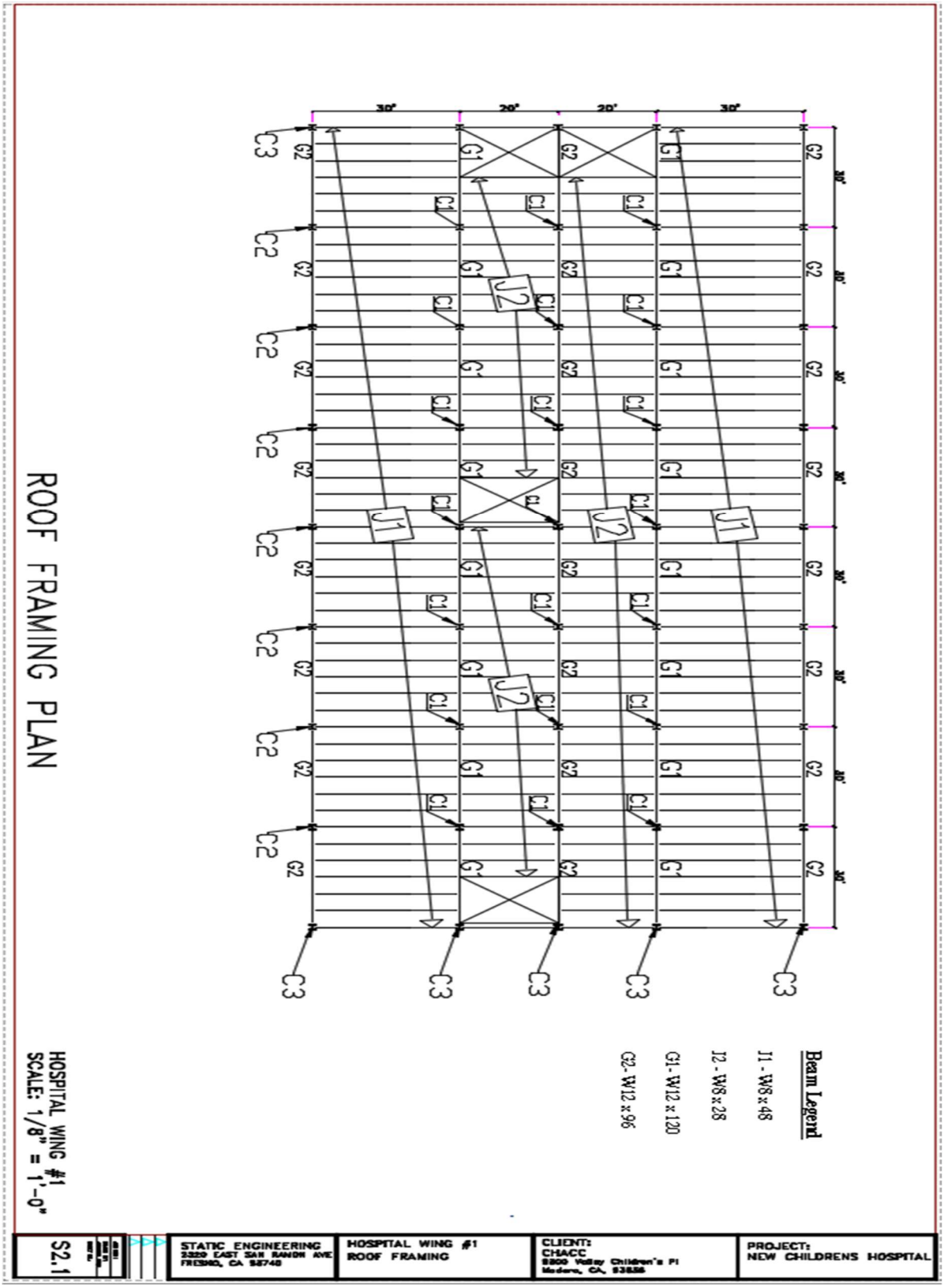


Figure 35: Hospital wing Roof Framing Plan.

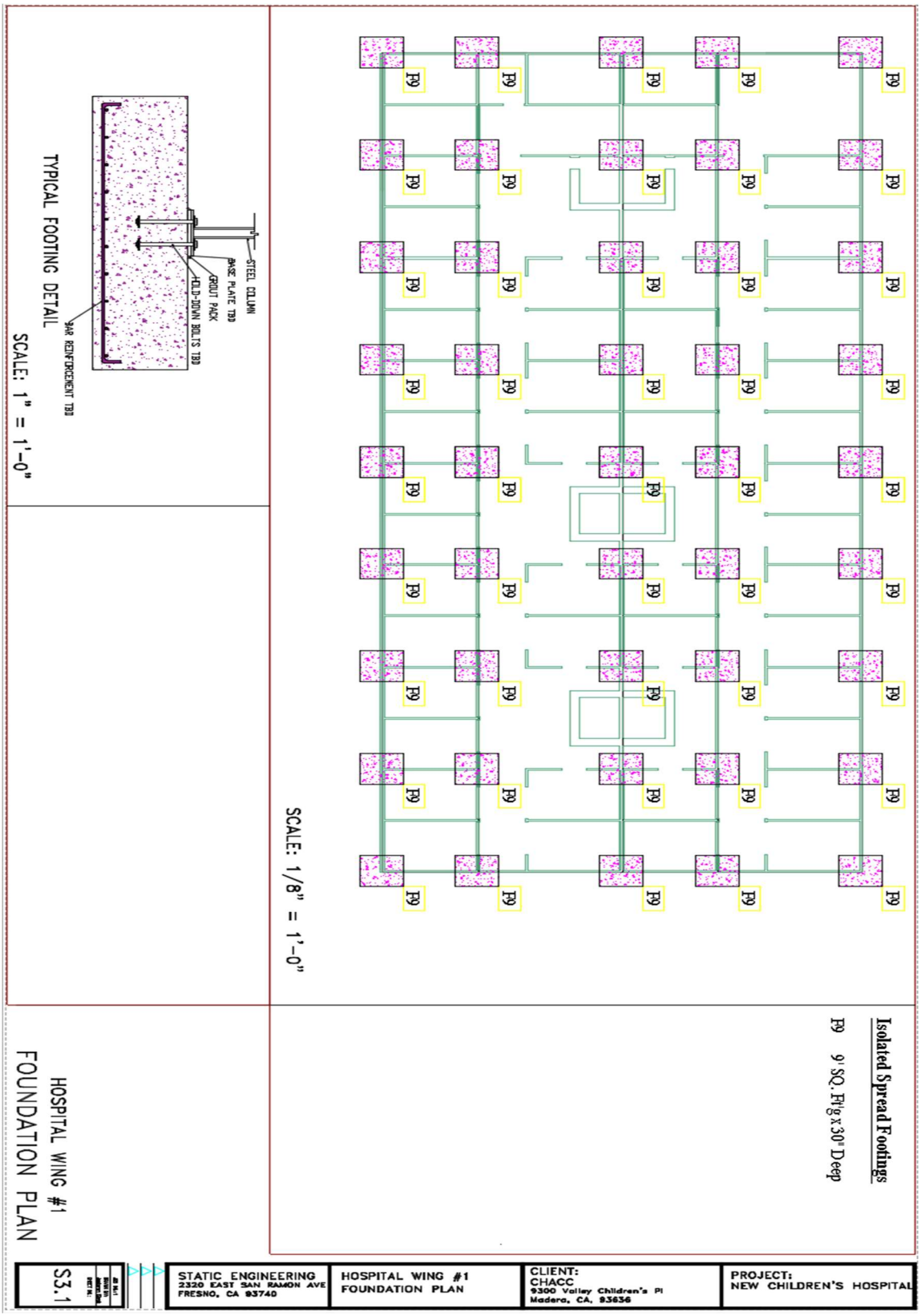


Figure 36: Hospital wing Foundation Plan.

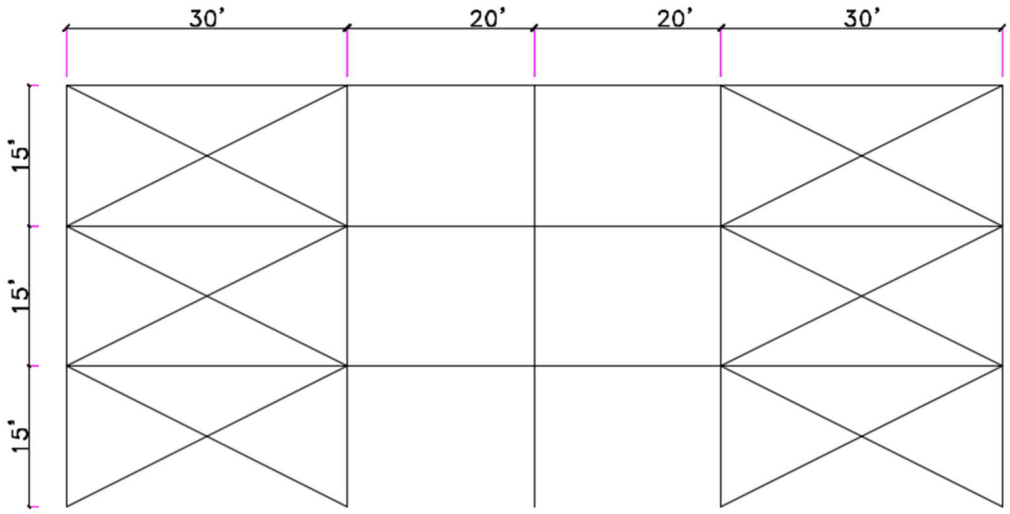


Figure 37: Hospital wing brace frame elevation.

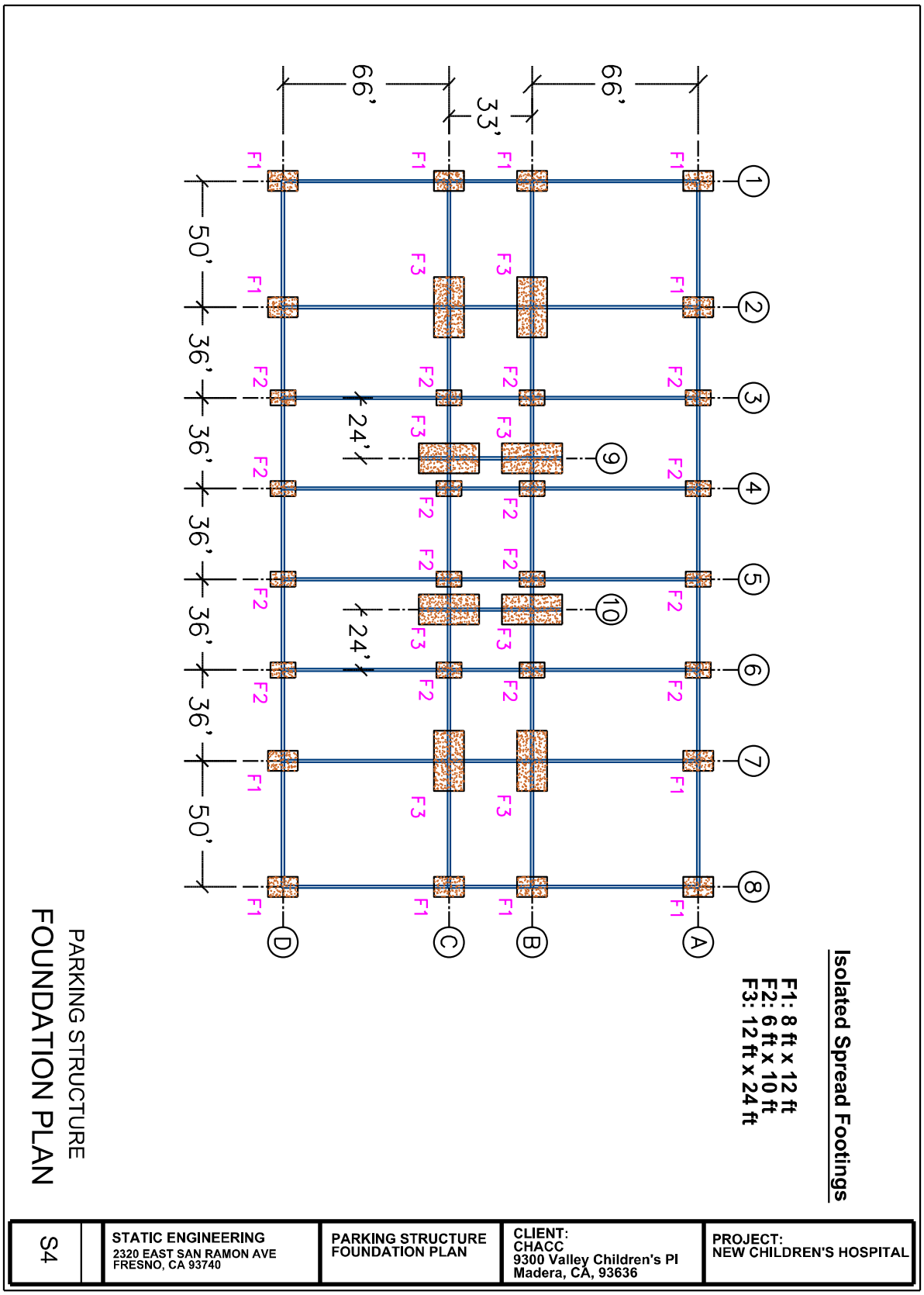
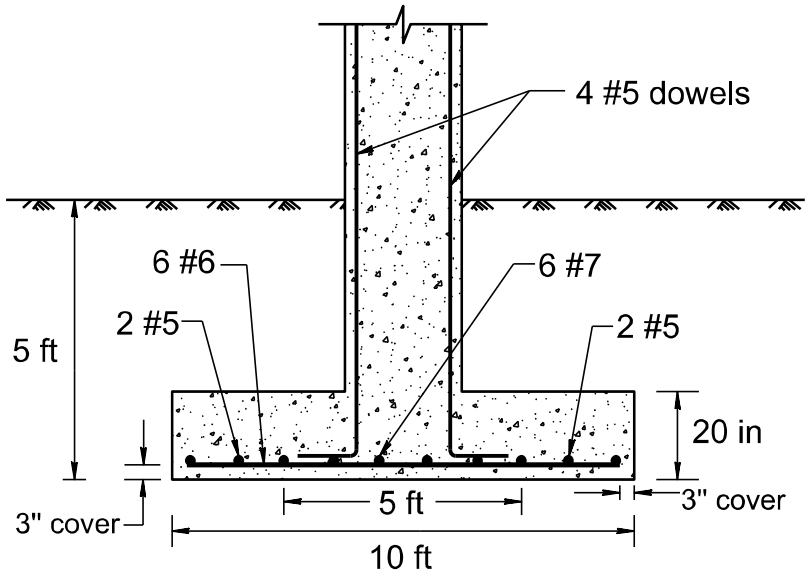
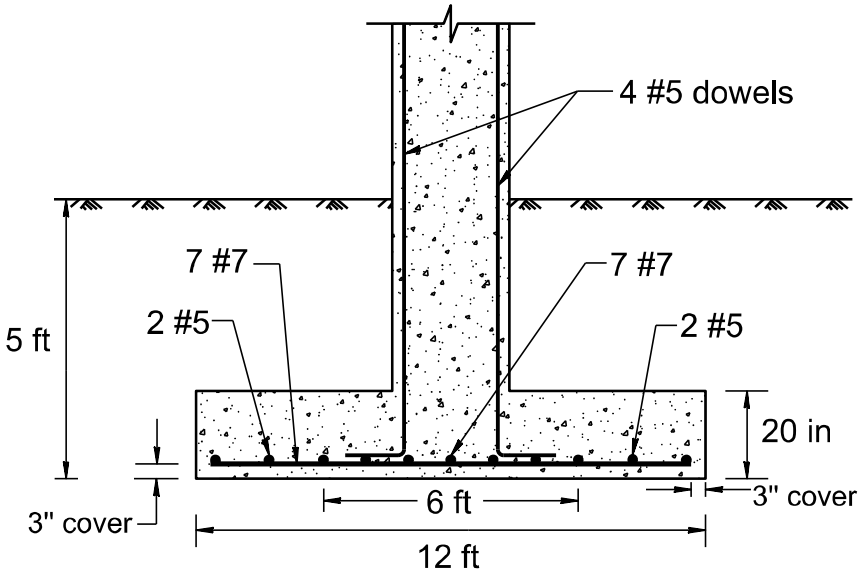


Figure 38: Parking Structure Foundation Plan



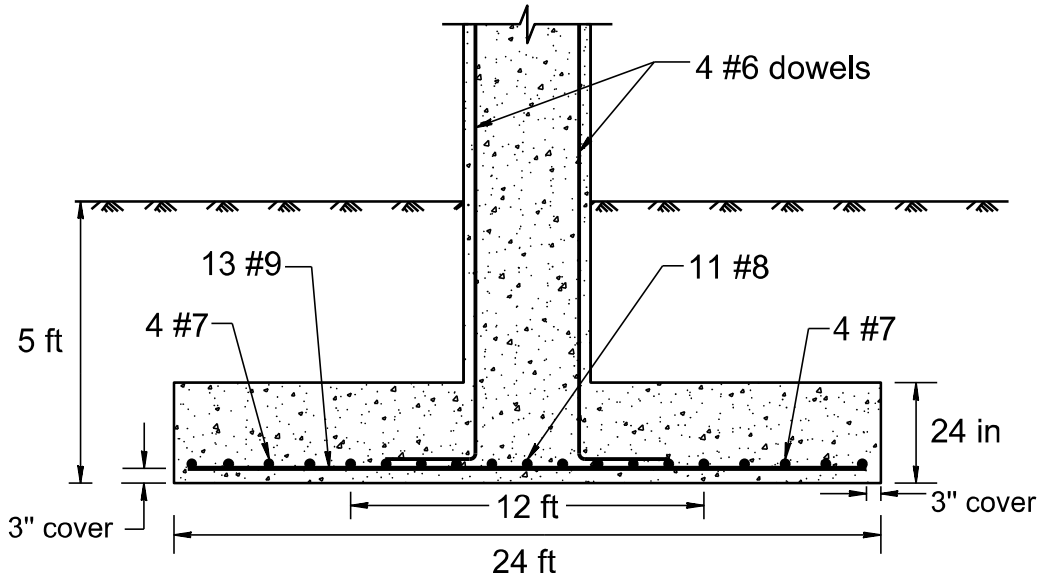
Footing Type F2



Footing Type F1

S5		STATIC ENGINEERING 2320 EAST SAN RAMON AVE FRESNO, CA 93740	PARKING STRUCTURE FOOTING DETAILS	CLIENT: CHACC 9300 Valley Children's Pl Madera, CA 93636	PROJECT: NEW CHILDREN'S HOSPITAL

Figure 39: Parking Structure Footing Details



Footing Type F3

SPECIFICATIONS

1. Compressive strength of concrete used for all footings shall be 4000 psi.
2. Clear cover of the reinforcement bars shall be 3 inches from the sides and bottom of the footing base in accordance with CBC Section 1808A and ACI 318-14.
3. Grade 60 steel shall be used for all reinforcement.
4. Concrete for the footings shall be mixed in accordance with ASTM C94: Standard Specification for Ready-Mixed Concrete.
5. Soil below the footings shall have a relative compaction of at least 95%.
6. Concrete mixture proportions shall satisfy ACI 318-14 durability requirements.
7. Design of the spread footings shall meet the 2016 California Building Code standards for shallow foundations.
8. All reinforcement shall be designed according to the standards of ACI 318-14.

S5	STATIC ENGINEERING 2320 EAST SAN RAMON AVE FRESNO, CA 93740	PARKING STRUCTURE FOOTING DETAILS	CLIENT: CHACC 9300 Valley Children's Pl Madera, CA 93636	PROJECT: NEW CHILDREN'S HOSPITAL

Figure 40: Parking Structure Footing Details

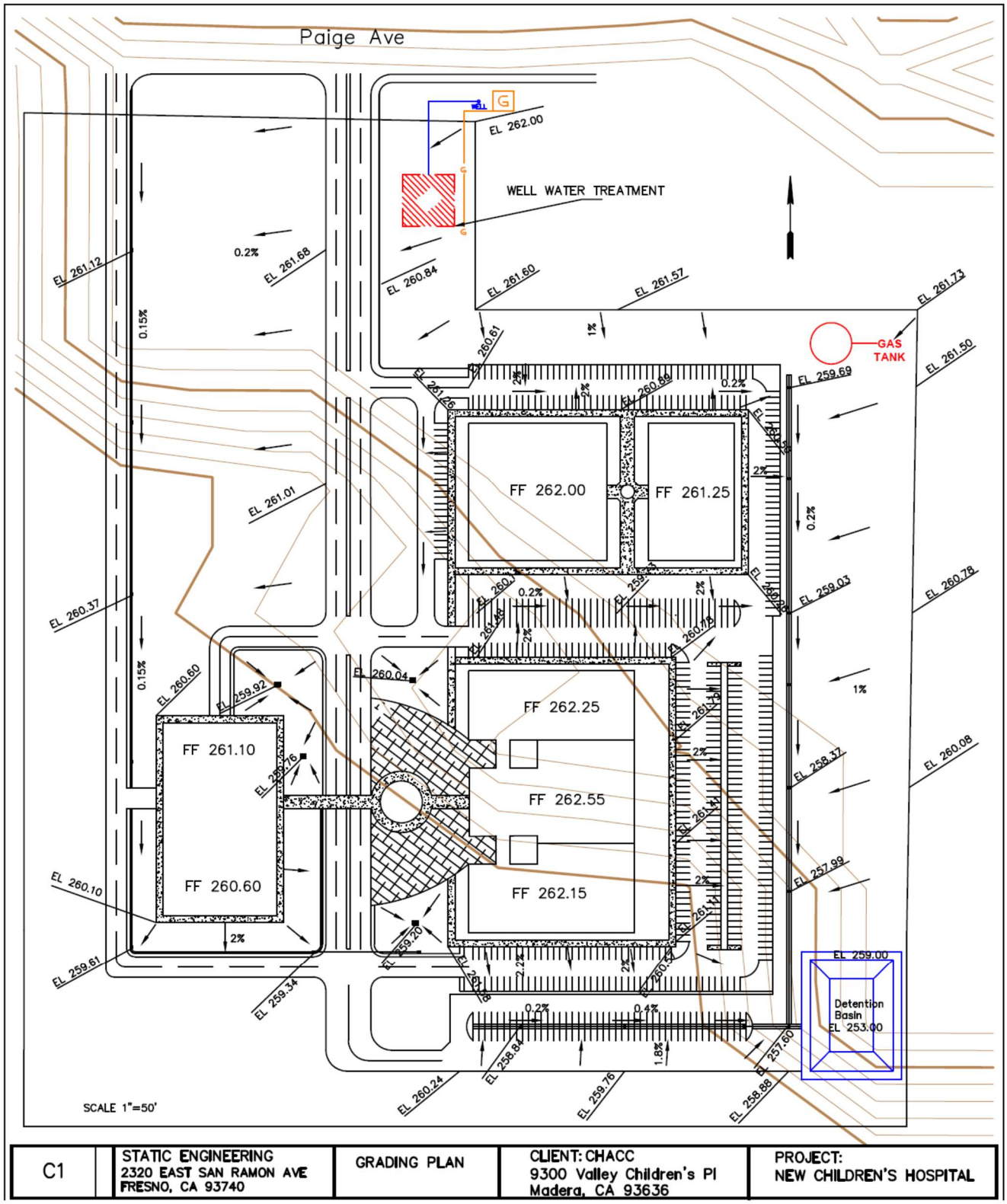


Figure 41: Grading Plan

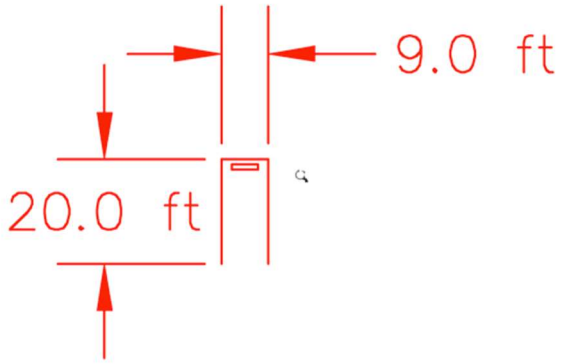


Figure 42: Typical Parking Design

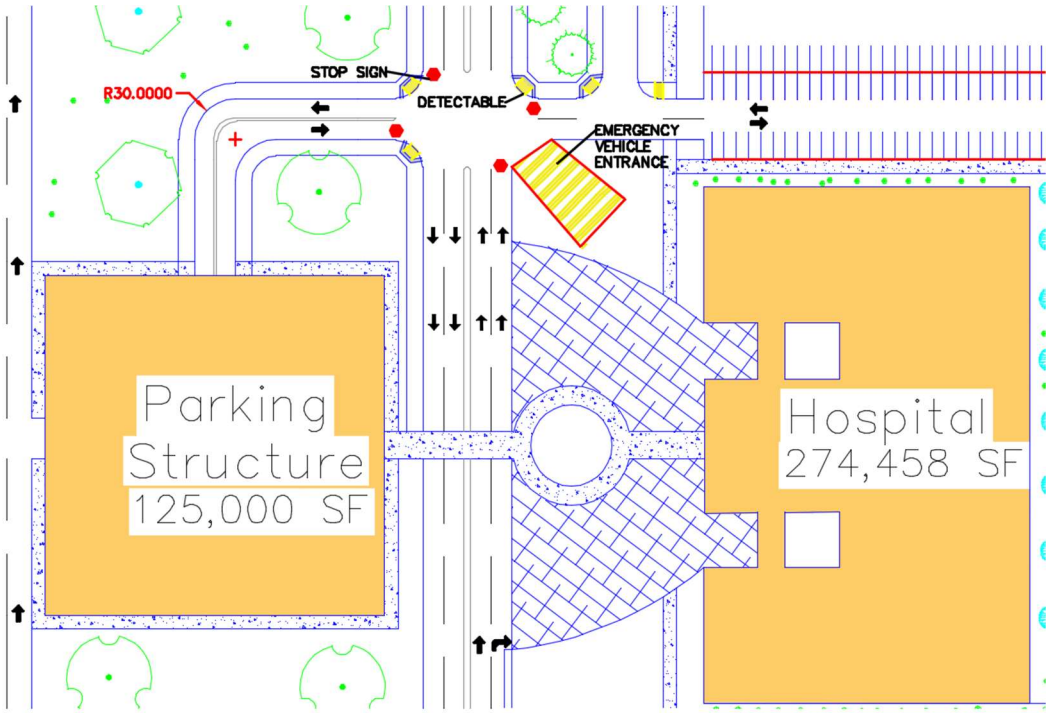


Figure 43: Intersection within the site

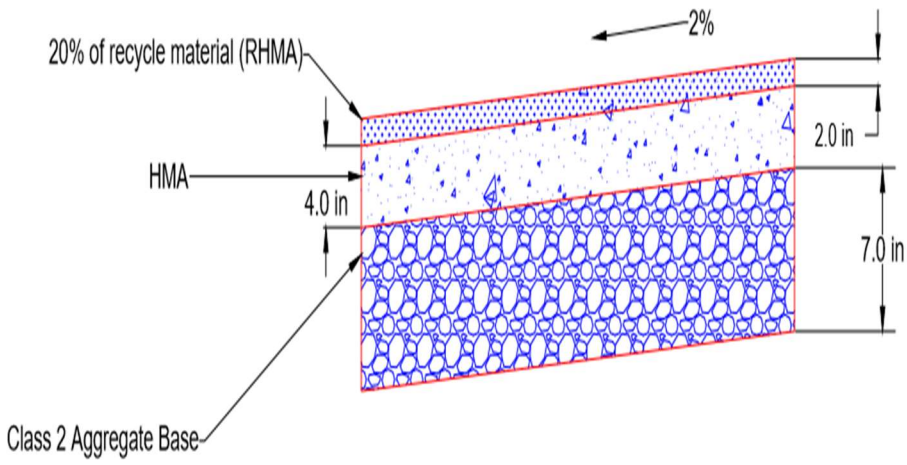


Figure 44: Typical pavement cross-section

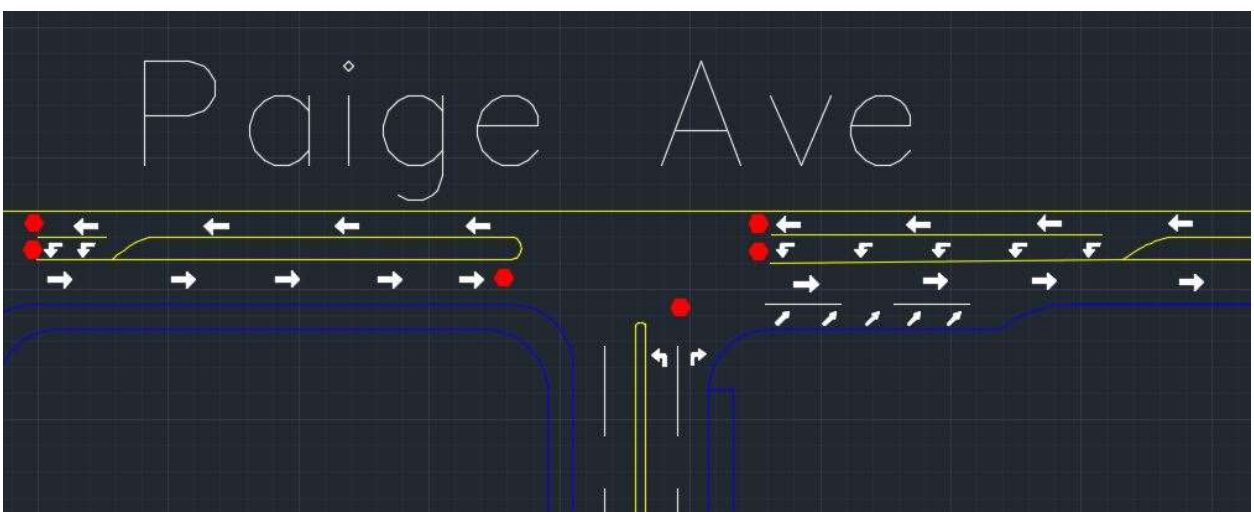


Figure 45: Median Crossover

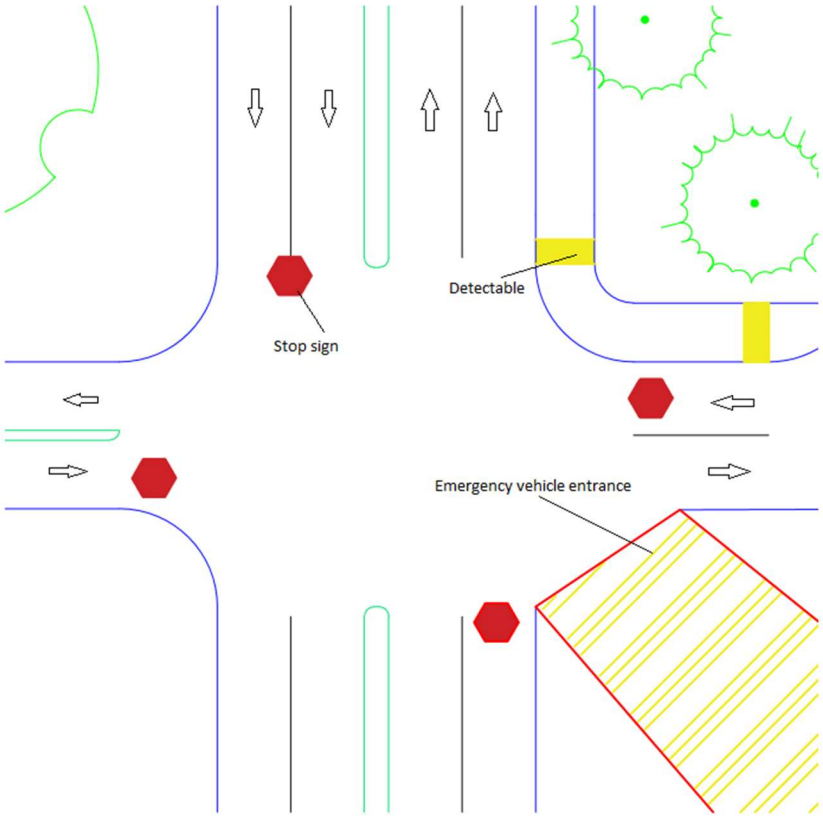


Figure 46: Intersection with Stop Sign by AutoCAD

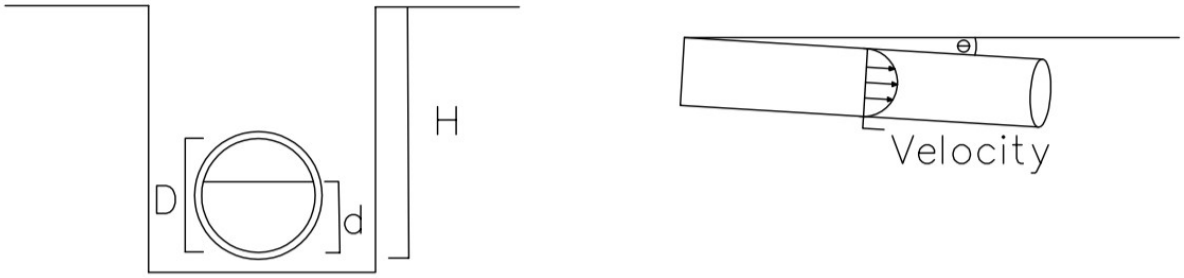


Figure 47: Sanitary Sewer Pipe

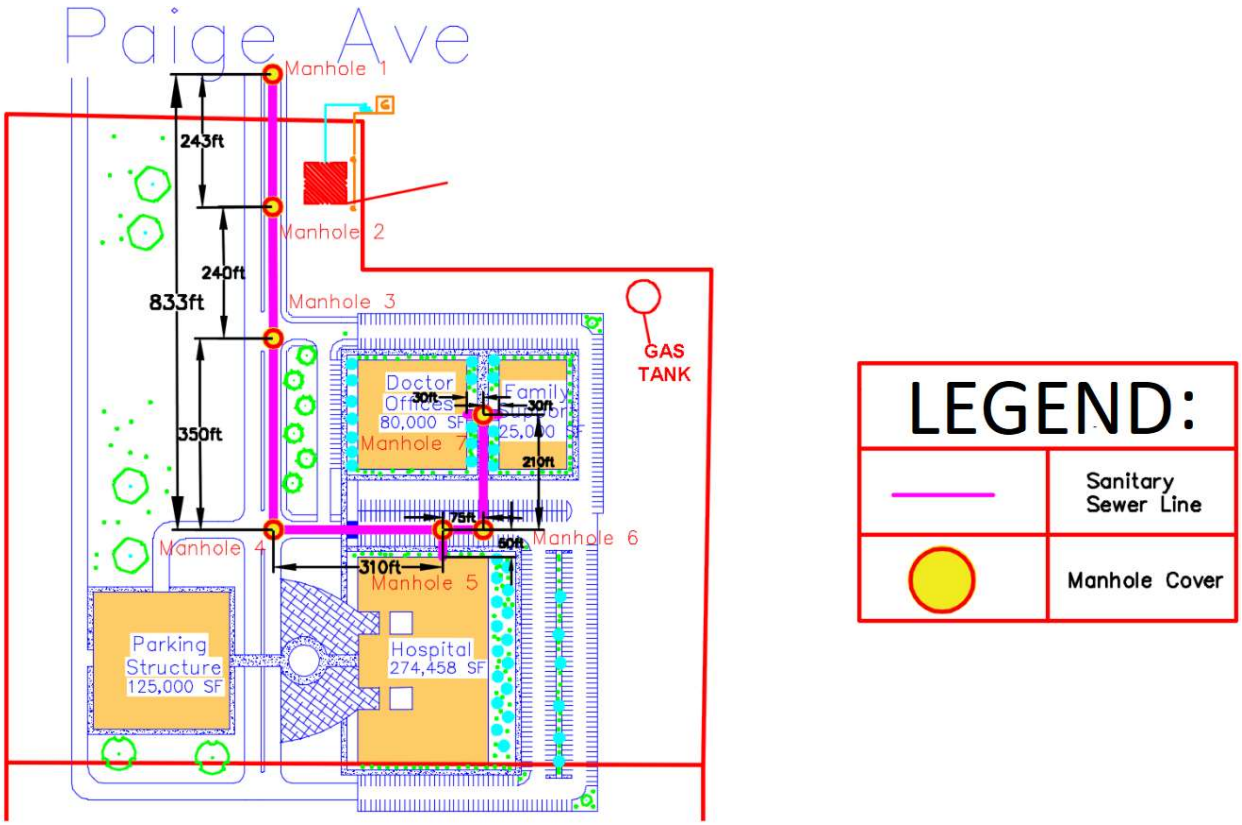


Figure 48: Layout of Sanitary Sewer Line

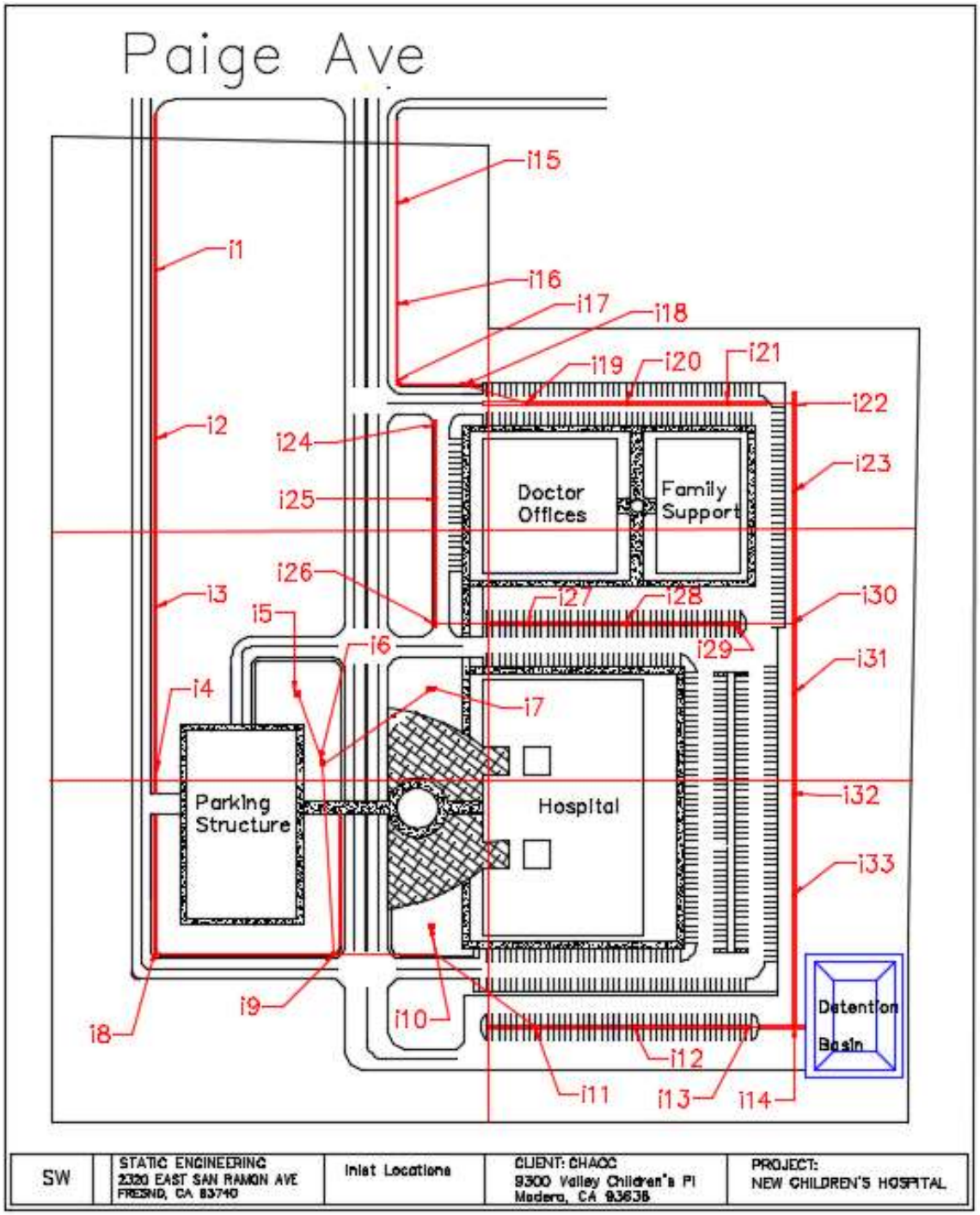


Figure 49: Location of planned, inlets, detention pond and storm sewer pipes

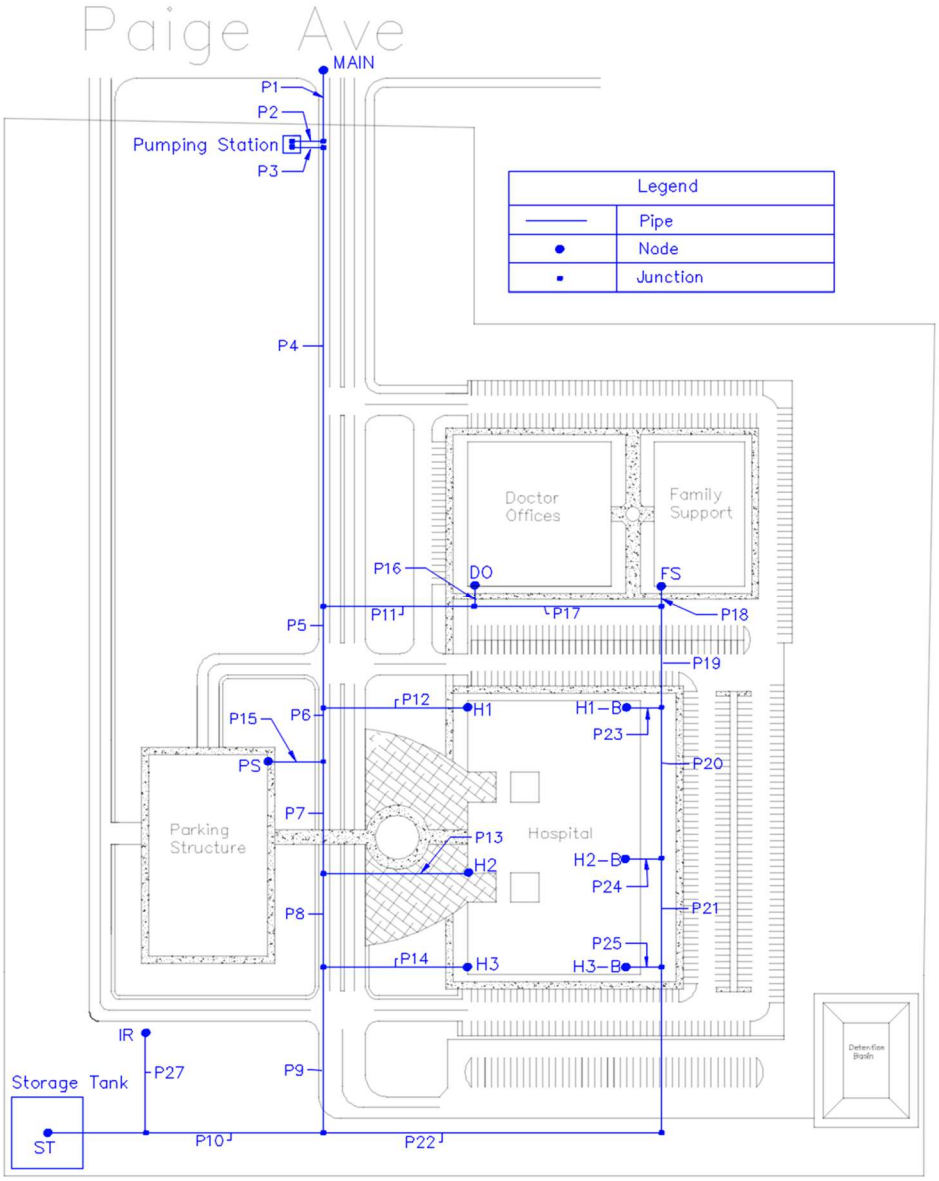


Figure 50: Layout of Water Distribution System

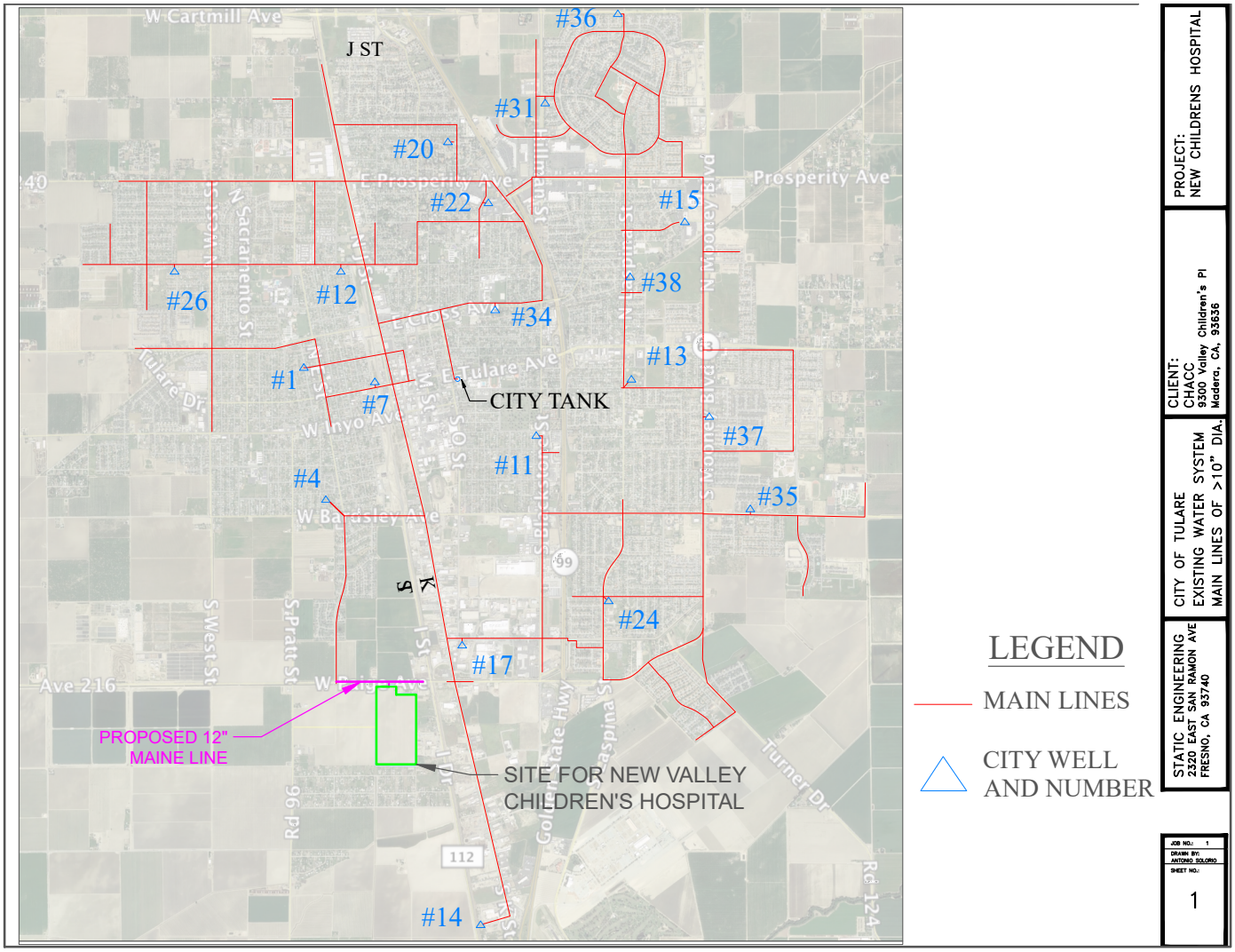
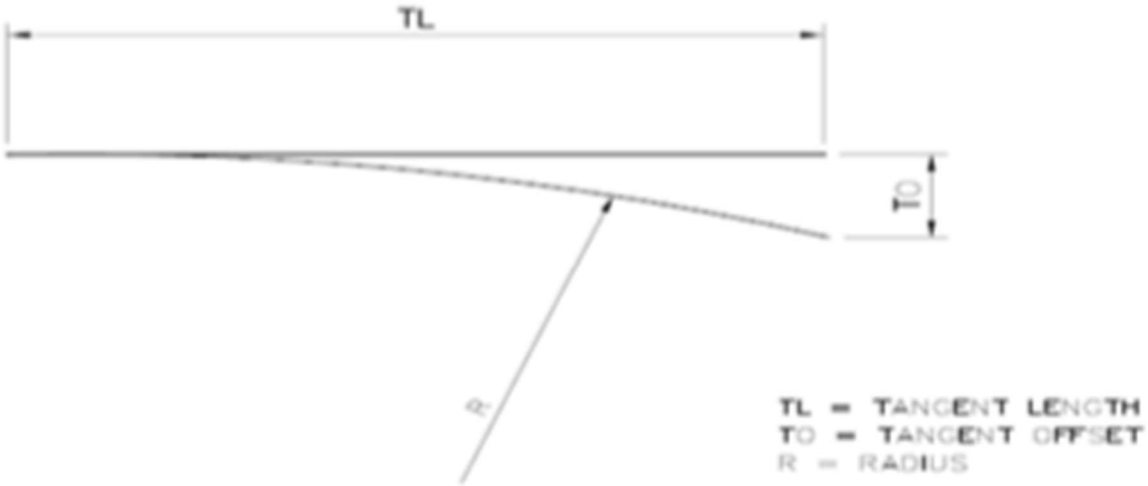


Figure 51*: AutoCAD Drawing of the Water Distribution System for the City of Tulare: Only Main Service Lines with Diameters of 10" or greater.*

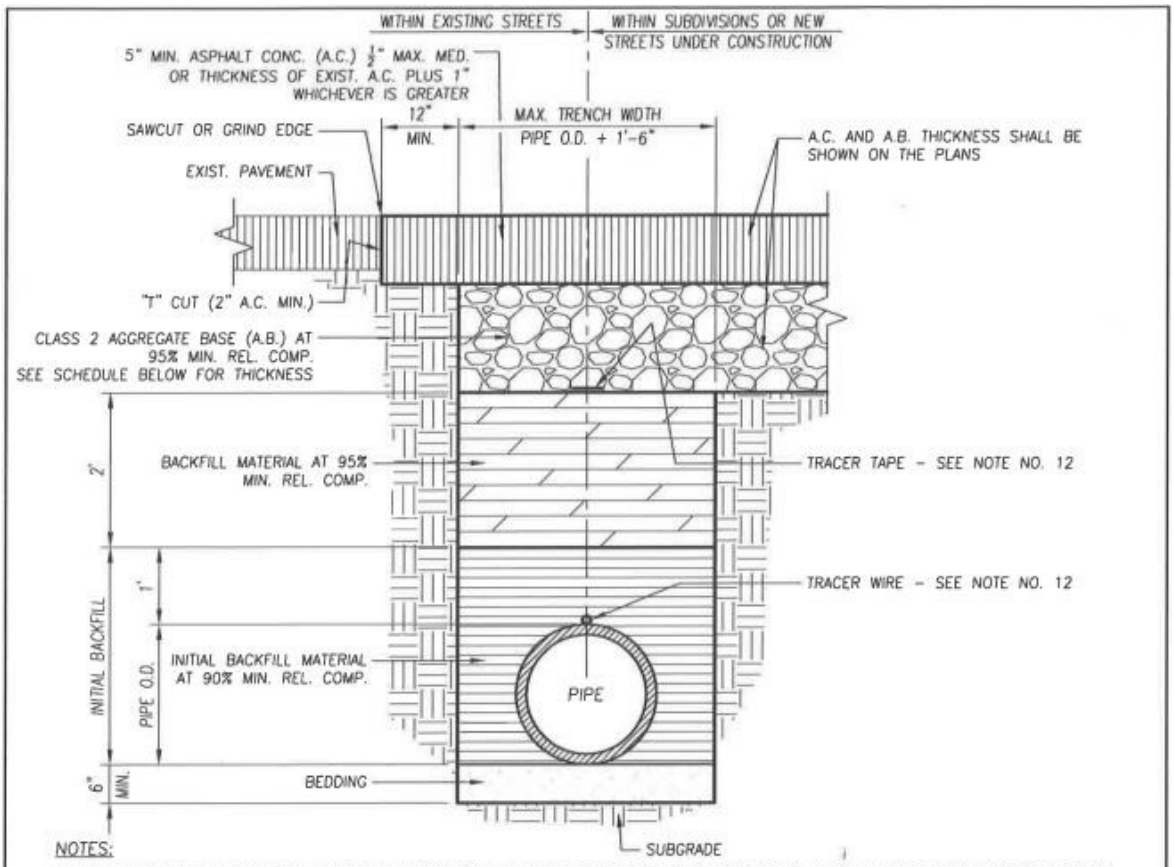
TABLE 16-2 Maximum Deflection for Polyvinyl Chloride Pipes



FLEXIBLE PIPELINE CURVE GEOMETRY

Nominal Pipe Size (Inches)	Maximum Tangent Offset - TO (Feet)											Minimum Radius of Curvature - R (Feet)
	Tangent Length - TL (Feet)											
	20	40	60	80	100	120	140	160	180	200	220	
6	1.0	4.0	9.2	16.7	26.8	40.0	57.2	80.	112.8	200.0	-	200
8	0.8	3.2	7.3	13.1	20.8	30.7	42.9	57.9	76.5	100.0	131.3	250
10	0.7	2.7	6.1	10.9	17.2	25.0	34.7	46.2	60.0	76.4	96.0	300
12	1.5	2.1	4.8	8.6	13.6	19.7	27.1	35.8	46.0	57.3	71.3	375

Figure 52: ***City of Tulare Specification for Maximum Deflection for Polyvinyl Chloride Pipes***



NOTES:

1. PREPARATION OF THE FOUNDATION TO RECEIVE MATERIAL SHALL BE THE RESPONSIBILITY OF THE CONTRACTOR, AND SHALL BE PLACED AS REQUIRED BY THE SOILS REPORT.
2. THE BEDDING AND INITIAL BACKFILL MATERIAL SHALL CONSIST OF A MINIMUM 30% SAND EQUIVALENT.
3. INITIAL BACKFILL FOR AREA AROUND THE PIPE AND 12" ABOVE THE TOP OF THE PIPE SHALL BE HAND-COMPACTED TO 90% MINIMUM RELATIVE COMPACTION.
4. HYDRO-HAMMERS SHALL NOT BE USED.
5. BACKFILL MATERIAL SHALL BE PLACED IN 6" MAX. LIFTS OF PROPERLY MOISTENED AND COMPACTED MATERIAL.
6. EXISTING PAVEMENT SURFACING SHALL BE CUT ("T" CUT) AS SHOWN AFTER TRENCH IS BACKFILLED AND AGGREGATE BASE MATERIAL IS IN PLACE AND COMPACTED AS REQUIRED.
7. ASPHALTIC PAINT BINDER, "TACK COAT", SHALL BE APPLIED TO ALL ADJACENT VERTICAL CONTACT SURFACES OF EXISTING PAVEMENT AND CONCRETE IMMEDIATELY PRIOR TO TRENCH PAVING. THE AREA MUST BE CLEANED BEFORE THE TACK COAT IS APPLIED.
8. AFTER TRENCH PAVING HAS BEEN COMPLETED APPLY A SEAL COAT TO ALL TRENCH EDGES WHICH ABUT EXISTING PAVEMENT AS DIRECTED BY THE PUBLIC WORKS INSPECTOR.
9. IN AREAS OF NO PAVEMENT, OR PHASED SUBDIVISIONS, OR WHERE NO A.C. PAVEMENT AND AGGREGATE BASE IS REQUIRED, THE BACKFILLED MATERIAL SHALL BE COMPACTED TO 92% MINIMUM RELATIVE COMPACTION.
10. PLACE AND MAINTAIN 2" MIN. TEMPORARY A.C. COLD MIX IN TRENCH IMMEDIATELY AFTER BACKFILL AND PRIOR TO OPENING UP TO TRAFFIC.
11. CLASS 2 AGGREGATE BASE MIN. THICKNESS SHALL BE:

    STATE HIGHWAYS.....AS DIRECTED BY CALTRANS
    ALLEYS.....4"
    LOCAL STREETS.....6"
    COLLECTOR STREETS.....8"
    MAJOR ARTERIALS.....10"

12. NON-METALLIC SEWERLINES SHALL BE INSTALLED WITH A MAGNETIC TRACER TAPE AS SHOWN ABOVE. ALL WATERLINES AND SEWER FORCE MAINS SHALL BE INSTALLED WITH 10-GA. INSULATED SOLID COPPER TRACER WIRE, SECURED TO THE CENTER OF THE TOP OF THE PIPE WITH TAPE AT 5' INTERVALS. ADJACENT TO EA. MANHOLE AND LIFT STATION, A TU-G05 UTILITY WELL SHALL BE INSTALL PER DRAWING NO. 6030 FOR ACCESS TO FORCE MAIN TRACER WIRE. TRACER WIRE TO BE INSTALLED PER DRAWING NO. 6325 WIRE SHALL BE CONTINUOUS AND TESTED FOR CONTINUITY. WIRE TO SERVICES, FIRE LINES, ETC. SHALL BE JOINED TO WIRE ON MAIN. ALL JOINTS IN WIRE TO BE SOLDERED AND WRAPPED WITH ELECTRICAL TAPE.

REVISIONS	DATE		CITY OF TULARE PUBLIC IMPROVEMENT STANDARD	
		CITY OF TULARE INCORPORATED APRIL 5, 1888	TRENCH BACKFILL / PATCH PAVING	DRAWING NO.: 6020
			Approved By: Michael W. Miller Date: 11/15/16 City Engineer	1 OF 1

Figure 53: ***Tulare City Standard for Trench Backfill***

APPENDIX B. EXCEL AND HAND CALCULATIONS

Name: Nicole Mahoney Project: Children's Hospital Date: 2/20/2018

Load Tables

Interior Walls	Psf
Steel Studs , 1/2" gypsum board each side	8
Insulation	4
Miscellaneous	3
Dead:	15

Exterior Walls	Psf
Steel Studs , 1/2" gypsum board each side	8
Insulation	4
Slate	10
Miscellaneous	3
Dead:	25

Roof	Psf
W 10 x 30 Joists	1.5
Suspended steel channel system	2
Suspended metal lath and gypsum plaster	10
Insulation	4
Roofing	3
Mechanical Duct	4
Miscellaneous	2.5
Dead:	27

Floor	Psf
W 10 x 33 Joists	1.65
Suspended steel channel system	2
Suspended metal lath and gypsum plaster	10
Insulation	4
6" Concrete slab	75
Mechanical Duct	4
Miscellaneous	2.5
Dead:	99.15

Live Loads	Psf
Corridor above first floor	80
Patient Rooms	40
Operating rooms, laboratories	60
Live Roof	20

Name: Nicole Mahoney Project: Children's Hospital Date: 2/14/2018

Seismic Base Shear

Risk Category:	IV	[ASCE 7-10 Table 1.5-1]
I_E:	1.5	[ASCE 7-10 Table 1.5-2]
S_{ds}	0.535	From USGS Seismic Design Report
S_{d1}	0.326	
T_L	12	
C_t	0.028	[ASCE 7-10 Table 12.8-2]
x	0.8	[ASCE 7-10 Table 12.8-2]
h_n	45	
T	0.588	[ASCE 7-10 Eq. 12.8-7]
R	8	[ASCE 7-10 Table 12.2-1]
C_s	0.100	[ASCE 7-10 Eq. 12.8-2]
$C_{s\ max}$	0.104	[ASCE 7-10 Eq. 12.8-3]
$C_{s\ min}$	0.035	[ASCE 7-10 Eq. 12.8-5]
$C_{s\ actual}$	0.100	

C_s	0.100	[LRFD]
C_s	0.070	[ASD]

Name: Nicole Mahoney Project: Children's Hospital Date: 2/20/2018

Seismic Vertical Distribution - Hospital Wing

k	1.044	[ASCE 7-10 Section 12.8.3]
W	6097.5	kips
Σwh^k	180044.3	

ASCE 7-10 Equation 12.8-11

$$F_x = C_{vx}V$$

ASCE 7-10 Equation 12.8-12

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$$

First Floor

h_1	15	ft
w_1	2634.6	kips

Second Floor

h_2	30	ft
w_2	2661	kips

Third Floor

h_3	45	ft
w_3	801.9	kips

	C_{vx}	V
First Floor	0.247	151.3
Second Floor	0.515	315.2
Third Floor	0.237	145.1

Name: Nicole Mahoney Project: Children's Hospital Date: 2/14/2018

Seismic Weigh Up - Hospital Wing

						Weight (kips)
Roof:	(240' x 100')					
	24000 sf		0.027	ksf		648
Floor:	2 x (240' x 100')					
	48000 sf		0.099	ksf		4759.2
Perimeter Walls:						
					ksf	
N/S	2 x (240') x (45' - 7.5')			18000	0.025	450
E/W	2 x (100') x (45' - 7.5')			7500	0.025	187.5
						637.5
					ksf	
Interior Walls:	2 x 1760					
	3520	sf			0.015	52.8
					Total:	6097.5

	W (kips)	V (kips)		
V	6097.5	0.100	611.7	[LRFD]
		0.070	428.2	[ASD]

Name:	Jaskaran Singh	Date:
Project:	Children's Hospital	2/14/2018
	Loading	
	Minimum Uniform distributed loads (dead)	Psf
Asce7-16	Mechanical duct allowance	4
	Suspended steel channel system	2
	suspended metal lath and gypsum plaster	10
	three-ply ready roofing	1
	insulation	4
	subflooring	3
	Terrazzo flooring type	19
	movable steel partitions	4
Exterior wall	2x4 @16-in, 5/8-in gypsum, isulated, 3/8-in siding	11
	2x6 @16-in, 5/8-in gypsum, isulated, 3/8-in siding	12
	Concrete flooring / roofing 6in. Thick	75
		3
AISC Design Example	Beams	4
	Joist	3
	Misc	5
	wall load	55
	ribbon window glazing sysem	15

Name:	Jaskaran Singh				
Project:	Children's Hospital	Date:	2/14/2018		
	Loading				
ASCE 7-10					
Table 1.5-1	**Risk Category**				
	4				
Table 1.5-2	**Wind Importance factor**				
	1				
Table 4-1	**Minimum Uniformly distributed loads (live)**				
	Hospital	(psf)			
	Operting rooms, Laboratories	60			
	Patients rooms	40			
	corridors above first floor	80			
	Roof Live				
	Ordinary Flat, pitched, and curved roofs	20			
Asce 7-10	**Wind Loading**				
Chapter 28 Part 2	Basic Wind speed V (mph)	115			
	Directionality Factor Kd	0.85			
	Exposure Categories	C			
	Topographic Factor Kzt	1			
	Gust Factor	0.85			
	Adjustment factor lamda	1.572			
	Ps30 simipified wind pressures (Psf)				
	Horizontal Pressures	A	B	C	D
		21	-10.9	13.9	-6.5
	Vertical Pressures	E	F	G	H
		-25.2	-14.3	-17.5	-11.1
	Ps Design Wind loads (psf)	33.012	-17.1348	21.8508	-10.218

Name:	Jaskaran Singh			
Project:	Children's Hospital	Date:	2/14/2018	
Floor Beams	G1	G2	J1	J2
Member Length(ft)	30	30	30	20
Trib Width (ft)	25	20	5	5
Dead Load (psf)	125	125	125	125
Live Load (psf)	80	80	80	80
Total LRSD (kip/ft)	6.95	5.56	1.39	1.39
Size	W14x176	W14x145	W10X60	W10X33
Column on First floor	C1	C2	C3	
Member length (ft)				
Trib Area (ft2)	750	450	225	
Dead Load (psf)	125	125	125	
Live Load (psf)	80	80	80	
Total LRSD (kips)	531	318.6	159.3	
Roof Beams	G1	G2	J1	J2
Member Length(ft)	30	30	30	20
Trib Width (ft)	25	20	5	5
Dead Load (psf)	100	100	100	100
Live Load (psf)	20	20	20	20
Total LRSD (kip/ft)	3.8	3.04	0.76	0.76
Size	W12x120	W12x96	W8x48	W8x28
roof load	114	68.4	34.2	
third floor	208.5	125.1	62.55	
second floor	208.5	125.1	62.55	

LRFD:	1	1.4D
	2	1.2D + 1.6L + 0.5(Lr or S or R)
	3	1.2D + 1.6(Lr or S or R) + (L or 0.5W)
	4	1.2D + 1.0W + L + 0.5(Lr or S or R)
	5	1.2D + 1.0E + L + 0.2S
	6	0.9D + 1.0W
	7	0.9D + 1.0E

B1	For roof:			LRFD Load Combinations	PSF	PLF
	Live	40	psf	Service	134.1	1341
	Dead	85	psf	1	119.0	1428.0
	Rain	0	psf	2	166.0	1992.0
	Snow	0	psf	3	142.0	1704.0
	Live roof	0	psf	4	142.0	1704.0
	Wind	0	psf	5	151.1	1813.2
	Seismic	9.1	psf	6	76.5	918.0
				7	85.6	1027.2
	Trib Width	12	LF			

B2	For roof:			LRFD Load Combinations	PSF	PLF
	Live	40	psf	Service	130.1	1301
	Dead	81	psf	1	113.4	1134.0
	Rain	0	psf	2	161.2	1612.0
	Snow	0	psf	3	137.2	1372.0
	Live roof	0	psf	4	137.2	1372.0
	Wind	0	psf	5	146.3	1463.0
	Seismic	9.1	psf	6	72.9	729.0
				7	82.0	820.0
	Trib Width	10	LF			

B3	For roof:			LRFD Load Combinations	PSF	PLF
	Live	40	psf	Service	130.1	1301
	Dead	81	psf +	1	113.4	1501.5
	Rain	0	psf	2	161.2	1927.0
	Snow	0	psf	3	137.2	1687
	Live roof	0	psf	4	137.2	1687
	Wind	0	psf	5	146.3	1778
	Seismic	9.1	psf	6	72.9	965.25
				7	82	1056.25
	Trib Width	10	LF			

B4	For roof:			LRFD Load Combinations	PSF	PLF
	Live	40	psf	Service	130.1	1301
	Dead	81	psf	1	113.4	1134.0
	Rain	0	psf	2	161.2	1612.0
	Snow	0	psf	3	137.2	1372.0
	Live roof	0	psf	4	137.2	1372.0
	Wind	0	psf	5	146.3	1463.0
	Seismic	9.1	psf	6	72.9	729.0
				7	82.0	820.0

	Trib Width	10	LF			

B5	For roof:			LRFD Load Combinations	PSF	PLF
	Live	40	psf	Service	130.1	1301
	Dead	81	psf	1	113.4	1134.0
	Rain	0	psf	2	161.2	1612.0
	Snow	0	psf	3	137.2	1372.0
	Live roof	0	psf	4	137.2	1372.0
	Wind	0	psf	5	146.3	1463.0
	Seismic	9.1	psf	6	72.9	729.0
				7	82.0	820.0
	Trib Width	10	LF			

G1	For roof:			LRFD Load Combinations	PLF
[B1]	Live	1360	plf	Service	5025
	Dead	3665	plf	1	5131.0
	Rain	0	psf	2	6574.0
	Snow	0	psf	3	5758.0
	Live roof	0	psf	4	5758.0
	Wind	0	psf	5	5767.1
	Seismic	9.1	psf	6	3298.5
				7	3307.6
	Trib Width	1.5	LF		

G2	For roof:			LRFD Load Combinations	PLF
[B2]	Live	1360	plf	Service	4889
	Dead	3529	plf	1	4940.6
	Rain	0	psf	2	6410.8
	Snow	0	psf	3	5594.8
	Live roof	0	psf	4	5594.8
	Wind	0	psf	5	5603.9
	Seismic	9.1	psf	6	3176.1
				7	3185.2

G3	For roof:			LRFD Load Combinations	PLF
[B1, B2]	Live	2720	plf	Service	9456
	Dead	6736	plf	1	9430.4
	Rain	0	psf	2	12435.2
	Snow	0	psf	3	10803.2
	Live roof	0	psf	4	10803.2
	Wind	0	psf	5	10812.3
	Seismic	9.1	psf	6	6062.4
				7	6071.5

G4	For roof:			LRFD Load Combinations	PLF
[B5]	Live	2400	plf	Service	7260
	Dead	4860	plf	1	6804.0
	Rain	0	psf	2	9672.0
	Snow	0	psf	3	8232.0
	Live roof	0	psf	4	8232.0
	Wind	0	psf	5	8241.1
	Seismic	9.1	psf	6	4374.0
				7	4383.1

C1	For roof:			LRFD Load Combinations	LB
[Outer corners]	Live	68000	lb	Service	244450
	Dead	176450	lb	1	247030.0
	Rain	0	lb	2	320540.0
	Snow	0	lb	3	279740.0
	Live roof	0	psf	4	279740.0
	Wind	0	psf	5	279749.1
	Seismic	9.1	psf	6	158805.0
				7	158814.1

C2	For roof:			LRFD Load Combinations	LB
	Live	48960	lb	Service	180900
	Dead	131940	lb	1	184716.0
	Rain	0	lb	2	236664.0
	Snow	0	lb	3	207288.0
	Live roof	0	psf	4	207288.0
	Wind	0	psf	5	207297.1
	Seismic	9.1	psf	6	118746.0
				7	118755.1

C3	For roof:			LRFD Load Combinations	LB
[Outer corners]	Live	58480	lb	Service	212675
	Dead	154195	lb	1	215873.0
	Rain	0	lb	2	278602.0
	Snow	0	lb	3	243514.0
	Live roof	0	psf	4	243514.0
	Wind	0	psf	5	243523.1
	Seismic	9.1	psf	6	138775.5
				7	138784.6

C4	For roof:			LRFD Load Combinations	LB
[Outer corners]	Live	68000	lb	Service	236400

	Dead	168400	lb	1	235760.0
	Rain	0	lb	2	310880.0
	Snow	0	lb	3	270080.0
	Live roof	0	psf	4	270080.0
	Wind	0	psf	5	270089.1
	Seismic	9.1	psf	6	151560.0
				7	151569.1

C5 [Outer corners]	For roof:			LRFD Load Combinations	LB
	Live	144000	lb	Service	435600
	Dead	291600	lb	1	408240.0
	Rain	0	lb	2	580320.0
	Snow	0	lb	3	493920.0
	Live roof	0	psf	4	493920.0
	Wind	0	psf	5	493929.1
	Seismic	9.1	psf	6	262440.0
				7	262449.1

Seismic:

TERM	SCALAR	UNITS	REFERENCE		
Risk Category	IV				
Snow importance	1.2				
Ice importance, thickness	1.25				
Ice importance, wind	1				
Seismic importance	1.5				
Site class	D		default		
Ss	0.611	g	(the S's are for lat 36.1779, long -119.347)		
Sms	0.801	g	USGS		
Sds	0.534	g			
S1	0.259	g	'		
Sms	0.488	g	'		
Sd1	0.325	g	'		
Fa	1.3112		T11.4-1		
Fv	1.918		T11.4-2		
Design category	D		(T11.6-1 and T 11.6-2)		
Ta =	0.11		eq 12.8-7	Ta	less
To	0.12		11.4.5	Ta	less
Ts	0.61		11.4.5		
TL			F 22-16		

Sa	0.50		eq 11.4-5	
Ct	0.02		T12.8-2	
hn	9.5		111.2	T
x	0.75		T12.8-2	

p	1		12.3.4.2 (check)	
Eh	**616.5**	**psf**	**eq 12.4-3**	
Ev	**9.1**	**psf**	**eq 12.4-4**	
Qe	616.5	psf	12.4.2.1	
V	616.5	psf	eq 12.8-1	(seismic base shear)
Qs	1.1		eq 12.8-2	(check)
W	577.3	psf	12.7.2	
R	3		T12.2-1	
Fp			12.11.1	

Wind:

Low rise building?		Yes	(h<60)	26.1.2.1
Open building?		No	(Ao>.8A)	26.1.2.1
V	115	mph	ASCE 26.5	
K_d	0.85		ASCE T26.6-1	
Exposure category	C		ASCE 26.7	
K_{zt}	1		ASCE Fig 26.8-1	
Enclosure	Closed		ASCE 26.10	
Gc_{pi}	0.18		ASCE T26.11-1	
q_h	24.5	psf	ASCE Eq 28.3-1	
K_z	0.85		ASCE T28.3.1	
K_h	0.85		ASCE T28.3.1	
q_z	24.5		ASCE Eq 28.3-1	
p	**-30.6**	**psf**	**ASCE Eq 28.4-1**	
p	**12.5**	**psf**	**ASCE Eq 28.4-2**	
Gc_{pf}	-1.07		ASCE Fig 28.4-1	
Gc_{pf}	0.69		ASCE Fig 28.4-1	

Seismic forces of 616.5 psf are greater than wind forces of -30.6 psf and 12.5 psf, so seismic governs.

USGS Design Maps Summary Report

User–Specified Input

Building Code Reference Document ASCE 7-10 Standard
(which utilizes USGS hazard data available in 2008)

Site Coordinates 36.1779°N, 119.347°W

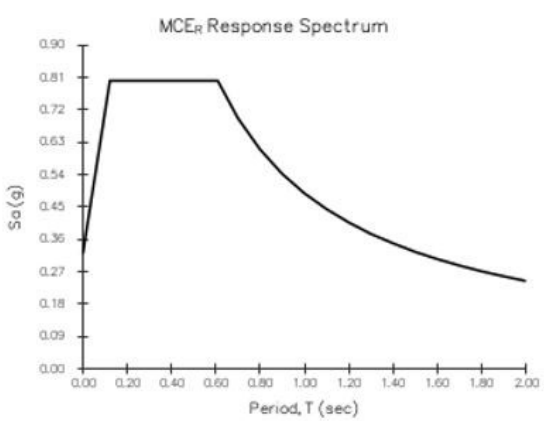
Site Soil Classification Site Class D – "Stiff Soil"

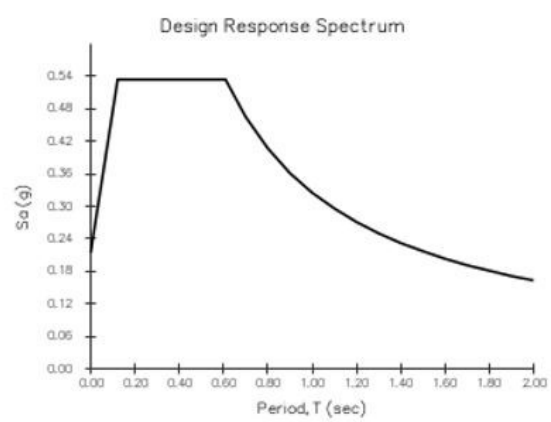
Risk Category IV (e.g. essential facilities)

USGS–Provided Output

S_S =	0.611 g	S_{MS} =	0.801 g	S_{DS} =	0.534 g
S_1 =	0.259 g	S_{M1} =	0.488 g	S_{D1} =	0.325 g

For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the "2009 NEHRP" building code reference document.





Flexural Strength	B1	B2	B3	B4	B5		
b_w	7	7				in	
Depth	34	34	34	34	34	in	
Cross section area	928	928	928	928	928	in^2	
trib width =	10	10	10	10	10	ft	
trib length =	60	68	68	60	60	ft	
Assume simply supported?	Y	Y	Y	Y	Y		
Max shear	67728	48360	65518	54808	48360	lb	
Max moment	1151376	725400	1113806	931736	725400	lb-ft	
Max shear	67.728	48.36	65.518	54.808	48.36	K	
Max moment	1151.376	725.4	1113.806	931.736	725.4	K-ft	
f'c	5000	5000	5000	5000	5000	psi	
Aps	1.84	1.84	1.84	1.84	1.84	in2	12 .5in dia, six each stem
As	0.88	0.88	0.88	0.88	0.88	in2	(2 #6)
As,min	0.841457	0.841457	0.841457	0.841457	0.841457	in2	PCI 4.2.1.4
Reinforcement check	Good	Good	Good	Good	Good		
C	1.06	1.06	1.06	1.06	1.06	(PCI 4.12.3)	
fy	60000	60,000	60000	60,000	60,000	psi	
Cωpu	0.023803	0.028564	0.028564	0.028564	0.028564		
ω	0.002157	0.002588	0.002588	0.002588	0.002588		
ω'	0	0	0	0	0		
fps	268000	268000	268000	268000	268000	psi	PCI Fig 4.12.3
dp	32	32	32	32	32	in	
a	0.892026	1.070431	1.070431	1.070431	1.070431		
c	1.049443	1.259331	1.259331	1.259331	1.259331		
Mn	17331553	17282855	17282855	17282855	17282855	lb-in	
Mn	1444296	1440238	1440238	1440238	1440238	lb-ft	
Mn	1444.296	1440.238	1440.238	1440.238	1440.238	K-ft	
ϕ	0.9	0.9	0.9	0.9	0.9		
ϕMn	1299.86	1296.21	1296.21	1296.21	1296.21	K-ft	

	6	4	4	4	4		
D/C	0.885765	0.718813	0.859276	0.718813	0.55963	Good	
γp	0.29	0.29	0.29	0.29	0.29		ACI T20.3.2.3.1
fps	269988.9	269988.5	269988.5	269988.5	269988.5	psi	ACI eq 20.3.2.3.1

Transverse Flexural Strength - B2, B3, B4			Transverse Flexural Strength – B1		
M_u	15.24543	kip-in/ft	M_u	15.3	kip-in/ft
A_s	0.16	in^2/ft	A_s	0.16	in^2/ft
a	0.188235		a	0.18824	
M_n	18296.47	lb-in/ft	M_n	18296.5	lb-in/ft
M_n	18.29647	K-in/ft	M_n	18.296.5	K-in/ft
ϕ	0.9		ϕ	0.9	
ϕM_n	16.46682	K-in/ft	ϕM_n	16.4668	K-in/ft
D/C	0.833244		D/C	0.929142	
A_s is from	W4 wire	at 3" OC			

Associated Design Equations:

$$M_n = A_{ps} f_{ps} \left(d_p - \frac{a}{2}\right) + A_s f_y (d$$

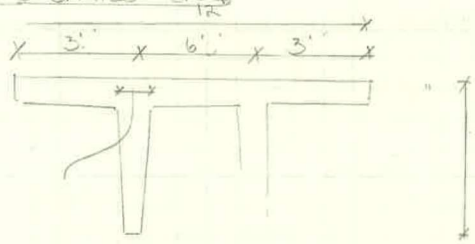
$$a = \frac{A_{ps} f_{ps} + A_s f_y}{}$$

$$C\omega_{pu} = C \frac{A_{ps} f_{pu}}{b\, d\, f'_c} + \frac{d}{d_p} (\omega - \omega') \quad \textit{(PCI Fig 4.12.3)}$$

$$\omega = \frac{A_s f_y}{b\, d\, f'_c}$$

$$A_{s,min} = 3 \frac{\sqrt{f'_c}}{f_y} b_w d \geq \frac{200\, b_w d}{f_y}$$

SAMPLE CALC



$f_y = 60$
$f'_c = 5$
$d = 34$

$$\frac{200 b_w d}{f_y}$$

$$= \frac{3\sqrt{5\,psi}}{60}\,(34'') =$$

$$= \frac{\quad}{\quad}$$

$$=$$

$$= \frac{\quad}{\quad} =$$

$$\frac{\rho_b f_y}{0.85 f'_c}$$

$$=$$

LEXURE CHECK SAMPLE

= [illegible]

= [illegible]

= [illegible]

∴ [illegible] = [illegible] 1500 lb-ft

(W IN FLEX RE)

Soil Properties		
Dry Density (γ_d)	100	pcf
Friction angle (Φ')	33	degrees
Friction angle (Φ')	0.5760	radians
Cohesion (c')	30	psf

$Q_{applied}$ (kips) =	531
$Q_{(all)}$ =	694

Meyerhof's Equation:

$$q_u = c'N_cF_{cs}F_{cd}F_{ci} + qN_qF_{qs}F_{qd}F_{qi} + \frac{1}{2}\gamma BN_\gamma F_{\gamma s}F_{\gamma d}F_{\gamma i}$$

Square footing		
B =	9	ft
L =	9	ft
Area =	81	ft^2
Depth (D_f) =	3	ft
q =	300	psf
Nc =	38.64	
Nq =	26.09	
Nγ =	35.19	
Fcs =	1.68	
Fqs =	1.65	
Fγs =	0.6	
Fcd =	1.093	
Fqd =	1.090	
Fγd =	1	
Fci =	1	
Fqi =	1	
Fγi =	1	
q_u =	25693.33	psf
$q_{(all)}$ =	8564.44	psf
$Q_{(all)}$ =	693719.84	lbs
F.S. =	3	
$Q_{(all)}$ =	693.72	kips

(from Eq. 4.26)

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Bearing Capacity - Parking Structure Footings

Footing: 12' X 24'		
$Q_{applied}$	435.6	kips
B	12	ft
L	24	ft
Area	288	ft^2
Depth (Df)	5	ft
q	525	psf
c'	30	psf
γ	105	pcf
Nc	38.64	
Nq	26.09	
Nγ	35.19	
Fcs	1.34	
Fqs	1.32	
Fγs	0.8	
Fcd	1.117	
Fqd	1.112	
Fγd	1	
Fci	1	
Fqi	1	
Fγi	1	
qu	39648.14	psf
$q_{(all)}$	13216.05	psf
F.S.	3	
Q_{all}	3806.22	kips

Footing: 6' X 10'		
$Q_{applied}$	181	kips
B	6	ft
L	10	ft
Area	60	ft^2
Depth (Df)	5	ft
q	525	psf
c'	30	psf
γ	105	pcf
Nc	38.64	
Nq	26.09	
Nγ	35.19	
Fcs	1.405	
Fqs	1.39	
Fγs	0.76	
Fcd	1.23	
Fqd	1.22	
Fγd	1	
Fci	1	
Fqi	1	
Fγi	1	
qu	33739.57	psf
$q_{(all)}$	11246.52	psf
F.S.	3.0	
Q_{all}	674.79	kips

Footing: 8' X 12'		
$Q_{applied}$	244.5	kips
B	8	ft
L	12	ft
Area	96	ft^2
Depth (Df)	5	ft
q	525	psf
c'	30	psf
γ	105	pcf
Nc	38.64	
Nq	26.09	
Nγ	35.19	
Fcs	1.45	
Fqs	1.433	
Fγs	0.733	
Fcd	1.175	
Fqd	1.168	
Fγd	1	
Fci	1	
Fqi	1	
Fγi	1	
qu	35744.75	psf
$q_{(all)}$	11914.92	psf
F.S.	3.0	
Q_{all}	1143.83	kips

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Settlement - Parking Structure Footings

Footing Size	**6' x 10'**	**8' x 12'**	**12' x 24'**	
P (applied load)	181	244.5	435.6	kips
Width	6	8	12	ft
Length	10	12	24	ft
Depth (D_f)	5	5	5	ft
Area	60	96	288	ft^2
W_f	45	72	216	kips
q	525	525	525	psf
q bar	3767	3297	2263	psf
C_1	0.919	0.905	0.849	
C_2	1.46	1.46	1.46	
Settlement (S_e)	1.08	1.36	1.71	inch

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Cut and Fill Calculations

FILL				
Designation	**Area**	**Distance**	**Volume**	
	SFT	**FT**	**CFT**	
F1	4,322.6			
F2	4,221.0	100.0	427,180	
F3	4,127.1	100.0	417,405	
F4	8,370.3	140.7	879,192	
F5	7,071.4	112.7	870,140	
F6	7,482.9	32.5	236,507	
F7	7,447.7	114.1	851,791	
F8	6,348.6	125.9	868,477	
F9	6,440.5	56.0	358,095	
F10	6,933.6	64.0	427,971	
F11	6,798.5	85.0	583,614	
F12	6,607.4	150.0	1,005,443	
F13	6,430.6	150.0	977,850	
F14	6,458.9	35.0	225,566	
F15	4,035.1	116.0	608,652	
F16	4,356.0	64.7	271,452	
		TOTAL	9,009,335	CFT
			333,679	**CY**

CUT				
Designation	**Area**	**Distance**	**Volume**	
	SFT	**FT**	**CFT**	
C1	3,959.9			
C2	3,913.2	100.0	393,655	
C3	3,892.1	100.0	390,265	
C4	7,829.0	140.7	824,581	
C5	7,191.8	112.7	846,423	
C6	6,956.6	32.5	229,912	
C7	5,907.8	114.1	733,914	
C8	5,597.8	125.9	724,278	
C9	5,507.4	56.0	310,946	
C10	5,332.7	64.0	346,883	
C11	5,100.8	85.0	443,424	
C12	4,290.8	150.0	704,370	
C13	3,576.1	150.0	590,018	
C14	3,560.0	35.0	124,882	
C15	3,436.7	116.0	405,809	
C16	3,146.8	64.7	212,976	
		TOTAL	7,282,334	CFT
			269,716	**CY**

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Quantity Calculations for Rebars

Rebar Quantity for 6ft x 10ft Footings						
	Length of Bar	No. of Bars per Footing	No. of Footings	Total length of rebar	Weight per foot	Weight
	LF			LF	lb/ft	LB
Long bar (#6)	9.5	6	16	912	1.502	1369.8
Short bar (#5)	5.5	4	16	352	1.043	367.1
Short bar (#7)	5.5	6	16	528	2.044	1079.2
Dowels (#5)	9	4	16	576	1.043	600.8
					Total	3417.0

Rebar Quantity for 8ft x 12ft Footings						
	Length of Bar	No. of Bars per Footing	No. of Footings	Total Length of rebar	Weight per foot	Weight
	LF			LF	lb/ft	LB
Long bar (#7)	11.5	7	12	966	2.044	1974.5
Short bar (#5)	7.5	4	12	360	1.043	375.5
Short bar (#7)	7.5	7	12	630	2.044	1287.7
Dowels (#5)	9	4	12	432	1.043	450.6
					Total	4088.3

Rebar Quantity for 12ft x 24ft Footings						
	Length of Bar	No. of Bars per Footing	No. of Footings	Total Length of rebar	Weight per foot	Weight
	LF			LF	lb/ft	LB
Long bar (#9)	23.5	13	8	2444	3.4	8309.6
Short bar (#8)	11.5	11	8	1012	2.67	2702.0
Short bar (#7)	11.5	8	8	736	2.044	1504.4
Dowels (#6)	10	4	8	320	1.502	480.6
					Total	12996.7

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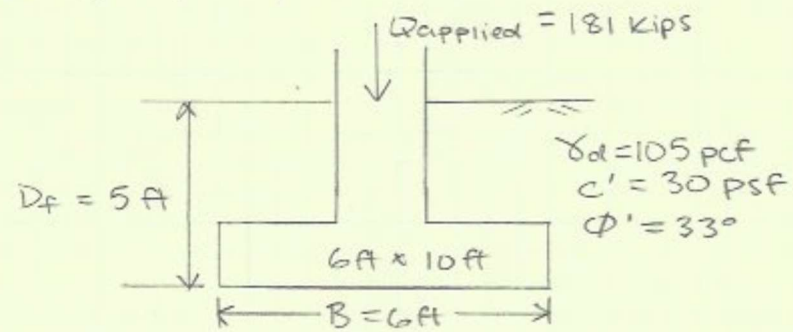
Quantity Calculations for Excavation, Backfill, and Concrete

Excavation, Backfill, and Concrete for 6ft x 10ft footings						
	Length	Width	Depth	Volume per footing	No. of Footings	Total Volume
	LF	LF	LF	CY		CY
Excavation	14	10	5	25.93	16	414.8
Concrete for footing	10	6	1.67	3.70	16	59.26
Concrete for stem	1.5	1	3.33	0.19	16	2.96
Backfill						352.60

Excavation, Backfill, and Concrete for 8ft x 12ft footings						
	Length	Width	Depth	Volume per footing	No. of Footings	Total Volume
	LF	LF	LF	CY		CY
Excavation	16	12	5	35.56	12	426.7
Concrete for footing	12	8	1.67	5.93	12	71.11
Concrete for stem	1.5	1	3.33	0.19	12	2.22
Backfill						353.34

Excavation, Backfill, and Concrete for 12ft x 24ft footings						
	Length	Width	Depth	Volume per footing	No. of Footings	Total Volume
	LF	LF	LF	CY		CY
Excavation	28	16	5	82.96	8	663.7
Concrete for footing	24	12	2	21.33	8	170.67
Concrete for stem	1.67	1.17	3	0.22	8	1.73
Backfill						491.31

Bearing capacity:



B = 6ft , L = 10ft
Area = 60 ft²
Df = 5 ft

Meyerhof's Bearing capacity equation:

$$q_u = c' N_c F_{cs} F_{cd} F_{ci} + q N_q F_{qs} F_{qd} F_{qi} + \tfrac{1}{2} \gamma B N_\gamma F_{\gamma s} F_{\gamma d} F_{\gamma i}$$

$$\phi' = 33° \begin{cases} N_c = 38.64 \\ N_q = 26.09 \\ N_\gamma = 35.19 \end{cases}$$

$$q = \gamma_d \cdot D_f = (105\ lb/ft^3)(5\ ft) = 525\ lb/ft^2$$

Shape Factors:

$$F_{cs} = 1 + \left(\frac{B}{L}\right)\left(\frac{N_q}{N_c}\right) = 1 + \left(\frac{6}{10}\right)\left(\frac{26.09}{38.64}\right) = 1.405$$

$$F_{qs} = 1 + \left(\frac{B}{L}\right)\tan\phi' = 1 + \left(\frac{6}{10}\right)\tan(33) = 1.39$$

$$F_{\gamma s} = 1 - 0.4\left(\frac{B}{L}\right) = 1 - 0.4\left(\frac{6}{10}\right) = 0.76$$

Depth Factors:

$$\frac{D_f}{B} = \frac{5}{6} = 0.83 < 1$$

$$\phi' = 33° > 0$$

$$F_{qd} = 1 + 2\tan\phi'(1 - \sin\phi')^2\left(\frac{D_f}{B}\right)$$

$$= 1 + 2\tan(33)(1 - \sin 33)^2\left(\frac{5}{6}\right) = 1.22$$

$$F_{cd} = F_{qd} - \frac{1 - F_{qd}}{N_c \tan\phi'}$$

$$= 1.22 - \frac{1 - 1.22}{(38.64)\tan(33)}$$

$$= 1.23$$

$$F_{\gamma d} = 1$$

Inclination Factors:

$$F_{ci} = F_{qi} = \left(1 - \frac{\beta}{90}\right)^2 = \left(1 - \frac{0}{90}\right)^2 = 1$$

$$F_{\gamma i} = \left(1 - \frac{\beta}{90}\right)^2 = \left(1 - \frac{0}{90}\right)^2 = 1$$

$$q_u = (30)(38.64)(1.405)(1.23)(1) + (525)(26.09)(1.39)(1.22)(1) + \tfrac{1}{2}(105)(6)(35.19)(0.76)(1)(1)$$

$$= 2003.27 + 23227.8 + 8424.486$$

$$= 33655 \text{ psf}$$

$$= 33.7 \text{ ksf}$$

$$q_{all} = \frac{q_u}{FS} = \frac{33655 \text{ psf}}{3} = 11218 \text{ psf} = 11.2 \text{ ksf}$$

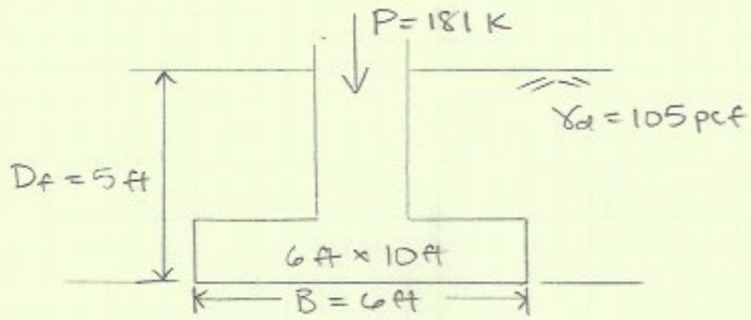
$$q_{all} = \frac{Q_{all}}{Area} \Rightarrow Q_{all} = q_{all} \times Area$$

$$= (11.218 \text{ ksf})(60 \text{ ft}^2)$$

$$= 673 \text{ kips} > 181 \text{ kips} = Q_{applied}$$

$\rightarrow$ safe

Settlement:



$W_f = B \times L \times D_f \times \gamma_{conc.} = 6 \times 10 \times 5 \times 150 \text{ pcf}$
$= 45000 \text{ lb}$

$q = \gamma_d \cdot D_f = (105 \text{ lb/ft}^2)(5 \text{ ft}) = 525 \text{ lb/ft}^2$

$\bar{q} = \frac{W_f + P}{A} = \frac{45000 + 181000}{60} = 3766.67 \text{ lb/ft}^2$

$C_1 = 1 - 0.5\left[\frac{q}{\bar{q} - q}\right] = 1 - 0.5\left[\frac{525}{3766.67 - 525}\right]$
$= 0.919$

$C_2 = 1 + 0.2\log\left(\frac{\text{time in years}}{0.1}\right) = 1 + 0.2\log\left(\frac{20}{0.1}\right) = 1.46$

$z_1 = \left(0.5 + 0.0555\left(\frac{L}{B} - 1\right)\right) \times B$
$= \left(0.5 + 0.0555\left(\frac{10}{6} - 1\right)\right) \times 6 = 3.222 \text{ ft}$

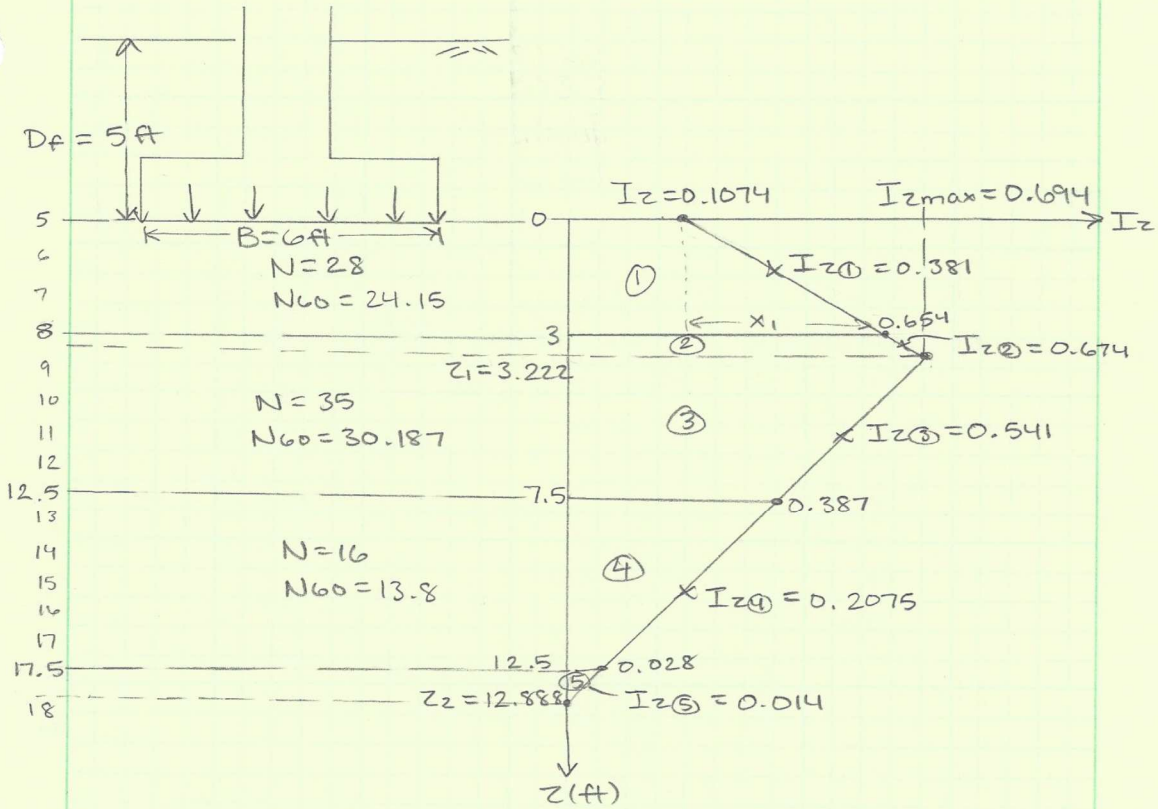
$z_2 = \left(2 + 0.222\left(\frac{L}{B} - 1\right)\right) \times B$
$= \left(2 + 0.222\left(\frac{10}{6} - 1\right)\right) \times 6 = 12.888 \text{ ft}$

$q'_{z(1)} = (D_f + z_1)\gamma_d = (5 + 3.222)(105) = 863 \text{ psf}$

$I_{z(max)} = 0.5 + 0.1\sqrt{\frac{\bar{q} - q}{q'_{z(1)}}}$

$= 0.5 + 0.1\sqrt{\frac{3766.67 - 525}{863}} = 0.694$

I_z for rectangular footing $= 0.1 + 0.0111\left(\frac{L}{B} - 1\right) \leq 0.2$
$= 0.1 + 0.0111\left(\frac{10}{6} - 1\right)$
$= 0.1074$



$$E_s = \alpha N_{60} P_a = 5 N_{60} P_a$$

$$N_{60} = \frac{N \eta_H \eta_B \eta_S \eta_R}{60}$$

e.g. N_{60} for Layer 2 $= \dfrac{35(60\%)(1.15)(1)(0.75)}{60}$

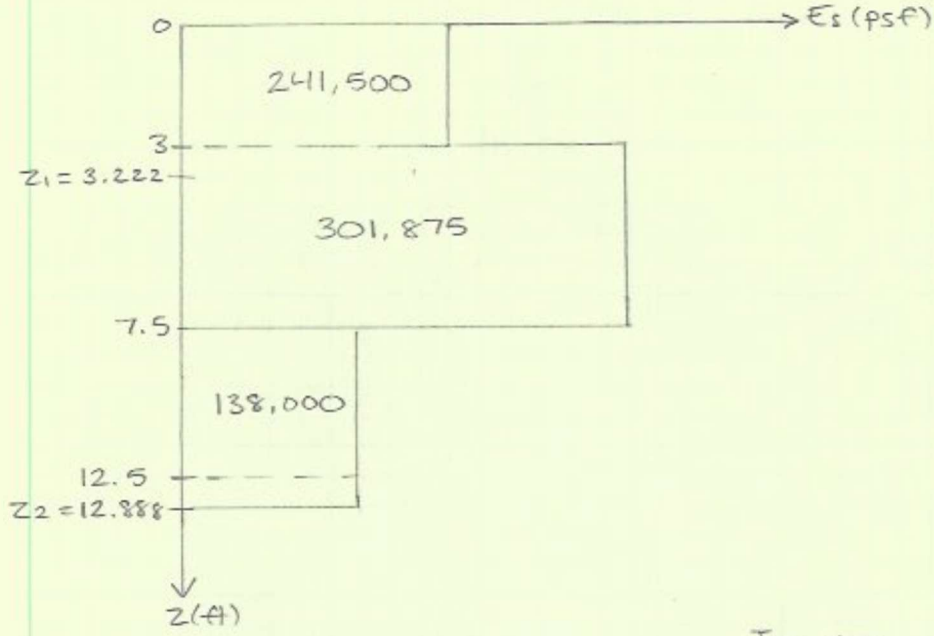
$= 30.1875$

$E_s = 5(2000)(30.1875) = 301{,}875$ psf

$$\frac{3}{x_1} = \frac{3.222}{0.694 - 0.1074}$$

$x_1 = 0.546 \qquad 0.546 + 0.1074 = 0.654$

$$I_{z①} = \frac{0.1074 + 0.654}{2} = 0.381$$



Layer No.	ΔZ (ft)	Es (psf)	Iz (ft)	$\frac{I_z}{E_s}\Delta Z$
1	3	241500	0.381	4.733×10^{-6}
2	0.22	301875	0.674	4.912×10^{-7}
3	4.28	301875	0.541	7.67×10^{-6}
4	5	138000	0.2075	7.52×10^{-6}
5	0.388	138000	0.014	3.936×10^{-8}
				$\Sigma = 2.405 \times 10^{-5}$

$$S_e = C_1 C_2 (\bar{q} - q) \sum_{0}^{z_2} \frac{I_z \Delta Z}{E_s} \quad \text{(Schmertmann Method)}$$

$$= 0.919(1.46)(3766.67 - 525)(2.045 \times 10^{-5})$$
$$= \boxed{1.08 \text{ in}}$$

Rebar Design for 6 ft x 10 ft footing:

Material properties:
Compressive strength of concrete $f'_c = 4000$ psi
Grade-60 steel: $f_y = 60000$ psi
Factored load $P = 181$ kips

Factored soil pressure $q_u = \frac{P}{Area} = \frac{181\ K}{60\ ft^2} = 3.02$ ksf

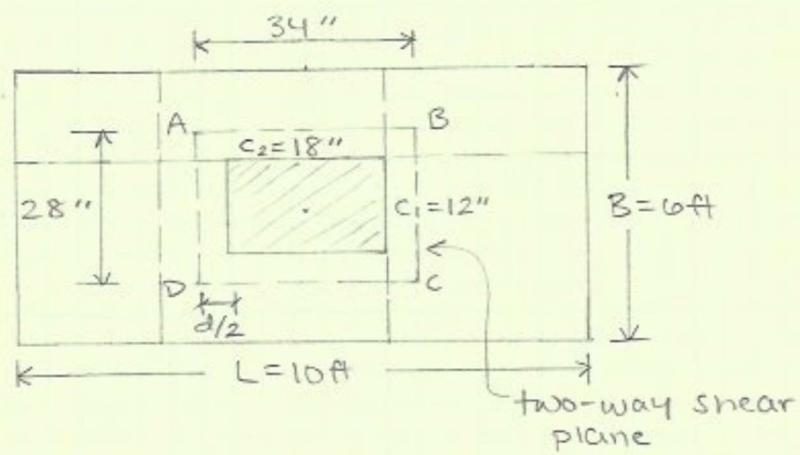
Two-way shear:

$\phi = 0.75$

Assume no shear reinforcement ($V_s = 0$)

$\phi V_n = \phi V_c$ (ACI 318 22.6.1.2)

Assume $h = 20$ in
$d = h - 3''(\text{cover}) - 1'' = 20 - 3 - 1 = 16$ in



b_o = Perimeter of ABCD $= 2(34\text{ in}) + 2(28\text{ in}) = 124$ in

ACI 318 Sec 22.6.5.2:

$\beta = \frac{18''}{12''} = 1.5$

$V_c = \left(2 + \frac{4}{\beta}\right)\sqrt{f'_c}\, b_o d$

$= \left(2 + \frac{4}{1.5}\right)\sqrt{4000}\,(124)(16) = 585$ kips

$V_c = \left(\frac{\alpha_s d}{b_o} + 2\right)\sqrt{f'_c}\, b_o d$

$= \left(\frac{40 \times 16}{124} + 2\right)\sqrt{4000}\,(124)(16) = 898$ kips

$V_c = 4\sqrt{f'c}\, b_o d = 4\sqrt{4000}\,(124)(16) = 502$ kips → controls

$\phi V_c = 0.75(502) = 376.5$ kips

$V_u = q_u$ (Area of footing − Area of ABCD)

$= (3.02\ \text{KSF})\left[60\ \text{ft}^2 - (34\times 28)\text{in}^2 \times \left(\frac{\text{ft}}{12\text{in}}\right)^2\right]$

$= 161$ Kips

check: $\phi V_n = \phi V_c = 376.5\ \text{K} > 161$ Kips $= V_u$ (OK)

<u>One-Way shear:</u>

$b_w = 6\text{ft} \times 12\text{in}/\text{ft} = 72$ in

$V_c = 2\lambda\sqrt{f'c}\, b_w d$ (ACI 318 Sec 22.5.5.1)

$\lambda = 1$ for normal weight concrete

$\phi V_c = \dfrac{0.75\,(2)(\sqrt{4000})(72)(16)}{1000} = 109.3$ Kips

$V_u = b_w \cdot q_u \left[(L/2) - \left(\dfrac{\frac{C_2}{2} + d}{12\ \text{in}/\text{ft}}\right)\right]$

$= (6\text{ft})(3.02\ \text{KSF})\left[\dfrac{10\text{ft}}{2} - \left(\dfrac{\frac{18''}{2} + 16''}{12\ \text{in}/\text{ft}}\right)\right]$

$= 52.85$ Kips

Check: $\phi V_n = \phi V_c = 109.3\ \text{K} > 52.85\ \text{K} = V_u$ (OK)

<u>Bearing Strength:</u>

$\phi = 0.65$ (ACI 318 21.2.1(d))

B_n = lesser of $\begin{cases} (a)\ \sqrt{\dfrac{A_2}{A_1}}\,(0.85 f'c\, A_1) \\ (b)\ 2(0.85 f'c\, A_1) \end{cases}$ ACI 318 Sec. 22.8.3.2

where A_1 = area of column $= (18\times 12)\ \text{in}^2 \times \left(\frac{\text{ft}}{12\text{in}}\right)^2 = 1.5\ \text{ft}^2$

A_2 = area of footing $= 60\ \text{ft}^2$

B_n = lesser of $\begin{cases} (a)\ \sqrt{\dfrac{60}{1.5}}\,(0.85\times 4\ \text{ksi}\times (18\times 12)\text{in}^2 = 4645\ \text{K} \\ (b)\ 2\times 0.85\times 4\ \text{ksi}\times (18\times 12)\text{in}^2 = 1469\ \text{K} \end{cases}$

Check: $\phi B_n = 0.65(1469\,k) = 954.85\,k > 181\,k = P$ (OK)

ACI 318 - Sec 16.3.4.1 $\longrightarrow$ dowel reinforcement area

$$A_s \geq 0.005 A_g = 0.005(18 \times 12) = 1.08\ in^2$$

A_g → area of column

Use 4 #5 dowels with $A_s = 4(0.31\ in^2) = 1.24\ in^2$

Development length of dowels (l_{dc}):

ACI 318 25.4.9.1:

$$l_{dc} = \text{greater of} \begin{cases} \left(\dfrac{f_y \psi_r}{50\lambda\sqrt{f'_c}}\right) d_b \\ 0.0003 f_y \psi_r d_b \\ 8'' \end{cases}$$

$\lambda = 1$
$\psi_r = 1$
(Table 25.4.9.3)

$$l_{dc} = \text{greater of} \begin{cases} \dfrac{(60{,}000\,psi)}{50\sqrt{4000\,psi}}(0.625\,in) = 11.86\,in \\ 0.0003(60000\,psi)(1)(0.625\,in) = 11.25\,in \\ 8'' \end{cases}$$

controls

$h = 20\,in > l_{dc} = 11.86\,in$

<u>Moment capacity in Long Direction:</u>

$$M_u = \frac{q_u(\text{moment arm})^2 b}{2}$$

$$\text{moment arm} = \frac{10\,ft - (18''/12)}{2} = 4.25\,ft$$

$$M_u = \frac{(3.02\,ksf)(4.25\,ft)^2(6\,ft)}{2} = 163.65\,k\text{-}ft$$

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{A_s(60\,ksi)}{0.85(4\,ksi)(6\,ft \times 12\,in/ft)} = 0.245 A_s$$

$\beta_1 = 0.85$ for $f'_c = 4000\,psi$ (ACI 318 - Sec 22.2.2.4.3)

$$\phi M_n = \phi A_s f_y \left(d - \frac{a}{2}\right)$$

$$\phi M_n = M_u = 0.9 A_s f_y \left(d - \frac{a}{2}\right)$$

$$163.65\,k\text{-}ft \times \frac{12\,in}{ft} = 0.9 A_s(60\,ksi)\left(16'' - \frac{0.245 A_s}{2}\right)$$

$$36.367 = 16A_s - 0.1225A_s^2 \rightarrow A_s = 2.31 \text{ in}^2$$

ACI 318 Sec 8.6.1.1

$$A_{s\,min} = 0.0018bh = 0.0018(6\text{ft} \times 12\text{in/ft})(20\text{ in})$$
$$= 2.592 \text{ in}^2$$

Use $A_s = 2.6 \text{ in}^2$

Use 6 #6 bars $\rightarrow A_s = 6(0.44) = 2.64 \text{ in}^2$

Center-to-center spacing:

$$s = \frac{(b'' - 3'' - 3'')}{\#\text{ of bars} - 1} = \frac{(6\text{ft} \times 12) - 6''}{6 - 1} = 13.2 \text{ in}$$

<u>Moment capacity in short direction:</u>

$$\text{moment arm} = \frac{6\text{ft} - 1\text{ft}}{2} = 2.5 \text{ ft}$$

$$M_u = \frac{q_u(\text{moment arm})^2(L)}{2}$$
$$= \frac{(3.02 \text{ ksf})(2.5\text{ ft})^2(10\text{ ft})}{2} = 94.4 \text{ k-ft}$$

$$a = \frac{A_s f_y}{0.85 f'_c L} = \frac{A_s(60 \text{ ksi})}{0.85(4 \text{ ksi})(10\text{ft} \times 12\text{in/ft})} = 0.147A_s$$

$$\phi M_n = \phi A_s f_y\left(d - \frac{a}{2}\right) = M_u$$

$$94.4 \text{ k-ft} \times \frac{12\text{in}}{\text{ft}} = 0.9A_s(60\text{ksi})\left(16'' - \frac{0.147A_s}{2}\right)$$
$$20.978 = 16A_s - 0.0735A_s^2$$
$$\Rightarrow A_s = 1.32 \text{ in}^2$$

$$A_{s\,min} = 0.0018Lh = 0.0018(10\text{ft} \times 12\text{in/ft})(20\text{ in})$$
$$= 4.32 \text{ in}^2$$

Use $A_s = 4.32 \text{ in}^2$

Distribute bars as follows:

$$\beta = \text{aspect ratio} = \frac{L}{B} = \frac{10}{6} = 1.67$$

$$\text{distribution ratio} = \frac{2}{\beta + 1} = \frac{2}{1.67 + 1} = 0.75$$

Reinforcement in 5ft center band $= 0.75A_s$
$= 0.75(4.32)$
$= 3.24 \text{ in}^2$

Use 6 #7 bars distributed uniformly across the center 5-ft band $\rightarrow A_s = 6(0.60) = 3.6\ in^2$

$$\text{spacing} = \frac{5\ ft \times 12\ in/ft}{6\ bars - 1} = 12\ in \text{ center-to-center}$$

Reinforcement outside central band:
$= 4.32 - 6(0.60) = 0.72\ in^2$

Use 4 #5 bars (2 each side) $\rightarrow A_s = 4(0.31) = 1.24\ in^2$

$$\text{Spacing (on each side)} = \frac{(2.5\ ft \times 12\ in/ft) - 3''\ (cover)}{2} = 13.5'' \text{ center-to-center}$$

Development length in long direction:

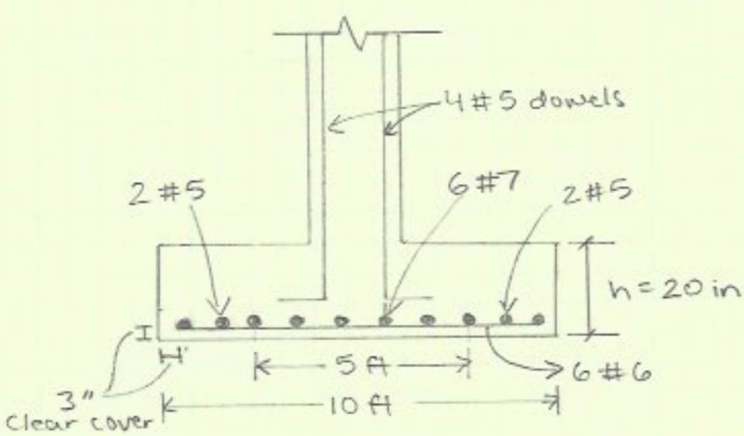
$$l_d = \left[\frac{3}{40} \cdot \frac{f_y}{\lambda\sqrt{f'_c}} \frac{\psi_t \psi_e \psi_s}{\left(\frac{c_b + K_{tr}}{d_b}\right)}\right] d_b \qquad \text{ACI 318 Eqn. 25.4.2.3a}$$

$$= \left[\frac{3}{40} \cdot \frac{60000}{\sqrt{4000}} \cdot \frac{0.8}{2.5}\right](0.75\ in) = 17.1\ in$$

$\hookrightarrow$ #6 d_b

$$l_{d(provided)} = \text{moment arm} - 3''\ (cover)$$
$$= \left(4.25\ ft \times \frac{12\ in}{ft}\right) - 3''$$
$$= 48\ in$$

$l_d = 17.1\ in < l_{d(prov.)} = 48\ in$
$\Rightarrow$ use straight #6 bars



Name: Dayadeepak Singh Project: Children's Hospital Date: 2/15/2018

Potable demand: Unit loading method- No. of people served

Hospital	185-317	gal/person per day		
Office	11-16	gal/person per day		
	gal/person/day			
Usage	Min	Max	Average	
Hospital	185	317	251	
Office	11	16	13.5	
Family supp	79	132	105.5	
		Min	Max	Average
Buildings		gpm	gpm	gpm
Wing 1	500	64.23611	110.069444	87.15278
Wing 2	500	64.23611	110.069444	87.15278
Wing 3	500	64.23611	110.069444	87.15278
Doctor's of	150	19.27083	33.0208333	26.14583
Family supp	150	19.27083	33.0208333	26.14583
	Total	231.25	396.25	313.75

Name: Dayadeepak Singh Project: Children's Hospital Date: 2/15/2018

Potable demand: Area

		gal/day/acre			
		min	max	average	
Office commercial		1100	5100	2030	
Multi family res.		2600	6600	4160	
High density		2600	6600	4160	
gal/min	area (sq ft)	Area ac	Min	Max	Average
			gpm	gpm	gpm
Wing 1	72000	1.652893	2.984	7.576	4.775
Wing 2	72000	1.652893	2.984	7.576	4.775
Wing 3	72000	1.652893	2.984	7.576	4.775
Doctor's of	50000	1.147842	0.877	4.065	1.618
Family supp	25000	0.573921	1.036	2.630	1.658
		Total	10.866	29.423	17.601

Name: Dayadeepak Singh Project: Children's Hospital Date: 2/15/2018

Potable demand: WSFU Calculation

		Toilet	Urinal	Sink	Bathtub	Laundry	Kitchen Sink	ngking Fountain	
		10 WSFU	10 WSFU	3 WSFU	4 WSFU	4WSFU	4WSFU	0.25 WSFU	Total WSFU
Hospital Wing 1		144	20	144	105	6	9	10	2554.5
Hospital Wing 2		21	10	10	2	6	5	10	394.5
Hospital Wing 3		84	15	30	70	6	7	10	1414.5
Doctors office		12	6	10	0	0	1	5	215.25
Family support		18	9	15	10	4	2	5	380.25
									4959

Name: Dayadeepak Singh Project: Children's Hospital Date: 2/15/2018

Potable demand: No. of Fixtures

			Design Load	Approximate	CPC
Building		WSFU	(gpm)	(gpm)	(gpm)
Hospital Wing 1		2554.5	260	360	345
Hospital Wing 2		394.5	110	150	125
Hospital Wing 3		1414.5	185	290	260
Doctors office		215.25	98	50	65
Family support		380.25	110	50	110
			763	900	905

Name: Dayadeepak Singh Project: Children's Hospital Date: 2/15/2018

Potable demand: No. of beds

Hospital	1310	L/day/bed	Peak	3450	L/day/bed
Building	3.8	L/day/m^2	Peak	21.2	L/day/m^2
		No. of Beds	Area m^2)	Min (gpm)	Max (gpm)
Hospital Wing 1		90	-	21.66005	57.04365
Hospital Wing 2		40	-	9.62669	25.35273
Hospital Wing 3		90	-	21.66005	57.04365
Doctor's office		-	50000	34.90594	194.7384
Family support		-	25000	17.45297	97.36919
			Total	105.3057	431.5476

Name: Dayadeepak Singh Project: Children's Hospital Date: 2/15/2018

Fire flow demand

$NFF = 18 * F * A^{0.5} * O * (1 + (X + P))$			
NFF = needed fire flow (gpm)			
F=	0.6	class of construction coefficient	
A=	61000	effective area (ft2)	
O =	0.85	ccupancy factor	
X=	0	Expose factor	
P =	0	communication factor	
NFF:	2267.293	gpm	
NFF:	2500	gpm (rounded to nearest 250 gpm)	

Potable Water Demand

* Method: unit loading method: Based on No. of People Served

Buildings:- Hospital Wing1, Wing2, Wing3, Doctor's office, family support

Table 3.3 - Water distribution systems Handbook.

Usage
Hospital - 185-317 gal/person/day
office 11 - 16 gal/person/day

No. of People Served.		× Demand		
Hospital Wing 1	- 500	× 317	=	158500 gal/day
Hospital Wing 2	- 500	× 317	=	158500 gal/day
Hospital Wing 3	- 500	× 317	=	158500 gal/day.
Doctors office	- 150	× 317	=	47550 gal/day
family support	- 150	× 317	=	47550 gal/day.

Wing 1	- 158500 gal/day	× 24 hr/day × 60 min/hr	=	110.1 gal/min
Wing 2	- 158500 gal/day	× 24 hr/day × 60 min/hr	=	110.1 gal/min
Wing 3	- 158500 gal/day	× 24 hr/day × 60 min/hr	=	110.1 gal/min
Doctor office	- 47550 gal/day	× 24 hr/day × 60 min/hr	=	33 gal/min
family support	- 47550 gal/day	× 24 hr/day × 60 min/hr	=	33 gal/min

$\boxed{396.3 \text{ gal/min.}}$

fire flow Demand

Hospital wing - 1

$N_{ff} = 18 \times f \times A^{0.5} \times O \times (1 + (x + P))$ (California Fire Code)

N_{ff} = fire flow (gpm)
f = construction class coefficient.
A = effective area. (sq. ft)
O = occupancy factor.
x = exposure factor.
P = communication factor

$f = 0.6$
$A = 61{,}000 \, ft^2$
$O = 0.85$
$x = 0$
$P = 0$

$N_{ff} = 18 \times f \times A^{0.5} \times O(1 + (x + P))$

$= 18 \times 0.6 \times \sqrt{61{,}000} \times 0.85 \, (1 + (0 + 0))$

$= 2267.3$ gpm.

Round to nearest 250 gpm

$\boxed{= 2500 \text{ gpm}}$

Irrigation Demand

- Irrigation Demand =

$$Demand = (Eto \times Pf \times Sf \times 0.62)\ (IE)$$

Sf = Area sq. ft
IE = irrigation efficiency
Pf = Plant factor.
Eto = water used by plant
Demand = gpm.

$Sf = 500 \times 500 = 250,000$ sq. ft

$IE = 80\% = 0.80$

$Pf = 0.8$ - (Both Lawn & Shrubs)

Eto = Peak in July = 0.26.

$$Demand = (Eto \times Pf \times Sf \times 0.62) / IE$$

$$= \frac{(0.26 \times 0.8 \times 250,000 \times 0.62)}{0.80}$$

$$= 40,300 \text{ gal}$$

- During Peak - irrigated daily for 8 hrs to Deliver this water.

$$\frac{40,300 \text{ gal}}{8 \text{ hr}} \times \frac{1 \text{ hr}}{60 \text{ min}} = 83.9 = \underline{84 \text{ gpm}}$$

Water Demand - 84 gpm for 8 hours

Preliminary Pipe design.

main Pipe - P-4

Demand = 2500 gpm - During event of fire.

Assume V = 5ft/s

$$V = \frac{Q}{A}$$

$$A = \frac{Q}{V}$$

$$= 2500 \frac{gal}{min} \times \frac{sec}{5ft} \times \frac{1min}{60sec} \times \frac{1ft^3}{7.48gal}$$

$$= 1.11 \, ft^2$$

$$A = \frac{\pi}{4} d^2$$

$$1.11 \, ft^2 = \frac{\pi}{4} d^2$$

$$d = 1.19 ft \times \frac{\cancel{1ft}}{\cancel{12in}} \times \frac{12in}{1ft} = 14.29 \, in$$

Demand During event of fire.

Try d = 12 in

$$\boxed{D = 12 in}$$

Head loss in Pipe

$$h_L = \frac{C_f L}{C^{1.852} D^{4.87}} Q^{1.852}$$

h_L = Head loss (ft)

L = Length (ft)

C = Hazen Williams C-factor

D = Diameter (ft)

Q = Pipeline flow rate (cfs, m³/s)

C_f = unit conversion factor (4.73 E, 10.7 SI)

→ Head loss for 1 ft of pipe.

$D = 12 in = 1 ft$

$A = \frac{\pi}{4}(1)^2$.

$$v = \frac{Q}{A} = 2500 gpm \times \frac{1 ft^3}{7.48 gal} \times \frac{1 min}{60 sec} = 5.57 cfs$$

$v = Q$

$$h_L = \frac{C_f L}{C^{1.852} D^{4.87}} Q^{1.852} = \frac{10.7 \times 1 ft}{150^{1.852} \times 1 ft^{4.87}} \times 5.57^{1.852}$$

$$= 0.024 \ ft/ft$$

Length of Pipe =

$$640 ft \times 0.024 \frac{ft}{ft} = \boxed{15.37 ft}$$

minor Head loss

$$V = \frac{Q}{A} = \frac{5.57\,cfs}{\frac{\pi}{4}(1)^2\,ft^2} = 7.09\ ft/s$$

minor Head loss $= K \times \frac{V^2}{2g}$

K = Entrance = 0.04
valve = 0.39 (open)
Bend = 0.16

$$\text{Headloss} = K \times \frac{V^2}{2g}$$

$$= (0.04 + 0.39 + 0.16) \times \frac{7.09\,ft^2/s^2}{32.2 \times 2}$$

$$= 0.46\,ft$$

Total Head loss in P-4 During fire

h_L = major + minor

$= 15.37' + 0.46'$

$$\boxed{= 15.83\ ft}$$

Pressure loss Due to friction

$$\frac{P_1}{\gamma} + \frac{V_1^2}{2g} + z_1 + h_p = \frac{P_2}{\gamma} + z_2 + \frac{V_2^2}{\gamma} + h_L + h_u$$

$P_1 = 60$ Psi after Booster Pump is ON.

$$\frac{P_1}{\gamma} + \cancel{\frac{V_1^2}{2g}} + z_1 + \cancel{h_p} = \frac{P_2}{\gamma} + \cancel{\frac{V_2^2}{2g}} + h_L + h_u$$

Same V₁ No Pump

$$\frac{60\,\frac{lb}{in^2}\left(\frac{12\,in}{1\,ft}\right)^2}{62.4\,\frac{lb}{ft^3}} + 262.5 = \frac{P_2\left(\frac{12\,in}{1\,ft}\right)^2}{62.4\,\frac{lb}{ft^3}} + 15.83\,ft + 260.35\,ft$$

$$= 138.46\,ft + 262.5\,ft = \frac{(x)(144)}{62.4} + 276.18\,ft$$

$$= 124.78\,ft = \frac{(x)(144)}{62.4} \qquad P_2 = 54.07\ \text{Psi}$$

$\Rightarrow$ Pressure loss: $60 - 54.07$

$$\boxed{\text{Pressure loss} \Rightarrow 5.93\ \text{Psi}}$$

Net Positive Suction Head

$$NPSH = h_{atm} + h_s - h_{vp} - h_L$$

Pump will be used to Boost pressure from 50 psi to 70 psi

Altitude. ≈200 ft

h_{atm} Atm Pres $= 14.7\ psi \times \frac{144\ in^2}{1\ ft^2} \times \frac{ft^3}{62.4\ lb} = 33.9\ ft$

$h_{vp} = 40°F = 0.1217\ psi\ (abs) \times \frac{144\ in}{1\ ft^2} \times \frac{1\ ft^3}{62.4\ lb} = 0.28\ ft$

$h_s = 50\ psi \times \frac{144\ in^2}{1\ ft^2} \times \frac{ft^3}{62.4\ lb} = 115.38\ ft$

$$NPSH = h_{atm} + h_s - h_{vp} - h_L$$

$$= 33.9\ ft + 115.38\ ft - 0.28\ ft =$$

$$\boxed{NPSH = 149.01\ ft}$$

Storage Volume Requirements.

Minimum fire flow = 2500 gpm for 2 hr.

$$2500 \frac{gal}{min} \times \frac{ft^3}{7.48 gal} \times \frac{60 min}{hr} \times 2 hr$$

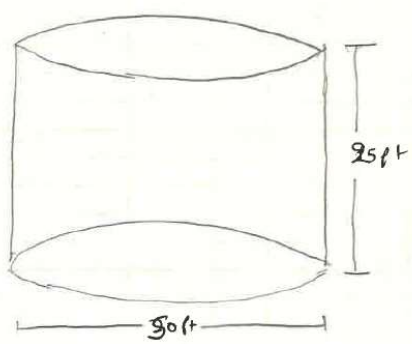
$$= 40106 \ ft^3$$

Diameter of Tank

Diameter of Tank = 20 ft

$$\frac{\pi}{4} D^2 = \frac{\pi}{4} \times 20^2 = 100\pi$$

$$\frac{\pi}{4} \times 50^2 \times 25' = 49087 ft^3$$



Minimum Tank Dimensions

EGL & HGL of Pipe -10 During fire flow

Pipe-10 length = 380 ft

Begin elevation = 255.25

End elevation = 253.5

flow = 2500 gpm

Velocity = 10.21 ft/s

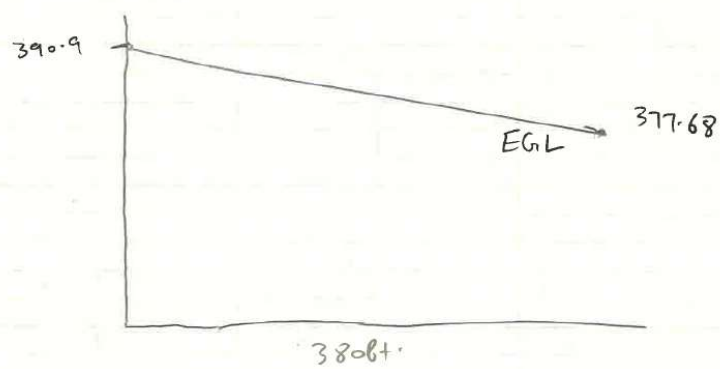
Begin Pressure = ~~28~~ 58.11 psi

End Pressure = 53.61 psi

~~Begin~~ head = Elev + velocity + Pressure

~~En~~ Begin = $255.25 + \frac{(10.21\ ft/s)^2}{2 \times 32.2\ ft/s^2} + \frac{58.11\ \frac{lbs}{in^2} \times 144}{62.4\ \frac{lbs}{ft^3}} = \boxed{390.9}$

End = $253.5 + \frac{10.21\ ft/s^2}{2 \times 32.2} + \frac{53.61\ psi \times 144}{62.4} = \boxed{377.68}$



Name: Nadun Makewita Project: Children's Hospital Date: 2/20/2018

Pre Development		
A	39.06	acre
I	0.475	in/hr
C	0.3	
Q	5.56605	cfs

Post Developmet						
Sub-basin #	C	A (acre)	Tc (min)	Td (min)	I (in/hr)	Q (cfs)
Sub-basin 1	0.8	8.48	27.3	27.3	1.32	8.95488
Sub-basin 2	0.8	5.6	17.8	17.8	1.72	7.7056
Sub-basin 3	0.8	7.68	20.8	20.8	1.59	9.76896
Sub-basin 4	0.8	7.57	20.8	20.8	1.59	9.62904
Sub-basin 5	0.8	5.54	17.8	17.8	1.72	7.62304
Sub-basin 6	0.8	4.2	24.5	24.5	1.43	4.8048
Overall Flow			27.3	27.3	1.32	41.25792

Table 43: Detention time calculations

Name:	Nadun Makewita		Project:	Children's Hospital			Date:	2/25/2018
Subbasin 1	Inlet	Overland length (ft)	Gutter Length (ft)	Kc	To (min)	Ts (min)	Tc (min)	Td (min)
	1	273	194	20	15.2	4.2	19.4	19.4
	2	273	246	20	15.2	5.3	20.5	20.5
	15	145	86	20	13.9	1.9	15.8	15.8
	16	135	150	20	13.5	3.2	16.7	16.7
	17	154	113	20	14.4	2.4	16.8	16.8
	18	90	92	20	11.0	2.0	13.0	13.0
	24	68	0	20	9.5	0.0	9.5	9.5
	25	68	108	20	9.5	2.3	11.9	11.9
Subbasin 2	Inlet	Overland length (ft)	Gutter Length (ft)	Kc	To (min)	Ts (min)	Tc (min)	Td (min)
	3	273	250	20	16.7	5.4	22.1	22.1
	4	31	242	20	5.6	5.2	10.8	10.8
	5	73	0	15	8.7	0.00	8.7	8.7
	6	60	0	15	7.8	0.00	7.8	7.8
	7	71	0	15	8.5	0.00	8.5	8.5
	26	68	190	20	8.4	4.09	12.4	12.4
Subbasin 3	Inlet	Overland length (ft)	Gutter Length (ft)	Kc	To (min)	Ts (min)	Tc (min)	Td (min)
	8	50	200	20	7.2	4.30	11.5	11.5
	9	206	257	20	14.5	5.53	20.1	20.1
	10	81	0	15	9.1	0.00	9.1	9.1
Subbasin 4	Inlet	Overland length (ft)	Gutter Length (ft)	Kc	To (min)	Ts (min)	Tc (min)	Td (min)
	11	118	68	20	4.5	1.46	6.0	6.0
	12	113	150	20	4.4	3.23	7.7	7.7
	13	62	175	20	3.5	3.77	7.3	7.3
	14	50	65	20	3.0	1.40	4.4	5.0
	32	168	148	20	5.6	3.18	8.8	8.8
	33	165	147	20	5.5	3.16	8.7	8.7
Subbasin 5	Inlet	Overland length (ft)	Gutter Length (ft)	Kc	To (min)	Ts (min)	Tc (min)	Td (min)
	27	61	60	20	3.4	1.29	4.7	5.0
	28	61	147	20	3.4	3.16	6.5	6.5
	29	240	158	20	6.7	3.40	10.1	10.1
	30	167	194	20	5.6	4.17	9.7	9.7
	31	155	100	20	5.4	2.15	7.5	7.5
Subbasin 6	Inlet	Overland length (ft)	Gutter Length (ft)	Kc	To (min)	Ts (min)	Tc (min)	Td (min)
	19	33	55	20	2.5	1.18	3.7	5.0
	20	30	150	20	2.36	3.23	5.6	5.6
	21	30	153	20	2.36	3.29	5.7	5.7
	22	180	0	20	5.8	0.00	5.8	5.8
	23	111	130	20	4.5	2.80	7.3	7.3

Table 44: Flow to the inlets calculations

Name:	Nadun Makewita		Project:	Children's Hospital			Date:	2/25/2018
Subbasin 1	Inlet	Area (acre)	C	Td (min)	I (in/hr)	Qp (cfs)		
	1	8.48	0.8	19.4	1.65	11.19		
	2	8.48	0.8	20.51	1.61	10.92		
	15	8.48	0.8	15.8	1.81	12.28		
	16	8.48	0.8	16.7	1.77	12.01		
	17	8.48	0.8	16.8	1.76	11.94		
	18	8.48	0.8	13.0	2.06	13.98		
	24	8.48	0.8	9.5	2.53	17.16		
	25	8.48	0.8	11.9	2.18	14.79		
Subbasin 2	Inlet	Area (acre)	C	Td (min)	I (in/hr)	Qp (cfs)		
	3	5.6	0.8	22.1	1.54	6.90		
	4	5.6	0.8	10.8	2.63	11.78		
	5	5.6	0.8	8.7	2.74	12.28		
	6	5.6	0.8	7.8	2.98	13.35		
	7	5.6	0.8	8.5	2.8	12.54		
	26	5.6	0.8	12.4	2.13	9.54		
Subbasin 3	Inlet	Area (acre)	C	Td (min)	I (in/hr)	Qp (cfs)		
	8	7.68	0.8	11.5	2.23	13.70		
	9	7.68	0.8	20.1	1.62	9.95		
	10	7.68	0.8	9.1	2.64	16.22		
Subbasin 4	Inlet	Area (acre)	C	Td (min)	I (in/hr)	Qp (cfs)		
	11	7.57	0.8	5	3.72	22.53		
	12	7.57	0.8	10.4	2.36	14.29		
	13	7.57	0.8	6.6	3.3	19.98		
	14	7.57	0.8	5	3.72	22.53		
	32	7.57	0.8	8.8	2.72	16.47		
	33	7.57	0.8	8.7	2.74	16.59		
Subbasin 5	Inlet	Area (acre)	C	Td (min)	I (in/hr)	Qp (cfs)		
	27	5.54	0.8	5	3.72	16.49		
	28	5.54	0.8	6.5	3.32	14.71		
	29	5.54	0.8	10.1	2.38	10.55		
	30	5.54	0.8	5.6	3.56	15.78		
	31	5.54	0.8	5.4	3.61	16.00		
Subbasin 6	Inlet	Area (acre)	C	Td (min)	I (in/hr)	Qp (cfs)		
	19	4.2	0.8	5	3.72	12.50		
	20	4.2	0.8	5.59	3.56	11.96		
	21	4.2	0.8	5.65	3.55	11.93		
	22	4.2	0.8	5.8	3.5	11.76		
	23	4.2	0.8	5	3.72	12.50		

Table 45: Calculations for maximum storage in detention pond

Name:	Nadun Makewita		Project:	Children's Hospital			Date:	2/25/2018
Td (mins)	Id (in/hr.)	Qd(cfs)	m	Avg Q (cfs	Vi (ft3)	Vo (ft3)	Si (ft3)	
5	8.81882	275.641	1.4	125.412	83381.41	37623.6	45757.81	
10	7.034102	219.8579	0.95	85.101	133014	51060.6	81953.42	
15	5.902519	184.4891	0.8	71.664	167423.9	64497.6	102926.3	
20	5.114473	159.858	0.725	64.9455	193428.1	77934.6	115493.5	
25	4.530861	141.6166	0.68	60.9144	214195.1	91371.6	122823.5	
30	4.079426	127.5065	0.65	58.227	231424.4	104808.6	126615.8	
35	3.718717	116.2322	0.628571	56.30743	246121.7	118245.6	127876.1	
40	3.423165	106.9944	0.6125	54.86775	258926.5	131682.6	127243.9	
45	3.176092	99.27195	0.6	53.748	270267.9	145119.6	125148.3	
50	2.966138	92.7096	0.59	52.8522	280446.6	158556.6	121890	
55	2.785276	87.0566	0.581818	52.11927	289680.8	171993.6	117687.2	
60	2.627672	82.13052	0.575	51.5085	298133.8	185430.6	112703.2	
80	2.156669	67.40886	0.55625	49.82888	326258.9	239178.6	87080.29	
100	1.841974	57.57275	0.545	48.8211	348315.1	292926.6	55388.52	
150	1.372081	42.88578	0.53	47.4774	389188.4	427296.6	-38108.2	
200	1.108036	34.63277	0.5225	46.80555	419056.5	561666.6	-142610	
250	0.936805	29.28077	0.518	46.40244	442871.7	696036.6	-253165	
300	0.815845	25.50007	0.515	46.1337	462826.2	830406.6	-367580	
350	0.725378	22.67242	0.512857	45.94174	480088.5	964776.6	-484688	
400	0.654893	20.46932	0.51125	45.79778	495357.6	1099147	-603789	
450	0.598261	18.69924	0.51	45.6858	509086.8	1233517	-724430	
500	0.551656	17.24255	0.509	45.59622	521587.1	1367887	-846300	
550	0.512557	16.02049	0.508182	45.52293	533081.8	1502257	-969175	
600	0.479235	14.97898	0.5075	45.46185	543737.1	1636627	-1092889	

Antonio Solorio	CE 1808 Hand Calculations	3/15/2018

Pipe Size:

Demand to Hospital during Fire Flow — 2,500 gpm

$1\ ft^3/s \Rightarrow 448.83\ gpm$

$$gpm \times \frac{1\ ft^3/s}{448\ gpm} = 5.58\ cfs$$

$Q = VA$

Q = Flowrate (cfs)

V = Velocity (ft/s)

A = Area of Pipe (ft^2)

Assume 7 ft/s

$$5.58\ ft^3/s = (7\ ft/s)(A)$$

$$A = .8\ ft^2$$

$$A = \pi r^2 \quad \text{or} \quad A = \frac{\pi d^2}{4}$$

$$\frac{\pi d^2}{4} = .8\ ft^2$$

$$d^2 = 1.01\ ft^2$$

$$d = 1.007''$$

$$d = 1.007\ ft \cdot \frac{12''}{1\ ft} \Rightarrow$$

$$\boxed{d \simeq 12''}$$

Antonio Solorio	CE 1808 Hand Calculations	4/3/2018

Height of Tank 1:

$\forall_{tank}$ = 150,000 gallon tank

1 ft^3 ≈ 7.48 gallons

$\forall_{tank}$ ≈ 20,050 ft^3

$\forall_{tank}$ = Volume of tank

$\forall_{tank} = A \cdot h$

A = Circular Area

h = height of tank

$A = \frac{\pi d^2}{4}$

d = diameter of tank

d = 25'

$20{,}050\ ft^3 = \frac{\pi (25')^2}{4} \cdot h$

$h = 40.8' \Rightarrow h \approx 41'$

Antonio Solorio | CE 180 B Hand Calculations | 4/1/2018

Water Demand Per Node:

Land-use Codes	Land Use water Demands (W.D.) gpd/acre	gpm/acre
Neighborhood Commercial - NC	1,300	.9
Low-Density Residential - LDR	2,400	1.67
Light Industrial - LI	2,000	1.39

Node 18 Acres

Low-Density Residential - 21.7 ac

Neighborhood Commercial - 5 ac

Light Industrial - 41.67 ac

Water demand to Node 18:

= (# of NC acres)(Land use W.D) + (# of LDR Acres)(Land use W.D) + (# of LI acres)(Land use W.D)

=> 5 acres (.9 gpm/acre) + 21.7 acres (1.67 gpm/ac) + 41.67 acres (1.39 gpm/acre)

Water Demand For Node 18 = * 85.8 gpm

*Before Peaking Factor is Applied

P.F = 3 for Tulare

Name:	Antonio Solorio	Project:	Children's Hospital	Date:	3/15/18
Total Water Used	18,626	26,822,026			
	gpm	gpd			

Data Table for Hospital During Peak Hour & Fire Flow Off Site (3,500 gpm at Fire Node)

Time	Demand	Head	Pressure
Hours	GPM	ft	psi
0:00	700	342	33
1:00	750	340	33
2:00	700	350	37

Proposed Pipe (East) - Characteristics during Fire Flow (Off Site)

Time	Length	Diameter	Flow	Velocity	Unit Headlos
Hours	ft	in	GPM	fps	ft/Kft
0:00	1320	12.0	442.1	1.3	0.43
1:00	1320	12.0	426.9	1.2	0.4
2:00	1320	12.0	397.9	1.1	8:24

Prposed Pipe (West) - Characteristics during Fire Flow (Off-Site)

Time	Length	Diameter	Flow	Velocity	Unit Headlos
Hours	ft	in	GPM	fps	ft/Kft
0:00	1320	12.0	257.9	0.7	0.16
1:00	1320	12.0	323.2	0.9	0.24
2:00	1320	12.0	302.1	0.9	0.21

RESULTS FOR FIRE FLOW OFF SITE DURING PEAK HOUR

	PRESSURE (PSI)	**DEMAND (GPM)**
HOSPITAL	33	750
	VELOCITY (FT/S)	**UNIT HEADLOSS (FT/KFT)**
PIPE (EAST)	1.2	0.4
PIPE (WEST)	1	0.24

Data Table for Hospital During Peak Hour & Fire Flow for Hospital (2,500 gpm for Hospital NFF)			
Time	Demand	Head	Pressure
Hours	GPM	ft	psi
0:00	700	343	26.98
1:00	750	343	26.77
2:00	700	353	31.17

Proposed Pipe (East) - Characteristics during Fire Flow for Hospital					
Time	Length	Diameter	Flow	Velocity	Unit Headlos
Hours	ft	in	GPM	fps	ft/Kft
0:00	1320	12.0	2494.6	7.1	9.38
1:00	1320	12.0	2480.6	7.0	9.28
2:00	1320	12.0	2448.4	7.0	9.06

Proposed Pipe (West) - Characteristics during Fire Flow for Hospital					
Time	Length	Diameter	Flow	Velocity	Unit Headlos
Hours	ft	in	GPM	fps	ft/Kft
0:00	1320	12	705.43	2.0	1.02
1:00	1320	12	769.42	2.2	1.2
2:00	1320	12	751.57	2.1	1.15

RESULTS FOR FIRE FLOW ON SITE DURING PEAK HOUR		
	PRESSURE (PSI)	**DEMAND (GPM)**
HOSPITAL	33	3250
	VELOCITY (FT/S)	**UNIT HEADLOSS (FT/KFT)**
PIPE (EAST)	7.0	9.4
PIPE (WEST)	2.2	1.2

APPENDIX C. COST ESTIMATES

Name of Project:	Valley Children's Hospital Parking Structure			
Name of Client:	Central Valley Children's Hospital Authority			
Date:	Feb 27 2018			
Quantities by:	REB	AREA:	82300	SF
Prices by:	REB	STALLS:	200	

NO.	ITEM	UNIT	QUANTITY	PRICE	AMOUNT
1	12DT34	SF	27000	$ 11.25	$ 303,750
2	10DT34	SF	20200	$ 11.25	$ 227,250
5	26LB36	LF	920	$ 225.00	$ 207,000
6	26IT44	LF	200	$ 240.00	$ 48,000
7	41IT32	LF	90	$ 245.00	$ 22,050
8	.67'x.67' Col	LF	280	$ 210.00	$ 58,800
9	1'x1' XCol	LF	40	$ 215.00	$ 8,600
10	Wheel Blocks	EA	200	$ 40.00	$ 8,000
11	Striping	LF	10736	$ 0.80	$ 8,589
12	CMU Barrier	SF	2401	$ 9.52	$ 22,858
13	Misc Metal	LB	10000	1.2	$ 12,000

Subtotal	$ 926,896.32
MOBILIZATION (30%)	$ 278,068.90
SUBTOTAL	$ 1,204,965.22
CONTINGENCIES (25%)	$ 301,241.30
TOTAL ($/SF)	$ 18.30
TOTAL ($/STALL)	$ 7,531.03
GRANT TOTAL	$ 1,506,206.52
FOR BUDGET PURPOSES, SAY	$ 1,507,000.00

Table 46: Total Cost Estimates for Parking Structure Foundations and Grading Plan

ITEM NO.	ITEM DESCRIPTION	UNIT OF MEASURE	QUANTITY	UNIT COST	AMOUNT
1	Staking	LS	1	$ 6,000.00	$ 6,000.00
2	Excavation	CY	1,506	$ 50.00	$ 75,300.00
3	Concrete for footing	CY	302	$ 1,050.00	$ 317,100.00
4	Concrete for stem	CY	7	$ 1,050.00	$ 7,350.00
5	Footing rebars	LB	18,970	$ 0.75	$ 14,227.50
6	Dowels	LB	1,532	$ 0.75	$ 1,149.00
7	Form work	SQYD	841	$ 12.00	$ 10,092.00
8	Backfill	CY	1,198	$ 120.00	$ 143,760.00
9	Grading	SQYD	217,000	$ 1.50	$ 325,500.00
10	Imported Borrow	CY	32,257	$ 15.00	$ 483,855.00
				Subtotal	$ 1,384,333.50
				Contingencies (10%)	$ 138,433.35
				Total	$ 1,522,766.85

Table 47: Cost estimates for Storm water Design

Item No	Description	Units	Quantity	Unit Cost	Extension
1	Inlets	EA	33	$3,500.00	$115,500.00
2	Strom sewer 21"	LF	698	$140.00	$97,720.00
83	Storm sewer 24"	LF	357	$100.00	$35,700.00
4	Storm sewer 27"	LF	361	$75.00	$27,075.00
5	Storm sewer 30"	LF	837	$56.00	$46,872.00
6	Storm sewer 36"	LF	795	$50.26	$39,956.70
7	Storm sewer 42"	LF	615	$85.00	$52,275.00
8	Storm sewer 48"	LF	524	$70.00	$36,680.00
9	Storm sewer 54"	LF	491	$95.00	$46,645.00
10	Storm sewer 60"	LF	620	$87.00	$53,940.00
11	Concrete for detention	Ton	50	$110.00	$5,500.00

	pond				
				Subtotal	$557,863.70
				Mobilization (30%)	$167,359.11
				Subtotal	$725,222.81
				Contingency (25%)	$181,305.70
				Total estimated cost	$1,631,751.32

Table 48: Cost estimates for Sanitary Sewer System

NO.	ITEM	UNIT	QUANTITY	PRICE	AMOUNT
1	Excavation	CY	149	$ 1,680.00	$ 250,320.00
2	4-in Pipe	LF	60	$ 17.29	$ 1,037.40
3	5-in Pipe	LF	285	$ 85.48	$ 24,361.80
4	15-in Pipe	LF	1193	$ 171.00	$ 204,003.00
				Total	**$ 479,722.20**

Name of Design:		Valley Children's Hospital Water Connection to City			
Name of Client:		Central Valley Children's Hospital Authority			
Date:		April 10 2018			
Prices by:		JM Eagle		&	IPEX USA LLC
NO.	**ITEM**	**UNIT**	**QUANTITY**	**PRICE**	**AMOUNT**
1	**12" x 12" C900 DR25 Pressure Gasketed Tee**	EA	1	$ 5,370.87	$ 5,371
2	**12" x 12" x 10" C900 DR25 Pressure Gasketed Tee**	EA	1	$ 4,760.18	$ 4,760
3	**12" x 12" C900 DR25 Pressure Gasketed Cross**	EA	1	$ 1,824.44	$ 1,824
4	**12" C900 DR 25 PVC Pipe**	LF	3580	$ 13.51	$ 48,366
		Subtotal			$ 60,321.29
		MOBILIZATION (30%)			$ 18,096.39
		SUBTOTAL			$ 78,417.68
		CONTINGENCIES (25%)			$ 19,604.42
		GRANT TOTAL			$ 98,022.10
		FOR BUDGET PURPOSES, SAY			$ 99,000.00

NO.	ITEM	UNIT	QUANTITY	PRICE	AMOUNT
1	B5ZPBH- Berkley pump	EA	1	$5,643.55	$ 5,644
2	Mitsibushi 25hp VFD	EA	1	$3,516.99	$ 3,517
3	10" DUCTILE IRON	LF	2110	$9.59	$ 20,235
4	8" DUCTILE IRON	LF	1670	$6.30	$ 10,521
5	6" DUCTILE IRON	LF	765	$3.00	$ 2,295
6	4" DUCTILE IRON	LF	60	$1.88	$ 113
7	10" DUCTILE IRON- LABOR	LF	2110	$7.32	$ 15,445
8	8" DUCTILE IRON-LABOR	LF	1670	$6.59	$ 11,005
9	6" DUCTILE IRON-LABOR	LF	765	$6.59	$ 5,041
10	4" DUCTILE IRON-LABOR	LF	60	$6.59	$ 395
11	10" DUCTILE IRON- BACKFILL	LF	2110	$6.50	$ 13,715
12	8" DUCTILE IRON-BACKFILL	LF	1670	$5.83	$ 9,736
13	6" DUCTILE IRON-BACKFILL	LF	765	$5.15	$ 3,940
14	4" DUCTILE IRON-BACKFILL	LF	60	$4.49	$ 269
15	12"x10"x12" T	EA	1	$580.00	$ 580
16	10" Elbow	EA	7	$366.00	$ 2,562
17	10" Valve	EA	6	$2,219.00	$ 13,314
18	10"X6" Reducer	EA	1	$173.00	$ 173
19	10"x5" Reducer	EA	1	$172.00	$ 172
20	Pump Flanges	EA	2	$100.00	$ 200
21	10"x8" T	EA	2	$436.00	$ 872
22	8"X4"T	EA	2	$242.00	$ 484
23	10"X6" Reducer	EA	4	$173.00	$ 692
24	10" T	EA	2	$654.00	$ 1,308
25	8" Reducer	EA	1	$243.00	$ 243
26	8" Elbow	EA	2	$160.00	$ 320
27	8"x8"x6" T	EA	3	$249.00	$ 747
28	Fire Hydrants	EA	8	$4,799.00	$ 38,392
29	CHECK VALVE	EA	2	$6,706.00	$ 13,412
30	Flow meter	EA	1	$3,507.00	$ 3,507
31	3" Valve	EA	1	$ 125.00	$ 125
32	Storage Tank	EA	1	$582,000.00	$ 582,000
33	Misc Fittings	LS	1	$5,000	$ 5,000

Subtotal	$ 765,974.74
MOBILIZATION (10%)	$ 76,597.47
SUBTOTAL	$ 842,572.21
CONTINGENCIES (15%)	$ 126,385.83
GRANT TOTAL	$ 968,958.05
FOR BUDGET PURPOSES, SAY	$ 969,000.00

Table 49: Unit price

Caltrans Code	Item Discribtion	Unit	Unit Price
390138	RUBBERIZED HOT MIX ASPHALT	Ton	$ 93.00
401000	CONCRETE PAVEMENT	CY	$ 95.00
260201	CLASS 2 AGGREGATE BASE	CY	$ 23.00
30064	PAINT PARKING STALL MARKING	LF	$ 4.00
MUTCD R1-1	STOP sign	EA	$ 150.00

Table 50: Cost estimates for Transportation

Cost of Project	
Parking Garage	
Parking pavement concrete	$ 593,750.00
Praking Stripes	$ 10,260.00
Base	$ 243,177.08
Parking Lot Pavement	
Parking pavement RHMA	$ 388,907.40
CLASS 2 AGGREGATE BASE	$ 168,317.45
Parking Lot Stripes	$ 64,800.00
Signs	
Stop Signs	$ 1,500.00

Total cost:	$ 1,470,711.93