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16. Abstract

A site near Victoria on U.S. 59 was selected as a site for the design of a prototype rest area comfort station. Rest room facilities were designed for 500,000 users per year with provisions made for additional units to be constructed later as the number of users increases. Separate parking facilities were provided for small and large vehicles. Rest room buildings were designed with dual men's and women's facilities to insure that at least one rest room will always be available, even during maintenance. Two doors are provided in each rest room. Rest rooms are designed to eliminate illegal sexual activities. Fixtures and materials were selected with ease of maintenance and durability in mind. Rest rooms are heated and air conditioned. The calculations of heating and cooling loads are shown, along with the design of the water and wastewater systems. Plans, elevations, sections, and details are included.

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Design Manual for Rest Area Comfort Stations

by

Kirby W. Perry
David W. Fowler
Carl W. Scharfe
Joseph F. Malina, Jr.

Research Report Number 442-5F

Research Project 3-18-86-442
Design of Rest Area Comfort Stations

conducted for

Texas State Department of Highways
and Public Transportation

in cooperation with the

U.S. Department of Transportation
Federal Highway Administration

by the

Center for Transportation Research

Bureau of Engineering Research
The University of Texas at Austin

November 1987

PREFACE

This report represents the last in a series on rest area comfort design. Many individuals, some of whom have been cited in previous reports, have contributed to the development of this report. Mr. E. W. (Bill) Wilson, (SDHPT) Technical Advisor, has been very supportive and helpful throughout the study and was always available to provide information and advice. Mr. Carl Ramert, former (SDHPT) District Engineer in the Yoakum District, was very supportive of using the site near Victoria for the prototype design. Edward Kristaponis, Federal Highway Administration, provided considerable encouragement throughout the study. Tom Straughan and Brian Rock performed many of the studies which led to this report. The help of Rose Rung in typing the report is greatly appreciated.

Kirby W. Perry
David W. Fowler
Carl Scharfe
Joseph F. Malina, Jr.
Gary C. Vliet

LIST OF REPORTS

Report 442-1, Volume 1, "Investigation of Rest Area Requirements," by W. T. Straughan, David W. Fowler and Kirby W.Perry, October, 1988.

Report 442-1, Volume 2, "Investigation of Rest Area Requirements, Appendix - Pertinent Rest Area Literature," by W. T. Straughan, David W. Fowler and Kirby W. Perry, October, 1988.

Report 442-2, "Evaluation of Energy Sources for Roadside Rest Areas," Brian A. Rock and Gary C. Vliet, December, 1986.

Report 442-3, "Water and Wastewater Systems at Highway Rest Areas," by Carl W. Scharfe and Joseph F. Malina, Jr., October, 1988.

Report 442-4, "Design Recommendations for Rest Areas," by W. Thomas Straughan, Brian A. Rock, Carl W. Scharfe, David W. Fowler, Joseph F. Malina, Jr., Kirby W. Perry and Gary C. Vliet, October, 1988.

ABSTRACT

A site near Victoria on U.S. 59 was selected as the site for the design of a prototype rest area comfort station. Rest room facilities were designed for 500,000 users per year with provisions made for additional units to be constructed later as the number of users increased. Separate parking facilities were provided for small and large vehicles. Rest room buildings were designed with dual men's and women's facilities to insure that at least one rest room would always be available, even during maintenance. Two doors were provided in the design for each rest room. Rest rooms were designed to eliminate illegal sexual activities. Fixtures and materials were selected with ease of maintenance and durability in mind. Rest rooms were designed to be heated and air conditioned. The calculations of heating and cooling loads are shown, along with the design of the water and wastewater systems. Plans, elevations, sections, and details are included.

SUMMARY

The design of a prototype rest area comfort station is presented using recommendations made in previous reports. The design includes the site layout, building, water and wastewater requirements, and electrical power requirements. The facility is designed for 500,000 users annually. Plans include a site plan, a floor plan, elevations, sections, and details.

IMPLEMENTATION STATEMENT

The comfort station design presented in this report can easily be incorporated in rest area construction. The facility will provide users with a moden attractive facility designed to provide security and require low maintenance.

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CHAPTER 1. INTRODUCTION

1.1 BACKGROUND

Rest areas are an integral part of the nation's highways, particularly the interestate system. Initially, rest areas consisted of parking areas adjacent to the highways where travellers could stop for a rest. Tables and simple shelters were added for convenience. Eventually a few locations had restrooms. With the advent of the interstate highway system, rest areas with comfort stations became much more common. Texas was one of the first states to develop an intrastate rest area system.

The rest areas in Texas are usually a few acres in size, with a single access road serving as parking for all vehicles. The restrooms usually have masonry walls which are partial height to permit air circulation and ventilation. The overhanging roofs protect the users from rain. Between the men's and women's restrooms is a mechanical/storage room.

Most of the other states have begun constructing fully enclosed, air conditioned/heated restrooms with ceramic tile floors and walls and stainless steel toilet partitions. The rest areas are 20 to 30 acres or larger. Many have uniformed attendants 24 hours per day, 7 days a week. They are attractively landscaped and well-lighted.

This research project was initiated to provide recommendations for the State Department of Highways and Public Transportation (SDHPT) for improving the design of rest areas and, in particular, comfort stations.

1.2 PREVIOUS RESEARCH

This report represents the last in a series on rest area comfort stations. The previous reports, which should be consulted for additional details, are

CTR 442-1, Investigation of Rest Area Requirements,

CTR 442-2, Evaluation of Energy Sources for Roadside Rest Areas,

CTR 442-3, Water and Wastewater Systems at Highway Rest Areas, and

CTR 442-4, Design Recommendations for Rest Areas.

1.3 SCOPE

This report presents design recommendations for two sites on U.S. 59 northeast of Victoria. Preliminary plans, outline specification, calculations for plumbing water and wastewater requirements, and heating and cooling loads are presented.

Chapter 2 contains the requirements for water.

Chapter 3 contains the requirements for wastewater.

Chapter 4 summarizes the heating and air conditioning and electric power requirements.

Chapter 5 contains the site and building design.

Chapter 6 provides a summary and conclusions.

CHAPTER 2. DESIGN OF WATER SYSTEMS

2.1 ASSUMPTIONS AND KNOWN VALUES

The design of the water and wastewater system at the rest area near Victoria, Texas, is based on estimates of several important parameters as well as site data provided by the SDHPT. These were the assumptions and known values used for the design of both water and wastewater systems.

2.1.1 Traffic Data

(1) Ten percent of mainline traffic stops at the rest area, based on old U.S.59 being a rural highway.
(2) The number of occupants per vehicle using the rest are facility is 2.25, based on 90 percent of 2.5 occupants per vehicle from the Hays County survey.
(3) The annual average daily traffic (ADT) is 13,600; the weekly peak ADT is 16,200, on Fridays.
(4) The ratio of the peak hourly traffic to average daily traffic is 0.16.

2.1.2 Water System

(1) Maximum fixture use is 30 users/ hour/fixture.
(2) Water use is 3 gallons per rest room user .
(3) Two hose bibs will be needed for sprinkling purposes. Water demand from these devices will be 5 gpm for each sprinkler.
(4) The two existing wells will be used for water supply. (Data for the well pumping tests and chemical analysis are available.)

2.1.3 Wastewater System

(1) Treated effluent will meet minimum state discharge standards of BOD of 20 mg/l and TSS of 20 mg/l. Design limits are BOD of 10 and TSS of 15 mg/l.
(2) Wastewater flow rates equal water demands less water used for sprinkling (i.e., there are no dump station facilities and drinking water demands are negligible).

2.1.4. Site Characteristics Data

(1) Land area is approximately 10 acres.
(2) Highest land elevation is 56 feet above mean sea level.
(3) Soil is Inez B based on U.S. Soil Survey for Victoria County
    (a) Grades of 0 to 2 percent.
    (b) Fine sandy loams near the surface grading to clays at 14 inches.in depth; clays predominate at depths below 14 in.
    (c) Percolation rates for depths below 14 in. are 0.06 to 0.2 in/hr; percolation rate at a depth of 3 ft is zero.
(4) Groundwater levels are approximately 30 ft below the surface in both wells at the site.
(5) The former picnic area (along old U.S. 59) is approximately 0.6 acre in size.

2.2 ANALYSIS OF TRAFFIC DATA

The design of the water systems is based on traffic data on the main highway for the east bound lane (EBL) of U.S. 59. The west bound lane (WBL) and the EBL rest area water systems will be identical. Separate systems were chosen because there are existing wells on each side of the highway at present, and one rest room facility can remain in operation, if the other rest area is closed for maintenance or repair. Hourly highway traffic data from a permanent traffic counter (station S116) 0.8 mile east of the site was obtained for the calendar year 1986 from the Transportation Planning Division of the SDHPT.

The traffic data were loaded from magnetic tape to the University's IBM mainframe computer and a statististical package called SAS was used to perform frequency analysis on (a) Two-way Average Daily Traffic (ADT), (b) Two-way Maximun Hourly Traffic (MXTHR), (c) Eastbound Maximum Hourly Traffic (MXHRE), and (d) Westbound Maximum Hourly Traffic (MXHRW). The frequencies calculated and used in design are for 1986 data only, the latest available. The design allows for expansion to accommodate increase in traffic flow or facility use. Ten percent of the vehicles on the main highway are assumed to stop at the rest area. This value is one percent greater than the SDHPT value used for rural highways.

The traffic data indicate that peak traffic days usually occur on holidays and that monthly average daily traffic (ADT) variation is minor, i.e., there are no seasonal effects. The annual ADT for US 59 is 13,600 vehicles. The highest ADT for an average week occurs on Friday and is ~16,200 vehicles.

2.3 FIXTURE REQUIREMENTS

The rest room fixture requirement for the Victoria site is eight fixures (six toilets and two urinals) per rest room, based on the Oregon design chart, which is the standard used by the Building Division Section of the Maintenance Division of the SDHPT. A cumulative frequency analysis for the eastbound lane (EBL) of U.S. Highway 59 will be used as a check.

A cumulative frequency analysis was performed on the maximum hourly traffic count for each day of 1986 The results, shown in Figure 2.1, indicate that a particular maximum hourly traffic corresponds to a cumulative percent. This percentage is the portion of the time that the maximum hourly traffic is equal to or less than a selected traffic count.

2.3.1 Calculating Peak Users Per Hour

Peak users per hour = number of fixtures x users/fixture/hr
= 8 fixtures x 30 users/fixture/hr
= 240 users/hr

2.3.2 Estimating Mainline Traffic

Mainline traffic = users/hr x vehicles/users x (1 + percentage of mainline traffic stopping)
= 240 x (1/2.25) x (1/0.10)
= 1067 veh/hr

2.3.3 Determining Number of Fixtures

The cumulative percent corresponding to 1,067 veh/hr is ~98 percent; therefore, eight fixtures will be sufficient 98 percent of the time.

2.4 WATER DEMANDS

2.4.1 Peak Hourly Rest Room Demand

The peak hourly demand is dictated by rest room usage and water used for lawn sprinkling purposes. Since a rest area attendant will be on duty, two mobile sprinklers can be used for lawn watering purposes.

(1) Rest room peak water demand.

Peak hourly demand is 240 users/hr x 3 gal/hr = 720 gal/hr or.12 gpm. However if the design is based on 300 users/hr (60 users/hr for urinals and 30 users/hr for toilets) the peak demand is 15 gpm. Therefore, the design water demand is 15 gpm

(2) The lawn sprinkling demand for two sprinklers is 2 x 5 gpm = 10 gpm.

(3) Total peak hourly demand = 15 + 10 = 25 gpm

2.4.2 Instantaneous Peak Demand

(1) The Hunter fixture method can be used. Therefore for six flush valve toilets, two flush urinals and eight sinks (Table 2.1, page 14 in Ref. 2).

Total fixture units = 6(10) + 2(5) + 8(1.5)
= 82 fixture units

Peak instantaneous
demand = 62 gpm from Figure 2.3

(2) If all eight fixtures are flushed simultaneously for 6 seconds at 30 gpm, the peak instanteous demand
= 8 fixtures x 30 gpm x 6 sec
+ 60 sec/min
= 25 gallons

This situation requires eight people to flush fixtures at exactly the same time, which is not likely; a more reasonable situation would be three fixtures flushing simultaneously, resulting in 10 gallons.

2.4.3 Peak Daily Water Demand

(1) The peak one-directional traffic for 1986 was 15,000 cars/day, on November 30, in the eastbound lane.

Peak daily demand = 15,000 cars/d x 2.25 users/veh x 3 gal/user x 0.10 (percentage of cars stopping at rest area)
= 10,125 gal/d.

(2 The lawn watering requirements for eight hours of grounds sprinkling are

Sprinkler demand = 10 gpm x 60 min/hr x 8 hours
= 4,800 gal.

(3) Total peak daily demand

Sprinkling is less likely in November, the second highest traffic day of the year is in July (8,400 gal/d); thus the peak daily demand is 13,200 gal/day when lawn sprinkling is taken into consideration.

2.4.4 Average Hourly Water Demand

(1) Average one-lane hourly main highway traffic
= 13,600 cars/d x d/24 hr
x 0.5 (one-lane)
= 282 cars/hr

(2) Average rest room water demand
= 282 x 2.25 x 3 x 0.10
= 190 gal/hr = 3.2 gpm

The conversion factor from vehicles on the main highway to gallons is equal to 0.675 (i.e., 10 percent stopping x 2.25 users/veh x 3 gal/user) and will be used in other conversions from mainline traffic to water use or wastewater produciton.

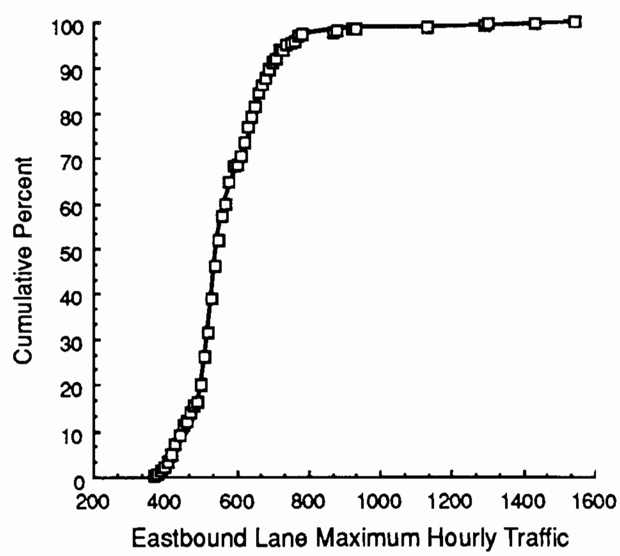


Fig 2.1. Cumulative frequency plot for eastbound lane traffic.

2.5 WATER SYSTEM DATA AND SYSTEM FLOWSHEET

2.5.1 Well Data

Data for the two water wells existing at the site are shown in Table 2.1. These data were provided by the SDHPT.

2.5.2 Water System Flowsheet

A schematic of the water system is presented in Figure 2.2. A storage tank and booster pumps are included because (a) the well pumps cannot meet peak hourly demands, (b) this design reduces starts/stops on the well pumps, and (c) this design allows ample chlorine contact time.

2.6 REST ROOM DESIGN

The sizing of pipes and maximum pressure losses arebased on the following guidelines:

Pipe velocity limits:	4 fps for fixture pipes
	8 fps for water supply main
Fixture flowrates:	30 gpm for flush valve toilets
	15 gpm for flush valve urinals
	3 gpm for lavatories
Pressure requirements:	25 psi for flush valve toilets
	15 psi for flush valve urinals.

2.6.1 Pipe Sizing Using Velocity Limits

(1) Copper type L pipe will be used inside the building.

(a) Toilets

Using a nominal 1-1/2 in. pipe (area = 0.012 sq ft) and a flow rate = 30 gpm (0.067 cu ft/sec, cfs), the

velocity = 0.067/0.012 = 5.4 fps

Since 5.4 fps > 4.0 fps, *use a nominal 2 in.pipe.*

(b) Urinals

Using a nominal 1-1/4 in. pipe (area = 0.0087 sq ft), and a flow rate = 15 gpm (.033 cfs) the

velocity = 0.033/0.0087 = 3.8 fps. *Use 1-1/4 or 1-1/2 in.pipe.*

(c) Lavatories

Using a nominal 1/2 in.pipe (area = 0.0016 sq ft) and a flow rate = 3 gpm (.0067 cfs) the

velocity = 0.0067/0.0016 = 4.12 fps

Since 4.12 fps > 4.0 fps, *use a nominal 3/4 in. pipe.*

(2) The velocity limit = 8 fps for the water main. Type K copper pipe will be used. The peak instantaneous demand is 62 gpm (via Hunter's method) and/or 100 gpm (10 gallons used in 6 second), using a flow rate = 100 gpm (0.233 fps) and a nominal 2-1/2 in. pipe (area = .032 sq ft) the

velocity = 0.233/0.032 = 7.1 fps. Therefore *use a nominal 2-1/2 in. pipe.*

2.6.2 Pressure Losses From Hydropneumatic Tank to Rest Room

The equivalent length for valves and appurtenances is 0.5 of the developed length of the pipe (for standard plumbing practice). Assume the head loss through the meter is 2 psi at 100 gpm, a friction factor of 0.02 for copper pipe, and the developed length from the farthest fixture to the pressure tank is 330 ft.

(1) Pipe equivalent lengh (PEL)

PEL = 330 + 0.5(330) = 500 ft.

TABLE 2.1. WATER WELL CHARACTERISTICS

Parameter	Well #1	Well #2
Traffic direction served	Eastbound	Westbound
Side of U.S. 59	North	South
Well pumps installed	1 HP, 14 Stage	1 HP, 14 Stage
Well pumps capacity	10 gpm assumed	
(actual capacity not available)	10 gpm assumed	
Well depth	104 ft.	95 ft.
Static well water level	30'8"	29'2"
Well pump depth below surface	70 ft.	70 ft.
Well pump test results	24 gmp	17 gmp
Water quality data (all in mg/l)		
Hardness (as $CaCO_3$)	722	636
Chloride	226	197
Manganese	.054	.044
Iron	.024	.027
Nitrate (as N)	3	2.5
Floride	.82291	.76283
Sulfate	(not available for either well)	
Bacteriological results		

(2) Friction losses for 2-1/2 in. diameter pipe; flowrate = 1 00 gpm.

$$h_L = f\,LV^2 / (D2g) = LQ^2 / (1.234D^5g)$$

where

- f = pipe friction factor
- L = pipe equivalent length, ft
- V = velocity, ft/sec
- D = pipe diameter, ft
- g = force of gravity, 32.ft/sec^2
- Q = flowrate, cu ft/sec

For D = 2-1/2 in. (0.20 ft) and Q = 100 gpm (.223 cfs)

$$h_L = (.02)(500)(0.223)^2 / (1.234 \times 32 \times .20^5) = 39 \text{ ft. } (\sim 17 \text{ psi})$$

(3) Friction losses for a 3 in. diameter pipe: Q = 100 gpm,

$$h_L = (.02)(500)(0.223)^2 / (1.234 \times 32 \times 0.242^5) = 5 \text{ ft. } (6.5 \text{ psi})$$

The difference in price between a 2-1/2-in.and 3-in. nominal pipe is approximately $2.80/ft; therefore the difference in capital costs for 500 ft. of pipe is ~$1,400. Since most commercially available pressure tanks can supply 70 psi or more use the 2-1/2-in. pipe size. The meter loss of 2 psi should be added to friction losses.

2.7 HYDROPNEUMATIC TANK DESIGN

2.7.1 Pressure Range

The minimum pressure for the peak instantaneous demand required for the fixtures is 25 psi + 17 psi + 2 psi, or 44 psi. Ideally a minimum pressure of 40 psi at the fixtures is desirable; therefore, the maximum required pressure is 60 psi. Therefore, design pressure range is 40 to 60 psi.

2.7.2 Tank Usable Volume

The maximum starts per hour for the booster pump are four for a 15-minute cycle time; since booster pumps will alternate in operation the actual cycling time can be cut in half, to 7.5 minutes . The pump flowrate capacity is 25 gpm to meet the peak hourly water demand. The usable volume (drawdown) in the tank is

$$V_u = (\text{Pumping rate} \times \text{cycling time}) / 4$$

$$= (25 \times 7.5) / 4 = \sim 47 \text{ gallons } (2)$$

This usable volume meets the instantaneous peak demands even if two peaks occur in succession. It should be noted that, if a shorter cycle time is used, the usable volume, although smaller, still will meet the instantaneous demands (for example, if 6 starts/hr are used, the usable volume is ~ 31 gallons).

2.7.3 Tank Size

The largest pre-charged standard tank that is readily available is 120 gallons. Tanks of this size can provide up to 37 gallons of drawdown and 70 psi. Two tanks in parallel can meet the peak instantaneous and peak hourly demands. Therefore use two 120-gallon tanks in parallel.

2.8 BOOSTER PUMPS AND STORAGE REQUIREMENTS

2.8.1 Booster Pump Size

The booster pump should be sized to deliver the peak hourly demand and meet the pressure requirements of the

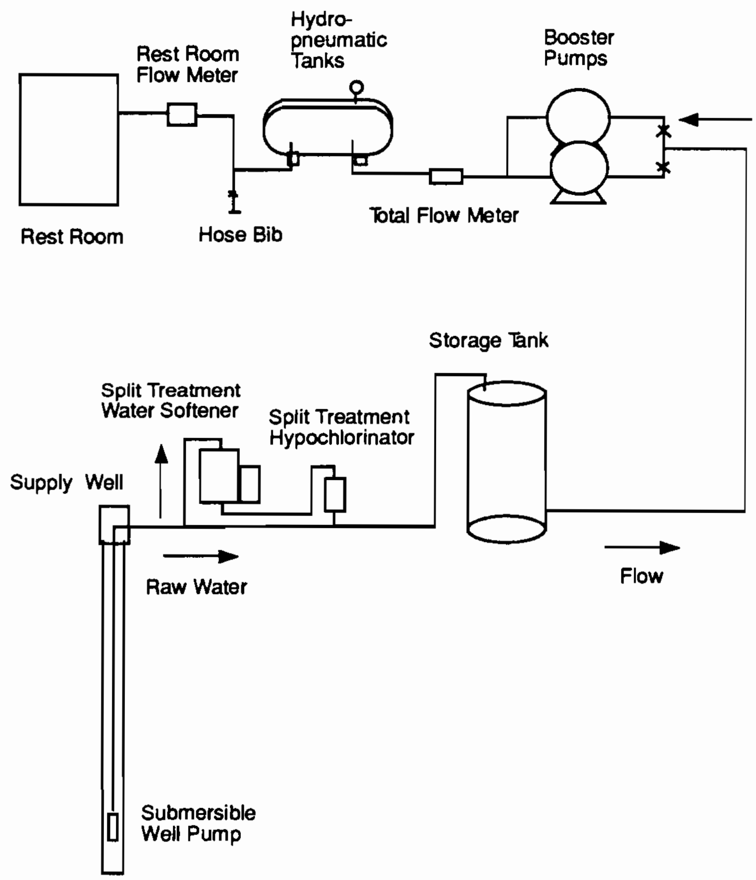


Fig 2.2. Rest area water system flowsheet.

pressure tank. At a maximum pressure of 60 psi and a meter friction loss of 2 psi the pump will have to provide ~ 150 ft of head.

Pump capacity,

Q_p = 0.0557 cu ft / sec (25 gpm)

Pump delivered head, H_p = 150 ft

Pump Horsepower (minimum) = $Q_p H_p$ x 62.4 lb_f /cu ft x 1 HP/ (550 lb_f / cu ft)

= ~ 0.95

In practice a 2- or 3-HP pump will be used since these pumps are readily available in manufacturers' catalogs.

2.8.2 Storage Requirement

Storage is required if the well pump cannot meet the peak hourly demand. The well pumps at the site are assumed to provide 10 gpm at maximum capacity. The hourly eastbound water demands based on multiplying hourly traffic values by 0.675 for the peak day of the year (November 30, 1986) are presented in Figure 2.3.

(1) Storage based on traffic.

The wells provide 10 gpm, or 600 gal/hr. The maximum demand based on 300 users/hr is 15 gpm. The storage volume needed isshown by the shaded area in Figure 2.3 and is approximately 1,800 gallons.

(2) Storage required for sprinkler demands.

Assume that sprinklers will operate 6 hours during the peak hours.

Sprinkler demands = 6 hrs x 10 gpm x 60 min/hr
= 3,600 gallons

(3) Total storage (worst case, sprinkling occurring in November).

Total storage = 1,800 + 3,600
= 5,400 gallons

2.8.3 Storage Tank Dimensions

A 10-ft-diameter cyclindrical tank 12 ft deep allows 2.5 ft of freeboard.

2.9 WATER SOFTENING

The hardness of the water is 722 mg/l. A softening unit is optional but is recommended if hot water is provided at the rest area. Split treatment design will be used to keep a calcium residual of ~100 mg/l. Therefore, six-sevenths of the flow from the well requires treatment. The chlorinator will use the water from the softener for chlorine mix water (Figure 2.2).

2.9.1 Grains of Hardness per Gallon

There are 17 mg/l of hardness (as $CaCO_3$) per 1 gr/gal., so

722 mg/l x (1 gr per gal/17 mg per l) = 42.2 gr/gal

2.9.2 Grains Removed per Softening Cycle

(1) The ADT for one-lane traffic will be used to design the softening unit.

Eastbound ADT = 6,800 veh/day

(2) Gallons to be softened per day.

Gal/d = 6,800 x 0.675 x 6/7 = 3,935 gal/d

(3) Grains of hardness to remove per day.

3,935 gal/d x 42.2 gr/gal = 166,027 gr/d = ~166 kgr/d

(4) The choice of an appropriate softening cycle depends on

(a) how often the maintenance crew can replace salt,
(b) the volume of waste generated over the backwash cycle and how it will be handled, and
(c) space available for the softening unit.

2.9.3 Typical Operating/Design Values for Softeners

Minimum resin bed depth	24 in.
Brine concentration for regeneration	10 % NaCl
Resin exchange capacity	20 - 30 kgr / cu ft
Regeneration level	15 lb NaCl / cu ft
Regeneration flowrate	1 gpm / cu ft
Rinse flowrate	1 gpm / cu ft, initial then 1.5 gpm / cu ft
Rinse water requirements	20 -50 gal / cu ft or two bed volumes
Backwash time	5 -15 minutes
Backwash flowrate	5 - 10 gpm per square foot of bed surface area
Service flowrate	2 gpm / cu ft

2.9.4 Design One-Day Softening Cycle

Calculations for a one-day cycle are

(1) Grains to be removed, 1-day cycle = 166 kgr/cycle

(2) Resin required, assume resin capacity is 25 kgr/cu ft.

Resin required (cu ft) = 166 kgr x 1 cu ft/25 kgr
= 6.6

(3) Gallons of brine required for regeneration.

(a) A brine concentration of 10 %per cent by weight requires 0.92 pounds of salt per gallon of water or 1.08 gallons of water per pound of salt.

(b) The gallons of brine needed for regeneration can be calculated as follows:

Gallons = 1.08 gal/lb NaCl x lb NaCl/cu ft x cu ft resin
= 1.08 x 15 x 6.6
= ~ 107 gallons per cycle.

(4) Rinse gallons required; assume 30 gal/cu ft is adequate,

Rinse gallons = 30 x 6.6 cu ft = ~200 gallons

(5) Backwash gallons required are usually given by manufacturer. For a 2-ft-diameter softener and a 8 gpm/cu ft backwash which runs for 10 minutes, 250 gallons are required.

(6) The actual performance of softeners is based on manufacturer's data. The following softener specifications are from the Bruner Corporation. Minimum and maximum exchange capacities, kilograins removed before backwash, are given in Table 2.2.

(7) These data indicate that for a one-day cycle the 140/210 kgr softener can be used. For a 140/210 kgr softener the brine maker would have to be refilled once a week if the maximum amount of salt for regeneration is used (700/105). The waste volume produced per day would be approximately 435 gallons. For two cycles per week the 500/750 could be used. The brine maker would have to refilled every two weeks if the maximum amount of salt for regeneration is used (900/225). The waste volume produced each regeneration cycle (every 3.5 days) would be approximately 1,008 gallons.

The 3.5 day regeneration cycle can be timed to occur Monday morning and early Thursday evening since these times will not coincide with peak water demand hours. The one-day regeneration cycle should be timed to occur in the early morning hours to avoid peak daily hourly demands.

(8) The brine produced must be handled by the wastewater treatment system. The volume of saltwater produced from the regeneration operation may have to be stored and slowly fed into the wastewater treatment system because of the high salt content (29,200 to 40,000 mg/l). The sizes of the softening tanks are not substantially different and are not a major concern in design. A 2-ft access above the softener must be provided for the tank.

(9) The power requirements for each cycle controller typically is 6 watts. Therefore power requirements are dependent on the number of control components using power and the number of softening cycles.

(10) At the 10-gpm flowrate provided by the well pump the maximum operating pressure drop across the resin will be about 10 psi and the drop across appurtenances about 2 psi. The well pump discharge pressure should be able to overcome this drop with no problem.

2.10 CHLORINATION

The Texas Department of Health requires disinfection of all groundwater drinking supplies. Automatic dosing will be required at the Victoria site because the flows can vary more than 50 percent above or below the mean flow rate. State codes require a minimum chlorine residual of 0.5 mg/l at all points in the system after the chlorination equipment (see Fig. 2.2).

2.10.1 Chlorine Demand

In general, chlorine demands should be determined by treating a series of well water samples with varying chlorine (or hypochlorite) dosages. The dosage that gives the desired

TABLE 2.2. MINIMUM AND MAXIMUM EXCHANGE CAPACITIES

	Softener Exchange Capacity (min/max. kgr)					
Parameter	140/210	200/300	300/450	400/600	500/750	600/900
Resin, cu ft	7	10	15	20	25	30
Tank dimensions						
Diameter, in.	24	30	30	36	36	42
Height, in.	54	54	60	60	72	60
Brine tank						
Dameter, in.	24	24	30	39	39	42
Height, in.	60	60	60	60	60	60
Brine maker						
Salt capacity, lb	700	600	900	1500	1500	1500
Regeneration, lb						
Salt per cycle						
Maximum	105	105	225	300	375	450
Minimum	42	60	90	120	150	180
Regeneration, gal/cycle	~115	~160	~245	~325	~400	~490
Backwash, gal/cycle (8 gpm /sq ft, 10 min.)	~250	~390	~390	~565	~565	~770
Rinse, gal/cycle (23.5 gal/ft depth)	~106	~106	~118	~118	~141	~118
Total waste, cycle	435	608	753	1,008	1,106	1,378

residual after an expected contact time can be determined. Since the bacteriological test results for well samples are not available and there are some reduced chemicals in the water, an estimate of 4.5 mg/l will be used as the chlorine demand for design purposes. This demand plus a 0.5 mg/l residual means 5 mg/l of chlorine have to be supplied; therefore,

Chlorine required = 5 mg l x average flowrate
= 5 mg/l x 4600 gal/d x 3.785 l/gal x 10^{-6} kg/mg x 2.2 lb/kg
= ~0.19 lb/d

2.10.2 Chlorine Safety Considerations

A liquid or solid hypochlorite system is safer and cheaper than a gas injection system for such a low chlorine requirement.

2.10.3 Chlorine Supply Requirements

(1) Calcium hypochlorite $Ca(OCL)_2$ will contain 70 percent available HOCL for disinfection. The quantity of hypochlorite needed to meet the chlorine requirement is

$Ca(OCl)_2$ = (0.19 lb/d) / .70 = 0.27 lb/d or about 8.2 lb/month.

(2) If a 5.25 percent sodium hypochlorite liquid bleach is used (0.44 lb HOCL/gal bleach), then the required volume of bleach per day needed is

Bleach required = 0.19 lb HOCL/d x 1 gal bleach / 0.44 lb HOCL
= 0.43 gal/d or about 13 gal/month.

(3) The chlorinated mix water will contain 100 mg/l of hardness after mixing with the raw water not treated, precipitation of calcium carbonate could be a problem if the alkalinity is high enough, thus chlorination with bleach is recommended

2.10.4 Hypochlorination Equipment Sizing

The size of the hypochlorinator pump and tank depends on the well pump flowrate, the desired intervals between hypochlorite additions, and the required chlorine residual. To satisify the average daily water demand the pump will operate for 7.66 hours each day. The hypoclorinator feeder pump also will run 7.66 hours per day. The chlorine is being feed into the split treatment line (flowrate equal to 8.57 gpm), but 5 mg/l must be maintained for the well pump flowrate (10 gpm).

(1) The chlorine concentration in the split treatment line can be calculated as

$$Cl_{split} = Q_w/Q_{split} \times Cl_d$$

where

Cl_{split} = Chlorine concentration in the split treat ment line, mg/l
Q_w = Well pump flowrate, gpm
Q_{split} = Split treatment line flowrate, gpm
Cl_d = Desired chlorine concentration, mg/l
Cl_{split} = 0/8.57 x 5
= 5.83 mg/l.

(2) The product of Cl_{tank} and Q_{fp} is a known constant and can be calculated as

$$Cl_{tank}Q_{fp} = Q_{split}Cl_{split}$$

where

Cl_{tank} = Chlorine concentration in the hypochlorinator tank, mg/l
Q_{fp} = Chlorinator feeder pump flowrate, gpm
Q_{split} = Split treatment line flowrate, gpm
Cl_{split} = Desired chlorine concentration in split treatment line, mg/l.

The values of Cl_{tank} and Q_{fp} are limited by available feed pump sizes and what concentrations of chlorine in the mixer tank are safe for the equipment.

(3) Variable speed diaphram chemical feeder pumps are available that deliver 3 to 30 gpd; select a value of 20 gpd (~0.014 gpm). The tank chlorine concentration is

Cl_{tank} = 8.57/0.014 x 5.83 mg/l = ~3,570 mg/l (~.03 lb/gal).

Tanks are made of high density polyethylene so that this chlorine concentration is acceptable.

(4) The volume of the hypochlorinator tank is determined by the frequency between refills of the tank. The pump will run for only 7.66 hours per day (~460 minutes) so the volume used per day will be:

Volume used per day = 460 x Q_{fp}

(5) Design for refill every 10 days; the refill schedule should be once/week, which means that ten days allows for holidays or a scheduling problem. Thus,

Hypochlorinator tank volume = 4,600 x Q_{fp}

(6) The tank volume can be calculated as

Tank volume = 4600 x 0.014 = ~65 gal.

(7) Tanks are commmonly available in the 15-to 75-gallon range. Use a 65-gallon tank.

(8) For a 65-gallon tank, approximately 4.4 gallons (563 oz.) of bleach are required for a chlorine concetration of 3,570 mg/l.

The capital costs of the water system are estimated using information from manufacturers' representatives and Austin contractors and companies. The operating costs are estimated in terms of man-hrs and kW-hrs rather than in dollars. Cost estimates for the water system are for one comfort station only.

2.11 COST ESTIMATES

2.11.1 Capital Costs

	Cost in dollars
(1) Well drilling and well pumps	none
(2) Hypochlorination unit	
(a) Chemical feeder pump	375
(b) Tank & platform	500
(c) Mixer unit	200
(3) Chlorine residual testing kit	40
(4) Softener (Bruner) Units	
(a) Fully automated with brine tank, softening tank, control panels and resin	
1) 1-day cycle (210,000 kgr cap.)	
single	2,455
twin	4,495
2) 3.5-day cycle (750,000 kgr cap.)	
single	6,106
twin	11,520
(5) Water meters, 2 meters at $600 each	1200
(6) Storage Tank Galvanized steel, $0.26/gal, tank size is 7,048 gallons (10-ft diameter x 12-ft high)	1,832
(7) Booster pumps Deming centrifugal, head = 140 ft., flowrate = 25 to 68 gpm, two at $450 per pump	900
(8) Hydropneumatic Pressure Tanks	
(a) Pre-charged, 30 to 70 psi rated, 37 gal drawdown volume =120 gallons, two at $400 each	800
(b) Control panels & associated equipment	500
(9) System Piping	
(a) Copper type K pipe, 2-1/2 in. 500 ft. (max. length) at $8.40/ft	4,200
(b) Copper type L pipe, 2 in. 50 ft. at $6.20/ft.	310
(10) Installation Cost Assume $20/hr/person for labor and it takes 10 laborers working for a total of 24 hours (half a week to install system)	4,800

2.11.2 Total Capital Cost (meters not included)

(1) 1-day softening cycle, single softener	17,762
(2) 3.5-day softening cycle, single softener	21,410
(3) Costs for two sides	42,000

2.11.3 Operating Costs

(1) Supply Costs	$/yr
(a) Bleach, $0.75/gal, 156 gal/yr	120
(b) Salt	
1) 1-day regeneration cycle $7.00/100 lb at max of 105 lb/cycle	2,675
2) 3.5-day regeneration cycle, maximum of 375 lb/cycle, $7.00/100 lb	2,730
(c) Water quality testing,	
1) Bacteriological analysis, one sample per month at $5/sample	60
2) Annual sanitary survey	25
(d) Total supply/sampling costs per year 2,940	
Cost for two sides	~5,900

(2) Man Power Requirements	Man-hrs/yr
(a) Hypochlorination mixing, 0.5 man-hr/week	5
(b) Chlorine residuals testing, 0.5man-hr/month	6
(c) Salt replacement in brine tank 0.5 man-hr/week, max.	26
(d) Routine equipment maintenance	10
(e) Total Requirements per year (both sides)	94
(3) Power Requirements	kW-hr/yr
(a) Well pumps, 1HP for 7.66 hrs/day, (1 HP = 1.341 kW)	3,750
(b) Hypochlorinator feeder pump & mixer, Assume both add to 0.5 HP, run for 7.66 hrs/d	1,875
(c) 2 controllers, cycle time of 1 hr	
1) 1 day softening cycle	4.4
2) 3.5 day cycle	1.25
(d) Booster pumps 3 HP pumps, running time 1.5 hrs/d	2,200
(e) Control panels for booster pumps, storage tank, and chlorine feeder pump	200
(f) Total power requirements	8,030
(g) Requirement for two sides	~16,000

The costs of the water system are presented in Table 2.3. The categories listed were chosen so that the various aspects of costs could be compared; calculations of the annual cost for major repairs are listed below the table. Labor costs are assumed to be $10/man-hr and power costs $0.07/kW-hr.

TABLE 2.3. COST FOR WATER SYSTEM (BOTH SIDES)

Capital ($)	Supply & Sampling ($/yr)	Manpower (man-hr/yr)	Power (kW-hrs/yr)	Annual Cost ($) [1]
41,000	14,000	100	8,030	13,600

[1] Cost of major repairs: assume Ca = $5,000 at x = 10

CHAPTER 3. WASTEWATER SYSTEM DESIGN

3.1 SELECTION OF SYSTEM

These alternative wastewater treatment systems were considered for the Victoria site:

(1) Pond systems (evaporative, overflow),

(2) Septic tank/leachfield system,

(3) Extended aeration treatment plant system, and

(4) Evapotranspiration bed system.

The systems were evaluated based on (a) environmental factors, and/or (b) Texas SDHPT management objectives. The assumptions and values given in Section 2.1 are applicable to the design of wastewater systems.

A major site constraint was the availability of land for a wastewater treatment system. Approximately 3.5 acres of land were available on the eastbound side and 2 to 2.5 acres on the westbound side. Most of the land available for a treatment system is densely wooded (approximately 75 to 90 percent) and the SDHPT expressed a desire to maintain these woods, if possible; this added a further constraint to design.

3.1.1 Pond Systems

The land constraints at the site eliminated evaporative ponds since approximately 3 acres are required. An example calculation is presented in CTR 442-3, Appendix F. The area required for an overflow pond is 0.6 acre plus a buffer zone. The use of ponds also requires extensive site modifications, such as removing trees and excavation.

3.1.2 Septic Tank/Leachfield System

The very low percolation rates for the soil at the site eliminated the use of septic tank systems because of the large land area required. In addition, the Texas Department of Health regulations [3] require a percolation rate of less than 60 minutes/inch [Fig. 3.5, page 48 of Reference 4]. The percolation rates at the site, based on U.S. Soil Survey data indicate a minimum rate of 300 minutes/inch.

3.1.3 Extended Aeration Package Plant System

An extended aeration treatment system was best suited for the site because of the small land area and minimal site modifications required. Discharge permits already had been approved for discharge to Garcitas Creek. Spray irrigation of the treated wastewater was not considered because land for a buffer zone was not available. An overland treatment system would have required an extensive runoff collection system because of the heavy rains possible in the region. Thus, a package plant system (or a sequential batch reactor system) with discharge to Garcitas Creek was the best option at the site.

3.1.4 Evapotranspiration Beds System

An evapotranspiration system requires a septic tank or some other type of solids settling component. Estimation of actual evapotranspiration rates is difficult for these systems; therefore, there is more risk associated with using an evapotranspiration bed at the site.

3.2 WASTEWATER FLOWRATES AND CHARACTERISTICS

3.2.1 Wastewater Flowrates

Wastewater flows were assumed to be equal to the water use since RV dump stations would not be provided at the site. The total highway traffic was used for design purposes since the wastewater treatment system would receive flows from comfort stations on each side.

(1) Average Daily Flow

Average daily flow = 13,600 veh / d x 0.675
= ~9,200 gal / d

(2) Peak Daily Flow

The peak daily traffic for 1986 occurred on November 26 and was 23,670 vehicles;

Peak daily flow = 23,670 veh / d x 0.675
= ~16,000 gal / d

(3) Minimum Daily Flow

The minimum traffic day for 1986 occurred on January 8 and was 10,140 vehicles; therefore the

Minimum daily flow = 10,140 veh / d x 0.675
= ~6,850 gal / d

(4) Peak Hourly Flows

Although the peak traffic day of 1986 occcurred on November 26 the highest peak hourly traffic occurred on November 30. The hydrographs for this day are shown in Fig. 3.1 and were calculated using SDHPT hourly trafffic counts. The adjustments on the November 30 hydrograph take into account the fact that toilets limit peak hourly flows. From the hydrograph:

Peak hourly sustained flow = 1,350 gal / hr

(5) Minimum Hourly Flows

Minimum hourly flows are in the 54 to 110 gal/hr range for five to six hours, based on traffic counts. The minimum flows can be zero, if there are no comfort station users at night.

3.2.2 Wastewater Characteristics

The wastewater characteristics estimated below are from limited data shown in Table 3.1 on page 28 of CTR 442-1 and best engineering judgement.

(1) BOD = 200 mg/l

(2) TSS = 200 mg/l

(3) TKN = 45 mg/l

(4) NH_3 = 30 mg/l

3.3 WASTEWATER TREATMENT FLOWSHEET

The flowsheet for the wastewater treatment system is shown in Figure 3.2, and the flowsheet for the extended aeration package plant is shown in Figure 3.3. Grinder pumps are used in the collection and pumping system because they (a) can be easily automated, () they are more economical than a pumping station for the flows experienced at the rest area, and (c) they allow the use of a smaller discharge pipe at the treatment plant.

3.4 WASTEWATER COLLECTION AND PUMPING

3.4.1 Texas State Codes for Pressure Collection Systems

(1) Sewers

(a) Flow velocities should be in the range of 2 to 5 ft/sec.

(b) Installation of cleanouts and a means of flushing all lines in the system are suggested.

(2) Pumps

(a) The wet well holding capacity should be capable of storing flows for power outages or equipment failures of short duration as specified by the Texas Department of Health.

(b) Dual grinder pumps should be provided (three may be required by the Texas Department of Health).

(c) Grinder pumps shall have backflow prevention devices.

3.4.2 Rest Room Piping

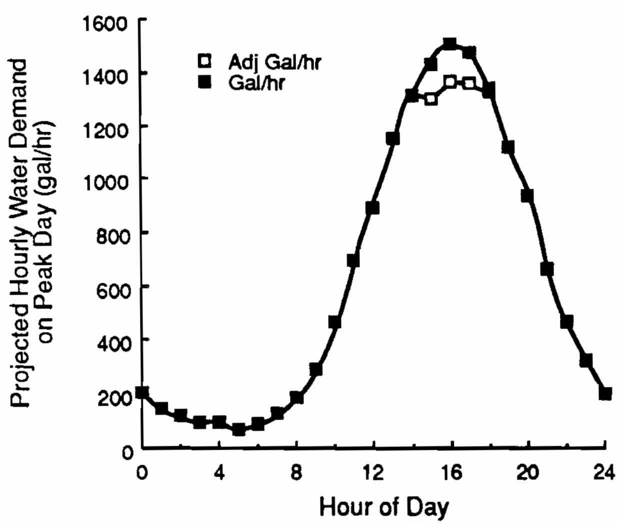


Fig 3.1. Peak daily hydrograph for two-way traffic (November 30, 1986).

(1) Minimum pipe sizes from fixtures should be as follows:

Flush water closets	3 in.
Urinals	2 in.
Lavatories	1.5 in.

(2) Cleanouts will be provided at each bend in the rest room wastewater pipes (i.e four cleanouts for each rest room building).

(3) The size of the main leaving the rest room will be based on an instantaneous demand of 62 gpm. The flow velocities for 3- and 4-in. pipe are

4 in.	1.56 fps
3 in.	2.81 fps

A 4-in. main should be used. The distance from the rest room to the grinder pump lift station is short (perhaps 10 to 20 ft) and the slope over this short distance (a 1 to 2-ft drop) should minimize settling of solids in the pipe.

3.4.3 Grinder Pump Lift Station

Grinder pump lift stations are a manufactured package unit and can be fully automated.

(1) Pump capacity

Maximum capacity is 15 gpm (identical to maximum hourly water usage minus sprinkler demands).

(2) Pressure requirements

(a) The maximum pipe length from the westbound side rest room to the package plant on the eastbound side is approximately 0.5 mile (2,640 ft).

(b) The treatment plant pipe inlet is about 15 ft above the pumps.

(c) The discharge pipe is 1-1/2 in. in diameter. The package unit includes a gate and a check valve on the discharge side of each grinder pump.

(d) The Darcy-Weisbach friction factor is equal to 0.02 and K values for the check and gate valves are 0.15 and 0.2, respectively. Assume there are two elbows with K values of 0.25.

(e) The pump head required (for $Q = 15$ gpm, $V = 2.72$ fps) is:

Pump Head = Fricton head loss + Elevation head + Minor losses

$$= fLV^2/(D2g) + 15' + 2K_{check}V^2/2g + 2K_{gate}V^2/2g + 2K_{el}V^2/2g$$

$$= 0.02(2{,}640)(2.72)^2 / [(.104)(64.4)] + 10' + (2.72)^2[0.15 + .2 + .25]/$$

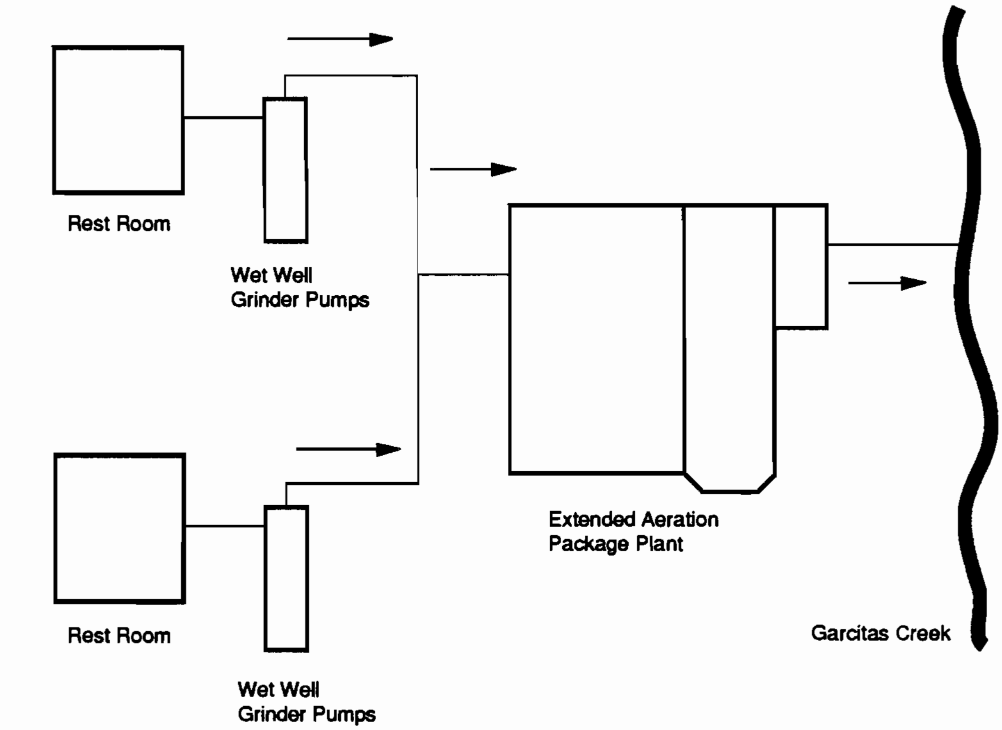


Fig 3.2. Rest area wastewater system flowsheet for package plant system.

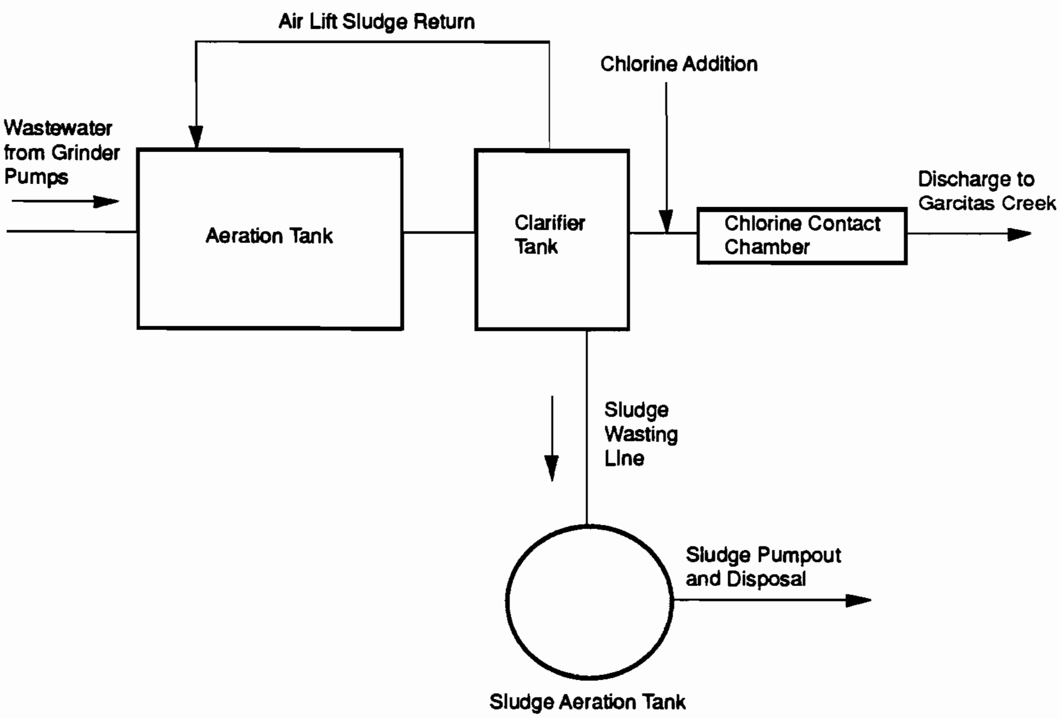


Fig 3.3. Extended aeration package plant flowsheet.

$$(64.4)$$
$$= 121' + 15' + \sim 0$$
$$= 80'$$

(f) A typical ready-to-install dual grinder pump lift station with a maximum storage volume of ~300 gallons will work. This includes two 2-HP, 3450 RPM pumps. The grinder pumps usually alternate pumping duties and will typically pump 8 to 10 gpm at a discharge pressure head of 90 ft. The pump operating conditions shoud be checked to ensure that the maximum total flow from both grinder lift stations to the treatment plant does not exceed 20 gpm.

3.5 EXTENDED AERATION PACKAGE PLANT DESIGN

The capacity of the package plant will be 10,000 gal/day, although the estimated average flow rate is 9,200 gal/day. The 10,000 gal/day figure occurs only 20 percent of the time, based on a frequency analysis using daily traffic data for 1986. The clarifier will be sized according to the peak flows shown (Figure 3.1) and an overflow rate of 600 gal/sq ft/day.

Equalization of flow is desirable for package plants, but the operation of equalization basins is difficult and requires constant oversight by a trained operator. The wastewater flows at rest areas in Texas are unknown and the goal of low operating requirements dictates that peak flows must be handled by sizing the clarifier for the peak hourly flow. Equalization volumes necessary for the average day and peak day are ~1,500 and 3,000 gallons [Figure 2.7, page 13, of Reference 4]. The design of the plant follows.

3.5.1 Design Criteria

(1) Texas Department of Water Resources and Texas Department of Health joint codes, "Design Criteria For Sewerage Systems," require

(a) Minimum air requirements, SCFM/lb BOD-d = 1.4.

(b) Clarifier maximum surface loadings

1) Peak flow = 800 gal/sq ft/day.

2) Design flow = 300 gal/sq ft/day.

(2) Aeration tank mixing requirements = 30 SCFM/10^3 cu ft aeration tank volume.

(3) MLSS = 4,000 mg/l

(4) MLVSS = 2,800 mg/l, based on MLVSS/MLSS = 0.7

(5) Solids concentration in recycle line (Sludge Density Index, SDI) = 10,000 mg/l.

(6) Sludge volume Index (SVI) = 1/SDI x 10^6 = 100

(7) Recycle ratio, Q_r/Q = MLSS / [SDI - MLSS] = .67

(8) Wasting will be accomplished via a valve at the bottom of the clarifier.

3.5.2 Calculate Hydraulic Detention Time

(1) Average flowrate = 9,200 gal/d

(2) Aeration tank volume = 10,000 gal.

(3) Hydraulic detention time, ø = 10,000/9,200 = 1.09 days

3.5.3 BOD loadings

(1) Average loading = (9,200 gal/d)(3.785 L gal)(2.2 lb/kg)(.0002 kg/d BOD)
= 15.3 lb/d

(2) Peak loadings = (16,000)(8.34)(.0002) = 26.7 lb/d

3.5.4 Air Requirements

Since the detention time in the aeration tank is sufficient for nitrification, oxygen requirements will include both BOD demands and demands for nitrification. The air supplied must ensure mixing in the aeration tank as well as biological oxidation of the organic matter and ammonia. The aeration tank may experience a buildup of biomass until wasting occurs; thus oxygen requirements are based on conversion of BOD and nitrogen demands without subtracting out biomass wasted.

(1) Oxygen requirements

$$O_2\,(\text{lb/d}) = [Q\,(S_o - S)\,8.34] / f + 4.57\,Q\,(N_o - N)8.34$$

where

Q = Flowrate, million gal per day
S_o = Influent soluble BOD_5, mg/l
S = Effluent soluble BOD_5, mg/l
N_o = Influent TKN, mg/l
N = Effluent TKN, mg/l
8.34 = Conversion factor (3.785 l/gal x 2.2 lb/kg)
4.57 = Conversion factor to ex press TKN in terms of oxygen equivalents
f = Factor to convert BOD_5 to ultimate BOD = 0.68

Using the peak flow rate expected at the rest area (16,000 gal/day) and assuming 90 percent conversion of BOD and a final TKN of 1 mg/l the oxygen requirements are

$$O_2\,(\text{lb/d}) = [0.016\,(200 - 10)\,8.34] / 0.68 + 4.57(0.016)(39)(8.34)$$
$$= \sim 50 \text{ lb/d}$$

(2) Calculate air requirements for treatment.

Assume air is composed of 23.2 percent oxygen by weight, the transfer efficency of equipment is 6 percent, and the density of air is .075 lb/cu ft;

Air (SCFM) = [50 lb/d] / [(.075 lb/cu ft)(.232)(.06)(1441 min/d)]

= 33 SCFM

(a) Air per unit volume = [47,890 cu ft/d] / [21,390 cu ft/d]

= 22 cu ft/cu ft

(b) Air per lb BOD/d = 33 SCFM/26.7 lbs d

= 1.23

(c) However, Texas state codes require a SCFM/ lb BOD per day of 1.4. Therefore, air requirements should be approximately 26.7 x 1.4 = 38 SCFM to meet state codes.

(3) Air requirements for mixing.

The aeration tank volume is 1,337 cu ft.

(a) Air for mixing = 30 x 1,337/1000 = ~40 SCFM

(b) Given the air requirements, seven orifices providing 6 SCFM each can be spaced on 2-ft centers (1.5-ft spacing from aeration tank walls) if the basin length is 15 ft., basin width is 8 ft, and the basin depth is 11 ft. The blower pressure required for 1/4-in. diameter orifices is about 5 psi.

(c) Total minimum air requirements = 42 + 37.5 (sludge handling) = 80 SCFM

3.5.5 Clarifier Design

The surface area of the clarifier is based on the peak hourly flow of 1350 gal/hr.

(1) Clarifier surface area

Clarifier surface area = (1350 gal /hr) / (25 gal /sq ft /d)

= 54 sq ft

Design for a peak flow equal to four times the average flow and an overflow rate of 800 gal/sq ft results in a surface area of ~75 sq ft for 10,000 gal/d flow. This design has been used at many rest areas in Texas so that clarifiers with this surface area size are readily available. Use a 8-ft x 9-ft clarifier.

(2) Clarifier depth

The design depth must be sufficient to allow sludge thickening and sludge storage. Sludge thickening will require a depth of about 3.5 ft, sludge storage for peak flow will require about 1 to 3 ft of depth for an aeration tank MLSS of 4,000 mg/l and a thickened sludge concentration of 10,000 mg/l. Add in 2 ft to ensure a steep bottom slope and 1.5 ft. to accommodate peak flows;

Minimum total liquid depth = 3.5 + 3 + 2 + 1.5 (freeboard)

= ~ 10 ft

In general clarifiers are commonly 2 ft. deeper than the aeration tank so the depth should be 13 ft.

3.5.6 Sludge Handling

Air lift pumps will be used for sludge return from the clarifier to the aeration tank and sludge storage facilities will be provided to make sludge hauling trips more economical.

(1) Air lift pump

Peak recycle flowrate = 1,350 gal/hr = 22.5 gpm

Inlet submergence = 11.5 ft.

Lift height above surface = 5 ft.

An air lift pump with a 0.5-inch-diameter pipe which can supply 10 SCFM is sufficient. Air for the scum skimmers will require about 7 SCFM each, and, therefore, 15 SCFM is required for skimmers.

(2) Sludge volume wasting

Two kilograms (4.4 lb. of solids will be produced in the aeration tank per day. The wasting to the aerobic sludge holding tank is to be done once per day. The thickened sludge concentration is 10,000 mg/l (1 percent solids) and the specific gravity of the sludge is approximately 1.01.

The volume of sludge to waste per day can be calculated by

$$V_s = M / [p \times S_{sl} \times P_s]$$

where

V_s = volume of sludge, cu m.

M = mass of dry solids, kg

p = density of water, 1,000 kg/cu m

S_{sl} = specific gravity of sludge

P_s = percent solids expressed as a decimal

for $M = 2$ kg, $S_{sl} = 1.01$, and $P_s = .01$

$$V_s = 2/[1000 \times 1.01 \times .01]$$

$$= 0.19 \text{ cu m}$$

$$= 0.19 \text{ cu m} \times 264 \text{ gal/cu m} = 52.3 \text{ gal/d}$$

The size of the sludge aeration tank is dependent on how many days between pumpouts. For 30-day sludge storage, the tank volume is equal to 1,570 gallons (209 cu ft).

(4) Air will be required to oxidize the volatile portion of the sludge. Assume the sludge is 80 percent volatile and that 40 percent of the solids can be oxidized completely. Oxygen requirements will be about 2.3 lb O_2

per lb cells oxidized.

O_2 needed = 4.4 x 0.8 x 0.4 x 2.3 = 3.2 lb/d

Air needed = (3.2 lb/d) / [(.075 lb / ft^3 x.232 x 0.06 x 1441)
= 2.12 SCFM

For mixing, 60 SCFM/1000 cu ft is required; thus, for an aeration sludge tank volume of 209 cu ft, 12.5 SCFM is needed.

(5) Total air for sludge handling = 37.5 SCFM.

3.5.7 Chlorine Contact Chamber Design

(1) Tank size

The contact time required is 30 minutes at peak flow.

Q_{peak} = 1350 gal/hr = 22.5 gal/min

Tank Volume = 22.5 gal/min x 30 min = 675 gallons

Dimensions = 8-ft. long x 2.5-ft. wide x 4.5-ft. deep

(2) Chlorine dose

Assume the chlorine demand is ~ 8 mg/l

Chlorine demand = (8 mg/l)(8.34)(.0092 MGD) = .61 lb/d.

As in the water system bleach will be used. The amount of bleach required is 1.4 gal/day (see water design section). Monthly, about 42 gallons will be required.

3.6 SEQUENTIAL BATCH REACTORS

Sequential batch reactors (SBR) offer another treatment alternative. Proprietory systems are available that can treat the flows expected at the rest area. A typical SBR operation is shown schematically in Figure 3.1b on page 59 of CTR 442-3.

Aeration cycles are typically anywhere from 2 to 6 hrs. with a 1 to 4-hour clarification cycle before discharge. Several tanks can be used so that the different tanks can be in different operational modes at the same time. For example, one tank could be receiving and aerating incoming wastewater for several hours while a second tank is in the clarification stage. After the clarification stage is over in the second tank, discharge is initiated and then the second tank acts as the aeration tank while the first starts a clarification stage. After the clarification stage is over in the second tank, discharge is initiated and then the second tank acts as the aeration tank while the first starts a clarification stage. Two tanks ,each with a detention time of 12 hours, could be set for a 6-hr aeration cycle, a 2-hr clarification cycle, and a 4-hr discharge and rest period. Float indicators could be used to initiate and terminate cycles and/or act as checks on a timed system.

An alternative SBR system which utilizes timed pump-

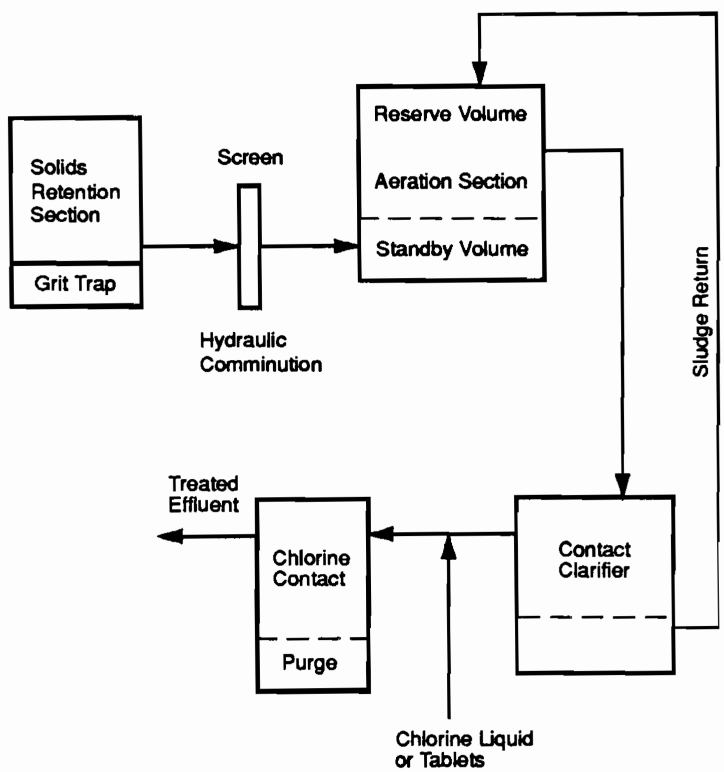


Fig 3.4. Sequential batch reactor treatment system. Courtesy of Cromaglass Corporation, P. O. Box 3215, Williamsport, PA 17701.

ing for all operations is shown in Fig. 3.4; a typical submersible pump type system for a 10,000 gal/day flow is shown in Fig. 3.5. Submersible aspirator pumps supply air via a venturi pipe, which extrains air from an air intake pipe opening, and, therefore, no compressors are required. Comminution is provided in a solids retention section which precedes the aeration tank; the break-up of solids is accomplished by air-induced turbulence in this section via the closest submersible pump in the aeration tank. The solids retention section requires periodic solids removal as large solids accumulate Submersible pumps also provide sludge return from the clarified basin to the aeration basin.

The SBR unit shown in Figures 3.4 and 3.5 can be located close to the rest rooms because (a) odors are minimized because oxygen levels in the aeration tank are maintained at 4 to 6 mg/l and the system solids retention section receives air for comminution, and (b) the unit is quiet since compressors are not used in the system. The system can be set to change batch cycles for peak flows and has standby capacity to help handle peak flows. The SBR operational requirements are not well established at present. Goronsky estimates 5 hrs/wk for the system [Figure 3.1b, page 59 of Reference 2]. For the pump driven system, operational requirements consist of raking the solids retention section (~15 min/week, maximum), checking the pump used for comminution for clogs (this should not be a big problem since the solids retention section is separated from the aeration section by a screen), replacement of chlorine (chlorination tank and equipment are not provided with the SBR unit except at extra cost), and sludge wasting (every six months as a conservative estimate). Pump replacement is the biggest maintenance item and takes about 20 minutes to accomplish. Pumps are likely to last three to five years before requiring replacement. Additional SBR units can be added to accommodate increases in future flows.

3.7 POND SYSTEM ALTERNATIVE

A pond system could be used if sufficient land were available. The following design calculations can serve as a guide in estimating land requirements for pond systems.

3.7.1 Evaporative Pond System

The monthly precipitation and evaporation data for climatalogical stations nearest to Victoria are presented in Table 3.1. The monthly two-way traffic flowrates also are shown.

(1) Evaporation rate excess
E (excess) = 78.26 - 35.34 = 42.92 in/yr,
= 42.92 in/yr x 1 ft/12 in. x 43,650 sq ft/acre
= 155,800 cu ft/acre/yr

(2) Yearly wastewater flowrate
Q (from Table 4) = 3,336,533 gal/yr x .1337 cu ft/gal
= ~ 446,100 cu ft/yr

(3) Compute surface area
SA = 446,100 cu ft/acre/yr ÷ 155,000 cu ft/yr
= 2.87 acres

(4) Volume of ponds
Vol. = 2.87 acres x 43,650 sq ft/acre x 4 ft = 501,474 sq ft

The method above is presented in Appendix H also.. Storm surge capacity (i.e., heavy rains and low evaporation occuring simultaneously) has not been included. Land area for dikes and for a buffer zone of at least 1.5 acres is also necessary. Multiple cells should be used to add flexibility to the system.

(5) The worst case surge capacity required for one day is based on the scenario of the period of record high rainfall for November (2.12 in.) occuring on the peak

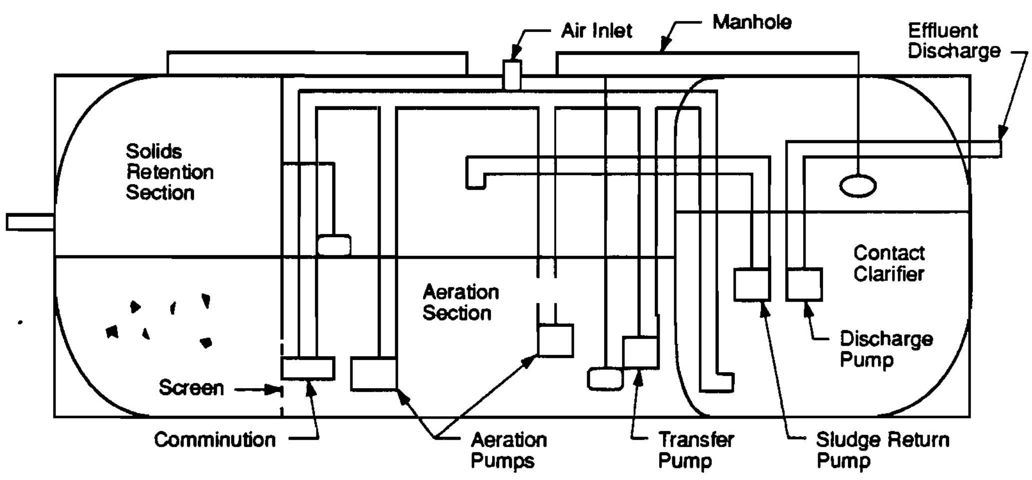


Fig 3.5. Sketch of Cromaglass SBR unit. Courtesy of Cromaglass Corporation, P. O. Box 3215, Williamsport, PA 17701.

traffic day in 1986 when the ponds are full.

Q_{rest} = 23,670 veh/d x 0.675 = 16,000 gal/d (2,139 cu ft/d)

Q_{rain} = 2.12 in./d x 1 ft/12in x 446,100 sq ft = 78,811 cu ft

Q_{tot} = ~81,000 cu ft = a .18 ft. rise in a one cell pond

These calculations are for one day. The cumulative effects of consecutive days of rainfall and little evaporation could increase the depth by at least one foot; therefore detailed analysis to evaluate the surges should be completed.

3.7.2 Overflow Pond System

Design of overflow ponds is more of an art than a science. The hydraulic character of ponds varies widely depending on wastewater flows and environmental conditions. The hydraulic characteristics of an overflow pond affect reaction rates and treatment capability. The oxygen production in a pond is dependent on sunlight and algae production of oxygen. The removal of BOD in a pond is dependent on the hydraulics of the pond and on the oxygen resources of the pond.

An empirical equation formulated by Oswald can be used to estimate pond oxygen production. Oswald's equation for oxygen production from sunlight is

$$Y = 0.25\,FS$$

where

Y = Oxygen production, lbs/acre-day

F = Oxygenation factor; this factor depends on BOD removal.

S = Solar radiation, cal/cm²-day

$S = S_{min} + p(S_{max} - S_{min})$

S_{min} = Minimum month solar radiation, cal/cm²-day

S_{max} = Maximum month solar radiation

p = Actual hours of sunlight divided by total possible hours of sunlight.

The Y value must equal the loading, if the entire depth is aerobic:

$$[BOD_{ul} \times Q] / A = 0.25\,FS$$

or

$$d/t = [0.25\,FS] / [BOD_{ul} \times \text{conversion factor}]$$

where

BOD_{ul} = Ultimate BOD = $BOD_5/.68$, mg/l

d = oxygenated pond depth, in.

t = detention time of pond, days

conversion factor = 0.226 for d in inches and t in days

F & S = as above

For Texas,

S_{min} = 76 cal/cm²-day for December

S_{max} = 184 cal/cm²-day (this is actually the minimum monthly solar radiation value associated

Fig 3.1. Peak daily hydrograph for two-way traffic (November 30, 1986)

Month	Days in Month	Tot. Precip. [1] (in/mo.)	Tot. Pan Evap. [2] (in/mo.)	Q in [3] (gal/mo.)
Jan.	31	2.17	2.81	260,035
Feb.	28	2.13	3.64	236,949
March	31	2.00	5.00	302,429
April	30	2.49	6.26	266,510
May	31	4.07	8.42	285,950
June	30	3.54	9.57	279,997
July	31	3.25	10.86	305,610
Aug.	31	2.91	10.25	303,664
Sept.	30	4.41	7.56	256,061
Oct.	31	3.41	6.22	266,857
Nov.	30	2.46	4.25	277,000
Dec.	31	2.50	3.42	265,482
Annual		35.34	78.26	3,336,533

[1] Weather of U>S> Cities, Gale Research Co., Detroit MI 48226 1981.

[2] Agro Climate Atlas of Texas, Agriculture Publication, Reed McKonal Bld., Texas A&M University, College Station, TX 77843, Dec. 1983.

[3] Values calculated using traffic data from traffic counter S116.

with the maximum month)

p = 8 / 14 = 0.57 (estimate for annual average).

F = 1.6 for 90 percent removal (design for 90% removal total system)

d = 5 feet (Texas Codes for facultative ponds)

= 60 in.

so

t = [60 x (200/.68) x .226] + [.25 x 1.6 x 139.5]

t = 71.5 days

The surface area is

SA= [Q x t]/d = [9,200 gal/d x 71.5 d x 1 cu ft/7.48 gal] ÷ 5 ft

= ~18,000 sq ft = 0.40 acre

Now check the loading rate:

L = QS_o/A = [.0092 x 200 x 8.34]/ .40 acres

= 38.36 lbs BOD_5/acre-d

The actual oxygenated depth in the winter will be about 3.2 ft. Subsequent pond areas can be calculated in a similiar manner (for 5-ft depths and 90 percent removal) yielding areas of 0.2 and .02 acres for a second and third pond. The total area is 0.62 acres by the Oswald method.

This area is minimum because the pond hydraulics were not taken into account and the winter oxygenated depth of the ponds is lower than that assumed for d to calculate the surface area. In practice at least 0.9 acre plus a buffer zone should be provided for a 3-cell system (Table 3.7, p. 53, CTR 442-3).

A second approach to pond design is to use Gloyna's empirical relationship:

$$V_{pond} = 3.5 \times 10^{-5} Q S_o \beta^{35-T} f f'$$

where

V_{pond} = Volume of pond in cu m.

Q = Wastewater flow rate in liters/d.

S_o = Ultimate total BOD (=BOD_5/.68), mg/l

ß = Temperature factor = 1.085

f = Algal toxicity factor

f' = Sulfide inhibition term

T = Ambient air temperature during coldest month, in degrees Celsius

The rest area wastewater pond, based on the following conditions,

Q = 10,000 gal/d x 3.785 liters/gal = 37,850 l/d

S_o = 200/.68 = 294 mg/l

f,f' = 1

T = 12°C

will have a volume of

V_{pond} = $3.5 \times 10^{-5}(37,850)(294)(1.085)^{23}$

= 2,5430 cu m = ~89,800 cu ft

This value is close to that obtained from the Oswald method (9,200 gal/d x 71.5 days x 1 cu ft/gal = ~88,000 cu ft) and also results in 0.40 acre for a 5-ft. deep pond with a 90 percent treatment efficiency. Additional smaller ponds are required to reach a final BOD of 10 mg/l.

3.8 EVAPOTRANSPIRATION BED SYSTEM ALTERNATIVE

The design of the evapotranspiration bed is based on the method shown in Appendix G in Reference 2. The monthly precipitation and evaporation data presented in Table 3.1 are used in design. The waste treatment system will require settleable solids removal before application of the supernatant to the beds. A septic tank can be used. The area where the beds are located must be cleared of trees. Runoff diversion from the site also is needed to minimize infiltration.

Tables 3.2 and 3.3 show effluent level calculations for beds with surface areas of 30,000 and 40,000 ft^2 (0.69 and 0.9 acre, respectively) for the eastbound traffic lane (EBL). The monthly evaporation values given in Table 3.3. were multiplied by 0.65 to estimate a minimum monthly evaporation to use in design calculations. The monthly inflows into the units are average monthly values based on traffic data for the EBL.

A comparison of the effluent levels for total bed surface areas of 0.69 and 0.9 acre, respectively, indicates that the effluent levels for the 0.9-acre system are relatively steady. Therefore the use of the larger area is the best choice. This method does not take into account plants transpiration so that effluent levels may be lower than calculated; thus there is some untapped storage capacity in the system but it cannot be quantified until a system is installed.

The septic tank system should be sized for the peak daily one-lane flow. A 10,000-gallon septic tank covers the peak flow for one directional traffic according to the peak traffic for the EBL. The effluent enters at the bottom of the bed (30 in. below the surface), and a gravity system can be used as long as the distance from the septic tank to the beds is not more than about 300 ft for 4-in. pipe (this provision means sewer pipe velocity limits can be met). The analysis presented relies on equal distribution of effluent to the beds.

Ten evapotranspiration beds of ~65 ft x 65 ft could be used to provide a surface area of 40,000 sq ft, and ten beds with dimensions of 55 ft. x 55 ft. could be used for a surface area of 30,000 sq ft. The depths of the beds are 2.5 feet so that the total volume for the larger beds is 105,625 cu ft (3,912 cu yd) and 76,625 sq ft (2,800 cu yd). One-half of the working depth should be filled with sand and one-quarter with gravel and soil, respectively. For the larger bed, approximately 650 feet of slotted pipe is required to distribute septic tank effluent to the beds; the smaller bed option will require approximately 550 ft of slotted pipe. An evapotranspiration bed system is required for each rest area because all calculations above are based on the EBL traffic.

The permeability results at the Victoria site indicate that lining the beds will not be necessary. This conclusion must be reviewed after review of the most currrent standards of the Texas Department of Health. The total area required, assuming two rows of five beds with 10 feet spread between beds in all directions, is approximately one acre per side of the highway. For the larger beds a total area per side of 1.4 acres is required to enclose these beds. A fence for the evapotranspiration system also is required.

3.9 REST AREA WASTEWATER SYSTEM COSTS

The capital costs of the wastewater system are estimated based on information from manufacturers and Austin contractors and companies. The operating costs are estimated in terms of man-hours and kW-hours rather than in dollars.

3.9.1 Capital Costs

	Cost in Dollars
(1) Package treatment plant, 10,000 gallon, delivered and installed, with hypochlorinator tank & associated equipment	50,000
(2) Grinder pump lift stations (package units), two 2-HP pumps, 36-in. x 72-in. deep fiberglass sump, 2 gate and check valves, and control panel. Installation not included. Two stations at 3,450 each.	7,000
(3) 1.5-in. discharge piping from grinder pump stations, 4,000 ft	
(a) PVC, $0.35/ft	1,400
(b) Steel, $1.42/ft	2,000
(4) Installations costs for grinder pumps, piping to treatment plant, and discharge piping, lump sum	3,000
(5) 4-in. PVC discharge piping, 500 ft at $1.30/ft	650
(6) Fence, 50 ft. x 25 ft., chain link with wooden inserts, eight-ft. high, $9.00/ft	1,350
(7) Total	
(a) Steel 1.5-in. discharge pipe	64,000
(b) PVC 1.5-in. discharge pipe	63,400

3.9.2 Operating Costs

(1) Supply Costs	$/yr
(a) $0.75/gal, 510 gal/yr.	383
(b) Effluent analysis costs, once/month samples for BOD, TSS, NH3, and PO4, $80/sample	960
(c) Total per year	1,343
(2) Man Power Requirement	Man-hr/yr
(a) Hypochlorinator mixing	52
(b) Effluent sampling and aeration tank sampling (2 hrs/week)	104
(c) Routine operational checks by rest area attendant, 1 man-hr/d [Chapter 3, p 66-67 of Ref 2]	365

TABLE 3.2. EFFLUENT BED LEVEL OVER TIME FOR BED SURFACE AREA OF 30,000 SQ FT

Month	Days in Month	Daily Pan Evap. gal/d-ft^2	ET Rate gal/d-ft^2	Q et gal/d-ft	Q in gal/d-ft	Δ Sys. gal	EL. Effl. Level Below Grade in.
Jan.	31	0.04	0.022	20,460	130,593	110,133	18.22
Feb.	28	0.05	0.055	46,200	117,974	71,774	22.32
March	31	0.07	0.028	26,040	151,936	125,896	16.53
April	30	0.08	0.055	49,500	134,136	84,636	20.95
May	31	0.11	0.028	26,040	142,583	116,543	17.53
June	30	0.13	0.062	55,800	139,644	83,844	21.03
July	31	0.14	0.028	26,040	152,313	126,273	16.49
Aug.	31	0.13	0.060	55,800	148,526	92,726	20.08
Sept.	30	0.10	0.030	27,000	129,276	102,276	19.06
Oct.	31	0.08	0.040	37,200	131,200	94,000	19.95
Nov.	30	0.06	0.040	36,000	136,789	100,789	19.2
Dec.	31	0.04	0.045	41,850	143,483	101,633	19.13

(d) Operational checks by trained operator, 4 hrs/month — 48
(e) Total per year — 570

(3) Power Requirements — kW-hr/yr
(a) Grinder lift stations, 2 HP for 10 hrs/d — 9,600
(b) Treatment plant blowers, 3 HP for 16 hrs/d — 23,500
(c) Total per year — 33,100

3.10 ANNUAL SYSTEM COSTS

3.10.1 Assumptions

(1) System life is 20 years
(2) Fixed interest rate is 75 percent (*Wall Street Journal* Municipal Bond Rates, 1987).

3.10.2 Annual Costs

Annual costs will be calculated using

$$AC = Ci + O\&M + i(C-S)/[(1+i)n -1] + CRi/[(1+i)x - 1]$$

from the tabulations previously made,

where

C = Capital costs from water and wastewater systems
O&M = Operation and maintenance from water or wastewater systems
S = Assume salvage value equals zero for conservative estimate.
CR = Cost of major repairs after x years = number of years until major repairs needed
i = current interest rate for municipal bonds

(1) Example for extended aeration package plant;
(a) Capital costs = $64,000
(b) Assume that tank capacity is adequate, but pumps/blowers may need to be replaced or have major overhauls after 10 years of use. If all pumps/blowers need replacement then cost of major repair is about $5,000, CR = $5,000 at x = 10.
(c) i = .075 (current interest rate on municipal bonds)
(d) O&M costs are based on labor costs of $10/ man-hr and power costs of $0.07/kW-hr.

so AC = 64,000 x (.075) + 12,400 + [64,000 x (.075)] / [1.075^{20} - 1] + [5,000 x .075] / [1.075^{10} - 1]
= 4,480 + 12,400 + 1,478 + 330
= ~$18,700

3.11 POND SYSTEM WASTEWATER SYSTEM COSTS

3.11.1 Capital Cost

Cost in Dollars

(1) Land Acquisiton, $ 3,000/acre, vacant land in Victoria County (a) evaporative pond (3.5 acres total area) — 10,500
(b) overflow pond (1.5 acres total area) — 4,500

(2) Excavation costs, $1.5/cu. yard
(a) 8,067 cu. yds for one acre pond(s), pond depths are 5 ft. — 12,100
(b) 19,360 cu. yds for 4-ft. deep evaporative pond(s). — 29,040

(3) Grinder pump stations (same as in treatment plant case) + installation — 10,000

(4) Synthetic liner, assume $0.3/ft2 for PVC
(a) Overflow pond is 500 ft long and 85 ft wide,

TABLE 3.3. EFFLUENT BED LEVEL OVER TIME FOR BED SURFACE AREA OF 40,000 SQ FT

Month	Days in Month	Daily Pan Evap. gal/d-ft^2	ET Rate gal/d-ft^2	Q et gal/d-ft	Q in gal/d-ft	Δ Sys. gal	EL. Effl. Level Below Grade in
Jan.	31	0.04	0.022	27,280	130,593	103,313	21.71
Feb.	28	0.05	0.030	33,600	117,974	84,374	23.23
March	31	0.07	0.022	27,280	151,936	124,656	20.00
April	30	0.08	0.018	21,600	134,136	112,536	20.97
May	31	0.11	0.025	31,000	142,583	111,583	21.05
June	30	0.13	0.028	33,600	139,644	106,044	21.49
July	31	0.14	0.025	31,000	153,313	121,313	20.27
Aug.	31	0.13	0.033	40,920	148,526	107,606	21.37
Sept.	30	0.10	0.028	33,600	129,276	95,676	22.32
Oct.	31	0.08	0.018	22,320	131,200	108,880	21.27
Nov.	30	0.06	0.025	30,000	136,789	106,789	21.43
Dec.	31	0.04	0.030	37,200	143,483	106,283	21.47

total ft^2 needed ~50,000 ft^2 15,000

(b) Evaporative pond, 3 acre, ~135,000 ft^2 40,500

(5) Fence
- (a) Evaporative, 1,500 ft., 8 ft high, at $6.50/ft 9,750
- (b) Overflow, 1,170 ft 7,600

(6) TOTAL CAPITAL COSTS
- (a) Evaporative pond
  - 1) With liner 96,540
  - 2) No liner 56,040
- (b) Overflow pond
  - 1) With liner 48,100
  - 2) No liner 33,100

3.11.2 Operating Costs

(1) Man Power Requirements	Man-hr/yr
(a) For all maintenance, 2 man-hr/wk	104
(see CTR 442-3, Chap. 3, page 58 for maintenance needs)	.
(b) Pond effluent sampling, once/month,	12
(c) Total man-hr/yr	116

(2) Effluent sampling analysis, once/month, $/yr 960

(3) Power Requirements	kW-hr/yr
Grinder lift station	9,600

3.12 EVAPOTRANSPIRATION BED WASTEWATER SYSTEM COSTS

3.12.1 Capital Costs

Cost in Dollars

(1) Land Acquisition , $3,000/acre, vacant land Victoria County.
- (a) Smaller beds, 1 acre per side 3,000
- (b) Large beds, 1.4 acres per side 4,200

(2) Bed Excavation, $1.5/$yd^3$ (off-site hauling not included)
- (a) Smaller beds, 2,800 yd^3 per side 8,400
- (b) Larger beds, 3,912 yd^3 per side 11,740

(3) Fill costs (one-quarter gravel, one-quarter soil, one-half sand), gravel $10/$yd^3$, sand $1.00/$yd^3$, soil will be obtained from excavation.
- (a) Smaller beds 16,800
- (b) Larger beds 23,460

(4) Liner costs, $0.30/ ft^2,
- (a) Smaller beds, 3,575 ft^2 each bed, 10 beds, 10,725
- (b) Larger beds, 4,875 ft^2 each bed, 10 beds 14,625

(5) Septic tank, 10,000 gallon capacity, one per side, $1/gal including installation 10,000

(6) Distribution piping, $3.00/ft for 4 inch slotted PVC,
- (a) Smaller beds, ~550 ft per side 1,650
- (b) Larger beds, ~650 ft per side 1,950

(7) Installation costs
Assume $20/hr/worker, and it requires 10 people a week to install each side 8,000

(8) TOTAL CAPITAL COSTS
- (a) Smaller beds, each side ~61,600
- (b) Larger beds, each side ~78,200

3.12.2 Operating Costs

(1) Man power requirements
- mowing of grass,checking septic tank, and effluent levels in beds, man-hr/yr 52

(2) Septic tank pumpout, once per six months, $/yr 300

3.13 SEQUENTIAL BATCH REACTOR SYSTEM COSTS

The costs listed below are based on manufacturer's estimates. The manufacturer recommends a chlorination system using solid sticks of calcium hypochlorite. If the stick-type chlorination feed system is not allowed via state codes a hypochlorinator system will have to be used, with the associated capital and labor costs outlined for the package plant system.

3.13.1 Capital Costs

	Cost in dollars
(1) SBR, 10,000 gal/day model, installed	37,000
(2) Chlorine contact tank, 3,000 gal.	3,000
(3) Discharge piping, 4PVC at $1.30/ft, 500 ft	650
(4) Shipping cost	1,000
(5) Installation of chlorine tank and discharge piping	1,000
(6) Grinder pump lift stations (package units)	7,000
(7) Installation costs for grinder pumps, piping to treatment plants, and discharge piping, lump sum	3,000
(8) 1.5-in. discharge piping from grinder pump stations, 4,000 ft	
(a) PVC, $0.35/ft	1,400
(b) Steel, $1.42/ft	2,000

(9) TOTAL CAPITAL COSTS ~56,000

3.13.2 Operating Costs

(1) Manpower requirements — Man-hr/yr
   (a) Raking of solids retention section, 15 min/wk. — 12
   (b) Effluent testing, 1 man-hr/sample — 52
   (c) Routine operation checks, 45 min/wk. — 39
   (d) Total man-hr/yr — 103

(2) Sampling/supply costs — $/yr
   (a) Sample analysis, once/month, $80/sample — 960
   (b) Chlorine stick replacement, 1 stick/d, $4/stick — 1,460
   (c) Total — 2,420

(4) Sludge wasting, pumpout 400-500 gallon per every six months (maximum) — 300
(5) Miscellaneous requirements — 1,200
(6) Power requirements — kW/yr
   52 kW-hrs/day, (mfg est.) — 19,000

3.14 RECOMMENDATIONS

The choice of an appropriate wastewater system for the Victoria rest area depends not only on costs shown in Table 3.4 but on management goals and site constraints. Based on annual costs listed in Table 3.4, an overflow pond system should be chosen. If minimization of manpower and power requirements is the prime goal then ET beds are the best choice.

The land available at the site is inadequate for the pond or ET bed systems; therefore, a treatment plant option is the best choice despite higher annual costs. If more land can be made available then the pond systems offer the least cost alternative.

A comparison of the package plant and SBR system cost favor the SBR plant in all categories. However, the cost figures listed for the SBR system should be viewed with caution. The SBR operation and maintenance requirements are not documented at present and figures reported by manufacurers may be low. It is likely that the operation and maintenance costs for the SBR plant will be closer to the package plant costs than those shown in Table 3.4.

The choice between the two systems hinges on risk assessment. SBR systems have a higher risk associated with them in terms of operation and maintenance. Package plant risks are associated with their ability to meet performance requirements under fluctuating wastewater flows and varying wastewater characteristics. A risk assessment has not been performed in this report but it is believed that the SBR plant is the best choice because of its ability to accommodate fluctuating flows and wastewater characteristics and because its operation can be modified without difficulty.

An SBR system is recommended.

TABLE 3.4. COST BY CATEGORY FOR WASTEWATER SYSTEMS

System Type	Capitol $	Supply & Sampling ($/yr)	Manpower (man-hr/yr)	Power (kW-hrs/yr)	Annual Costs($)[1]
Package plant	64,000	1,343	570	33,100	15,650
Pond system					
Evaporative	96,540	960	104	9,600	11,500
Overflow	48,100	960	156	9,600	7,850
ET Beds (two sides)					
30,000 ft	123,200	600	52	0	13,400
40,000	156,400	600	52	0	16,850
SBR plant	56,000	2,843	103	19,000	11,000

[1] Costs of major repairs after ten years (CR) per system are as follows:

Extended Aeration Package Plant	Assume all blowers and pumps are replaced in ten years; cost of major repairs (Cr) is $5,000, x = 10 yrs.
Pond Systems	Assume sludge is removed, grinder pumps replaced CR = $5,000 at x = 10.
ET Beds	Assume liners in half the beds need replacement in ten years, CR = $8,000 (small beds) and CR = $10,000, for (large beds) at x = 10
Sequential Batch Systems	Assume replacement of all pumps after each 5 years, CR = 9x 250 = 2,250 at x = 5.

CHAPTER 4. REQUIREMENTS FOR HEATING, COOLING, AND LIGHTING

4.1 BASIS FOR ANALYSIS

The structures evaluated have the construction, materials, and orientation as specified in Chapter 5. The orientation is assumed to be such that the "cross" design of the building is NE/SE/SW/NW.

In summary, the exterior walls are exterior stucco on concrete masonry units, with 1-in. closed cell foam insulation and interior tile with an overall U factor of 0.127. The flat and sloped roof portions include 6 in. of figerglass insulation and have an overall U factor of 0.04. Glazing is double pane (U=0.56). Doors are 3 ft x 6 ft 8 in. and have steel frames and facings with polystyrene cores and thermal breaks (U=0.47).

The infiltration is assumed to occur primarily due to door openings and is assumed to be two air changes per hour for winter (heating) and one air change for summer (cooling), i.e., 120 and 60 cfm for winter and summer, respectively.

A major load for a restroom facility is due to the ventilation requirements, which are nominally selected to be 2 cfm/ft^2 and 1 cfm/ft^2 of floor area for the toilet and core areas, respectively, when occupancy is near the normal maximum. It should be noted that in the subsequent results, scenarios are considered where ventilation requirements may be reduced during periods of low occupancy to reduce the peak demands for heating and/or cooling

For Victoria, the winter design temperature is 28°F and the summer design conditions are 97°F dry bulb and 77°F wet bulb. The nominal winter (heating) season inside temperature is set at 70°F. The nominal summer (cooling) season inside conditions are set at 77°F and 50 percent relative humidity. Note that, to reduce peak electrical loads and equipment capacities, different scenarios for these inside conditions will be considered.

The HVAC equipment to be used is recommended to be electrically driven heat pump/air conditioning units with average winter heating and summer cooling COP's of 2.0 and 3.0, respectively.

People loads of one person per 128 sq ft at 245 and 205 BTU/hr/person for sensible and latent heat, respectively, were assumed. The interior lighting levels are designed for 2.0 W/m^2 and thus amount to 2.6kW per building.

The occupancy (visitor) distribution through the day was based on the actual survey data reported in Reference 4. This survey in particular points out that during the early morning peak winter heating period occupancy is about 25 percent, while during the late afternoon peak summer cooling period occupancy is near 100 percent.

4.2 HEATING AND COOLING LOAD CALCULATIONS

The heating and cooling load calculations were performed using Carrier's E20-II (1984 version) program run on an IBM-PC. The program was used primarily to assess the heating and cooling loads during peak demand periods for the "nominal" facility. In performing the computations, weather/solar data for Houston were used.

The "nominal" facility assumes the winter and summer design outdoor and indoor conditions, with the recommended ventilation rate per unit floor area and no ventilation heat recovery. The project peak load data for each of winter (heating) and summer (cooling) are shown as Case 1 in Table 4.1. The total (sensible) heating load is shown for the winter season, while the sensible, latent, and total cooling loads are indicated for the summer season. Also indicated are the design electrical demands for each of heating and cooling, where heating and cooling COP's of 2.0 and 3.0 are assumed. The design electrical demands are 27.4 and 21.0 kW respectively. A major impact on both of these loads is the nominal ventilation requirements

Other scenarios can be assumed for the heating season. The design heating condition occurs in the early morning, when the facility occupancy is about 25 percent, and during this period a reduced (25 percent of nominal) ventilation may be acceptable. Also, since there are few visitors and their stays in the facility are short, a 60°F rather than a 70°F inside temperature may be acceptable. The heating demands for a 60°F inside temperature and 25 percent of nominal ventilation are shown as Case 2 (a).

This, however, may not be the most restrictive case, since the occupancy survey data show that by 10 AM occupancy is up to near 100 percent, requiring full ventilation. Data show that during design periods the outdoor temperature at 10 AM is typically about 7°F above the 28°F design value. Case 2(b) indicates the heating load for full ventilation but with an outside design temperature elevated to 35°F and an inside temperature increased to 65°F because of the greater occupancy. As seen, the 19.5 kW demand for heating near 10 AM does, in fact, exceed the 12.2kW early morning demand when reduced ventilation is acceptable.

A final scenario, in addition, incorporates a 60 percent (sensible) heat recovery unit for the ventilation air. Cases 3(a) and 3(b) for early morning and 10 AM, respectively, show that electrical demands are now reduced to 9.6 and 12.2 kW, respectively, with the 10 AM condition controlling the electrical demand (design capacity) of the heat pump.

Similar scenarios can be considered for the cooling season. However, the peak cooling demand typically occurs during the late afternoon during periods of nominally 100 percent rated occupancy, so ventilation cannot be reduced. One possibility is to design for an 82°F (rather than 77°F) inside temperature, again the reasoning being that a person's stay in the facility will be only a few minutes and that he has come in from a hot outside condition. The lower part of Table 4.1 gives the electrical demand for cooling, assuming inside conditions of 82°F and 50 percent R.H.. Raising inside temperature influences sensible building envelope loads as well as infiltration and ventilation loads (both sensible and latent). The peak electrical cooling demand is seen to decrease to 17.4 kW (from 21 kW) for this scenario.

A final possibility (Case 3) is to incorporate a 60 percent sensible heat recovery unit for the ventilation air. This further decreases the electrical demand for cooling to 15.9 kW.

Thus, based on reduced ventilation requirements, accepting a higher or lower than ideal inside temperature and incorporating a heat recovery unit for the ventilated air, the peak heating and cooling season electrical demands are projected to be 12.2 kW and 15.9 kW, respectively. Therefore, an installed electrical requirement capacity for HVAC is set at approximately 18 kW.

4.3 EXTERIOR LIGHTING

Assuming a site of approximately 360 ft by 800 ft (288,000 ft^2) with an average illumination of 0.5 ft. candles (0.5 lumens/ft^2) and approximately 100 lumens/watt, the exterior lighting requirement is 1.5 kW (assume 2 kW).

4.4 OTHER ELECTRICAL DEMANDS

Other electrical demands are listed in Table 4.2 along with the HVAC requirements discussed above. A 20 percent excess capacity (7.5 kW) above the 37.5 kW nominal peak demand is included, for a recommended installed capacity of 45 kW per building.

4.5 SUMMARY OF ELECTRIC REQUIREMENTS

Assuming a site on either side of the roadway, the recommended electrical capacity is 96 kw for one building per site or 186 kW for two buildings per site. These capacities assume use of ventilation waste heat recovery, decreased ventilation during low occupancy winter peak demand periods and relaxation of indoor temperature during both summer and winter peak demand periods.

TABLE 4.1. HEATING/COOLING LOAD SUMMARY

Heating	Sensible (1,000) Btu/hr)	Latent (1,000) Btu/hr)	Total (1,000) Btu/hr)	Demand* (1,000) Btu/hr)
Nominal Facility	172	----	172	27.4
Early morning 25% ventilation T = 60°F; To = Td = 28°F	76.5	----	76.5	12.2
10 am 100% ventilation T 65°F; To = Td + 7 = 35°F	122.4	----	122.4	19.5
Early morning Plus 60% vent. heat recovery	60.3	----	60.3	9.6
10 am plus 60% vent. heat recovery	76.8	----	76.8	12.2
Cooling				
Nominal Facility	126.1	72.1	198.2	21.0
Relax inside temp. T = 82°F. (50% RH)	108.7	55.3	164.0	17.4
Plus 60% sensible heat recovery on vent. air	94.3	55.3	149.6	15.9

* Based on assumed COP's of 3.0 and 2.0 for the cooling and the heating modes of electrically driven units.

TABLE 4.2. SUMMARY OF ELECTRICAL LOADS

Per Building	**kW**
HVAC (based on cooling)	16.0
Water heater	5.0
Interior lighting	2.6
Outlets	3.6
Electrical hand dryer (2)	1.0
Fans	2.0
Air compressor	0.8
Water/wastewater	3.5
Vending machines	2.0
Signs	1.0
Excess Capacity (20 percent)	7.5
Total	45.0
Per Site: (one on each side of the road)	
Exterior lighting	2.0
Signs	1.0
Total	3.0

Therefore, for one building on each side of the road, the peak electrical load is about 96 kW, while for two buildings on each site, the peak electrical load is about 186 kW.

CHAPTER 5. SITE AND BUILDING DESIGN

5.1 DESIGN PROGRAM REQUIREMENTS

This section presents the design program requirements for the prototype facility. They are based on findings in the previous reports issued in this study [References 2, 4, 5 and 6]. These reports should be consulted for additional information concerning design recommendations.

5.1.1 A minimum clean, operational toilet facility shall be available for public use at all times.

Design solutions:

(1) At least two rest rooms for men and women will be provided so maintenance can be performed with one unit remaining open.
(2) All plumbing and services will be concealed in a central service area for protection and maintenance.
(3) All fixtures are off floor for ease of maintenance.
(4) High ceilings are provided for natural light, ventilation and aesthetics.

5.1.2 The facility shall be arranged and sited to discourage group gatherings, loitering and illicit sexual solicitations.

Design solutions:

(1) Adjacent stalls will not be used.
(2) Alternate extrance/exit doors will be provided.
(3) Mirrors will be placed away from lavatories.
(4) Facility will be well-lighted.
(5) There will be implied or actual staff presence at all times with radios to be operated continuously in central service area.
(6) The units will be small with two stalls maximum.

5.1.3 The facility shall be planned for maximum security for those using the site.

Design solutions:

(1) The facility will be well-lighted, using natural and artificial light.
(2) Entrances will be visible from parking areas.
(3) No low vegetation or visual barriors will be used near walks or doors.
(4) There will be alternate entrance/exit doors.
(5) The rest rooms will be small, with no concealed hiding places.
(6) Uniformed personnel should be present 24 hours each day, seven days a week.

5.1.4 The facilities shall be well-ventilated and the interior spaces air conditioned by heating and/or cooling to moderate temperature and humidity levels. (This does not, however, require residential comfort levels.)

5.2 ILLUSTRATIVE DESIGN

5.2.1 Site

Northeast of Victoria on U.S. Highway 59 in District 13, the site consists of two areas, one on the north side and one on the south side, of approximately ten acres each (Fig. 5.1). The sites are approximately 400 feet deep, which allows for a truck lane near the highway lanes and an automobile parking lane behind the comfort station, to separate large and small vehicle traffic. Many nice hardwood trees, including live oaks and elms, cover the site, allowing a canopy of green to cover the approach to the building from parking areas which will be cleared of low underbrush. One building will be designed on each rest area with two men's and two women's units per building, which are designed to serve 95 percent of the estimated maximum number of users. Use figures were based on a maximum annual traffic of 500,000 vehicles per year passing the site in each direction. Provisions have been made at each site for one additional building, allowing growth for up to 1,000,000 vehicles per year passing the sites in each direction. The design requirements for the number of fixtures in the rest rooms are discussed in Chapter 3.

Each rest area site has a shallow well to provide water. Sewer wastewater requirements are described in Chapter 2. Picnic tables and shelters, charcoal boxes, and waste receptacles are provided in the utility area and on the periphery of the site. Where possible, pedestrian traffic from parking areas should not be required to cross auto or truck lanes to reach the comfort station.

5.2.2 Prototype Building Design

The building plans, elevations and details, are shown in Figs. 5.1 to 5.6. A brief description of the building follows:

(1) Due to building size, a slab-on-grade foundation with integral grade beams is recommended for the preliminary design of the foundation. For final design, soil tests should be conducted to permit an engineering design to be made.
(2) A steel frame superstructure will be used with wood joists or trusses for nailable roof ceiling panels.
(3) Plaster on masonry walls using earthtone colors will be provided for low maintenance and durability. (Brick and rock veneer are possible alternates.)
(4) Sheet metal roofs with standing seams are specified on slopes and exposed parapets. A flat-seam roof will be used on flat roofs and fascias; parapets are sufficiently high to hide roof vents and mechanical equipment.
(5) Steel door frames will be filled with grout for strength, stability, and durability.

(6) Borrowed light will be provided from clerestory windows in annodized aluminum frames above the plate height to provide natural light and to require low maintenance

(7) Exterior soffits and beams will be rough sawn cedar sealed and stained for acoustical and aesthetic purposes and will be above the reach of users.

5.2.3 Building Function Description

(1) To allow cleaning and maintenance of units without denying access to facilities, the building is designed with two separate, complete and identical toilet rooms (units) for each sex. Toilet stalls and urinal stalls are separated by the lavatory area to eliminate the opportunity for illicit sex between adjacent stalls.

(2) No mirrors will be placed over lavatories to prevent shaving, combing of hair, applying makeup, etc. in front of, and thus blocking access to, lavatories. Mirrors over a shelf will be located between the two doors on the outside wall. Electrical outlets for razors, hair dryers, etc. will be provided. The shelves will be sufficiently wide for changing diapers.

(3) Having two doors to each toilet unit provides users an alternate escape route should they be feel threatened and, primarily, the two doors prevent sexual deviants from easily locking or securing a single door, thus making the area unattractive for sexual activities. Users can, however, exit the same door they entered.

(4) All plumbing will penetrate the interior wall to the utility room, where maintenance access to the equipment is readily available.

(5) Toilet partitions shall be tubular stainless steel supports with double width stainless steel clad marine plywood panels forming a 4-in. wide partition. This allows paper and supplies dispensers to be recessed into the partitian and makes drilling sexual liaison holes in partitions impractical for illicit use due to the width of the partition. Partition panels are demountable, all one size, custom fabricated composite modules, which can be replaced if damaged. Panels will be attached to tubular supports with tamper-proof head stainless steel screws.

(6) The plumbing fixtures and toilet stall partitions will be suspended above the floor. The walls and floors will be of ceramic tile with curved corners. These features allow easy cleaning of the floor and walls by use of high pressure water or steam equipment, with no cracks or corners to catch and hold dirt.

(7) The porches between units on the corners of the building provide exterior waiting space outside the units, where trash receptacles, drinking fountains, bulletin boards, maps, and public telephones may be provided under cover. These porches also mark obvious extrance/exit access to the buildings from parking areas. Drinking fountains will be attached to the structure rather than be free standing, to minimize vandalism.

(8) Units and porches will be well-lighted with general illumination, indirect high intensity, metal vapor lighting. The lighting over lavatories will be strip flourescent over luminar grids. The flourescent strips will also provide fast start emergency lighting with battery backup in case of momentary or longer power loss which would shut off the metal vapor lights for at least five minutes.

(9) Compressed air, rather than heated air, hand driers will be provided at each lavatory from a central compressed air tank and compressor.

(10) Individual soap dispensers at each lavatory will be connected to a central tank located in the central utility room, which permits constant monitoring.

(11) Portable steam cleaning units are recommended for cleaning the interiors of toilet rooms, sidewalks, and picnic tables.

5.2.4 Specifications

Outline specifications are included in Appendix A.

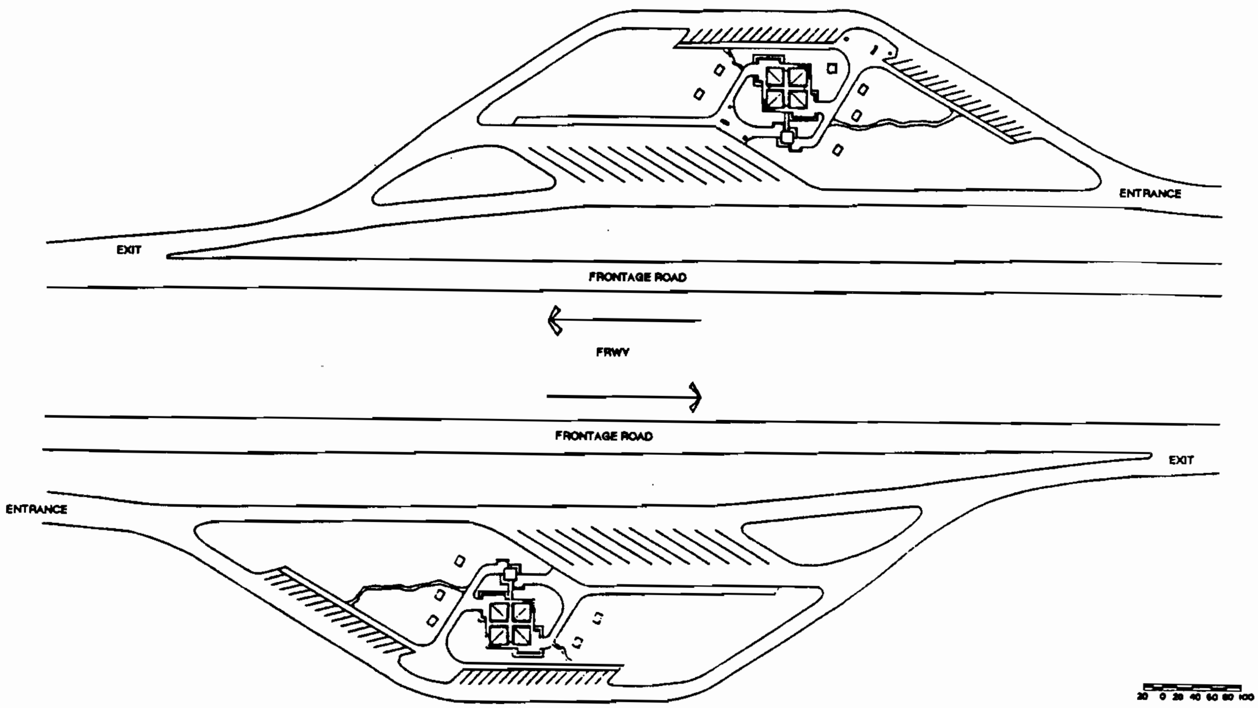


Fig 5.1. Overall site plan.

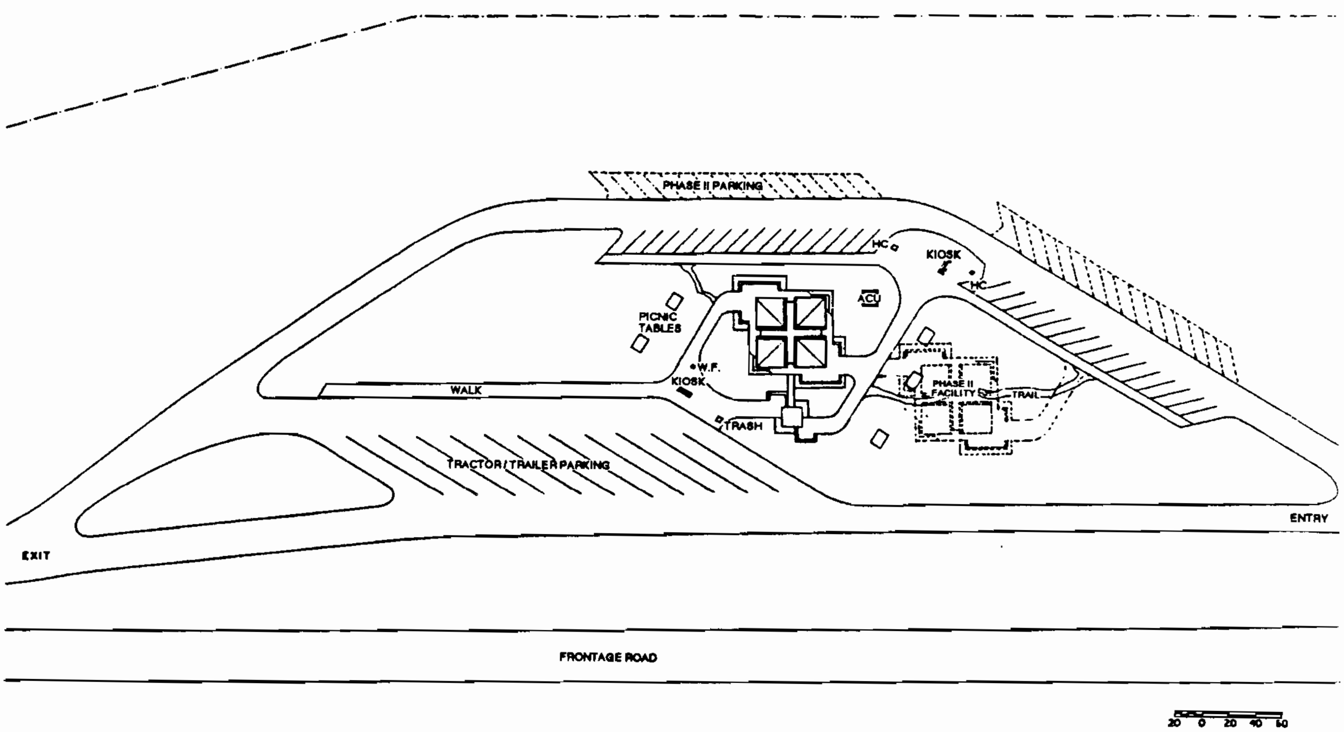


Fig 5.2. Site plan.

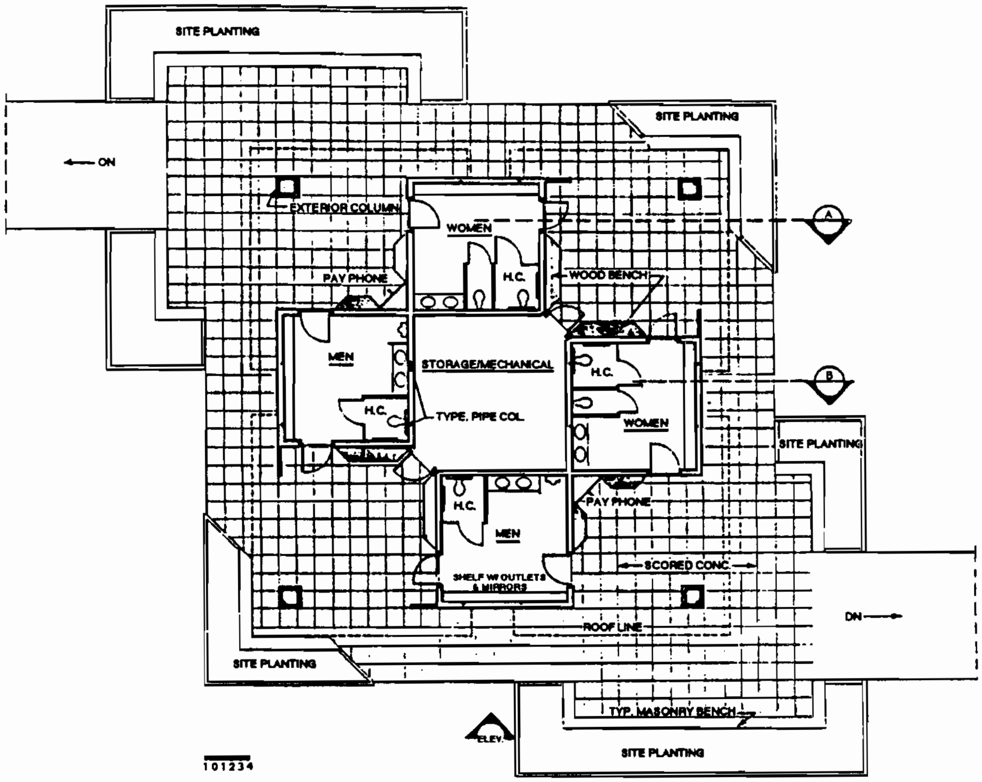


Fig 5.3. Floor plan.

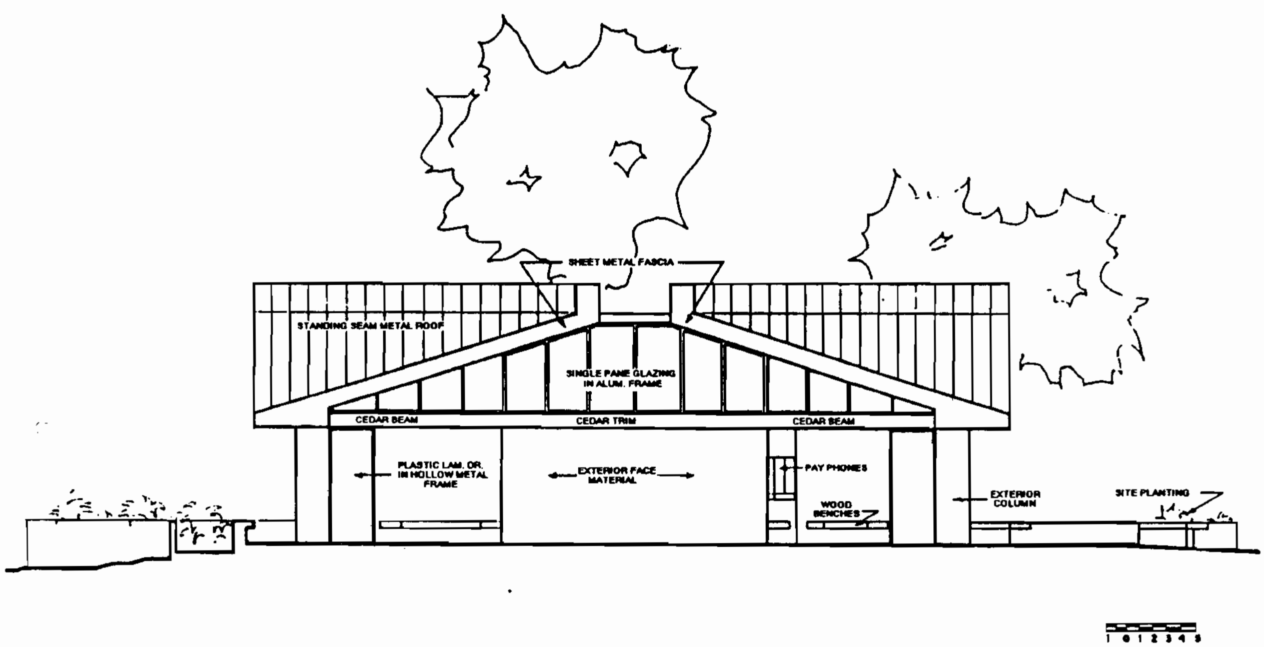


Fig 5.4. Elevation.

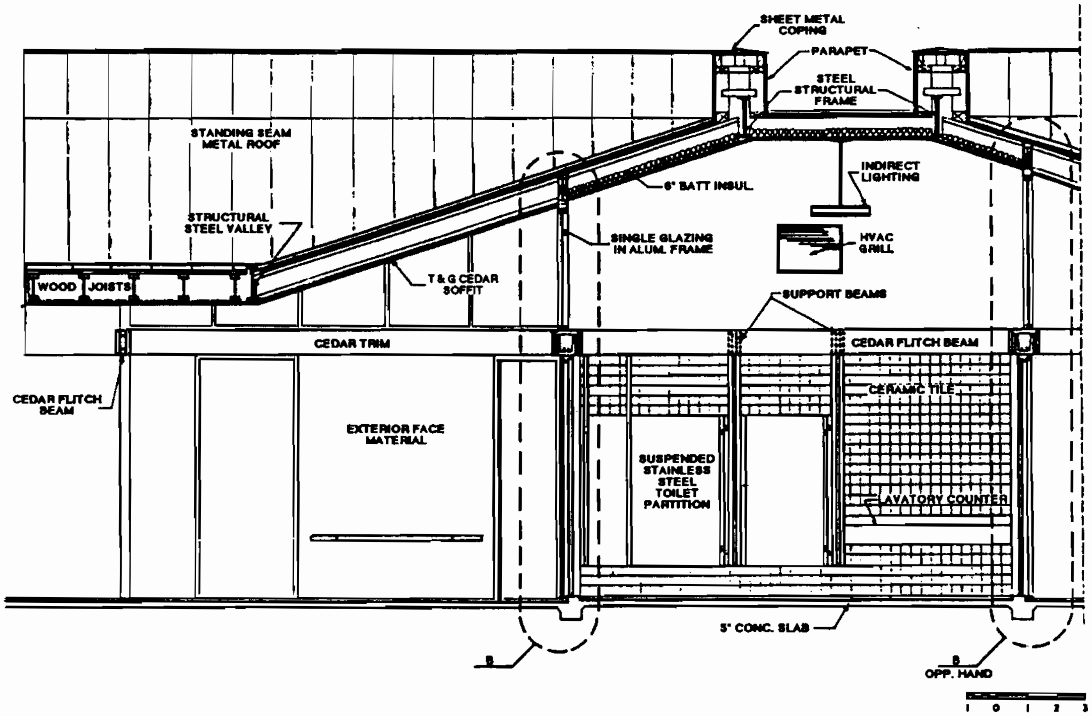


Fig 5.5. Building section 'A.'

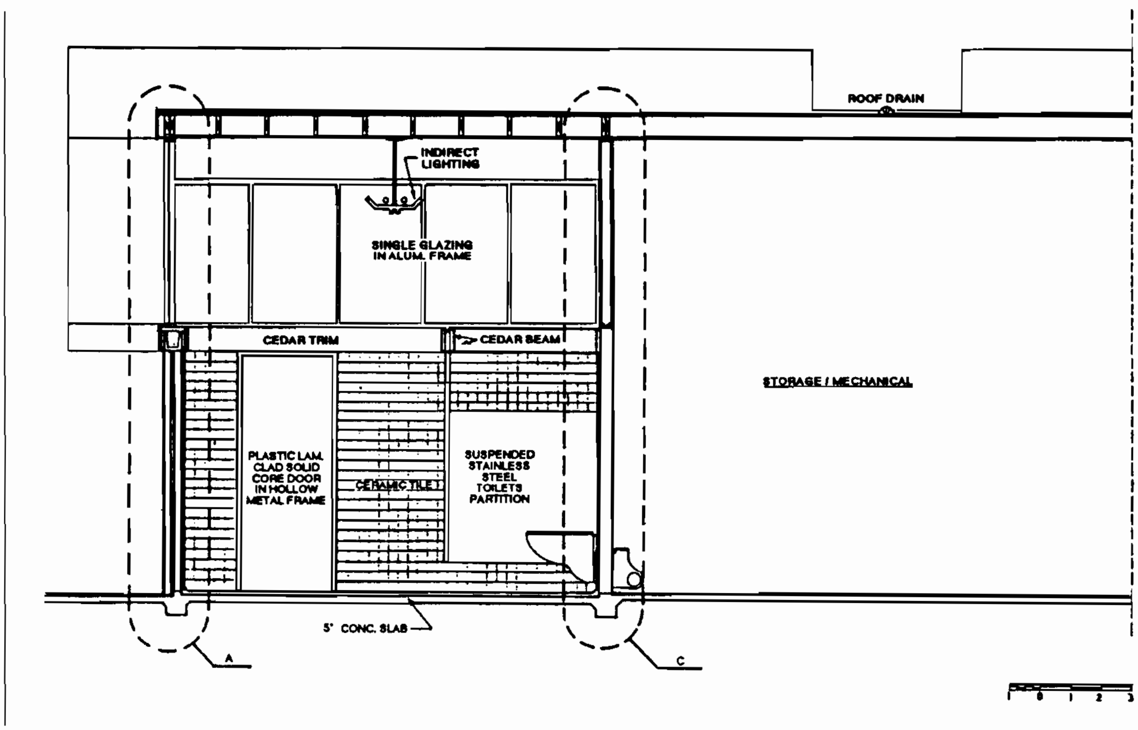


Fig 5.6. Building section 'B.'

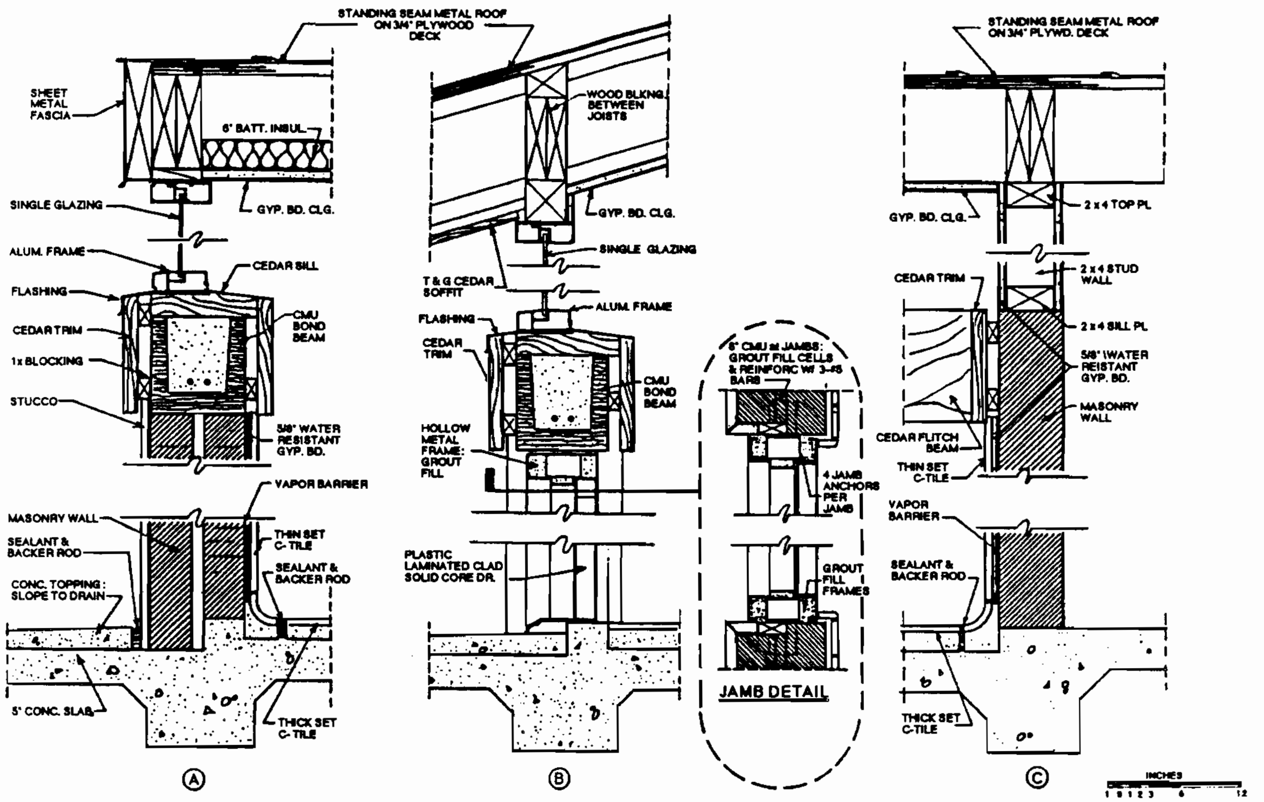


Fig 5.7. Wall sections.

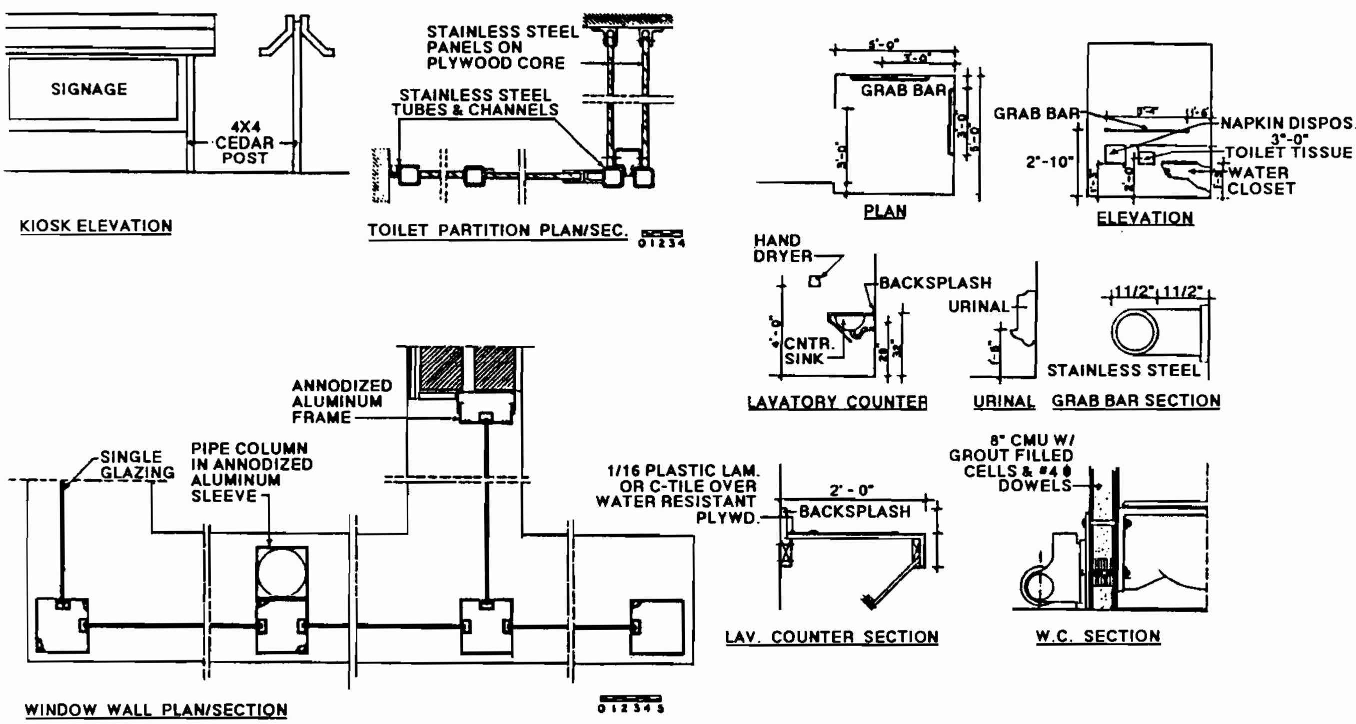
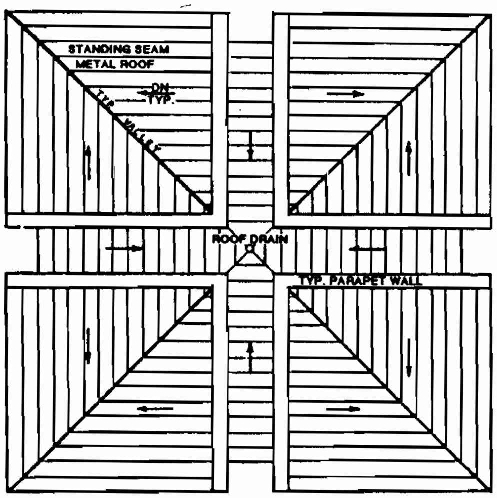
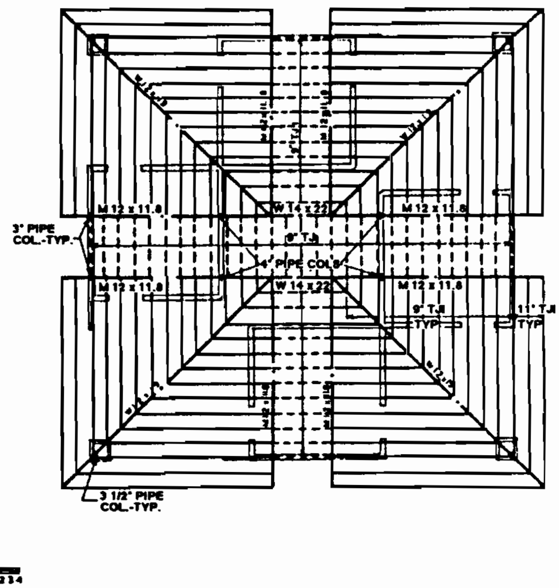


Fig 5.8. Interior elevations and details.





(a) Roof plan.

(b) Roof framing.

Fig 5.9.

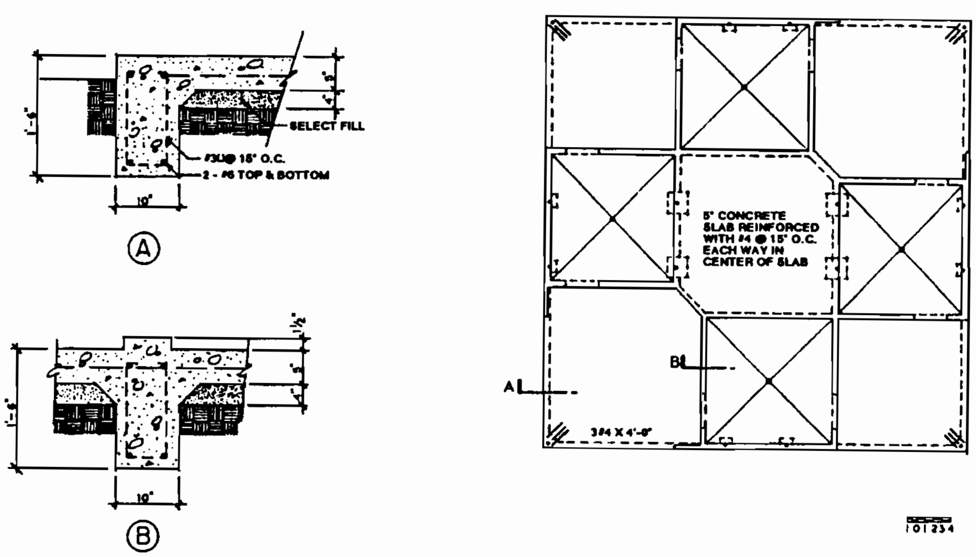


Fig 5.10. Foundation plan.

CHAPTER 6. SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

6.1. SUMMARY

A prototype comfort station design has been presented in this report. A site on U.S. 59, northeast of Victoria, was selected. Each comfort station is designed to accommodate about 500,000 users per year. The design of the water and wastewater system is presented. Design requirements of heating, air conditioning, and electrical power are given. The design program requirements and the prototype design are included.

6.2 CONCLUSIONS

A satisfactory rest area comfort station design involves many considerations: number of users; location, site, and terrain of the site; parking for large and small vehicles; landscaping; lighting, shelters, tables, and other ancillary features; comfort station buildings; water supply; wastewater and sewage disposal; and energy sources. Maintenance, safety, and security must rank high on the priority list of design considerations.

These design recommendations will result in attractive, safe, and low maintenance facilities and provide the travelling public with modern, functional rest area comfort stations.

6.3 RECOMMENDATIONS

It is recommended that

(1) One or more prototype rest areas should be constructed.

(2) The rest areas should be evaluated for user satisfaction, energy and water consumption, and maintenance costs.

(3) The design recommendations made in this study should be refined or modified based on the evaluations.

REFERENCES

1. "Soil Survey of Victoria County, Texas," United States Department of Agriculture, Soil Conservation Service, May, 1982.

2. Scharfe, C. W., and Malina, J. F., Jr., "Water and Wastewater Treatment Systems at Highway Rest Areas," Research Report No. 442-3, Center for Transportation Research, The University of Texas at Austin, November, 1987.

3. "Construction Standards for Private Sewage Facilities," Texas Department of Health," July, 1987.

4. Straughan, W. T., Fowler, D. W., and Perry, K. W., "Investigation of Rest Area Requirements," Research Report No. 442-1, Center for Transportation Research, The University of Texas at Austin, December, 1986.

5. Rock, B. A., and Vliet, G. C., "Evaluation of Energy Sources for Roadside Rest Areas," Research Report No. 442-2, Center for Transportation Research, The University of Texas at Austin, December, 1986.

6. Fowler, D. W., Malina, J. F., Jr., Perry, K. W., and Vliet, G. C., "Design Recommendations for Rest Areas," Research Report No. 442-4, Center for Transportation Research, The University of Texas at Austin, November, 1987.

APPENDIX
OUTLINE SPECIFICATIONS FOR COMFORT STATIONS

1. GENERAL REQUIREMENTS

The project will be constructed according to the Uniform General Conditions for All State of Texas Building Construction Projects and shall conform to the requirements of Article 7, Article 601b, Vernon's Texas Civil Statutes (Elimination of Architectural Barriers Act).

2. SITE

Curbs and Gutters:	Reinforced concrete.
Walks:	Reinforced concrete, tooled edges and joints and, non-slip broom finish.
Finish Grades:	Top soil, suitable for growing grass, shaped to drain water away from the building.

3. CONCRETE BUILDING FOUNDATION AND FLOOR

Concrete slab on engineered fill with vapor barrier.

4. MASONRY

Concrete masonry units plastered on exterior.

Interior Walls:	Concrete masonry units
Reinforcing:	Truss type, No. 9 gauge in bed joints spaced 16-in. o. c. verti cally. Reinforce concrete masonry units at each door jamb and at each wall hung plumbing fixture with #4 reinforcing bonded in cell filled with mortar.

5. STRUCTURAL METALS

Miscellaneous steel sections, columns and beams conforming to ASTM A-36.

6. CARPENTRY

Roof framing TJ1 series wood Truss Joists. Rough sawn western cedar trim. Laminated plastic lavatory counter top and splash and wall shelf.

7. MOISTURE PROTECTION

Galvanized metal standing seam roof applied over resin size paper.

Building Insulation:	Glass fiber batts in roof structure and styrofoam in walls.
Sealant and Caulking:	Seal around exterior door frames and store front. Expansion joints in ceramic tile con structed with sealant and backer rod.

8. DOORS, WINDOWS AND GLASS DOORS

Solid core with laminated plastic veneer

Steel door frames:	Sixteen ga. filled with mortar.
Storefront:	Extruded aluminum flush glaze system with class 1 anodic coating on exposed surfaces.
Butts:	Stainless steel, non-removable pin, ball bearing hinges with door closers.
Closure:	Overhead concealed.
Dead lock:	Mortise type, heavy duty.
Push-pulls:	Stainless steel.
Kick plates:	Stainless steel type 302, 18 gauge.
Glazing:	Lexan polycarbonate sheet, 1/4-in. thick, ultra violet and mar resistant.

9. FINISHES

Plaster:	Portland cement plaster on self-furring lath with expansion beads, maximum spacing 12 ft. o. c. hori zontally on 10 ft. o. c. vertically. Finish coat: Thorocoat.
Ceramic tile:	Standard grade meeting the require ments of ANSI A137.1
Wall tile:	Colored matte or semi-matte with cushion edge with trim pieces such as bull-nose corners and cove base.
Ceramic mosaic:	Unglazed, all purpose edge mounted in sheets.

<u>Gypsum Drywall</u>

Gypsum board:	5/8-in. thick "Firecode" type S. W. joints filled, taped and floated.

<u>Painting</u>

General:	Paint all wood surfaces, gypsum board, concrete masonry units ex posed to view and ferrous metal both interior and exterior.
Gypsum board:	Light texture and satin enamel fin ish.
Ferrous metal:	Metal primer, semi-gloss enamel finish.
Interior wood:	Semi-transparent stain, satin varnish finish.

Exterior wood: Semi-transparent stain.

10. SPECIALTIES

Toilet partitions and urinal screens: Beam hung, custom fabricated from 3/4-in. marine plywood panels covered with 20-ga. type 302 stain less steel. Supports are type 302 stainless steel. Tubes, sizes as shown on drawings.

Toilet Accessories

Grab bars: 1-1/4-in. diameter stainless steel with concealed fasteners.

Mirror: Stainless steel frame, secured to wall from front with tamper resistant screws. Reflective surface shall be type 430 bright polished stainless steel mirror.

Toilet tissue dispenser: Double roll, heavy duty cast alu minum bracket, and spindles shall have retractable pin and concealed locking mechanism. Spindle can be removed with special key furnished with unit.

Coat hooks: Heavy duty stainless steel attached with concealed tamper proof screws.

Mechanical: Electric powered heat pump air conditioning unit, 80,000 BTU/ hr heating / 50,000 BTU/hr. cooling.

Plumbing Fixtures

Water closet: Vitreous china, wall hung elongated siphon jet with concealed flush con trols and tamper-proof bolts.

Urinal: Vitreous china, wall hung, washout urinal with integral flushing rim with concealed flush controls.

Lavatory: Vitreous china, counter mounted, self rimming with spring-driven automatic closing/mixing faucet.

Lighting

General illumination: HID metal halide in toilet rooms and exterior spaces under the roof.

Flourescent lighting: Mount over lavatories and in me chanical room.