PRELIMINARY SEWER STUDY

Coyne Ranch at Sunbeam Lake

West of Bennett Road, and North of W Ross Road
Imperial County, CA

Prepared By

Fuscoe Engineering
6390 Greenwich Dr. Ste 170
San Diego, CA 92122
(858) 554-1500

REGISTERED PROFESSIONAL ENGINEER
EXP. 9-30-16
3/24/16
CIVIL
STATE OF CALIFORNIA

January 2016

TABLE OF CONTENTS

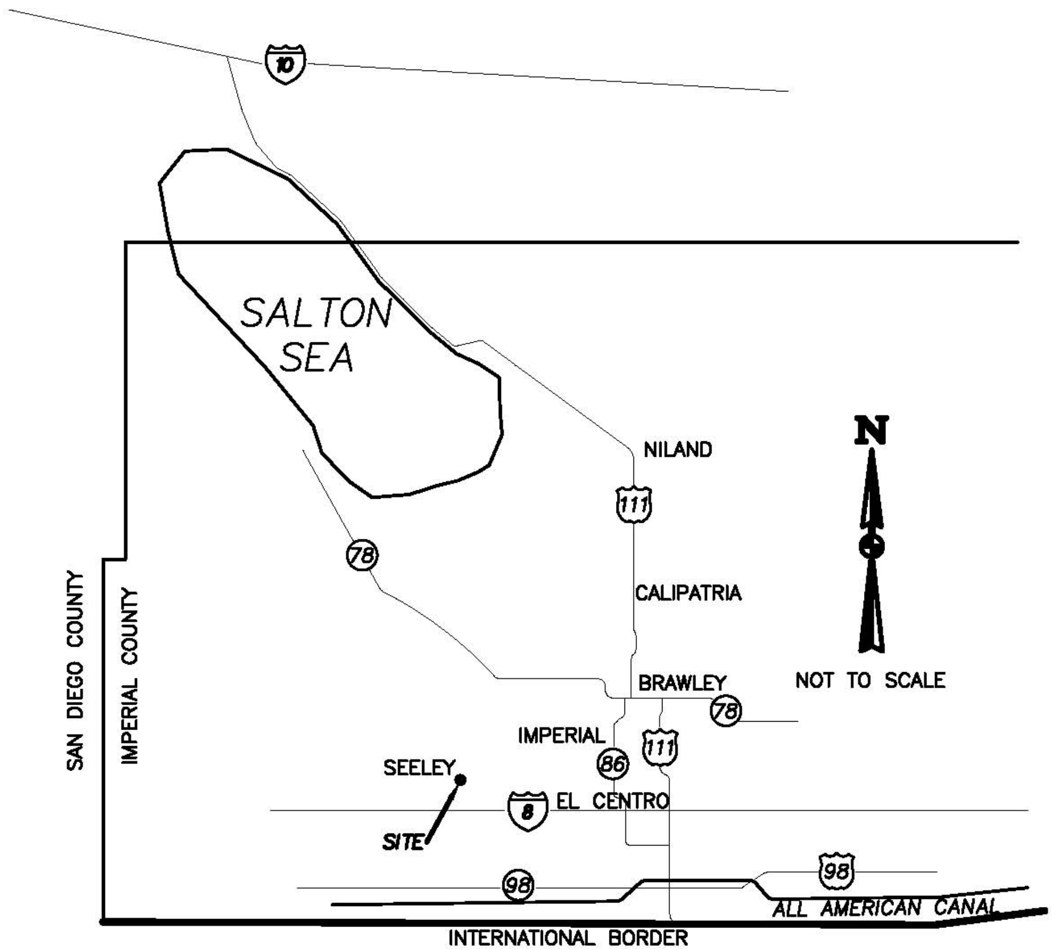


Figure 1 Vicinity Map

1.0 INTRODUCTION

This preliminary sewer study is prepared per Seeley County Water District (SCWD) requirements and is intended to address the on-site wastewater flow of the proposed project. The City of San Diego Sewer Design Guide (May 2015) is used for standards in design. This document is subject to revisions as needed by the engineer.

1.1 PROJECT SITE DESCRIPTION

The proposed project site is a rectangular shaped area west of Bennett Rd, and north of W Ross Road, Imperial County, CA. The site is approximately 0.6 miles north of Interstate 8 and approximately 0.9 miles southeast of Seeley, CA. The existing site consists of 128 acres of agricultural fields, irrigation delivery ditches, and some tile drains.

The proposed development is a 450 lot residential subdivision, which includes residential streets and a large extended detention basin. 444 lots will be single-family residences and 6 lots are multifamily residential developments. Please see Figure 1 on the preceding page for a vicinity map.

1.2 EXISTING CONDITIONS

Currently, there is no existing sewer system in the immediate vicinity of the project. The Seeley County Water District has an existing gravity system to the northwest of the project. They also have an existing wastewater treatment plant, which receives all flows from Seeley.

1.3 PROPOSED CONDITIONS

A private sewer line is proposed for the project, serving all of the lots. The system will be an underground gravity system within the project limits. The system will consist of 8"-10" PVC main lines with concrete sewer manholes. Please see Attachment A for Sewer Layout Exhibit. Since there is no sewer system in the area, the project will propose to connect to the Seeley County Water District wastewater treatment plant to the northeast of the project in the town of Seeley via a proposed pump station and force main. This proposed force main will be located in unpaved areas where feasible to minimize installation costs.

2.0 METHODOLOGY AND CRITERIA

2.1 INTRODUCTION

Sewer calculations from the Coyne Ranch project were performed in accordance with the City of San Diego Sewer Design Guide and per capita demand estimates provided by representatives of Dynamic Consulting Engineers (DCE), contract engineer for SCWD.

2.2 DEMAND AND POPULATION ESTIMATES

For the project demand estimates, a daily demand of 90 gallons per capita was used. For the single-family houses, a unit density of 4 persons per dwelling unit was assumed at one dwelling unit per lot. For the multi-family lots, and unit density of 3.3 persons per dwelling unit was assumed at 29 dwelling units per acre. This yields a population of 2,499 persons, and a sewer demand of 224,910 gallons per day. See Table 1 and Table 2 for estimates summary.

Table 1 – Population Estimate

Description	Density (DU)	Unit Density (persons/DU)	Units	Population
Single-Family	1 per Lot	4	444 Lots	1,776
Multi-Family	29 per Acre	3.3	7.55 Acres	723
Total				2,499

Table 2 – Sewer Demand Estimate

Population	Demand per capita (gpdc)	Total Demand (gal/day)
2,499	90	224,910

2.3 WASTEWATER TREATMENT PLANT

As mentioned previously, the project proposes to route flow to the SCWD sewer system to the northeast of the project. There is an existing wastewater treatment plant (WWTP) with an existing capacity of 250,000 gallons per day (gpd). The current demand on the plant from the town of Seeley is 150,000 gpd, leaving 100,000 gpd available. Acceptance of flow from the proposed project will exceed the existing capacity of the treatment plant by approximately 125,000 gpd, and additional capacity is required.

Per discussion with DCE, to provide the necessary capacity for the Coyne Ranch at Sunbeam Lake project an additional 250,000 gpd WWTP could be constructed. The additional WWTP would be identical in footprint, and adjacent to, the existing plant.

2.4 PUMP STATION

Due to the flat terrain within and adjacent to the project, a pump station will be necessary to pump flows from the project to the wastewater treatment plant. The pump station is proposed at the northwest corner of the project. The proposed route of the sewer force main is as follows: From the pump station location in the northwest portion of the project site, the force main will extend north along the westerly property line. To the north of the property, the force main will run north in the unpaved farm road, crossing the railroad tracks, and turning west along Evan Hughes Highway (County Highway S80). Along Evan Hughes Highway, the force main would be located in the unpaved shoulder to the south of the edge of pavement. The force main would turn north at Mount Signal Avenue, again running the unpaved shoulder along the west side of the street. The force main would transition to gravity flow at a new manhole near the intersection with Main Street. A short length of gravity sewer would then connect to an existing sewer manhole. The total length of sewer force main is approximately 8,080 lf.

2.5 PROPOSED IMPROVEMENTS

Improvements proposed with the project include the following:

1. On-site sewer mains and laterals
2. On-site pump station
3. Off-site force main and gravity main connecting to SCWD infrastructure

2.5.1 PUMP STATION

PUMP STATION DEMAND

The on-site pump station is sized to receive and discharge flow from the Coyne Ranch at Sunbeam Lake project. The average and peak flows to the pump station are provided in Table 3 below. Peaking factors are determined from the equation below:

Peaking Factor = $6.2945 \times (\text{population})^{-0.1342}$

Table 3 – Pump Station Demand

Condition	Avg Daily Flow (gpd)	Avg Daily Flow (gpm)	Population	Peaking Factor	Peak Flow (gpm)	Peak Flow (cfs)
Developed	224,918	156	2,499	2.22	346	0.77

Given that the ultimate Average Daily Flow (ADF) on the pump station is less than 3.0 MGD, constant speed pumps are proposed.

WET WELL DESIGN

The pump station will house 2 pumps and a wet well. The wet well is preliminarily designed to cycle 6 times per hour in both the initial and ultimate condition. The pumps are preliminarily sized to provide a pumping capacity equal to the peak flow of 346 gpm. Table 4 below presents the process to determine the duration of each cycle, assuming 6 cycles per hour and a pumping capacity of 346 gpm.

Table 4 – Wet Well Cycle Duration

Condition	Avg Daily Flow (gpd)	Avg Hourly Flow (gph)	Peaking Factor	Peak Hourly Flow (gph)	Cycles per Hour	Flow per Cycle (gal)	Pumping Capacity (gpm)	Cycle Duration (min)
Developed	224,918	9,372	2.22	20,806	6	3,468	346	10.0

Assume a cycle duration of 10.0 minutes. The required wet well volume is determined from:

$V = (Q \times t) / 4$

V = Operational Volume, gal
Q = Pumping Capacity, gpm
t = Cycle Duration, minutes

The operational volumes are provided below for the initial and ultimate condition in Table 5.

Table 5 – Wet Well Operational Volume

Condition	Pumping Capacity (gpm)	Cycles / hour	Cycle Time (min)	Operational Volume (gal)	Operational Volume (cu.ft.)
Developed	346	6	10	865	116

For the initial and ultimate conditions, assuming a 9' diameter wet well, the operational depths are determined as shown in Table 6 below:

Table 6 – Wet Well Operational Depth

Condition	Operational Volume (cu.ft.)	Wet Well Diameter (ft)	Cross-Sectional Area (sf)	Operational Depth (ft)
Developed	116	9	63.6	1.8

In summary, the pump station will house 2 pumps, each with a pumping capacity of 346 gpm that will cycle 6 times per hour. The cycle duration will be 10.0 minutes. The wet well is sized at 9' diameter, with an operational depth of 1.8 ft.

FORCE MAIN

The desired range of velocity in the force main is 2.0 fps minimum (to maintain cleansing velocities) and 8.0 fps maximum (to decrease frictional losses). For a pumping capacity of 346 gpm (0.77 cfs), flow velocities for various pipe diameters is provided in Table 7 below.

Table 7 – Pipe Velocity

Diameter (in)	Velocity (fps)
4	8.8
6	3.9
8	2.2

From Table 7, A 6" diameter force main is acceptable. The estimated length of force main is approximately 8,080'; due to the length of force main, the higher cost of the 8" main may be ultimately justified by the decrease in frictional losses that will occur, but a 6" force main is selected for planning purposes at this time.

PRELIMINARY PUMP SELECTION

Pipe sizing and selection software available from Hydromatic Pumps were utilized to preliminarily size the pumps. Given an 8,080', 6" PVC, a water level at the pump station of 10' and a hydraulic head difference of 6' between the pump and the end of the force main at the intersection of Main Street and Mt Signal Avenue, a required head of approximately 34' is required at a flow rate of 346 gpm. A Hydromatic submersible non-clog pump, series S4L is preliminarily selected for the project. The pump data sheet and additional information is provided in the Appendix.

GRAVITY FLOW TO CONNECTION POINT

The proposed force main will transition to gravity flow at a new manhole near the intersection of Main Street and Mount Signal Avenue. A short length of gravity sewer would then connect to an existing sewer manhole in this intersection.

Based on conversation with DCE, the connection point at Mt. Signal and West Main Street is quite deep and greater than 15' below ground surface. Assuming the force main will be approximately 4' deep at the point where it transitions from pressure to gravity flow, there is adequate depth at the connection point to allow for gravity flow to the existing manhole.

3.0 HYDRAULIC ANALYSIS

3.1 INTRODUCTION

An analysis of the overall sewer system was conducted to find the Average Daily Dry Weather Flows and the corresponding peak flows. The system was designed per the City of San Diego Sewer Design Guidelines. A summary of design guidelines are below:

1. Minimum Pipe Size = 8" for residential
1. Peak Factor = 2.22 for a population of 2,499
2. Manning's Coefficient = 0.013 for PVC
3. Maximum dn/D ratio = 0.5 for all mains less than 15" in diameter
4. Velocities: 2 ft/s < Effluent < 5ft/s. Where flows are less than 2ft/s, a minimum slope of 1% is used.

3.2 CALCULATIONS

The sewer capacities were calculated using Manning's equation (via Bentley FlowMaster software) to calculate depth of flow and velocity in each sewer main segment. Attachment B: Sewer Summary Table presents the hydraulic results for each pipe segment.

3.3 RESULTS

The 8"-12" onsite sewer main system will have adequate capacity to convey the peak flow from the project site while flowing less than half-full for the majority of flows. Minimum and maximum velocities are consistent with the criteria set forth in City of San Diego Sewer Design Guide. The Coyne Ranch project will contribute a total sewer peak flow of 0.77 cfs.

4.0 SUMMARY AND CONCLUSIONS

The Coyne Ranch at Sunbeam Lake project will provide housing for an estimated 2,499 people. The estimated average daily flow (ADF) of wastewater generated by this population is 224,918 gpd (156 gpm). The peaking factor for the project is 2.22, yielding a peak flow of 346 gpm. Given an existing demand for the town of Seeley of 150,000 gpd, the ultimate demand for the WWTP will increase to approximately 374,918 gpd in the fully built out condition. To accommodate flow from the two projects, the SCWD system should be upgraded based on this future demand.

A pump station is planned near the northwest corner of the Coyne Ranch at Sunbeam Lake project to serve the two developments. The pump station will consist of 2 constant speed pumps and a 9' diameter wet well, as well as all required operational controls. The wet well will be sized to have an operational volume of 116 cubic feet and an operational depth of 1.8 feet.

The force main from the pump station will deliver approximately 532 gpm of wastewater and can consist of a 6" PVC. The force main will run from the pump station to the intersection of Main Street and Mount Signal Avenue – a distance of approximately 8,080 feet. The main will transition to gravity flow near an existing manhole located at this intersection. The total head to be delivered by the pump is approximately 34', which can be provided by a Hydromatic Series S4L submersible non-clog pump, or equivalent.

The 8"-12" onsite private sewer main system will be a gravity system within the Coyne Ranch project. It will flow to the northwest corner of the project to the pump station as discussed in this study. Hydraulic calculations indicate that the system will flow at a d/D ratio of less than 0.5. Where flow velocity is less that 2.0 fps, the pipe slope is to be increased to 1.0% minimum, consistent with referenced standards.

5.0 APPENDICES

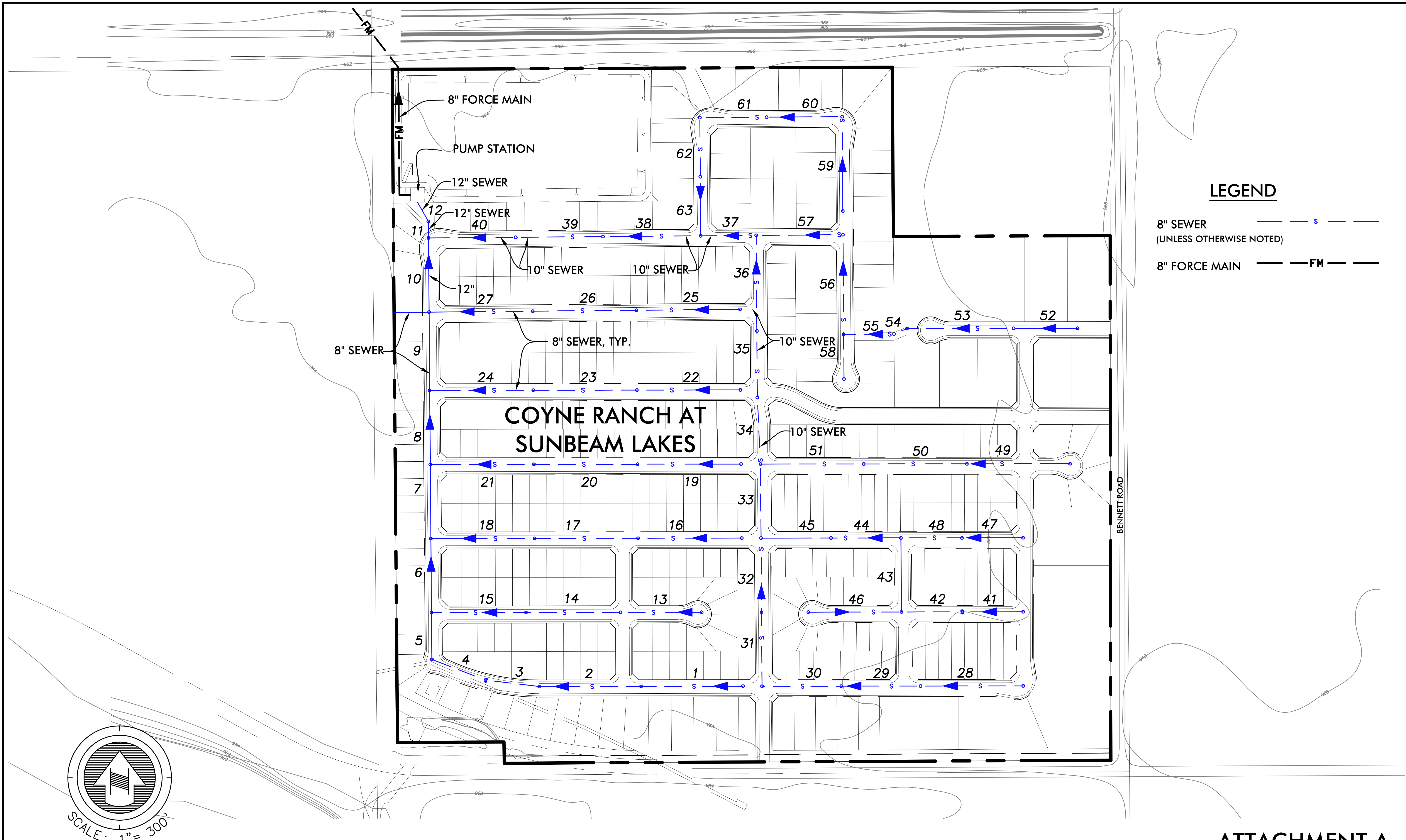
Attachment A *Sewer Layout Exhibit*

Attachment B *Sewer Summary Table*

Attachment C *Pump Data*

Attachment D *Referenced Standards*

ATTACHMENT A
SEWER LAYOUT EXHIBIT



ATTACHMENT A
SEWER LAYOUT EXHIBIT
COYNE RANCH AT SUNBEAM LAKE

ATTACHMENT B
SEWER SUMMARY TABLE

Sewer Summary

Line Label	Length	Inv Upstrm	Inv Dwnstrm	Design Slope (%)	DU Served	Population Served	Cummulative Population	ADWF (gpd)	ADWF(gpm)	ADWF (cfs)	Peak Dry Weather Flow* (gpm))	Peak Dry Weather Flow (cfs)	Line Size (Inches)	dn (in)	dn/D	Vel** (fps)
1	357.73	959.69	956.11	1.0%	11	44	44	3,960	3	0.006	6	0.014	8	0.61	0.08	1.16
2	357.73	956.01	952.44	1.0%	9	36	80	7,200	5	0.011	11	0.025	8	0.80	0.10	1.39
3	190.14	952.34	950.44	1.0%	4	16	96	8,640	6	0.013	13	0.030	8	0.87	0.11	1.46
4	201.50	950.34	948.32	1.0%	1	4	100	9,000	6	0.014	14	0.031	8	0.88	0.11	1.48
5	164.10	948.22	946.58	1.0%	4	16	116	10,440	7	0.016	16	0.036	8	0.95	0.12	1.54
6	260.01	946.48	943.88	1.0%	4	16	260	23,400	16	0.036	36	0.081	8	1.40	0.18	1.97
7	260.00	943.77	941.18	1.0%	4	16	408	36,720	26	0.057	57	0.127	8	1.75	0.22	2.25
8	260.02	941.08	938.48	1.0%	5	20	564	50,760	35	0.079	79	0.175	8	2.06	0.26	2.47
9	274.90	938.38	935.63	1.0%	4	16	716	64,440	45	0.100	100	0.222	8	2.32	0.29	2.64
10	260.12	935.63	934.42	0.5%	5	20	872	78,480	55	0.121	122	0.271	12	2.66	0.22	2.10
11	58.23	934.32	934.09	0.4%	0	0	2,499	224,918	156	0.348	348	0.776	12	4.86	0.41	2.60
12	77.29	933.99	933.68	0.4%	0	0	2,499	224,918	156	0.348	348	0.776	12	4.86	0.41	2.60
13	284.98	960.70	957.85	1.0%	12	48	48	4,320	3	0.007	7	0.015	8	0.63	0.08	1.18
14	331.65	957.75	954.44	1.0%	12	48	96	8,640	6	0.013	13	0.030	8	0.87	0.11	1.46
15	331.65	954.34	946.58	2.3%	8	32	128	11,520	8	0.018	18	0.040	8	0.82	0.10	2.13
16	363.70	958.14	954.50	1.0%	12	48	48	4,320	3	0.007	7	0.015	8	0.63	0.08	1.18
17	363.70	954.40	950.77	1.0%	11	44	92	8,280	6	0.013	13	0.029	8	0.86	0.11	1.45
18	363.70	950.67	945.44	1.9%	10	40	132	11,880	8	0.018	18	0.041	8	0.87	0.11	2.01
19	363.74	957.56	953.93	1.0%	12	48	48	4,320	3	0.007	7	0.015	8	0.63	0.08	1.18
20	363.74	953.83	950.19	1.0%	12	48	96	8,640	6	0.013	13	0.030	8	0.87	0.11	1.46
21	363.74	950.09	944.30	2.4%	10	40	136	12,240	9	0.019	19	0.042	8	0.83	0.10	2.20
22	363.50	956.56	952.92	1.0%	12	48	48	4,320	3	0.007	7	0.015	8	0.63	0.08	1.18
23	363.50	952.82	949.19	1.0%	12	48	96	8,640	6	0.013	13	0.030	8	0.87	0.11	1.46
24	363.50	949.09	943.16	2.9%	10	40	136	12,240	9	0.019	19	0.042	8	0.79	0.10	2.35
25	363.85	957.39	953.75	1.0%	12	48	48	4,320	3	0.007	7	0.015	8	0.63	0.08	1.18
26	363.85	953.65	950.01	1.0%	12	48	96	8,640	6	0.013	13	0.030	8	0.87	0.11	1.46
27	363.85	954.28	941.96	5.1%	10	40	136	12,240	9	0.019	19	0.042	8	0.69	0.09	2.87
28	360.68	958.88	957.43	1.0%	12	511	511	45,995	32	0.071	71	0.159	8	1.96	0.25	2.40
29	278.13	957.33	956.22	1.0%	6	24	535	48,155	33	0.075	75	0.166	8	2.00	0.25	2.43
30	278.13	956.12	955.01	1.0%	8	32	567	51,035	35	0.079	79	0.176	8	2.06	0.26	2.47
31	260.06	954.91	953.87	1.0%	0	0	567	51,035	35	0.079	79	0.176	8	2.06	0.26	2.47
32	260.06	953.77	950.76	1.0%	0	0	567	51,035	35	0.079	79	0.176	8	2.06	0.26	2.47
33	259.89	950.66	945.90	0.4%	0	0	1,087	97,838	68	0.151	152	0.338	8	3.72	0.47	2.12
34	233.76	945.80	944.87	0.4%	0	0	1,251	112,598	78	0.174	174	0.388	10	3.62	0.36	2.18
35	233.56	944.77	943.84	0.4%	4	16	1,267	114,038	79	0.176	177	0.393	10	3.65	0.37	2.18
36	335.68	943.74	942.39	0.4%	4	16	1,283	115,478	80	0.179	179	0.398	10	3.67	0.37	2.19
37	194.92	942.39	941.61	0.4%	2	8	1,403	126,278	88	0.195	196	0.436	10	3.63	0.36	2.18
38	342.90	941.51	940.09	0.4%	11	44	1,555	139,958	97	0.217	217	0.483	10	3.86	0.39	2.25
39	306.23	939.99	938.77	0.4%	10	40	1,595	143,558	100	0.222	222	0.495	10	3.92	0.39	2.27
40	306.23	938.67	937.44	0.4%	8	32	1,627	146,438	102	0.227	227	0.505	10	3.96	0.40	2.28
41	213.66	958.09	957.24	1.0%	11	162	162	14,539	10	0.022	23	0.050	8	1.11	0.14	1.70
42	213.72	957.14	956.28	1.0%	6	24	186	16,699	12	0.026	26	0.058	8	1.19	0.15	1.78
43	260.00	956.18	952.92	1.0%	0	0	254	22,819	16	0.035	35	0.079	8	1.39	0.17	1.95

44	245.50	952.82	951.84	0.4%	11	44	488	43,923	31	0.068	68	0.152	8	2.42	0.30	1.71
45	245.50	951.74	950.76	0.4%	8	32	520	46,803	33	0.072	72	0.161	8	2.49	0.31	1.73
46	326.66	963.14	956.28	2.1%	17	68	68	6,120	4	0.009	9	0.021	8	0.62	0.08	1.69
47	214.41	957.31	955.17	1.0%	9	154	154	13,905	10	0.022	22	0.048	8	1.09	0.14	1.68
48	214.41	955.07	952.92	1.0%	9	36	190	17,145	12	0.027	27	0.059	8	1.20	0.15	1.79
49	362.20	956.97	953.35	1.0%	15	60	60	5,400	4	0.008	8	0.019	8	0.70	0.09	1.28
50	362.20	953.25	949.63	1.0%	14	56	116	10,440	7	0.016	16	0.036	8	0.95	0.12	1.54
51	362.20	949.53	945.90	1.8%	12	48	164	14,760	10	0.023	23	0.051	8	0.97	0.12	2.11
52	224.43	959.92	956.78	1.4%	5	20	20	1,800	1	0.003	3	0.006	8	0.38	0.05	1.01
53	373.47	956.68	952.94	1.0%	5	20	40	3,600	3	0.006	6	0.012	8	0.57	0.07	1.10
54	49.79	952.84	952.35	1.0%	0	0	40	3,600	3	0.006	6	0.012	8	0.57	0.07	1.10
55	177.00	952.25	950.63	1.0%	0	0	40	3,600	3	0.006	6	0.012	8	0.57	0.07	1.10
56	349.93	950.53	947.03	1.0%	8	32	100	9,000	6	0.014	14	0.031	8	0.88	0.11	1.48
57	304.65	946.93	942.49	1.5%	3	12	112	10,080	7	0.016	16	0.035	8	0.85	0.11	1.77
58	156.83	958.84	950.63	5.2%	7	28	28	2,520	2	0.004	4	0.009	8	0.33	0.04	1.83
59	331.51	961.95	956.64	1.6%	9	36	36	3,240	2	0.005	5	0.011	8	0.49	0.06	1.27
60	265.22	956.54	952.30	1.6%	5	20	56	5,040	4	0.008	8	0.017	8	0.60	0.08	1.44
61	234.35	952.20	948.45	1.6%	4	16	72	6,480	5	0.010	10	0.022	8	0.67	0.08	1.58
62	207.36	948.35	945.03	1.6%	6	24	96	8,640	6	0.013	13	0.030	8	0.78	0.10	1.72
63	207.36	944.93	941.61	1.6%	3	12	108	9,720	7	0.015	15	0.034	8	0.82	0.10	1.79

*Flow Rate calculated using 90 gpd per person assuming 4 people per DU for SFR and 29.3 for MF and a peaking factor of 2.22 Figure 1-1 of City of San Diego Sewer Design Guide

** Minimum slope of 1.0% was used for Pipe segments with Velocity less than 2 fps per 3-302.2

ATTACHMENT C
PUMP DATA

Pump Data Sheet - HYDROMATIC

PENTAIR

HYDROMATIC®

Company: Fuscoe Engineering
Name: James Moser
Date: 1/26/2016

Pump:

Size: S4L/S4LX

Type: SUBMERSIBLE SH-4
Synch speed: 1800 rpm

Speed: 1750 rpm
Dia: 9.375 in

Curve: S4L1750

Impeller:

Specific Speeds:

Ns: ---
Nss: ---

Dimensions:

Suction: ---
Discharge: 4 in

Pump Limits:

Temperature: 140 °F
Pressure: 125 psi g
Sphere size: 3.25 in

Power: ---
Eye area: ---

Search Criteria:

Flow: 346 US gpm

Head: 67 ft

Fluid:

Water
SG: 1
Viscosity: 1.105 cP
NPSHa: 33.3 ft

Temperature: 60 °F
Vapor pressure: 0.2563 psi a
Atm pressure: 14.7 psi a

Motor:

Consult HYDROMATIC to select a motor for this pump.

Pump Selection Warnings:

Catalog does not contain data to verify that NPSHa is sufficient.

---- Data Point ----

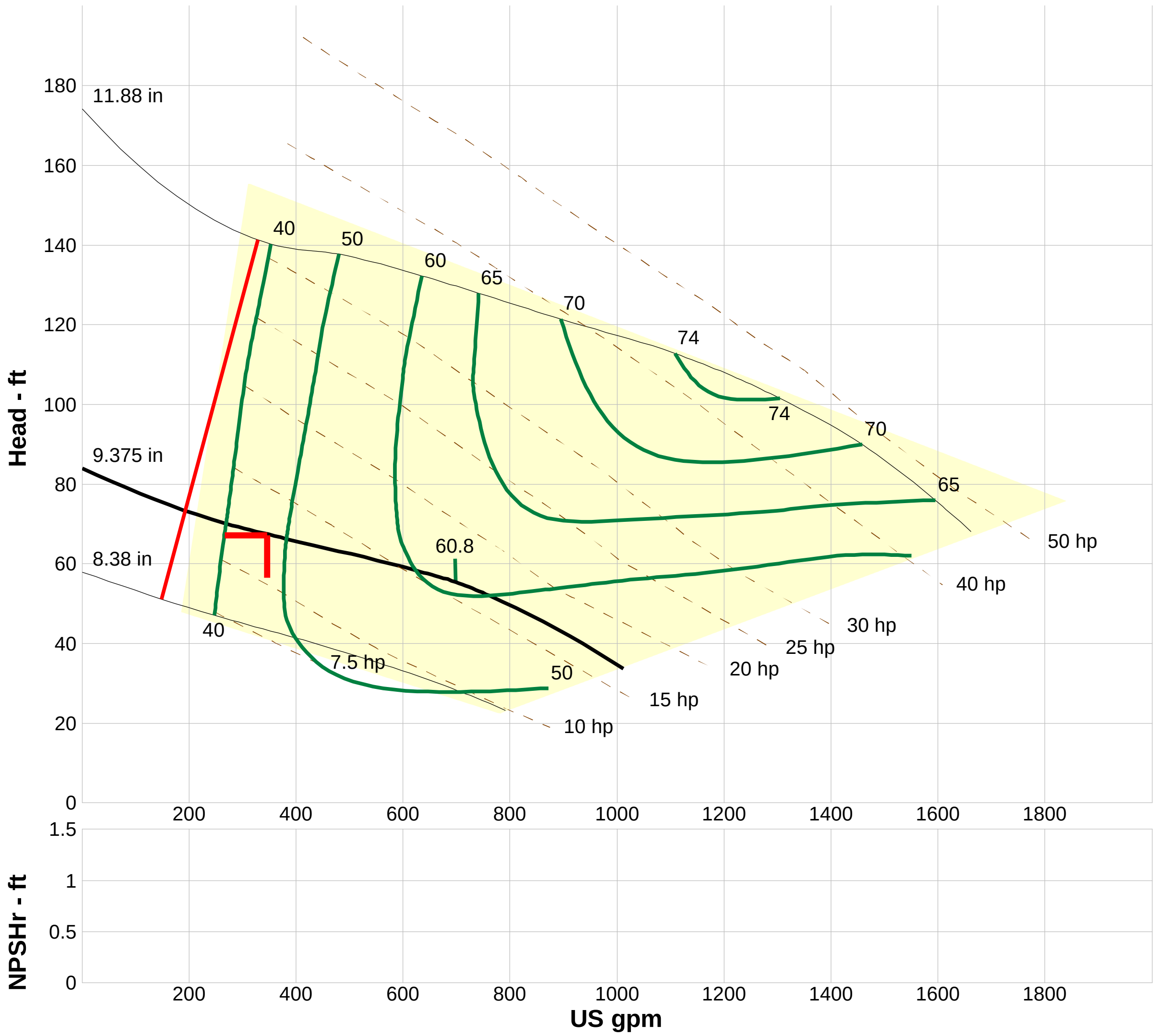
Flow:	346 US gpm
Head:	67.5 ft
Eff:	47%
Power:	12.5 hp
NPSHr:	---

---- Design Curve ----

Shutoff head:	84 ft
Shutoff dP:	36.4 psi
Min flow:	188 US gpm
BEP:	61% @ 697 US gpm
NOL power:	16.6 hp @ 760 US gpm

-- Max Curve --

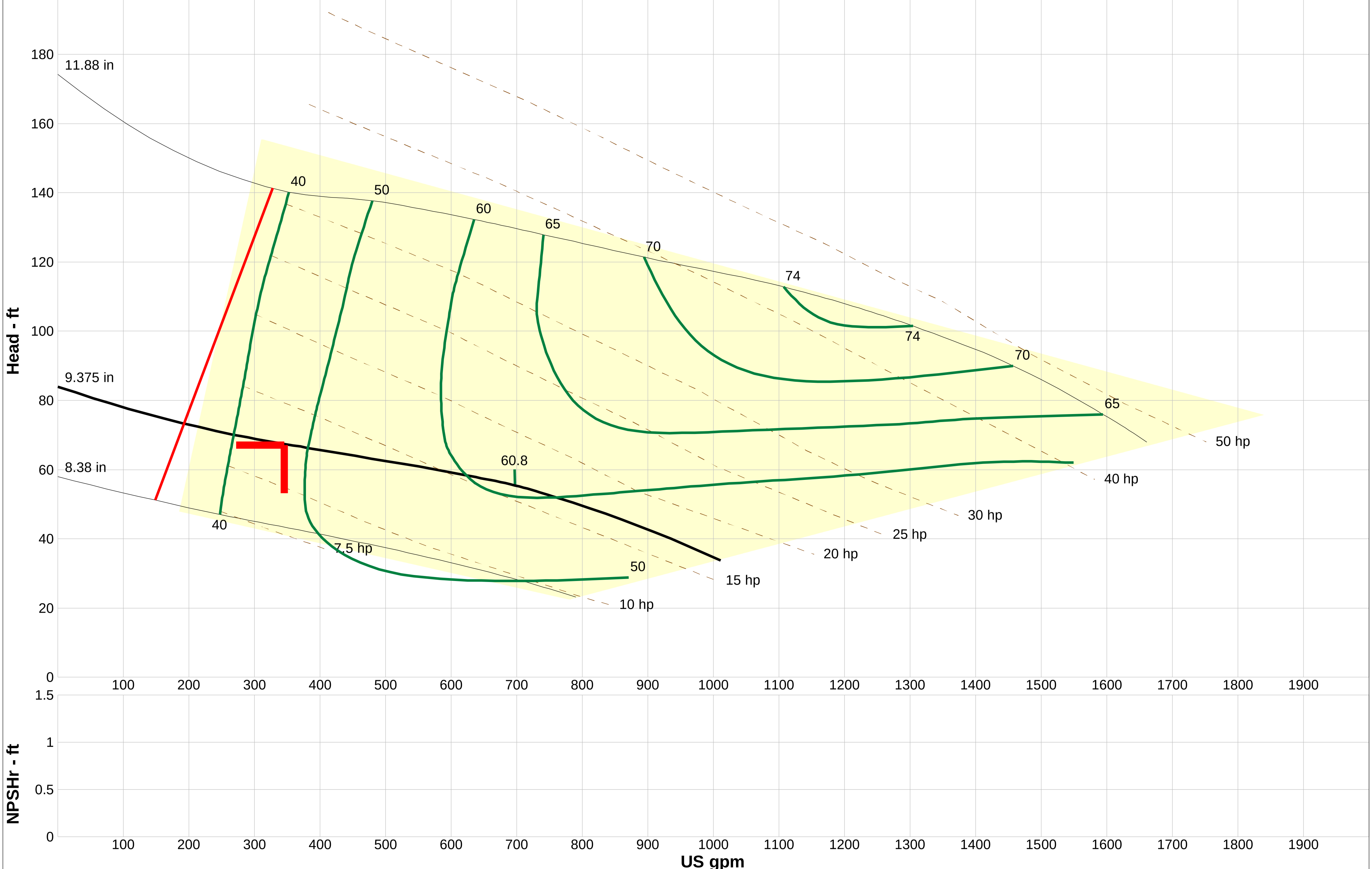
Max power:
47.3 hp @ 1457 US gpm



Performance Evaluation:

Flow US gpm	Speed rpm	Head ft	Efficiency %	Power hp	NPSHr ft
415	1750	65.2	51	13.1	---
346	1750	67.5	47	12.5	---
277	1750	69.8	41	11.9	---
208	1750	73.2	35	11.4	---
138	1750	---	---	---	---

Selected from catalog: sub solids handling.60 Vers: Sep2011



Company: Fuscoe Engineering
Name: James Moser
1/26/2016

HYDROMATIC
Catalog: sub solids handling.60, Vers Sep2011
SUBMERSIBLE SH-4 - 1800
Design Point: 346 US gpm, 67 ft

Size: S4L/S4LX
Speed: 1750 rpm
Dia: 9.375 in
Curve: S4L1750

PENTAIR
HYDROMATIC®

ATTACHMENT D
REFERENCED STANDARDS

PUBLIC UTILITIES DEPARTMENT

PEAKING FACTOR FOR SEWER FLOWS
(Dry Weather)

Ratio of Peak to Average Flow*
Versus Tributary Population

Population	Ratio of Peak to Average Flow	Population	Ratio of Peak to Average Flow
200	4.00	4,800	2.01
500	3.00	5,000	2.00
800	2.75	5,200	1.99
900	2.60	5,500	1.97
1,000	2.50	6,000	1.95
1,100	2.47	6,200	1.94
1,200	2.45	6,400	1.93
1,300	2.43	6,900	1.91
1,400	2.40	7,300	1.90
1,500	2.38	7,500	1.89
1,600	2.36	8,100	1.87
1,700	2.34	8,400	1.86
1,750	2.33	9,100	1.84
1,800	2.32	9,600	1.83
1,850	2.31	10,000	1.82
1,900	2.30	11,500	1.80
2,000	2.29	13,000	1.78
2,150	2.27	14,500	1.76
2,225	2.25	15,000	1.75
2,300	2.24	16,000	1.74
2,375	2.23	16,700	1.73
2,425	2.22	17,400	1.72
2,500	2.21	18,000	1.71
2,600	2.20	18,900	1.70
2,625	2.19	19,800	1.69
2,675	2.18	21,500	1.68
2,775	2.17	22,600	1.67
2,850	2.16	25,000	1.65
3,000	2.14	26,500	1.64
3,100	2.13	28,000	1.63
3,200	2.12	32,000	1.61
3,500	2.10	36,000	1.59
3,600	2.09	38,000	1.58
3,700	2.08	42,000	1.57
3,800	2.07	49,000	1.55
3,900	2.06	54,000	1.54
4,000	2.05	60,000	1.53
4,200	2.04	70,000	1.52
4,400	2.03	90,000	1.51
4,600	2.02	100,000+	1.50

***Based on formula:** **Peak Factor = $6.2945 \times (pop)^{-0.1342}$**
(Holmes & Narver, 1960)

FIGURE 1-1

b. Where an existing main is replaced by a City contract and it terminates in a dead end street, such as in Subsection 2.5.12, the main shall be extended and standard laterals shall be constructed perpendicular to the main.

2.3.2 Distance between Manholes

The maximum distance between manholes shall not be greater than those shown in Table 2-4.

TABLE 2-4
DISTANCE BETWEEN MANHOLES

Sewer Size (Inches)	Maximum Distance Between Manholes (Feet)
8 - 15	400
18 and over	800

2.3.3 Design of Manhole Shelves

a. **Shelf Width:** The width of the shelf in manholes shall be of approximately equal size on either side of the main channel and a minimum distance of 18 inches from the edge of the channel to the manhole wall. The shelf provides a working platform for sewer maintenance personnel (see SDS-106 and SDS-107). In manholes with changes in direction of sewer flow or a side inlet, the outboard shelf may be reduced to 12 inches (moving the pipe out of the centerline of the manhole) where a long transition is needed to maintain laminar flow or to reduce the standing wave. In no case will the manhole size be reduced. The manhole may be increased in diameter or a vault may be used to increase the flow curve radius.

b. **Outboard Shelf (Standing Wave)** Manhole bases that accommodate a change in direction of flow shall be designed with sufficient freeboard on the "outboard shelf" to keep the entire flow cross-section within the manhole channel without spillage onto the "outboard shelf". The shelf elevations shall be shown on the plans and shall be of equal height on both sides of the channel. The following formula shall be used to determine the minimum required shelf elevations:

Therefore: $$\Delta D = \frac{V^2 B}{gr} + 0.25$$

$$D_{sw} = D_N + \Delta D$$

(Reference: Brater and King, "Handbook of Hydraulics")

Where:

$\Delta D =$ Depth of outboard water surface crest above normal depth at peak flow (feet)
$V =$ Velocity (feet per second)
$B =$ Width (feet) of water surface (horizontal projection)
$g =$ Gravitational constant (32.2 feet/second^2)
$r =$ Radius of bend (feet)
$D_{sw} =$ Depth of water surface above invert (standing wave)* (feet)
$D_N =$ Normal depth at peak flow (feet)

$$D_{sw} \text{ max.} = \frac{3}{4} d$$

$d =$ Pipe diameter (feet)

2.3.4 Manhole Frames and Covers

Manhole frames and covers shall be non-rocking and shall conform to the requirements of ASTM A48, Class 30. Unless otherwise indicated, manhole frames and covers shall be heavy-duty cast-iron type with a 36-inch opening. Manhole cover inserts shall be 24-inch diameter with lettering "CITY OF SAN DIEGO" and "SEWER" similar to what is indicated on SDRSD M-1.

2.3.5 Manhole Lining and Grouting

2.3.5.1 Bases

Bases shall be coated for all conditions listed in Subsection 2.3.5.3. The manhole base shall be primed with epoxy and lined with a 100-mil dry film thickness (DFT) of 100 percent solids elastomeric polyurethane with a minimum, Shore D, hardness of 55 in accordance with *SSPWC Section 500-2.4 ─ Air-Placed Concrete and Polyurethane Protective Lining Manhole Rehabilitation*, or other methods and materials included in the City's Approved Materials List for municipal sewer applications. The lining shall be continuous, without seams, and free from any defects, holes, or surface irregularities. The CONTRACTOR shall furnish a minimum of two plugs per manhole to permit verification of the applied thickness.

2.3.5.2 Riser Joints

Polymer mortar shall be used for riser joints on manholes to create water-tight joints to prevent or minimize infiltration.

* Determination of the location of the standing wave is not required since the shelf is horizontal.

2.3.5.3 **Risers**

Manhole risers in the wastewater collection system shall be epoxy-grouted and lined with PVC, (or T-Lock, or other methods and materials included in the City's Approved Materials List for municipal sewer applications) in any of the following cases (See Figure 2-3 for typical application.):

a. Manholes for all trunk sewers 18-inch and larger in diameter
b. Manholes in all coastal communities
c. At locations of force main discharge
d. Manholes where high concentrations of hydrogen sulfide exist, (e.g. sealed manholes in canyons, manholes in areas downstream of sewer pump stations, manholes downstream of hydraulic jumps where the sewage is more than 4 hours old from the farthest source, and manholes upstream from siphons)
e. Manholes where groundwater is present

2.3.5.4 **Exterior Walls**

Waterproofing of the exterior walls with a coal tar emulsion (waterproofing agent) shall be required for all manholes in canyons, below the water table, in coastal communities, with base elevations less than mean sea level plus seven (MSL + 7) feet, or in soils with elevated chloride ion (>300 ppm) or sulfate ion (>2,000 ppm) concentrations. The coal tar emulsion shall be applied in no less than two coats for a total dry film thickness of 25 to 35 mils.

2.3.6 **Minimum Invert Drop Across a Manhole**

The invert drop across a manhole or transition structure shall be calculated to provide smooth laminar flow through the manhole and shall not be arbitrarily established.

2.3.6.1 **Manholes With The Same Inlet And Outlet Diameter**

Manholes shall be hydraulically designed to prevent head losses through the manhole such that solids do not fall out of suspension and accumulate in the main downstream of the manhole. Mains of any size that have a peak wet weather flow below the spring-line will not be affected by the flow changing from a "U" channel back into a circular pipe.

a. **Straight-Through Flow for Trunk Sewers with Velocity ≥ 3 fps and All Small Diameter Mains:** For trunk sewers, where the peak design velocity is 3 fps or greater, and for all small diameter mains, the slope of the pipe shall be carried through the manhole, provided the wet-weather flow d/D is 0.4 or less.

b. **Straight-Through Flow for Trunk Sewers with Velocity < 3 fps:** For sewers 18-inch and larger in diameter with peak design velocity less than 3 fps, the invert drop across the manhole shall be equal to the inside diameter (D) of the manhole in feet multiplied by the average slope of the inlet (s_1) or outlet (s_2) sewers. However, a minimum invert drop of 0.1 feet shall be required:

$$\text{Invert Drop} = \left[D \text{ x} \left(\frac{s_1 + s_2}{2} \right) \right] \geq 0.1 \text{ft}$$

c. **Side Inlet for All Pipe Sizes:** The invert drop across the manhole shall be the inside diameter (D) of the manhole in feet multiplied by the average slope of the side inlet (s_1) and outlet (s_2) sewers, plus 0.10 feet of additional drop. However, a minimum invert drop of 0.20 feet shall be required:

$$\text{Invert Drop} = \left[D \text{ x} \left(\frac{s_1 + s_2}{2} \right) + 0.1 \right] \geq 0.2 \text{ft}$$

d. **Changes in Direction:** Same requirements as for side inlets unless the standing wave is below the spring-line in peak wet weather flow conditions, in which case the slope of the pipe shall be carried through the manhole. Also see Subsection 2.2.2.2 for more information.

2.3.6.2 **Outlet Pipe Larger Than Inlet** (See Figure 2-5)

When the outlet pipe is larger than the inlet pipe, the same calculations as shown above in Subsection 2.3.6.1(a) or (b) shall be used, and the drop shown in Table 2-5 shall be added to the result of each calculation.

Table 2-5 is based on matched water surface profile from the upstream pipe to the downstream pipe when flowing ½ full. The Table may be used in lieu of calculating invert drops across a manhole due to the change in pipe diameter. However, if the sewer main is in an area where the slopes are flat, individual invert drops may be calculated based on the planning study design d_n/D at peak flow.

TABLE 2-5
INVERT DROPS ACROSS MANHOLES

Diameter of Outlet (inches)	**Diameter of Inlet (inches) Inlet Drop to be added (feet)**		
	8	10	12
10	0.08	-	-
12	0.17	0.08	-
15	0.29	0.21	0.13
18	0.42	0.33	0.25

Outlet pipes smaller in diameter than the inlet pipe shall not be allowed. In lieu of calculating the drop through a manhole, where there are good slopes and proper manhole channelization, the crown of the pipes may be matched.

2.3.7 Maximum Invert Drop across Manhole

Maximum invert drop across a manhole for sewers 15- inch in diameter and smaller shall be 0.60 feet for straight through flow and 1.00 feet for side inlet flow.

2.3.8 Minimum Manhole Size

The minimum manhole base diameter shall be 4 feet per SDS-107, but not less than the pipe diameter plus 3 feet.

2.3.9 Large Diameter Manholes

For sewer mains greater than 36 inches in diameter, special design and structural details for the manholes or vaults shall be shown on the plans. Vaults shall require a minimum of two access manholes. A separate structural permit is required.

2.3.10 Deep Manholes

For sewer mains that exceed 25 feet in depth, vaults shall be provided with a minimum of two access manholes each. Calculations shall be provided to show that the vault structure is designed to accommodate the design depth. A separate structural permit is required.

2.3.11 Inspection of Existing Manholes

Removal of existing City manhole covers by unauthorized personnel is not permitted as potentially lethal, poisonous, or explosive gases may be present. If access to any existing City manhole is necessary for design or construction purposes, please contact the Public Utilities Department, Wastewater Collection Division at (858) 654-4154.

2.3.12 Raising Manhole Covers

The maximum depth of rings above the cone per SDRSD SDS-106 and SDS-107 is 18 inches. If there is a fill placed near an existing manhole, the distance above the cone can be increased to a maximum of 18 inches. Greater fill shall require that the cone be removed and the riser extended as needed to conform to the standard drawing.

2.4 PIPE BEDDING

2.4.1 Normal Bedding Requirements

Normal bedding is full rock encasement. All sewers, including laterals with normal cover, shall be adequately bedded according to City of San Diego Drawings SDS-100 and SDS-110(C). The induced trench method of construction in which the trench is excavated in compacted fill and refilled with loose compressible materials shall not be allowed.

2.4.2 Special Considerations

Where the possibility exists for erosion, migration, separation, or segregation of sands, silts and clay from the trench wall into the pipe bedding or where the sewer pipe is installed below the water table, the rock envelope shall be wrapped with an engineering geotextile fabric.

2.4.3 Load Factors for Clay Pipe

Vitrified clay pipe shall be bedded based on the calculated loads and a safety factor of 1.5. Bedding should be selected based on a load factor of 2.2 for rock encasement and a load factor of 4.5 for concrete encasement.

SECTION 7.2 SUMMARY OF FACILITY CAPACITY AND HYDRAULIC DESIGN CRITERIA

7.2.1 PURPOSE

This Section provides the basic criteria for determining the required facility capacity and hydraulic design requirements of the pump station facility. The DESIGN ENGINEER shall also be responsible for determining the required capacity and design of other facility subsystems not addressed here per normally accepted design practice.

7.2.2 DESIGN CAPACITY CALCULATIONS

7.2.2.1 **Pump Station Design Capacity Calculation:** Sewer pump station pumping capacity shall be calculated as described in Chapter 1, Subsection 1.5.1.

7.2.3 PUMP AND SYSTEM CALCULATIONS

7.2.3.1 **Constant versus Variable Speed Pumps:** Constant speed pumps shall be used where pump station design capacity is less than 3 million gallons per day (mgd) or 2000 gallons per minute (gpm). Variable speed pumps shall be evaluated for use where pump station design capacity is greater than 3 mgd capacity and as directed by the Senior Civil Engineer. Where pump station capacity is 1.5 mgd to 3 mgd, the facility shall similarly be evaluated for variable speed if required by special site conditions and/or inflow conditions.

7.2.3.2 **Variable Speed Pumps: (<u>Special Station Requirement</u>):** Variable speed pumps may be used for pump stations greater than 3 mgd capacity (see above for stations with 1.5 to 3 mgd capacity) as approved by the Senior Civil Engineer. In the preliminary design report for the facility, the DESIGN ENGINEER shall prepare an alternative analysis that calculates the pumping operation/cycling of constant speed versus variable speed pumps to determine if variable speed is the best apparent alternative for the facility. This shall include an evaluation of operation/cycling that will occur during periods of minimum inflow rate vs. periods of maximum inflow rate. The relative life-cycle cost comparison of constant versus variable speed pumps for pumping stations shall include the cost of all structure(s), mechanical and electrical equipment that would be affected by the pump selection. The City shall thereafter direct the DESIGN ENGINEER to incorporate constant or variable speed pumps in its design.

7.2.3.3 **Uniform Sizing and Number of Service and Standby Pumps:** All installed pumps shall generally be of the same size. The minimum number of pumps per station shall be two. In stations with two pumps, each pump shall be capable of pumping the design flow with the second pump acting as a full standby. In stations with more than two pumps, an identical "standby pump" of the same size and capacity as the other service pumps shall be installed.

7.2.3.4 **Calculation of Hydraulic Losses:** Procedures to be used for calculating dynamic losses shall follow those presented in the most current edition of "*Pumping Station Design"* by Garr M. Jones, et al; Butterworths-Heinemann Publishers.

7.2.3.5 **Allowable Pipe Velocities:** In general, the maximum recommended suction pipe velocity is 5 fps. Velocity at the suction bell shall not exceed 3.5 fps. Install a larger suction line than the pump inlet diameter if required to reduce velocity and inlet head losses, in order to provide the required net positive suction head (NPSH) according to the Hydraulic Institute, and prevent cavitations for high flow rate pumps.

The maximum recommended velocity in the station discharge piping is 8 fps.

Refer to Section 7.9 for allowable force main velocities.

Suction and discharge pipe design shall follow Hydraulic Institute recommendations for items not addressed in this Section.

7.2.3.6 **NPSHA Calculation:** Net positive suction head available (NPSHA) shall be calculated for all pumps other than column pumps. NPSHA shall be calculated on the basis of the static suction head in feet of water (pool elevation) in the wet well, minus the elevation of the center of the pump, plus the absolute barometric pressure (in feet) minus the vapor pressure of water (in feet) at 85 deg. F at sea level, minus the calculated losses from the wet well to the pump connection. Pump specifications shall include NPSHA values for all anticipated operating conditions. NPSHA shall always be more than net positive suction head required (NPSHR) by the selected pump(s).

NPSHR shall mean the NPSHR determined in accordance with ANSI/HI 1.6 or 2.6, as applicable for the proposed pump. The DESIGN ENGINEER shall require the Contractor to document the method used to determine NPSHR for the proposed pump in its pump submittal material.

The pump station design and pump selections shall be made such that NPSHA is equal to or exceeds the greater of: NPSHR plus 5.0 feet or 1.35 times NPSHR.

7.2.3.7 **Pump and System Curves:** Calculations and curves shall be developed for each station, as described in the following paragraphs.

7.2.3.7.1 **Calculation of System Curves:** Station system curves shall include static lift and all dynamic losses from the station suction piping to the point of discharge. Dynamic losses and plotted system curves (total dynamic head) shall be calculated on the basis of Hazen and Williams C values of 110, 130 and 140.

7.2.3.7.2 **Selection of Candidate Manufacturer's Pump Curves:** For each of the above calculated C values, select a pump curve from a manufacturers' catalogue that meets the required design operating point(s). Each pump curve shall be accomplished by the same model pump, with only the diameter of the impeller varying (note: refer to comments below on purpose of pump curve plots).

7.2.3.7.3 "**Flat" Pump Curves:** Avoid pumps with "flat" pump curves where a small change in total dynamic head (TDH) will result in a large change in pump flow.

7.2.3.7.4 **Plotted System and Pump Curve Information on Design Drawings:** For each of the C value condition, provide a plot of the calculated system curve and the associated selected pump curve.

7.2.3.7.5 **Multiple Pump Operation Curves:** Where multiple pump operation is designed (i.e. multiple pumps will operate in series or parallel), provide combined pump curves for multiple pump operation required to meet pumping capacity requirements. Should variable speed pumps be selected, pump curve plots over the full range of variable speed pumping, and for multiple variable speed pumps in operation shall be provided.

7.2.3.7.6 **Other Information and Pump Curves:** The plots of the associated system and candidate manufacturers' pump curves required as design submittals under Section 7.2.3.7 shall include the following information: Head versus Q, NPSHR versus Q, Hp versus Q, and efficiency versus Q for the candidate pumps at the required operating speed(s). These curves also shall have the manufacturers' allowable operating regions (ANSI/HI 9.6.3) plotted on them to demonstrate that all specified continuous duty operating points are within the candidate manufacturers' recommended pump operating regions. The selected motor shall be non-overloading throughout the maximum speed curves. The DESIGN ENGINEER shall require the Contractor to submit the information described above and to demonstrate that his proposed pumps meet the same requirements and those described below.

7.2.3.7.7 **Pump Selection:** The selected pump must provide for stable operation at all operating points falling between the boundary conditions established by the worst (i.e., greatest static lift and lowest pipeline C value) and best (i.e., least static lift and highest pipeline C value) set of assumptions used for development of the station system curves. These boundary conditions must be within the limits of the pump manufacturers' allowable operating region (ANSI/HI 9.6.3). The selected pump must also meet the criteria of Subsection 7.1.1.2 - Energy Efficient Designs.

The selected pumps shall operate without damaging cavitation or vibration over the entire design range of flow and head conditions (operating points) including those produced by multiple pump operation and/or variable speed.

Pump NPSHR shall be checked against the NPSHA to assure the pump design requirements of subsection 7.2.3.6 are met.

Unless otherwise noted or specified, pump Head/Q curves shall slope in one continuous curve within the specified operating conditions. No points of reverse slope inflection capable of causing unstable operation will be permitted within the specified zone of continuous duty operation. Pumps with Head/Q curves as described in paragraph 9.6.3.3.12 of ANSI/HI 9.6.3 are specifically prohibited if these characteristics will cause unstable operation within the specified range of operating conditions and where startup/shutdown conditions entail operation against a slow opening/closing valve.

Pumps shall be designed in accordance with applicable portions of ANSI/HI 1.1-1.6, 2.1-2.6, and 9.1-9.6. The pumps shall be specifically designed to pump raw wastewater and shall operate without clogging or fouling caused by material in the pumped fluid at any operating condition within the range of service specified.

7.2.3.7.8 **Design Pump Rating and Requirements:** The specified pump shall be rated to deliver the station design capacity at the worst combination of static head and pipeline C value, and also selected to operate in the manufacturers' Preferred Operating Region (ANSI/HI 9.6.3) at the Head/Q curve intersection with the system curve established by the best combination of static lift and pipeline C value.

The rated condition and all other continuous duty operating conditions specified for full speed operation in the detailed specification section shall fall within the manufacturers' Preferred Operating Region as defined in ANSI/HI 9.6.3. The Preferred Operating Region shall be not less than that specified in paragraph 2.1.12 of API 610. Proposed pumps shall be selected to allow not less than a five percent increase in head, as specified in paragraph 2.1.4 of API 610. Variable speed operation to achieve this objective shall not be considered. Pump selections proposing impeller diameter greater than 90% of the maximum size for the proposed pump model and casing size shall not be accepted.

7.2.3.7.9 **Impeller Information for Plotted System and Pump Curves:** The purpose of providing separate plotting of the above associated system and pump curves on the design drawings is to show that for the above various C values, the candidate manufacturers' pump can be made to operate at the required design points. This will be accomplished by only varying (replacing) the impeller diameter. This is to assure that the pump station pumps can be configured and designed to operate through the "C" value changes that typically occur during the extended service of the facility (i.e., grease coating and corrosion occurring inside the pipe reducing the C factor value over the service life.

7.2.3.7.10 **Specification of Design Pumps:** Based on the above calculations, the candidate manufacturers' design pump to be listed in the project specifications and supplied during construction shall be specified so the installed impeller shall be the correct size to operate with the C = 130 curve. In no case, shall the maximum impeller diameter available for a particular model pump be selected (Ref. Subsection 7.2.3.7.8).

7.2.4 MASS ELASTIC SYSTEMS AND CRITICAL SPEED CALCULATIONS

Each pumping unit, consisting of pump, intermediate shafting, couplings, motor, supports and all attached appurtenances shall have no dangerous critical or resonant frequencies or multiples of resonant frequencies within 20 percent above and 15 percent below the speed (range) required by the pump to meet the indicated operating conditions. A dangerous critical speed shall be defined as one which produces a torsional stress exceeding 3500 psi. The DESIGN ENGINEER shall require the pump manufacturer, through the Contractor, to be responsible for the analysis of critical speeds and the complete mass elastic system, which shall be analyzed and certified by a registered professional engineer regularly engaged in this type of work. Analysis shall be at least equal to the industry standard technique developed by Dunkerly and Holzer.

7.2.5 SURGE PRESSURE CALCULATIONS

7.2.5.1 **Surge Analysis Methodology:** All pumping stations shall be independently evaluated by the DESIGN ENGINEER for the potential for hydraulic transients. Computer programs for transient analysis shall be approved by the City on a case-by-case basis. Current state-of-the-art computer programs for transient analysis, such as LIQT developed by Stoner Associates, Inc., or SURGE 5 developed by the University of Kentucky, or NETWORK-SURGE developed by John List, or other programs approved by the City, shall be used for evaluation of all transient phenomena and proposed control measures.

Each program is unique in terms of its capabilities and must be assessed in each situation to make sure the program can handle the complexities of the analysis involved.

7.2.5.2 **Submittal of Hydraulic Transient Memorandum:** Prior to initiating detailed design of a pumping station, the DESIGN ENGINEER shall submit to the City a Hydraulic Transient Memorandum describing and summarizing the transient analyses preformed including assumptions made, analysis program input and output tables, graphs, figures etc. as necessary. The memorandum shall also contain a narrative description of any potential for hydraulic transients and the steps recommended by the DESIGN ENGINEER for further action or mitigation of the hydraulic transients. Based on the contents of this submittal, the City may direct the DESIGN ENGINEER to design the necessary means for mitigation of hydraulic transients. The memorandum shall be signed and sealed by a registered professional engineer.

7.2.5.3 **Transient Control Measures:** Devices for transient control shall be considered in design, and installed as required to reduce pressure surges with pump starts and stops. Transient control measures to be considered singly or in combination for wastewater systems are limited to the following and listed in the order of preference:

7.2.5.3.1 **Shaft-Mounted Flywheels:** to increase moment of inertia for systems subject to column separation.

7.2.5.3.2 **Force Main Alignment:** revisions to eliminate potential column separation zones.

7.2.5.3.3 **Vacuum Relief Valves and Pressure Release Valves (Combination Type):** Locate at critical locations along the force main to prevent column separation and damaging vacuum conditions following pump shutoff.

7.2.5.3.4 **Slow-Closing, Hydraulically-Operated Pump Discharge Valves:** to control head rise in the pressurized discharge pipelines.

7.2.5.3.5 **Vacuum Relief Valves or Check Valves (Vented from Wet Well):** for entry of air into the line to prevent column separation following pump shutoff.

7.2.5.3.6 **Non-Approved Measures:** Surge tanks are specifically prohibited as water hammer control measures for wastewater pumping systems.

7.2.6 WET WELL CALCULATIONS

7.2.6.1 **Flow Data Table:** Provide a table of flow data on the design drawings for the sewer line discharging into the wet well up to 500 feet upstream, as shown in Table 7.2-1:

TABLE 7.2-1

FLOW DATA

Pipe Section (MH# to MH#)	Peak Q	"N"	d/D

7.2.6.2 **Wet Well Inlet:** The wet well inlet sewer invert shall be above the normal high water operating level. The wet well inlet sewer shall be designed to minimize turbulence and odor generation, with no free fall discharge into the wet well under any operating condition. In addition, the influent pipe shall not discharge directly on top of the suction elbow of a pump. The wet well inlet shall be designed in accordance with *ANSI/HI 9.8 Pump Intake Design Standard for Solid-Bearing Liquids.*

7.2.6.3 **Wet Well Operating Volume:** The wet well operating volume and pump(s) sequencing start/stop call levels shall be configured to meet minimum inflow conditions through peak wet weather inflow conditions. The total wet well operating volume is the volume between the first pump on start level in the wet well to the all pumps on stop level. For periods of very low inflow, the volume to be pumped by the first pump call shall be as small as possible to allow regular pumping down of the wet well volume to prevent septic action from taking place. However, the wet well must be large enough to provide at least 5 minutes pump running time at minimum flow to prevent overheating of the electric motor and controls (refer to minimum operating volume calculation in Subsection 7.2.6.5).

Where variable speed pumps are installed (i.e. to provide the required variation in pumping rate for minimum inflow through peak wet weather inflow conditions), the pump(s) start/stop call levels in the wet well shall be configured to satisfy the above requirements over the entire range of design pumping rates and pump sequencing.

7.2.6.4 **Minimum Inflow Calculation:** In the sizing of a pump station wet well, determination of minimum flow is also important to control cycling of constant speed pumps. Wet wells should be large enough to provide at least 5 minutes of pump running time to prevent overheating of the motor, but not too large in order to prevent septic conditions in the wet well. Table 7.2-2 shall be used to determine minimum flow (note: typically 20% to 30% of the average daily flow dependent on population and flow (Source: WPCF Manual of Practice No. 9). No reference to Table 7.2-2

TABLE 7.2-2

RATIO OF MINIMUM TO AVERAGE FLOW

Average Flow (mgd)	Minimum Flow Factor
Less than 1	0.2
2	0.24
3	0.26
4	0.27
5	0.28
7	0.30
10	0.32

7.2.6.5 **First Pump Call Level in the Wet Well Operating Volume:** The minimum wet well operating volume (i.e. first pump call operating volume based on start and stop levels) shall be equal to the following (Ref. Subsection 1.3.2.2):

First Pump Call Wet Well Operating Volume = [(Pump Station Design Capacity) – ($Q_{\text{Minimum Inflow}}$)] x 5 Minutes

Where:

$Q_{\text{Minimum Inflow}}$ = (Average Dry Weather Flow) x (Minimum Flow Factor, per Table 7.2-2)

7.2.6.6 **Wet Well Operating and Alarm Levels:** The wet well low and high operating water levels and alarm levels shall be indicated on the design drawings. The pump automatic shut-off level shall be located above the pump volute level to ensure sufficient net positive suction head per Section 7.2.3.6. Minimum submergence of the pump suction bells (this defines the low flow level) shall be not less than that determined in accordance with Section 9.8.7 of the *Hydraulic Institute Pump Intake Design Standard*. The automatic low level shut-off feature shall be inoperable during cleaning cycles of self-cleaning trench type wet wells.

7.2.6.7 **Emergency Storage Volume:** Separate from the wet well operating volume, the DESIGN ENGINEER shall provide an emergency storage volume sufficient to accommodate storage of a two-hour inflow at peak wet weather flow. The total pump station sewage storage volume (i.e., volume of the wet well above the station HIGH WATER ALARM to the lowest sewage spill point) can be accomplished by the following measures singly or in combination, and listed in order of preference: additional storage in the wet well above the operating volume, separate overflow tank and storage in the inlet line to the spill level.

This "emergency repair holding time" will allow operating personnel at least two (2) hours to respond to a station failure alarm and/or to shut off all pumps to perform emergency repairs to correct a failure condition. In addition, this storage is also available to be utilized for flow equalization during large storm events should peak wet weather inflow exceed the pump station design capacity.

7.2.6.8 **Influent Line Storage:** The wet well influent sewer shall not be designed to accommodate storage except as required for “emergency repair holding time” as described in Section 7.2.6.7 (note: this causes grease buildup problems in the inlet line). This storage shall be utilized where it is not practical to provide two-hour emergency storage in the wet well and/or a separate overflow storage tank.

7.2.6.9 **Spill Location Indication:** Influent sewer and pump station spill locations shall be indicated on the design drawings (lowest upstream elevation or wet well cover elevation where backup spill will occur). Mean sea level (MSL) elevation shall be included for information for spill location.

7.2.7 SIX-HOUR EMERGENCY STORAGE (<u>SPECIAL STATION REQUIREMENT</u>)

7.2.7.1 **Closed Tanks:** In areas where protection from spillage must be provided, the size of the pump station emergency storage capacity will be determined by the Wastewater Collection Senior Civil Engineer. For example, in areas where maximum protection from a sewer spill would enter a potable water supply reservoir, six-hour emergency overflow storage (at peak wet weather inflow rate) shall be required. This storage requirement is in addition to the wet well operational storage. The emergency storage can be an underground structure or a separate tank that is normally empty but can drain by gravity back into the wet well.

In environmentally sensitive or public contact areas such as at a beach, the Wastewater Collection Senior Civil Engineer shall determine the size of the emergency storage capacity.