DEXTER WILSON ENGINEERING, INC.

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CONSULTING ENGINEERS

WATER SERVICE ANALYSIS FOR THE EMERALD HILLS PROJECT IN THE CITY OF SAN DIEGO

November 5, 2024

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WATER SERVICE ANALYSIS
FOR THE EMERALD HILLS PROJECT
IN THE CITY OF SAN DIEGO

November 5, 2024

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11-5-2024

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Job No. 688-034

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November 5, 2024 688-034

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2280 Wardlow Circle, Suite 100
Corona, CA 92880

Attention: Dan Boyd, Vice President - Entitlements

Subject: Water Service Analysis for the Emerald Hills Project in the City of San Diego

Introduction

This report provides a public water study and fire flow analysis for the Emerald Hills project in the City of San Diego. The project proposes 123 single-family residential units. The 31.2 gross acre project site is located in the Emerald Hills neighborhood of the City located on the west side of 60th Street between Tooley Street and Old Memory Lane. The project site previously consisted of a radio tower and will be redeveloped with the proposed residential project use.

Topography of the buildable portion of the site slopes to a high point in the middle and northeast portion. The site will be designed to connect to existing public water lines in 60th Street and Old Memory Lane. Figure 1 provides a Vicinity Map for the project.

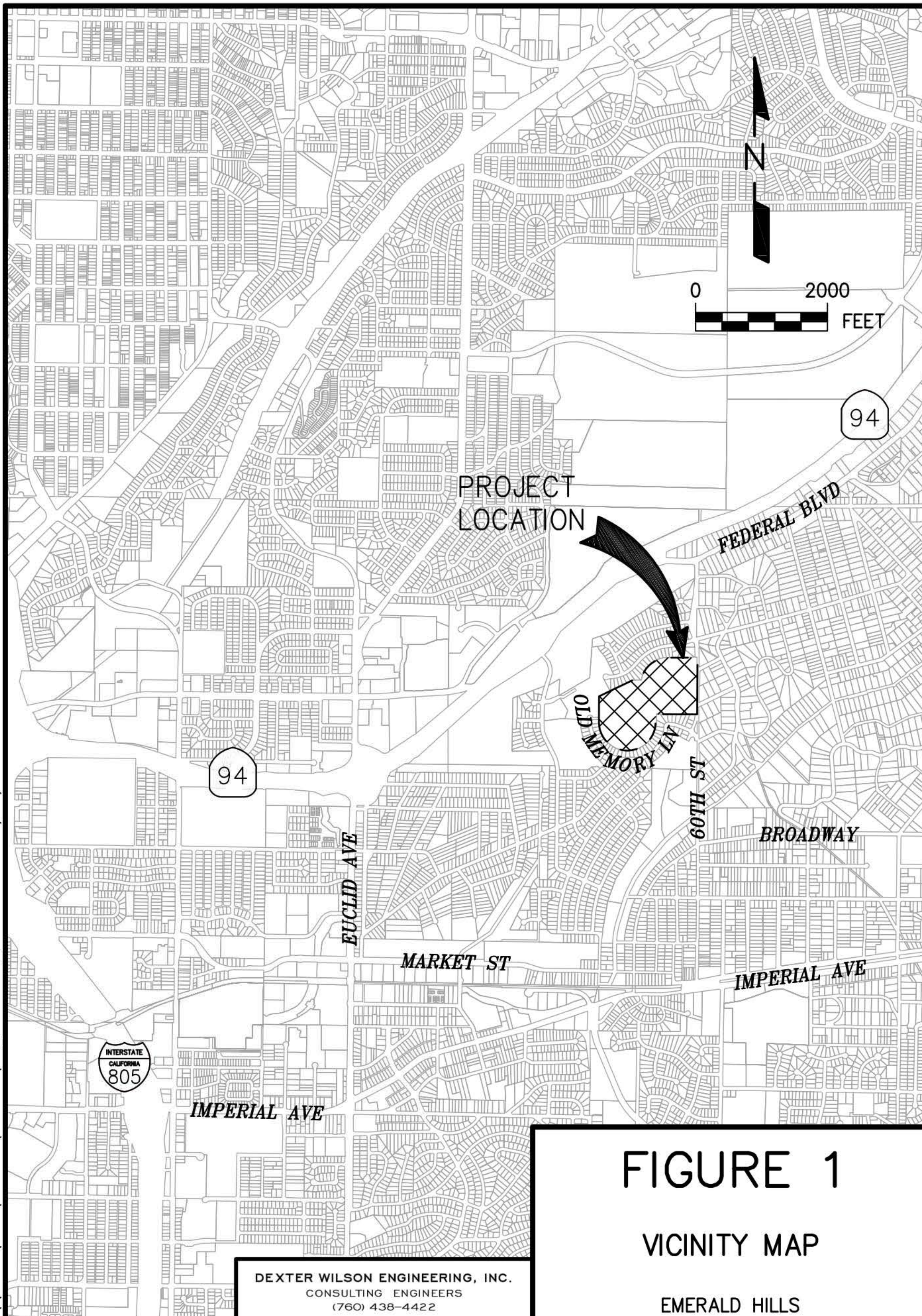

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FIGURE 1

VICINITY MAP

EMERALD HILLS

\\ARTIC\DWG\688034\REPORT\EH_FIGURE-1_VICMAP.DWG 11/13/2023 3:42:27 PM LAYOUT:8x11 USER:Donald

Purpose of Study

The purpose of this study is to confirm the adequacy of the existing and proposed public water system and provide recommended water system improvements for the Emerald Hills project if needed. This report will verify that the any public improvements comply with the City of San Diego Water Department water system design standards. The onsite water facilities for the project are proposed to be public.

Study Area

The study area for this report is the boundary of the Emerald Hills project. The extent of the existing water system which was incorporated into the analysis of the project site was based on the existing Paradise Mesa 610 Zone distribution system that serves the area. The adjacent water mains were assessed by a hydrant flow test performed by the City in the area of the project. This was performed to ensure that the dynamics of the existing water system were analyzed sufficiently without modeling the entire pressure zone.

Figure 2 presents the existing and proposed water facilities in the project area as well as the location of the hydrant flow test. Detailed information regarding the hydrant flow test is included in Appendix A.

Emerald Hills Project Water Demands

The water demands were developed in accordance with the City of San Diego Design Guidelines and Standards. Residential water demand is estimated based on density and a unit water demand of 150 gpd/person. The project proposes 123 residential units over 31.2 gross-acres. A gross acreage of 31.2 acres equates to 25.0 net-acres which equals a net-density for Emerald Hills of 4.9 units per acre. Table 2-1 in the City of San Diego Design Guidelines and Standards, attached as Appendix B, indicates that 4.9 units per acre falls in the range of 3.5 persons per dwelling unit (equivalent to RS-1 zoning). A dwelling unit density of 3.5 persons per dwelling unit and a unit water demand of 150 gpd/person results in a water demand rate of 525 gpd per dwelling unit at the project.

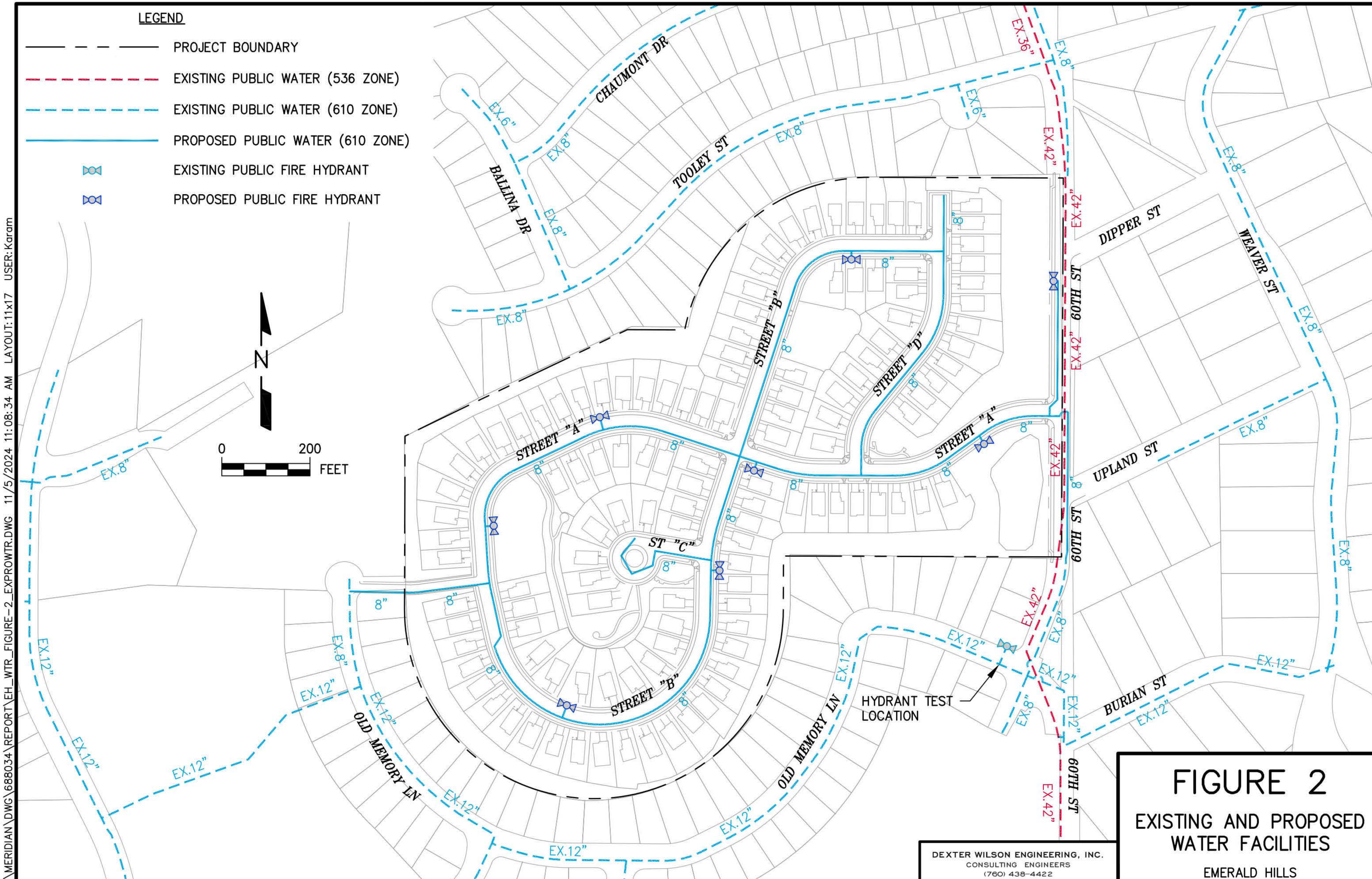

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FIGURE 2

EXISTING AND PROPOSED WATER FACILITIES

EMERALD HILLS

Table 1 presents the projected potable water demand for the project.

TABLE 1 EMERALD HILLS PROJECT POTABLE WATER DEMAND			
Land Use	**Quantity**	**Demand Factor**	**Average Water Use, gpd**
Single-Family Residential (4.9 DUs/net acre)	123 Units	525 gpd/DU	64,575
TOTAL			**64,575 = 45 gpm**

From the City of San Diego Guidelines and Standards, Figure 2-2, the maximum day demand to average annual demand ratio is approximately 1.7 based on the RS residential peaking curve, resulting in an estimated maximum day demand in the pressure zone of 109,778 gpd (76 gpm).

From the City of San Diego Guidelines and Standards, Figure 2-1, the peak hour demand to average annual demand ratio is approximately 2.5 x 1.5 based on the RS residential peaking curve, resulting in an estimated peak hour demand of 242,156 gpd (168 gpm).

Appendix B of this report presents the reference and backup data for determining these peaking factors.

As stated previously, the existing site has been developed as a radio tower and this land use will be replaced with the proposed residential project. Since there was not previous water demand for the radio tower the estimated water demand shown in Table 1 will all be new water use for the project site.

City of San Diego Design Criteria

Book 2 of the City of San Diego Guidelines and Standards was used to analyze the existing water system.

A summary of the design criteria from Book 2 is presented as Table 2.

**TABLE 2
CITY OF SAN DIEGO
WATER SYSTEM DESIGN CRITERIA**

Criteria	Design Requirement
Minimum Static Pressure	65 psi
Maximum Static Pressure	120 psi
Maximum Pressure Drop	25 psi
Minimum Pressure – Peak Hour	40 psi
Minimum Pressure – Max Day plus Fire	20 psi
Maximum Pipeline Velocity (Fire Flow)[1]	15 fps
Maximum Pipeline Velocity (Normal Operating Conditions)[2]	5 fps

[1] Section 3.3.1 E
[2] Section 3.10.1

Static Pressures

Maximum static pressures within the project are calculated based on the Paradise Mesa 610 Water Service Pressure Zone. Pad elevations onsite range from 346 feet to 390 feet. Using the maximum potential hydraulic gradeline of 610 feet, maximum static pressures within the project will range between 95 psi and 114 psi.

Existing and Proposed Water System

There are existing public water facilities directly adjacent to the project site. The existing facilities are part of the Paradise Mesa 610 Zone and Alvarado 536 Zone. Figure 2 presents the existing and proposed water system.

610 Zone. There is an existing 610 Zone 8-inch diameter public water line in Old Memory Lane adjacent to the project. There is also an existing 610 Zone 8-inch diameter public water line in 60^{th} Street which stubs at the northeast and southeast corners of the project site. The project will need to construct an 8-inch water line in 60^{th} Street to interconnect the dead-end stub from the southeast corner up to a new hydrant in 60^{th} St. The project will be connecting its onsite water systems at these 610 Zone locations in Old Memory Lane and 60^{th} Street.

536 Zone. There is an existing 536 Zone 42-inch transmission water line (Otay 2^{nd} Pipeline) in 60^{th} Street adjacent to the project site. The project will not be making any connections to this pipeline. As part of the street improvements to 60^{th} Street there will be a reduction in minimum cover at select locations. The minimum cover over the existing Otay 2^{nd} Pipeline will be reduced from 4.8 feet to 3.8 feet.

City criteria for transmission water mains (i.e., 16-inches in diameter and larger) require a minimum of 5 feet of cover to the top of the pipe. Less cover may be acceptable when supported with engineering calculations and approved by the Public Utilities Department. Calculations are provided in Appendix C which confirms that pipe deflection under the proposed condition will be acceptable (i.e. ~1%).

Onsite Water System. All onsite public water lines are proposed as 8-inch diameter.

Fire Flow

The fire flow requirement for the project site was estimated based on City Design Criteria. The City Design Criteria states a fire flow requirement of 1,500 gpm for single family residential thus this water study will assume a fire flow requirement of 1,500 gpm for the proposed project.

Water System Computer Model and Fire Flow Analysis

Appendix A presents the hydrant flow test data performed by the City. The available pressure under a fire flow requirement of 1,500 gpm was extrapolated from the hydrant flow tests through the utilization of an additional spreadsheet calculation. That spreadsheet calculation is included in Appendix A as well. Results from the hydrant flow test show a residual pressure of approximately 105 psi at the 1,500 gpm fire flow requirement for the project at the hydrant flow test location (60th Street and Old Memory Lane intersection (adjacent to southeast corner of project).

Model Development. Analysis using the KYPIPE computer software program developed by the University of Kentucky determined residual pressures throughout the fire protection system. This computer software utilizes the Hazen-Williams equation for determining headloss in pipes. The Hazen-Williams "C" value used for all pipe sizes in our analysis is 120.

Appendix D presents the computer modeling results and Exhibit A presents the corresponding Node and Pipe Diagram. The fire flow requirement of 1,500 gpm was modeled at several locations within the project site. A pipe break scenario was also modeled within the public water system.

Under normal operating conditions (all pipes open) the fire flow requirement of 1,500 gpm is being met with a minimum residual pressure of greater than 68 psi and a maximum pipeline velocity of 9.6 feet per second (fps) in the existing public water system

Under pipe break conditions, the fire flow requirement of 1,500 gpm is being met with minimum residual pressures of greater than 49 psi and a maximum pipeline velocity of 10.1 fps in the existing public water system.

The results of the computer hydraulic analyses for the Emerald Hills project indicate that the existing public water system can provide sufficient flow and pressure for the Emerald Hills's projects' domestic and fire protection service needs.

Conclusions and Recommendations

The following conclusions and recommendations are summarized based on the water system and fire flow analysis prepared for the proposed Emerald Hills project.

1. The project will be supplied from the Paradise Mesa 610 Zone system.

2. Maximum static pressure within the residential project will range between 95 psi and 114 psi.

3. Figure 2 presents the existing and proposed water system surrounding the project.

4. A maximum day demand plus 1,500 gpm fire flow scenario can be met at the project site with existing piping with all residual pressures greater than the 20 psi requirement. The hydrant flow test performed by the City in the project vicinity, and headloss in the distribution piping around the project, show residual pressures of approximately 68 psi under a 1,500 gpm fire flow condition within the project site.

5. Offsite water system improvements are needed for the Emerald Hills project in order to provide adequate connectivity to the site. The project will need to construct an 8-inch water line in 60th Street to interconnect the dead-end stub from the southeast corner of the project site frontage up to a new hydrant in 60th St.

6. Peak Hour Demand is met at a residual pressure of 91 psi.

7. The recommended material specification for all new potable water lines is AWWA C900 PVC DR18 Class 235.

8. If any water lines to be constructed by this development are metallic, a California Licensed Corrosion Engineer will be required to perform a soil corrosivity study and to design a Corrosion Control System.

If you have any questions regarding the information or conclusions and recommendations presented in this report, please do not hesitate to contact the undersigned.

Dexter Wilson Engineering, Inc.

Steven Henderson, P.E.

SH:ah

Attachments

APPENDIX A

CITY OF SAN DIEGO
HYDRANT FLOW TEST INFORMATION

City of San Diego
Development Services
Attention: **Hydrant Flow Request**
1222 First Ave., MS-401
San Diego, CA 92101
(619) 446-5000

Hydrant Flow Request

FORM
DS-160
OCTOBER 2016

Fill out the information below completely for all sprinkler system flow requests, including NFPA 13, 13D and 13R systems. E-mail form to: DSDHydrantFlow@sandiego.gov, or mail request to the above address.

Please print or type legibly.

Company Requesting Hydrant Flow:
Dexter Wilson Engineering, Inc.

Telephone No:	Fax No:	E-mail Address:
760-438-4422	760-438-0173	steven@dwilsoneng.com

Project Number for the Building Permits:
N/A

Location of Hydrants:
Southwest Corner of Old Memory Lane and 60th Street

Cross Street:	City:	State:	ZIP Code:
Old Memory Lane/60th Street	San Diego	CA	92114

FOR CITY USE ONLY

Facility Sequence Number: (FSN): H538938

Static: 116.9 PSI
Elevation: 334' FEET

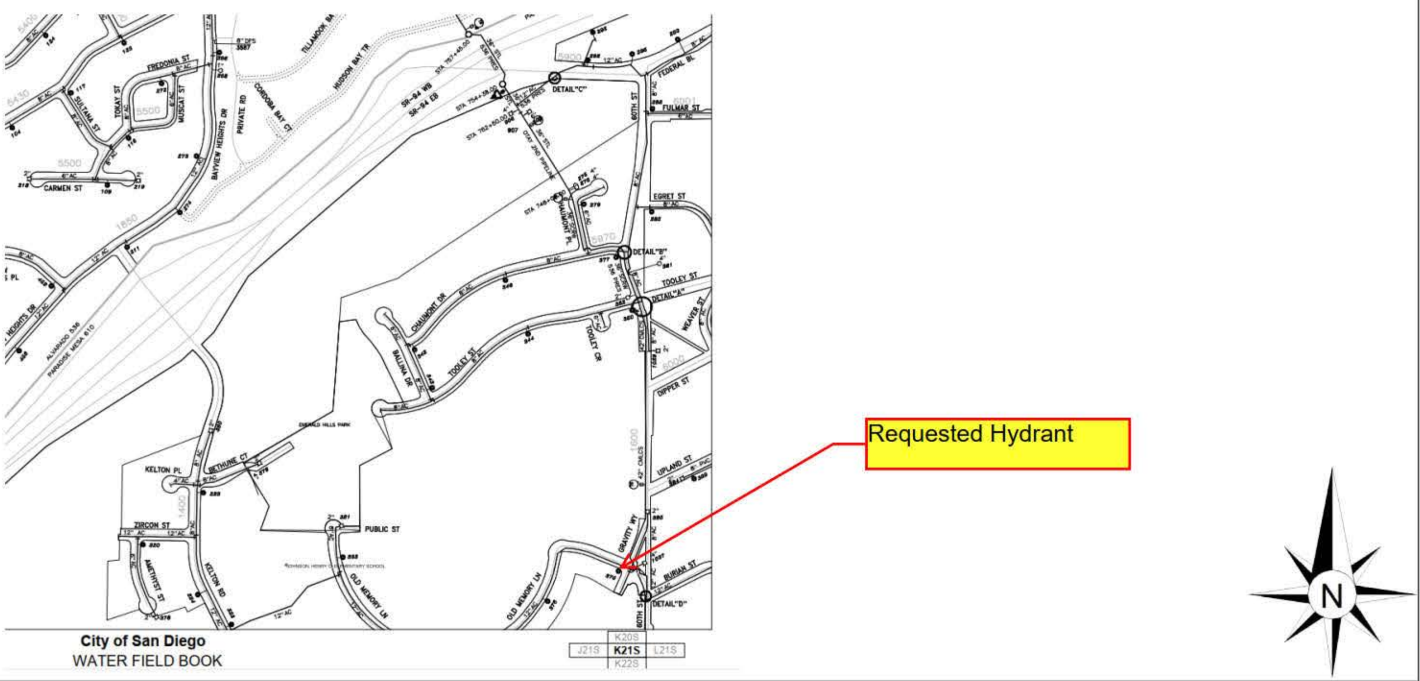
Pitot: model PSI
Residual: 102.7 PSI

Date: 10/11/2022
Flow: 1652 GPM

Researched in database by: O. Paraiso

The information provided above is based upon a water model. It is the contractor's responsibility to confirm the available static pressure at the system point of connection. If a discrepancy is noticed at that time, notify DSDHydrantFlow@sandiego.gov as soon as possible.

Please draw an accurate map for fire hydrant data



Fire Hydrant Flow Test Date 10/11/2022
Fire Hydrant Flow Test Location 60th Street and Old Memory Lane

Input Flow Test Results

Static Pressure	116.9 PSI		
Residual Pressure	102.7 PSI		
Hydrant Flow	1652 GPM		
Actual Hydrant Elevation	334 Feet	HGL	603.8 Feet
Estimated Hydrant Elevation	334 Feet	HGL	603.8 Feet

Equation $\Delta H = k\,Q^{1.85}$

k = 3.64927E-05

Extrapolated Calculations

Q, gpm	Residual Pressure	Available HGL
500	115.3 psi	600.2 ft
750	113.6 psi	596.2 ft
1000	111.3 psi	590.9 ft
1250	108.4 psi	584.2 ft
1500	105.0 psi	576.4 ft
2000	96.7 psi	557.1 ft
2500	86.3 psi	533.3 ft
3000	74.1 psi	505.0 ft
3500	59.9 psi	472.4 ft
4000	44.0 psi	435.5 ft
4500	26.2 psi	394.6 ft
5000	6.7 psi	349.5 ft
5500	-14.5 psi	300.5 ft

Residual Pressure, psi	Available Flow, gpm
0 psi	5,163
10 psi	4,919
20 psi	4,665
30 psi	4,398
40 psi	4,117
50 psi	3,818
60 psi	3,498
70 psi	3,151
80 psi	2,768
90 psi	2,333
100 psi	1,815
110 psi	1,118
120 psi	Residual Pressure Exceeds Static Pressure

APPENDIX B

CITY OF SAN DIEGO
DESIGN CRITERIA

Chapter
2

WATER DEMANDS AND SERVICE CRITERIA

2.1 General

This chapter outlines planning procedures to estimate water demands and fire flows. Water system service requirements are also defined in terms of water pressure and reservoir storage.

2.2 Service Area

The DESIGN CONSULTANT defines the project's service area and identifies the pressure zones in which it is located. The Senior Civil Engineer in charge of either Water Planning or new development approves the service area boundaries.

2.3 Land Use and Residential Population

The DESIGN CONSULTANT develops present and future land use maps for the service area to define the following land use categories: residential (by zone in accordance with **Table 2-1**), central business district, commercial and institutional, parks, hospitals, hotels, industrial, office, and schools.

The DESIGN CONSULTANT estimates the residential population in the service area based on present and future allowable land use. Unless more accurate population density estimates are available, the residential population in the service area is estimated based on the figures presented in **Table 2-1**.

Table 2-1
Residential Population Density

Zone	Dwelling Unit Density (dwelling unit/ net acre)	Unit Density (persons/ dwelling unit)	Population Density (persons/ net acre)
AR-1-1	0.1	3.5	0.4
AR-1-1	0.2	3.5	0.7
AR-1-2	1	3.5	3.5
RS-1-1/RS-1-8	1	3.5	3.5
RS-1-2/RS-1-9	2	3.5	7.0
RS-1-4/RS-1-11	4	3.5	14

Zone	Dwelling Unit Density (dwelling unit/ net acre)	Unit Density (persons/ dwelling unit)	Population Density (persons/ net acre)
RS-1-7/RS-1-14	9	3.5	32
RM-1-1	14	3.2	45
RM-2-5	29	3.0	87
RM-3-7	43	2.6	112
RM-3-9	73	2.2	161
RM-4-10	109	1.8	196
RM-4-11	218	1.5	327

Dwelling unit density in **Table 2-1** is based on net area. The net area is measured in acres, and is 80% of the gross area for each residential zone.

2.4 Average Annual Water Demands

For most projects, average annual water demands are determined based on the unit water demand criteria presented in **Table 2-2**.

Table 2-2
Unit Water Demands

Land Use Category	Unit Water Demand
Residential	150 gallons/person-day
Central Business District	6000 gallons/net acre-day
Commercial and Institutional	5000 gallons/net acre-day
Fully Landscaped Park	4000 gallons/net acre-day
Hospitals	22500 gallons/net acre-day
Hotels	6555 gallons/net acre-day
Industrial	6250 gallons/net acre-day
Office	5730 gallons/net acre-day
Schools	4680 gallons/net acre-day

Average annual water demands are calculated as the sum of: (1) the residential water demand, and (2) other water demands for each land use category as follows:

Residential Water Demand (gallons/day) = Residential Population x 150 gallons/person-day

Other Water Demand (gallons/day) = Land Use Area by Category (net acres) x Unit Water Demand for Each Land Use Category (gallons/net acre-day)

Average Annual Water Demand (gallons/day) = Residential Water Demand + Other Water Demands

On some projects, particularly large residential developments, using the unit water demands in **Table 2-2** may generate unrealistically high estimates of water requirements. For these large projects, the DESIGN CONSULTANT or developer may request that the Senior Civil Engineer consider an alternative approach, making use of the City's water demand distribution data developed for macroscale planning purposes. Similarly, the Senior Civil Engineer may also consider alternative unit water demand estimates for specific land use types where such estimates are based on detailed demand evaluations. Recent projects of similar size, nearby location and similar character may be used for comparative demand analysis.

2.5 Peak Water Demands

Unless the project involves a large development that calls for an alternative approach, peak hour and maximum day water demands are estimated using the peaking factors presented in **Figures 2-1 and 2-2**. Peaking day factors correspond to the zones identified in the Public Utilities Department Water System HGL Zones.

Peak water demands are estimated as follows:

Peak Hour Demand = Average Annual Water Demand * Peak Day Factor * 1.5

Maximum Day Demand = Average Annual Water Demand * Peak Day Factor

Figure 2-1 Peak-Hour Demand Factor by Development Type and Size

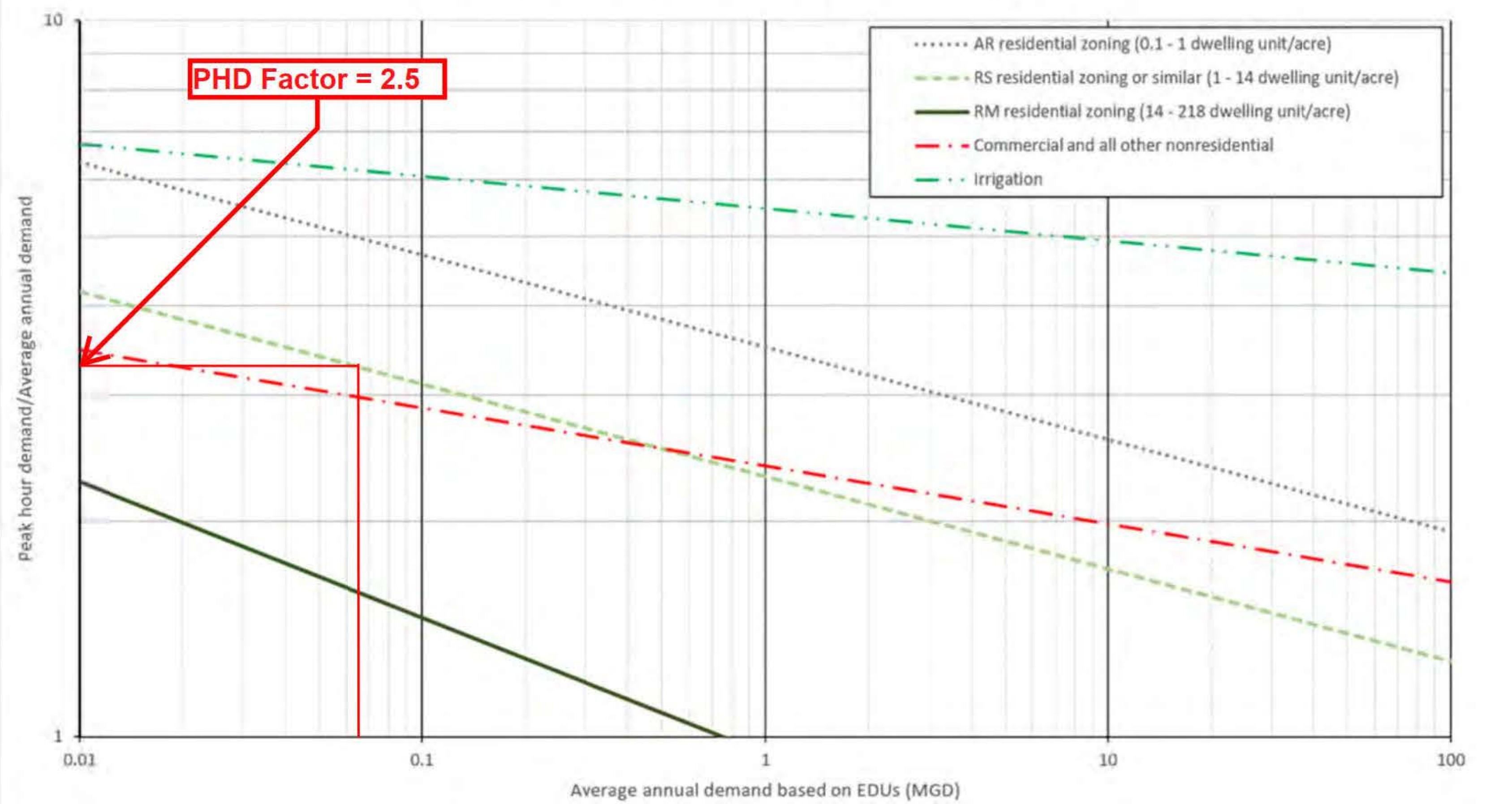
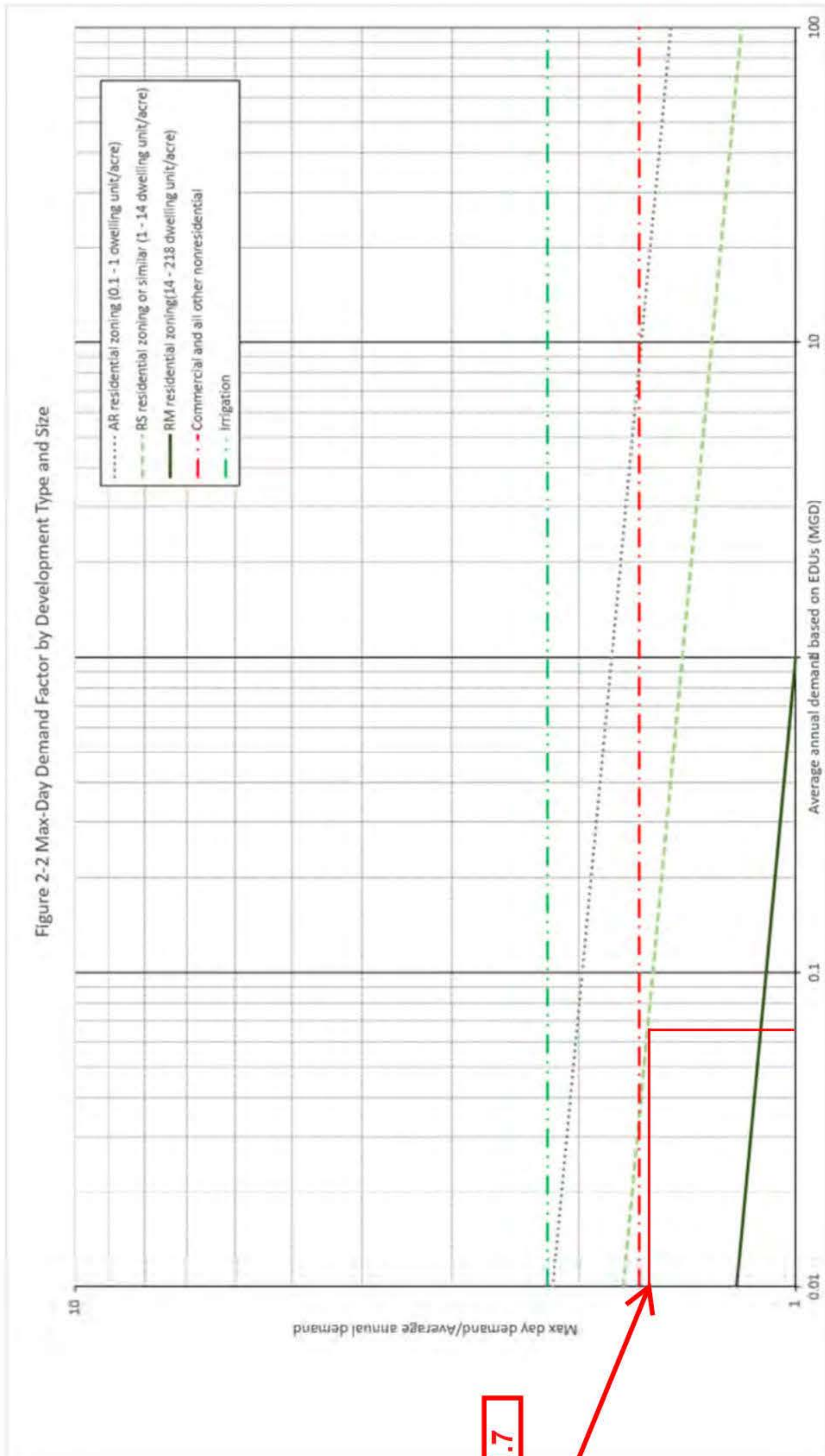


Figure 2-2 Max-Day Demand Factor by Development Type and Size



2.6 Fire Demands

The DESIGN CONSULTANT shall use the minimum required fire demands for design shown in **Table 2-3**. The fire flow duration for planning purposes is at least five hours. Note that the values in **Table 2-3** are the minimum design criteria for public infrastructure. Privately owned facilities shall follow the guidelines described in Appendix B of the California Fire Code (CFC).

Table 2-3
Fire Demands for Design Purposes

Development Type	Fire Demand (gpm)
Single family residential up to Fourplexes	1,500
Condominiums and apartments	3,000
Commercial	4,000
Industrial	6,000

Should application of the CFC Appendix B result in figures lower than those shown in **Table 2-3**, the firm or Civil Engineer, in consultation with the fire department, CIP City Project Manager may approve the CFC figures on a case-by-case basis following submittal of supporting calculations. In no case shall the approved fire flow rate and flow duration be less than the flow rate and duration values required by Appendix B of the CFC based on the anticipated or proposed type of building construction and total building floor area.

The required fire demand must be supplied from public and private on-site fire hydrants located as required by CFC Appendix C.

2.7 Pressure Criteria

2.7.1 Design Pressures

Water systems must be designed to provide the minimum residual pressures under:

- Maximum day demands plus fire demand conditions, or
- Peak hour demand conditions.

In analyzing the supply to a pressure zone, the minimum hydraulic grade line elevation available from the water source is used, a level that typically occurs during dry weather conditions. A water supply source is defined as a treatment plant clearwell, flow control facility, pump station, pressure regulating station or reservoir. Supply sources occur at discrete points in a system of water mains and control both flow and pressure at the supply point. Water mains are not supply sources but rather conveyance facilities. The maximum static pressure in gravity systems is determined from reservoir overflow elevations and/or the discharge control setting on pressure reducing valves, whichever is greater. The maximum static pressure in pumped systems is determined from reservoir overflow elevations or pump shutoff levels, whichever is greater. There are two important pressure criteria used in water system design: Domestic Pressure and Fire Pressure. For systems supplying only domestic demand, only the Domestic Pressure criteria will apply. Similarly, for systems providing only fire demand, only the Fire Pressure criteria will apply. Systems supplying both types of demand, both criteria will apply and must be independently checked.

2.7.2 Domestic Pressure Criteria

The domestic pressure criteria for water system design are shown in **Figure 2-3**. Every water main in each pressure zone must be capable of supplying a minimum static pressure of 65 psi. Domestic pressures must fall no more than 25 psi below the static pressure, and residual water main pressure must be at least 40 psi. Domestic pressures are determined in the distribution system pipelines, excluding losses through service connections and building plumbing, and are measured relative to adjacent building pad elevations.

When analyzing a system with one source of supply out of service, domestic pressures may fall more than 25 psi below static pressure, but the domestic pressure shall not fall below 40 psi.

2.7.3 Pressure Requirements During Fires

For the simulation of fire conditions, a minimum operating pressure of 20 psi is required at the fire hydrant locations.. The residual pressure is determined given the fire demand among one or more hydrants and with the simultaneous water consumption occurring at the maximum day demand. The hydrants considered in this simulation must be sufficiently near to the fire location to be classified as “available” to that location as defined by the California Fire Code.

For water systems with available storage, the residual pressures in the distribution system during a fire are maintained given the following conditions:

- The water level in the storage facility at the time of the fire is at or near the minimum operating level
- The prescribed fire duration set by the California Fire Code, occurring under maximum day conditions.

2.8 System Reliability

Water systems must be designed to meet the operating pressure criteria with one critical source out of service. Water mains must be designed so that no more than one, average-sized city block (approximately 30 homes) is out of service at any time, and no more than two fire hydrants (excluding fire services) are on a dead end or are out of service at any time. These provisions do not apply under earthquake conditions.

Water mains serving more than two hydrants or more than 30 homes must be looped, fed from two sources, or provided with a reservoir of sufficient capacity to supply the emergency needs (contingency and fire storage) as described below in **subsection 2.9**.

All water mains relied upon for looping and source redundancy shall be in separate streets. Dual mains in the same street or alignment require the DESIGN ENGINEER to prepare a request for deviation using the format of ATTACHMENT 1, which is included as a part of this document. Where dual mains are relied upon for looping or source redundancy, the mains shall be spaced at least 10 feet apart from outer edge to outer edge.

For City CIP work in already-built-out areas, where looping of mains or connection to two sources of supply is not feasible, water mains may be constructed require the DESIGN ENGINEER to prepare a request for deviation using the format of ATTACHMENT 1, which is included as a part of this document. Additional design considerations shall be made to minimize the chance of pipe breakage, such as use of a higher class of pipe.

to be considered include the primary fault offset, and the zone in which secondary fault offset might occur. An acceptable level of fault-offset displacement is that based on a 16% chance of exceedance at the particular pipe location, given that a characteristic earthquake on the fault occurs. Lacking site-specific data, it may be reasonable to design a pipe to accommodate a fault offset of the Rose Canyon fault for 5 feet of right-lateral movement, accompanied by 0.5 feet of vertical movement.

Pipelines that traverse the liquefaction zones should either be relocated, or if this is not feasible, designed in accordance with **Section 8.4**.

Utilities and pipelines penetrating basement walls are especially vulnerable. Measures should be taken to provide adequate flexibility and / or ductility at such penetrations. The potential for settlement at a site should be investigated by the geotechnical engineer concurrently with the investigation for liquefaction.

3.8 Bedding, Backfill, Cover and Surface Restoration

3.8.1 Bedding

When unstable soils are encountered and over-excavation is required, foundation stabilization material must be specified for use at the base of the trench. The pipe zone is considered to include the full width of the excavated trench from the bottom of the trench to a point at least 12 inches above the top outside surface of the barrel of the pipe. The minimum depth of bedding material beneath the pipe should be selected based on the expected unevenness of the trench bottom and the diameter of the pipe. Typically 6 inches is adequate; however, the DESIGN CONSULTANT selects the appropriate thickness to be used. Particular attention should be given to the area of the pipe zone from the invert to the centerline of the pipe to ensure that firm support is obtained to prevent any pipe lateral movement during final backfill of the pipe zone. Using controlled low strength material (CLSM) in this area can mitigate this concern. Hand tamping with approved tamping bars, supplemented by compacting with mechanical tamping equipment, is allowed.

3.8.2 Trench Backfill

The DESIGN CONSULTANT shall review the trench backfill provisions in the in the latest standard specifications for Public Works Construction "GREENBOOK" and the WHITEBOOK – Standard Specifications for the City of San Diego.

3.8.3 Cover

The standard depth of cover on water distribution mains, up to and including 16-inches in diameter, is 3 to 5 feet. Transmission water mains (i.e., 16-inches in diameter and larger) require a minimum of 5 feet of cover to the top of the pipe. Less cover may be acceptable when supported with engineering calculations and approved by the Public Utilities Department. The maximum depth of cover for distribution and transmission mains is 8 feet. Less than 3 feet or more than 8 feet of cover require loading, deflection, and safety calculations. When supported by calculations, special design must be submitted to the Public Utilities Department for approval.

3.8.4 Surface Restoration

The DESIGN CONSULTANT refers to City of San Diego standard drawings. Surface restoration is required to match or exceed existing conditions according to the requirements of each agency with jurisdiction in the project area.

3.9 Pipe Design

All pipes are designed by the DESIGN CONSULTANT. Steel pipe specials and fittings may be designed by the pipe manufacturer.

3.9.1 External Loads

External loads on a pipeline include dead loads (the weight of the soil and any improvements constructed above the pipe), live loads (caused by construction traffic and/or vehicular traffic traveling above the pipe), vacuum pressures, and pressures from groundwater.

3.9.1.1 Dead Loads

Dead loads attributable to the weight of the backfill are computed in accordance with AWWA standards as per specific pipe material using the compacted soil weight determined in the geotechnical investigation. Dead loads caused by improvements constructed near or above the pipeline are computed by standard geotechnical engineering practices.

3.9.1.2 Live Loads

Live loads caused by standard highway loadings (HS-20) or railroad loadings (E-80) are computed in accordance with applicable AWWA standards. Live loads from heavy construction vehicles should be analyzed. Construction loads may be present when the pipeline has no or only minimal cover.

3.9.1.3 Vacuum Pressure and Groundwater

The DESIGN CONSULTANT computes the allowable internal vacuum pressure using the formulas described in AWWA standards for buried pipeline. For non-buried pipeline (e.g., above grade crossings or pipelines in casings without control density fill or similar situations), use the applicable AWWA standards. If the calculations indicate that the backfilled pipe could be collapsed by a full vacuum, the DESIGN CONSULTANT must notify the City Project Manager in writing. The pipe

APPENDIX C

PIPE DEFLECTION CALCULATIONS FOR
42-INCH OTAY 2ND PIPELINE IN 60TH STREET

60th Street – Load Calculations for Existing 42" Steel Pipeline

Dead Load Calculation

$W_c = wH_c(D_c/12)$ AWWA M11 Equation 5-3

Where,

W_c = prism load = dead load on conduit, lb/ft of pipe

w = unit weight of fill, lb/ft^3

H_c = height of fill above top of pipe, ft

D_c = outside diameter of pipe, in.

So W_c = 120 x 3.8 x (42/12) = **1,596 lb/ft**

Live Load Calculation

$W_l = D_cL$

Where,

W_l = Live load on conduit, lb/ft

D_c = outside diameter of pipe, ft

L = HS-20 Loading per AWWA Table 5-1 (600 lb/ft^2 @ 3 ft. cover)

So W_l = (42/12) x 600 = **2,100 lb/ft**

Total Load Calculation

Total Load, W = $W_c + W_l$

W = 1,596 + 2,100 = **3,696 lb/ft**

60th Street Existing 42" Steel Pipeline - Deflection Calculations

$$x = D_l \frac{k\, W\, r^3}{EI + 0.061\, E'\, r^3}$$ AWWA M11 Equation 5-4

Where,

x = Horizontal delection, in.
D_l = detection lag factor = 1.0
K = bedding constant = 0.1
W = load per unit length, lb/in
r = conduit radius, in.
EI = wall stiffness
E' = modulus of soil reaction, lb/in^2 (from AWWA M11 Table 5-3)

Calculate E I

E_{steel} = 30,000,000 psi
$E_{concrete}$ = 4,000,000 psi
I = t^3/12, t = conduit thickness, in.

t_{steel} = 0.25 in.
t_{cement} = 0.75 in.

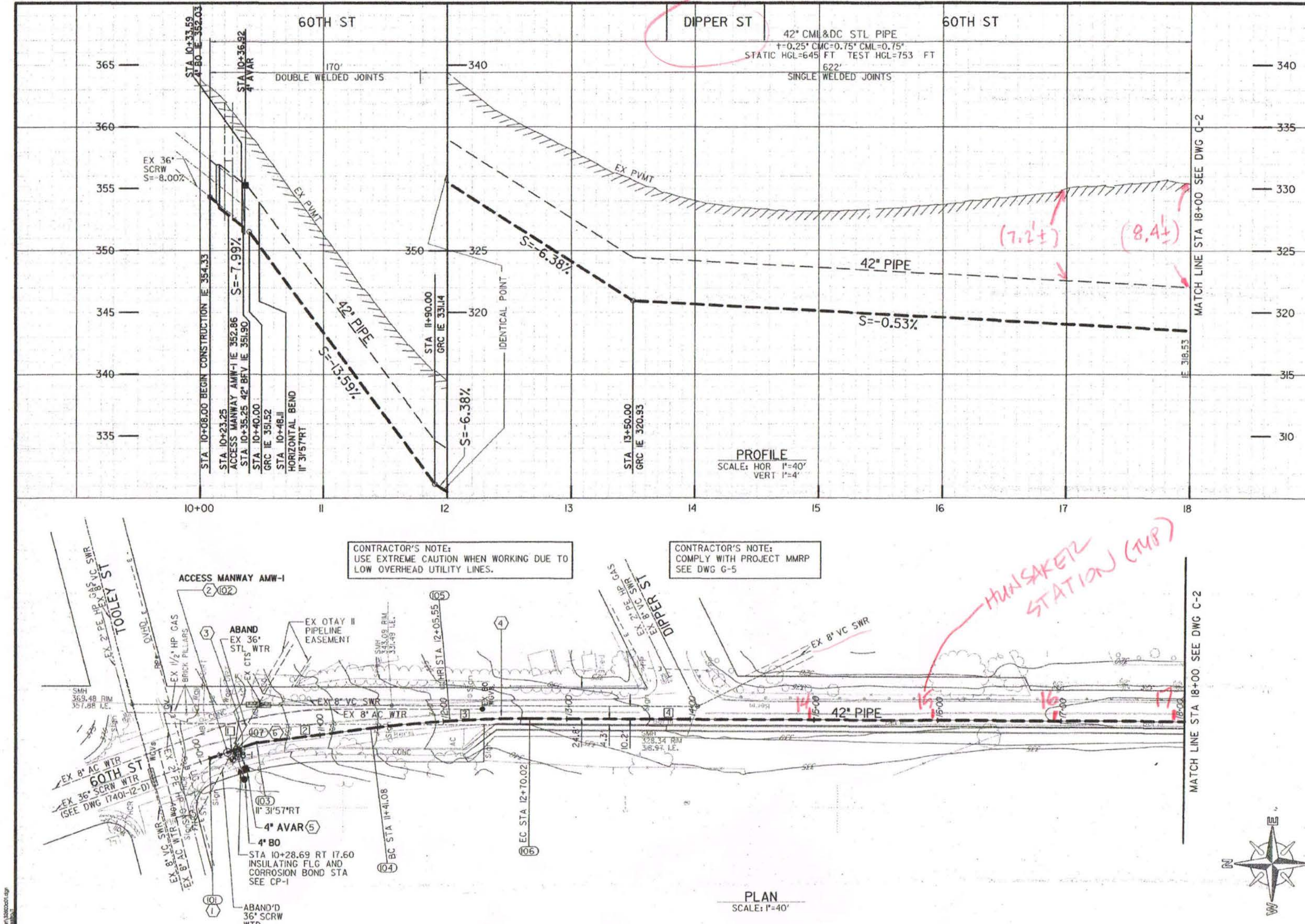
EI_{steel} = 30,000,000 x $(0.4375)^3$/12 = 39,063 lb in
EI_{cement} = 4,000,000 x $(.75)^3$/12 = 140,625 lb in
EI_{total} = 179,688 lb in

Calculated Deflection

W = 3696 lb/ft = 309 lb/in
42" CML & DC Steel Pipe - OD = 44.0", r = 22.0 in.
E' = 750 lb/in^2

$$x = 1.0 \frac{(0.1)(309)(22.0)^3}{179{,}688 + (0.061)(750)(22.0)^3}$$

x = 0.49 in.
% x = (0.49/44.0) X 100 = **1.12% (<2.0% OK)**



COORDINATE TABLE			
NO.	NORTHING	EASTING	DESCRIPTION
101	1844108.09	6310039.58	STA 10+08.00
102	1844093.65	6310044.46	STA 10+23.25
103	1844070.09	6310052.44	STA 10+48.11
104	1843977.86	6310064.04	STA 11+41.08
105	1843913.80	6310072.10	STA 12+05.55
106	1843849.24	6310071.86	STA 12+70.02
107	1844086.07	6310047.03	STA 10+35.25

PIPE DATA TABLE				
NO.	Δ/BEARING	RADIUS	LENGTH	REMARKS
1	S18° 42'09"E	-	40.11'	42" STL CML&DC
2	S07° 10'12"E	-	92.97'	42" STL CML&DC
3	07° 23'16"RT	1000.00'	128.94'	42" STL CML&DC
4	S00° 13'03"W	-	529.98'	42" STL CML&DC

(1) BY CONTRACTOR
FURNISH & INSTALL
STA 10+08.00
TEMPORARY DISCONNECT/PLUG EX 36" MAIN AT EX 36" FLANGE. AFTER ACCEPTANCE OF NEW 42" MAIN RECONNECT TO EX 36" MAIN W/ 4' CLOSURE PIPE
SEE DET. 2 / C-10

(2) BY CONTRACTOR
FURNISH & INSTALL
STA 10+23.25
1 - 36" ACCESS MANWAY
SEE DET. 1 / C-14

(3) BY CONTRACTOR
FURNISH & INSTALL
STA 10+32.25 4.5'L
CUT AND PLUG
EX 36" STL WTR
ABANDON EX 36" STL WTR
SEE DWG G-3

(4) BY CONTRACTOR
FURNISH & INSTALL
STA 10+08 TO STA 18+00
REMOVE AND DISPOSE INTERFERING PORTIONS OF EX 36" STL PIPE AND VALVE.
CONSTRUCT NEW 42" STL PIPE.
TEST AND DISINFECT NEW 42" MAIN.

(5) BY CONTRACTOR
FURNISH & INSTALL
STA 10+35.98
1 - 4" AVAR

(6) BY CONTRACTOR
FURNISH & INSTALL
STA 10+35.25
1 - 42" BFV W/4" GV BYPAS
SEE DET 2 / C-12

FIELD NOTES:

WATER:	17401-3-D, 17401-4-D, 17401-12-D
SEWER:	6330-AD, 6346 -D
STORM DRAIN:	
GAS:	36-424
ELECTRIC:	202-1746
CABLE TV:	198-746
TELEPHONE:	182054
IMPROVEMENTS:	
WATER FIELD BOOK:	K215
SEWER FIELD BOOK:	K215
100' SCALE:	
THOMAS BROS.:	1290-C1, 1290-C2

RETIREMENTS: 5043' 36" STL WTR (C-1 TO C-8) ONE 36" BFV

OTAY II PIPELINE IMPROVEMENTS

CIP NO. 73-286.8 SPEC NO. 2408 C-1

OTAY II PIPELINE IMPROVEMENTS
NORTH ENCANTO REPLACEMENT
60TH ST STA 10+08 TO STA 18+00

CITY OF SAN DIEGO, CALIFORNIA
SHEET 7 OF 73 SHEETS
WATER W.O. 186991

APPROVED Hoa Cir Ajar 8/5/10 SENIOR CIVIL ENGINEER DATE
SUBMITTED BY John Stotts ASSOCIATE ENGINEER
Brandon Tappan ASSISTANT ENGINEER

DESCRIPTION	BY	APPROVED	DATE	FILMED
ORIGINAL				
AS-BUILT	ADC/JDO		4/01/10	

CONTROL CERTIFICATION 202-1746 LAMBERT COORDINATES
32600-07-D

DRAWING STATUS

NO.	DATE	REVISION DESCRIPTION	DWN	CKD	RVD	PE

CONSULTANT
AECOM
7807 CONVOY COURT, SUITE 200
SAN DIEGO, CA 92111
(858)268-8080
15398.01-0001

ENGINEER OF WORK
AMANDA D. COMBS C 67297 Exp. 03/30/09 CIVIL STATE OF CALIFORNIA
A.D.Combs 10-29-07
DATE

DES ADC
DWN JP
CKD NJS

WATER DEPARTMENT
City of San Diego

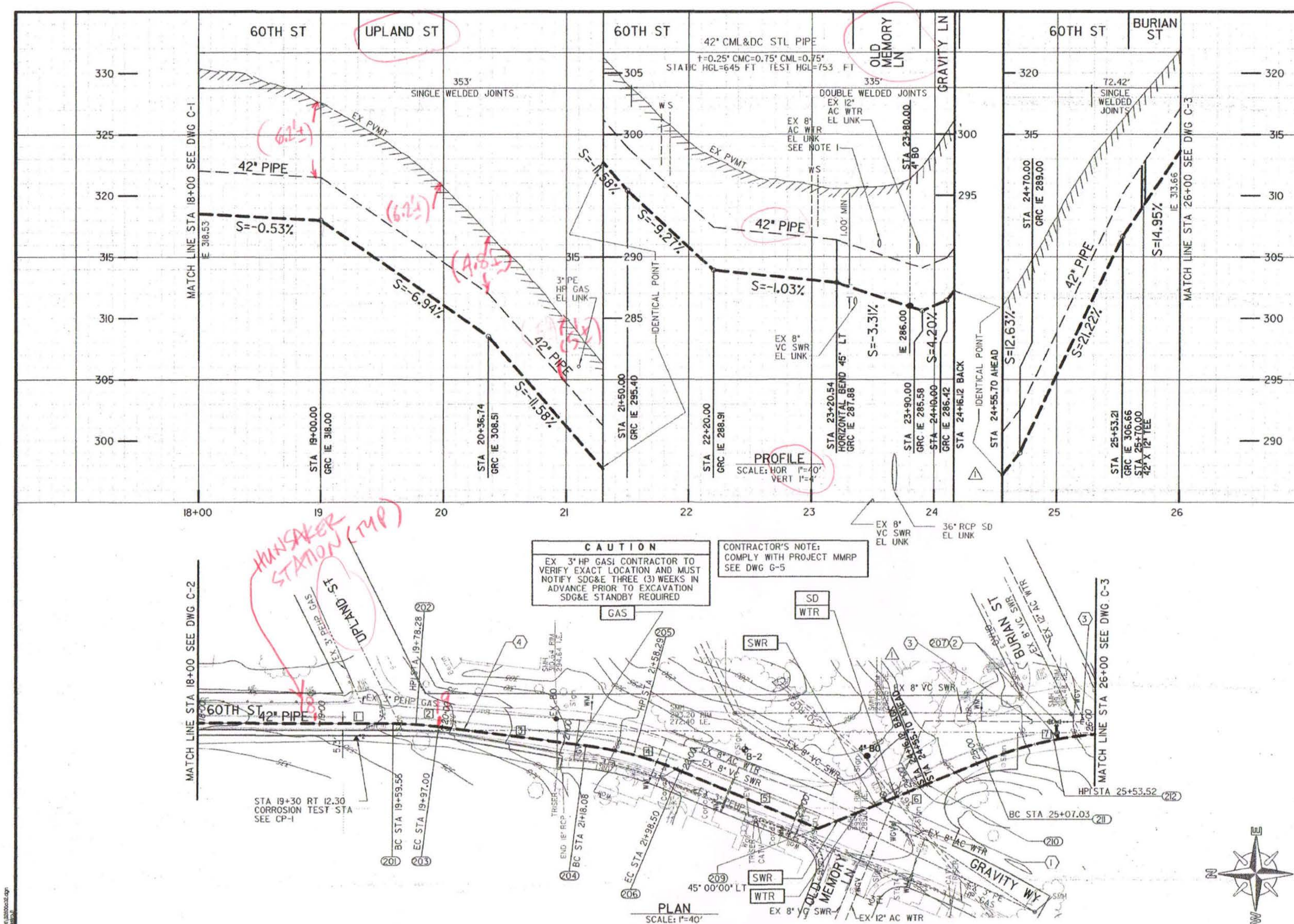
SCALE:
VERT: 1"=4'
HOR: 1"=40'

FOR CITY ENGINEER
C. Wo
AS-BUILT ENGINEER
INSPECTOR
AS-BUILT ONLY

CONTRACTOR T.C. CONSTRUCTION DATE STARTED 6/2/08
INSPECTOR R.E.P. HANNA DATE COMPLETED 12/1/09

AS-BUILT

42" W.L. RECD. 8-17-23



COORDINATE TABLE			
NO.	NORTHING	EASTING	DESCRIPTION
201	1843159.71	6310069.24	STA 19+59.55
202	1843140.97	6310069.17	STA 19+78.28
203	1843122.38	6310066.76	STA 19+97.00
204	1843002.30	6310051.23	STA 21+18.08
205	1842962.18	6310046.05	STA 21+58.29
206	1842924.86	6310030.42	STA 21+98.50
207	1842616.37	6310056.48	STA 25+70.00
209	1842812.30	6309983.27	STA 23+20.54
210	1842723.85	6310019.49	STA 24+16.12/24+55.70
211	1842676.36	6310038.94	STA 25+07.03
212	1842621.14	6310061.56	STA 25+53.52

PIPE DATA TABLE				
NO.	Δ/BEARING	RADIUS	LENGTH	REMARKS
1	S00° 13'03"W	-	159.55'	42" STL CML&DC
2	07° 09'01"RT	300.00'	37.45'	42" STL CML&DC
3	S07° 22'05"W	-	121.08'	42" STL CML&DC
4	15° 21'34"RT	300.00'	80.42'	42" STL CML&DC
5	S22° 43'38"W	-	122.04'	42" STL CML&DC
6	S22° 16'22"E	-	146.91'	42" STL CML&DC
7	17° 45'53"RT	300.00'	92.97'	42" STL CML&DC

(1) BY CONTRACTOR FURNISH & INSTALL
STA 23+80.00
1 - 4" BO

(2) BY CONTRACTOR FURNISH & INSTALL
STA 25+70.00
CONNECT EX 12" MAIN TO NEW 42" PIPELINE
SEE DET. 3 / C-10

(3) BY CITY FORCES AHD OF CONTRACTOR
STA 25+70.00
TEMPORARY DISCONNECT/PLUG EX 12" MAIN (20' LT AND RT OF NEW 12" TEE).
RECONNECT AFTER ACCEPTANCE.

(4) BY CONTRACTOR FURNISH & INSTALL
STA 18+00 TO STA 26+00
CONSTRUCT NEW 42" STL PIPE.
TEST AND DISINFECT NEW 42" MAIN.

NOTE:
1. RELOCATE PER DET. 4/C-10 IF MINIMUM 6-INCH VERTICAL CLEARANCE CANNOT BE ACHIEVED. SEE NOTE 16 ON DWG G-4.

FIELD NOTES:

WATER:	13268-6-D, 15355-2-D, 15355-6-D
SEWER:	6329-D, 6342 -D, 6345-D, 6349-D
STORM DRAIN:	15355-2-D
GAS:	36-408A, 36-418
ELECTRIC:	202-1746
CABLE TV:	198-746
TELEPHONE:	182023
IMPROVEMENTS:	6345-D
WATER FIELD BOOK:	K2IS
SEWER FIELD BOOK:	K2IS
100' SCALE:	
THOMAS BROS.:	1290-C2

RETIREMENTS:

OTAY II PIPELINE IMPROVEMENTS

C-2

CIP NO. 73-286.8 SPEC NO. 2408

OTAY II PIPELINE IMPROVEMENTS
NORTH ENCANTO REPLACEMENT
60TH ST STA 18+00 TO STA 26+00

CITY OF SAN DIEGO, CALIFORNIA
SHEET 8 OF 73 SHEETS

WATER W.O. 186991

APPROVED Hossein Ajideh 8/5/10 SENIOR CIVIL ENGINEER DATE
SUBMITTED John Stohr ASSOCIATE ENGINEER
Brandon Sapper ASSISTANT ENGINEER

DESCRIPTION	BY	APPROVED	DATE	FILMED
ORIGINAL				
AS-BUILT	ADC/JDC		11/10/10	

CONTROL CERTIFICATION 202-1746
LAMBERT COORDINATES
32600-08-D

CONTRACTOR T.C. CONSTRUCTION DATE STARTED 6/2/08
INSPECTOR R.E.R HANNA DATE COMPLETED 12/1/09

DRAWING STATUS

NO.	DATE	REVISION DESCRIPTION	DWN	CKD	RVD	PE
1	09/09	CORRECT AHEAD AND BACK LABELS AT 24+16.12 /24+55.70	ADC	NJS	NJS	ADC

CONSULTANT
AECOM
7807 CONVOY COURT, SUITE 200
SAN DIEGO, CA 92111
(858)268-8080
15358.01-0001

AMANDA D. COMBS C 67287 Exp. 09/30/10 CIVIL STATE OF CALIFORNIA

ENGINEER OF WORK
A.D. Combs 10-29-07
DATE

DES ADC
DWN JP
CKD NJS

WATER DEPARTMENT
City of San Diego

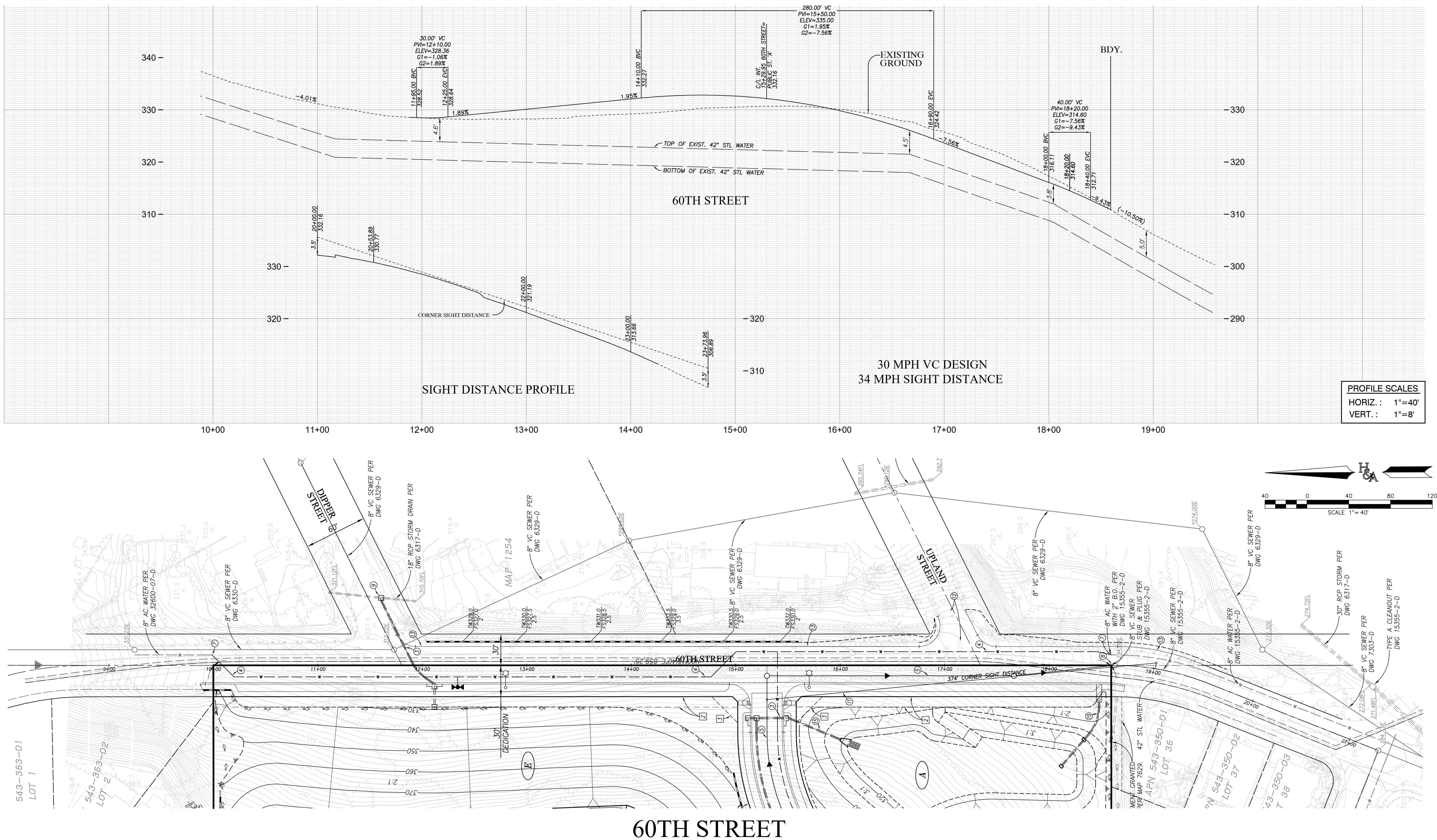
SCALE:
VERT: 1"=4'
HOR: 1"=40'

FOR CITY ENGINEER
ASSOCIATE ENGINEER
INSPECTOR

AS-BUILT ONLY

AS-BUILT

42" W.L. RECD 8-17-23



PREPARED BY:

HUNSAKER & ASSOCIATES
SAN DIEGO, INC.

PLANNING 9707 Waples Street
ENGINEERING San Diego, Ca 92121
SURVEYING PH(858)558-4500 · FX(858)558-1414

60TH STREET PLAN / PROFILE

EMERALD HILLS

CITY OF SAN DIEGO, CALIFORNIA

SHEET C6 OF 16

W.O. 1375-0238

R:\1671\&Pln\Emerald Hills VTM - Sht C6.dwg[]Oct-24-2023:22:24

Chapter 5

External Loads on Buried Pipe

External loads on buried pipe are generally comprised of the weight of the backfill (earth load) combined with the live load including impact. As described in chapter 4, internal pressure is used to determine pipe wall thickness. Analysis of the effects of external loads and the principles of pipe/soil interaction are used to determine an adequate pipe-soil system for the design wall thickness. To ensure satisfactory performance of the pipe, proper bedding and backfilling installation must also be followed to enable pipe deflection to be controlled. The prudent choice of pipe embedment soil type and the backfill compaction level are the most important factors affecting the performance of the steel pipe-soil system.

EARTH LOAD

The first solution to the problem of soil-induced loads on buried pipe was published by Professor Anson Marston at Iowa State University in 1913. The Marston load theory is simply the weight of the backfill reduced by the friction of the sidefill against the undisturbed trench walls.

The Marston trench load for rigid pipe is defined as

$$W_d = C_d w B_d^2 \qquad \text{(Eq 5-1)}$$

That equation was modified by M.G. Spangler circa 1930 for flexible pipe to

$$W_d = C_d w B_d B_c \qquad \text{(Eq 5-2)}$$

Where:

W_d = trench load on conduit, lb/lin ft of pipe
C_d = load coefficient for ditch conduits (Spangler 1951)
w = unit weight of backfill, lb/ft^3
B_d = horizontal width of trench at top of pipe, ft
B_c = outside diameter of pipe, ft, also = $D_c/12$
D_c = outside diameter of coated pipe, in.

Equations 5-1 and 5-2 have also been used to calculate loads on buried pipe in a narrow trench condition. When these equations are used, care must be taken that the trench width as built does not exceed the design trench width B_d.

In the 1960s, significant testing was done at Utah State University, which concluded that due to the variability and imprecision of soil properties along the alignment of the pipeline and the design engineer's inability to represent soil parameters to the same level of precision as those with other engineering materials such as structural steel or concrete, the conservative prism load should be considered for buried flexible pipe. Simply stated, the prism load is the weight of the column of backfill directly over the pipe without considering soil friction or arching effects and, therefore, is the maximum load that can be imposed by the backfill on flexible pipe over time. The prism load is a conservative design approach, as the actual earth load on a flexible pipe lies somewhere between the Marston/Spangler trench load and the more conservative prism load. Over time the soil load may approach the prism load due to consolidation of the soils.

Prism load:

$$W_c = wH_c \frac{D_c}{12} \qquad \text{(Eq 5-3)}$$

Where:

W_c = prism load = dead load on the conduit, lb/lin ft of pipe

w = unit weight of fill, lb/ft^3

H_c = height of fill above top of pipe, ft

D_c = outside diameter of coated pipe, in.

LIVE LOADS

In addition to supporting dead loads created by soil cover, buried pipelines can also be subjected to superimposed loads: concentrated live loads or distributed live loads. Concentrated live loads are generally caused by truck-wheel loads or railway-car loads. Distributed live loads are caused by surcharges, such as piles of material or temporary structures. The effects of live loads on a pipeline depend on the depth of soil cover over the pipe.

Live-load effects (W_L), when applicable, are added to soil load and are generally based on AASHTO HS-20 truckloads or Cooper E-80 railroad loads as indicated in Table 5-1. These values are given in pounds per square foot and include a 50 percent impact factor. There is no appreciable live-load effect for HS-20 loads when the earth cover exceeds 8 ft or for E-80 loads when the earth cover exceeds 30 ft.

CONSTRUCTION LOADS

During construction operations, it may be necessary for heavy construction equipment to travel over an installed pipe. Before heavy construction equipment is permitted to cross over a pipe, earth fill should be placed over the pipe. A generally accepted minimum elevation for the fill is at least 3 ft over the top of the pipe; however, additional analysis of the heavy equipment load may be necessary. The fill should be of sufficient width to prevent possible lateral displacement of the pipe and should be maintained to ensure rutting does not decrease the effective cover over the pipe and to prevent excessive impact loading on the pipe.

Table 5-1 Live-load effect

Highway HS-20 Loading*		Railroad E-80 Loading*	
Height of Cover	Load†	Height of Cover	Load†
(ft)	*(lb/ft²)*	*(ft)*	*(lb/ft²)*
1	1,800	2	3,800
2	800	5	2,400
3	600	8	1,600
4	400	10	1,100
5	250	12	800
6	200	15	600
7	176	20	300
8	100	30	100

Source: ASTM A796 (ASTM International).

* Neglect live load when less than 100 lb/ft^2; use dead load only.

† Other HS and E loads can be calculated by applying a ratio such as 25/20 to HS-20 for HS-25 loading.

EXTREME EXTERNAL LOADING CONDITIONS

An occasional need to calculate extreme external loading conditions may arise. One example is to determine off-highway loading from heavy construction equipment. A convenient method of solution for such load determination using modified Boussinesq equations is presented by Handy and Spangler (2007).

As an example (using the Newmark coefficient in Table 5-2):

Assume:

Live load from a large dump truck:
Weight on one set of dual wheels = 42,300 lb
Tire pattern is 44 in. × 24 in.
Height of cover, $H_c = 2$ ft

Calculation:

Using Figure 5-1 as reference, calculate:
Tire pattern area or load surface area:
(44) (24) = 1,056 in.^{2} = 7.33 ft^2 (for dual wheels)
Surface pressure:
42,300/7.33 = 5,771 lb/ft^2

The distance from the center to the corner of the surface load is located by (see Figure 5-1)

$$A_T = (1/2)\,(44/12) = 1.83 \text{ ft}$$
$$B_T = (1/2)\,(24/12) = 1.0 \text{ ft}$$

The m and n values can then be found by

$$m = A_T/H_c = 1.83/2 = 0.915 \qquad n = B_T/H_c = 1.0/2 = 0.50$$

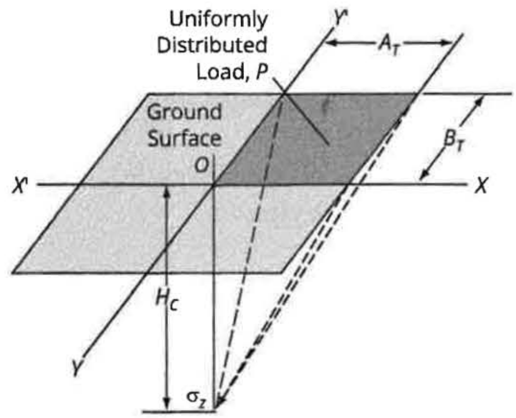
Using m and n, the vertical influence coefficient interpolated from Table 5-2 = 0.117.
The product of A_T and B_T is ¼ of the total tire pattern (see Figure 5-1).
Therefore,

$$W_L = (4)(0.117)(5{,}771) = 2{,}701 \text{ lb/ft}^2$$

Table 5-2 Newmark vertical influence coefficients

	$m = A_T/H_c$ or $n = B_T/H_c$																	
m or *n*	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.5	2.0	2.5	3.0	5.0	10.0	∞
0.1	0.005	0.009	0.013	0.017	0.020	0.022	0.024	0.026	0.027	0.028	0.029	0.030	0.031	0.031	0.032	0.032	0.032	0.032
0.2	0.009	0.018	0.026	0.033	0.039	0.043	0.047	0.050	0.053	0.055	0.057	0.059	0.061	0.062	0.062	0.062	0.062	0.062
0.3	0.013	0.026	0.037	0.047	0.056	0.063	0.069	0.073	0.077	0.079	0.083	0.086	0.089	0.090	0.090	0.090	0.090	0.090
0.4	0.017	0.033	0.047	0.060	0.071	0.080	0.087	0.093	0.098	0.101	0.106	0.110	0.113	0.115	0.115	0.115	0.115	0.115
0.5	0.020	0.039	0.056	0.071	0.084	0.095	0.103	0.110	0.116	0.120	0.126	0.131	0.135	0.137	0.137	0.137	0.137	0.137
0.6	0.022	0.043	0.063	0.080	0.095	0.107	0.117	0.125	0.131	0.136	0.143	0.149	0.153	0.155	0.156	0.156	0.156	0.156
0.7	0.024	0.047	0.069	0.087	0.103	0.117	0.128	0.137	0.144	0.149	0.157	0.164	0.169	0.170	0.171	0.172	0.172	0.172
0.8	0.026	0.050	0.073	0.093	0.110	0.125	0.137	0.146	0.154	0.160	0.168	0.176	0.181	0.183	0.184	0.185	0.185	0.185
0.9	0.027	0.053	0.077	0.098	0.116	0.131	0.144	0.154	0.162	0.168	0.178	0.186	0.192	0.194	0.195	0.196	0.196	0.196
1.0	0.028	0.055	0.079	0.101	0.120	0.136	0.149	0.160	0.168	0.175	0.185	0.193	0.200	0.202	0.203	0.204	0.205	0.205
1.2	0.029	0.057	0.083	0.106	0.126	0.143	0.157	0.168	0.178	0.185	0.196	0.205	0.212	0.215	0.216	0.217	0.218	0.218
1.5	0.030	0.059	0.086	0.110	0.131	0.149	0.164	0.176	0.186	0.193	0.205	0.215	0.223	0.226	0.228	0.229	0.230	0.230
2.0	0.031	0.061	0.089	0.113	0.135	0.153	0.169	0.181	0.192	0.200	0.212	0.223	0.232	0.236	0.238	0.239	0.240	0.240
2.5	0.031	0.062	0.090	0.115	0.137	0.155	0.170	0.183	0.194	0.202	0.215	0.226	0.236	0.240	0.242	0.244	0.244	0.244
3.0	0.032	0.062	0.090	0.115	0.137	0.156	0.171	0.184	0.195	0.203	0.216	0.228	0.238	0.242	0.244	0.246	0.247	0.247
5.0	0.032	0.062	0.090	0.115	0.137	0.156	0.172	0.185	0.196	0.204	0.217	0.229	0.239	0.244	0.246	0.249	0.249	0.249
10.0	0.032	0.062	0.090	0.115	0.137	0.156	0.172	0.185	0.196	0.205	0.218	0.230	0.240	0.244	0.247	0.249	0.250	0.250
∞	0.032	0.062	0.090	0.115	0.137	0.156	0.172	0.185	0.196	0.205	0.218	0.230	0.240	0.244	0.247	0.249	0.250	0.250

Source: Newmark 1935.



Source: Spangler and Handy 1982.

Figure 5-1 Vertical stress under an imposed area load

PREDICTING DEFLECTION

The Iowa deflection formula was first proposed by M.G. Spangler in 1941. It was later modified by Watkins and Spangler in 1958 and has since been presented in various forms. The Iowa formula was developed as a tool to predict horizontal ring deflection of unpressurized pipe buried in soil. The formula is not the basis for design of a pipeline; it is an estimate or *prediction* of long-term horizontal deflection of unpressurized pipe responding to earth and live loads. It is not applicable to pipe that is pressurized. When pressurized, the internal pressure will tend to reround the pipe. A detailed explanation of the rerounding effect can be found in *Structural Mechanics of Buried Pipes* (Watkins and Anderson 2000).

One of the principal variables in the modified Iowa formula is the modulus of soil reaction E', an empirical representation of soil stiffness in the pipe-soil system. The use of E' in the modified Iowa formula is simplistic and has proven to be reliable over time. The formula also shows that ring deflection is controlled primarily by the soil rather than the pipe; therefore, changing the E' has a much greater effect on controlling deflection than does increasing the pipe stiffness.

As an example, when varying E' values and relative pipe cylinder thicknesses independently, the greater significance of increasing E' versus the impact of increasing cylinder thickness is apparent (see appendix A). In most cases, the pipe stiffness only contributes about 1 to 10 percent of the total resistance to deflection, while the soil contributes 90 to 99 percent. In summary, the benefits of improving the quality of the bedding and backfill as the primary means of controlling pipe deflection are much more cost-effective than increasing the thickness of the steel pipe wall.

In one of its most common historical forms, the modified Iowa formula for predicting deflection is

$$\Delta x = D_l \frac{KWr^3}{EI + 0.061E'r^3} \quad \text{(Eq 5-4)}$$

This equation in basic terminology is

$$\text{Predicted Deflection} = \frac{\text{Load}}{\text{Pipe Stiffness} + \text{Soil Stiffness}}$$

Where:

Δx = predicted horizontal deflection of pipe, in.

D_l = 1.0*

K = bedding constant (0.1)

W = load per unit of pipe length, lb/lin in.

= $[W_c/12 + W_L D_c/144]$

D_c = outside diameter of coated pipe, in.

r = mean radius of the pipe, in.

= $(D_c - t - t_L - t_C) / 2$

E' = modulus of soil reaction of the embedment material, psi

Tables 5-3 and 5-5 show values and nomenclature for various embedment materials.

EI = pipe wall stiffness, lb in.

= $E_S I_S + E_C I_L + E_C I_C$

Where:

E = modulus of elasticity [30,000,000 psi for steel (E_S) and 4,000,000 psi for cement mortar (E_C)]

I = transverse moment of inertia per unit length of individual pipe wall components (for steel cylinder (I_S), for cement-mortar lining (I_L), and for cement-mortar coating (I_C))

= $t^3/12$, in.^{3}

t = pipe cylinder wall thickness, in.

t_L = cement-mortar lining thickness, in.

t_C = cement-mortar coating thickness, in.

Under external loads, the individual elements—the mortar lining, the steel cylinder, and the cement-mortar coating—work together as laminated rings ($E_S I_S + E_C I_L + E_C I_C$). The pipe wall stiffness of these individual elements is additive. Structurally, the combined action of these elements increases the moment of inertia of the pipe section above that of the steel cylinder alone, thus increasing its ability to resist external loads.

In the original Iowa equation, the Marston load (W_d) was used, which partially reduces the earth load by frictional shear forces. If the Marston load W_d is used in place of the prism load W_c in the deflection formula, the use of a D_l greater than 1.0 should be evaluated.

Predicted deflection limits for steel pipe installations are as follows:

- 5 percent of D_c for flexible lining and coating such as liquid epoxy linings and coatings, polyurethane linings and coatings, or tape coatings
- 3 percent of D_c for cement-mortar lining and flexible coating
- 2 percent of D_c for cement-mortar coating

These deflection limits are based on the performance limits of the steel pipe with the various coating options.

A buried steel ring is stable up to a uniform elliptical deflection of approximately 20 percent. Cement-mortar lining will withstand deflections of up to 6 to 10 percent. Cement-mortar coating will withstand deflections of up to 3 to 4 percent. Installed deflection conditions that have stabilized but exceed the predicted deflection limits may still be considered acceptable.

* Deflection lag factor (D_l) accounts for long-term deflection as a result of consolidation or settlement of backfill material at the sides of the pipe. Because the steel pipe is designed for the maximum possible soil load directly over the outside diameter of the pipe when the prism load (W_c) is used, D_l is 1.0.

Table 5-3 Soil stiffness, *E'*, for pipe embedment materials (psi)*

Soil Stiffness Category (SC)	Soil Type†	AASHTO Soil Groups‡	Depth of Cover (ft)	Compaction Level§ 85%	90%	95%	100%
SC1	Clean, coarse-grained soils: SW, SP, GW, GP, or any soil beginning with one of these symbols with 12% or less passing a No. 200 sieve	A1, A3	2–5	700	1,000	1,600	2,500
			5–10	1,000	1,500	2,200	3,300
			10–15	1,050	1,600	2,400	3,600
			15 +	1,100	1,700	2,500	3,800
SC2	Coarse-grained soils with fines: GM, GC, SM, SC, or any soil beginning with one of these symbols more than 12% fines. Sandy or gravelly fine-grained soils: CL, ML (or CL-ML, CL/ML, ML/CL) with more than 25% retained on a No. 200 sieve	A-2-4, A-2-5, A-2-6, or A-4 or A-6 soils with more than 25% retained on a No. 200 sieve	2–5	600	1,000	1,200	1,900
			5–10	900	1,400	1,800	2,700
			10–15	1,000	1,500	2,100	3,200
			15+	1,100	1,600	2,400	3,700
SC3**	Fine-grained soils: CL, ML (or CL-ML, CL/ML, ML/CL) with 25% or less retained on a No. 200 sieve	A-2-7, or A-4 or A-6 soils with 25% or less retained on a No. 200 sieve	2–5	500	700	1,000	1,500
			5–10	600	1,000	1,400	2,000
			10–15	700	1,200	1,600	2,300
			15+	800	1,300	1,800	2,600

NOTE: Soils that have 30% or less retained on the ¾-in. sieve are covered in the standard Proctor test, but the test on some clean gravels can be difficult to complete and the results difficult to interpret. Clean gravel (GP and GW) materials such as "crushed rock" are considered to be the stiffest embedment soils and when consolidated are at least equivalent to 100% standard Proctor compaction for the selection of E' values.

* Derived from Hartley and Duncan 1987.

† ASTM D2487.

‡ AASHTO M145.

§ Standard Proctor densities in accordance with AASHTO T 99 and ASTM D698 are used with this table. See Table 5-4 for comparative soil density tests.

** SC3 soils may require more restricted values for material passing the No. 200 sieve and liquid limit when used with large-diameter (>60 in.) pipe.

Table 5-4 Comparison of standard density tests*

Test	Hammer Weight (lb)	Hammer Drop (in.)	Number of Soil Layers	Blows/Layer	Compactive Energy per Unit Volume ($ft\text{-}lb_f/ft^3$)
Standard Proctor (AASHTO T 99/ ASTM D698)	5.5	12	3	25	12,400
Modified Proctor (AASHTO T 180/ ASTM D1557)	10	18	5	25	56,250

* Natural in-place deposits of soils have densities from 60% to 100% of maximum obtained by the standard AASHTO compaction method. The designer should be sure that the E' value used in design is consistent with this specified degree of compaction and the method of testing that will be used during construction.

For free-draining soils, the relative density should be at least 70 percent as determined by ASTM D4253 and ASTM D4254.

In addition to other considerations, the allowable pipe deflection is also dependent on the type of jointing system being utilized. Contact the pipe manufacturers for additional joint deflection limitations.

Actual percent pipe deflection at a location in an installed pipeline can be determined by measuring the inside diameter along the horizontal axis (D_x) and the vertical axis (D_y). The deflection is then calculated by the following:

Table 5-5 Unified soil classification

Symbol	Description
GW	Well-graded gravels, gravel-sand mixtures, little or no fines
GP	Poorly graded gravels, gravel-sand mixtures, little or no fines
GM	Silty gravels, poorly graded gravel-sand-silt mixtures
GC	Clayey gravels, poorly graded gravel-sand-clay mixtures
SW	Well-graded sands, gravelly sands, little or no fines
SP	Poorly graded sands, gravelly sands, little or no fines
SM	Silty sands, poorly graded sand-silt mixtures
SC	Clayey sands, poorly graded sand-clay mixtures
ML	Inorganic silts and very fine sand, silty or clayey fine sands
CL	Inorganic clays of low to medium plasticity
MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
CH	Inorganic clays of high plasticity, fat clays
OL*	Organic silts and organic silt-clays of low plasticity
OH*	Organic clays of medium to high plasticity
PT*	Peat and other highly organic soils

Source: ASTM Standard D2487.

*Typically not suitable for pipe backfill material.

$$\% \text{ Deflection} = \left(\frac{D_x - D_y}{D_x + D_y}\right) \times 100 \tag{Eq 5-5}$$

Modulus of soil reaction, E', is a measure of stiffness of the pipe embedment material, which surrounds the pipe as shown in the trench detail (Figure 5-2) and is not generally dependent on soils outside the trench walls. (For poor soils with blow counts of four or less per foot refer to ASCE MOP No. 119, *Buried Flexible Steel Pipe* [ASCE 2009]). E' is a hybrid modulus that has been derived empirically to eliminate the spring constant used in the original Iowa formula. It is the product of the modulus of passive resistance of the pipe zone soil used in Spangler's early derivation and the radius of the pipe. E' increases with depth of cover, is not a fundamental material property, and cannot be measured either in the field or in a geotechnical laboratory using soil samples.

Values of E' were originally determined by measuring deflections of actual installations of metal (corrugated and smooth wall) culvert pipe and then by back-calculating the effective soil reaction.

In the same way, the minimum required E' value can be calculated by rewriting the modified Iowa formula as follows:

$$E' = 16.4\left[\frac{C_\Delta \times W}{r} - \frac{EI}{r^3}\right] \tag{Eq 5-6}$$

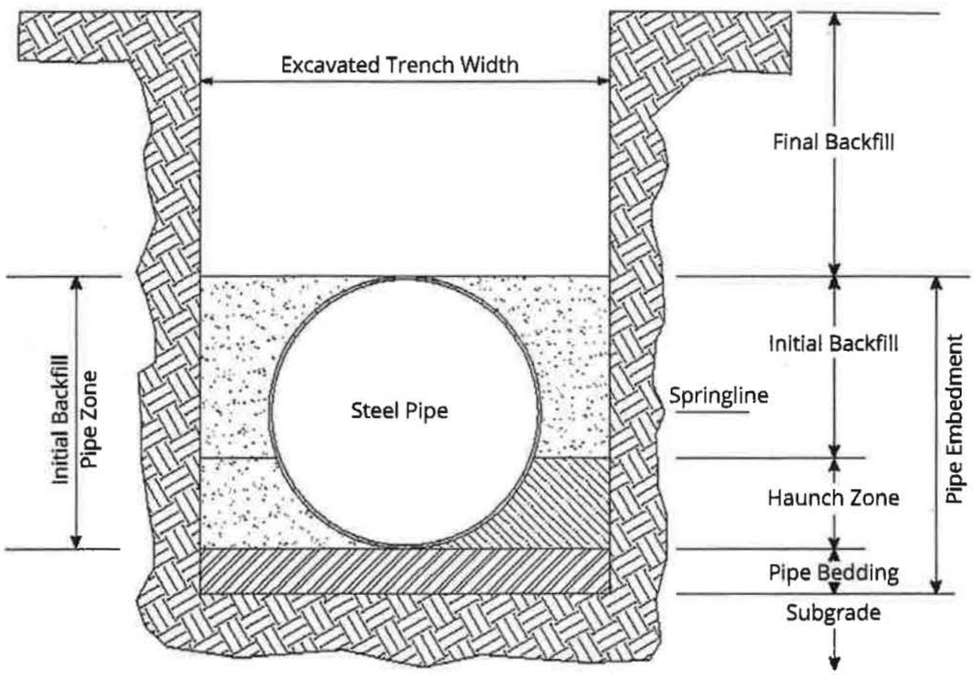
Where:

C_Δ = factor to account for predicted pipe deflection limit

W = load per unit of pipe length, lb/lin in.

$= [W_C/12 + W_L D_c/144]$

r = mean radius of the pipe, in.



Notes:
—Pipe embedment materials may be SC1, SC2, SC3 or as specified. Materials shall be placed evenly on both sides of pipe and compacted to the density specified by the purchaser.
—Subgrade may need to be replaced or modified if trench bottom material is unacceptable or unstable.
—Trench width shall be adequate to assure elimination of voids in the launch area and/or proper placement and compaction of initial backfill materials.

Source: ANSI/AWWA C604-11.

Figure 5-2 Trench detail

Therefore,

C_Δ = 1 for flexible lining and coating

C_Δ = 1.67 for cement-mortar lining and flexible coating

C_Δ = 2.5 for cement-mortar coating

Using Eq 5-6 and Table 5-3, the most efficient soil type and compaction level of embedment material can be determined at a selected depth of cover.

Example Problem. Is 85 percent compaction of a native fine-grained silt (ML) weighing 110 lb/ft^3 adequate for a 48-in. nominal diameter pipe with a D/t ratio of 240, cement-mortar lined with flexible (liquid or tape) coating, buried with 25 ft of cover?

D_o = 49.75-in. OD cylinder

t = 0.20 in.

t_L = 0.5 in.

C_Δ = 1.67 for cement-mortar lining and flexible coating

Soil stiffness category = SC3

Solving for E' using Eq 5-6,

$$W = (110 \times 25 \times 49.75/12)/12 = 950 \text{ lb/lin in.}$$

$$r^3 = [(49.75 - 0.20)/2]^3 = 15{,}207 \text{ in.}^3$$

$$E_S I_S + E_C I_L = 30{,}000{,}000 \times 0.20^3/12 + 4{,}000{,}000 \times 0.5^3/12 = 20{,}000 + 41{,}667 = 61{,}667 \text{ lb in.}$$

$$E' = 16.4\,[1.67 \times 950/[(49.75 - 0.20)/2] - 61{,}667/15{,}207]$$

Therefore minimum E' = 984 psi

APPENDIX D

COMPUTER HYDRAULIC MODELING OUTPUT

NODE AND PIPE DIAGRAM REFERENCE:

Exhibit A

The following conditions were modeled:

0. Average Day Demand.
1. Peak Hour Demand.
2. Maximum Day Demand plus 1,500 gpm Fire Flow at Node 7.
3. Maximum Day Demand plus 1,500 gpm Fire Flow at Node 2.
4. Maximum Day Demand plus 1,500 gpm Fire Flow at Node 7. Pipe 13 closed.

Project: Emerald Hills
Date: 6/6/2024
Job Number: 688-034

Scenario: All Pipes Open - Average Day Demand

Node No.	**Node El. Ft.**	**HGL Zone Ft. (Static)***	**Static P psi**	**Model Run P, psi**	**Delta P from Static**
Hyd Test		--	--	--	--
J-1	373	610	102.7	99.7	3.0
J-2	375	610	101.8	98.8	3.0
J-3	350	610	112.7	109.6	3.0
J-4	305	610	132.1	129.1	3.0
J-5	366	610	105.7	102.7	3.0
J-6	370	610	104.0	101.0	3.0
J-7	391	610	94.9	91.9	3.0
J-8	380	610	99.7	96.6	3.0
J-9	330	610	121.3	118.3	3.0
J-10	377	610	101.0	97.9	3.0
J-11	310	610	130.0	127.0	3.0

Project: Emerald Hills
Date: 6/6/2024
Job Number: 688-034

Scenario: Peak Hour Demand

Node No.	Node El. Ft.	HGL Zone Ft. (Static)*	Static P psi	Model Run P, psi	Delta P from Static
Hyd Test		--	--	--	--
J-1	373	610	102.7	99.4	3.3
J-2	375	610	101.8	98.6	3.3
J-3	350	610	112.7	109.4	3.2
J-4	305	610	132.1	129.0	3.2
J-5	366	610	105.7	102.5	3.2
J-6	370	610	104.0	100.7	3.2
J-7	391	610	94.9	91.6	3.2
J-8	380	610	99.7	96.4	3.2
J-9	330	610	121.3	118.1	3.2
J-10	377	610	101.0	97.7	3.3
J-11	310	610	130.0	126.8	3.2

Project: Emerald Hills
Date: 6/6/2024
Job Number: 688-034

Scenario: Maximum Day Demand plus 1500 gpm Fire Flow at Node 7

Node No.	Node El. Ft.	HGL Zone Ft. (Static)*	Static P psi	Model Run P, psi	Delta P from Static
Hyd Test		--	--	--	
J-1	373	610	102.7	80.4	22.3
J-2	375	610	101.8	79.5	22.3
J-3	350	610	112.7	91.7	20.9
J-4	305	610	132.2	114.6	17.6
J-5	366	610	105.7	83.8	22.0
J-6	370	610	104.0	81.4	22.6
J-7	391	610	94.9	68.7	26.2
J-8	380	610	99.7	77.1	22.6
J-9	330	610	121.3	101.6	19.8
J-10	377	610	101.0	79.2	21.7
J-11	310	610	130.0	111.1	18.9

Project: Emerald Hills
Date: 6/6/2024
Job Number: 688-034

Scenario: Maximum Day Demand plus 1500 gpm Fire Flow at Node 2

Node No.	Node El. Ft.	HGL Zone Ft. (Static)*	Static P psi	Model Run P, psi	Delta P from Static
Hyd Test		--	--	--	--
J-1	373	610	102.69	78.11	24.6
J-2	375	610	101.82	71.09	30.7
J-3	350	610	112.65	91.05	21.6
J-4	305	610	132.15	114.35	17.8
J-5	366	610	105.72	83.61	22.1
J-6	370	610	103.99	81.57	22.4
J-7	391	610	94.89	72.85	22.0
J-8	380	610	99.65	77.92	21.7
J-9	330	610	121.32	102.06	19.3
J-10	377	610	100.95	77.59	23.4
J-11	310	610	129.98	110.65	19.3

Project: Emerald Hills
Date: 6/6/2024
Job Number: 688-034

Scenario: Maximum Day Demand plus 1500 gpm Fire Flow at Node 7, Pipe 13 Closed

Node No.	Node El. Ft.	HGL Zone Ft. (Static)*	Static P psi	Model Run P, psi
Hyd Test		--	--	--
J-1	373	610	102.7	60.2
J-2	375	610	101.8	59.3
J-3	350	610	112.7	70.1
J-4	305	610	132.2	89.6
J-5	366	610	105.7	63.2
J-6	370	610	104.0	61.5
J-7	391	610	94.9	49.4
J-8	380	610	99.7	58.4
J-9	330	610	121.3	90.8
J-10	377	610	101.0	58.4
J-11	310	610	130.0	87.5

Project: Emerald Hills
Date: 6/6/2024
Job Number: 688-034

Scenario: All Pipes Open - Average Day Demand

Pipe No.	Pipe Size (inches)	Model Run Flow (gpm)	Model Run Velocity (fps)
1	8	-7.23	0.1
2	8	5.63	0.0
3	8	3.57	0.0
4	8	24.48	0.2
5	8	9.66	0.1
6	8	9.2	0.1
7	8	4.03	0.0
8	8	24.48	0.2
9	8	-0.43	0.0
10	8	-8.87	0.1
11	8	6.06	0.0
12	8	-20.56	0.1
13	12	24.48	0.1
14	8	-20.56	0.1

Project: Emerald Hills
Date: 6/6/2024
Job Number: 688-034

Scenario: Peak Hour Demand

Pipe No.	Pipe Size (inches)	Model Run Flow (gpm)	Model Run Velocity (fps)
1	8	-27.13	0.2
2	8	21.11	0.1
3	8	13.38	0.1
4	8	91.81	0.6
5	8	36.21	0.2
6	8	34.49	0.2
7	8	15.1	0.1
8	8	91.81	0.6
9	8	-1.61	0.0
10	8	-33.25	0.2
11	8	22.72	0.2
12	8	-77.09	0.5
13	12	91.81	0.3
14	8	-77.09	0.5

Project: Emerald Hills
Date: 6/6/2024
Job Number: 688-034

Scenario: Maximum Day Demand plus 1500 gpm Fire Flow at Node 7

Pipe No.	Pipe Size (inches)	Model Run Flow (gpm)	Model Run Velocity (fps)
1	8	331.99	2.1
2	8	9.57	0.1
3	8	430.28	2.8
4	8	810.12	5.2
5	8	360.7	2.3
6	8	439.85	2.8
7	8	351.13	2.2
8	8	810.12	5.2
9	8	691.24	4.4
10	8	61.45	0.4
11	8	819.76	5.2
12	8	-767.88	4.9
13	12	810.12	2.3
14	8	-767.88	4.9

Project: Emerald Hills
Date: 6/6/2024
Job Number: 688-034

Scenario: Maximum Day Demand plus 1500 gpm Fire Flow at Node 2

Pipe No.	Pipe Size (inches)	Model Run Flow (gpm)	Model Run Velocity (fps)
1	8	-977.66	6.2
2	8	1511	9.6
3	8	293.72	1.9
4	8	865.35	5.5
5	8	552.48	3.5
6	8	303.29	1.9
7	8	542.91	3.5
8	8	865.35	5.5
9	8	-205.87	1.3
10	8	-487.64	3.1
11	8	215.44	1.4
12	8	-712.65	4.6
13	12	865.35	2.5
14	8	-712.65	4.6

Project: Emerald Hills
Date: 6/6/2024
Job Number: 688-034

Scenario: Maximum Day Demand plus 1500 gpm Fire Flow at Node 7, Pipe 13 Closed

Pipe No.	**Pipe Size (inches)**	**Model Run Flow (gpm)**	**Model Run Velocity (fps)**
1	8	-27.25	0.17
2	8	9.57	0.1
3	8	-20.6	0.1
4	8	0	0.0
5	8	1.46	0.0
6	8	-11.03	0.1
7	8	-8.11	0.1
8	8	0	0.0
9	8	619.21	4.0
10	8	-676.63	4.3
11	8	891.79	5.7
12	8	-1578	10.1
13	12	PIPE CLOSED	
14	8	-1578	10.1

```
* * * * * * * * * * * * *  K Y P I P E  * * * * * * * * * * * * *
*                                                                *
*  Pipe Network Modeling Software                                *
*                                                                *
*  CopyRighted by KYPIPE LLC (www.kypipe.com)                    *
*  Version: 10.009  10/01/2019                                   *
*  Company:  Dexter          Serial #:  592169                   *
*  Interface:  Classic                                           *
*  Licensed for Pipe2018                                         *
*                                                                *
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Date & Time: Wed Jun 05 18:11:07 2024

Master File : \\artic\eng\688034\emerald hills ky pipe june 2024.KYP\emerald hills ky pipe june 2024.P2K

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************************************************
 S U M M A R Y   O F   O R I G I N A L   D A T A
************************************************
```

U N I T S   S P E C I F I E D

FLOWRATE .... = gallons/minute
HEAD (HGL) .... = feet
PRESSURE .... = psig

P I P E L I N E   D A T A

STATUS CODE: XX -CLOSED PIPE   CV -CHECK VALVE

P I P E N A M E	NODE NAMES #1	#2	LENGTH (ft)	DIAMETER (in)	ROUGHNESS COEFF.	MINOR LOSS COEFF.
P-1	J-1	J-6	245.00	8.00	120.0000	0.00
P-2	J-1	J-2	312.00	8.00	120.0000	0.00
P-3	J-5	J-6	321.00	8.00	120.0000	0.00
P-4	J-11	J-3	323.00	8.00	120.0000	0.00
P-5	J-3	J-10	575.00	8.00	120.0000	0.00
P-6	J-3	J-5	505.00	8.00	120.0000	0.00
P-7	J-10	J-1	409.00	8.00	120.0000	0.00
P-8	J-4	J-11	218.00	8.00	120.0000	0.00
P-9	J-6	J-7	778.00	8.00	120.0000	0.00
P-10	J-6	J-8	282.00	8.00	120.0000	0.00
P-11	J-8	J-7	565.00	8.00	120.0000	0.00
P-12	J-8	J-9	504.00	8.00	120.0000	0.00
P-13	Hyd Test	J-4	1939.00	12.00	120.0000	0.00
P-14	J-9	Hyd Test	683.00	8.00	120.0000	0.00

P U M P/L O S S   E L E M E N T   D A T A

THERE IS A DEVICE AT NODE Hyd Test DESCRIBED BY THE FOLLOWING DATA: (ID= 1)

HEAD (ft)	FLOWRATE (gpm)	EFFICIENCY (%)
603.00	0.00	75.00 (Default)
600.00	500.00	75.00 (Default)
590.00	1000.00	75.00 (Default)
576.00	1500.00	75.00 (Default)
557.00	2000.00	75.00 (Default)
533.00	2500.00	75.00 (Default)
505.00	3000.00	75.00 (Default)

N O D E   D A T A

NODE NAME	NODE TITLE	EXTERNAL DEMAND (gpm)	JUNCTION ELEVATION (ft)	EXTERNAL GRADE (ft)
Hyd Test		0.00	0.00	
J-1		5.63	373.00	
J-2		5.63	375.00	
J-3		5.63	350.00	
J-4		0.00	305.00	
J-5		5.63	366.00	
J-6		5.63	370.00	
J-7		5.63	391.00	
J-8		5.63	380.00	
J-9		0.00	330.00	
J-10		5.63	377.00	
J-11		0.00	310.00	

O U T P U T   O P T I O N   D A T A

OUTPUT SELECTION: ALL RESULTS ARE INCLUDED IN THE TABULATED OUTPUT
MAXIMUM AND MINIMUM PRESSURES = 3
MAXIMUM AND MINIMUM VELOCITIES = 3

S Y S T E M   C O N F I G U R A T I O N

NUMBER OF PIPES ....(P) = 14
NUMBER OF END NODES ....(J) = 11
NUMBER OF PRIMARY LOOPS ....(L) = 3
NUMBER OF SUPPLY NODES ....(F) = 1
NUMBER OF SUPPLY ZONES ....(Z) = 1

==

Case: 0

RESULTS OBTAINED AFTER 10 TRIALS: ACCURACY = 0.50530E-06

S I M U L A T I O N D E S C R I P T I O N (L A B E L)

AVERAGE DAY DEMAND

P I P E L I N E R E S U L T S

STATUS CODE: XX -CLOSED PIPE CV -CHECK VALVE

P I P E N A M E	NODE NUMBERS #1	#2	FLOWRATE gpm	HEAD LOSS ft	MINOR LOSS ft	LINE VELO. ft/s	HL+ML/ 1000 ft/f	HL/ 1000 ft/f
P-1	J-1	J-6	-7.23	0.00	0.00	0.05	0.00	0.00
P-2	J-1	J-2	5.63	0.00	0.00	0.04	0.00	0.00
P-3	J-5	J-6	3.57	0.00	0.00	0.02	0.00	0.00
P-4	J-11	J-3	24.48	0.01	0.00	0.16	0.02	0.02
P-5	J-3	J-10	9.66	0.00	0.00	0.06	0.00	0.00
P-6	J-3	J-5	9.20	0.00	0.00	0.06	0.00	0.00
P-7	J-10	J-1	4.03	0.00	0.00	0.03	0.00	0.00
P-8	J-4	J-11	24.48	0.00	0.00	0.16	0.02	0.02
P-9	J-6	J-7	-0.43	0.00	0.00	0.00	0.00	0.00
P-10	J-6	J-8	-8.87	0.00	0.00	0.06	0.00	0.00
P-11	J-8	J-7	6.06	0.00	0.00	0.04	0.00	0.00
P-12	J-8	J-9	-20.56	0.01	0.00	0.13	0.02	0.02
P-13	Hyd Test	J-4	24.48	0.01	0.00	0.07	0.00	0.00
P-14	J-9	Hyd Test	-20.56	0.01	0.00	0.13	0.02	0.02

P U M P/L O S S E L E M E N T R E S U L T S

NAME	FLOWRATE gpm	INLET HEAD ft	OUTLET HEAD ft	PUMP HEAD ft	EFFIC- ENCY %	USEFUL POWER Hp	INCREMTL COST $	TOTAL COST $	#PUMPS PARALLEL	#PUMPS SERIES	NPSH Avail. ft	Case
Hyd Test	45.04	0.00	602.98	603.0	75.00	7.	0.3	0.3	**	**	33.2	0.0000

N O D E   R E S U L T S

NODE NAME	NODE TITLE	EXTERNAL DEMAND gpm	HYDRAULIC GRADE ft	NODE ELEVATION ft	PRESSURE HEAD ft	NODE PRESSURE psi
Hyd Test		0.00	602.98			
J-1		5.63	602.96	373.00	229.96	99.65
J-2		5.63	602.96	375.00	227.96	98.78
J-3		5.63	602.96	350.00	252.96	109.62
J-4		0.00	602.98	305.00	297.98	129.12
J-5		5.63	602.96	366.00	236.96	102.68
J-6		5.63	602.96	370.00	232.96	100.95
J-7		5.63	602.96	391.00	211.96	91.85
J-8		5.63	602.96	380.00	222.96	96.62
J-9		0.00	602.97	330.00	272.97	118.29
J-10		5.63	602.96	377.00	225.96	97.92
J-11		0.00	602.97	310.00	292.97	126.95

M A X I M U M   A N D   M I N I M U M   V A L U E S

P R E S S U R E S

JUNCTION NUMBER	MAXIMUM PRESSURES psi	JUNCTION NUMBER	MINIMUM PRESSURES psi
J-4	129.12	J-7	91.85
J-11	126.95	J-8	96.62
J-9	118.29	J-10	97.92

V E L O C I T I E S

PIPE NUMBER	MAXIMUM VELOCITY (ft/s)	PIPE NUMBER	MINIMUM VELOCITY (ft/s)
P-4	0.16	P-9	0.00
P-8	0.16	P-3	0.02
P-14	0.13	P-7	0.03

S U M M A R Y   O F   I N F L O W S   A N D   O U T F L O W S

(+) INFLOWS INTO THE SYSTEM FROM SUPPLY NODES
(-) OUTFLOWS FROM THE SYSTEM INTO SUPPLY NODES

NODE NAME	FLOWRATE gpm	NODE TITLE
Hyd Test	45.04	

NET SYSTEM INFLOW = 45.04
NET SYSTEM OUTFLOW = 0.00
NET SYSTEM DEMAND = 45.04

==
Case: 1

C H A N G E S   F O R   N E X T   S I M U L A T I O N (Change Number = 1)

Peak Hour Demand

JUNCTION DEMANDS CHANGED - PLEASE SEE RESULTS TABLE

RESULTS OBTAINED AFTER 3 TRIALS: ACCURACY = 0.20810E-06

P I P E L I N E   R E S U L T S

STATUS CODE: XX -CLOSED PIPE CV -CHECK VALVE

PIPE NAME	NODE NUMBERS #1	#2	FLOWRATE gpm	HEAD LOSS ft	MINOR LOSS ft	LINE VELO. ft/s	HL+ML/ 1000 ft/f	HL/ 1000 ft/f
P-1	J-1	J-6	-27.13	0.01	0.00	0.17	0.03	0.03
P-2	J-1	J-2	21.11	0.01	0.00	0.13	0.02	0.02
P-3	J-5	J-6	13.38	0.00	0.00	0.09	0.01	0.01
P-4	J-11	J-3	91.81	0.08	0.00	0.59	0.25	0.25
P-5	J-3	J-10	36.21	0.03	0.00	0.23	0.05	0.05
P-6	J-3	J-5	34.49	0.02	0.00	0.22	0.04	0.04
P-7	J-10	J-1	15.10	0.00	0.00	0.10	0.01	0.01
P-8	J-4	J-11	91.81	0.06	0.00	0.59	0.25	0.25
P-9	J-6	J-7	-1.61	0.00	0.00	0.01	0.00	0.00
P-10	J-6	J-8	-33.25	0.01	0.00	0.21	0.04	0.04
P-11	J-8	J-7	22.72	0.01	0.00	0.15	0.02	0.02
P-12	J-8	J-9	-77.09	0.09	0.00	0.49	0.18	0.18
P-13	Hyd Test	J-4	91.81	0.07	0.00	0.26	0.04	0.04
P-14	J-9	Hyd Test	-77.09	0.13	0.00	0.49	0.18	0.18

P U M P/L O S S   E L E M E N T   R E S U L T S

NAME	FLOWRATE gpm	INLET HEAD ft	OUTLET HEAD ft	PUMP HEAD ft	EFFIC-ENCY %	USEFUL POWER Hp	INCREMTL COST $	TOTAL COST $	#PUMPS PARALLEL	#PUMPS SERIES	NPSH Avail. ft	Case
Hyd Test	168.90	0.00	602.70	602.7	75.00	26.	0.3	0.7	**	**	33.2	1.0000

N O D E   R E S U L T S

NODE NAME	NODE TITLE	EXTERNAL DEMAND gpm	HYDRAULIC GRADE ft	NODE ELEVATION ft	PRESSURE HEAD ft	NODE PRESSURE psi
Hyd Test		0.00	602.70			
J-1		21.11(3.75)	602.46	373.00	229.46	99.43
J-2		21.11(3.75)	602.46	375.00	227.46	98.56
J-3		21.11(3.75)	602.49	350.00	252.49	109.41
J-4		0.00	602.63	305.00	297.63	128.97
J-5		21.11(3.75)	602.47	366.00	236.47	102.47
J-6		21.11(3.75)	602.47	370.00	232.47	100.74
J-7		21.11(3.75)	602.47	391.00	211.47	91.64
J-8		21.11(3.75)	602.48	380.00	222.48	96.41
J-9		0.00	602.57	330.00	272.57	118.11
J-10		21.11(3.75)	602.47	377.00	225.47	97.70
J-11		0.00	602.57	310.00	292.57	126.78

M A X I M U M   A N D   M I N I M U M   V A L U E S

P R E S S U R E S

JUNCTION NUMBER	MAXIMUM PRESSURES psi	JUNCTION NUMBER	MINIMUM PRESSURES psi
J-4	128.97	J-7	91.64
J-11	126.78	J-8	96.41
J-9	118.11	J-10	97.70

V E L O C I T I E S

PIPE NUMBER	MAXIMUM VELOCITY (ft/s)	PIPE NUMBER	MINIMUM VELOCITY (ft/s)
P-4	0.59	P-9	0.01
P-8	0.59	P-3	0.09
P-12	0.49	P-7	0.10

S U M M A R Y   O F   I N F L O W S   A N D   O U T F L O W S

(+) INFLOWS INTO THE SYSTEM FROM SUPPLY NODES
(-) OUTFLOWS FROM THE SYSTEM INTO SUPPLY NODES

NODE NAME	FLOWRATE gpm	NODE TITLE
Hyd Test	168.90	

NET SYSTEM INFLOW = 168.90
NET SYSTEM OUTFLOW = 0.00
NET SYSTEM DEMAND = 168.90

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Case: 2

C H A N G E S   F O R   N E X T   S I M U L A T I O N (Change Number = 2)

Maximum Day Demand plus 1,500 gpm Fire Flow at Node 7

JUNCTION DEMANDS CHANGED - PLEASE SEE RESULTS TABLE

RESULTS OBTAINED AFTER 5 TRIALS: ACCURACY = 0.85086E-07

P I P E L I N E   R E S U L T S

STATUS CODE: XX -CLOSED PIPE CV -CHECK VALVE

PIPE NAME	NODE NUMBERS #1	#2	FLOWRATE gpm	HEAD LOSS ft	MINOR LOSS ft	LINE VELO. ft/s	HL+ML/ 1000 ft/f	HL/ 1000 ft/f
P-1	J-1	J-6	331.99	0.67	0.00	2.12	2.75	2.75
P-2	J-1	J-2	9.57	0.00	0.00	0.06	0.00	0.00
P-3	J-5	J-6	430.28	1.43	0.00	2.75	4.44	4.44
P-4	J-11	J-3	810.12	4.63	0.00	5.17	14.35	14.35
P-5	J-3	J-10	360.70	1.84	0.00	2.30	3.21	3.21
P-6	J-3	J-5	439.85	2.34	0.00	2.81	4.63	4.63
P-7	J-10	J-1	351.13	1.25	0.00	2.24	3.05	3.05
P-8	J-4	J-11	810.12	3.13	0.00	5.17	14.35	14.35
P-9	J-6	J-7	691.24	8.32	0.00	4.41	10.69	10.69
P-10	J-6	J-8	61.45	0.03	0.00	0.39	0.12	0.12
P-11	J-8	J-7	819.76	8.28	0.00	5.23	14.66	14.66
P-12	J-8	J-9	-767.88	6.55	0.00	4.90	12.99	12.99
P-13	Hyd Test	J-4	810.12	3.86	0.00	2.30	1.99	1.99
P-14	J-9	Hyd Test	-767.88	8.87	0.00	4.90	12.99	12.99

P U M P/L O S S   E L E M E N T   R E S U L T S

NAME	FLOWRATE gpm	INLET HEAD ft	OUTLET HEAD ft	PUMP HEAD ft	EFFIC- ENCY %	USEFUL POWER Hp	INCREMTL COST $	TOTAL COST $	#PUMPS PARALLEL	#PUMPS SERIES	NPSH Avail. ft	Case
Hyd Test	1578.00	0.00	573.25	573.3	75.00	229.	1.3	2.0	**	**	33.2	2.0000

N O D E   R E S U L T S

NODE NAME	NODE TITLE	EXTERNAL DEMAND gpm	HYDRAULIC GRADE ft	NODE ELEVATION ft	PRESSURE HEAD ft	NODE PRESSURE psi
Hyd Test		0.00	573.25			
J-1		9.57(1.70)	558.54	373.00	185.54	80.40
J-2		9.57(1.70)	558.54	375.00	183.54	79.53
J-3		9.57(1.70)	561.63	350.00	211.63	91.71
J-4		0.00	569.39	305.00	264.39	114.57
J-5		9.57(1.70)	559.29	366.00	193.29	83.76
J-6		9.57(1.70)	557.86	370.00	187.86	81.41
J-7		1511.00(**)	549.55	391.00	158.55	68.70
J-8		9.57(1.70)	557.83	380.00	177.83	77.06
J-9		0.00	564.38	330.00	234.38	101.56
J-10		9.57(1.70)	559.79	377.00	182.79	79.21
J-11		0.00	566.26	310.00	256.26	111.05

M A X I M U M   A N D   M I N I M U M   V A L U E S

P R E S S U R E S

JUNCTION NUMBER	MAXIMUM PRESSURES psi	JUNCTION NUMBER	MINIMUM PRESSURES psi
J-4	114.57	J-7	68.70
J-11	111.05	J-8	77.06
J-9	101.56	J-10	79.21

V E L O C I T I E S

PIPE NUMBER	MAXIMUM VELOCITY (ft/s)	PIPE NUMBER	MINIMUM VELOCITY (ft/s)
P-11	5.23	P-2	0.06
P-4	5.17	P-10	0.39
P-8	5.17	P-1	2.12

S U M M A R Y   O F   I N F L O W S   A N D   O U T F L O W S

(+) INFLOWS INTO THE SYSTEM FROM SUPPLY NODES
(-) OUTFLOWS FROM THE SYSTEM INTO SUPPLY NODES

NODE NAME	FLOWRATE gpm	NODE TITLE
Hyd Test	1578.00	

NET SYSTEM INFLOW = 1578.00
NET SYSTEM OUTFLOW = 0.00
NET SYSTEM DEMAND = 1578.00

==
Case: 3

C H A N G E S   F O R   N E X T   S I M U L A T I O N (Change Number = 3)

Maximum Day Demand plus 1,500 gpm Fire Flow at Node 2

JUNCTION DEMANDS CHANGED - PLEASE SEE RESULTS TABLE

RESULTS OBTAINED AFTER 5 TRIALS: ACCURACY = 0.74466E-04

P I P E L I N E   R E S U L T S

STATUS CODE: XX -CLOSED PIPE CV -CHECK VALVE

PIPE NAME	NODE NUMBERS #1	#2	FLOWRATE gpm	HEAD LOSS ft	MINOR LOSS ft	LINE VELO. ft/s	HL+ML/ 1000 ft/f	HL/ 1000 ft/f
P-1	J-1	J-6	-977.66	4.98	0.00	6.24	20.32	20.32
P-2	J-1	J-2	1511.00	14.20	0.00	9.64	45.51	45.51
P-3	J-5	J-6	293.72	0.70	0.00	1.87	2.19	2.19
P-4	J-11	J-3	865.35	5.24	0.00	5.52	16.21	16.21
P-5	J-3	J-10	552.48	4.06	0.00	3.53	7.06	7.06
P-6	J-3	J-5	303.29	1.17	0.00	1.94	2.33	2.33
P-7	J-10	J-1	542.91	2.80	0.00	3.47	6.84	6.84
P-8	J-4	J-11	865.35	3.53	0.00	5.52	16.21	16.21
P-9	J-6	J-7	-205.87	0.88	0.00	1.31	1.13	1.13
P-10	J-6	J-8	-487.64	1.58	0.00	3.11	5.60	5.60
P-11	J-8	J-7	215.44	0.70	0.00	1.38	1.23	1.23
P-12	J-8	J-9	-712.65	5.70	0.00	4.55	11.31	11.31
P-13	Hyd Test	J-4	865.35	4.36	0.00	2.45	2.25	2.25
P-14	J-9	Hyd Test	-712.65	7.73	0.00	4.55	11.31	11.31

P U M P/L O S S   E L E M E N T   R E S U L T S

NAME	FLOWRATE gpm	INLET HEAD ft	OUTLET HEAD ft	PUMP HEAD ft	EFFIC- ENCY %	USEFUL POWER Hp	INCREMTL COST $	TOTAL COST $	#PUMPS PARALLEL	#PUMPS SERIES	NPSH Avail. ft	Case
Hyd Test	1578.00	0.00	573.25	573.3	75.00	229.	11.4	13.3	**	**	33.2	3.0000

N O D E   R E S U L T S

NODE NAME	NODE TITLE	EXTERNAL DEMAND gpm	HYDRAULIC GRADE ft	NODE ELEVATION ft	PRESSURE HEAD ft	NODE PRESSURE psi
Hyd Test		0.00	573.25			
J-1		9.57(1.70)	553.26	373.00	180.26	78.11
J-2		1511.00(**)	539.07	375.00	164.07	71.09
J-3		9.57(1.70)	560.12	350.00	210.12	91.05
J-4		0.00	568.89	305.00	263.89	114.35
J-5		9.57(1.70)	558.94	366.00	192.94	83.61
J-6		9.57(1.70)	558.24	370.00	188.24	81.57
J-7		9.57(1.70)	559.12	391.00	168.12	72.85
J-8		9.57(1.70)	559.82	380.00	179.82	77.92
J-9		0.00	565.52	330.00	235.52	102.06
J-10		9.57(1.70)	556.06	377.00	179.06	77.59
J-11		0.00	565.35	310.00	255.35	110.65

M A X I M U M   A N D   M I N I M U M   V A L U E S

P R E S S U R E S

JUNCTION NUMBER	MAXIMUM PRESSURES psi	JUNCTION NUMBER	MINIMUM PRESSURES psi
J-4	114.35	J-2	71.09
J-11	110.65	J-7	72.85
J-9	102.06	J-10	77.59

V E L O C I T I E S

PIPE NUMBER	MAXIMUM VELOCITY (ft/s)	PIPE NUMBER	MINIMUM VELOCITY (ft/s)
P-2	9.64	P-9	1.31
P-1	6.24	P-11	1.38
P-4	5.52	P-3	1.87

S U M M A R Y   O F   I N F L O W S   A N D   O U T F L O W S

(+) INFLOWS INTO THE SYSTEM FROM SUPPLY NODES
(-) OUTFLOWS FROM THE SYSTEM INTO SUPPLY NODES

NODE NAME	FLOWRATE gpm	NODE TITLE
Hyd Test	1578.00	

NET SYSTEM INFLOW = 1578.00
NET SYSTEM OUTFLOW = 0.00
NET SYSTEM DEMAND = 1578.00

==
Case: 4

C H A N G E S F O R N E X T S I M U L A T I O N (Change Number = 4)

Maximum Day Demand plus 1,500 gpm Fire Flow at Node 7, Pipe 13 Closed

JUNCTION DEMANDS CHANGED - PLEASE SEE RESULTS TABLE

Pipe P-13 is CLOSED

RESULTS OBTAINED AFTER 4 TRIALS: ACCURACY = 0.50232E-05

P I P E L I N E R E S U L T S

STATUS CODE: XX -CLOSED PIPE CV -CHECK VALVE

P I P E N A M E	NODE NUMBERS #1	#2	FLOWRATE gpm	HEAD LOSS ft	MINOR LOSS ft	LINE VELO. ft/s	HL+ML/ 1000 ft/f	HL/ 1000 ft/f
P-1	J-1	J-6	-27.25	0.01	0.00	0.17	0.03	0.03
P-2	J-1	J-2	9.57	0.00	0.00	0.06	0.00	0.00
P-3	J-5	J-6	-20.60	0.01	0.00	0.13	0.02	0.02
P-4	J-11	J-3	0.00	0.00	0.00	0.00	0.00	0.00
P-5	J-3	J-10	1.46	0.00	0.00	0.01	0.00	0.00
P-6	J-3	J-5	-11.03	0.00	0.00	0.07	0.01	0.01
P-7	J-10	J-1	-8.11	0.00	0.00	0.05	0.00	0.00
P-8	J-4	J-11	0.00	0.00	0.00	0.00	0.00	0.00
P-9	J-6	J-7	619.21	6.78	0.00	3.95	8.72	8.72
P-10	J-6	J-8	-676.63	2.90	0.00	4.32	10.28	10.28
P-11	J-8	J-7	891.79	9.68	0.00	5.69	17.14	17.14
P-12	J-8	J-9	-1578.00	24.85	0.00	10.07	49.31	49.31
P-13-XX	Hyd Test	J-4						
P-14	J-9	Hyd Test	-1578.00	33.68	0.00	10.07	49.31	49.31

P U M P/L O S S E L E M E N T R E S U L T S

NAME	FLOWRATE gpm	INLET HEAD ft	OUTLET HEAD ft	PUMP HEAD ft	EFFIC-ENCY %	USEFUL POWER Hp	INCREMTL COST $	TOTAL COST $	#PUMPS PARALLEL	#PUMPS SERIES	NPSH Avail. ft	Case
Hyd Test	1578.00	0.00	573.25	573.3	75.00	229.	11.4	24.7	**	**	33.2	4.0000

N O D E   R E S U L T S

NODE NAME	NODE TITLE	EXTERNAL DEMAND gpm	HYDRAULIC GRADE ft	NODE ELEVATION ft	PRESSURE HEAD ft	NODE PRESSURE psi
Hyd Test		0.00	573.25			
J-1		9.57(1.70)	511.81	373.00	138.81	60.15
J-2		9.57(1.70)	511.81	375.00	136.81	59.28
J-3		9.57(1.70)	511.81	350.00	161.81	70.12
J-4		0.00	511.81	305.00	206.81	89.62
J-5		9.57(1.70)	511.81	366.00	145.81	63.19
J-6		9.57(1.70)	511.82	370.00	141.82	61.45
J-7		1511.00(**)	505.03	391.00	114.03	49.41
J-8		9.57(1.70)	514.72	380.00	134.72	58.38
J-9		0.00	539.57	330.00	209.57	90.81
J-10		9.57(1.70)	511.81	377.00	134.81	58.42
J-11		0.00	511.81	310.00	201.81	87.45

M A X I M U M   A N D   M I N I M U M   V A L U E S

P R E S S U R E S

JUNCTION NUMBER	MAXIMUM PRESSURES psi	JUNCTION NUMBER	MINIMUM PRESSURES psi
J-9	90.81	J-7	49.41
J-4	89.62	J-8	58.38
J-11	87.45	J-10	58.42

V E L O C I T I E S

PIPE NUMBER	MAXIMUM VELOCITY (ft/s)	PIPE NUMBER	MINIMUM VELOCITY (ft/s)
P-12	10.07	P-5	0.01
P-14	10.07	P-7	0.05
P-11	5.69	P-2	0.06

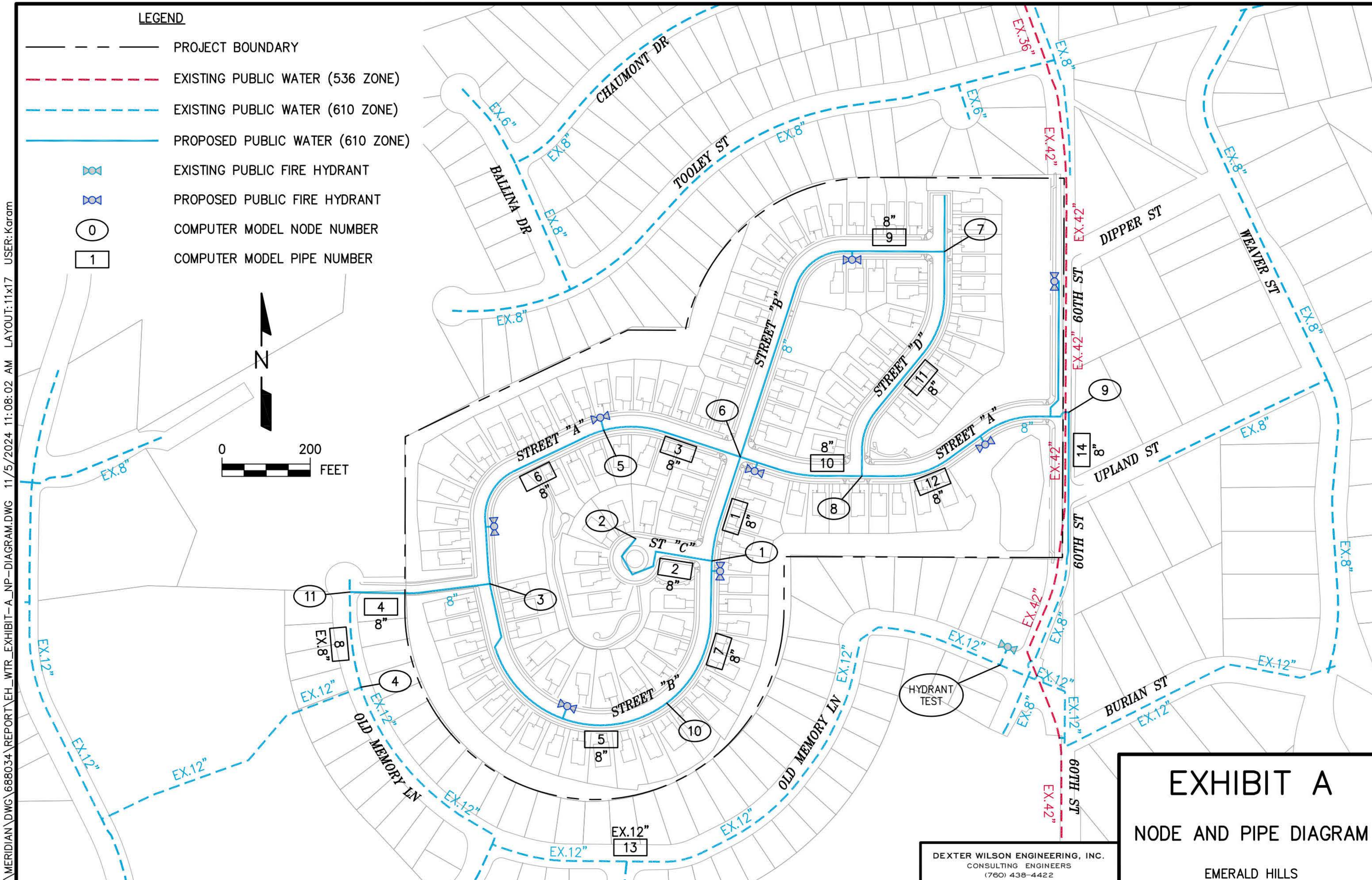
S U M M A R Y   O F   I N F L O W S   A N D   O U T F L O W S

(+) INFLOWS INTO THE SYSTEM FROM SUPPLY NODES
(-) OUTFLOWS FROM THE SYSTEM INTO SUPPLY NODES

NODE NAME	FLOWRATE gpm	NODE TITLE
Hyd Test	1578.00	

NET SYSTEM INFLOW = 1578.00
NET SYSTEM OUTFLOW = 0.00
NET SYSTEM DEMAND = 1578.00

***** HYDRAULIC ANALYSIS COMPLETED *****



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