Amended Drainage Report / Final Drainage Report

For

Midtown Courts

1417 Railroad Avenue

McKinleyville, Humboldt County, California

ENGINEERING, CONSULTING, & PERMITTING

REGISTERED PROFESSIONAL ENGINEER
DAVID T. NICOLETTI
No. 76814
12/31/22
CIVIL
STATE OF CALIFORNIA

1. PROJECT SUMMARY

1.1. INTRODUCTION

This report is an amendment to the Preliminary Drainage Report for the Brookview Tract a Planned Unit Development Dated March 2013 (Exhibit A).

1.2. PROJECT LOCATION

The project site is located in unincorporated town of McKinleyville, Humboldt County, California. It is located just west of the Central Avenue at 1417 Railroad Avenue, APN 510-121-026-000 (FIGURE 1)

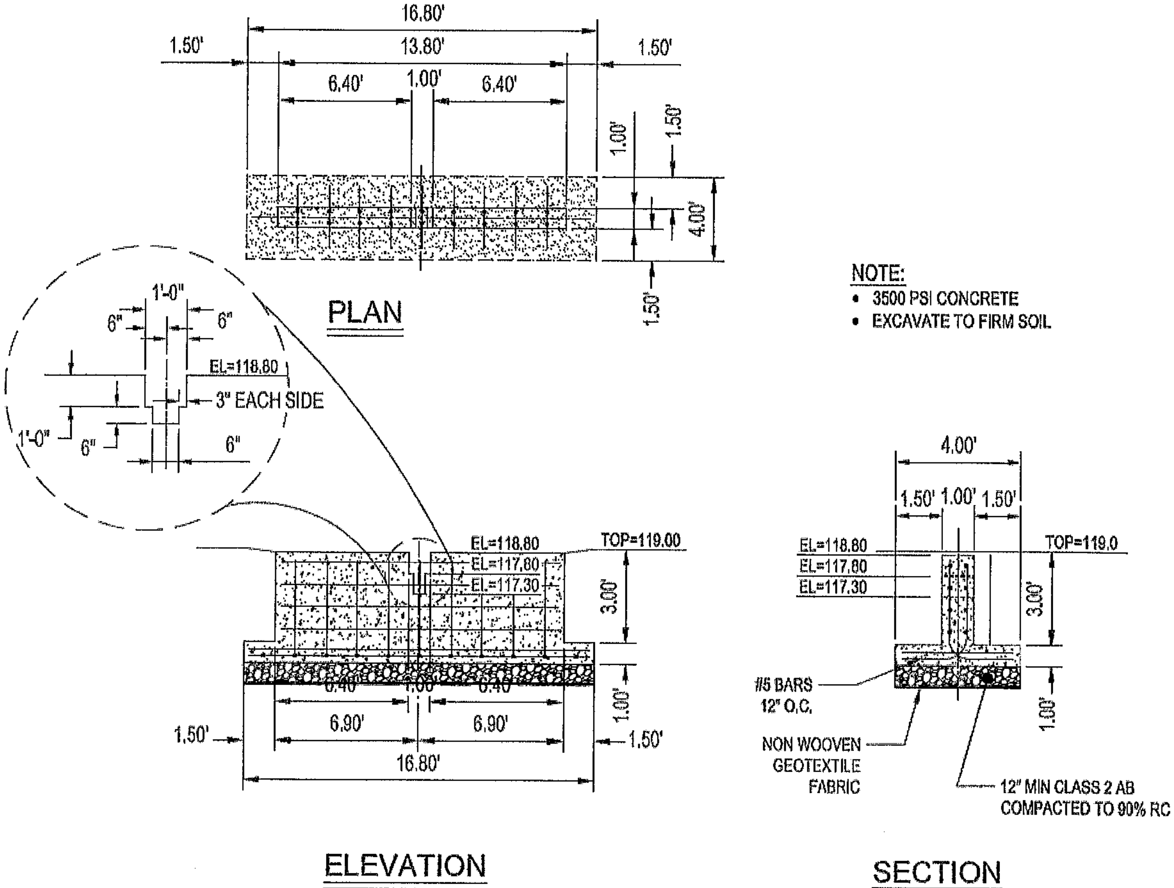
FIGURE 1

1. 3. MODIFICATIONS TO THE PRE-LIMINARY DRAINAGE REPORT

The pre-liminary drainage report remains unchanged except for one design consideration in Section 3.2 Outlet and Overflow Structure Design. This section details the overflow connection into the existing DI and as shown in the preliminary drainage report and it has been identified as a potential clogging situation presenting challenges with maintenance.

The revised approach incorporates a notched concrete weir (Exhibit B) at the outlet from the drainage basin. The sizing of all drainage facilities remains as called on in the preliminary report and the low flow and high flow overflow elevations of the weir and the outlet structure designs are shown below.

Weir Detail at Basin:



Outlet Structure Detail:

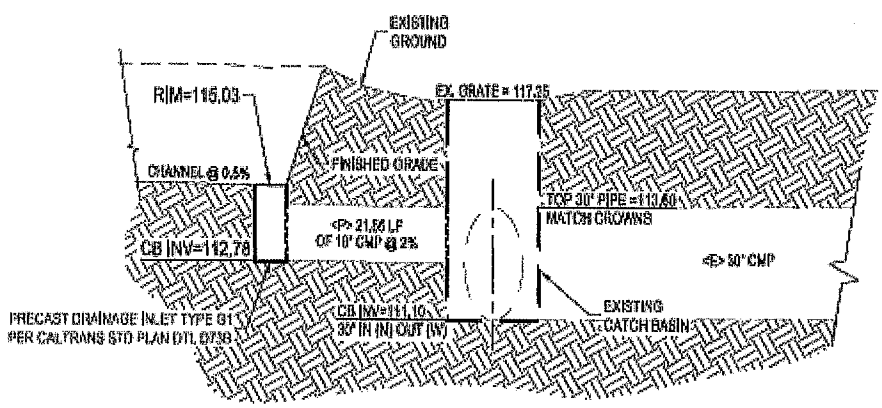
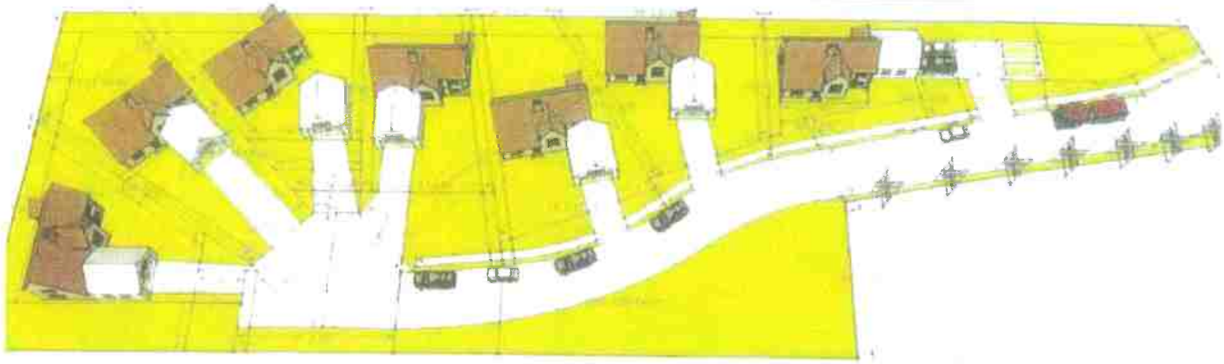


Exhibit A

Preliminary Drainage Report for the

Brookview Tract

a Planned Unit Development



Located in:
Mckinleyville, California
APN: 510-121-026

Owner:
Lynn Pettlon
1417 Railroad Avenue
Mckinleyville, CA 95519

Prepared by:

REGISTERED PROFESSIONAL ENGINEER
MAX A. SCHILLINGER
78866
CIVIL
STATE OF CALIFORNIA

Max Schillinger, P.E.
PO Box 4207
Palmer, AK 99645

March 2013

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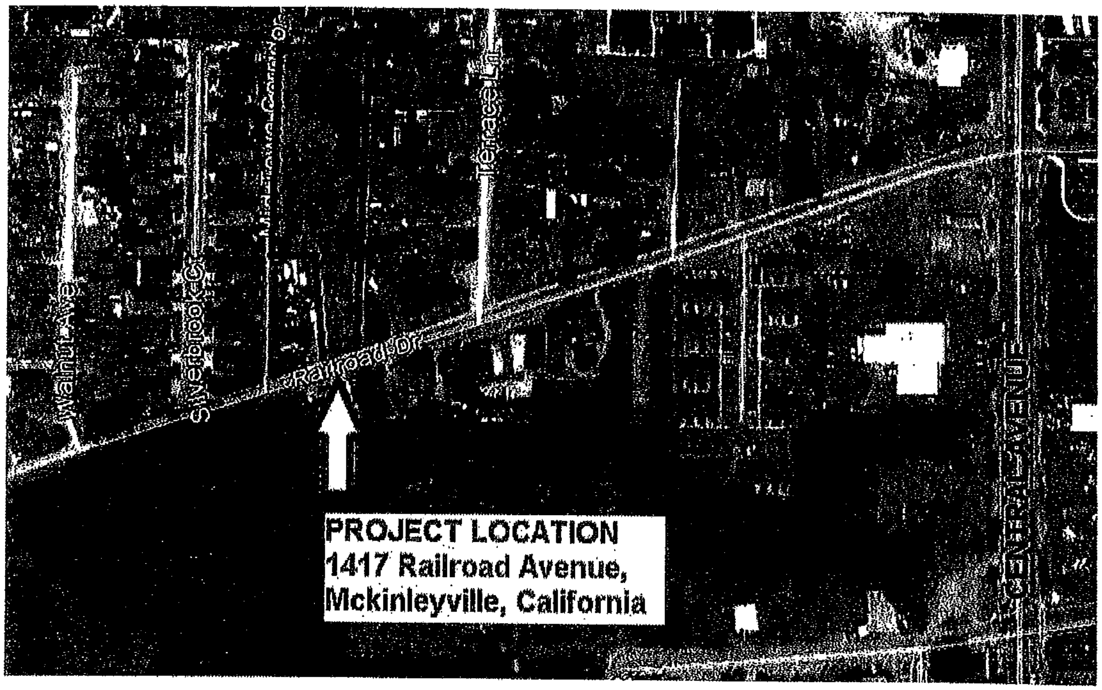
G Grate Spacing
H Orifice and Weir Charts
I Time of Concentration References

1.0 INTRODUCTION

This section presents general information about the proposed subdivision, and the existing project topography and geology.

1.1 Project Location

The project site is located in unincorporated town of McKinleyville, Humboldt County, California. It is located just west of the Central Avenue at 1417 Railroad Avenue. See the Location Map below. The project is on AP No. 510-121-026.



Location Map: not to scale

1.2 Project Description

This project develops a vacant 1.58-acre parcel into 7 residential lots, 1 access road, and 1 stormwater detention basin/bioswale area. The lots are sized between 5000 and 11000 square feet. This project will require grading, construction of underground utilities, roads, detention basin and other infrastructure. See Appendix A-Tentative Map by Points West Surveying Company.

1.3 Project Topography and Geology

The 1.58-acre parcel is surrounded by residential development, including a paved trail on the west side of the project. The existing topography gently slopes northwesterly (appx 1%). There is an existing drainage inlet at the northwest corner of the parcel, to which existing and future drainage will flow.

The soil onsite is currently not explored. However, the soil onsite is expected to be similar to that of neighboring Central Terrace Estates Subdivision, Terrace Estates Planned Unit Delevopment, and Shadowbrook Subdivision. These subdivisions all yielded thick rich layer of sod and topsoil (appx 2' deep) followed by light brown sandy soil below. Although there is little standing water nor any wetland areas are evident onsite, seasonal perched groundwater is expected to be present at shallow depths (5-6 feet), as was encountered at Terrace Estates.

Typical percolation rate for this native sandy soil is around 30-60 minutes per inch. However, compaction by construction equipment, and silt sedimentation during the construction process can greatly compromise this percolation rate.

Project Site, view southeast from northwest corner of project

2.0 HYDROLOGY ANALYSIS

This section presents general hydrology calculations for pre-development and post-development.

2.1 Hydrology Introduction

The County of Humboldt requires that macro hydrology be analyzed for subdivision development. The purpose of this requirement is to reasonably verify that onsite development doesn't adversely effect the offsite watershed as a whole.

To accomplish this goal, calculations for the Pre-Development 2-year storm (Q2) and Post development 100-year storm (Q100) are made. The Q2 flow calculations are used to size drainage structures for the allowable discharge during common storm events of the subdivision (typically via basin orifice flow). The Q100 calculations are used to size drainage structures for high flow capacity. In accordance to County policies, the difference in flow between these two values is detained.

For flow calculations, the Rational Method was used: $Q = CiA$

Where:
- Q = Flow (cfs)
- C = Runoff Coefficient (=0.25 ag. land, 0.88 pavement, see Appendix D)
- I = Rainfall intensity (in/hr), which is dependent on frequency of event, duration, and time of concentration. (See Appendix B)
- A = DrainageArea (acres)

For the small drainage area, a minimum time of concentration (Tc) of 10 minutes was selected by engineering convention. The calculations of time of concentration for small sites vary greatly according to formulae used. See Appendix I for a summary of Tc formula.
Thus given a Tc of 10 minutes, rainfall intensities per Appexdix B are:

I2 = 2 year storm intensity = 1.25in/hr
I100 = 100-year storm intensity = 3.2 in/hr

2.2 Pre-Development Flow Calculations

For pre-development 2-year flow analysis, a runoff coefficient of 0.25 (agricultural land) was selected.

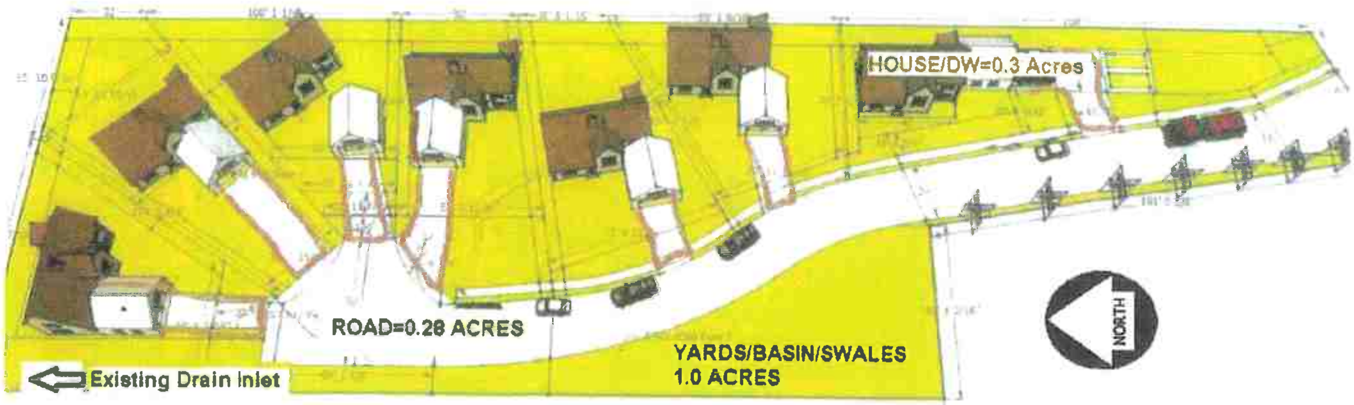
Macro Onsite Pre-Development Flow

Q2 onsite =	C(pre)	* I(pre)	* A		
	0.25	*1.25	*1.58	=	**Q2 onsite= 0.5 cfs**

Note that this 0.5cfs is the amount allowable that can be released offsite in Post-Development conditions.

2.3 Post-Development Flow Calculations

For post-development 100-year flow analysis, a weighted runoff coefficient was considered:
As seen on the Post-Development Concept Drawing below, the site will be development into three major areas: 1) impervious houses and driveways, 2) paved road, and 3) vegetated yards/basins/swales. Common runoff coefficients are shown in Appendix B.



Post-Development Concept Drawing: not to scale

1) For the houses and driveways area, a footprint of 1900 s.f. per lot, or 0.30 acres for all 7 lots. A runoff coefficient of 0.95 was selected.

2) For the paved road area, the 12,000 s.f. footprint, or 0.28 acres was used. A runoff coefficient of 0.85 was selected.

3) The remaining area is 1 acre. (=1.58 total -0.30-0.28) This area will be vegetated yards/basins/swales. A runoff coefficient of 0.20 was selected.

Macro Post Development Flow Table:

Sub Area	Acreage	C factor	C*A
Houses/Driveways	0.30 Acres	0.95	0.29
Paved Road	0.28	0.85	0.24
Vegetated	1.00 Acres	0.25	0.25
Totals	1.58 Acres		0.78→ thus weighted C = 0.723/1.58 = 0.49

→ use weighted coefficient of 0.5

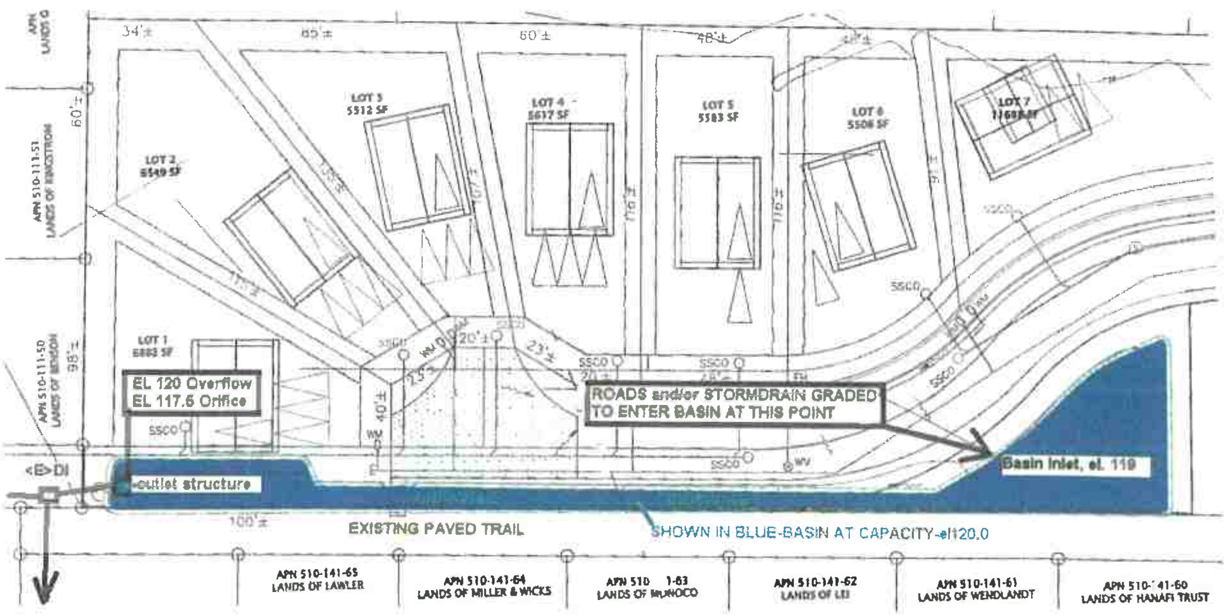
Q100 (post)	=	C(post)	*I(post)	*A		
		0.5	*3.20	*1.58	=	**Q100 (post) = 2.5 cfs**

Note that this 2.5 cfs is the minimum the onsite drainage improvements need to be able to handle for overflow conditions.

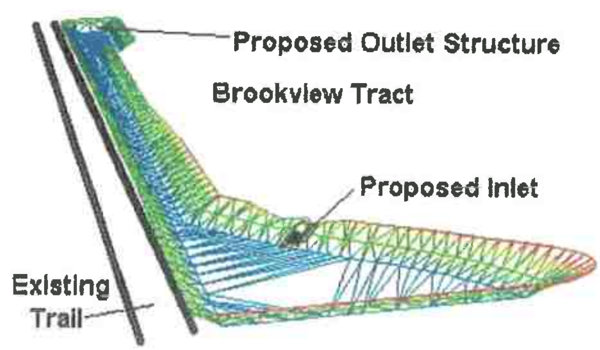
3.0 PROPOSED DRAINAGE IMPROVEMENTS

This section outlines guidelines and designs for several drainage improvements. The subdivision will feature roads and lots with runoff directed towards a basin, bioswale, and outlet structure. As the improvement plans are not finalized at this time, exact grades of roads, valley gutters, and drainage inlets are not established. However, this section of the report outlines several drainage improvements to verify that the proposed subdivision will be compliant with County drainage criteria.

Below are proposed improvements which will be discussed in the following pages:



Basin and Outlet Structure Plan: not to scale



Basin and Outlet Model Isometric View: looking northerly

3.1 Basin Storage Capacity Required and Provided

Per section 2.2 and 2.3, Q2 is 0.5 cfs and Q100 is 2.5 cfs.

Basin Sizing Estimate
(By Triangular Method – aka Skupe Method, assuming 10min Tc = 600 seconds)

Volume =	[K	*(Q100-Q2)	* (3Tc)]/2		
	[1.5	*(2.5-0.5)	* (3*600)]/2	=	**Vrequired = 5400 cf**

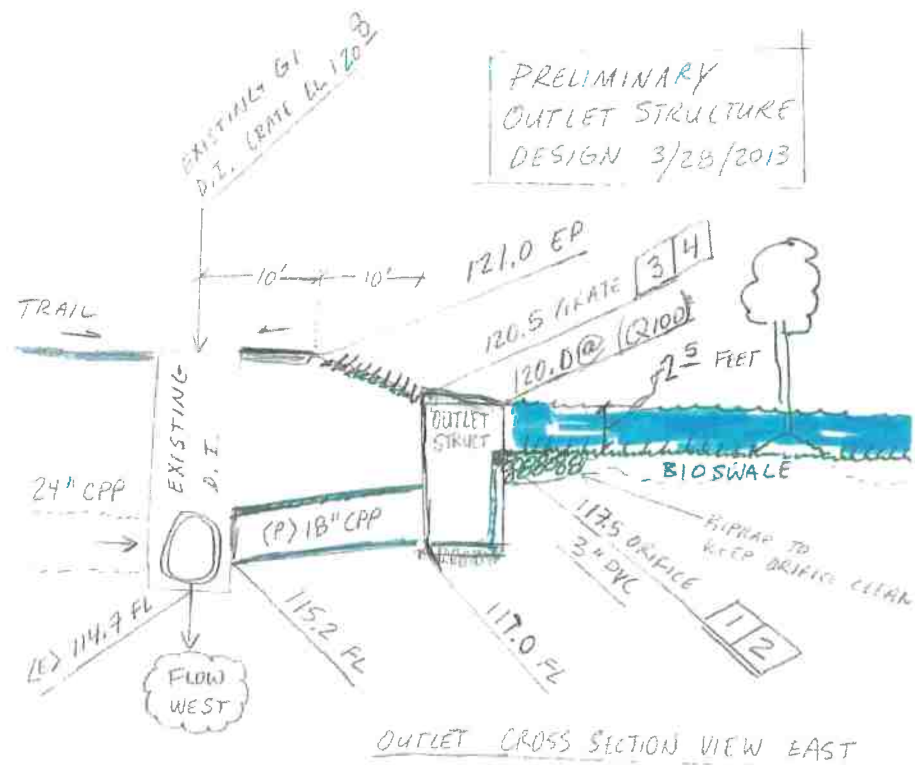
The Basin shown on the previous page has the following stage storage levels:

Elevation	Quantity (cu.ft)	Remarks
117.5	0	3" Orifice FL Elevation
118.0	210	
118.5	610	
119.0	2,160	Water Reaches south end of Basin
119.5	3,970	
120.0	**6,130**	**Vprovided at Bottom Overflow Grate**
120.5	8,550	Top of Overflow Grate---Basin Max. Capacity
120.8	flooding	Grate Elevation of Existing D.I.

From the above table, it is apparent that sufficient storage is provided for the subdivision.

3.2 Outlet and Overflow Structure Design

The Outlet Structure modulates storm flow to a 2 year value. The basin overflow allows the 100 year storm event to bypass the basin in a controlled manner. Proposed is a modified type G1 Drainage Inlet, including a 3" orifice, as the outlet structure of the project.



Outlet Structure Cross Section, view east: not to scale

This Overflow Structure will has the following specifications:

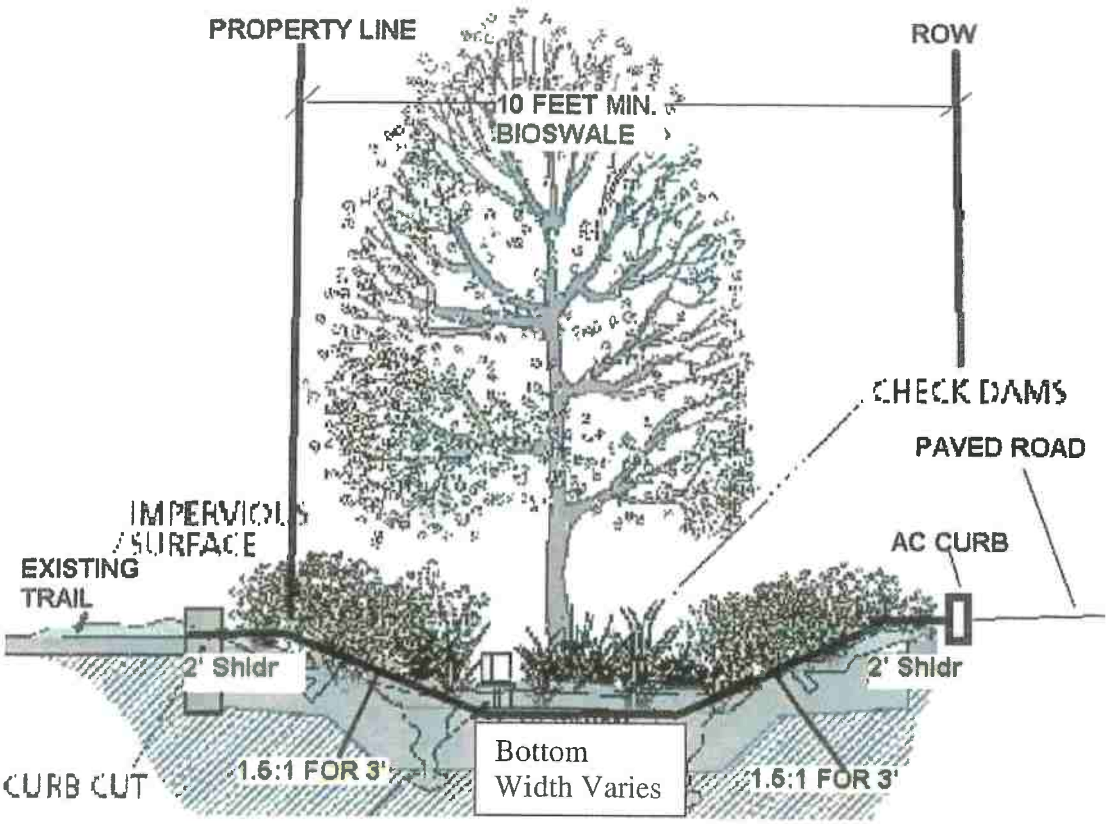
1. Basin outlet Q2 low flow discharge required = 0.5 cfs, from section 2.2.
2. Diameter of low flow orifice selected to provide low flow = 3 inch PVC

   (Calculated from Orifice Formula, see Appendix H)
3. Outlet structure overflow elevation = 120.0 ft
4. Outlet structure overflow capacity = 10 cfs (note only 2.5 cfs req'd per section 2.3)

   (Calculated by Grate Capacity, see Section 3.5 and Appendices F & G)
5. Note: Storage volume (El 117.5 to El. 120.0) = 6130 cu.ft. (5400+ req'd)

3.3 Bioswale Capacity

The section below will be used for the swale:



Given this two +/- foot deep swale will be designed with 1' natural check dams, the actual overflow capacity available is 1 foot. Using manning equation, assuming a minimum of 1 foot water depth, 0.5% slope, and rough grass n=0.03, and varying bottom widths:

$Q=(1.49/n)$ * A * $R^{2/3}$ * $S^{1/2}$

Try 2' Bottom width:

$Q=(1.49/0.03)$ 3.5* $(0.62)^{2/3}$* $(0.005)^{1/2}$ = **Min. Qswale=9 cfs.**

Assume 0' Bottom width:

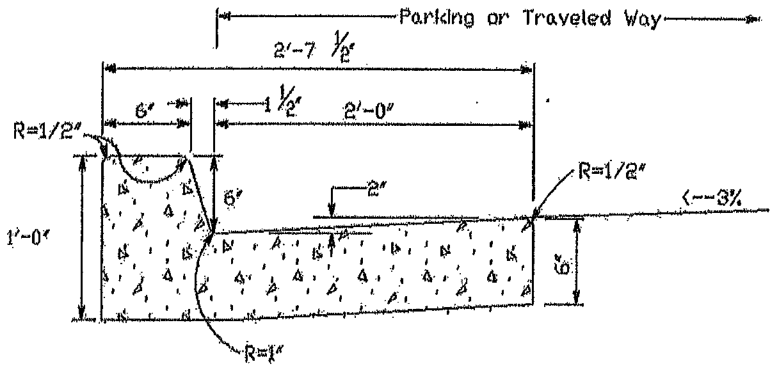
$Q=(1.49/0.03)$ 1.5* $(0.42)^{2/3}$* $(0.005)^{1/2}$ = **Min. Qswale=3 cfs.**

Note: Minumum Qrequired = Q100 2.5 cfs→Thus 0' Bottom width OK!

Since flow velocity will be under 3 ft per second at 0.5%, natural vegetation should be sufficient to prevent erosion, and no turf reinforced mats are specified at this time. However, other rolled erosion control products may be desirable on the 1.5:1 sideslopes to prevent erosion.

3.4 Gutter flow calculations

The purpose of this section is to analyze how far water could flood parking or travelled way lanes. Ideally, Q100 flow floods at most the parking lane (8'). Q100 also cannot overtop the A2-6 curb. The project has this A2-6 curb typical section:



From the section above, the flow table was created using Manning's Equation.

$$V = \frac{k}{n} R_h^{2/3} \cdot S^{1/2}$$

Where:

- V is the cross-sectional average velocity (ft/s, m/s)
- k is a conversion constant equal to 1.486 for U.S. customary units
- n is the **Manning coefficient**, =0.011 for concrete, and 0.015 for pavement
- R_h is the hydraulic radius (ft, m)
- S is the slope of the water surface in the direction of road travel

The table below shows flow values associated with various curb ponding depths:

		Street Steepness in direction on travel (ft/ft)					
Ponding Depth	Ponding Width (ft)	0.005	0.01	0.02	0.03	0.04	0.05
1"	1	1.0	1.4	2.0	2.4	2.8	3.1
2"	2	2.0	2.8	4.0	4.9	5.6	6.3
3"	4.8	2.2	3.1	4.3	5.3	6.1	6.9
4"	7.6	3.1	4.4	6.3	7.7	8.9	9.9
5"	10.3	5.4	7.6	10.7	13.1	15.2	16.9
		Q (cfs) resulting from depths and slopes					

For this project, the Q100 was calculated to be 2.5 cfs. It is apparent that even with a Street Steepness of 0.5%, this amount will have a ponding width under 8 feet, and a ponding depth under 4". Thus, even though the roads are not designed at this point in the project, the can be designed at a 0.5% minimum grade and have enough capacity to not flood driving lanes.

3.5 Inlet Capacity Calculations

Although the improvement plans are not yet finalized for the subdivision, Caltrans type G0 drainage inlets with A2-6 Curb and Gutters are commonly used. Per Appendix F, the capacity of those drainage inlets at a depth of 0.5' (curb depth) is 10.6 cfs. This capacity exceeds the individual Q100 in flow for the entire subdivision, so no further analysis was required.

Also note that for the outlet structures, typically Caltrans Type 24-9x Grate is prescribed, as it clogs less than grates designed for bicycle traffic. See Appendix G. Due to the close proximity this outlet structure is to a public trail, a bicycle rated grate may be prudent. The grate capacity is expected to exceed the Q100 required.

3.6 Pipe Capacity Calculations

Although the improvement plans are not yet finalized for the subdivision, 12" and 18" corrugated plastic pipes may be required to connect drain inlets. Assuming a manning n of 0.012 for smooth wall plastic pipe, and a minimum slope of 0.5%:

$Q=(1.49/n) * \quad A * \quad R^{2/3} * \quad S^{1/2}$

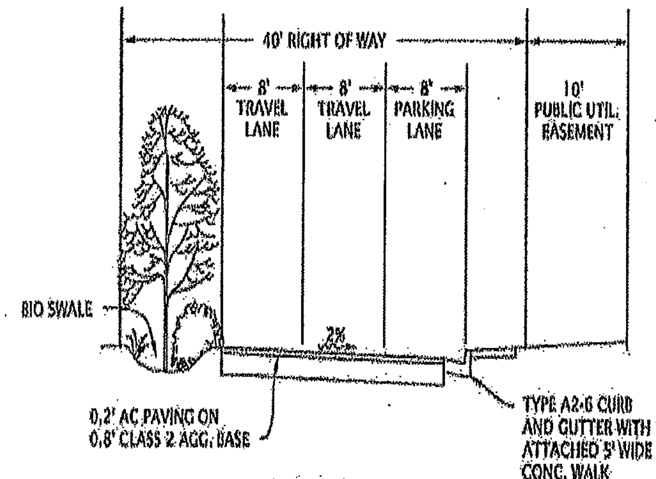
For 12" CPP (1'diameter)

$Q=(1.49/0.012) \quad 0.785* \quad (0.25)^{2/3}* \quad (0.005)^{1/2} =$ **12" CAPACITY = 2.8 cfs.**

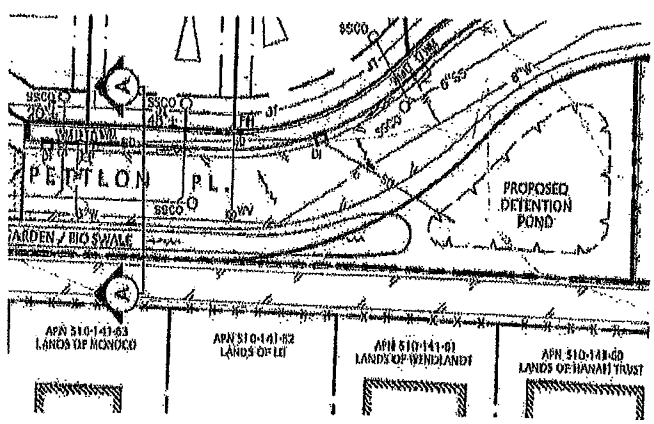
Even though the 12" pipe can carry Q100 for this subdivision, 18" pipes are preferred as they are able to be cleaned and allow room for deposition of sediment. See also Appendix C for an Inlet Control Nomograph for pipe capacities at varying headwater depths.

3.7 Road and Other Improvement Details

The lots and road will be constructed such as to drain towards the basin, at the location labeled as "Basin Inlet" on the plan shown on sheet 7. How the water drains into the basin from the road is not yet finalized, but several alternatives exist. One option depicted on the typical map included the following road section and plan:



SECTION A-A
NO SCALE



As shown, Pettlon Place is graded right (east) and into stormdrain system and into the basin. As outlined in this report, 12" storm drain pipe would have enough capacity for the subdivision. One potential problem with this system is that cover for the stormdrain pipes may insufficient (typ. 1' min) depending on the final road grades. Other alternatives exist, including but not limited to:

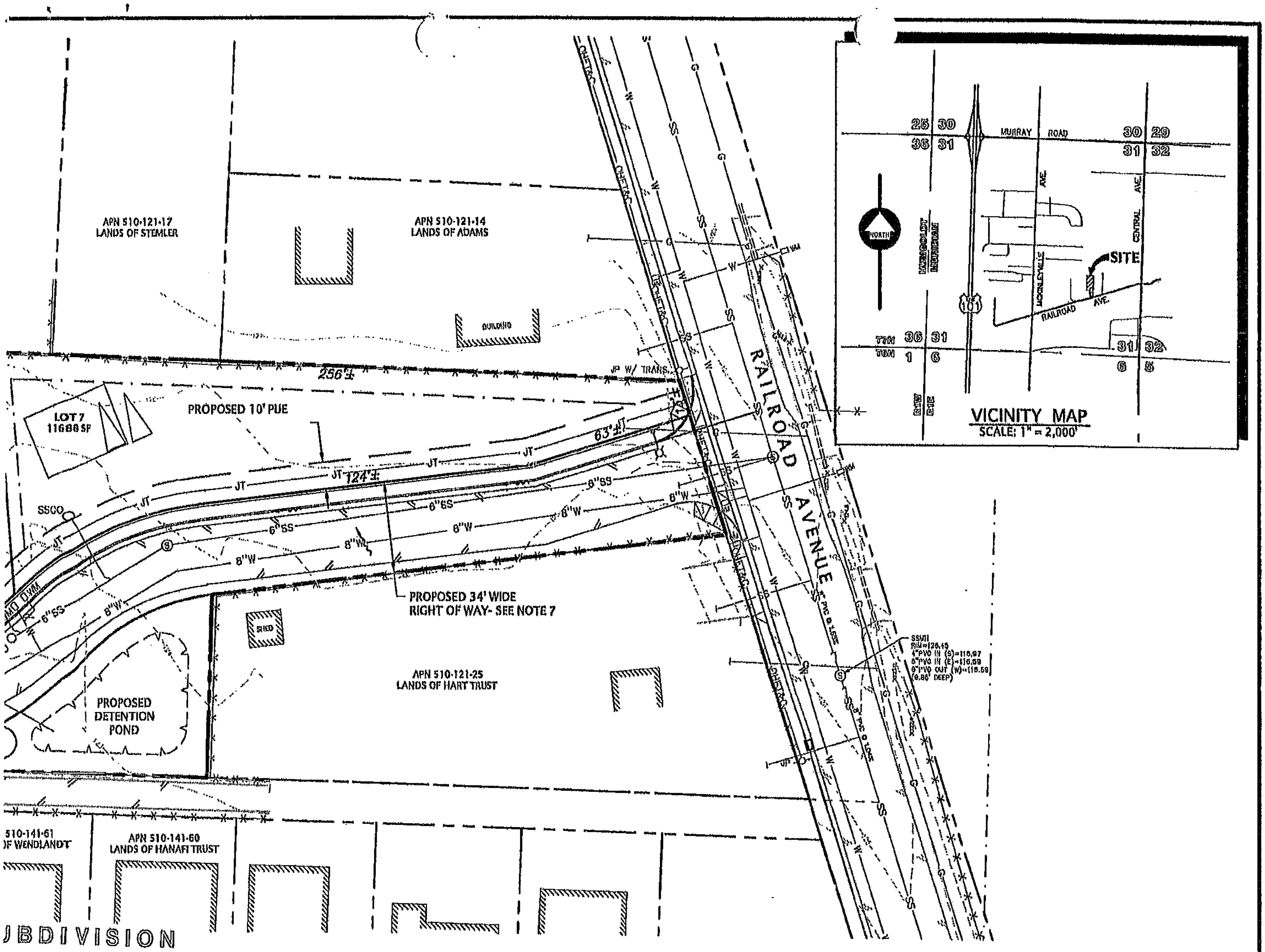
(1) Using section A-A, but at the position shown for the stormdrain crossing Pettlon Place, install a valley gutter system graded left (west), including a riprap swale entering the basin.

(2) Instead of section A-A, use a typical section that is graded left (west) at 2%. Install a Caltrans type A1-6 Curb on the right (east) side of the road, and A2-6 on the left (west) side of the road. At the position of the basin inlet, end the curb and have a riprap swale entering the basin.

3.7 Conclusions

This report has demonstrated that the preliminary layout shown on the tentative map can accommodate drainage improvements which mitigate adverse offsite drainage impacts. The basin outlined in this section complies with the typical storage capacity required by the county. The bioswale and other onsite best management practices should enhance natural percolation of water within the basin system, thereby reducing the demands required of the outlet structure.

Other specific drainage details, such as road drainage inlets and/or valley gutters, road profiles will be outlined in the Improvement Plans for the project.



Notes:

1) This map represents a subdivision of the lands described in Document No. 1998-4854 into seven parcels. Total gross area of property is 1.58 acres.

2) The existing boundary lines and easements are based on a Title Report prepared by Humboldt land Title Company, Order No. 00142882-DS, dated 7/23/2012. All easements of record and new easements show on the Tentative Map and will show on the Final Map.

3) The property is not shown on official maps to be subject to flooding per Firm Community Panel No. 060060 0625C.

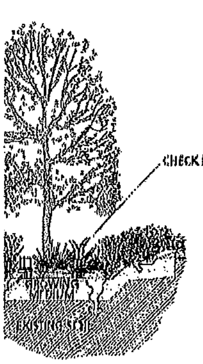
4) Utilities:

Water and Sewer	McKinleyville Community Services District
Electric and Gas	Pacific Gas and Electric
Telephone	AT&T
Cable TV	Sudden Link

5) Topography is shown at 1 foot contour intervals based on field survey performed by Points West Surveying in January 2013.

6) The existing property is vacant. The proposed use is residential single family residences. The existing use on adjacent parcels is residential.

7) The proposed Planned Unit Development presented proposes a right of way of varying width. The entry road is proposed to be a 34 foot right of way widening to a forty foot right of way as shown. Additionally, the front setback is proposed to be ten feet from the right of way to allow flexibility in home citing and to facilitate solar exposure.



BIO-SWALE SECTION
NO SCALE

PROJECT DATA

Owner / Applicant:	Lynn Pettion
APN:	510-121-026
Address:	1417 Railroad Avenue McKinleyville, CA 95519
Agent:	David Crivelli Points West Surveying
General Plan:	RL (MCCP) Residential Low Density
Prinicipal Zoning:	R-1 Residential Single Family 5000 sf minimum lot size
Building Setbacks:	Front: 10' Interior Side: 5' Exterior Side: 5' Rear: 10'

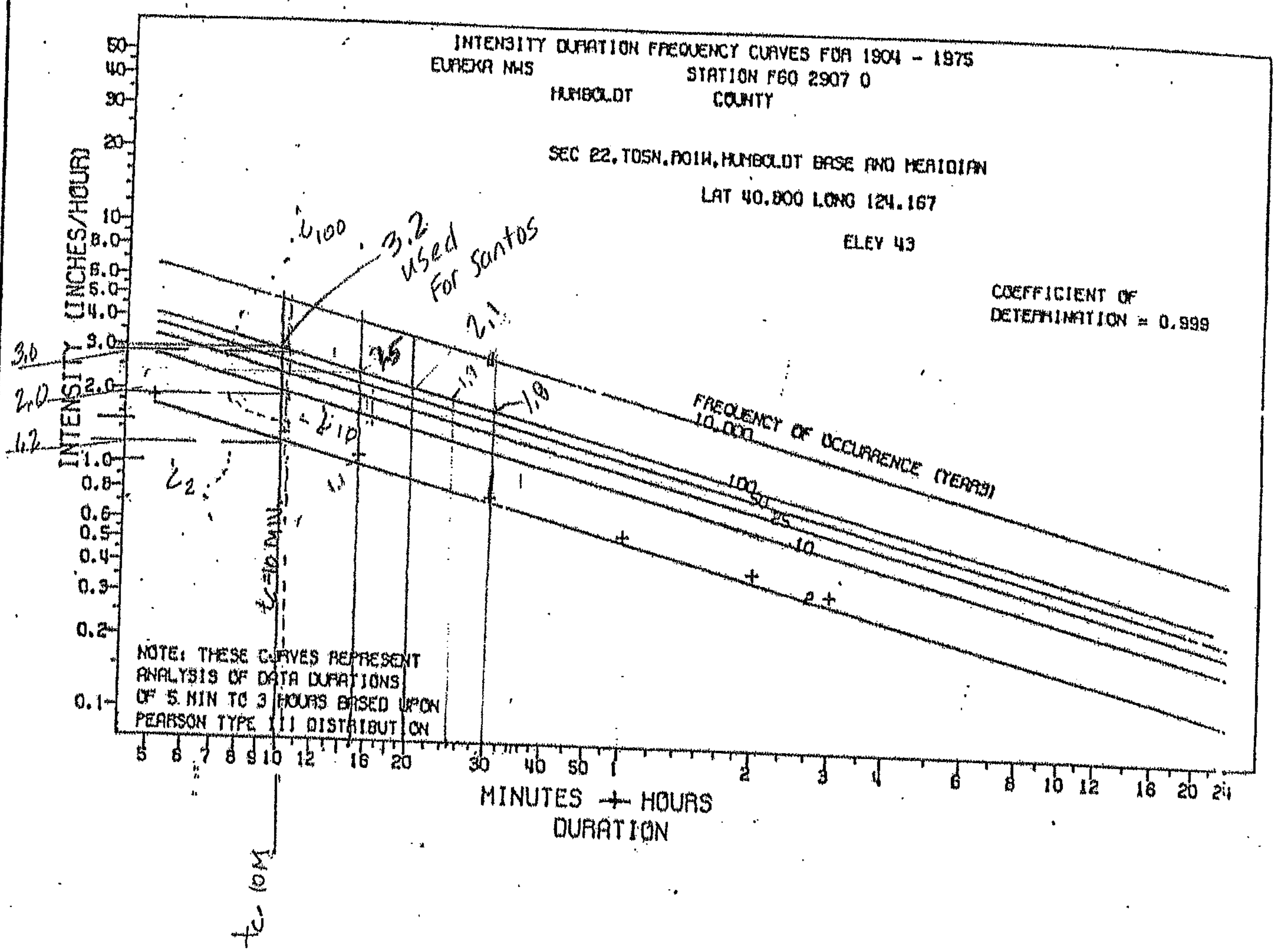
APN 510-121-026
TENTATIVE MAP
for
Brookview Tract

A PLANNED UNIT DEVELOPMENT
SECTION 31, T7N, R1E,
HUMBOLDT MERIDIAN

IN THE UNINCORPORATED AREA OF
HUMBOLDT COUNTY, STATE OF CALIFORNIA
SCALE: 1" = 30' FEBRUARY 2013 SHEET 1 OF 1

POINTS WEST SURVEYING CO.
5201 Carlson Park Dr., Suite 3 - Arcata, CA 95521
707-840-9510 - Phone 707-840-9542 - Fax

APPENDIX B



t_c = 10 MIN;
FIND i_2, i_{10}, i_{100}

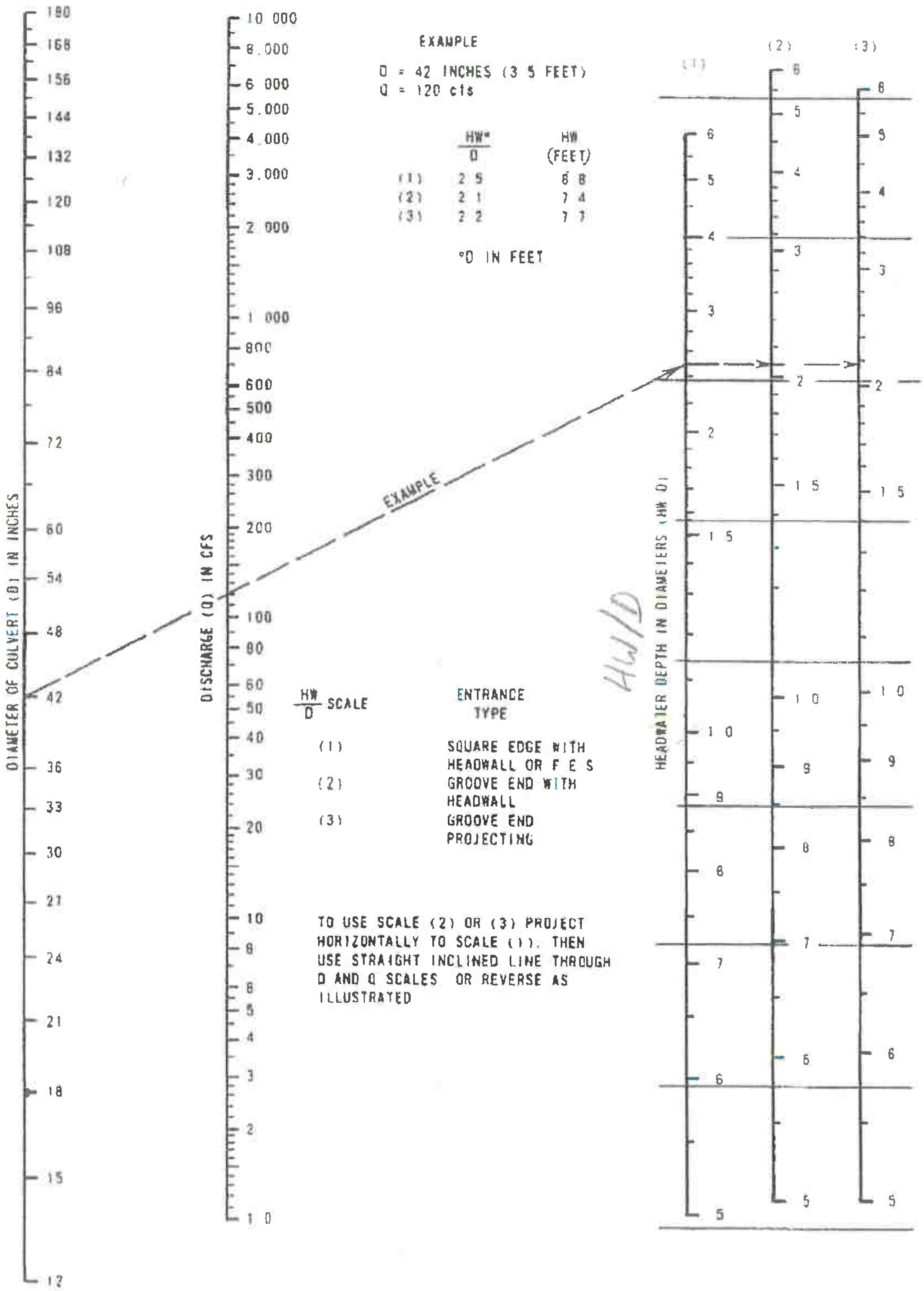
From: "Rainfall Analysis For Drawing Design Vol. I." Bulletin 195, Oct. 1976. California Department of Water Resources.

INTENSITY DURATION FREQUENCY CURVES - 1904 - 1975

B IDF Curve for Eureka, CA 1(b)

Inlet Control Nomograph for Concrete Pipe

APPENDIX C



APPENDIX D - Rational Method C Coefficients

categorized by (face

NOTE: FOR THIS PROJECT, C=0.85 WAS SELECTED FOR ROAD AREAS, C=0.95 WAS SELECTED FOR HOUSE/DRIVEWAY AREAS.

forested	0.059–0.2
asphalt	0.7–0.95
brick	0.7–0.85
concrete	0.8–0.95
shingle roof	0.75–0.95
lawns, well-drained (sandy soil)	
up to 2% slope	0.05–0.1
2% to 7% slope	0.10–0.15
over 7% slope	0.15–0.2
lawns, poor drainage (clay soil)	
up to 2% slope	0.13–0.17
2% to 7% slope	0.18–0.22
over 7% slope	0.25–0.35
driveways, walkways	0.75–0.85

categorized by use

NOTE: COMMON USAGE IS C=0.25 FOR MCKINLEYVILLE PASTURE/AGRICULTURAL LAND

C=0.20 FOR LAWNS AND GREENBELT

farmland	0.05–0.3
pasture	0.05–0.3
unimproved	0.1–0.3
parks	0.1–0.25
cemeteries	0.1–0.25
railroad yards	0.2–0.35
playgrounds (except asphalt or concrete)	0.2–0.35
business districts	
neighborhood	0.5–0.7
city (downtown)	0.7–0.95
residential	
single family	0.3–0.5
multiplexes, detached	0.4–0.6
multiplexes, attached	0.6–0.75
suburban	0.25–0.4
apartments, condominiums	0.5–0.7
industrial	
light	0.5–0.8
heavy	0.6–0.9


APN 510-121-17
LANDS OF STEMLER
APN 510-121-14
LANDS OF ADAMS
LOT 7
11688 SF
256'±
PT = 0+91.05
PC = 0+74.61
63'±
124'±
APN 510-121-25
LANDS OF HART TRUST
510-141-61
APN 510-141-60
LANDS OF HANAFI TRUST

APPENDIX F

COUNTY OF HUMBOLDT – DEPARTMENT OF PUBLIC WORKS		
QUANTITY CALCULATIONS		
DC-CEM-4801 (OLD HC-52 REV.11/92) 7451-3520-0	SHEET 1 OF 2	
JOB STAMP	ITEM	FILE NO.
WEIR FLOW VERSUS ORFICE FLOW CONTROLLING CONDITIONS & FLOWRATE	LOCATION	SEGREGATION YES NO
	CALC. BY DPJ	DATE 07-15-2010
	CHK. BY	DATE

4.4.5.1. Grate Inlets in Sags

A grate inlet in a sag location operates as a weir to depths dependent on the size of the grate and as an orifice at greater depths. Grates of larger dimension will operate as weirs to greater depths than smaller grates.

see STD PLAN (CALTRANS) D77A, D77B

The capacity of grate inlets operating as weirs is:

use for DI's @ curbs

$$Q_i = C_w P d^{1.5} \qquad (4\text{-}26)$$

1.56 clear

3'4"

where:

P = perimeter of the grate in m (ft) disregarding the side against the curb
C_w = 1.66 (3.0 in English units)
d = average depth across the grate; 0.5 ($d_1 + d_2$), m (ft)

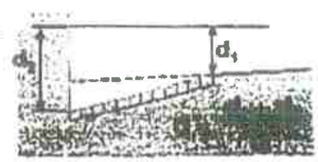


Figure 4-17. Definition of depth.

24-9
SPACING 2"
Q = 0.67 * 5.25

The capacity of a grate inlet operating as an orifice is:

$$Q_i = C_o A_g (2 g d)^{0.5} \qquad (4\text{-}27)$$

where:

C_o = orifice coefficient = 0.67
A_g = clear opening area of the grate, m² (ft²)
g = 9.81 m/s² (32.16 ft/s²)

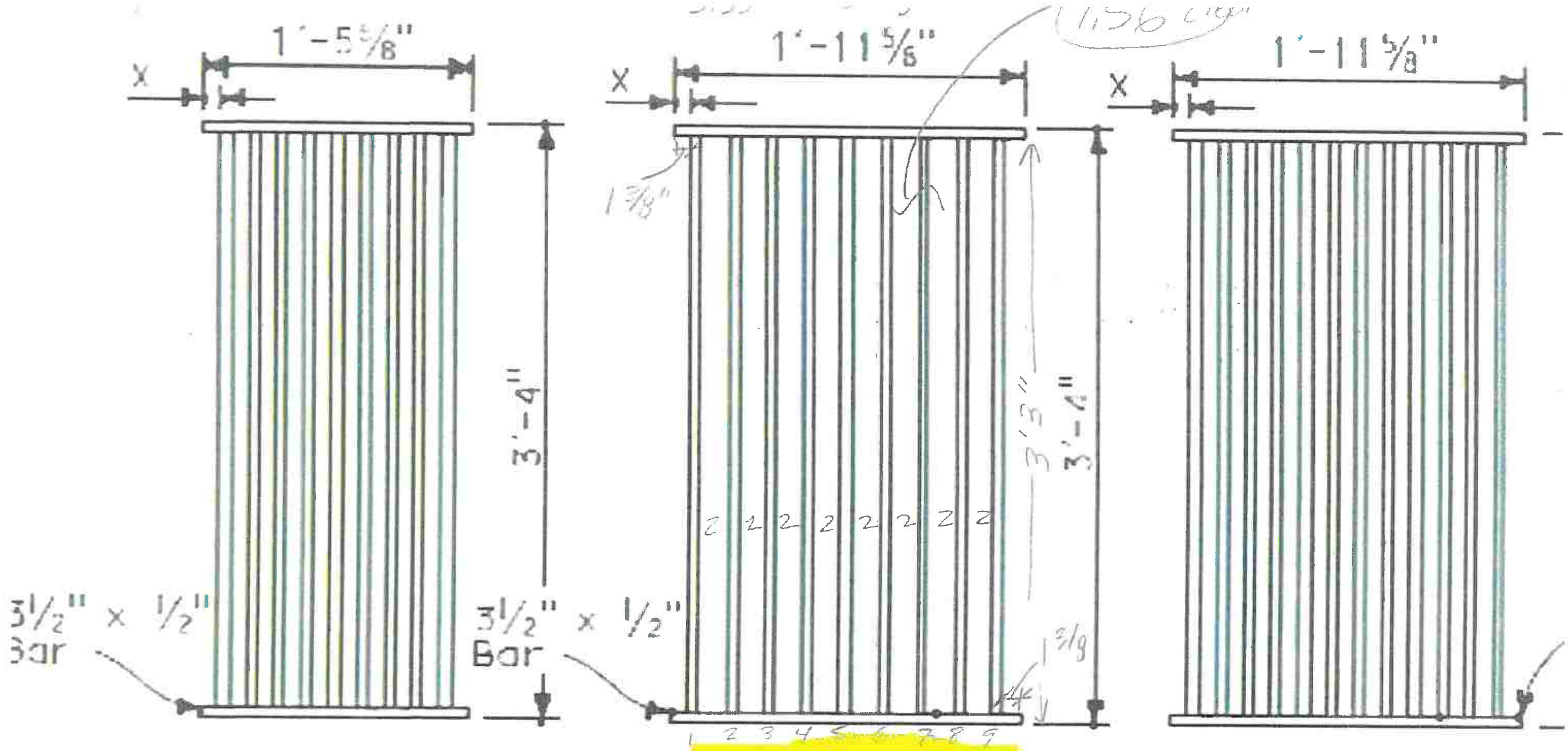
TPE 24-9
$A_g = 1.56 \times 3.33 = 5.20$

use for outlet struct basin
$A_g = 5.20$

WEIR FLOW VERSUS ORFICE FLOW CONTROLLING CONDITIONS & FLOWRATE - COMB. INLET									
	WEIR FLOW				ORFICE FLOW				
DEPTH,d	C_W	P	$d^{1.5}$	Q_i	C_O	A_g	g	$(2gd)^{0.5}$	Q_i
[ft]		[ft]		[cfs]		[ft²]	[ft/sec²]		[cfs]
0.1	3.0	10.0	0.032	0.95	0.67	4.875	32.2	2.538	8.29
0.2	3.0	10.0	0.089	2.68	0.67	4.875	32.2	3.589	11.72
0.3	3.0	10.0	0.164	4.93	0.67	4.875	32.2	4.395	14.36
0.4	3.0	10.0	0.253	7.59	0.67	4.875	32.2	5.075	16.58
0.5	3.0	10.0	0.354	10.61	0.67	4.875	32.2	5.675	18.53
0.6	3.0	10.0	0.465	13.94	0.67	4.875	32.2	6.216	20.30
0.7	3.0	10.0	0.586	17.57	0.67	4.875	32.2	6.714	21.93
0.8	3.0	10.0	0.716	21.47	0.67	4.875	32.2	7.178	23.44
0.9	3.0	10.0	0.854	25.61	0.67	4.875	32.2	7.613	24.87
1	3.0	10.0	1.000	30.00	0.67	4.875	32.2	8.025	26.21
1.1	3.0	10.0	1.154	34.61	0.67	4.875	32.2	8.417	27.49
1.2	3.0	10.0	1.315	39.44	0.67	4.875	32.2	8.791	28.71
1.3	3.0	10.0	1.482	44.47	0.67	4.875	32.2	9.150	29.89
1.4	3.0	10.0	1.657	49.70	0.67	4.875	32.2	9.495	31.01
1.5	3.0	10.0	1.837	55.11	0.67	4.875	32.2	9.829	32.10

Q = 0.67 x 5.2 * (5.675) = 19.8 cfs

APPENDIX G - Grate spacing



TYPE 18-9

1⅜" Clear spacing. Use within the roadbed on highways where bicycles and pedestrians are excluded.

TYPE 24-9

2" Clear spacing. Use in locations off the roadbed on all types of highways.

3.36 × 1.56 = Ag = 5.2

TYPE 24-12

1⅜" Clear spacing. Use within the roadbed on highways where bicycles and pedestrians are excluded.

RECTANGULAR GRATE DETAILS

(See table below)

APPENDIX I

TABLE 1 Summary of Time of Concentration Models

Publication and Year	Equation for Time of Concentration (min)	Remarks
Williams (1922) (*6*)	$t_c = 60LA^{0.4}D^{-1}S^{-0.2}$ L = basin length, mi A = basin area, mi^2 D = diameter (mi) of a circular basin of area S = basin slope, %	The basin area should be smaller than 50 mi^2 (129.5 km^2).
Kirpich (1940) (*7*)	$t_c = KL^{0.77}S^{y}$ L = length of channel/ditch from headwater to outlet, ft S = average watershed slope, ft/ft For Tennessee, $K = 0.0078$ and $y = -0.385$ For Pennsylvania, $K = 0.0013$ and $y = -0.5$	Developed for small drainage basins in Tennessee and Pennsylvania, with basin areas from 1 to 112 acres (0.40 to 45.3 ha).
Hathaway (1945) (*8*), Kerby (1959) (*9*)	$t_c = 0.8275\,(LN)^{0.467}S^{-0.233}$ L = overland flow length, ft S = overland flow path slope, ft/ft N = flow retardance factor	Drainage basins with areas of less than 10 acres (4.05 ha) and slopes of less than 0.01.
Izzard (1946) (*10*)	$t_c = 41.025(0.0007i + c)L^{0.33}S^{-0.333}i^{-0.667}$ i = rainfall intensity, in./h c = retardance coefficient L = length of flow path, ft S = slope of flow path, ft/ft	Hydraulically derived formula; values of c range from 0.007 for very smooth pavement to 0.012 for concrete pavement to 0.06 for dense turf.
Johnstone and Cross (1949) (*11*)	$t_c = 300L^{0.5}S^{-0.5}$ L = basin length, mi S = basin slope, ft/mi	Developed for basins with areas between 25 and 1624 mi^2 (64.7 and 4206.1 km^2).
California Culvert Practice (1955) (*12*)	$t_c = 60(11.9L^3/H)^{0.385}$ L = length of longest watercourse, mi H = elevation difference between divide and outlet, ft If expressed as $T_c = kL^a n^b S^{-y} i^{-z}$ format: $t_c = KL^{0.77}S^{-0.385}$ K = conversion constant	Essentially the Kirpich (*7*) formula; developed for small mountainous basins in California.
Henderson and Wooding (1964) (*13*)	$t_c = 0.94(Ln)^{0.6}S^{-0.3}i^{-0.4}$ L = length of overland flow, ft n = Manning's roughness coefficient S = overland flow plane slope, ft/ft i = rainfall intensity, in./h	Based on kinematic wave theory for flow on an overland area.
Morgali and Linsley (1965) (*14*), Aron and Erborge (1973) (*15*)	$t_c = 0.94L^{0.6}n^{0.6}S^{-0.3}i^{-0.4}$ L = length of overland flow, ft n = Manning roughness coefficient S = average overland slope, ft/ft i = rainfall intensity, in./h	Overland flow equation from kinematic wave analysis of runoff from developed areas.
FAA (1970) (*16*) AKA RATIONAL METHOD	$t_c = 1.8(1.1 - C)L^{0.5}S^{-0.333}$ C = rational method runoff coefficient L = length of overland flow, ft S = surface slope, ft/ft	Developed from airfield drainage data assembled by U.S. Corps of Engineers.
U.S. Soil Conservation Service (1975, 1986) (*17, 18*)	$t_c = (1/60)\Sigma(L/V)$ L = length of flow path, ft V = average velocity in ft/s for various surfaces (The exponent of S, if converted from Manning's equation, will be −0.5)	Developed as a sum of individual travel times. V can be calculated using Manning's equation.
Papadakis and Kazan (1986) (*2*)	$t_c = 0.66L^{0.5}n^{0.52}S^{-0.31}i^{-0.38}$ L = length of flow path, ft n = roughness coefficient S = average slope of flow path, ft/ft i = rainfall intensity, in./h	Developed from USDA Agricultural Research Service data of 84 small rural watersheds from 22 states.
Chen and Wong (1993) (*19*), Wong (2005) (*20*)	$t_c = 0.595(3.15)^{0.33k}C^{-0.33}L^{0.33(2-k)}S^{-0.33}i^{-0.33(1+k)}$ For water at 26°C C, k = constants (for smooth paved surfaces, $C = 3$, $k = 0.5$. For grass, $C = 1$, $k = 0$) L = length of overland plane, m S = slope of overland plane, m/m i = net rainfall intensity, mm/h	Overland flow on test plots of 1 m wide by 25 m long. Slopes of 2% and 5%.
TxDOT (1994) (*21*)	$t_c = 0.702(1.1 - C)L^{0.5}S^{-0.333}$ C = rational method runoff coefficient L = length of overland flow, m S = surface slope, m/m	Modified from FAA (*16*).
Natural Resources Conservation Service (1997) (*22*)	$t_c = 0.0526[(1000/CN) - 9]L^{0.8}S^{-0.5}$ CN = curve number L = flow length, ft S = average watershed slope, %	For small rural watersheds.

NOTE: 1 mi = 1.61 km; 1 ft = 0.3048 m; 1 in. = 25.4 mm.

Tc= 40 min

Kirpich

$Tc = (0.0078) * (L^{0.77}) * (S^{-0.385})$

L = 2300 ft
S = 0.06 ft/ft
Tc = 9 min (overland flow on bare soil and earth lined ditches)
Tc = 4 min (overland flow on concrete or asphalt)
Tc = 2 min (concrete channels)

Kinematic Wave Formula

$Tc = (0.94 * (L^{0.6}) * (n^{0.6})) / ((I^{0.4}) * (s^{0.3}))$

L = 2300 ft
n = 0.13
I = 0.5 in/hr (kinematic)
S = 0.06 ft/ft
Tc = 86 min

NOTE: *Tc is calculated based upon an assumed Intensity (I). Use calculated Tc to obtain I and re-iterate with new I. Re-iterate as required.*

Izzard

$Tc = (41.025 * ((0.0007 * I) + C) * (L^{0.33})) / ((S^{0.333}) * (I^{0.667}))$

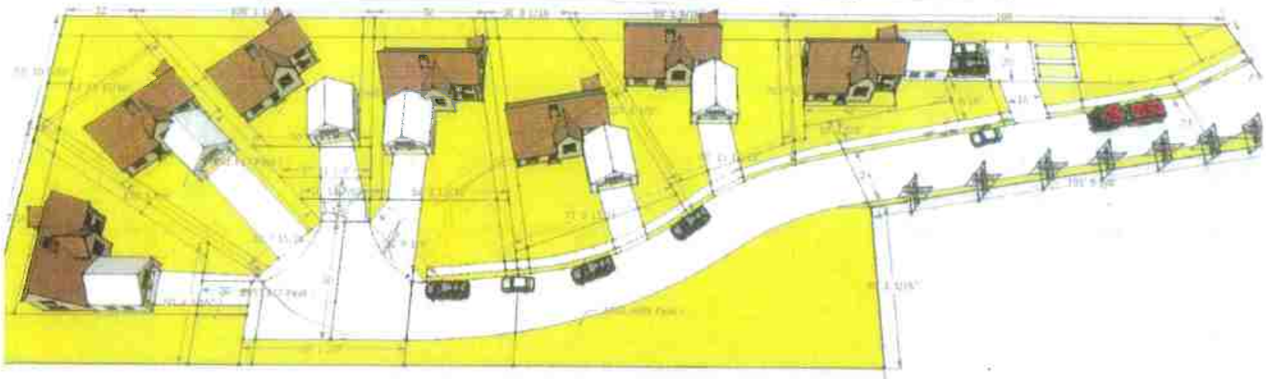
L = 2300 ft
I = 1.4 in/hr
S = 0.08 ft/ft
Tc = 8 min for C = 0.007 very smooth pavement
Tc = 13 min for C = 0.012 for concrete pavement
Tc = 60 min for C = 0.060 for dense turf

NOTE: *Tc is calculated based upon an assumed Intensity (I). Use calculated Tc to obtain I and re-iterate with new I. Re-iterate as required.*

Preliminary Drainage Report for the

Brookview Tract

a Planned Unit Development



Located in:
Mckinleyville, California
APN: 510-121-026

Owner:
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Mckinleyville, CA 95519

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March 2013

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APPENDICES

A Tentative Map by Points West Surveying Company
B IDF Curve for Eureka, CA
C Culvert Nomograph
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E Basin Plan Overlaid on the Tentative Map
F Inlet Capacity Chart per County of Humboldt
G Grate Spacing
H Orifice and Weir Charts
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1.0 INTRODUCTION

This section presents general information about the proposed subdivision, and the existing project topography and geology.

1.1 Project Location

The project site is located in unincorporated town of McKinleyville, Humboldt County, California. It is located just west of the Central Avenue at 1417 Railroad Avenue. See the Location Map below. The project is on AP No. 510-121-026.

Location Map: not to scale

1.2 Project Description

This project develops a vacant 1.58-acre parcel into 7 residential lots, 1 access road, and 1 stormwater detention basin/bioswale area. The lots are sized between 5000 and 11000 square feet. This project will require grading, construction of underground utilities, roads, detention basin and other infrastructure. See Appendix A-Tentative Map by Points West Surveying Company.

1.3 Project Topography and Geology

The 1.58-acre parcel is surrounded by residential development, including a paved trail on the west side of the project. The existing topography gently slopes northwesterly (appx 1%). There is an existing drainage inlet at the northwest corner of the parcel, to which existing and future drainage will flow.

The soil onsite is currently not explored. However, the soil onsite is expected to be similar to that of neighboring Central Terrace Estates Subdivision, Terrace Estates Planned Unit Delevopment, and Shadowbrook Subdivision. These subdivisions all yielded thick rich layer of sod and topsoil (appx 2' deep) followed by light brown sandy soil below. Although there is little standing water nor any wetland areas are evident onsite, seasonal perched groundwater is expected to be present at shallow depths (5-6 feet), as was encountered at Terrace Estates.

Typical percolation rate for this native sandy soil is around 30-60 minutes per inch. However, compaction by construction equipment, and silt sedimentation during the construction process can greatly compromise this percolation rate.

Project Site, view southeast from northwest corner of project

2.0 HYDROLOGY ANALYSIS

This section presents general hydrology calculations for pre-development and post-development.

2.1 Hydrology Introduction

The County of Humboldt requires that macro hydrology be analyzed for subdivision development. The purpose of this requirement is to reasonably verify that onsite development doesn't adversely effect the offsite watershed as a whole.

To accomplish this goal, calculations for the Pre-Development 2-year storm (Q2) and Post development 100-year storm (Q100) are made. The Q2 flow calculations are used to size drainage structures for the allowable discharge during common storm events of the subdivision (typically via basin orifice flow). The Q100 calculations are used to size drainage structures for high flow capacity. In accordance to County policies, the difference in flow between these two values is detained.

For flow calculations, the Rational Method was used: Q = CiA

Where:
Q = Flow (cfs)
C = Runoff Coefficient (=0.25 ag. land, 0.88 pavement, see Appendix D)
I = Rainfall intensity (in/hr), which is dependent on frequency of event, duration, and time of concentration. (See Appendix B)
A = DrainageArea (acres)

For the small drainage area, a minimum time of concentration (Tc) of 10 minutes was selected by engineering convention. The calculations of time of concentration for small sites vary greatly according to formulae used. See Appendix I for a summary of Tc formula.
Thus given a Tc of 10 minutes, rainfall intensities per Appexdix B are:

I2 = 2 year storm intensity = 1.25in/hr
I100 = 100-year storm intensity = 3.2 in/hr

2.2 Pre-Development Flow Calculations

For pre-development 2-year flow analysis, a runoff coefficient of 0.25 (agricultural land) was selected.

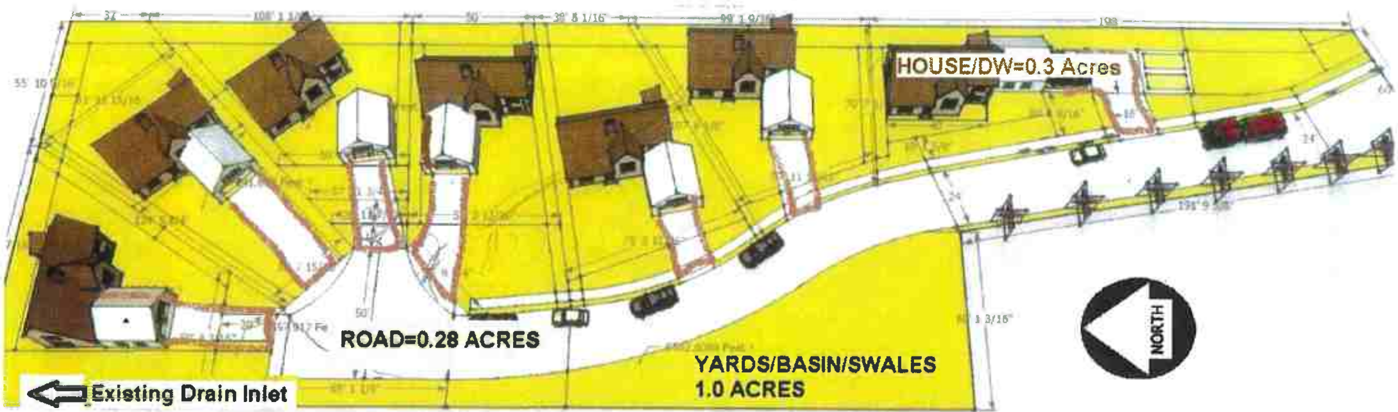
Macro Onsite Pre-Development Flow

Q2 onsite =	C(pre)	* I(pre)	* A		
	0.25	*1.25	*1.58	=	**Q2 onsite= 0.5 cfs**

Note that this 0.5cfs is the amount allowable that can be released offsite in Post-Development conditions.

2.3 Post-Development Flow Calculations

For post-development 100-year flow analysis, a weighted runoff coefficient was considered:
As seen on the Post-Development Concept Drawing below, the site will be development into three major areas: 1) impervious houses and driveways, 2) paved road, and 3) vegetated yards/basins/swales. Common runoff coefficients are shown in Appendix B.



Post-Development Concept Drawing: not to scale

1) For the houses and driveways area, a footprint of 1900 s.f. per lot, or 0.30 acres for all 7 lots. A runoff coefficient of 0.95 was selected.

2) For the paved road area, the 12,000 s.f. footprint, or 0.28 acres was used. A runoff coefficient of 0.85 was selected.

3) The remaining area is 1 acre. (=1.58 total -0.30-0.28) This area will be vegetated yards/basins/swales. A runoff coefficient of 0.20 was selected.

Macro Post Development Flow Table:

Sub Area	Acreage	C factor	C*A
Houses/Driveways	0.30 Acres	0.95	0.29
Paved Road	0.28	0.85	0.24
Vegetated	1.00 Acres	0.25	0.25
Totals	1.58 Acres		0.78→ thus weighted C = 0.723/1.58 = 0.49

→ use weighted coefficient of 0.5

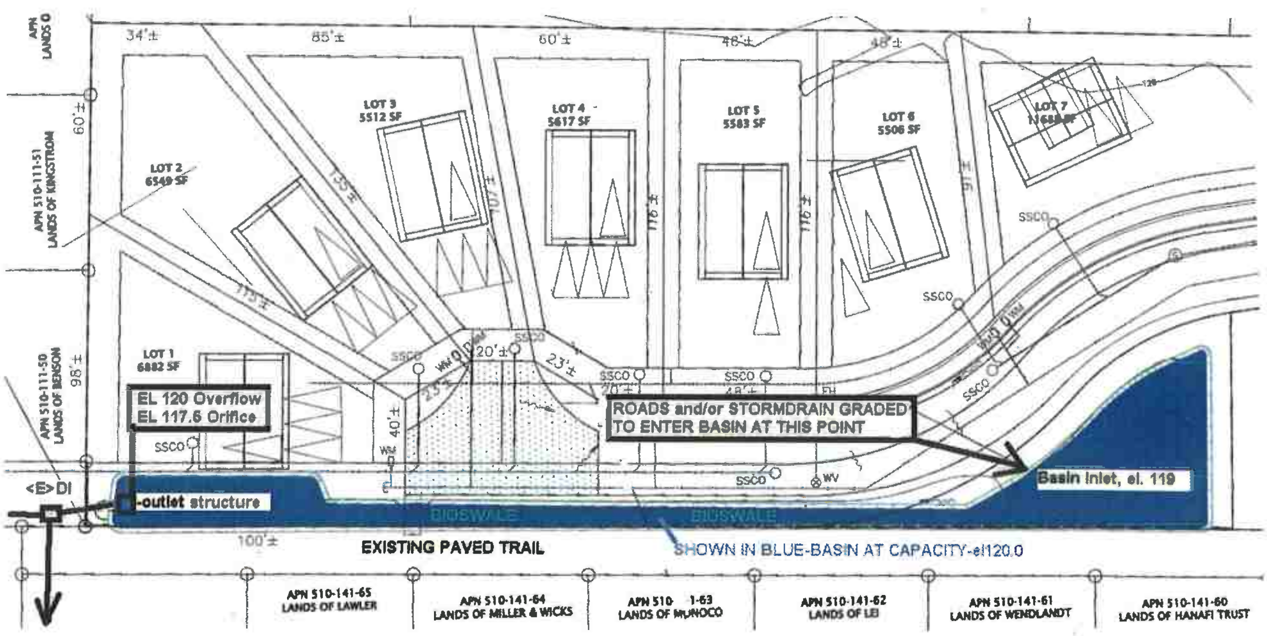
Q100 (post)	=	C(post)	*I(post)	*A	
		0.5	*3.20	*1.58	= **Q100 (post) = 2.5 cfs**

Note that this 2.5 cfs is the minimum the onsite drainage improvements need to be able to handle for overflow conditions.

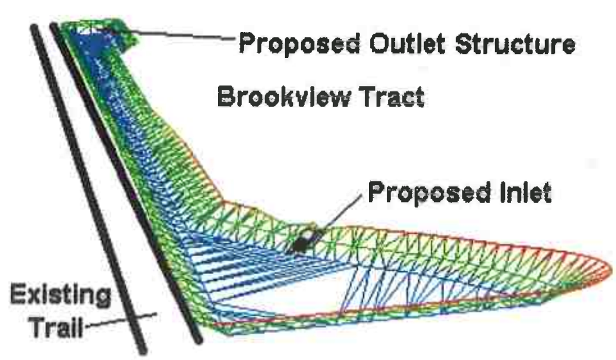
3.0 PROPOSED DRAINAGE IMPROVEMENTS

This section outlines guidelines and designs for several drainage improvements. The subdivision will feature roads and lots with runoff directed towards a basin, bioswale, and outlet structure. As the improvement plans are not finalized at this time, exact grades of roads, valley gutters, and drainage inlets are not established. However, this section of the report outlines several drainage improvements to verify that the proposed subdivision will be compliant with County drainage criteria.

Below are proposed improvements which will be discussed in the following pages:



Basin and Outlet Structure Plan: not to scale



Basin and Outlet Model Isometric View: looking northerly

3.1 Basin Storage Capacity Required and Provided

Per section 2.2 and 2.3, Q2 is 0.5 cfs and Q100 is 2.5 cfs.

Basin Sizing Estimate
(By Triangular Method – aka Skupe Method, assuming 10min Tc = 600 seconds)

Volume =	[K	*(Q100-Q2)	* (3Tc)]/2		
	[1.5	*(2.5-0.5)	* (3*600)]/2	=	**Vrequired = 5400 cf**

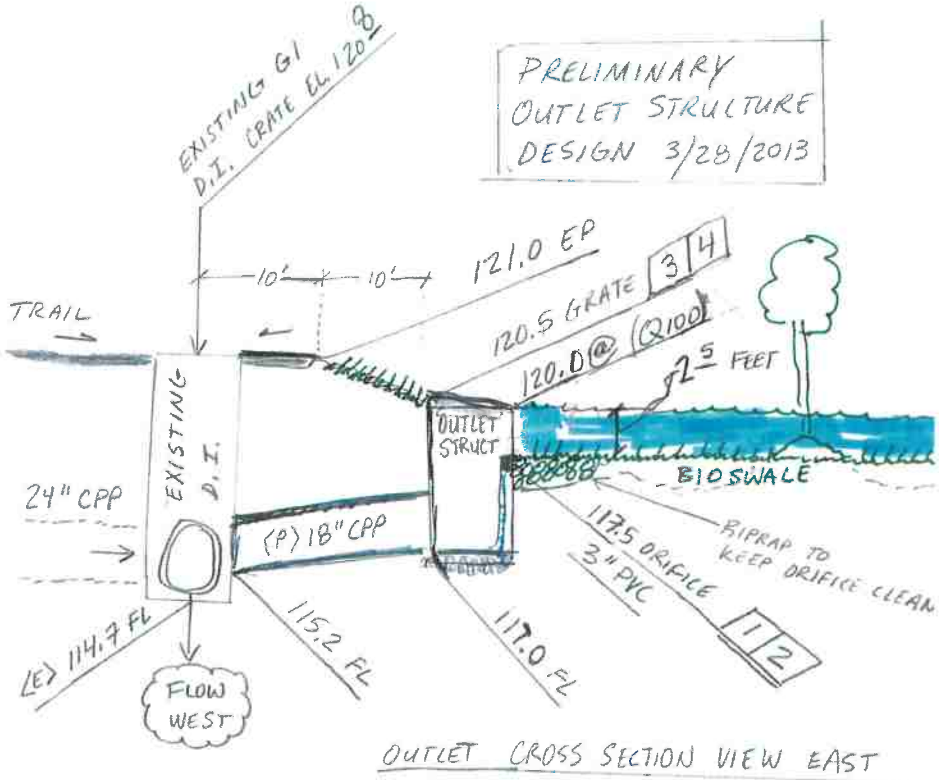
The Basin shown on the previous page has the following stage storage levels:

Elevation	Quantity (cu.ft)	Remarks
117.5	0	3" Orifice FL Elevation
118.0	210	
118.5	610	
119.0	2,160	Water Reaches south end of Basin
119.5	3,970	
120.0	**6,130**	**Vprovided** at Bottom Overflow Grate
120.5	8,550	Top of Overflow Grate---Basin Max. Capacity
120.8	flooding	Grate Elevation of Existing D.I.

From the above table, it is apparent that sufficient storage is provided for the subdivision.

3.2 Outlet and Overflow Structure Design

The Outlet Structure modulates storm flow to a 2 year value. The basin overflow allows the 100 year storm event to bypass the basin in a controlled manner. Proposed is a modified type G1 Drainage Inlet, including a 3" orifice, as the outlet structure of the project.



Outlet Structure Cross Section, view east: not to scale

This Overflow Structure will has the following specifications:

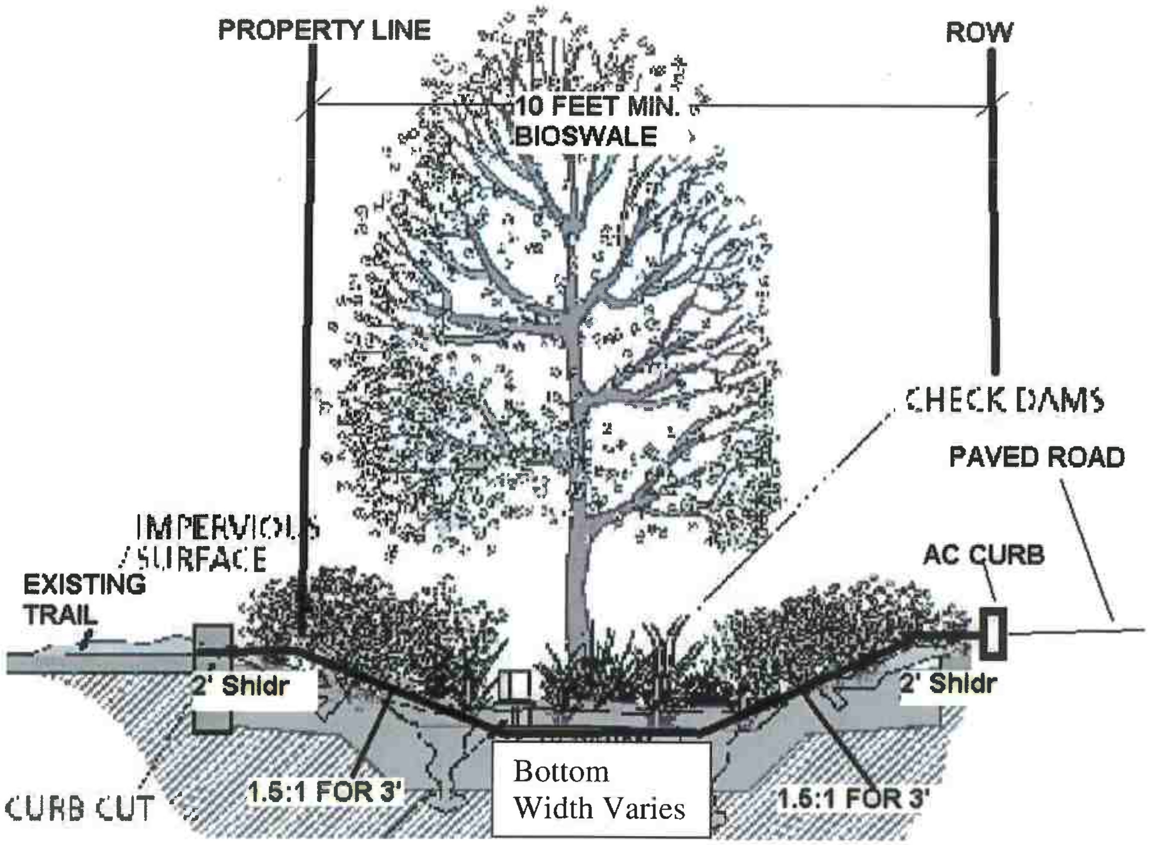
1. Basin outlet Q2 low flow discharge required = 0.5 cfs, from section 2.2.
2. Diameter of low flow orifice selected to provide low flow = 3 inch PVC

   (Calculated from Orifice Formula, see Appendix H)
3. Outlet structure overflow elevation = 120.0 ft
4. Outlet structure overflow capacity = 10 cfs (note only 2.5 cfs req'd per section 2.3)

   (Calculated by Grate Capacity, see Section 3.5 and Appendices F & G)
5. Note: Storage volume (El 117.5 to El. 120.0) = 6130 cu.ft. (5400+ req'd)

3.3 Bioswale Capacity

The section below will be used for the swale:



Given this two +/- foot deep swale will be designed with 1' natural check dams, the actual overflow capacity available is 1 foot. Using manning equation, assuming a minimum of 1 foot water depth, 0.5% slope, and rough grass n=0.03, and varying bottom widths:

$Q=(1.49/n)$ * A * $R^{2/3}$ * $S^{1/2}$

Try 2' Bottom width:

$Q=(1.49/0.03)$ 3.5* $(0.62)^{2/3}$* $(0.005)^{1/2}$ = **Min. Qswale=9 cfs.**

Assume 0' Bottom width:

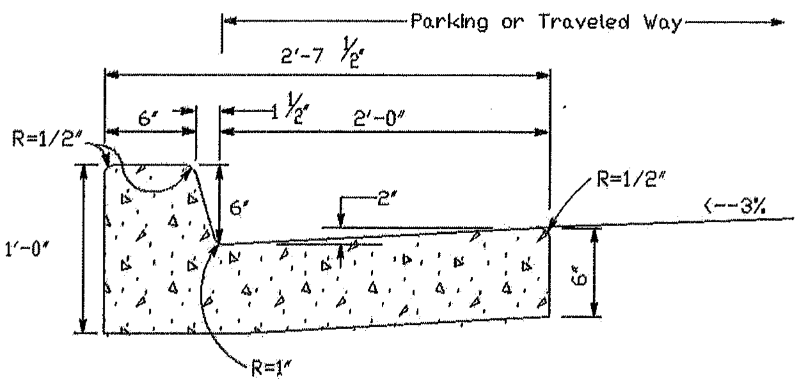
$Q=(1.49/0.03)$ 1.5* $(0.42)^{2/3}$* $(0.005)^{1/2}$ = **Min. Qswale=3 cfs.**

Note: Minumum Qrequired = Q100 2.5 cfs→Thus 0' Bottom width OK!

Since flow velocity will be under 3 ft per second at 0.5%, natural vegetation should be sufficient to prevent erosion, and no turf reinforced mats are specified at this time. However, other rolled erosion control products may be desirable on the 1.5:1 sideslopes to prevent erosion.

3.4 Gutter flow calculations

The purpose of this section is to analyze how far water could flood parking or travelled way lanes. Ideally, Q100 flow floods at most the parking lane (8'). Q100 also cannot overtop the A2-6 curb. The project has this A2-6 curb typical section:



From the section above, the flow table was created using Manning's Equation.

$$V = \frac{k}{n} R_h^{2/3} \cdot S^{1/2}$$

Where:

- V is the cross-sectional average velocity (ft/s, m/s)
- k is a conversion constant equal to 1.486 for U.S. customary units
- n is the **Manning coefficient**, =0.011 for concrete, and 0.015 for pavement
- R_h is the hydraulic radius (ft, m)
- S is the slope of the water surface in the direction of road travel

The table below shows flow values associated with various curb ponding depths:

		Street Steepness in direction on travel (ft/ft)					
Ponding Depth	Ponding Width (ft)	0.005	0.01	0.02	0.03	0.04	0.05
1"	1	1.0	1.4	2.0	2.4	2.8	3.1
2"	2	2.0	2.8	4.0	4.9	5.6	6.3
3"	4.8	2.2	3.1	4.3	5.3	6.1	6.9
4"	7.6	3.1	4.4	6.3	7.7	8.9	9.9
5"	10.3	5.4	7.6	10.7	13.1	15.2	16.9
		Q (cfs) resulting from depths and slopes					

For this project, the Q100 was calculated to be 2.5 cfs. It is apparent that even with a Street Steepness of 0.5%, this amount will have a ponding width under 8 feet, and a ponding depth under 4”. Thus, even though the roads are not designed at this point in the project, the can be designed at a 0.5% minimum grade and have enough capacity to not flood driving lanes.

3.5 Inlet Capacity Calculations

Although the improvement plans are not yet finalized for the subdivision, Caltrans type G0 drainage inlets with A2-6 Curb and Gutters are commonly used. Per Appendix F, the capacity of those drainage inlets at a depth of 0.5’ (curb depth) is 10.6 cfs. This capacity exceeds the individual Q100 in flow for the entire subdivision, so no further analysis was required.

Also note that for the outlet structures, typically Caltrans Type 24-9x Grate is prescribed, as it clogs less than grates designed for bicycle traffic. See Appendix G. Due to the close proximity this outlet structure is to a public trail, a bicycle rated grate may be prudent. The grate capacity is expected to exceed the Q100 required.

3.6 Pipe Capacity Calculations

Although the improvement plans are not yet finalized for the subdivision, 12” and 18” corrugated plastic pipes may be required to connect drain inlets. Assuming a manning n of 0.012 for smooth wall plastic pipe, and a minimum slope of 0.5%:

$Q=(1.49/n)$ * A * $R^{2/3}$ * $S^{1/2}$

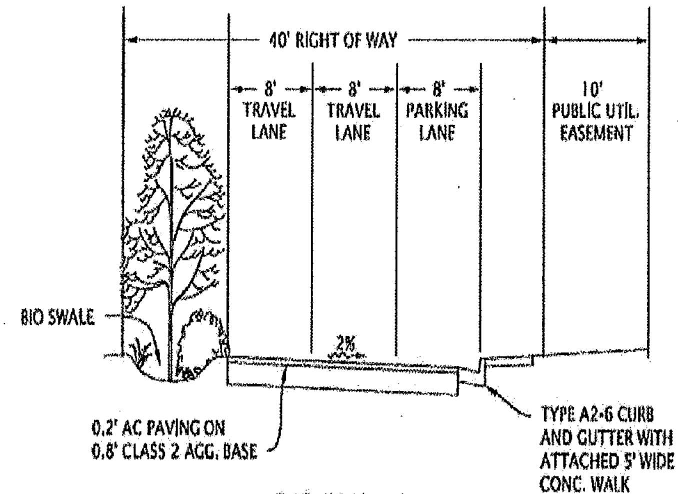
For 12” CPP (1’diameter)

$Q=(1.49/0.012)$ 0.785* $(0.25)^{2/3}$* $(0.005)^{1/2}$ = **12” CAPACITY = 2.8 cfs.**

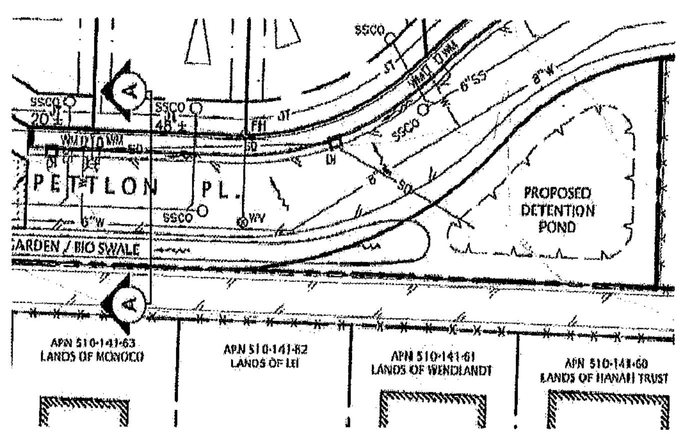
Even though the 12” pipe can carry Q100 for this subdivision, 18” pipes are preferred as they are able to be cleaned and allow room for deposition of sediment. See also Appendix C for an Inlet Control Nomograph for pipe capacities at varying headwater depths.

3.7 Road and Other Improvement Details

The lots and road will be constructed such as to drain towards the basin, at the location labeled as "Basin Inlet" on the plan shown on sheet 7. How the water drains into the basin from the road is not yet finalized, but several alternatives exist. One option depicted on the typical map included the following road section and plan:



SECTION A-A
NO SCALE



As shown, Pettlon Place is graded right (east) and into stormdrain system and into the basin. As outlined in this report, 12" storm drain pipe would have enough capacity for the subdivision. One potential problem with this system is that cover for the stormdrain pipes may insufficient (typ. 1' min) depending on the final road grades. Other alternatives exist, including but not limited to:

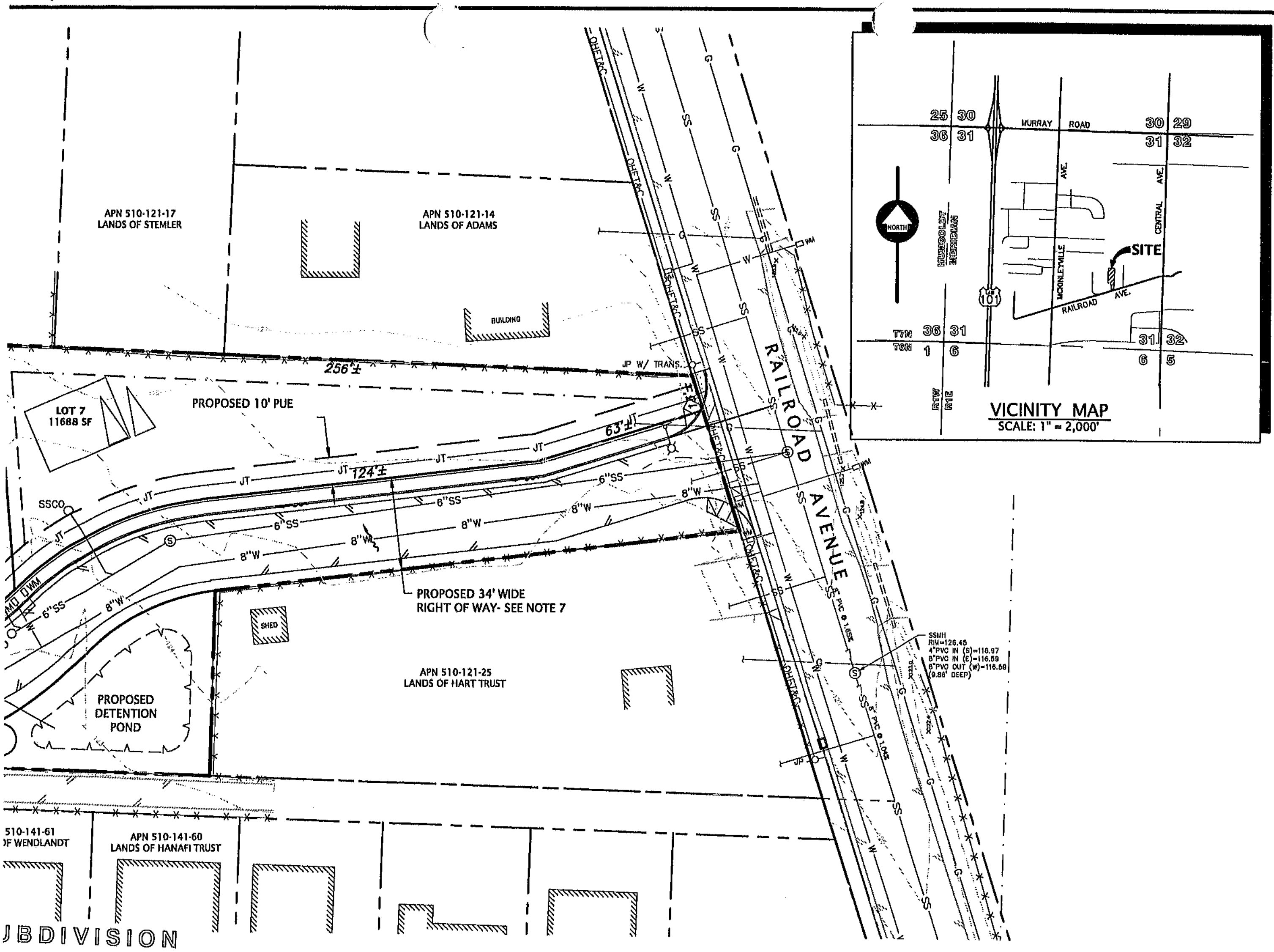
(1) Using section A-A, but at the position shown for the stormdrain crossing Pettlon Place, install a valley gutter system graded left (west), including a riprap swale entering the basin.

(2) Instead of section A-A, use a typical section that is graded left (west) at 2%. Install a Caltrans type A1-6 Curb on the right (east) side of the road, and A2-6 on the left (west) side of the road. At the position of the basin inlet, end the curb and have a riprap swale entering the basin.

3.7 Conclusions

This report has demonstrated that the preliminary layout shown on the tentative map can accommodate drainage improvements which mitigate adverse offsite drainage impacts. The basin outlined in this section complies with the typical storage capacity required by the county. The bioswale and other onsite best management practices should enhance natural percolation of water within the basin system, thereby reducing the demands required of the outlet structure.

Other specific drainage details, such as road drainage inlets and/or valley gutters, road profiles will be outlined in the Improvement Plans for the project.



Notes:

1) This map represents a subdivision of the lands described in Document No. 1998-4854 into seven parcels. Total gross area of property is 1.58 acres.

2) The existing boundary lines and easements are based on a Title Report prepared by Humboldt land Title Company, Order No. 00142882-DS, dated 7/23/2012. All easements of record and new easements show on the Tentative Map and will show on the Final Map.

3) The property is not shown on official maps to be subject to flooding per Firm Community Panel No. 060060 0625C.

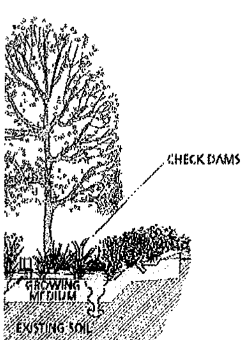
4) Utilities:

Water and Sewer	McKinleyville Community Services District
Electric and Gas	Pacific Gas and Electric
Telephone	AT&T
Cable TV	Sudden Link

5) Topography is shown at 1 foot contour intervals based on field survey performed by Points West Surveying in January 2013.

6) The existing property is vacant. The proposed use is residential single family residences. The existing use on adjacent parcels is residential.

7) The proposed Planned Unit Development presented proposes a right of way of varying width. The entry road is proposed to be a 34 foot right of way widening to a forty foot right of way as shown. Additionally, the front setback is proposed to be ten feet from the right of way to allow flexibility in home citing and to facilitate solar exposure.



BIO-SWALE SECTION
NO SCALE

PROJECT DATA

Owner / Applicant: Lynn Pettion
APN: 510-121-026
Address: 1417 Railroad Avenue
McKinleyville, CA 95519

Agent: David Crivelli
Points West Surveying

General Plan: RL (MCCP) Residential Low Density
Prinicipal Zoning: R-1 Residential Single Family
5000 sf minimum lot size

Building Setbacks:

Front:	10'
Interior Side:	5'
Exterior Side:	5'
Rear:	10'

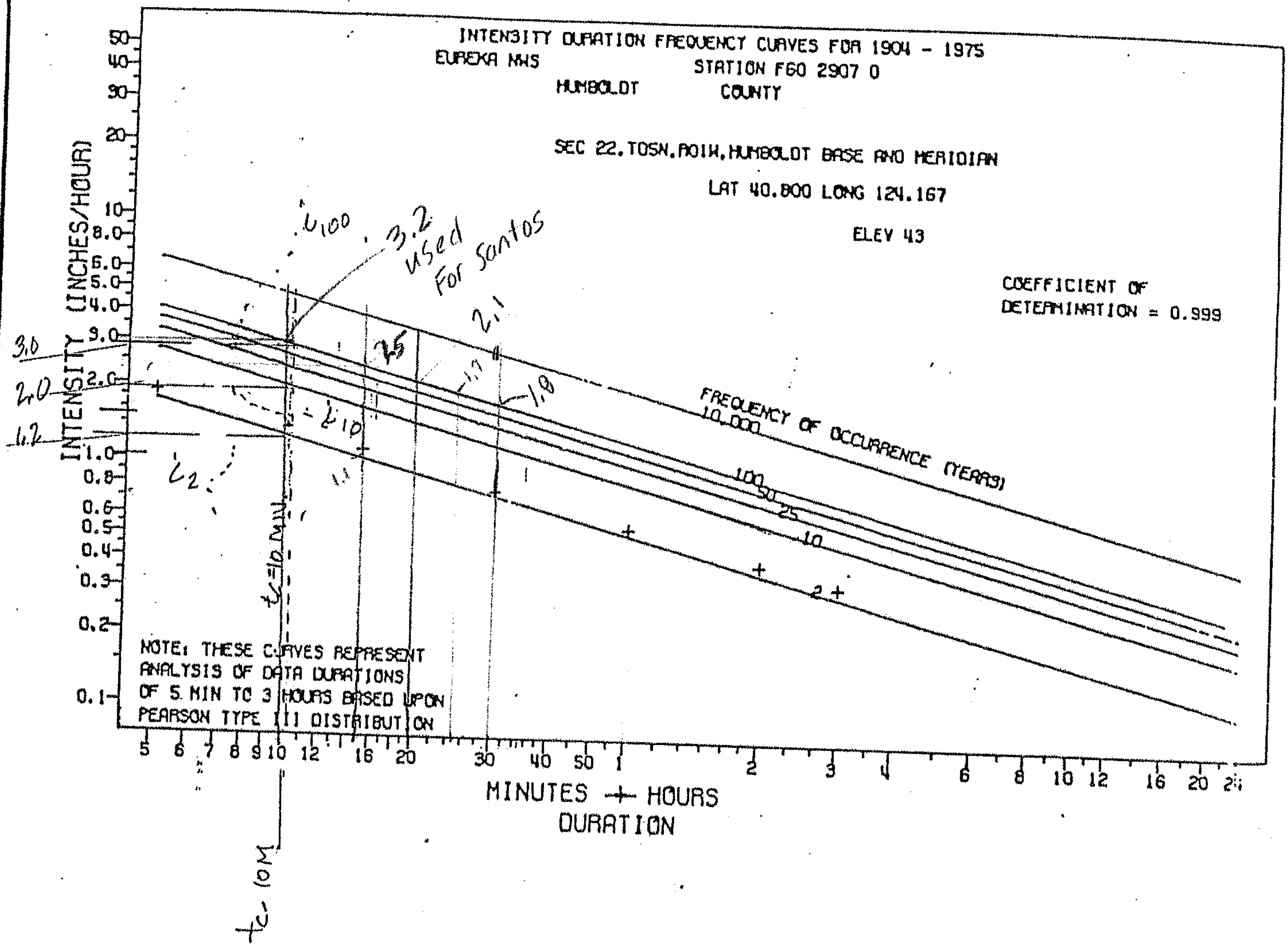
APN 510-121-026
TENTATIVE MAP
for
Brookview Tract

A PLANNED UNIT DEVELOPMENT
SECTION 31, T7N, R1E,
HUMBOLDT MERIDIAN

IN THE UNINCORPORATED AREA OF
HUMBOLDT COUNTY, STATE OF CALIFORNIA
SCALE: 1" = 30' FEBRUARY 2013 SHEET 1 OF 1

POINTS WEST SURVEYING CO.
5201 Carlson Park Dr., Suite 3 - Arcata, CA 95521
707·840·9510 · Phone 707·840·9542 · Fax

APPENDIX B



$t_c = 10$ MIN;
FIND i_2, i_{10}, i_{100}

From: "Rainfall Analysis For Drawing Design Vol. 1." Bulletin 195, Oct. 1976.
California Department of Water Resources.

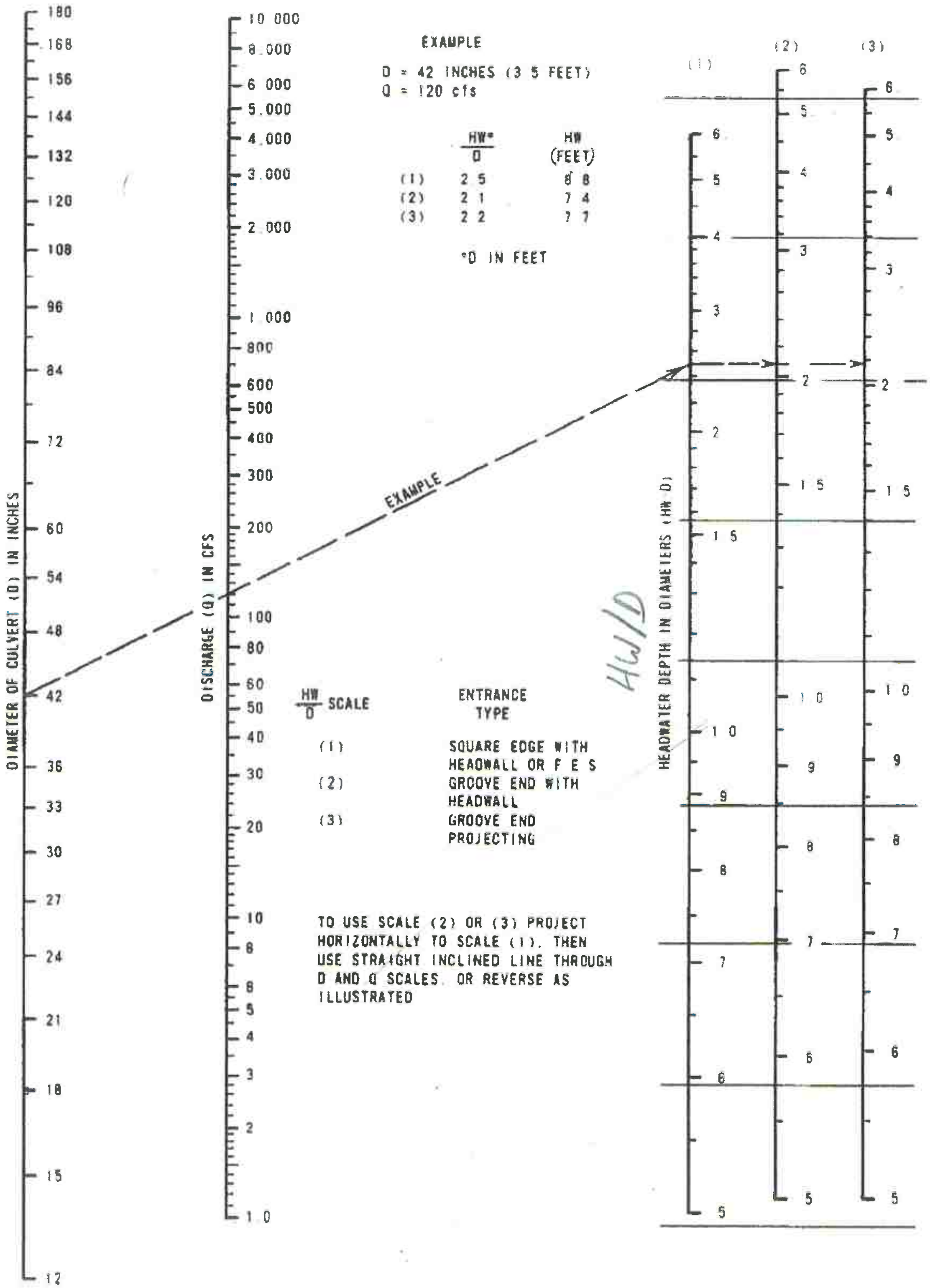
INTENSITY DURATION FREQUENCY CURVES - 1904 - 1975

B IDF Curve for Eureka, CA

1(b)

Inlet Control Nomograph for Concrete Pipe

APPENDIX C



APPENDIX D - Rational Method C Coefficients

categorized by surface

NOTE: FOR THIS PROJECT, C=0.85 WAS SELECTED FOR ROAD AREAS, C=0.95 WAS SELECTED FOR HOUSE/DRIVEWAY AREAS.

forested	0.059–0.2
asphalt	0.7–0.95
brick	0.7–0.85
concrete	0.8–0.95
shingle roof	0.75–0.95
lawns, well-drained (sandy soil)	
up to 2% slope	0.05–0.1
2% to 7% slope	0.10–0.15
over 7% slope	0.15–0.2
lawns, poor drainage (clay soil)	
up to 2% slope	0.13–0.17
2% to 7% slope	0.18–0.22
over 7% slope	0.25–0.35
driveways, walkways	0.75–0.85

categorized by use

NOTE: COMMON USAGE IS C=0.25 FOR MCKINLEYVILLE PASTURE/AGRICULTURAL LAND

C=0.20 FOR LAWNS AND GREENBELT

farmland	0.05–0.3
pasture	0.05–0.3
unimproved	0.1–0.3
parks	0.1–0.25
cemeteries	0.1–0.25
railroad yards	0.2–0.35
playgrounds (except asphalt or concrete)	0.2–0.35
business districts	
neighborhood	0.5–0.7
city (downtown)	0.7–0.95
residential	
single family	0.3–0.5
multiplexes, detached	0.4–0.6
multiplexes, attached	0.6–0.75
suburban	0.25–0.4
apartments, condominiums	0.5–0.7
industrial	
light	0.5–0.8
heavy	0.6–0.9


APN 510-121-17
LANDS OF STEMLER
APN 510-121-14
LANDS OF ADAMS
256'±
LOT 7
11688 SF
126
PT = 0+91.05
PC = 0+74.61
63'±
124'±
126
SSCO
WM
APN 510-121-25
LANDS OF HART TRUST
PROPOSED
DETENTION
POND
510-141-61
F WENDLANDT
APN 510-141-60
LANDS OF HANAFI TRUST

APPENDIX F

COUNTY OF HUMBOLDT – DEPARTMENT OF PUBLIC WORKS		
QUANTITY CALCULATIONS		
DC-CEM-4801 (OLD HC-52 REV.11/92) 7451-3520-0	SHEET 1 OF 2	
JOB STAMP	ITEM	FILE NO.
WEIR FLOW VERSUS ORFICE FLOW CONTROLLING CONDITIONS & FLOWRATE	LOCATION	SEGREGATION YES NO
	CALC. BY DPJ	DATE 07-15-2010
	CHK. BY	DATE

4.4.5.1. Grate Inlets in Sags

A grate inlet in a sag location operates as a weir to depths dependent on the size of the grate and as an orifice at greater depths. Grates of larger dimension will operate as weirs to greater depths than smaller grates.

see STD PLAN (CALTRANS) D77A, D77B

The capacity of grate inlets operating as weirs is:

use for DI's @ curbs

$$Q_i = C_w P d^{1.5} \qquad (4\text{-}26)$$

1.56 clear

3'4"

where:

P = perimeter of the grate in m (ft) disregarding the side against the curb
C_w = 1.66 (3.0 in English units)
d = average depth across the grate: $0.5(d_1 + d_2)$, m (ft)

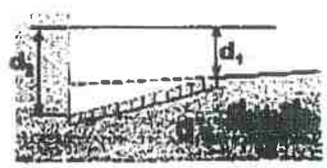


Figure 4-17. Definition of depth.

24-9 SPACING 2"

5.25

$Q = 0.67 *$ ~~[illegible]~~ $= $ ~~[illegible] cfs~~

The capacity of a grate inlet operating as an orifice is:

$$Q_i = C_o A_g (2gd)^{0.5} \qquad (4\text{-}27)$$

where:

C_o = orifice coefficient = 0.67
A_g = clear opening area of the grate, m^2 (ft^2)
g = 9.81 m/s^2 (32.16 ft/s^2)

TPE 24-9 ~~[illegible]~~
$A_g = 1.56 \times 3.33 = 5.20$

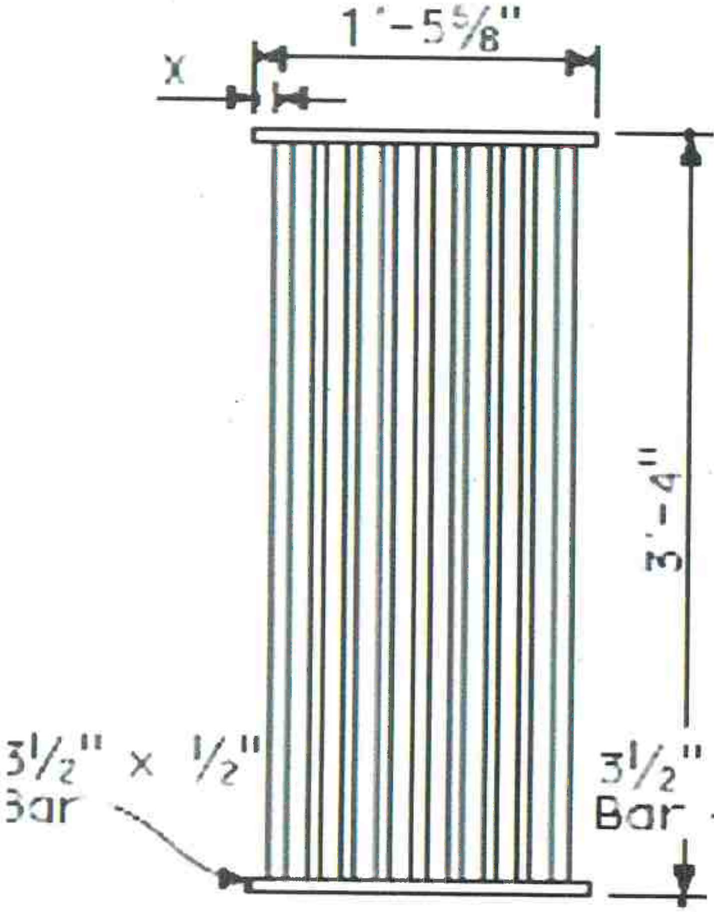
use for outlet struct basin
$A_g = 5.20$

WEIR FLOW VERSUS ORFICE FLOW CONTROLLING CONDITIONS & FLOWRATE - COMB. INLET									
	WEIR FLOW				ORFICE FLOW				
DEPTH,d	C_W	P	$d^{1.5}$	Q_i	C_O	A_g	g	$(2gd)^{0.5}$	Q_i
[ft]		[ft]		[cfs]		[ft^2]	[ft/sec^2]		[cfs]
0.1	3.0	10.0	0.032	0.95	0.67	4.875	32.2	2.538	8.29
0.2	3.0	10.0	0.089	2.68	0.67	4.875	32.2	3.589	11.72
0.3	3.0	10.0	0.164	4.93	0.67	4.875	32.2	4.395	14.36
0.4	3.0	10.0	0.253	7.59	0.67	4.875	32.2	5.075	16.58
0.5	3.0	10.0	0.354	10.61	0.67	4.875	32.2	5.675	18.53
0.6	3.0	10.0	0.465	13.94	0.67	4.875	32.2	6.216	20.30
0.7	3.0	10.0	0.586	17.57	0.67	4.875	32.2	6.714	21.93
0.8	3.0	10.0	0.716	21.47	0.67	4.875	32.2	7.178	23.44
0.9	3.0	10.0	0.854	25.61	0.67	4.875	32.2	7.613	24.87
1	3.0	10.0	1.000	30.00	0.67	4.875	32.2	8.025	26.21
1.1	3.0	10.0	1.154	34.61	0.67	4.875	32.2	8.417	27.49
1.2	3.0	10.0	1.315	39.44	0.67	4.875	32.2	8.791	28.71
1.3	3.0	10.0	1.482	44.47	0.67	4.875	32.2	9.150	29.89
1.4	3.0	10.0	1.657	49.70	0.67	4.875	32.2	9.495	31.01
1.5	3.0	10.0	1.837	55.11	0.67	4.875	32.2	9.829	32.10

$Q = 0.67 * 5.2 * (5.675) = 19.8$ cfs

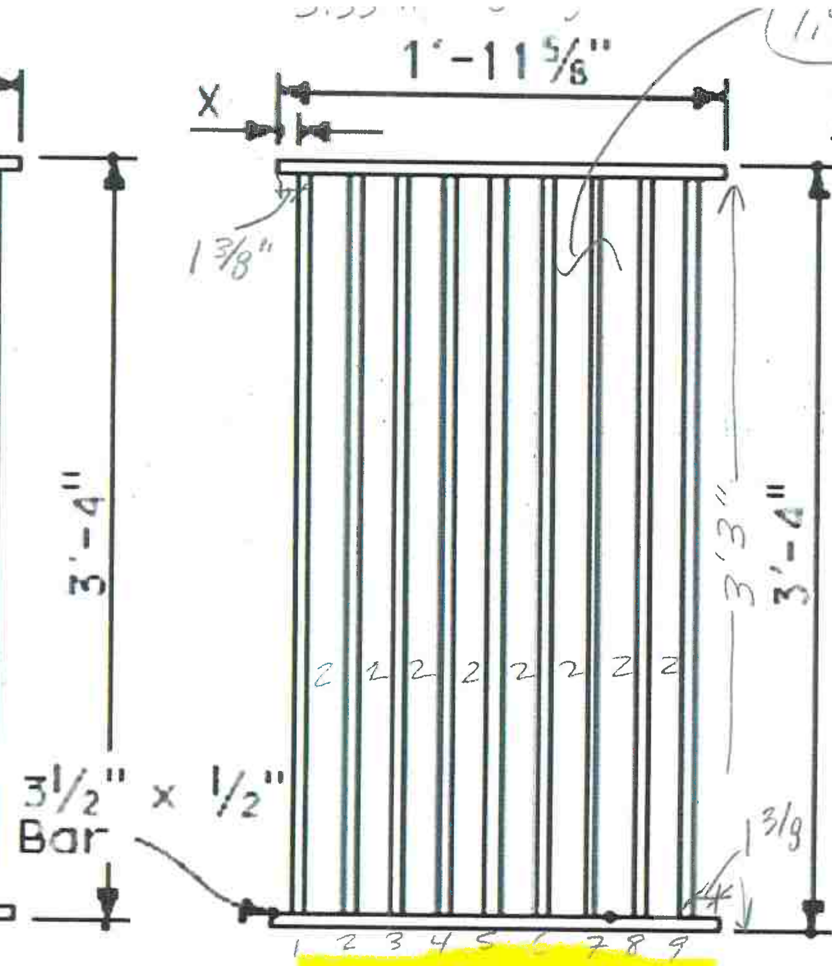
POSTED BY	DATE	POSTED TO
CEM-4801 (HC-52 REV. 11/92)		92 21305

APPENDIX G - Grate spacing



TYPE 18-9

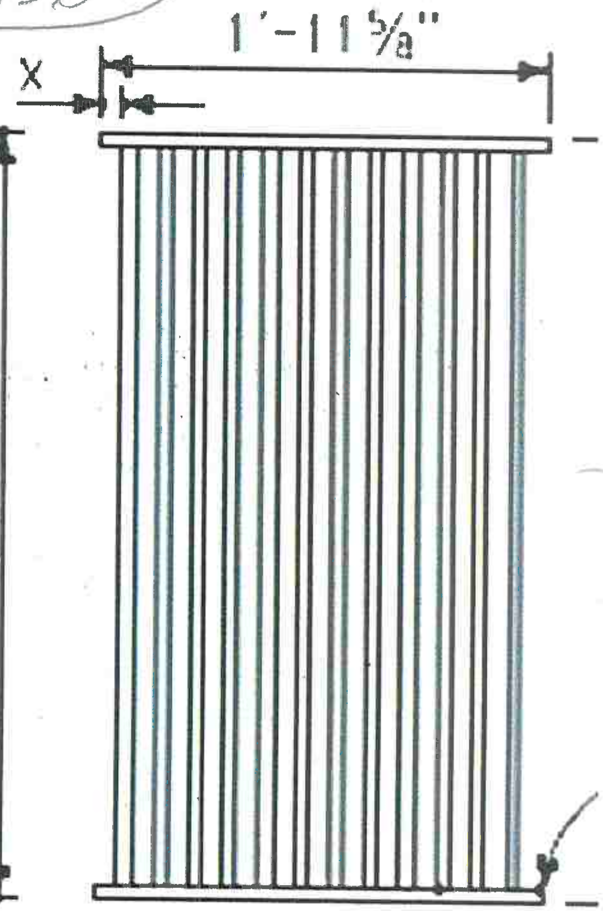
1⅜" Clear spacing.
Use within the roadbed on highways where bicycles and pedestrians are excluded.



TYPE 24-9

2" Clear spacing.
Use in locations off the roadbed on all types of highways.

3.36 x 1.56 = Ag = 5.2



TYPE 24-12

1⅜" Clear spacing.
Use within the roadbed on highways where bicycles and pedestrians are excluded.

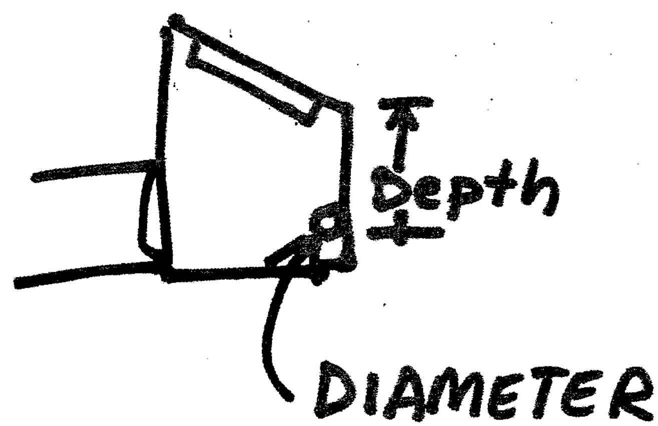
RECTANGULAR GRATE DETAILS

(See table below)

APPENDIX H

PIPE DIA:	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	24	30	36
Area	0.02	0.05	0.09	0.14	0.20	0.27	0.35	0.44	0.55	0.66	0.79	0.92	1.07	1.23	1.40	1.58	1.77	3.14	4.91	7.07
$K=CA(2g)^{1/2}$	0.10	0.24	0.42	0.65	0.94	1.28	1.68	2.12	2.62	3.17	3.77	4.42	5.13	5.89	6.70	7.57	8.48	15.08	23.56	33.93
Depth (ft)	FLOW (cfs)																			
5.75	0.3	0.6	1.0	1.6	2.3	3.1	4.0	5.1	6.3	7.6	9.0	10.6	12.3	14.1	16.1	18.1	20.3	36.2	56.5	81.4
6.00	0.3	0.6	1.0	1.6	2.3	3.1	4.1	5.2	6.4	7.8	9.2	10.8	12.6	14.4	16.4	18.5	20.8	36.9	57.7	83.1
6.25	0.3	0.6	1.0	1.6	2.4	3.2	4.2	5.3	6.5	7.9	9.4	11.1	12.8	14.7	16.8	18.9	21.2	37.7	58.9	84.8
6.50	0.3	0.6	1.1	1.7	2.4	3.3	4.3	5.4	6.7	8.1	9.6	11.3	13.1	15.0	17.1	19.3	21.6	38.4	60.1	86.5
6.75	0.3	0.6	1.1	1.7	2.4	3.3	4.4	5.5	6.8	8.2	9.8	11.5	13.3	15.3	17.4	19.7	22.0	39.2	61.2	88.1
7.00	0.3	0.6	1.1	1.7	2.5	3.4	4.4	5.6	6.9	8.4	10.0	11.7	13.6	15.6	17.7	20.0	22.4	39.9	62.3	89.8
7.25	0.3	0.6	1.1	1.8	2.5	3.5	4.5	5.7	7.0	8.5	10.2	11.9	13.8	15.9	18.0	20.4	22.8	40.6	63.4	91.4
7.50	0.3	0.6	1.1	1.8	2.6	3.5	4.6	5.8	7.2	8.7	10.3	12.1	14.1	16.1	18.4	20.7	23.2	41.3	64.5	92.9
7.75	0.3	0.7	1.2	1.8	2.6	3.6	4.7	5.9	7.3	8.8	10.5	12.3	14.3	16.4	18.7	21.1	23.6	42.0	65.6	94.5
8.00	0.3	0.7	1.2	1.9	2.7	3.6	4.7	6.0	7.4	9.0	10.7	12.5	14.5	16.7	19.0	21.4	24.0	42.7	66.6	96.0

ORIFICE
CHART



APPENDIX I

TABLE 1 Summary of Time of Concentration Models

Publication and Year	Equation for Time of Concentration (min)	Remarks
Williams (1922) (*6*)	$t_c = 60LA^{0.4}D^{-1}S^{-0.2}$ L = basin length, mi A = basin area, mi^2 D = diameter (mi) of a circular basin of area S = basin slope, %	The basin area should be smaller than 50 mi^2 (129.5 km^2).
Kirpich (1940) (*7*)	$t_c = KL^{0.77}S^{y}$ L = length of channel/ditch from headwater to outlet, ft S = average watershed slope, ft/ft For Tennessee, $K = 0.0078$ and $y = -0.385$ For Pennsylvania, $K = 0.0013$ and $y = -0.5$	Developed for small drainage basins in Tennessee and Pennsylvania, with basin areas from 1 to 112 acres (0.40 to 45.3 ha).
Hathaway (1945) (*8*), Kerby (1959) (*9*)	$t_c = 0.8275\,(LN)^{0.467}S^{-0.233}$ L = overland flow length, ft S = overland flow path slope, ft/ft N = flow retardance factor	Drainage basins with areas of less than 10 acres (4.05 ha) and slopes of less than 0.01.
Izzard (1946) (*10*)	$t_c = 41.025(0.0007i + c)L^{0.33}S^{-0.333}\,i^{-0.667}$ i = rainfall intensity, in./h c = retardance coefficient L = length of flow path, ft S = slope of flow path, ft/ft	Hydraulically derived formula; values of c range from 0.007 for very smooth pavement to 0.012 for concrete pavement to 0.06 for dense turf.
Johnstone and Cross (1949) (*11*)	$t_c = 300L^{0.5}S^{-0.5}$ L = basin length, mi S = basin slope, ft/mi	Developed for basins with areas between 25 and 1624 mi^2 (64.7 and 4206.1 km^2).
California Culvert Practice (1955) (*12*)	$t_c = 60(11.9L^3/H)^{0.385}$ L = length of longest watercourse, mi H = elevation difference between divide and outlet, ft If expressed as $T_c = kL^a n^b S^{-y} i^{-z}$ format: $t_c = KL^{0.77}S^{-0.385}$ K = conversion constant	Essentially the Kirpich (*7*) formula; developed for small mountainous basins in California.
Henderson and Wooding (1964) (*13*)	$t_c = 0.94(Ln)^{0.6}S^{-0.3}i^{-0.4}$ L = length of overland flow, ft n = Manning's roughness coefficient S = overland flow plane slope, ft/ft i = rainfall intensity, in./h	Based on kinematic wave theory for flow on an overland area.
Morgali and Linsley (1965) (*14*), Aron and Erborge (1973) (*15*)	$t_c = 0.94L^{0.6}\,n^{0.6}S^{-0.3}i^{-0.4}$ L = length of overland flow, ft n = Manning roughness coefficient S = average overland slope, ft/ft i = rainfall intensity, in./h	Overland flow equation from kinematic wave analysis of runoff from developed areas.
FAA (1970) (*16*) AKA RATIONAL METHOD	$t_c = 1.8(1.1 - C)L^{0.5}S^{-0.333}$ C = rational method runoff coefficient L = length of overland flow, ft S = surface slope, ft/ft	Developed from airfield drainage data assembled by U.S. Corps of Engineers.
U.S. Soil Conservation Service (1975, 1986) (*17, 18*)	$t_c = (1/60)\Sigma(L/V)$ L = length of flow path, ft V = average velocity in ft/s for various surfaces (The exponent of S, if converted from Manning's equation, will be −0.5)	Developed as a sum of individual travel times. V can be calculated using Manning's equation.
Papadakis and Kazan (1986) (*2*)	$t_c = 0.66L^{0.5}n^{0.52}S^{-0.31}i^{-0.38}$ L = length of flow path, ft n = roughness coefficient S = average slope of flow path, ft/ft i = rainfall intensity, in./h	Developed from USDA Agricultural Research Service data of 84 small rural watersheds from 22 states.
Chen and Wong (1993) (*19*), Wong (2005) (*20*)	$t_c = 0.595(3.15)^{0.33k}C^{-0.33}L^{0.33(2-k)}S^{-0.33}i^{-0.33(1+k)}$ For water at 26°C C, k = constants (for smooth paved surfaces, $C = 3$, $k = 0.5$. For grass, $C = 1$, $k = 0$) L = length of overland plane, m S = slope of overland plane, m/m i = net rainfall intensity, mm/h	Overland flow on test plots of 1 m wide by 25 m long. Slopes of 2% and 5%.
TxDOT (1994) (*21*)	$t_c = 0.702(1.1 - C)L^{0.5}S^{-0.333}$ C = rational method runoff coefficient L = length of overland flow, m S = surface slope, m/m	Modified from FAA (*16*).
Natural Resources Conservation Service (1997) (*22*)	$t_c = 0.0526[(1000/CN) - 9]L^{0.8}S^{-0.5}$ CN = curve number L = flow length, ft S = average watershed slope, %	For small rural watersheds.

NOTE: 1 mi = 1.61 km; 1 ft = 0.3048 m; 1 in. = 25.4 mm.

Tc Calculator

by R.W. Bronkall on 11/20/2001
revised 01/25/2005 by RWB

Data Entry

General Variables

2300	feet	Length of longest watercourse
160	ft	Elevation of highest point on watercourse
16	ft	Elevation of lowest point on watercourse
0.25	--	"C"- Rational method runoff coefficient

C=1.00 impervious; C=0.55 Med. Density (2-8 units/acre)
C=0.40 Low Density (0-2 units/acre); C=0.30 rural (5 ac min)
C=0.85 commercial / Light Manufacturing / Tourist;
C=0.25 Ag. (20 ac min); C=0.20 Open Space/Forest

Kinematic Wave formula specific varaibles:

0.5	in/hr	"I" - Intensity - (iterative)
0.13	--	"n" - mannings roughness coefficient

Izzard formula specific variables

1.4	in/hr	"I"- Intensity. (assumed for Izzard formula)

Executive Summary of All Methods

Tc=	9	min. ~ Calfiornia Culvert Practice
Tc=	40	min. ~ FAA
Tc=	9	min. ~ Kirpich (overland flow on bare soil and earth lined ditches)
Tc=	4	min. ~ Kirpich (overland flow on concrete or asphalt)
Tc=	2	min. ~ Kirpich (concrete channels)
Tc=	86	min. ~ Kinematic Wave *** iterative- based upon assumed I***
Tc=	8	min. ~ Izzard (smooth pavement) *** iterative- based upon assumed I ***
Tc=	13	min. ~ Izzard (concrete pavement) *** iterative- based upon assumed I ***
Tc=	60	min. ~ Izzard (dense turf) *** iterative- based upon assumed I ***

California Culvert Practice

$$Tc = 60 * (11.9 * (L^3) / H)^{0.385}$$

L=	0.44	mile
H=	144	ft
Tc=	9	min

FAA

$$Tc = 1.8 * (1.1 - C) * (L^{0.50}) / (S^{0.333})$$

L=	2300	ft
C=	0.25	
S=	6.26	%

Tc=	40	min

Kirpich

$Tc = (0.0078) * (L^{0.77}) * (S^{-0.385})$

L =	2300	ft
S =	0.06	ft/ft
Tc =	9	min (overland flow on bare soil and earth lined ditches)
Tc =	4	min (overland flow on concrete or asphalt)
Tc =	2	min (concrete channels)

Kinematic Wave Formula

$Tc = (0.94 * (L^{0.6}) * (n^{0.6})) / ((I^{0.4}) * (s^{0.3}))$

L =	2300	ft
n =	0.13	
I =	0.5	in/hr (kinematic)
S =	0.06	ft/ft
Tc =	86	min

NOTE: *Tc is calculated based upon an assumed Intensity (I). Use calculated Tc to obtain I and re-iterate with new I. Re-iterate as required.*

Izzard

$Tc = (41.025 * ((0.0007 * I) + C)* (L^{0.33})) / ((S^{0.333})*(I^{0.667}))$

L =	2300	ft
I =	1.4	in/hr
S =	0.08	ft/ft
Tc =	8	min for C = 0.007 very smooth pavement
Tc =	13	min for C = 0.012 for concrete pavement
Tc =	60	min for C = 0.060 for dense turf

NOTE: *Tc is calculated based upon an assumed Intensity (I). Use calculated Tc to obtain I and re-iterate with new I. Re-iterate as required.*

LINDBERG GEOLOGIC CONSULTING
David N. Lindberg, Certified Engineering Geologist

ENGINEERING-GEOLOGIC SOILS REPORT

Proposed Subdivision of APN 510-121-026

1417 Railroad Drive, McKinleyville,
Humboldt County, California

RECEIVED
FEB 4 2014
Humboldt County
Planning Division

Prepared for:

Points West Surveying

CERTIFIED ENGINEERING GEOLOGIST
DAVID N. LINDBERG
No. 1895
EXP. 2/28/14
STATE OF CALIFORNIA

APPROVED
Humboldt County
Building Inspection Division
FEB 14 2014
By: Damian

David N. Lindberg, CEG 1895, Exp. 02/28/14

Engineer to inspect
footing/excavations.

January 25, 2013

LGC Project No. 0054.00

Post Office Box 306 Cutten, California 95534 (707) 442-6000

TABLE OF CONTENTS

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1.0 INTRODUCTION 1
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ENGINEERING GEOLOGIC SOILS REPORT

Proposed Subdivision of APN 510-121-026
1417 Railroad Drive, McKinleyville,
Humboldt County, California

1.0 INTRODUCTION

1.1 Site and Project Description

This report presents the results of our soils exploration conducted on the property located in McKinleyville, California (Figure 1). The parcel number assigned by the Humboldt County Assessor is 510-121-026 (Figure 2). Pertinent project site location information is listed in Table 1 below.

TABLE 1 - PROJECT LOCATION INFORMATION

Latitude and Longitude*	40.9464° N and 124.1054° W
Legal Description	NE ¼ of Southeast ¼ of Section 31 Township 7 N, Range 1 E; HB&M.
Parcel Size	1.4157 acre
USGS Quadrangle	Arcata North 7.5-minute topographic quadrangle.

* Centroid of parcel per Humboldt County Web GIS

Lindberg Geologic Consulting (LGC) was retained by Points West Surveying, representing the property owner(s). The owner is proposing to subdivide the property, presently one parcel, into seven new residential parcels served by one new street. Off-street parking will be provided on each lot, with additional parking provided in a dedicated area in the southern part of the proposed subdivision. Local utilities (water, sewer, power, etc.) are available through the McKinleyville Community Services District, PG&E and other local service providers. Ingress and egress to the new parcels will be from a new street heading north from Railroad Drive, to be constructed as part of this proposed subdivision development.

Included in this report are assessments of the potential geologic hazards associated with the site, and recommendations as necessary and appropriate to help mitigate some of the potential effects of such hazards. Also provided in this report are recommendations for design professionals such as architects and engineers, to utilize for planning and design of future site developments.

1.2 Scope of Work

The Scope of Services for this investigation included identifying potential geologic hazards with a potential to affect the proposed developments, characterizing the subgrade soils, developing recommendations, and preparation of this Report. The following information, recommendations, and design criteria are presented in this report:

- Description of site terrain and local geology.
- An interpretation of subsurface soil and groundwater conditions based on our exploration.
- Logs of soil profile characteristics observed within test boring locations.
- Assessment of potential earthquake-related geologic and geotechnical hazards including surface fault rupture, liquefaction, differential settlement, and site slope instability.
- Discussion of potential geo-hazard mitigation measures as necessary.
- Seismic design parameters per the applicable sections of the 2010 California Building Code (CBC), including Seismic Design Category, Site Class, and Spectral Response Accelerations.
- Brief discussion of generally-appropriate foundation design options.
- Recommendations regarding foundation elements, including:
    - Allowable bearing pressures (dead, live, and seismic loads)
    - Evaluation of potential foundation settlement
    - Minimum foundation embedment
- Recommendations for earthwork; site and subgrade preparation; fill material; fill placement and compaction requirements; and criteria for temporary excavation support.
- Recommendations for construction materials observation and testing.

An environmental site assessment for the presence or absence of any hazardous materials was not included in our scope of work. Although we have explored subsurface conditions, we have not conducted any analytical laboratory testing for the presence of hazardous material of samples obtained.

1.3 Limitations

This report has been prepared for the exclusive use of Points West Surveying, the property owner, their consultants and subcontractors, and appropriate public authorities, for specific application to this proposed project. LGC has endeavored to comply with generally-accepted engineering geologic practice common to the local area at the time this report was prepared. LGC makes no other warranty, express, or implied.

The analyses and recommendations contained in this report are based on data obtained from subsurface exploration. Our methods indicate subsurface conditions only at specific locations where samples were obtained, only at the time they were obtained, and only to the depths penetrated. Samples cannot always be relied on to accurately reflect stratigraphic variations that exist between sampling locations, nor do they necessarily represent conditions at any other time.

The recommendations included in this report are based, in part, on assumptions about subsurface conditions that may only be verified during earthwork. Accordingly, the validity of these recommendations is contingent upon LGC being retained to provide a complete professional service. LGC cannot assume responsibility or liability for the adequacy of the recommendations when they are applied in the field unless LGC is retained to observe construction. We will be glad to discuss a schedule of such observations required to provide assurance of the validity of our recommendations.

Do not apply any of this report's conclusions or recommendations if the nature, design, or location of any of the proposed new developments is changed. If or when changes are contemplated, LGC should be consulted to review their impact on the applicability of the recommendations in this report. LGC is not responsible for any claims, damages, or liability associated with any third party's interpretation of the subsurface data, or reuse of this report for any future projects or at any other locations without our express prior written authorization.

2.0 FIELD EXPLORATION

2.1 Field Exploration Program

A Certified Engineering Geologist from our office visited the project site on January 14, 2013. The field investigation was performed to assess the *in-situ* soil and groundwater conditions, and to estimate the engineering characteristics and properties of the subsurface materials at the project site. Our exploration included five hand-augered borings distributed throughout APN 510-121-026. The hand auger borings were located to provide insight into subsurface conditions on this parcel. Soils observed in the test boring were field-logged and classified in general accordance with ASTM D-2488 visual-manual procedures. The hand auger boring locations are depicted on the site plan included as Figure 3, and soil profile logs are attached as Figures 5 through 9.

2.2 Laboratory Testing

No soil samples were retained from the field exploration and no laboratory analyses were performed for this project. Subsurface soils appeared to be uniformly-distributed across this site and, in stratigraphic order, consist of topsoil, soft to medium soft sandy silt and silt, and medium dense silty sand.

3.0 SITE AND SUBSURFACE CONDITIONS

3.1 Topography and Site Conditions

The existing subject parcel is approximately 1.4 gently-sloping acres and is situated in a residential area of McKinleyville at an elevation of approximately 124 feet above mean sea level, according to Points West Surveying. The parcel slopes to the west, with gradients less than 5 percent. The nearest mapped perennial water course is Widow White Creek, which flows northwesterly, approximately 1,400 feet north of the subject parcel (Figure 1).

3.2 Geologic Setting

This parcel is located within California's northern Coast Ranges Geomorphic Province, a seismically active region in which large earthquakes are expected to occur during the economic life span (50 years) of any developments on the subject property. Mapping by McLaughlin *et al.,* (2000), shows that the site is underlain by Quaternary marine terrace deposits (Qm, Figure 4). The marine terrace surface on which this proposed subdivision will be built is one of a number of emergent terraces in the area. These terraces are preserved as a result of high local uplift rates and fluctuating sea level. The McKinleyville marine terraces were deposited in a near-shore, shallow-water marine environment during late Pleistocene high sea level stands. Carver (1985), assigned this marine terrace to isotope stage 5a, and named it the Savage Creek Terrace. Carver and Burke (1992) thus assigned an age of 83,000 years before present to this feature. Kelly (1984) mapped marine terraces (Qmts) at the site. In the time since this surface has been elevated above sea level, a layer of aeolian silt and sand from the nearby beaches has blanketed the area, leaving a soft, organic-rich topsoil across the McKinleyville area. The McKinleyville aeolian cap is several feet thick near the shore, and thins downwind and inland.

Marine terrace deposits encountered in the on-site exploratory hand auger borings under the topsoils and the aeolian cap, consist of a relatively consistent profile of brown and gray, medium dense, moist to wet, silty sand (SM). On nearby parcels, sands generally contained less than 15 percent silt or gravel. Free water was encountered at between 4.5 to 6.0 feet below the ground surface (bgs), or about 119 feet MSL on January 14, 2013. This elevation (~119') is interpreted to represent seasonally-perched groundwater, and not the true phreatic surface elevation. Similar soil conditions, and medium dense to dense sands were observed on the McKinleyville High School campus in deep borings to 50 feet bgs, where the groundwater was found to be about 15 feet bgs in February 2012 (LGC, 2012).

Underlying the marine terrace deposits at some undetermined depth at the subject property, middle and early Pleistocene sediments consisting of poorly consolidated sandstone is shown on the geologic map (QTw, Figure 4). These deposits include units named nearby by earlier workers, including the Falor Formation of Manning and Ogle (1950), and the Crannel Sands identified by Woodward-Clyde Consultants, (1980). Geologic mapping by Carver (1989), mapped middle and early Pleistocene sediments at an elevation of less than 80 feet, a short distance northwest of this site. The middle and early Pleistocene fluvial and marine sediments are commonly referred to as Falor Formation. Quaternary deposits are underlain at some greater depth, by Cretaceous rocks; mélange and sandstone of the Central Belt Franciscan Complex (Kelly, 1984) which was not encountered in this exploration.

The near-surface soils are composed predominantly of silt with fine sand. Soils, based on our exploratory hand auger borings, are interpreted to be generally flat-lying and uniformly distributed across the subject parcel. Within the areas explored, the soil profile consists of approximately 1 to 1.5 feet of soft organic-rich topsoil overlying 1 to 3 feet of very soft to medium soft sandy silt. Marine terrace deposits consisting of medium dense silty sand were encountered at depths between 2 and 2.5 feet in borings HB-1 through HB-4. While in exploratory boring HB-5, marine terrace sands were encountered at 3.5 feet.

3.3 Seismicity

This project site is located within a seismically active region in which large earthquakes from a variety of sources have the potential to occur during the economic life span (50-years) of a typical structure. North of Cape Mendocino and the Mendocino triple junction, the regional tectonic framework is controlled by the Cascadia subduction zone (CSZ), wherein the Gorda and Juan de Fuca oceanic plates are being actively subducted beneath the North American continental plate.

According to the Alquist-Priolo Special Studies Zone map of the Arcata North quadrangle (CDMG, 1983), there are no faults zoned for special studies on the subject parcel. However, the property is located within the Mad River fault zone, between two fault strands which are zoned as active by the State. These two strands are the McKinleyville fault and the Mad River fault. The McKinleyville fault is located less than 1.5 miles to the northeast. The Mad River fault located less than one mile to the west-southwest.

3.4 Regional Seismicity

Regionally, the project site is subject to ground motion from a number of seismic sources beyond the Mad River fault zone (USGS, 2006, CGS, 1999). These more distal sources include the Little Salmon fault 10 to 12 miles to the southwest, and the Cascadia subduction zone which is less than 40 miles to the west-southwest, and considered capable of producing a great earthquake with an estimated magnitude (moment magnitude, M_W) of 9.0. The subducting Gorda plate is a common source of historic earthquakes felt in the vicinity of McKinleyville, and recent (since ~1850) Gorda plate earthquakes have ranged in magnitude to 7.4 in the earthquake of November 1980.

Faults within the Mad River fault zone, and other proximal North American intraplate faults are considered capable of generating earthquakes with moment magnitudes 7.3 (CGS 1999 & 2000). Other more distal active faults include: the Garberville Briceland fault (~50 mi., M_W 6.9), the Mendocino fault (~50 mi., M_W 7.4), and the San Andreas fault (~50 mi., M_W 7.6).

3.5 Subsurface Conditions

On the day of our field investigation, to explore soil and groundwater conditions, five hand auger borings were extended to maximum depths of up to 7.5 feet below ground surface (bgs). The soil profile as exposed in the test borings was described in general accordance with ASTM D 2488 standards. More-detailed descriptions of the subsurface stratigraphy encountered within our test borings are provided in the attached boring logs (Figures 5 through 9).

Stratigraphy within the upper 2 to 3.5 feet of the soil profile consists of a uniform profile of organic-rich, soft topsoil and sandy silt. Our hand auger borings exposed an intact soil profile without notable modification of the original ground surface due to historic grading or filling. The sod and topsoil in the vicinity of the hand auger borings was approximately 1 to 1.5 feet thick.

3.6 Groundwater Conditions

Groundwater was encountered at a depth of 6 feet bgs in hand borings HB-1 and HB-2 during our field investigation. Secondary porosity was observed to be well-developed in the soil above the water table. Some soil mottling, indicative of transient high groundwater conditions, was also observed near the water table. Groundwater levels on this site will fluctuate somewhat with seasonal or long-term climatic variations and changes in land use.

Due to the subject parcel being underlain by soil materials with well-developed secondary porosity, groundwater is not expected to be encountered at foundation depths during dry-season (May through September) earthwork to depths up to approximately 15 feet bgs. Earthwork during the wet season (October through May) could be adversely affected by soils subject to temporary, season saturation within five feet below anticipated foundation depths. Groundwater conditions are not anticipated to negatively affect foundation performance, or foundation construction during the dry season, but seasonally-perched groundwater is likely to make earthwork problematic during the wet winter months.

4.0 GEOLOGIC HAZARDS

The focus of our geologic hazard assessment for this project site primarily included seismic ground shaking due to near and far seismic sources, the potential for liquefaction of the shallow saturated soils, tsunami, and differential settlement due to undocumented fill soils. Our assessment of these and other common potential geologic hazards is presented below.

4.1 Seismic Ground Shaking and Surface Fault Rupture

As noted in Section 3.3, the project site is situated within a seismically active area proximal to several seismic sources capable of generating moderate to strong ground motions. Given the proximity of the Mad River fault zone to the site, and the Cascadia subduction zone (offshore to the west), as well as other active faults within and offshore of northern California (e.g. Little Salmon fault), the project site is expected to experience strong ground shaking during the economic life span of any proposed developments.

The McKinleyville fault is located less than 1.5 miles northeast of the subject parcel and strands of the Mad River fault zone are less than one mile to the southwest, and is the closest recognized active fault (CDMG, 1998 and 2000). The subject parcel, however, is not located within an Alquist-Priolo earthquake fault zone, in which the State requires special studies for structures for human occupancy. Due to the distance from the project site to the nearest recognized active fault, and based on the information available, the potential for ground surface fault rupture to occur at the project site is considered low.

4.2 Liquefaction

Liquefaction is the loss of soil strength, resulting in fluid mobility through the soil. Liquefaction typically occurs when loose, uniformly-sized, saturated sands or silts that are subjected to repeated shaking in areas where the groundwater is less than 50 feet bgs. In addition to the necessary soil and groundwater conditions, the ground acceleration must be high enough, and the duration of the shaking must be sufficient, for liquefaction to occur. Strong ground shaking and groundwater less than 50 feet bgs are liquefaction conditions that appear to have been met at this site; however, loose, uniformly-sized, saturated sands or silts were not encountered in our explorations.

Based on the planning scenario (CDMG, 1995), the site is located outside of any areas of high liquefaction potential. Within our hand auger borings we encountered soft sandy silt soils and medium dense silty sand at approximate, anticipated foundation load bearing depths. Groundwater was encountered approximately 4.5 to 6 feet below the ground surface in our hand auger borings. Loose saturated sands are unlikely to occur in the shallow subsurface deeper than our hand auger borings. Based on the geological age and density of the native soils at the site, the potential for liquefaction-related settlement or other liquefaction-related phenomenon is considered low at this site.

4.3 Settlement

Based on our exploratory borings, undocumented, non-engineered fill soils do not appear to be present on the subject property. If encountered, undocumented, non-engineered fill soils should be considered unsuitable as foundation load bearing soils due to the potential for excessive total and differential settlement. An apparent lack of fill soils on the site suggests that all present-day foundation elements may be founded on shallow, in-place and undisturbed native soils, and designed for uniform settlement.

For foundation systems designed in accordance with our recommendations, and the standard of care for civil engineering, we estimate that total and differential settlement can be minimized through the design and construction process.

4.4 Landsliding

The subject property is located on a generally flat-lying surface ground surface at an elevation of approximately 124 feet above mean sea level. There are no significant slopes in the vicinity of the project, therefore slope instability or landsliding is not anticipated to have any impact on the proposed project.

4.5 Flooding

The subject parcel is not located adjacent to any watercourses. The Humboldt County Web GIS system shows the parcel to be out of the 100-year flood zone. Consequently, there is no FEMA FIRM Flood rating or Panel number for the site. The hazard of flooding of the existing parcel and the new parcels being created by this subdivision is low.

4.6 Tsunami

As mapped by the State and County, this site is well-above and outside of the Tsunami Hazard zone. Based on the published mapping, the hazard of tsunami inundation is low.

4.7 Soil Swelling or Shrinkage Potential

Subsurface soils at foundation load bearing depths consist predominantly of low-plasticity sandy silt and silty sand. Due to seasonal precipitation, soils were moist to the ground surface at existing grade. Soils are permeable and well-drained. Generally low or lacking in clay, these soils do not appear to be subject to shrink swell associated with cyclic seasonal wetting and desiccation. Additionally, in McKinleyville's moist coastal climate, these soils are unlikely to desiccate to a depth sufficient to affect a typical foundation system built according to current building codes. The hazard to any future structures associated with potential swelling or shrinkage of the soils beneath a spread footing foundation is therefore low.

5.0 CONCLUSIONS AND DISCUSSION

Based on the results of our explorations, it is our opinion that the project site is suitable for its intended use as described in this report. The subject parcel will be subdivided and developed similarly to the majority of the surrounding parcels. Beyond the site plan, there are no plans for any new construction with which this office has been provided. The proposed subdivision will create seven new parcels which will be suitable for construction of lightly-loaded, one or two story wood framed structures supported on foundation systems consisting of either reinforced concrete perimeter spread footings with interior footings, or reinforced, monolithic slab(s) on grade with continuous concrete perimeter footings, and interior spread footings and pads. Due to the soft soils in the shallow subsurface, we will recommend that the foundation loads bear in the medium dense silty sand occurring at depths between 2.5 to 4.5 feet, based on our exploratory borings.

Footings DEPTH

6.0 RECOMMENDATIONS

6.1 Setback Recommendations

From an engineering geologic viewpoint, the potential geologic hazards at this site cannot be mitigated through setbacks. The proposed parcels are situated on gently-sloping ground, away from steep slopes and watercourses. The subject parcel is surrounded by similarly-developed parcels with single-family residences and paved streets.

6.2 Site Preparation

All earthwork, including, but not limited to, site clearing, grubbing, and stripping should be conducted during dry weather conditions. All topsoil (approximately 1.0-1.5 feet), should be removed from within building footprints, from areas 3 feet beyond building perimeters, and from beneath pavement and concrete flatwork areas.

Any undocumented fill soils, and any other debris encountered at or below the existing ground surface should be removed to a depth sufficient to expose firm, undisturbed native mineral soil material. Topsoil should be stockpiled on-site for later use as landscaping material or other nonstructural fill. Approved erosion and sediment controls appropriate for the season, and compliant with State and County regulations, should be emplaced. When the ground is wet, vehicle and equipment traffic should be restricted, and care should be taken to avoid rutting and mixing of disturbed soils or topsoil with the underlying native bearing soils.

6.3 Subgrade Preparation

Areas to receive fill should be stripped of all topsoil, graded to provide a smooth flat bearing surface, scarified to a depth of 8 inches, moisture conditioned and compacted in accordance with our compactions standards (below) to a firm and unyielding surface sufficient to support the anticipated loads. If the exposed subgrade soil is soft or disturbed, or if it proves difficult to compact, it should be excavated additionally to expose more-competent native soils. The resulting subgrade should be scarified and conditioned as recommended above. The excavated material should be replaced with compacted engineering fill as necessary.

6.4 Temporary Excavations

All temporary construction slopes (if any) should be designed and excavated in strict compliance with applicable local, state, and federal safety regulations including the current OSHA Excavation and Trench Safety Standards.

Construction equipment, building materials, excavated soil, vehicular traffic, and other similar loads should not be allowed near the top of any unshored or unbraced excavation. Where the stability of adjoining buildings, walls, pavements, or other similar improvements is, or may be endangered by excavation operations, support systems such as shoring, bracing, or underpinning, may be required to provide structural stability and to protect personnel working in the excavation.

Since excavation operations are dependent on construction methods and scheduling, the contractor should be solely responsible for the design, installation, maintenance, and performance of all shoring, bracing, underpinning, and other similar systems. LGC assumes no responsibility for temporary excavations, the safety thereof, or the design, installation, maintenance, and performance of any shoring, bracing, underpinning, or other similar systems.

6.5 Cut and Fill Slopes

No cut or fill slopes are anticipated for this site. Structural fill on sloping ground (if any) should be placed on a suitably prepared subgrade surface with a slope of no greater than 4H:1V and should be compacted mechanically to reduce any potential for excessive settlement.

6.6 Fill Materials

Aggregate Base

Aggregate base material may be used for pavement subgrade, placed beneath footings or floor slabs, or used as trench backfill. This material should meet the requirements in the Caltrans Standard Specifications for Class 2 Aggregate Base (3/4-inch maximum particle size).

Select Fill

In the case of new construction requiring select fill, that should consist of granular material that may be used as non-expansive fill beneath floor slabs and for the upper portion of pavement subgrade. Select fill should be a soil/rock mixture free of organic material and other deleterious material; on-site native soils are probably not suitable for use as select fill. Select fill material should contain low plasticity clay, well-graded sand, and/or gravel. The material should contain no particles larger than 3 inches in greatest dimension, nor more than 15 percent larger than 2-inches. Additionally, the material should meet the following specifications:

Plasticity index (PI):	<12
Liquid Limit (LL):	<30
Percent passing No. 200 sieve:	50 maximum, 5 minimum

6.7 Compaction Standard

Structural fill and backfill material shall be compacted in accordance with the specifications listed in Table 3 below. Material should be placed in horizontal lifts that do not exceed 8-inches in uncompacted thickness. A qualified field technician should be present to perform field density tests at random locations throughout each lift to verify that the specified compaction is being achieved by the contractor.

Where trenches closely parallel a footing and the trench bottom is within a two horizontal to one vertical plane, projected outward and downward from any structural element, controlled low-strength material (CLSM) concrete slurry should be utilized to backfill that portion of the trench below this plane. The use of slurry backfill is not required where a narrow trench crosses a footing at or near a right angle.

TABLE 3 – STRUCTURAL FILL PLACEMENT SPECIFICATIONS

Fill Placement Location	Compaction Recommendation (ASTM D 1557-Modified Proctor)	Moisture Content (Percent of Optimum)
Granular cushion beneath Floor Slab	90%	-1 to +3 percent
Structural fill supporting Footings	90%	-1 to +3 percent
Structural fill placed within 5-feet beyond the perimeter of the building pad	90%	-1 to +3 percent
Roadway fill placed within 2-feet of the final Pavement grade	95%	-1 to +3 percent
Structural fill placed below the base of the Roadway fill (>2' below final grade)	90%	-1 to +3 percent
Utility trenches within building and beneath pavement areas	95%	-1 to +3 percent
Utility trenches beneath Landscape Areas	90%	-1 to +3 percent

6.8 Seismic Design Parameters

As required by the 2010 CBC, the project site is classified as a Site Class D consisting of "a stiff soil profile" (Section 1613.5.2, CBC, 2010). The following parameters listed in Table 4 are based on this classification and were determined in accordance with the ASCE 7 Standard, Minimum Design Loads for Buildings and Other Structures (USGS, 2012).

TABLE 4: SPECTRAL RESPONSE ACCELERATIONS

Site Location - Latitude: 40.9464° N Longitude: -124.1054° W	
Occupancy Category – II	
Seismic Design Category – E	
Site Class – D	
S_s	**2.698**
S_1	**1.054**
Spectral Response Accelerations (F_a=1.0, F_v=1.5):	
S_{MS}	**2.698**
S_{M1}	**1.581**
S_{DS}	**1.799**
S_{D1}	**1.054**

Engineer to inspect footing/excavations.

6.9 Foundation Design

No specific foundation plans were provided to us for the proposed developments. The following foundation recommendations assume that only typical, lightly-loaded, wood frame, one- or two-story residential structures will be constructed on the new parcels created by this subdivision. In our opinion, these proposed structures can be supported by a monolithic slab on grade with continuous concrete perimeter footing in combination with isolated interior spread footings, or on a perimeter spread footing with interior footings. Foundation of these types are suitable for site conditions provided they are designed and constructed in accordance with our recommendations, and meet the minimum standards of the currently in-force edition of the CBC.

Footings

- Foundations are not anticipated to be located in areas of undocumented fill soils, however there is a possibility that unobserved, undocumented fills could exist on the site. Foundation systems for this site should be designed to limit potential structural damage due to differential settlement resulting from undocumented, compressible fill soils.
- To mitigate undocumented fill soils, excavate and replace with suitable engineered fill, placed and compacted as recommended. Alternately, footings may be built at 1.5 feet below grade on controlled low strength material (CLSM, e.g. concrete slurry) backfilled footing trenches, excavated into the bearing soil indicated in this report.
- Foundations should be embedded a minimum of 18 inches into suitably dense, undisturbed native bearing soils. Based on the soil profile observed on this parcel, the suitably dense, undisturbed native bearing soils usually occur at approximately 12 to 18

ENGINEER MUST INSPECT ALL - FOOTINGS & ISOLATED PIERS inches below existing grade, at minimum. Very soft soils were encountered in HB-5 between 2.5 and 3.5 bgs; similar areas of deep unsuitably-soft soils may exist on-site.

- Minimum width of footings should be 15 inches, and the minimum thickness should be 6 inches, per CBC Section 1809.

Engineer to inspect footing/excavations.

Floor Slab Design

- Concrete floor slabs-on-grade should have a minimum thickness as specified by the CBC or the design engineer, and should be reinforced. Floor slabs should be underlain by at least 8 inches of compacted select fill consisting of Class 1, Type A permeable material (per Caltrans), or an approved equivalent, to act as a capillary moisture break, plus 1 inch of sand as described below.
- To reduce the possibility of moisture migration through a floor slab-on-grade, a minimum 6 mil plastic membrane (vapor retarder) should be placed on the prepared Class 1, Type A gravel subgrade or the approved equivalent.
- Joints between the sheets and utility piping openings should be lapped and taped.
- Care should be taken during construction to protect the plastic membrane against punctures. To protect the membrane during steel and concrete placement, and to provide for a better concrete finish, cover the membrane within at least 1 inch of clean sand.
- The difference, if any, between the 8 inches of select fill plus 1 inch of sand under the slab, and the depth to suitably-firm undisturbed native soil should be made up with additional select fill or engineered fill that is placed as specified in the Structural Fill section of this report.

Allowable Soil Bearing Pressures

- For design of foundation elements embedded into suitably-dense undisturbed firm granular soils, we recommend an allowable bearing pressure of 1,500 pounds per square foot for dead load plus long-term live load, in accordance with Table 1806.2 (CBC, 2010). Lateral bearing pressure is 100 pounds per square foot per foot below native grade. For lateral sliding resistance use a coefficient of friction 0.2 multiplied by the dead load, or a cohesion value of 130 psf.
- The allowable bearing pressure may be increased by one-third when using alternate load combinations in Section 1605.3.2 (CBC, 2010) that include wind or earthquake loads. At minimum, all footings should be designed and sized to be not less than 15 inches wide and 6 inches thick per Section 1809.7 (CBC, 2010).

6.10 Grading, Drainage and Erosion Control

Grade this site to provide positive drainage away from the foundation elements of all structures. With the exception of stormwater detention or retention facilities, no water should be allowed to pond anywhere on the site, nor to migrate beneath any structure.

- A minimum gradient of two percent away from foundations should be maintained for all hardscaped areas.
- At minimum, a five percent gradient should be maintained for landscaped areas within 10-feet of the structure.
- Design the grading to promote sheet runoff and avoid concentrating stormwater runoff.
- Roof storm drainage should be controlled with the installation of gutters and downspouts.
- Connect downspouts to tightlines to convey roof storm runoff away from foundations to suitable outlet points where no erosion will occur.
- Runoff from hardscaped areas, including sidewalks and parking areas, and other impermeable surfaces should be contained, controlled, and directed to suitable outlet points; preferably the gutter in the proposed street.

6.11 Pavement Design Recommendations

This proposed subdivision includes an access road and a small off-street parking area. Based on the soil borings, pavement areas will be underlain by native soils consisting of soft to medium soft sandy silt. We have assumed an R-Value of 15 for sandy silt, and a traffic index of 5.5, per Caltrans' Highway Design Manual (12/1/1981, Table 7-651.5). Based on these assumptions and our field explorations, we recommend design pavement sections consisting of 0.25 foot of AC pavement, over 1.0 foot of Class-2 aggregate base rock, placed and compacted as recommended. As an alternative, the 0.25 foot of AC may be underlain by 0.4 foot of Class-2 aggregate base rock and 0.6 foot of aggregate sub-base or select fill, placed and compacted as recommended.

Subgrade soils to support the design pavement section, or to support structural fill that will in-turn support the pavement section, should first be stripped of unsuitable surface materials (including 1.0 to 1.5 feet of topsoil), cultural debris and any undocumented fill materials.

Procedures and materials specifications for pavements should be in accordance with the current Caltrans Standard Specifications except that 95 percent relative compaction per ASTM D-1557 should be obtained in the upper 6 inches of the subgrade soil and in the Class-2 aggregate base rock in the pavement section. Any soil subgrades or structural fills more than 2.0 feet below the final pavement grade should be compacted to 90 percent relative compaction per ATSM D-1557.

Pavement subgrade soils should be proof-rolled with a minimum 10-ton vibratory steel drum roller or with an approved equivalent. As outlined in Table 3 above, scarify, moisture condition, and compact the upper 6 inches of the native subgrade to a minimum of 95 percent of the maximum dry density (per ASTM D698-91). Moisture content should be controlled to -1 to +3 percent of optimum. The subgrade should be tested and approved prior to placement of any fill.

7.0 ADDITIONAL SERVICES

7.1 Review of Grading and Foundation Plans and Excavations

The conclusions and recommendations provided in this report are based on the assumption that soil conditions encountered during grading and/or foundation construction will be essentially as exposed during our site exploration, and that the general nature of the grading and use of the property will be as described above. In order to provide a complete professional service, Lindberg Geologic Consulting should be retained to provide observation services to assure conformance with the specific recommendations contained within this report including:

- Review of the native soils underlying the roadway and parking area following stripping, and during compaction of the roadway native subgrade.
- Observation of foundation excavations prior to placement of fill, forms or reinforcing steel.

Engineer to inspect footing/excavations.

8.0 REFERENCES

McLaughlin, R. J., S. D. Ellen, M. C. Blake Jr., A. S. Jayko, W. P. Irwin, K. R. Aalto, G. A. Carver, and S. H. Clarke, Jr., 2000, Geology of the Cape Mendocino, Eureka, Garberville, and Southwestern Part of the Hayfork 30x 60 Minute Quadrangles and Adjacent Offshore Area, Northern California.

Carver, G.A., 1985, Quaternary tectonics north of the Mendocino Triple Junction -- the Mad River Fault Zone, in M.E. Savina, ed., American Geomorphological Field Group Field Conference Guidebook, p. 169-83.

Carver, G.A., 1989, Unpublished maps of the Arcata North, Korbel, and parts of Arcata South and Blue Lake quadrangles, 1:24,000 scale, 1989.

CBC [California Building Code], 2010 edition, California Code of Regulations, Title 24, California Building Standards Commission.

CDMG [California Division of Mines and Geology], 2000, Digital Images of Official Maps of Alquist-Priolo Earthquake Fault Zones of California, Northern and Eastern Region.

CDMG, 1998, State of California Special Studies Zones, Fields Landing 7.5' Quadrangle, Humboldt County, California.

CDMG, 1995, Planning Scenario in Humboldt and Del Norte Counties, California, for a Great Earthquake on the Cascadia Subduction Zone, Special Publication 115.

CDMG, 1996, Probabilistic Seismic Hazard Assessment for the State of California. Mark D. Petersen, William A. Bryant, Chris H. Cramer, Tianqing Cao, and Michael Reichle, California. Department of Conservation, Division of Mines and Geology, Arthur D. Frankel, U.S. Geological Survey, Denver, Colorado. James J. Lienkaemper, Patricia A. McCrory, and David P. Schwartz, U.S. Geological Survey, Menlo Park, California, Open-File Report 96-08

Kelley, F.R., 1984, DMG Open-File Report 84-38, Geology and Geomorphic Features Related to Landsliding, Arcata North 7.5' Quadrangle, Humboldt County, California Scale 1:24,000

Manning, G. A., and Ogle, B. A., 1950, Geology of the Blue Lake Quadrangle, California; California Division of Mines and Geology Bulletin 148, 35 p.

Maps of Known Active Fault Near-Source Zones in California and Adjacent Portions of Nevada.

Satake, K., Wang, K., Atwater, B., 2003, Fault slip and seismic moment of the 1700 Cascadia earthquake inferred from Japanese tsunami descriptions. Journal of Geophysical Research, Vol. 108, No. B11, 2535.

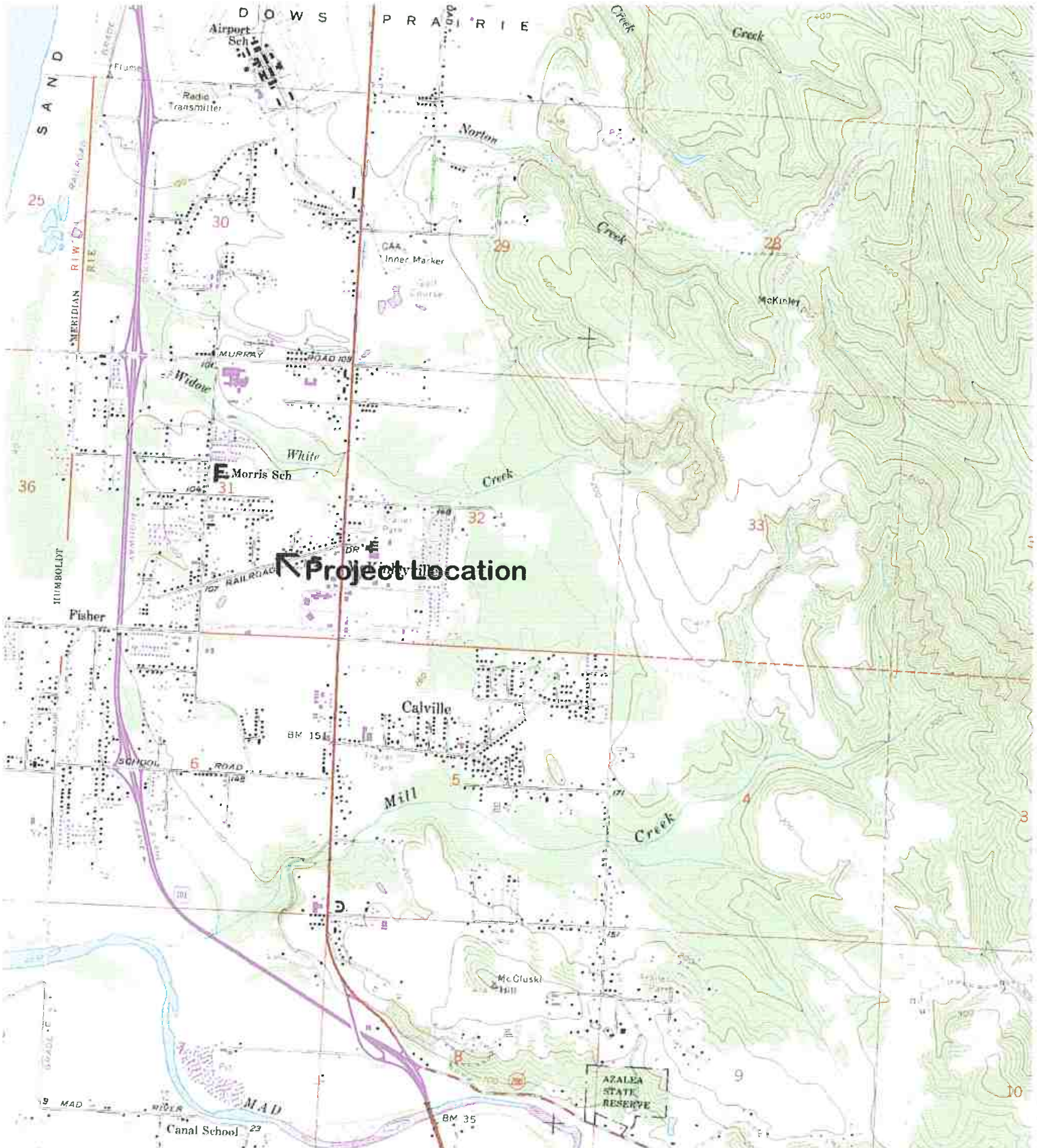
USGS, 2012, Seismic Design Values for Buildings, website, URL: http://earthquake.usgs.gov/research/hazmaps/design/index.php

Woodward-Clyde Consultants, 1980, Evaluation of the potential for resolving the geologic and seismic issues at the Humboldt Bay Power Plant Unit No. 3, Appendixes, Woodward-Clyde Consultants, San Francisco, California.

9.0 LIST OF FIGURES AND APPENDICES

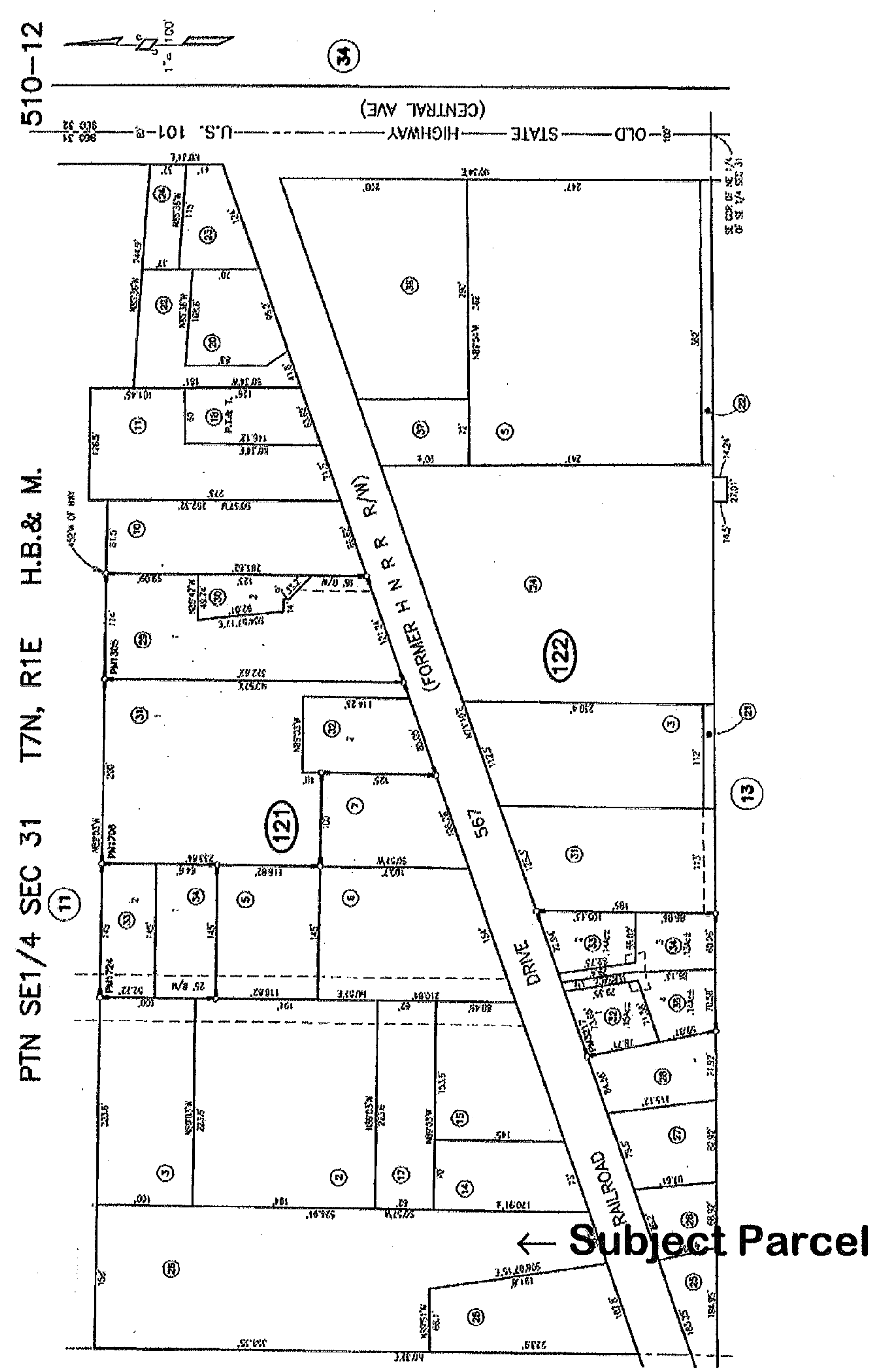
Figure 1: Location Map
Figure 2: Assessor's Parcel Map
Figure 3: Site Map
Figure 4: Geologic Map
Figure 5: Hand Auger Boring Log HB-1
Figure 6: Hand Auger Boring Log HB-2
Figure 7: Hand Auger Boring Log HB-3
Figure 8: Hand Auger Boring Log HB-4
Figure 9: Hand Auger Boring Log HB-5

Lindberg Geologic Consulting	Engineering Geologic Soils Report, APN 510-121-026	Fig. No. 1
P. O. Box 306	1417 Railroad Drive, McKinleyville, California	Date Jan 20, 2013
Cutten, CA 95534	Points West Surveying	Proj. 0054
(707) 442-6000	Location Map, 1" ≅ 2,750'	

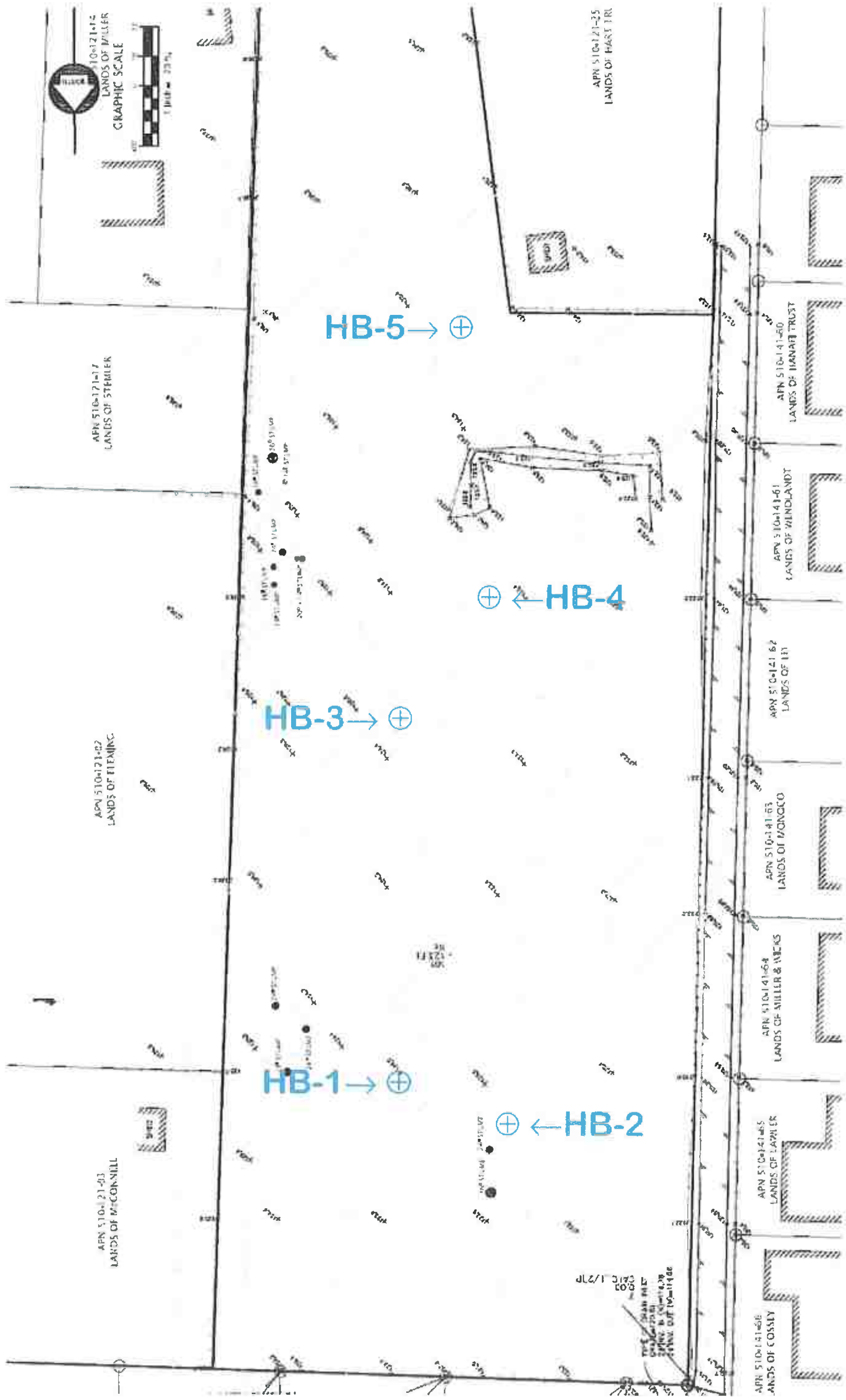


From: USGS Arcata North 7.5' Quadrangle map 1972

Lindberg Geologic Consulting	Engineering Geologic Soils Report, APN 510-121-026	Fig. No. 2
P. O. Box 306	1417 Railroad Drive, McKinleyville, California	Date Jan 20, 2013
Cutten, CA 95534	Points West Surveying	Proj. 0054
(707) 442-6000	Assessor's Parcel Map; Not to scale	

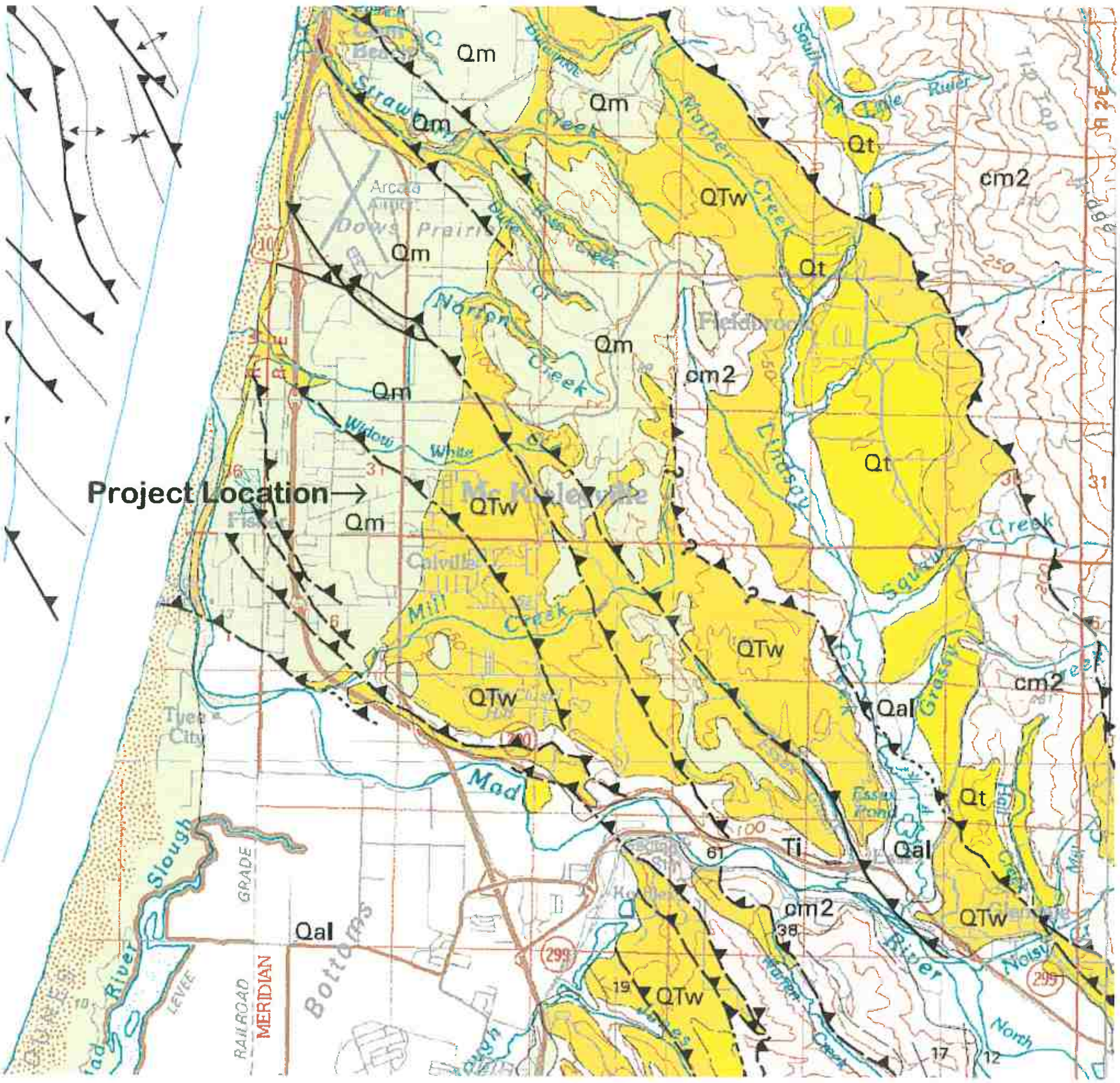


Lindberg Geologic Consulting	Engineering Geologic Soils Report, APN 510-121-026	Fig. No. 3
P. O. Box 306	1417 Railroad Drive, McKinleyville, California	Date Jan 24, 2013
Cutten, CA 95534	Points West Surveying	Proj. 0054
(707) 442-6000	Boring Location Map, (not to scale, all locations approximate)	



From: Points West #605-12_Pettlon_TMAP_11x17

Lindberg Geologic Consulting	Engineering Geologic Soils Report, APN 510-121-026	Fig. No. 4
P. O. Box 306	1417 Railroad Drive, McKinleyville, California	Date Jan 20, 2013
Cutten, CA 95534	Points West Surveying	Proj. 0054
(707) 442-6000	Geologic Map; Not to scale	



From: McLaughlin, et.al., 2000

LABORATORY				FIELD					
Dry Density (pcf)	Moisture Content (%)	Cohesion; Friction Angle (psf; degrees)	Other Tests	Blows/foot*	Sample	Depth (feet)	Graphic Lithology	U.S.C.S. Designation	SOIL DESCRIPTION
						1		ML	Topsoil: Silt with fine sand, very dark brown, soft, moist, organics-rich, trace sand, common fine roots.
						2		ML	Silt with sand, dark brown, soft grading to medium soft, moist, few roots.
						3		SM	Silty sand with clay, yellowish brown, medium dense, moist, slightly plastic, slightly sticky. Clay content decreases with depth to;
						4		SM	Silty sand, yellowish brown, medium dense, moist, friable. Becomes more gray with depth.
						5		SM	Silty sand: Light olive gray, medium dense, moist to wet, with common strong brown mottling.
						6			Groundwater flooded the boring to 6 feet.
						7		SM	Silty sand: Olive gray, medium dense, wet, with occasional strong brown motting.
									HB-1 was located approximately 43 feet northeasterly of Points West's Control Point #1, and was backfilled with cuttings upon completion at 7.5 feet below grade.

* The blow counts have been converted to standard N-value blow counts

SURFACE ELEVATION: 126 Feet

TOTAL DEPTH: 7.5 Feet

GROUNDWATER DEPTH: 6.0 Feet

LOGGED BY: David N. Lindberg, CEG

BOREHOLE DIAMETER: 3.5 Inches

EQUIPMENT: Hand Auger

HAMMER TYPE: None

LINDBERG GEOLOGIC CONSULTING

PROJECT NUMBER: 0054 DATE: Jan. 14, 2013

LOG OF EXPLORATORY BORING/EXCAVATION

HB-1 1417 Railroad Drive Soils

Figure No. **5**

LABORATORY				FIELD					
Dry Density (pcf)	Moisture Content (%)	Cohesion; Friction Angle (psf; degrees)	Other Tests	Blows/foot*	Sample	Depth (feet)	Graphic Lithology	U.S.C.S. Designation	SOIL DESCRIPTION
								ML	Topsoil: Silt with fine sand, very dark brown, soft, moist, organics-rich, common roots.
						1			
						2		ML	Sandy silt, dark brown, medium soft, moist, occasional roots.
						3			
						4		SM	Silty sand, olive brown with strong brown mottling, loose to medium dense, moist to wet slightly plastic and sticky to friable. Slightly clayey from 2.5 to 3.0 feet and decreasing clay with depth.
						5			
						6			Groundwater flooded boring to 6 feet.
								SM	Silty sand, olive gray, medium dense, wet, with occasional strong brown mottling.
						7			HB-2 was located approximately 45 feet northwesterly of Points West's Control Point #1, and was backfilled with cuttings upon completion at 6.2 feet below grade.

* The blow counts have been converted to standard N-value blow counts

SURFACE ELEVATION: 126 Feet

TOTAL DEPTH: 6.2 Feet

GROUNDWATER DEPTH: 6.0 Feet

LOGGED BY: David N. Lindberg, CEG

BOREHOLE DIAMETER: 3.5 Inches

EQUIPMENT: Hand Auger

HAMMER TYPE: None

LINDBERG GEOLOGIC CONSULTING

PROJECT NUMBER: 0054 DATE: Jan. 14, 2013

LOG OF EXPLORATORY BORING/EXCAVATION

HB-2 1417 Railroad Drive Soils

Figure No. **6**

LABORATORY				FIELD					
Dry Density (pcf)	Moisture Content (%)	Cohesion; Friction Angle (psf; degrees)	Other Tests	Blows/foot*	Sample	Depth (feet)	Graphic Lithology	U.S.C.S. Designation	SOIL DESCRIPTION
								ML	Topsoil: Silt with sand, very dark brown, soft, moist, organics-rich, trace sand, common fine roots.
						1		ML	Sandy silt, dark brown, soft grading to medium soft, moist, few roots.
						2		SM	Silty sand with clay, olive brown, medium dense, moist, slightly plastic and sticky.
						3		SM	Silty sand, yellowish brown, medium dense, moist, friable, occasional strong brown mottling.
						4		SM	Silty sand, strong brown, medium dense to dense, moist, friable.
						5			HB-3 was located approximately 83 feet southwesterly of Points West's CP #1, and was backfilled upon completion at 4.5 feet below grade. No groundwater was encountered.
						6			
						7			

* The blow counts have been converted to standard N-value blow counts

SURFACE ELEVATION: 126 Feet

TOTAL DEPTH: 4.5 Feet

GROUNDWATER DEPTH: >4.5 Feet

LOGGED BY: David N. Lindberg, CEG

BOREHOLE DIAMETER: 3.5 Inches

EQUIPMENT: Hand Auger

HAMMER TYPE: None

LINDBERG GEOLOGIC CONSULTING

PROJECT NUMBER: 0054 DATE: Jan. 14, 2013

LOG OF EXPLORATORY BORING/EXCAVATION

HB-3 1417 Railroad Drive Soils

Figure No. 7

LABORATORY				FIELD					
Dry Density (pcf)	Moisture Content (%)	Cohesion; Friction Angle (psf; degrees)	Other Tests	Blows/foot*	Sample	Depth (feet)	Graphic Lithology	U.S.C.S. Designation	SOIL DESCRIPTION
								ML	Topsoil: Silt with sand, very dark brown, soft, moist, trace fine sand, organics-rich, common fine roots,
						1			
						2		ML	Sandy silt, dark brown, soft to medium soft, moist, few roots.
						3		SM	Silty sand with clay, olive brown, medium dense, moist, slightly plastic, slightly sticky, strong brown mottling. Clay decreases with depth.
						4		SM	Silty sand, yellowish brown to strong brwon, medium dense, moist, friable.
						5			HB-4 was located approximately 120 feet southerly of Points West's Control Point #1, and was backfilled with cuttings upon completion. Groundwater was not encountered.
						6			
						7			

* The blow counts have been converted to standard N-value blow counts

SURFACE ELEVATION: 127 Feet

TOTAL DEPTH: 4.2 Feet

GROUNDWATER DEPTH: >4.2 Feet

LOGGED BY: David N. Lindberg, CEG

BOREHOLE DIAMETER: 3.5 Inches

EQUIPMENT: Hand Auger

HAMMER TYPE: None

LINDBERG GEOLOGIC CONSULTING

PROJECT NUMBER: 0054 DATE: Jan. 14, 2013

LOG OF EXPLORATORY BORING/EXCAVATION

HB-4 1417 Railroad Drive Soils

Figure No. **8**

LABORATORY				FIELD					
Dry Density (pcf)	Moisture Content (%)	Cohesion; Friction Angle (psf; degrees)	Other Tests	Blows/foot*	Sample	Depth (feet)	Graphic Lithology	U.S.C.S. Designation	SOIL DESCRIPTION
								ML	Topsoil: Silt with fine sand, very dark brown, soft, moist, trace sand, organis-rich with common fine roots.
						1			
								ML	Silty sand, dark brown, soft to medium soft, moist. few roots.
						2			
						3		ML	Silty sand, dark brown, vey soft from 2.5 to 3.5 feet, moisture increasing.
						4		SM	Silty sand, yellowish brown with strong brown mottling, medium dense. Clay content decreases and moisture increases with depth.
									Groundwater flooded the boring to 4.5 feet.
								SM	Silty sand, yellowish brown and strong brown, medium dense, wet, friable.
						5			
						6			HB-5 was located approximately 22 feet easterly of the northeastern corner of APN 510-121-025, and was backfilled with cuttings upon completion at 5.2 feet.
						7			

* The blow counts have been converted to standard N-value blow counts

SURFACE ELEVATION: 127 Feet

TOTAL DEPTH: 5.2 Feet

GROUNDWATER DEPTH: 4.5 Feet

LOGGED BY: David N. Lindberg, CEG

BOREHOLE DIAMETER: 3.5 Inches

EQUIPMENT: Hand Auger

HAMMER TYPE: None

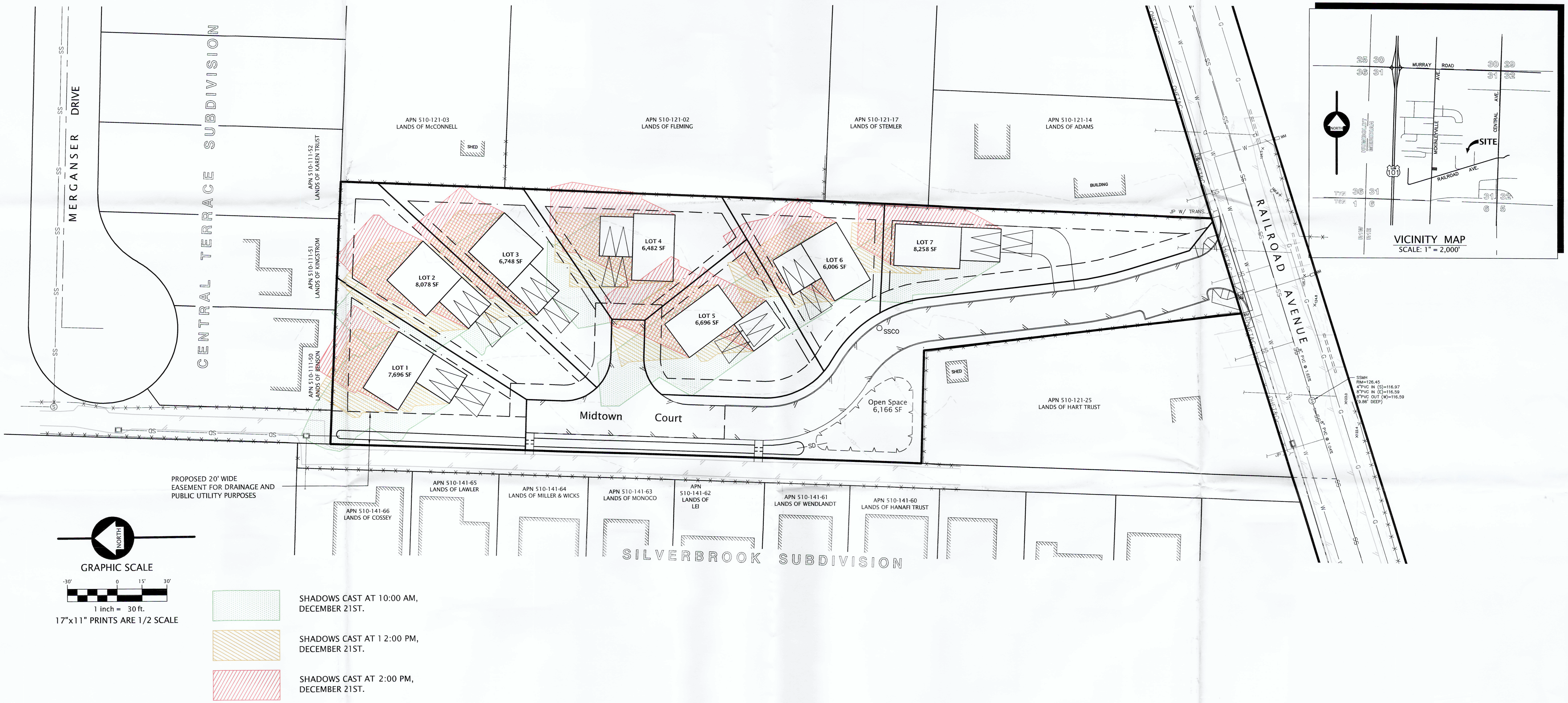
LINDBERG GEOLOGIC CONSULTING

PROJECT NUMBER: 0054 DATE: Jan. 14, 2013

LOG OF EXPLORATORY BORING/EXCAVATION

HB-5 **1417 Railroad Drive Soils**

Figure No. **9**



APN 510-121-026

SOLAR SHADING PLAT

for

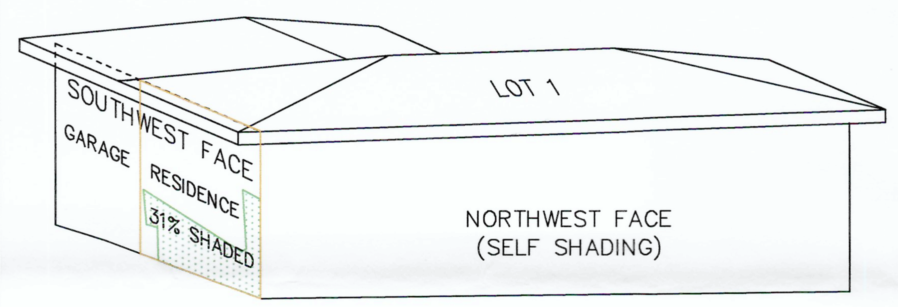
Midtown Court Tract

A PLANNED UNIT DEVELOPMENT
SECTION 31, T7N, R1E,
HUMBOLDT MERIDIAN

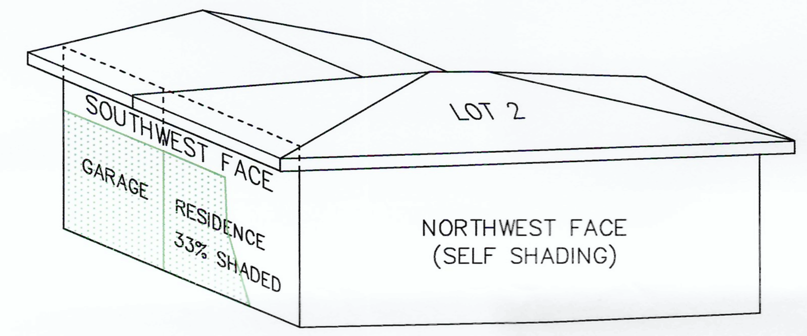
IN THE UNINCORPORATED AREA OF
HUMBOLDT COUNTY, STATE OF CALIFORNIA

SCALE: 1" = 30' APRIL 2015 SHEET 1 OF 2

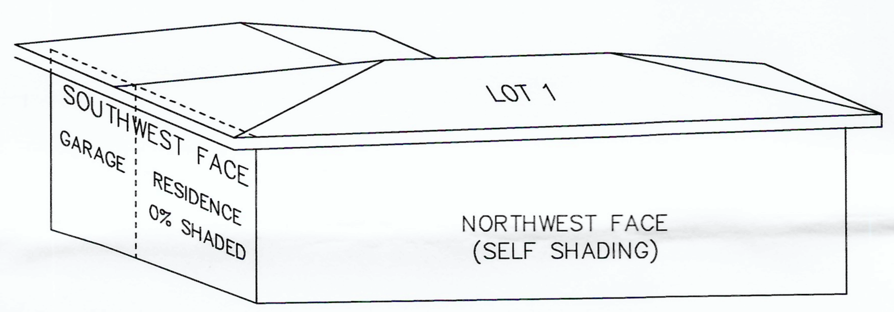
POINTS WEST SURVEYING CO.
5201 Carlson Park Dr., Suite 3 · Arcata, CA 95521
707 · 840 · 9510 · Phone 707 · 840 · 9542 · Fax



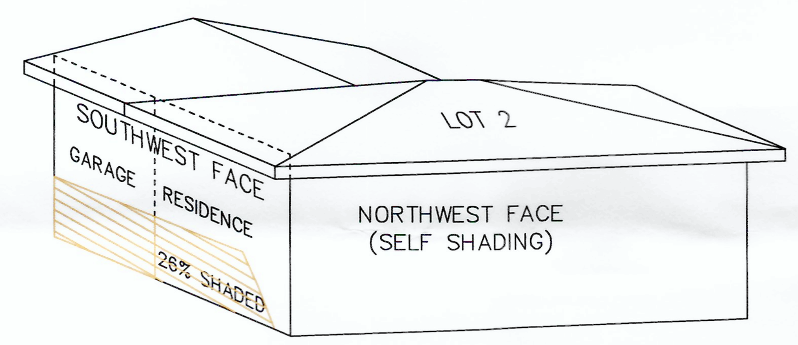
LOT 1 - 10:00 AM
DECEMBER 21



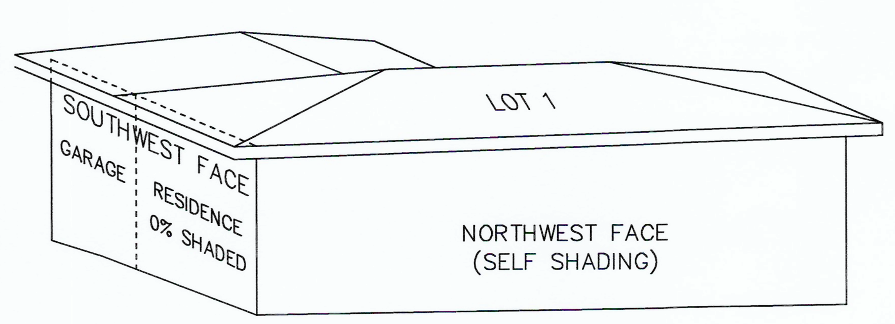
LOT 2 - 10:00 AM
DECEMBER 21



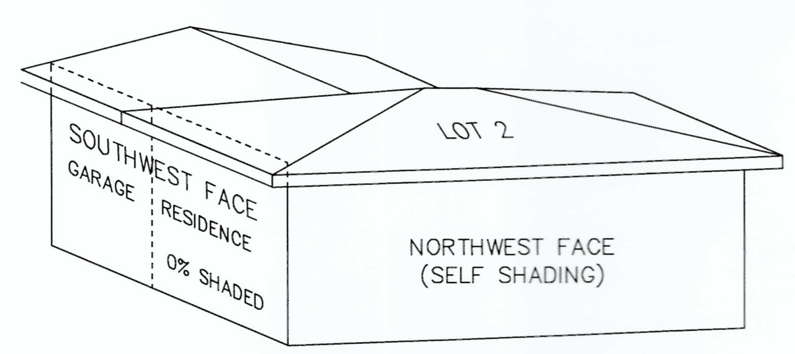
LOT 1 - 12:00 PM
DECEMBER 21



LOT 2 - 12:00 PM
DECEMBER 21



LOT 1 - 2:00 PM
DECEMBER 21



LOT 2 - 2:00 PM
DECEMBER 21

LOT 1 - 10:00 AM THRU 2:00 PM DECEMBER 21
AVERAGE SHADING =10.4%

LOT 2 - 10:00 AM THRU 2:00 PM DECEMBER 21
AVERAGE SHADING =19.7%

NOTES:

Lots 3 through 7 are not shaded in any significant manner by houses created in this subdivision.

Lot 6 is shaded by Lot 7 less than 20% at 10:00 am on December 21st, less than 5% at noon and not at all at 2:00 pm. This equals an average shading of under 9% on December 21st.

SHADOWS CAST AT 10:00 AM, DECEMBER 21ST.

SHADOWS CAST AT 12:00 PM, DECEMBER 21ST.

SHADOWS CAST AT 2:00 PM, DECEMBER 21ST.

APN 510-121-026

SOLAR SHADING PLAT

for

Midtown Court Tract

A PLANNED UNIT DEVELOPMENT
SECTION 31, T7N, R1E,
HUMBOLDT MERIDIAN

IN THE UNINCORPORATED AREA OF
HUMBOLDT COUNTY, STATE OF CALIFORNIA
APRIL 2015

SCALE: 1" = 30'

SHEET 2 OF 2

POINTS WEST SURVEYING CO.
5201 Carlson Park Dr., Suite 3 · Arcata, CA 95521
707·840·9510 · Phone 707·840·9542 · Fax

GRAPHIC SCALE

0 10 20

(IN FEET)
1 inch = 20 ft.
Contour Interval = 1'

EROSION CONTROL NOTES

A. GENERAL

1. THIS PLAN WAS PREPARED BY A QUALIFIED ENGINEER FROM WHITCHURCH ENGINEERING, INC WHO HAS TRAINING AND EXPERIENCE TO HAVE EXPERT KNOWLEDGE OF EROSION AND SEDIMENT CONTROL METHODS.

2. THE SOURCE OF THE BMP'S USED IN THIS PLAN PREPARATION ARE FROM CALIFORNIA STORM WATER BEST MANAGEMENT PRACTICE HANDBOOK AND STATE WATER RESOURCES CONTROL BOARD BEST MANAGEMENT PRACTICE CONSTRUCTION HANDBOOK.

3. THE IMPLEMENTATION OF BMP'S WILL OCCUR WITH THE ONSET OF CONSTRUCTION, IMMEDIATELY AFTER SOIL IS DISTURBED AT SITE. AS SHOWN ON SITE MAP A SILT FENCE WILL BE INSTALLED PARALLEL TO CONSTRUCTION SITE. OTHER EROSION CONTROL ACTIVITIES (HAY BALES, ETC) SHALL BE IMPLEMENTED AS DEEMED BY INSPECTOR.

4. A REPRESENTATIVE FROM WHITCHURCH ENGINEERING, INC SHALL INSPECT EROSION CONTROL MEASURES AFTER A SIGNIFICANT RAIN EVENT. A LETTER FOR EACH INSPECTION SHALL BE SUBMITTED TO THE JOB FILE. THE EROSION CONTROL MEASURES SHALL BE INSPECTED BY THE ENGINEER AS NECESSARY. ANY REQUIRED REPAIRS SHALL BE MADE IMMEDIATELY.

5. THIS PROJECT MAY COMMENCE DURING THE WINTER MONTHS (OCTOBER 15–APRIL 15) THEREFORE EFFORTS WILL BE MADE TO MINIMIZE LAND DISTURBANCES.

B. WATER COURSES

1. EXISTING AND PROPOSED DRAINAGE PATTERNS, CHANNELS AND FACILITIES ARE SHOWN ON ATTACHED PLAN.

2. CHANGES IN FLOW QUANTITIES AND VELOCITIES ARE NEGLIGIBLE; EXISTING SLOPES AND DRAINAGE CHANNELS ARE TO REMAIN UNALTERED. SURFACE WATER FLOW IS BY SHEET FLOW. SLOPE PROTECTION MEASURES SHALL CONSIST OF APPLYING A PROTECTIVE LAYER OF STRAW OR ANOTHER SUITABLE MATERIAL TO SOIL SURFACE AREA.

3. TEMPORARY SLOPE STABILIZATION MEASURES SHALL CONSIST OF MULCHING WITH PROTECTIVE COVERINGS. APPLICATION OF THIS MEASURE SHALL COMMENCE WITH START OF CONSTRUCTION.

4. TEMPORARY CHANNEL TO CONTROL SURFACE WATER FLOW OVER CUT AND FILL SLOPES SHALL BE AN A.D.S PLASTIC PIPE DIRECTED TO ESTABLISHED DRAINAGE.

5. EXISTING GRASS VEGETATED FIELD AREA WILL SERVE TO REDUCE DRAINAGE FLOW VELOCITIES.

6. A TEMPORARY SEDIMENT DETENTION BASIN IS NOT NECESSARY FOR THIS PROJECT.

7. ALL LOOSE SOIL AND DEBRIS SHALL BE REMOVED FROM THE STREET AREAS UPON STARTING OPERATIONS AND PERIODICALLY THEREAFTER AS DIRECTED BY THE INSPECTOR. ALL ENTRANCES SHALL BE MAINTAINED IN A CONDITION THAT WILL PREVENT TRACKING OR FLOWING OF SEDIMENT ONTO PUBLIC RIGHT–OF–WAY.

C. DISPOSAL OF EXCAVATED MATERIALS

1. EXCAVATED MATERIALS SHALL BE HAULED OFF SITE OR USED IN LANDSCAPING ON–SITE.

D. DUST CONTROL

1. EXCESSIVE DUST SHALL BE CONTROLLED AT ALL TIMES DURING CONSTRUCTION AND UNTIL FINAL COMPLETION. THE CONTRACTOR, WHEN HE OR HIS SUBCONTRACTOR ARE OPERATING EQUIPMENT ON SITE, SHALL PREVENT THE FORMATION OF EXCESSIVE AIRBORNE NUISANCES BY WATERING AND/OR TREATING THE SITE OF THE WORK IN SUCH A MATTER THAT WILL CONFIRM DUST PARTICLES TO THE IMMEDIATE SURFACE OF THE WORK AREA. THE CONTRACTOR SHALL BE RESPONSIBLE FOR ANY DAMAGE CAUSED BY DUST FROM HIS OWN ACTIVITIES OR HIS SUBCONTRACTORS ACTIVITIES IN PERFORMING THE WORK UNDER THIS CONTRACT AND SHALL BE RESPONSIBLE FOR ANY CITATIONS, FINES,OR CHARGES RESULTING FROM DUST NUISANCES. DUST CONTROL WILL BE DONE ON A DAILY BASIS.

E. REMOVAL OF VEGETATION AND REVEGETATION

1. VEGETATION REMOVAL IS TO BE LIMITED TO AREA DIRECTLY UNDER PROPOSED CONSTRUCTION.

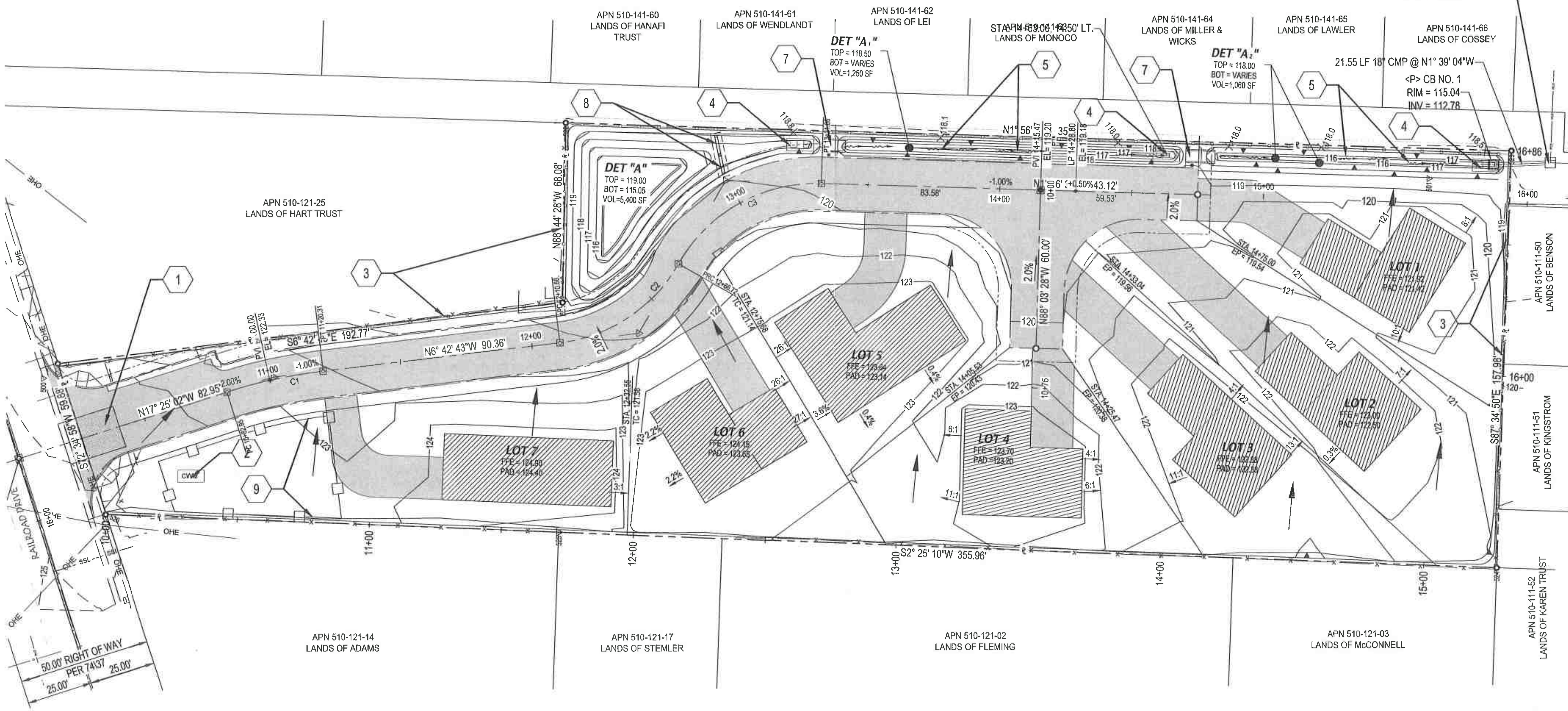
F. FINAL REPORTS AND NOTIFICATION OF COMPLETION

1. UPON COMPLETION OF THE PERMITTED ROUGH GRADING WORK AND AT THE FINAL COMPLETION OF THE WORK, A SET OF REPORTS, DRAWINGS AND SUPPLEMENTS THERETO ARE REQUIRED FOR ENGINEERED GRADING, OR WHEN PROFESSIONAL INSPECTION IS PERFORMED FOR REGULAR GRADING, AS APPLICABLE.

2. THE BUILDING OFFICIAL SHALL BE NOTIFIED WHEN THE GRADING OPERATION IS READY FOR FINAL INSPECTION AFTER ALL WORK AND ALL EROSION–CONTROL MEASURES HAVE BEEN COMPLETED IN ACCORDANCE WITH THE FINAL APPROVED GRADING PLAN, AND THE REQUIRED REPORTS HAVE BEEN SUBMITTED

CENTERLINE CURVE DATA

CURVE No.	LENGTH	RADIUS	Δ
C1	37.37	200.00	10° 42' 19"
C2	56.04	60.00	53° 30' 55"
C3	65.17	60.06	62° 10' 10"



STORM WATER POLLUTION CONTROL CONSTRUCTION KEYNOTES

1. STABILIZE CONSTRUCTION ENTRANCE/EXIT. SEE TC-1 FOR INSTALLATION INSTRUCTIONS.
2. CONCRETE WASTE MANAGEMENT. SEE WM-8 FOR INSTALLATION INSTRUCTIONS.
3. INSTALL SILT FENCE CONTROLS BELOW THE TOE OF SLOPE. SEE SC-1 FOR INSTALLATION INSTRUCTIONS AS INDICATED.
4. SEDIMENT TRAP. SEE SC-3 FOR INSTALLATION INSTRUCTIONS AND SIZING.
5. INSTALL EARTH DIKES/DRAINAGE SWALES AND LINED DITCHES. SS-9 FOR INSTRUCTIONS.
6. EXISTING MUNICIPAL STORM DRAIN INLET
7. 4.0' W x 8" H CELL CONCRETE SCUPPER.
8. CONCRETE STAGED OUTLET WEIR.
9. SURGE AREA; STORAGE AREA & TANKS; SHIPPING & RECEIVING; FUELING; AND VEHICLE & EQUIPMENT STORAGE & MAINTENANCE

LEGEND

- TC-1 STABILIZED CONSTRUCTION ENTRANCE
- SC-3 SEDIMENT TRAP
- CWM CONCRETE WASH-OUT AREA
- SC-1 SILT FENCES
- EXISTING SURFACE FLOW
- SS-9 EARTH DRAINAGE SWALE
- BUILDINGS, COVERED AREAS & ROOFED AREAS
- PAVED AREAS

RECEIVED
JUN 27 2023
Humboldt County
PLANNING

NO.	HISTORY / REVISIONS	BY	CHCK.	DATE

DTN
ENGINEERING
& CONSULTING
dnicoletti@dtnengineering.com

STORM WATER POLLUTION PREVENTION PLAN
MIDTOWN COURT TRACT
SEC 31, T7N, R1E; HUMBOLDT MERIDIAN
APN: 510-121-026-000

DRAWN	SjSa
CHECK	DTN
APPROVED	
DATE	Dec-19
JOB NUMBER	SWPPP
SHEET NO.	1 of 1

USER: steve FILE: D:\DTN_ENG\Humboldt\Midtowne Court\Midtowne Court SWPPP.dwg 3/13/2022 - 8:18am

Preliminary Stormwater Control Plan (CDP, CUP, and SP ≥ 5000 sf)

The flow chart outlines the basic process for discretionary project and subdivision approvals. This is only a guide; not all projects are the same nor is every department. Check with your jurisdictional office for further details on the exact approval process.

Low Impact Development Discretionary Process (CDP/CUP/SP) Flow Chart

Pre-Application

- County provides handouts, worksheets, and over the counter information on LID requirements
- Pre-application meeting occurs and gives the Planner a chance to more thoroughly explain requirements & review preliminary SCP plans
- Final LID worksheets to be completed by Applicant/Agent are included in subdivision application package

Application

- Submit Complete Application, includes: a completed and signed Preliminary SCP with worksheets 1 and 2, Site Plan, Checklists, and any additional requested information

Processing

- Planner completes review of Preliminary SCP worksheet and Tentative Map at project intake
- Completed LID worksheets are included with all referrals

Decision

- Staff Report/ CEQA Document includes a finding of consistency with LID requirement
- Conditions of Approval to record a Notice of Development Plan with notes and details that include proposal/approved LID Improvement
- Final SCP to be completed and submitted with Building/ Grading Permits

Satisfy Conditions of Approval

- Record Notice of Development Plan (Any Treatment/ Baseline Hydromodification Facility must be recorded as well [This will appear on title reports for the property])
- Final SCP and site plan must be completed and submitted with Building/Grading Permits

Construction

- Complete LID improvements concurrent with other construction included in applicable building/grading permits.
- Final sign off of permits, allowing occupancy or initiation of use contingent upon completion of LID improvements to the satisfaction of the Building division
- Annual Self-Certification Compliance Check (O&M) Required for all Treatment/Baseline Hydromodification LID Features utilized in the project

Preliminary Stormwater Control Plan (CDP, CUP, and SP ≥ 5000 sf)

For Office Use Only
Application No.____________________
Received By: ____________________

Instructions

The following worksheet is used to demonstrate that for each and every lot, the intended use can be achieved with a design which disperses runoff from the roofs, driveways, sidewalks, streets and other impervious areas to self-retaining pervious areas. It is also used to demonstrate that drainage to treatment and/or flow control facilities is feasible and that the project is in overall compliance with the MS4 permit. Use this form to assist you in designing your project to comply with the design standards for Multi-Parcel Regulated projects. The completed, signed Preliminary SCP for Subdivision Projects, a site map, plus any additional applicable information, must be submitted with your application to the Planning Department.

Project Name: MIDTOWN COURT TRACT

Physical Site Address: APN 510-121-026

Project Applicant: ANDY WU

Mailing Address: 6456 WHITE LILY ST. EASTVALE, CA, 92880

Phone: (909) 628-5248

Consultant's Information

Name: DAVID NICOLETTI PE QSD

Firm: DTN ENGINEERING

Address: 2731 K ST UNIT A EUREKA, CA, 95501

Email: dnicoletti@dtnengineering.com

Phone: (916) 215-7769

A. Project Information

1a. Does Project create or replace 1-acre or more of impervious surface?	☑ **Yes** (see question below)	☐ **No** (skip question 1b.)
b. If 'Yes' to the above question than does project increase impervious surface from pre-project conditions?	☑ **Yes** (hydromodification requirements must be met)	☐ **No** (regulated project requirements must be met)
Total pre-project Impervious Surface (sf):	~~16,210 SF~~ Ø SF	
Total new or replaced Impervious Surface Area (square feet) [Sum of impervious area that will be constructed as part of the project]	16,210 SF	

Preliminary Stormwater Control Plan (CDP, CUP, and SP ≥ 5000 sf)

B. Summary Table of Pervious to Impervious Surface

The following table will be used by staff to ensure that adequate measures have been utilized within the project design to capture, retain, and/or infiltrate the design storm.
Each DMA shown in the table shall be designated with the same name on the site plan. All site design measures used to meet the runoff reduction goals and all treatment facilities utilized to capture remaining runoff volumes must be shown on the site plan at an appropriate scale. Please use the Flowchart as a reference of the process.

1. **Utilize Worksheet 1 to Calculate Impervious to Pervious Ratio to determine if further runoff reduction is needed**
2. **Utilize the Runoff Reduction Calculator (Worksheet 2*) to increase reduction**
3. **Utilize Bioretention or equivalent if reduction cannot be achieved using site design measures**

DMA Name	Does pervious to impervious ratio Achieve 3.5:1 or better, Worksheet 1(Yes or No)	Does runoff reduction with site design measures equal 100% or greater, Box DD (Worksheet 2)	Value from Box BB (worksheet 2) Impervious surface amount that must be treated using additional methods	Bioretention facility name and size (sf) (Use a sizing factor of 0.04 to calculate bioretention facility size or equivalent sizing technique if different treatment/baseline hydromodification facility is proposed)
(A)	(B)	(C)	(D)	
Example A	Yes	Yes	-----------	------------
Example B	No	Yes	------------	--------------
Example C	No	No	1350 sf	C : (1350 X .04)=54 sf
DMA 1 ETIRE SUB.	NO	YES	9620 SF	SEE DRAINAGE REPORT

*Worksheet 1 and 2 showing calculations for each DMA must be included with the Preliminary SCP
Attach additional sheets as needed for the table above

Preliminary Stormwater Control Plan (CDP, CUP, and SP ≥ 5000 sf)

C. Preliminary Site Plan Checklist –items that must be include on the site plan

- ☐ Topographic lines (2 ft. contours)
- ☐ On-site waterways/drainages, vegetation and areas to be left undisturbed all shown with appropriate buffers
- ☐ DMAs clearly delineated and labeled with name and area (square feet)
- ☐ Location of site design measures used in worksheet 2
- ☐ Location, size, and name of Bioretention/Treatment Facility
- ☐ Flow direction that clearly demonstrates the ability of self-retaining areas, ~~infiltration site design measure~~, and treatment facilities to capture runoff from impervious surfaces
- ☐ Hydrologic soil class

D. Operation and Maintenance Plan Requirements

Each Bioretention facility or equivalent will be required to have an operation and maintenance plan attached to the final SCP and shall include all details found in Appendix 3, 4, and 5 of the LID Manual.

E. Additional Requirements

A detailed final Stormwater Control Plan with narrative sections will need to be submitted prior to issuance of a grading/building permit (see, Appendix 1. However, by completing the Preliminary SCP a more efficient and timely review of the final SCP is enabled.

F. Signature and Certification

I, the below signed, confirm that I have accurately described my project to the best of my ability, and that I have not purposely omitted any detail affecting my project's classification for storm water regulation. I hereby certify that the site design measures and storm water flow treatment measures identified herein as being incorporated into my project have been designed in accordance with the approved BMP Fact Sheet or equivalent, and are included in the final site plans. I also hereby certify that my project meets the storm water runoff reduction criteria identified in Worksheet 2, or as determined through other approved means.

[signature] PE
Signature

6/27/23
Date

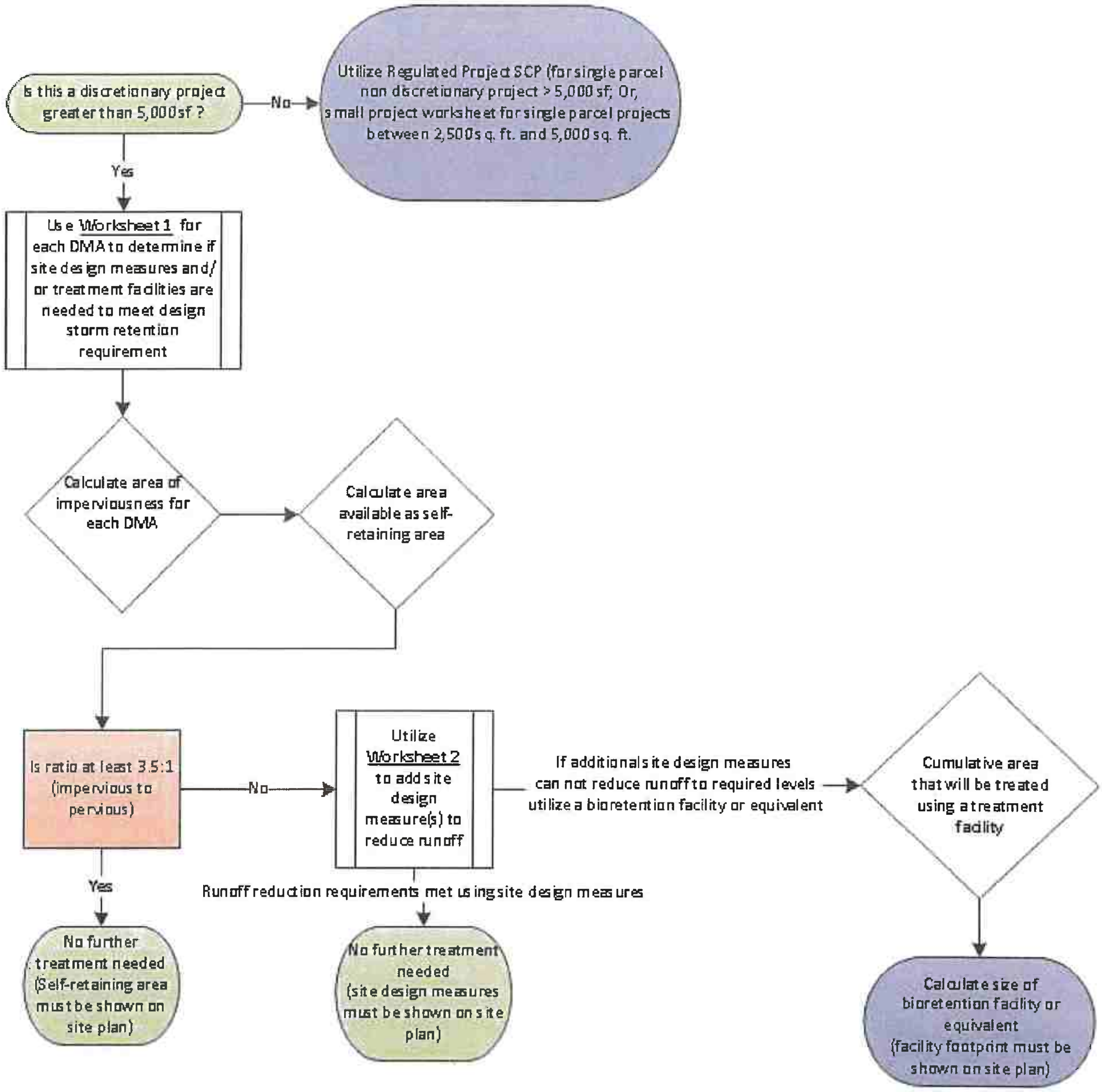
DAVID NICOLETTI
Print Name

I am the:

☐ Property Owner ☐ Applicant ☐ Contractor

Preliminary Stormwater Control Plan (CDP, CUP, and SP ≥ 5000 sf)

The following example illustrates the elements necessary for evaluating a project for compliance with the MS4 permit only. Additional requirements will most likely be needed for compliance with other regulations please consult the full planning submission checklist to make certain all required elements are presented on the preliminary site plan.



Preliminary Stormwater Control Plan (CDP, CUP, and SP ≥ 5000 sf)

Worksheet 1 Example

Regulated Projects Worksheet 1 - Humboldt Low Impact Development Stormwater Manual

DMA Name	Total Post Project Impervious Surface Area (square feet)	Pervious Self-Retaining Area[1] (square feet)	Ratio of Impervious Surface Area to Self-Retaining Pervious Surface Area	Does Ratio Achieve 3.5 : 1 ratio or better of Impervious Surface Area to Self-Retaining Pervious Surface Area (Yes or No)[2]
Example A	500	150	3.3 : 1	YES
Example B	500	100	5.0 : 1	NO
DMA [illegible]	16,210	[illegible]	[illegible]	YE[illegible]
DMA 1 (ENTIRE SUB)	16,210	6210	[illegible]	2.6:1 YES

1: Self-Retaining Areas where impervious surface runoff is directed to the Pervious Self-Retaining Area in accordance with Humboldt LID Manual - Part C, Section 6.0

2: If "Yes", Ratio of Impervious Surface Area to Self-Retaining Pervious Surface Area is equal to 3.5:1 or better (1.3:1 or better in the Shelter Cove MS4 area), then compliance with runoff reduction measures have been met for DMA.
If "No", Ratio of Impervious Surface Area to Self-Retaining Pervious Surface Area does not achieve 3.5:1 or better (1.3:1 in Shelter Cove), then compliance with runoff reduction measures have not been met for DMA (Complete Worksheet 2).

Preliminary Stormwater Control Plan (CDP, CUP, and SP ≥ 5000 sf)

Worksheet 2: (Use one Worksheet for each DMA as applicable)

Regulated Projects Worksheet 2 Humboldt Low Impact Development Stormwater Manual					
Project Information					*Formulas/Notes*
DMA Name:					
Total Post-Project Impervious Surface Area (square feet)	27,210	A		square feet	
24 hour - 85th Percentile Design Storm		B	.65	inch	B = Select Design Storm Value (0.65-inch Humboldt Bay Area, 1.3-inch Shelter Cove)
Impervious Surface Runoff Value **(Potential Stormwater Runoff due to impervious surface area and design storm value)**	10,986	C		Gallons per 24 hours	C = A x B x 0.083 x 7.48
Pervious Self-Retaining Area (SRA) Credit (if applicable, if none enter 0)					
Self-Retaining Area (square feet)	3105	SRA Credit	6210	square feet	SRA Credit = Self-Retaining Area x Multiplier Select Multiplier (3.5 Humboldt Bay Area, 1.3 Shelter Cove)
Site Design Measure Credits					
Tree Planting and Preservation					
New Trees	# of trees				
100 square feet per deciduous tree	D	Ø	E	square feet	E = D x 100
200 square feet per evergreen tree	F	Ø	G	square feet	G = F x 200
Existing Trees (Credit for 50% of existing canopy area)	Canopy diameter (feet)				
Tree #1	H_1	Ø	J_1	square feet	$J_1 = 3.14 \times (H_1/2)^2 \times 0.50$
Tree #2	H_2	Ø	J_2	square feet	$J_2 = 3.14 \times (H_2/2)^2 \times 0.50$
Tree #3	H_3	Ø	J_3	square feet	$J_3 = 3.14 \times (H_3/2)^2 \times 0.50$
Rain Barrel or Cisterns (55 gallon minimum)					
Square foot credit per gallon based on 24-hour, 85th Percentile Design Storm	K	Ø			K = Select square foot credit per gallon (2.48 Humboldt Bay Area, 1.24 Shelter Cove)
	Gallons				
Rain Barrels	L	Ø	M	square feet	M = L x K
Cisterns	N	Ø	O	square feet	O = N x K
~~Infiltration Trench/Basin (55 gallon minimum ~ 21 ft^3**)~~	cubic feet				
~~volume(ft^3) = length x width x depth~~	P		~~Q~~	~~square feet~~	~~Q = P x R x K x 7.48~~
~~porosity (approximate %)~~	R	~~35%~~			
~~**Subsurface Infiltrators (55 gallon minimum)**~~					
~~Proprietary units vary, insert estimated storage in ft^3~~	S		~~T~~	~~square feet~~	~~T = S x 7.48~~
Impervious Area Disconnection Credit per square foot of impervious area feeding into pervious area	11,380	U		square feet	U = Enter square foot value
Soil Quality Improvement					
Credit per square foot of soil quality improvement		V		square feet	V = Enter square foot value
Green Roof					
Credit per square foot of green roof installation		W	0	square feet	W = Enter square foot value
PPPP (Porous Asphalt, Pervious Concrete, Permeable Pavers)					
Credit per square foot of PPPP		X	0	square feet	X = Enter square foot value
Vegetated Swales					
Credit per square foot of vegetated swale		Y	6210	square feet	Y = Enter square foot value
Stream Setbacks and Buffers					
Credit per square foot of stream setback and buffer#		Z		square feet	Z = Enter square foot value
Credits Total	17,590	AA		square feet	AA = SRA Credit + E + G + J_1 + J_2 + J_3 + M + O + Q + T + U + V + W + X + Y + Z
Post-Project Impervious Surface Area minus Site Design Measure Credits	9620	BB		square feet	BB = A - AA
NEW Impervious Surface Runoff Value **(Potential Stormwater Runoff due to impervious surface area and design storm after implementation of Site Design Measures)**	5973	CC		Gallons per 24 hours	CC = BB x B x 0.083 x 7.48
Percent reduction in Impervious Surface Runoff Value*		DD	46	%	DD = ((C - CC) / C) x %100

***If value for DD is not greater than or equal to %100 then bioretention is required for treating remaining runoff from impervious area indicated by value BB. Design and implement bioretention facility in accordance with Humboldt LID Stormwater Manual - Part C.**

****Infiltration Trench/Basin calculations are based on porosity (35%). Increased trench dimensions (volume) are required to meet 55 gallon minimum capacity.**

Green	Fill in (Enter Value)	Conversions Used:
Red	Calculated Value	1 inch = 0.083 feet
Black	**Fixed Value/Selectable Value**	1 cubic foot = 7.48 gallons

Regulated Projects Worksheet 2, Version 2.0 - June 29, 2016

check with agency with project area jurisdiction for requirements