DRAINAGE REPORT

PROJECT: 22A267

PCD File No. PPR2325

Town of Ramah Wastewater System Improvements

PREPARED FOR:

Town of Ramah
113 S. Commercial Street
Ramah, CO 80832

PREPARED BY:

EVstudio, LLC
5335 W. 48th Ave Suite 300
Denver, CO 80212
Prepared By: Timothy Osborn, EIT
Reviewed By: Brian Welch, P.E.
(303) 670-7242 x50

DATE: June 12, 2024

Engineer's Statement:

The Attached Drainage plan and report were prepared under my direction and supervision and are correct to the best of my knowledge and belief. Said drainage report has been prepared according to the criteria established by the El Paso County for drainage reports and said report is in conformity with the master plan of the drainage basin. I accept responsibility for any liability caused by any negligent acts, errors, or omissions on my part in preparing this report.

SIGNATURE: ______________________________

Registered Professional Engineer State

of Colorado No. 52002

(Affix Seal)

COLORADO LICENSED
BRIAN M. WELCH
52002
PROFESSIONAL ENGINEER

Owner/Developer's Statement:

I, the owner/developer, have read and will comply with all of the requirements specified in this drainage report and plan.

Cindy Tompkins, Town Clerk 6-20-24

Name: Date

Town of Ramah
113 S. Commercial Street
Ramah, CO 80832

El Paso County:

Filed in accordance with the requirements of the Drainage Criteria Manual, Volumes 1 and 2, El Paso County Engineering Criteria Manual and Land Development Code as amended.

______________________________ ____________

Joshua Palmer, P.E. Date
County Engineer / ECM Administrator

Conditions:

Table of Contents

I. SITE SCOPE & DESCRIPTION

A. PROJECT NAME AND LOCATION

Project/Site Name: Town of Ramah Wastewater System Improvements
Location: Section 1, Township 11 South, Range 61 West of the 6^{th} P.M.
City: Town of Ramah
County: El Paso County
Latitude: 39.1157° N **Longitude:** -104.1611° W

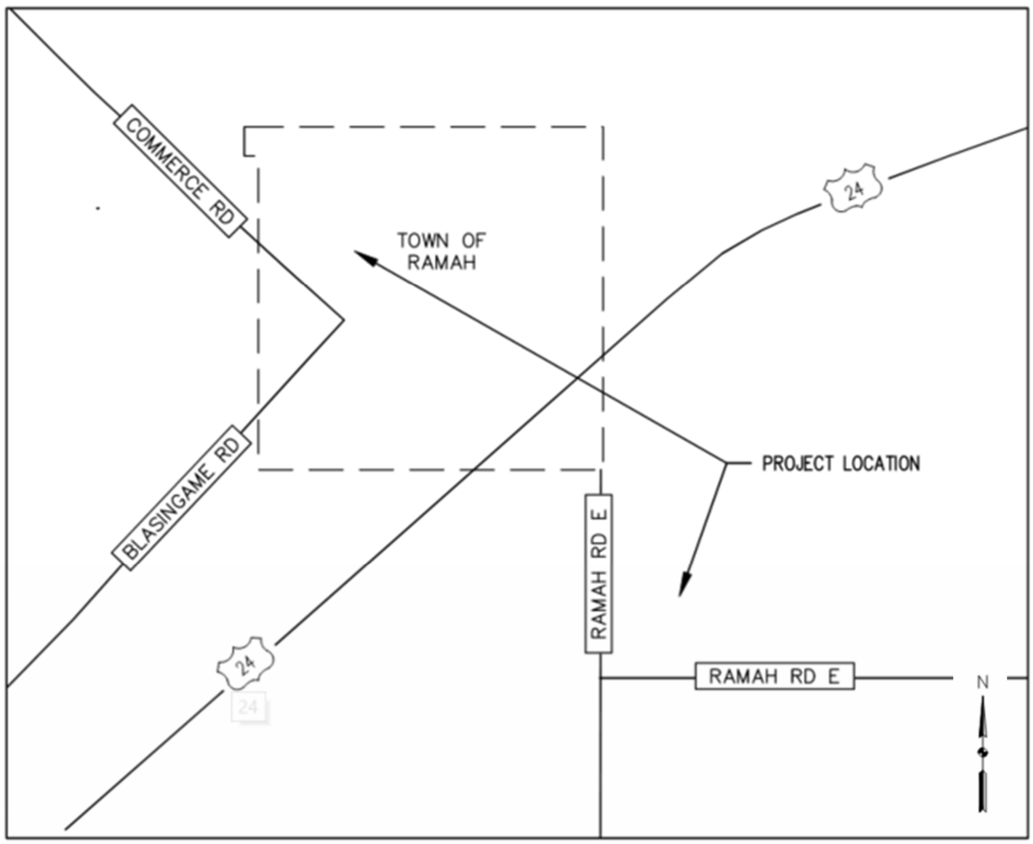


Figure 1. Vicinity Map of Town of Ramah Wastewater System Improvements

The Town of Ramah Wastewater System Improvements is located within Section 1, Township 11 South, Range 61 West of the 6^{th} Principal Meridian, Town of Ramah, County of El Paso, State of Colorado. The site is bounded by an existing street, Ramah Road E, to the south and west. To the west, adjacent to the site is Ramah Cemetery.

B. Description of Property

The site area and the area of the proposed land disturbing activities at the site is approximately 17 acres. The existing vegetation on the property consists primarily of native grasses, shrubs, and weeds with an estimated pre-existing ground cover estimate of 60%. Pre-existing vegetation was determined through the evaluation of satellite imagery of the site. Natural topography at the evaporative pond site slopes to the north towards an unnamed swale (discharges to Big Sandy Creek) at slopes ranging from 2% to 7%. There are no wetlands or receiving waterways within the project area or within 50 feet of the project area.

According to the Geotechnical Engineering Study for the Ramah Wastewater Treatment Plant by Kumar & Associates, Inc. Soils on the site consist of clay with sand (CL) or sandy clay (CL). Because of the predominance of clay in the soil the hydraulic soil group throughout the site will be considered as Class "C".

The nearest waterway is Big Sandy Creek and is the property adjacent to a non-perennial stream that leads into the Big Sandy Creek. No construction is planned within the flood plain. Historical runoff from the site enters this tributary and into Big Sandy Creek.

There are no irrigation facilities observed on the site. and there are no other utilities observed on the site.

C. Scope

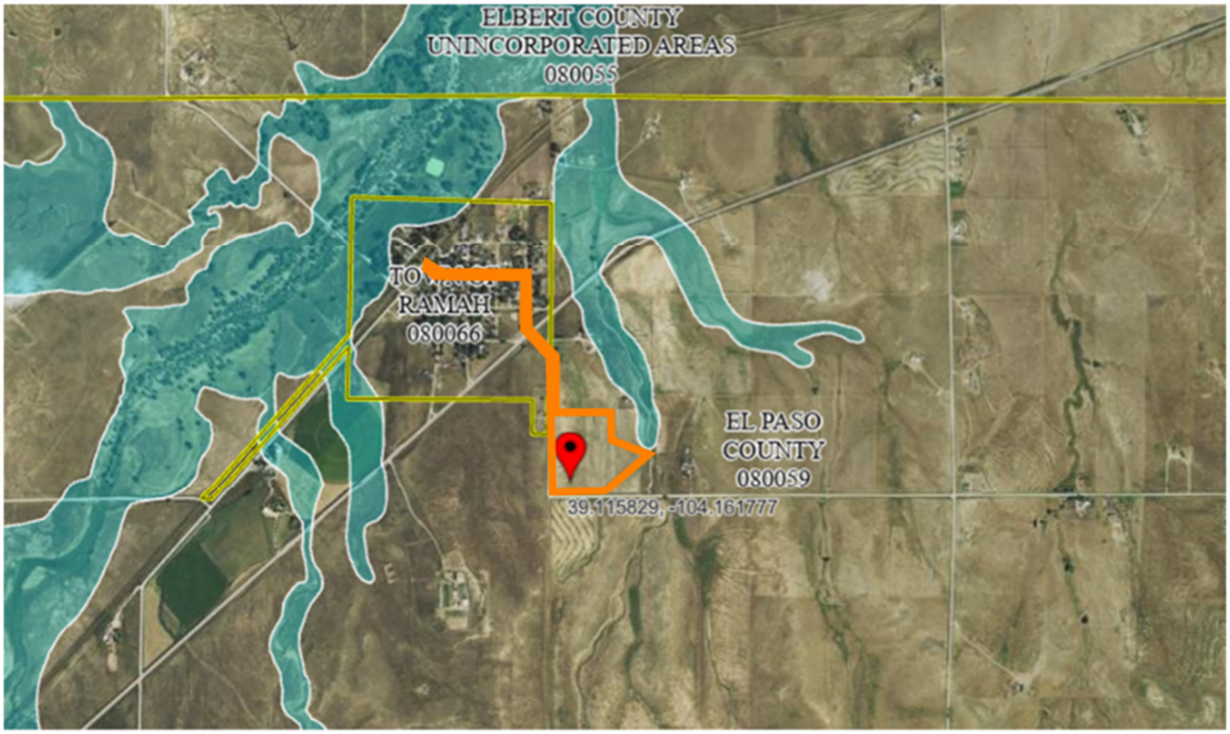
This Project will include the addition of a lift station inside the northeast bend of Rock Island Ave. Sanitary utility will then be added down the following roads: N Commercial St., 2nd St., S Chestnut St., across Highway 24, and finally to Ramah Rd. E. finally leading to the new wastewater treatment ponds.

The building of the wastewater treatment ponds will involve mostly the grading of the ponds and a drainage swale.

II. DRAINAGE BASINS AND SUBBASINS

A. Major Basin Description

The Ramah wastewater treatment plant is located inside the Ramah Drainage Basin which is part of the watershed of Big Sandy Creek, and it is located adjacent to an unnamed non-perennial stream that flows into Big Sandy creek and is part of the 100-year flood plain. There are no portions of the proposed wastewater treatment plant that are inside the flood plain.



There are no portions of the proposed wastewater treatment plant that are inside the flood plain. The flood plain is located in the plains of Colorado and contains low brush and the occasional tree and continues southeast until it joins the Arkansas River.

B. Sub-basin Description

Runoff from the site starts at the crown of Ramah Road East that surrounds the south and west side of the site. Runoff from Ramah Road enters the site and crosses the site in a northeast direction until reaches the stream that flows north into the Big sandy creek.

Ramah Road that runs along the south and west sides of the project site has a remnant of a roadside ditch that regularly spills flows across the site. Ramah road prevents additional offsite runoff from entering the site.

C. Existing Conditions

There are 2 main discharge points on the property where the evaporative wastewater treatments ponds will be placed, and they have been labeled as Labeled EX-1 and EX-2. Additionally, a minor change to a small area in the town of Ramah where a wastewater lift station will be built.

EX-1: Starts at the high point along the county road on the west side of the site. Is the largest portion of the site with slopes between 8% - 2% in the northeast direction towards a low point in the north of the site. These flows then travel to the ephemeral steam to the east of the site. The runoff leaving this site during a 5-year storm is 7.17 cfs, and for the 100-year storm 51.0 cfs.

EX-2: Starts at the high point along the county road on the south side of the of the site. The slopes proceed to the northeast at 2% where it enters a rundown leading into the ephemeral stream to the east of the site. The runoff leaving this site during a 5-year storm is 1.92 cfs, and for the 100-year storm 10.5 cfs.

EO-1 and **EO-2** Is the portions of Ramah Rd that produces runoff and flows into the property. The runoff from for EO-1 is 4.9 cfs, and for EO-2 is 11.3 cfs.

EX-LS: The Lift station is a small area where some excavation for a lift station will take place. Its slopes towards the southwest at 4%-2% into an open space before eventually entering the Big Sandy Creek. The runoff leaving this site during a 5-year storm is 0.19 cfs, and for the 100-year storm 0.7 cfs.

Basin runoff						
Basin ID		C Values		Runoff (cfs)		Comments
		5-year	100-year	5-year	100-year	
Existing						
EX-1	1	0.16	0.50	7.17	51.0	
EO-1	1	0.19	0.52	0.81	4.9	
EO-2	1	0.20	0.52	1.86	11.3	
EX-2	2	0.21	0.53	1.92	10.5	
EX-LS	4	0.40	0.65	0.19	0.7	
EX Total		0.17	0.51	11.95	78.36	

III. DESIGN CRITERIA

A. Development Criteria Reference

The drainage plan is developed to follow the El Paso County Engineering Criteria Manual (ECM), the El Paso County Drainage Criteria Manual, and the Mile High Flood District Urban Storm Drainage Criteria Manual.

B. Hydrologic criteria

The Four Step Process

Step 1: The site has minimal added impervious. The top of the berms that surround the ponds will be a gravel road, A grass swale is used along the south side of the treatment ponds. And the flows to the north will travel through reseeded grasses.

Step 2: Stabilized drainage ways. The channel along the south side of the ponds will be a grassed swale. The properties of the swale can be found in section IV.B "Channelized Flow" of this drainage report. The natural Drainage way receiving this flow will be receiving flows at a velocity that results in a Froude number below 0.8 and it used by El Paso County as an indicator of erosion potential in channels. The Existing flow path of channelized flow runs down the side of the ephemeral stream. This steep area can be armored against erosion with 18.8 in thick Class L riprap with running slopes between 8:1-5:1.

Step 3: Provide Water Quality Capture Volume. The evaporative wastewater treatment ponds comply with ECM App I.7.1.C.3. The Compliance calculations are found in section IV.B "Water Quality Compliance" of this drainage report.

Step 4: Consider Need for Industrial and Commercial BMPs. The site will involve the treatment of wastewater and is designed to comply with the evaporative wastewater treatment ponds section 8.4.0 of CDPHE Wastewater Design Criteria. Because of this storm water may only leave the site through evaporation. Process Design Report (PDR) provides confirmation that the water supply rights are adjudicated and/or permitted for consumptive use through evaporative disposal systems and no obligations exist for discharge to satisfy downstream water rights ownership.

Water Quality Criteria

ECM App I.7.1.C allows for projects to utilize one of 6 water quality standards. This design does not follow the common Water Quality Control Volume Standard ECM App I.7.1.C.1 Because the design for these evaporative wastewater treatment ponds complies with the Runoff Reduction Standard of ECM App I.7.C.3 without any additional design alterations.

The water quality for the wastewater treatment plant will follow ECM Runoff Reduction standards "The control measures is designed to infiltrate into the ground where site geology permits, evaporate, or evapotranspire a quantity of water equal to 60% of what the calculated Water Quality Control Volume (WQCV) would be if all impervious area for the applicable development site discharged without infiltration" (ECM App I.7.1.C.3). Additionally, any area that is excluded or that is outside of the limits of construction will not contribute to the calculation of the WQCV.

Water Quality Exclusions

The installation of a new wastewater main will fall under the Aboveground and Underground Utilities exception "Activities for installation or maintenance of underground utilities or infrastructure that does not permanently alter the terrain, ground cover, or drainage patterns from those present prior to the construction activity. This exclusion includes, but is not limited to, activities to install, replace, or maintain utilities under roadways or other paved areas that return the surface to the same condition" (I.7.1.B.4 ECM).

Design Storms

For the purpose of this report the design storm for the site will be based on the 25-year 24-hour design storm (3.39 in.) because of the concurrent requirements for wastewater evaporative ponds. This modification to perspective results in considering a total runoff volume that is greater than the 100-year 1-hour design (2.97 in.) when assuming no infiltration. No infiltration is assumed when sizing the pond because of the proposed clay liner that is designed to prevent wastewater effluent from entering ground water. When calculating the peak runoff, the 100-year 1-hour was chosen because it represents the greatest peak flow generated by the design storm. Rainfall projections are from NOAA Atlas 14 Point Precipitation Frequency Estimates - Simla Station.

Open Channel Design Criteria

According the El Paso County ECM 6.5 Open channel Design criteria the Flow capacity will be calculated using the Mannings equation. The Channel Velocity will not Exceed 6.0 ft/sec. and the Froude Number will not be greater than 0.9 in the 100-year storm.

IV. DRAINAGE FACILITY DESIGN

A. General Concept

The detention area for the wastewater treatment ponds is defined by the crown of the access road that surrounds the ponds. The only runoff that leaves the site is from areas outside of the pond. Because of the berms that surround the proposed ponds there are two flow paths that will surround the site. A swale is proposed on the south side of the site and will direct flows from the road to the nearby stream. The north half of the site Sheet flows off the site flowing northwest along natural drainage patterns into the stream that enters the Big Sandy Creek.

Because of the large area occupied by the WWTP a large portion of the runoff is detained by the ponds and 40 cfs of projected runoff is being detained by the ponds. The total runoff from disturbed area in basins A-1 (8.5 cfs), A-2 (12.1 cfs), and A-3 (7.6 cfs) and are 28.2 cfs combined during a 100-year storm. This proposed runoff is less than the existing Basin EX-1 which represents the disturbed area with a runoff of 51.0 cfs during a 100-year storm. With 28.2 cfs of runoff from the site being less than the existing 51.0 cfs, the site complies with state and county requirements that the runoff release rate is not greater than 90% of the existing release rate.

B. Specific Details

Water Quality Compliance

The water quality criteria that will govern the site is the runoff reduction standard, and this standard requires that at least 60% of the total WQCV from the site is detained and infiltrated or evaporated from the site. The rainfall and the generated water quality control volume that falls within the wastewater treatment pond will all be evaporated; these ponds have no outflow and will treat 100% of the WQCV from the WWTP Basin.

The Remaining areas utilize infiltration BMPs such as grass buffers and grass swales.

Basin A-1:The added impervious areas from around the north and east portions of the treatment ponds will 70 or more feet of grassed area before leaving the site. This grass buffer will treat 100% of the water quality Control Volume.

Basin A-2: the Added impervious area from the south portion of the treatment ponds enters a large, grassed swale before leaving the site to the east. This swale will treat 100% of WQCV generated by the added impervious.

Basin A-3: Is an area that will receive some impact over the course of construction, but no planned grading will happen in this area. This area will be reseeded to reflect existing conditions after the course of construction.

Basins OS-1, OS-2, and OS-3 Is the portions of Ramah Rd that produces runoff and flows into the property. With OS-3 being composed of most of EX-2 and is not considered as part of the comparison of the proposed site runoff.

Basin LS is the Lifts station and will utilize the grasses area withing the construction area to treat the WQCV. This grass buffer will treat 70% of the WQCV from the lift station.

The Water quality reduction percentages for the infiltration BMPs were calculated using the UD-BMP-v3.07 Runoff Reduction spreadsheet. A copy of this spreadsheet can be found in the appendix

Runoff and Release

The ponds release water through evaporation only. Because of the shared design use of the wastewater ponds no runoff can be safely released from the ponds. Instead, the ponds are sized to allow for the evaporation of the major rain events throughout the year. No Emergency Spillway will be provided.

There are four design points that represent the runoff the leaved the disturbed areas of the properties. These basins are combined additively for this section of the report for ease of comparison and review. The values in this section are not used for the design of conveyance facilities such as grass swales.

Design Point 1 Flows north and receives flows from A-1 (8.5 cfs), and OS-1 (4.9 cfs). The total runoff that is allowed to flow north via sheet flow contributes 13.4 cfs. The type of flows entering the stream or adjacent properties or streams is considered to be channelized flow because the rational method assumes that sheet flow will channelize within 300 feet. Because of this assumption channelized flow across grassed fields is considered for all flow paths and is not considered to be a change to the types of flows onto adjacent sites. The runoff from A-1 is 16.6% of the existing runoff from EX-1

Design Point 2 is composed A-2 (12.1 cfs), OS-2 (11.3 cfs), and OS-3 (9.6 cfs). Historically the areas that contribute to OS-2 and A-2 flowed towards design point 1, however because of the placement of the pond, runoff from these areas will be more concentrated and concentrate sooner than existing conditions would expect. If this flow was directed north the concentrated flows could negatively impact the adjacent property; therefore, A bypass Swale is proposed to route flows into the nearby stream adjacent to the property. The total runoff leaving the site this way total 33.0 cfs. This existing rundown that was being utilized by EX-2 was already experiencing observable erosion. Because of the added flows and to protect against the existing erosion the rundown is proposed to be armored with class m riprap. Details for the rundown are available in the appendices. The runoff from A-2 is 23.7% of the existing runoff from EX-1

Design Point 3 comes from flows on the east side to the treatment pond and are composed of the flows from area A-3 (7.6 cfs). This area will be revegetated and will act as a grass buffer for the runoff in this area. The runoff from A-3 is 14.9% of the existing runoff from EX-1. This design point combines with the runoff from design point 1. Combined A-1 and A-3 combine to 16.1 cfs for 31.6% of the existing runoff from EX-1. When combined with offsite flows the runoff to the north of the site will be 21.0 cfs and is less than the historic runoff of 55.9 cfs from EX-1 and EO-1.

Design Point 4 Is the runoff that leaves the wastewater Lift station LS (0.7 cfs) as it passes over the revegetated area and resumes with existing flow patterns. No significant contribution to the runoff from this area is observable due to the limited scope of the impact and the minimum time of concentration.

Due to downstream water rights stormwater detention ponds have a requirement to drain the EURV in 72 hours, and the 100-year runoff volume in 120 hours. Because the evaporative ponds will not release water downstream the Process Design Report (PDR) provides confirmation that the water supply rights are adjudicated and/or permitted for consumptive use through evaporative disposal systems and no obligations exist for discharge to satisfy downstream water rights ownership.

Basin runoff						
Basin ID		C Values		Runoff (cfs)		Comments
		5-year	100-year	5-year	100-year	
Existing						
EX-1	1	0.16	0.50	7.17	51.0	
EO-1	1	0.19	0.52	0.81	4.9	
EO-2	1	0.20	0.52	1.86	11.3	
EX-2	2	0.21	0.53	1.92	10.5	
EX-LS	4	0.40	0.65	0.19	0.7	
EX Total		0.17	0.51	11.95	78.4	
Basin Runoff						
Basin ID	DESIGN POINT	C Values		Runoff (cfs)		Comments
		5-year	100-year	5-year	100-year	
Proposed						
WWTP		0.19	0.52	6.50	40.0	Contained on site
A-1	1	0.18	0.51	1.29	8.5	
OS-1	1	0.19	0.52	0.81	4.9	
A-2	2	0.17	0.52	2.02	12.1	
OS-2	2	0.20	0.52	1.86	11.3	
OS-3	2	0.22	0.53	1.77	9.6	
A-3	3	0.15	0.50	1.03	7.6	28.2 cfs < 51 cfs
LS	4	0.44	0.68	0.21	0.7	
Detained Runoff					40.0	cfs
Site Runoff					28.2	cfs
Bypassed Runoff					25.8	cfs
Total Site Runoff					54.7	cfs
Design Point 1 Sum					13.4	cfs
Design Point 2 Sum					33.0	cfs

The runoff generated by the wastewater treatment ponds (WWTP) will be fully detained as required by CDPHE Wastewater Design Criteria: The Runoff potential of 6.5 Cfs and 40 cfs is based on what would leave the site if it was not detained by the WWTP. Resulting in the total runoff from the major storm will always be less than existing conditions. The primary purpose of the ponds is wastewater treatment and the detention and evaporation of runoff from the wastewater treatment ponds is regulated by the Colorado Department of Public Health and Environment.

The Lift Station (LS) will compose of mostly underground utility improvements. The added imperviousness from the monitoring equipment and access is relatively minor and when applying the rational method when comparing those changes with the C values provided by El Paso County ECM the resulting runoff was found to not increase. Because of this no additional storm water conveyance is proposed for the LS.

The design for the wastewater treatment ponds follows the Colorado Department of Public Health and Environment (CDPHE) Wastewater Design Criteria Policy which lays out the requirements for the Wastewater treatment ponds and dictates the requirements for the evaporation calculations. Calculations for the pond sizing can be found in Appendix B.

Channelized flow

The equation used for the open channel analysis for the proposed ditch that runs along the south side of the wastewater treatment ponds is the Mannings equation

Where:

$$Q = 1.486 / n\ R^{2/3} S^{1/2} A$$

Q = Flow (cfs)
R = Hydraulic Radius
S = Slope (ft/ft)
A = Area (ft ^{2})
n = Roughness Coefficient (0.020)

The manning number used for this design is 0.030 and represents the vegetation that will be seeded inside the swale. This N Value is also representative of the channel before the grasses are established. It matches what would be present both before and after the seedling is established.

The channel that runs around the south of the site. 4:1 side slopes and a channel slope of 0.006 ft/ft. The peak flow for this channel was calculated with the basins flowing together as a whole with an overall time of concentration of 17 minutes, and an average runoff coefficient of 0.50. The peak flow for the swale during a 100-year storm is 33.0 cfs. The flow depth required to safely convey these flows in the designed swale is 2.71 feet deep at the last turn passing the WWTP.

The free board for the end of the swale is:

$$\text{Freeboard (feet)} = 1.0 + 0.025(Q/A)(d)^{0.33} = 1.09$$

The total required depth by the end of the swale is 2.81 feet. The depth of this swale at this point of the channel is 4 feet.

A Froude Number of 0.8 is used by El Paso County for grass swales when giving approval. The Froude Number is the ratio of the velocity divided by the clarity of the water in the channel. The Froude Number for the channel was found to be 0.62-0.74.

	Bottom width	Side slope 1	Side Slope 2	Channel Depth	Mannings	channel slope	Q (cfs)	Flow Area	Velocity	Froude	Requaired Freeboard	Requiared Depth
start	0.00	0.25	0.25	0.74	0.03	0.006	5.13	2.20	2.33	0.68	1.05	1.79
SW corner	0.00	0.25	0.25	0.98	0.03	0.006	10.7	3.81	2.80	0.71	1.07	2.05
SE corner	0.00	0.25	0.25	1.29	0.03	0.006	22.3	6.61	3.37	0.74	1.09	2.38
Run Down	0.00	0.25	0.25	1.62	0.04	0.01	33.3	10.50	3.17	0.62	1.09	2.71

C. Costs and Fees

Drainage basin Fee

This development takes please in the Ramah Drainage Basin which is located in the Big Sandy Creek River basin. This Basin is not listed in El Paso County Drainage Basins Fees Resolution No. 22-442 and 23-35. Because this is a non-platting project there is no drainage basin fee.

Drainage Construction estimates

The evaporative wastewater treatment ponds satisfy the drainage requirements for the project. The following pricing is a list of the related costs for building the ponds and is not an addition to the pond itself.

STORMWATER CONSTRUCTION ITEMS				
DESCRIPTION	QTY	UNIT	UNIT PRICE	SUBTOTAL
Pond Excavation	125,500	CY	$ 3.50	$ 439,250.00
Evaporation Pond Ballasting	1	LS	$ 100,000.00	$ 100,000.00
Miscellaneous Site Work	1	LS	$ 5,000.00	$ 5,000.00
Seeding/Erosion Control/Site Restoration	1	LS	$ 69,000.00	$ 69,000.00
Rundown riprap	1	LS	$ 8,000	$ 8,000

V. REFERENCES

Urban Storm Drainage Criteria Manual (USDCM) Volumes 1-3, Mile High Flood District (MHFD)

El Paso County Drainage Basins Fees Resolution No. 22-442 and 23-35, El Paso County.

Drainage Criteria manual of El Paso County, Colorado

Engineering Criteria Manual County of El Paso, Colorado

Wastewater Design Criteria Policy, Colorado Department of Public Health, and Environment (CDPHE)

USDA- Agricultural Research Service Equations, *Design of Rock Chutes* (by K.M. Robinson, et al, USDA-ARS, 1998 Transactions of ASAE) and An Excel Program to Design Rock Chutes for Grade Stabilization, (K.M. Robinson, et al, USDA- ARS, 2000 ASAE Meeting Presentation). This method is based on laboratory data for slopes from 2 to 40 percent.

APPENDIX A: HYDROLOGIC COMPUTATIONS

PROJECT: **Element Eng - Ramah Lift Station**
SUBJECT: Imperviousness and Runoff C Calculations

JOB #: **22A267**
DATE: **6/12/2024**
BY: **TRO**

			Pasture/Meadow		Paved		Gravel		Soil Type "C" Composite Runoff Factors		
Basin Name	**Square Footage**	**Acres**	**A&B SQ-FT**	**C&D SQ-FT**	**A&B SQ-FT**	**C&D SQ-FT**	**A&B SQ-FT**	**C&D SQ-FT**	**C_5**	**C_{100}**	**I %**
EXISING CONDITIONS											
EX-1	726451	16.7	0	711051	0	0	0	15400	0.16	0.51	1.70%
EX-2	125185	2.9	0	109787	0	0	0	15398	0.21	0.53	9.84%
EO-1	56887	1.31	0	51739	0	0	0	5148	0.19	0.52	7.24%
EO-2	137291	3.15	0	124292	0	0	0	12999	0.20	0.52	7.57%
EX-LS	5884	0.14	0	3952	0	1932	0	0	0.40	0.65	32.83%
EX Total	1051698	24.1	0	1000821	0	1932	0	48945.00	0.17	0.51	3.91%
PROPOSED CONDITIONS											
WWTP	415923	9.55	0	379530	0	0	0	36393	0.19	0.52	7.00%
A-1	102563	2.35	0	96879	0	0	0	5684	0.18	0.51	4.43%
A-2	128471	2.95	0	122607	0	0	0	5864	0.17	0.51	3.65%
A-3	103548	2.38	0	102563	0	0	0	985	0.15	0.50	0.76%
LS	5884	0.14	0	3644	0	2240	0	0	0.44	0.68	38.07%
Total Disturbed	756389	17.4	0	705223	0	2240	0	48927	0.18	0.52	5.47%
OS-1	56887	1.31	0	51739	0	0	0	5148	0.19	0.52	7.24%
OS-2	137291	3.15	0	124292	0	0	0	12999	0.20	0.52	7.57%
OS-3	100237	2.30	0	86365	0	0	0	13872	0.22	0.53	11.07%
Total Basin	1050804	24.1	0	981491	0	2240	0	67073.85	0.18	0.52	5.32%

		Type C 5-year		Type C 100-year	
Lans Use	**Imp**	HSG A&B	HSG C&D	HSG A&B	HSG C&D
Paved	100%	0.90	0.90	0.96	0.96
Gravel	80%	0.59	0.63	0.70	0.74
Pasture/Meadow	0%	0.08	0.15	0.35	0.50

PROJECT: Element Eng - Ramah Lift Station
SUBJECT: TIME OF CONCENTRATION

JOB #: 22A267
DATE: 6/12/2024
BY: TRO

TIME OF CONCENTRATION

			TIME (Ti) *[Max. 300']*					TRAVEL TIME (Tt)							Tc CHECK (Urbanized Basins)			FINAL Tc	Time to	
Basin	Area	5Yr.	Elevations		Dist.	Slope	Ti	Elevations		Dist.	Slope		Vel.	Tt		Length	Regeion	10	Peak**	Remarks
No.	(acres)	co-eff.	Upstream	Downstream	(ft)	(%)	(min)	Upstream	Downstream	(ft)	(%)	*	(fps)	(min)	Tc	(ft)	(ft)	(min)	Flow	
								EXISTING												
EX-1	16.68	0.16	6166	6145	300	7.0	15.5	6145	6121	1250	0.5	5	1.10	18.9	34.4	1550	18.6	10.0	**18.6**	
EX-2	2.87	0.21	6165	6159	50	12.0	5.0	6159	6128	1154	2.7	5	2.46	7.8	12.8	1204	16.7	10.0	**12.8**	
EO-1	1.31	0.19	6154	6146	200	4.0	14.6	6146	6115	300	10.3	5	5.10	1.0	15.6	500	12.8	10.0	**12.8**	
EO-2	3.15	0.20	6154	6138	140	11.4	1.9	6138	6128	1154	0.9	5	1.50	12.8	14.7	1294	17.2	10.0	**14.7**	
EX-LS	0.14	0.40	6153	6137	10	25.0	1.4			50	1.0	5	1.60	0.5	1.9	60	10.3	10.0	**10.0**	
								PROPOSED												
WWTP	9.55	0.19			20	100.0	1.6	0		100	0.5	5	1.10	1.5	3.1	120	10.7	10.0	**10.0**	
A-1	2.35	0.18	6142	6139	100	3.0	11.6	6137	6115	600	0.5	5	1.10	9.1	20.7	700	13.9	10.0	**13.9**	
OS-1	1.31	0.19	6154	6146	200	4.0	14.6	6146	6115	300	0.5	5	1.10	4.5	19.2	500	12.8	10.0	**12.8**	
A-1 + OS-1	**3.66**	**0.16**	**6154**	**6137**	**140**	12.1	**1.93**	**6137**	**6115**	**600**	**3.7**	**5**	3.06	**3.27**	**5.2**	**740**	**14.1**	**10.0**	10.0	
A-2	2.95	0.17	6138	6137	70	1.4	2.7	6137	6130	460	1.5	5	1.80	4.26	7.0	530	12.9	10.0	**10.0**	
A-3	2.38	0.15	6138	6137	140	0.7	4.9	6137	6128	1154	0.8	5	1.40	13.74	18.7	1294	17.2	10.0	**17.2**	
OS-2	3.15	0.20	6154	6138	140	11.4	1.9	6138	6128	1154	0.9	5	1.50	12.82	14.7	1294	17.2	10.0	**14.7**	
OS-3	2.30	0.22	6169	6161	100	8.0	1.76	6161	6128	1154	2.9	5	2.58	7.45	9.2	1254	17.0	10.0	**10.0**	
A-3 + OS-2 + OS-3	**8.40**	**0.16**	**6153**	**6137**	**140**	11.4	**1.96**	**6137**	**6128**	**1154**	**0.6**	**6**	1.44	**13.36**	**15.3**	**1294**	**17.2**	**10.0**	**15.3**	
LS	0.14	0.44			10	25.0	0.29			50	1.0	5	1.60	0.52	**0.8**	60	10.3	10.0	**10.0**	

**NOTE: EL PASO REQUIRES MIN. Tc=10 MIN. FOR NON-URBANIZED & Tc=5 MIN. FOR UBRANIZED

<u>* Type of Land Surface for Overland Travel Time</u>	<u>VELOCITY COEFFICIENTS</u>
1 = Heavy Meadow	2.5
2 = Tillage / Field	5
3 = Short pasture & lawns	7
4 = Nearly bare ground	10
5 = Grassed waterway	15
6 = Paved areas and shallow paved swales	20

STANDARD FORM SF-3
STORM DRAINAGE SYSTEM DESIGN
(RATIONAL METHOD PROCEDURE)

CALCULATED BY:	**TRO**	**JOB NO:**	**22A267**
DATE:	**6/12/24**	**PROJECT:**	**Element Eng - Ramah Lift Station**
CHECKED BY:	**BMW**	**DESIGN STORM:**	**5 Year**

			DIRECT RUNOFF						
BASIN	Design Point	AREA DESIG.	AREA (Acres)	RUNOFF COEFF	Tc (min)	C A (Acres)	I (in/hour)	Q (cfs)	REMARKS
EX-1	1	**EX-1**	16.68	0.16	18.61	2.67	2.68	**7.17**	
EX-2	2	**EX-2**	2.87	0.21	12.83	0.60	3.20	**1.92**	
EO-1	1	**EO-1**	1.31	0.19	12.78	0.25	3.20	**0.81**	
EO-2	1	**EO-2**	3.15	0.20	14.71	0.62	3.02	**1.86**	
EX-LS	3	**EX-LS**	0.14	0.40	10.00	0.05	3.54	**0.19**	
WWTP		**WWTP**	9.55	0.19	10.00	1.83	3.54	**6.50**	
A-1	1	**A-1**	2.35	0.18	13.89	0.42	3.10	**1.29**	
OS-1	1	**OS-1**	1.31	0.19	12.78	0.25	3.20	**0.81**	
A-1 + OS-1	1	**A-1 + OS-1**	3.66	0.18	10.00	0.67	3.54	**2.37**	
A-2	3	**A-2**	2.95	0.19	10.00	0.57	3.54	**2.02**	
A-3	2	**A-3**	2.38	0.15	17.19	0.37	2.80	**1.03**	
OS-2	2	**OS-2**	3.15	0.20	14.71	0.62	3.02	**1.86**	
OS-3	2	**OS-3**	2.30	0.22	10.00	0.50	3.54	**1.77**	
A-3 + OS-2 + OS-3	2	**A-3 + OS-2 + OS-3**	8.40	0.19	17.19	1.62	2.80	**4.54**	
LS	4	**LS**	0.14	0.44	10.00	0.06	3.54	**0.21**	

STANDARD FORM SF-3
STORM DRAINAGE SYSTEM DESIGN
(RATIONAL METHOD PROCEDURE)

CALCULATED BY: TRO | **JOB NO:** 22A267
DATE: 6/12/24 | **PROJECT:** Element Eng - Ramah Lift Station
CHECKED BY: BMW | **DESIGN STORM:** 100 YEAR

			DIRECT RUNOFF						
BASIN	DESIGN POINT	AREA DESIG.	AREA (Acres)	RUNOFF COEFF	Tc (min)	C A (Acres)	I (in/hour)	Q (cfs)	REMARKS
EX-1	1	**EX-1**	16.68	0.50	18.61	8.38	6.08	**51.0**	
EX-2	2	**EX-2**	2.87	0.50	12.83	1.44	7.26	**10.5**	
EO-1		**EO-1**	1.31	0.52	12.78	0.68	7.26	**4.9**	
EO-2		**EO-2**	3.15	0.52	14.71	1.65	6.85	**11.3**	
EX-LS		**EX-LS**	0.14	0.65	10.00	0.09	8.04	**0.7**	
WWTP		**WWTP**	9.55	0.52	10.00	4.97	8.04	**40.0**	Detained on site and released by evaporation
A-1		**A-1**	2.35	0.51	13.89	1.21	7.02	**8.5**	
O-1		**O-1**	1.31	0.52	12.78	0.68	7.26	**4.9**	
A-1 + OS-1	1	**A-1 + OS-1**	3.66	0.51	10.00	1.88	8.04	**15.1**	Combinded proposed design Runoff (40.8 CFS) for Design point 1 is less than
A-2	1	**A-2**	2.95	0.51	10.00	1.51	8.04	**12.1**	
A-3		**A-3**	2.38	0.50	17.19	1.19	6.35	**7.6**	
O-2		**O-2**	3.15	0.52	14.71	1.65	6.85	**11.3**	
O-3		**O-3**	2.30	0.52	10.00	1.19	8.04	**9.6**	
A-3 + OS-2 + OS-3	2	**A-3 + OS-2 + OS-3**	8.40	0.50	17.19	4.22	6.35	**26.8**	Enters an ephemeral stream upon leaving the property
LS		**LS**	0.14	0.68	10.00	0.09	8.04	**0.7**	Equivelant runoff from EX-LS

Basin runoff						
Basin ID		C Values		Runoff (cfs)		Comments
		5-year	100-year	5-year	100-year	
Existing						
EX-1	1	0.16	0.50	7.17	51.0	
EO-1	1	0.19	0.52	0.81	4.9	
EO-2	1	0.20	0.52	1.86	11.3	
EX-2	2	0.21	0.53	1.92	10.5	
EX-LS	4	0.40	0.65	0.19	0.7	
EX Total		0.17	0.51	11.95	78.36	
Basin Runoff						
Basin ID	POINT	C Values	Runoff (cfs)			Comments
		5-year	100-year	5-year	100-year	
Proposed						
WWTP		0.19	0.52	6.50	40.0	Contained on site
A-1	1	0.18	0.51	1.29	8.5	
OS-1	1	0.19	0.52	0.81	4.9	
A-2	2	0.17	0.52	2.02	12.1	
OS-2	2	0.20	0.52	1.86	11.3	
OS-3	2	0.22	0.53	1.77	9.6	
A-3	3	0.15	0.50	1.03	7.6	28.2 CFS < 51 CFS
LS	4	0.44	0.68	0.21	0.7	
Detained Runoff					40.0	CFS
Site Runoff					28.2	CFS
Bypassed Runoff					25.8	CFS
Total Site Runoff					54.7	CFS
Design Point 1 Sum					13.4	CFS
Design Point 2 Sum					33.0	CFS

Design Procedure Form: Runoff Reduction

UD-BMP (Version 3.07, March 2018)

Designer:
Company:
Date: March 15, 2024
Project:
Location:

SITE INFORMATION (User Input in Blue Cells)

WQCV Rainfall Depth: 0.60 inches

Depth of Average Runoff Producing Storm, d_6 = 0.43 inches (for Watersheds Outside of the Denver Region, Figure 3-1 in USDCM Vol. 3)

Area Type	UIA:RPA	UIA:RPA	UIA:RPA								
Area ID	PR-A1	PR-A2	PR-LS								
Downstream Design Point ID	1	2	3								
Downstream BMP Type	None	None	None								
DCIA (ft^2)	--	--	--								
UIA (ft^2)	5,684	2,531	2,240								
RPA (ft^2)	8,000	8,000	1,200								
SPA (ft^2)	--	--	--								
HSG A (%)	0%	0%	0%								
HSG B (%)	0%	0%	0%								
HSG C/D (%)	100%	100.0%	100%								
Average Slope of RPA (ft/ft)	0.200	0.006	0.025								
UIA:RPA Interface Width (ft)	400.00	80.00	35.00								

CALCULATED RUNOFF RESULTS

Area ID	PR-A1	PR-A2	PR-LS								
UIA:RPA Area (ft^2)	13,684	10,531	3,440								
L / W Ratio	0.09	1.65	2.81								
UIA / Area	0.4154	0.2403	0.6512								
Runoff (in)	0.00	0.00	0.10								
Runoff (ft^3)	0	0	28								
Runoff Reduction (ft^3)	237	105	65								

CALCULATED WQCV RESULTS

Area ID	PR-A1	PR-A2	PR-LS								
WQCV (ft^3)	237	105	93								
WQCV Reduction (ft^3)	237	105	65								
WQCV Reduction (%)	100%	100%	70%								
Untreated WQCV (ft^3)	0	0	28								

CALCULATED DESIGN POINT RESULTS (sums results from all columns with the same Downstream Design Point ID)

Downstream Design Point ID	1	2	3								
DCIA (ft^2)	0	0	0								
UIA (ft^2)	5,684	2,531	2,240								
RPA (ft^2)	8,000	8,000	1,200								
SPA (ft^2)	0	0	0								
Total Area (ft^2)	13,684	10,531	3,440								
Total Impervious Area (ft^2)	5,684	2,531	2,240								
WQCV (ft^3)	237	105	93								
WQCV Reduction (ft^3)	237	105	65								
WQCV Reduction (%)	100%	100%	70%								
Untreated WQCV (ft^3)	0	0	28								

CALCULATED SITE RESULTS (sums results from all columns in worksheet)

Total Area (ft^2)	27,655
Total Impervious Area (ft^2)	10,455
WQCV (ft^3)	436
WQCV Reduction (ft^3)	407
WQCV Reduction (%)	94%
Untreated WQCV (ft^3)	28

APPENDIX B: HYDRAULIC CALCULATIONS

Town of Ramah
Evaporation Pond Calculations

Table of Contents

1. Precipitation and Evaporation Data

Month	Precipitation (in)	Evaporation Distribution	Evaporation Rate (in)
January	0.34	3.0%	1.41
February	0.36	3.5%	1.645
March	0.97	5.5%	2.585
April	1.675	9.0%	4.23
May	2.37	12.0%	5.64
June	2.085	14.5%	6.815
July	2.775	15.0%	7.05
August	2.805	13.5%	6.345
September	1.29	10.0%	4.7
October	0.88	7.0%	3.29
November	0.57	4.0%	1.88
December	0.385	3.0%	1.41
Total	16.505	100%	49

Precipitation Source: WRCC Station Hugo 1 NW
Evaporation Source: NOAA Tech Atlas 33 Gross Evaporation Map No. 3 – Annual Free Water Surface Evaporation and the State Engineer's Office recommended distribution of evaporation for elevations below 6,500 feet

25-Year 24-Hour Precipitation Event		
25-Year 24-Hour Precipitation Event	3.39	inches
Source: NOAA Atlas 14 Point Precipitation Frequency Estimates - Simla Station		

2. Evaporation Pond Design

Total Bottom Surface Area	310,000	ft^2	7.12	acres
Total Top of Berm Surface Area (Precip Area)	372,049	ft^2	8.54	acres
Total Required Land Area	448,154	ft^2	10.29	acres

Pond Bottom Surface Area per Pond	103,333	ft^2	2.37	acres
Pond Top Surface Area per Pond	124,016	ft^2	2.85	acres

Sludge depth	0.5	ft
Max Operating WL	3	ft
Freeboard	2	ft
Side Slope Ratio	3	
Pond Width to Length Ratio	2	
No. of Ponds	3	
Radius of Corner at Bottom	20	ft
Radius of Corner at Top	35	ft
SA Sub for Corner at Bottom	343	sf
SA Sub for Corner at Top	1052	sf
Width of Pond At Bottom	228	ft
Length of Pond at Bottom	455	ft
Width of Pond At Top	258	ft
Length of Pond at Top	485	ft
Width of Road	20	ft

3. Water Balance - Design Operating Conditions

Month	Days/Month	WW Inflow (gal/day)	WW Inflow (gal)	WW Inflow (ft3)	Avg. Precip (in)	Avg. Evap (in)	Avg. Precip (ft3)	Avg. Evap (ft3)	OUTFLOW Evaporation (ft3)	INFLOW WW + Precip (ft3)	Net Inflow (ft3)
January	31	15,000	465,000	62,161	0.34	1.41	10,541	40,070	40,070	72,703	32,632
February	28	15,000	420,000	56,146	0.36	1.65	11,161	46,749	46,749	67,307	20,559
March	31	15,000	465,000	62,161	0.97	2.59	30,074	73,462	73,462	92,235	18,773
April	30	15,000	450,000	60,156	1.675	4.23	51,932	120,211	120,211	112,088	-8,123
May	31	15,000	465,000	62,161	2.37	5.64	73,480	160,282	160,282	135,641	-24,640
June	30	15,000	450,000	60,156	2.085	6.82	64,644	193,674	193,674	124,800	-68,874
July	31	15,000	465,000	62,161	2.775	7.05	86,036	200,352	200,352	148,198	-52,154
August	31	15,000	465,000	62,161	2.805	6.35	86,966	180,317	180,317	149,128	-31,189
September	30	15,000	450,000	60,156	1.29	4.70	39,995	133,568	133,568	100,152	-33,416
October	31	15,000	465,000	62,161	0.88	3.29	27,284	93,498	93,498	89,445	-4,052
November	30	15,000	450,000	60,156	0.57	1.88	17,672	53,427	53,427	77,829	24,401
December	31	15,000	465,000	62,161	0.385	1.41	11,937	40,070	40,070	74,098	34,028
Total	**365**	**180,000**	**5,475,000**	**731,901**	**16.505**	**47.00**	**511,723**	**1,335,680**	**1,335,680**	**1,243,624**	**-92,056**
					30.50 Deficit						

Notes:
1. Calculations assume no seepage
2. Precipitation volume calculating using top of berm surface area
3. Evaporation volume calculated using average of top and bottom pond surface areas

Month	Year 1	Year 2	Year 3	Year 4	Year 5	Max Pond Stage (ft)	Yr 5 Pond Stage (ft)
July	0	0	0	0	0	0.00	0.00
August	0	0	0	0	0	0.00	0.00
September	0	0	0	0	0	0.00	0.00
October	0	0	0	0	0	0.00	0.00
November	24,401	24,401	24,401	24,401	24,401	0.08	0.08
December	58,429	58,429	58,429	58,429	58,429	0.19	0.19
January	91,062	91,062	91,062	91,062	91,062	0.29	0.29
February	111,620	111,620	111,620	111,620	111,620	0.36	0.36
March	130,393	130,393	130,393	130,393	130,393	0.42	0.42
April	122,270	122,270	122,270	122,270	122,270	0.39	0.39
May	97,630	97,630	97,630	97,630	97,630	0.31	0.31
June	28,756	28,756	28,756	28,756	28,756	0.09	0.09

Notes:
1. Values listed = volume in pond at corresponding year and month in ft^3
2. Values calculated by adding Net Inflow for corresponding month to previous month. If net volume < 0, 0 is listed
3. Max Pond Stage = max volume in pond during corresponding month over five years divided by total pond bottom surface area
4. Yr 5 Pond Stage = volume in pond at year 5 during corresponding month divided by total pond bottom surface area

4. Water Balance - 25-Year 24-Hour Precipitation Event Conditions

Month	Year 1	Year 2	Year 3	Year 4	Year 5	Year 6	Year 7	Year 8	Year 9	Year 10	Max Pond Stage (ft)
July	0	0	81,706	0	0	0	0	0	0	0	0.26
August	0	0	50,517	0	0	0	0	0	0	0	0.16
September	0	0	17,101	0	0	0	0	0	0	0	0.06
October	0	0	13,048	0	0	0	0	0	0	0	0.04
November	24,401	24,401	37,450	24,401	24,401	24,401	24,401	24,401	24,401	24,401	0.12
December	58,429	58,429	71,477	58,429	58,429	58,429	58,429	58,429	58,429	58,429	0.23
January	91,062	91,062	104,110	91,062	91,062	91,062	91,062	91,062	91,062	91,062	0.34
February	111,620	111,620	124,668	111,620	111,620	111,620	111,620	111,620	111,620	111,620	0.40
March	130,393	235,497	143,441	130,393	130,393	130,393	130,393	130,393	130,393	130,393	0.76
April	122,270	227,374	135,318	122,270	122,270	122,270	122,270	122,270	122,270	122,270	0.73
May	97,630	202,734	110,678	97,630	97,630	97,630	97,630	97,630	97,630	97,630	0.65
June	28,756	133,860	41,804	28,756	28,756	28,756	28,756	28,756	28,756	28,756	0.43
Month	**Year 11**	**Year 12**	**Year 13**	**Year 14**	**Year 15**	**Year 16**	**Year 17**	**Year 18**	**Year 19**	**Year 20**	**Max Pond Stage (ft)**
July	0	0	0	0	0	0	0	0	0	0	0.00
August	0	0	0	0	0	0	0	0	0	0	0.00
September	0	0	0	0	0	0	0	0	0	0	0.00
October	0	0	0	0	0	0	0	0	0	0	0.00
November	24,401	24,401	24,401	24,401	24,401	24,401	24,401	24,401	24,401	24,401	0.08
December	58,429	58,429	58,429	58,429	58,429	58,429	58,429	58,429	58,429	58,429	0.19
January	91,062	91,062	91,062	91,062	91,062	91,062	91,062	91,062	91,062	91,062	0.29
February	111,620	111,620	111,620	111,620	111,620	111,620	111,620	111,620	111,620	111,620	0.36
March	130,393	130,393	130,393	130,393	130,393	130,393	130,393	130,393	130,393	130,393	0.42
April	122,270	122,270	122,270	122,270	122,270	122,270	122,270	122,270	122,270	122,270	0.39
May	97,630	97,630	97,630	97,630	97,630	97,630	97,630	97,630	97,630	97,630	0.31
June	28,756	28,756	28,756	28,756	28,756	28,756	28,756	28,756	28,756	28,756	0.09

Notes:
1. Values listed = volume in pond at corresponding year and month in ft^3
2. Values calculated by adding Net Inflow for corresponding month to previous month. If net volume < 0, 0 is listed
3. Max Pond Stage = max volume in pond during corresponding month over five years divided by total pond bottom surface area
4. Yr 5 Pond Stage = volume in pond at year 5 during corresponding month divided by total pond bottom surface area
5. Yellow month and year indicates month that 25-year 24-hour precipitation event is added

Flow & Loading

The system has been designed for an influent flow rate of 15,000 gpd and 32 lbs of BOD/day.

Precipitation

Precipitation data was obtained from Western Regional Climate Center, Station Rush 1N and Eastonville 2NNW. The data set for Ramah was determined by calculating the average of the Rush and Eastonville stations. The data from Rush and Eastonville is presented in Table 1. The calculated data for Ramah is presented in Table 2.

Table 1 - WRCC Precipitation Data

Month	Precipitation (in)	Month	Precipitation (in)
January	0.44	January	0.24
February	0.47	February	0.25
March	1.29	March	0.65
April	2.09	April	1.26
May	2.56	May	2.18
June	2.3	June	1.87
July	3.18	July	2.37
August	3.07	August	2.54
September	1.47	September	1.11
October	1.12	October	0.64
November	0.79	November	0.35
December	0.52	December	0.25
Total	**19.30**	**Total**	**13.71**
Source: WRCC Station Eastonville 2 NNW		Source: WRCC Station Rush 1N	

Table 2 - Ramah Precipitation Data

Month	Precipitation (in)
January	0.34
February	0.36
March	0.97
April	1.68
May	2.37
June	2.09
July	2.78
August	2.81
September	1.29
October	0.88
November	0.57
December	0.39
Total	**16.51**
Source: Average of WRCC Station Rush 1N & Eastonville 2 NNW	

Evaporation

Evaporation data was taken from NOAA Tech Atlas 33 Gross Evaporation map No.3 – Annual Free Water Surface Evaporation and the State Engineer's Office recommended distribution of evaporation for elevations below 6,500 feet. Monthly evaporation rates are presented in Table 3. The relevant information from these sources is attached to this memo.

Table 3 - Monthly Evaporation Rates

Month	Evaporation Distribution	Evaporation Rate (in)
January	3%	1.41
February	4%	1.645
March	6%	2.585
April	9%	4.23
May	12%	5.64
June	15%	6.815
July	15%	7.05
August	14%	6.345
September	10%	4.7
October	7%	3.29
November	4%	1.88
December	3%	1.41
Total	**100%**	**47**
Source: NOAA Tech Atlas 33 Gross Evaporation Map No. 3 – Annual Free Water Surface Evaporation and the State Engineer's Office recommended distribution of evaporation for elevations below 6,500 feet		

Evaporation Calculations

The above information was utilized to perform an iterative process to determine the appropriate sizing for an evaporative system. The size of the ponds was increased until the ponds will completely evaporate once per year. The total land required will be approximately 10.3 acres. See the evaporative pond system calculation table attached at the end of this memo.

BOD Loading

Colorado Department of Public Health and Environment (CDPHE) Design Criteria for Domestic Wastewater Treatment Works requires that "the total BOD loading shall not exceed 0.5 pounds per 1,000 square feet of water surface per day based on the total water surface in the stabilization pond". The total minimum operating water level of the ponds is 310,000 SF.

$$\left(32\ lbs\frac{BOD}{day}\right) \div \left(310{,}000\frac{SF}{1{,}000}\right) = 0.10 lbs/1{,}000\ SF$$

The total BOD loading is calculated to be 0.10 lbs BOD/1,000 SF at the proposed permitted influent organic loading. This meets CDPHE criteria. The BOD loading breakdown is displayed in table 4.

Table 4 - Evaporation Ponds BOD Loading

Condition	Loading	Units	Design Criteria
Permitted Organic Loading	32	lbs BOD/day	
Total Bottom of Pond Area (Three Ponds)	310,000	SF	
Total Bottom of Pond Area with One Pond Out of Service	206,667	SF	
BOD Loading with Three Ponds Online	0.103	lbs BOD/day/1,000 SF	0.5
BOD Loading with One Pond Out of Service	0.155	lbs BOD/day/1,000 SF	1

Design Values

Angular D_{50} dia. = 9.4 in.
$Rock_{chute}$ thickness = 18.8 in.
Inlet apron length = 9 ft.
Outlet apron length = 12 ft.
Radius = 26 ft.
Will bedding be used? Yes

Rock Gradation Envelope

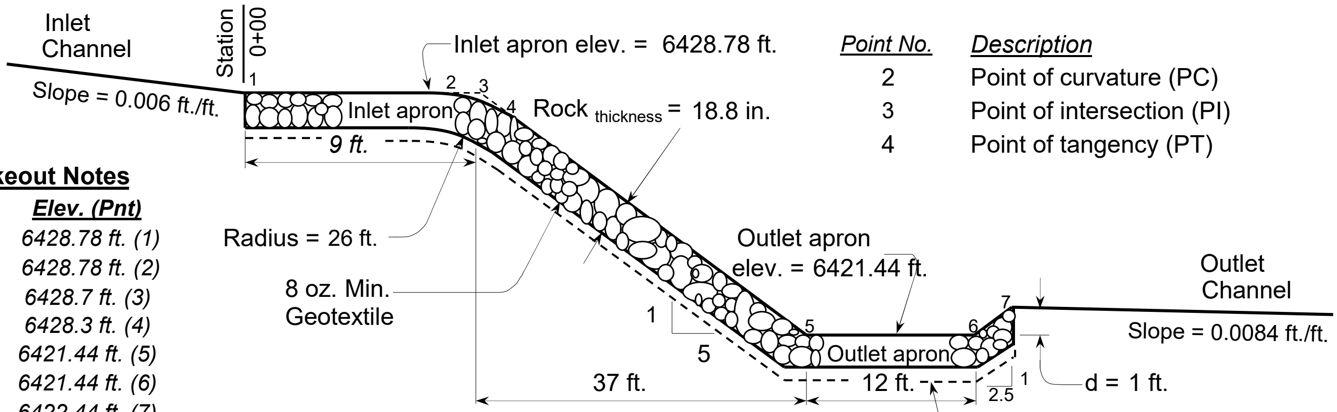
% Passing		*Diameter, in. (weight, lbs.)*
D_{100}	---------	14 - 19 (204 - 484)
D_{85}	---------	12 - 17 (133 - 353)
D_{50}	---------	9 - 14 (61 - 204)
D_{10}	---------	8 - 12 (31 - 133)

Coefficient of Uniformity, $(D_{60})/(D_{10}) \leq 2.0$

Quantities [a]

Angular Rock = 132 yd^3
Geotextile (8 oz.)[b] = 338 yd^2
Bedding (6 in.) = 61 yd^3
Excavation = 700 yd^3
Earthfill = 500 yd^3
Seeding = 1.0 acres

Notes: [a] *Rock, bedding, and geotextile quantities are determined from x-section below (neglect radius).*
[b] *Geotextile shall be overlapped (18-in. minimum) and anchored (18-in. minimum along sides and 24-in. minimum on the ends) --- <u>quantity not included</u>.*

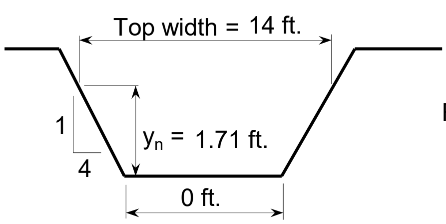


Point No.	*Description*
2	Point of curvature (PC)
3	Point of intersection (PI)
4	Point of tangency (PT)

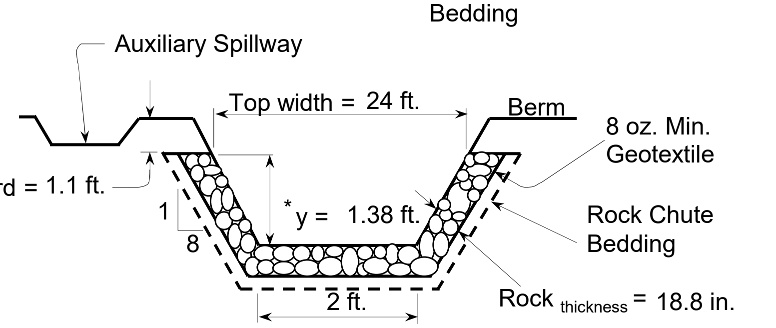
Stakeout Notes

Sta.	***Elev. (Pnt)***
0+00	*6428.78 ft. (1)*
0+6.4	*6428.78 ft. (2)*
0+9	*6428.7 ft. (3)*
0+11.5	*6428.3 ft. (4)*
0+46	*6421.44 ft. (5)*
0+58	*6421.44 ft. (6)*
0+60.5	*6422.44 ft. (7)*

<u>Profile Along Centerline of Rock Chute</u>

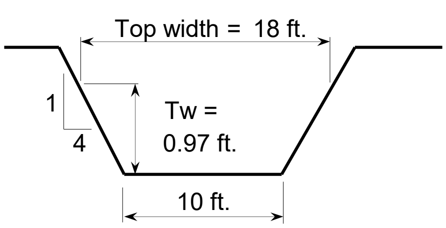


<u>Inlet Channel Cross Section</u>



<u>Rock Chute Cross Section</u>

* Use H_p throughout chute but not less than z_2.



<u>Outlet Channel Cross Section</u>

Profile, Cross Sections, and Quantities

Project: Town of Ramah WWTP Drainage Plan
Location: El Paso County

U.S. Department of Agriculture
Natural Resources Conservation Service

Designed: Timothy Osborn	Approved by: ________________		
Drawn: NRCS Standard Dwg.	Title: ________________		
	Title: ________________		
Traced: ________________	Sheet No.		Drawing No.
Checked: ________________	of		

EVstudio

PROJECT: Element Eng - Ramah Lift Station
SUBJECT: Imperviousness and Runoff C Calculations

JOB #: 22A267
DATE: 1/26/2024
BY: TRO

	Bottom width	Side slope 1	Side Slope 2	Channel Depth	Mannings	channel slope	Q (cfs)	Flow Area	Velocity	Froude	Requaired Freeboard	Requiared Depth
start	0.00	0.25	0.25	0.74	0.03	0.006	5.13	2.20	2.33	0.68	1.05	1.79
SW corner	0.00	0.25	0.25	0.98	0.03	0.006	10.69	3.81	2.80	0.71	1.07	2.05
SE corner	0.00	0.25	0.25	1.29	0.03	0.006	22.28	6.61	3.37	0.74	1.09	2.38
Run Down	2.0	0.13	0.13	1.0	0.0	0.0	33.9	10.7	3.2	0.74	1.08	2.12

Design Procedure Form: Runoff Reduction

UD-BMP (Version 3.07, March 2018)

Designer:
Company:
Date: March 8, 2024
Project:
Location:

SITE INFORMATION (User Input in Blue Cells)

WQCV Rainfall Depth: 0.60 inches

Depth of Average Runoff Producing Storm, d_6 = 0.43 inches (for Watersheds Outside of the Denver Region, Figure 3-1 in USDCM Vol. 3)

Area Type	UIA:RPA	UIA:RPA	UIA:RPA								
Area ID	PR-A1	PR-A2	PR-LS								
Downstream Design Point ID	1	2	3								
Downstream BMP Type	None	None	None								
DCIA (ft^2)	--	--	--								
UIA (ft^2)	5,684	2,531	2,240								
RPA (ft^2)	8,000	8,000	1,200								
SPA (ft^2)	--	--	--								
HSG A (%)	0%	0%	0%								
HSG B (%)	0%	0%	0%								
HSG C/D (%)	100%	100.0%	100%								
Average Slope of RPA (ft/ft)	0.200	0.006	0.025								
UIA:RPA Interface Width (ft)	400.00	80.00	35.00								

CALCULATED RUNOFF RESULTS

Area ID	PR-A1	PR-A2	PR-LS								
UIA:RPA Area (ft^2)	13,684	10,531	3,440								
L / W Ratio	0.09	1.65	2.81								
UIA / Area	0.4154	0.2403	0.6512								
Runoff (in)	0.00	0.00	0.10								
Runoff (ft^3)	0	0	28								
Runoff Reduction (ft^3)	237	105	65								

CALCULATED WQCV RESULTS

Area ID	PR-A1	PR-A2	PR-LS								
WQCV (ft^3)	237	105	93								
WQCV Reduction (ft^3)	237	105	65								
WQCV Reduction (%)	100%	100%	70%								
Untreated WQCV (ft^3)	0	0	28								

CALCULATED DESIGN POINT RESULTS (sums results from all columns with the same Downstream Design Point ID)

Downstream Design Point ID	1	2	3								
DCIA (ft^2)	0	0	0								
UIA (ft^2)	5,684	2,531	2,240								
RPA (ft^2)	8,000	8,000	1,200								
SPA (ft^2)	0	0	0								
Total Area (ft^2)	13,684	10,531	3,440								
Total Impervious Area (ft^2)	5,684	2,531	2,240								
WQCV (ft^3)	237	105	93								
WQCV Reduction (ft^3)	237	105	65								
WQCV Reduction (%)	100%	100%	70%								
Untreated WQCV (ft^3)	0	0	28								

CALCULATED SITE RESULTS (sums results from all columns in worksheet)

Total Area (ft^2)	27,655
Total Impervious Area (ft^2)	10,455
WQCV (ft^3)	436
WQCV Reduction (ft^3)	407
WQCV Reduction (%)	94%
Untreated WQCV (ft^3)	28

APPENDIX C: REFERENCES

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Geotechnical and Materials Engineers
and Environmental Scientists

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GEOTECHNICAL ENGINEERING STUDY
RAMAH WASTEWATER TREATMENT PLANT
IMPROVEMENT PROJECT
RAMAH, COLORADO

Prepared By:
Arben F. Kalaveshi, P.E.

COLORADO REGISTERED PROFESSIONAL ENGINEER
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04-22-2022

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Lakewood, Colorado 80228

Project No. 22-2-102

April 22, 2022

TABLE OF CONTENTS

SUMMARY

1. This study was conducted in two areas. Borings 1 and 2, drilled southeast of The Town of Ramah encountered a layer of topsoil overlying sands and clays extending to the maximum drilled depth of 20 feet. Boring 3, drilled on the northwest end of town encountered topsoil overlying man placed fill extending to a depth of about 6 feet. Sands were found below the fill, and extended to the maximum drilled depth of 30 feet, but included a layer of sandy lean clay from about 27 to 29 feet.

2. Groundwater was encountered in Boring 3 both during drilling and when measured again six days later. The water depth at the time of our final reading was 15.3 feet below the ground surface. Although no groundwater was measured in the other two borings, perched surface water may occur within the sands above less permeable clays, particularly after precipitation events.

3. Borings 1 and 2 were drilled for new evaporative ponds to replace the existing system. The subsurface soil profile at this location included a clay zone from about 2½ feet to 9½ feet below the existing ground surface. The clays tested had a moderate to high swell potential upon wetting and are anticipated to have a relatively low permeability. While these soils will probably work well for use as evaporative ponds, the construction of shallow foundations here will be difficult due to the swell potential. If movement sensitive structures are constructed in this area, we recommend that they be constructed on deep foundations such as helical piers that extend to the underlying granular soils found below the clays.

4. Boring 3 was drilled for the construction of a new lift station. Because undocumented fill was encountered in this area, we recommend that it be removed and replaced with suitable materials where it is present below proposed shallow foundations. Alternatively, deep foundations may be considered. Based on the subsurface profile encountered at this location, foundations will need to extend to a depth of about 6 feet or greater to bear on native soils, but the depth and lateral extent of the existing fill was not determined beyond the boring location. Fill may extend to greater depths in the area of the proposed foundations.

5. We anticipate that gravel access drives may be constructed at each of the proposed sites. Based on their intended use, we have assumed an EDLA of 10 for these areas. Based on the subsurface conditions encountered and the relatively light estimated traffic volumes, we recommend the pavement section alternatives presented in the following table.

Pavement Section Thickness (in.)	
Area	Aggregate Base Course
Access Drives	8

PURPOSE AND SCOPE OF STUDY

This report presents the results of a geotechnical engineering study for the construction of various improvements to the existing wastewater treatment system in Ramah, Colorado. The project site is shown on Fig. 1. The study was conducted in accordance with our Proposal No. C22-104, dated January 10, 2022, to provide recommendations for foundations and gravel pavement section thickness.

This report has been prepared to summarize the data obtained during this study, and to present our conclusions and recommendations based on the proposed construction and the subsurface conditions encountered. Design parameters and a discussion of geotechnical engineering considerations related to the proposed construction are included in the report.

PROPOSED CONSTRUCTION

We understand the project will include a new lift station for a force main that will connect to a new evaporative pond system located about half a mile away. The new ponds will include three adjacent cells, and will replace the existing one located on the north end of town, which will be decommissioned. The lift station is anticipated to have a depth of about 12 feet, and the ponds will have sloped basins ranging in depth from about 2 to 8 feet. The approximate pond site layout is shown on the attached Fig. 1A.

We anticipate that bearing loads will be light for the proposed structures. Permanent grading will mostly consist of cuts, with depths up to about 8 feet for the proposed ponds. If the proposed construction is significantly different from that described above or depicted in this report, we should be notified to reevaluate the recommendations contained in this report.

SITE CONDITIONS

The proposed evaporation pond site is located southeast of the Town of Ramah Cemetery, and is bordered by Ramah Road on the south and west sides, and open fields on the north and east. The area is relatively flat, with some small hills and draws, with a gentle slope down to the northeast. An ephemeral tributary to Big Sandy Creek is located about 1,000 feet east of the site, and was dry at the time of our study. This area appeared to be actively used as an agricultural field, and the vegetation had been tilled.

The site of the proposed lift station is located on the northwest part of town, just west of the intersection of Rock Island Avenue and Pikes Peak Avenue. The site is bordered to the north and east by Pikes Peak avenue, and to the south and west by private property. Houses and other small structures are located near the site. This area is relatively flat with a light downward slope to the north. Big Sandy Creek is located about 500 feet to the northwest, and the existing evaporation pond (lagoon) is located about 1,000 feet to the northeast. Vegetation in this area consisted of a grass lawn and several trees.

SUBSURFACE CONDITIONS

Information on subsurface conditions was obtained by conducting a site reconnaissance and drilling three exploratory borings at the approximate locations shown on Fig. 1. The boring logs and corresponding legend and notes are shown on Figs. 2 and 3. The results of swell-consolidation tests and gradation tests conducted on selected soils are presented on Figs. 4 through 6, and Figs. 7 and 8, respectively. A summary of the test results is presented on Table I. The laboratory testing was conducted in general accordance with applicable ASTM standards.

Borings 1 and 2 were drilled at the site of the proposed evaporation ponds. Below a layer of topsoil the subsurface soil profile at this location consisted of clayey sands extending to a depth of about 2½ feet underlain by a layer of lean clay with varied amounts of sand, followed by discontinuous layers of clayey sand and well graded sand with silt extending to the maximum depth explored of 20 feet. Based on vertical expansion ranging from about 3.4 to 5.7 percent upon wetting under a surcharge pressure of 1,000 psf, the clays in this area possess a moderate to high swell potential.

Boring 3 was drilled at the site of the proposed lift station. Below a layer of vegetated topsoil, the subsurface soil profile at this location consisted of man placed fill extending to a depth of about 6 feet, and underlain by clayey sand extending to a depth of 9½ feet. Well graded sand was found below the clayey sand, and extended to a depth of about 27 feet, where it was underlain by a layer of sandy lean clay. The clay layer was relatively thin, and was underlain by clayey sand from 29 feet to the maximum explored depth of 30 feet. The fill tested did not appear to possess a significant swell potential based on a vertical expansion of 0.2 percent upon wetting under a surcharge pressure of 1,000 psf.

Detailed descriptions of the soils and the depths at which they were encountered can be found on Figs. 2 and 3.

ENGINEERING CONSIDERATIONS

Existing fill was encountered to a depth of about 6 feet at the location of the proposed lift station. The lateral or vertical extents of the fill were not determined in the scope of this study, but we understand that the base of this structure will be about 12 feet below the ground surface, and if this is the case for all foundations, the existing fill is not likely to be a factor for the design of shallow foundations. If portions of the structure or ancillary structures will be constructed at shallower depths, fill may be present below the base of shallow foundations. In all cases, fill should be removed and replaced with suitable material where it is present below foundations. Alternatively, foundations extending to native soils or deep foundations may be considered. Recommendations for both footing/pad foundations and deep helical foundations have been presented in this report.

Groundwater was measured at a depth of about 15.3 feet in the boring drilled for the proposed lift station measured six days after drilling. This depth is near the elevation of the base of the lift station, and groundwater may be a construction consideration at this site. A detailed discussion is presented in the "Site Grading and Earthwork" Section.

The subsurface soil profile at the location of the proposed evaporation ponds included a clay zone from about 2½ feet to 9½ feet below the existing ground surface. The clays tested had a moderate to high swell potential upon wetting and are anticipated to have a relatively low permeability. While these soils will probably work well for use as evaporative ponds, the construction of shallow foundations here will be difficult due to the swell potential. If structures that are sensitive to heave related movement are constructed in this area, we recommend that they be constructed on deep foundations such as helical piers that extend to the underlying granular soils found below the clays.

The clay soils encountered in our study will have relatively low permeability, but are natural materials and will vary throughout the site area. An engineered liner system should be implemented at the basin of each pond if specific permeability limits are required for this project.

FOUNDATIONS

Shallow Foundations: The design and construction criteria presented below should be observed for a shallow footing system. The construction details should be considered when preparing project documents.

1. The maximum net allowable bearing pressure for footings placed on native granular soils or suitable fill will be a function of the embedment depth of the foundation considered. Allowable pressures for the anticipated foundation depths have been presented in the following table. These values may be increased by a factor of 1/3 for transient loading.

Foundation Bury Depth (feet)	Allowable Bearing Pressure (psf)
3	2,500
12	4,500

   Mat foundations that are not considered rigid may use a design modulus of vertical subgrade reaction of 150 pci. This value is for a 1 ft. x 1 ft. square plate and should be corrected for the shape and size of the actual mat.

2. We estimate total settlement for shallow foundations designed and constructed as discussed in this section will not exceed approximately 1 inch.

3. Continuous footings should have a minimum width of 16 inches, and isolated pads should have a minimum width of 24 inches.

4. Exterior footings and footings beneath unheated areas should be provided with adequate soil cover above their bearing elevation for frost protection. Based on our experience with similar projects, we recommend the foundations be placed at least 36 inches below the existing grade.

5. The lateral resistance of a foundation placed on properly compacted fill material or bedrock will be a combination of the sliding resistance of the footing on the foundation materials and passive earth pressure against the side of the footing. Resistance to sliding at the bottoms of the footings may be calculated based on an allowable coefficient of friction of 0.35. Passive pressure against the sides of the footings may be calculated

using an allowable equivalent fluid unit weight of 190 pcf. These values are working values. The specifications for compaction of fill against the sides of foundations to resist lateral loads are presented under the “Site Grading and Earthwork” section of this report.

6. Earthwork recommendations for shallow foundations are presented in the “Site Grading and Earthwork” section of this report.

7. Existing fill, or areas of loose material encountered within the foundation excavation should be removed and the footings extended to adequate natural bearing material.

8. A representative of the geotechnical engineer should observe all footing excavations prior to fill and concrete placement.

<u>Helical Pier Foundations</u>: The axial design load of helical piers should be determined in general accordance with the current International Building Code (IBC), which states the allowable axial design load, P_a, should be determined as follows:

P_a= 0.5 P_u, where P_u (the ultimate load) is the least value of:

1. Sum of the areas of the helical bearing plates times the ultimate bearing capacity of the soil or rock comprising the bearing stratum.

2. Ultimate capacity determined from well-documented correlations with installation torque.

3. Ultimate capacity determined from load tests.

4. Ultimate capacity of pile shaft.

5. Ultimate capacity of pile couplings.

6. Sum of the Ultimate axial capacity of helical bearing plates affixed to pile.

Items 1 through 3 are related to the geotechnical capacity of the piers; Items 4 through 6 are related to the structural capacity and should be evaluated by the structural engineer. The owner and structural designer should be aware that certain proprietary helical pier systems have been subjected to acceptance testing administered by the International Code Council (ICC), while other systems provided by specialty contractors may be fabricated according to designs by registered professional engineers. The certified systems have documentation that addresses many of the structural capacity issues, while the non-certified systems require structural design by an engineer. Many of the lighter-duty helical pile systems available, with working capacities on the order of 50 kips or less, are certified, which can simplify the design and submittal process. However, higher capacity systems, where single piers may have working capacities of 200 kips or more, sometimes referred to as screw piles, are often designed and fabricated and are not certified, manufactured systems.

Based on consideration of bearing capacity theory and published correlations of boring penetration resistance values with ultimate bearing capacity, we recommend an ultimate bearing capacity of 10 ksf for a helical pile embedded in the native sands. We anticipate it will be possible to achieve adequate capacities at nominal depths of about 15 feet by using the appropriate size and number of bearing plates. Nominal depths should be measured from the topmost bearing plate. A greater bearing capacity will be achievable if the piers extend to the underlying claystone bedrock.

Helical piers are typically very slender foundation elements with a low capacity for resisting lateral loads. Lateral restraint of a helical pile foundation system is normally provided through the use of passive pressure on pile caps or foundation walls, or through the use of battered piers. It is normally assumed that a battered pile can be designed for the same axial load as a vertical pile, with the lateral restraint being provided by the horizontal component of the battered pile. Helical piers are often assumed to have tension capacities similar to the axial compressive capacity, although that should be evaluated through load testing or otherwise addressed by the specialty contractor’s submittal.

Acceptance of helical pile installation should be based on attaining a specified torque in the recommended bearing stratum determined in accordance with correlations of installation torque to capacity based on calibrated torque measurements and axial load test data. In our opinion, the ultimate bearing capacity recommended above may be exceeded if supported by adequate site-specific load test data. If site-specific load tests are not performed, the specialty helical pile contractor’s submittal should contain torque-to-capacity data for their pile system in similar soil conditions. If that information cannot be provided, site-specific load tests should be performed in accordance with ASTM D 1143.

We recommend that a qualified helical pile specialty contractor be retained to provide the required design submittal and to provide and install the helical piers. The project design should include a performance specification indicating required capacities, structural requirements, and submittal requirements. At a minimum, the submittal should be required to contain information supporting capacity determination, a description of equipment and installation procedures that will ensure penetration to the required depths, and acknowledgement that the helical bearing plates will be installed into the recommended bearing stratum, as well as all necessary information to satisfy the requirements of the project structural designer.

We should be retained to review the contractor's submittal, and to provide installation observation including monitoring depths and general conformance with the plans and specifications. Our observation and testing services will be intended to document that all of the helix bearing plates on the piers are installed into an adequate bearing stratum.

RETAINING STRUCTURES

Structures such as retaining or foundation walls should be designed for the lateral pressure generated by the backfill, which is a function of the degree of rigidity of the retaining structure and the type of backfill material used. Cantilevered retaining structures that can deflect sufficiently to mobilize the active earth pressure condition maybe designed using the active equivalent fluid pressure (EFP) presented in the following table. Retaining structures that are not expected to deflect should be designed using the at-rest EFP presented in the same table.

Condition	Soil Type	Equivalent Fluid Pressure (pcf)	
		Active	At-rest
Unsubmerged	Suitable On-Site Soil	50	70
Unsubmerged	CDOT Class 1 Structure Backfill	40	60
Submerged	Suitable On-Site Soil	88	99
Submerged	CDOT Class 1 Structure Backfill	83	94

All foundation and retaining structures should be designed for appropriate hydrostatic and surcharge pressures such as adjacent footings, traffic, construction materials and equipment. The unsubmerged pressures recommended above assume drained conditions behind the walls and a horizontal backfill surface. The buildup of water behind a wall or an upward sloping backfill surface will increase the lateral pressure imposed on a foundation wall or retaining structure. Retaining structures may be designed using the values presented for unsubmerged soils if adequate drainage is provided to prevent the buildup of hydrostatic pressure. This can be accomplished using an underdrain or weep holes. If such measures are not implemented, the structures should be designed using the submerged values presented.

WATER SOLUBLE SULFATES

The concentrations of water soluble sulfates measured in samples of the native clay and fill obtained from the exploratory borings ranged from 0.01 to 0.06 percent. These concentrations of water soluble sulfates represent a Class 0 severity of exposure to sulfate attack on concrete exposed to these materials. The degree of attack is based on a range of Class 0 to Class 3 severity of exposure as presented in ACI 201. Based on the laboratory data and our experience, special sulfate resistant cement will not be required for concrete exposed to the on-site soils.

SEISMIC DESIGN CRITERIA

Using estimated shear wave velocities for the subgrade materials encountered based on standard penetration testing, calculations indicate a design Site Class D per the International Building Code (IBC). Based on the subsurface profile and the anticipated ground conditions, liquefaction is not a design consideration.

SURFACE DRAINAGE

Providing proper surface drainage, both during construction and after the construction has been completed, is very important for acceptable performance of the development. The following recommendations should be used as guidelines and changes should be made only after consultation with the geotechnical engineer.

1. Excessive wetting or drying of the foundation and structure subgrades should be avoided during construction.

2. Care should be taken when compacting around the foundation walls to avoid damage to the structure.

3. The ground surface surrounding the exterior of the building should be sloped to drain away from the foundation in all directions. We recommend a minimum slope of 6 inches in the first 10 feet in unpaved areas. Site drainage beyond the 10-foot zone should be

designed to promote runoff and reduce water infiltration. A minimum slope of 3 inches in the first 10 feet is recommended in the paved areas. These slopes may be changed as required for handicap access points in accordance with the Americans with Disabilities Act.

4. Ponding of water should not be allowed on backfill material or within 10 feet of the foundation walls, whichever is greater.

5. Roof downspouts and drains should discharge well beyond the limits of all backfill.

SITE GRADING AND EARTHWORK

We recommend the following criteria be used when preparing the site grading plans.

Fill Material Specifications: The following material specifications are presented for fills on the project site.

1. *Fill Beneath and Beside Foundations*: The on-site granular soils with the exception of any deleterious materials and rock larger than 4 inches in diameter will be suitable for reuse as structural fill. Import fill, if required, should consist of a minus 2-inch non-expansive soil having a maximum 35% passing the No. 200 sieve and a maximum plasticity index of 15. New fill should extend down from the edges of the foundations at a minimum 1:1 horizontal to vertical projection.

2. Gravel Pavement Subgrade Areas: Same as #1 above.

3. *Pipe Bedding Material*: Pipe bedding material should be a free draining, coarse-grained sand and/or fine gravel having a maximum size of 1 inch. We do not anticipate that the near surface on-site natural soils will be suitable for bedding due to the presence of larger particles.

4. *Utility Trench Backfill*: Materials excavated from the utility trenches may be used for trench backfill above the pipe zone fill provided they do not contain unsuitable material or particles larger than 4 inches.

5. *Material Suitability*: All fill material should be free of vegetation, brush, sod and other deleterious substances. The geotechnical engineer should evaluate the suitability of all proposed fill materials prior to placement.

Subgrade Preparation: The ground surface shall be stripped of vegetation/organics prior to foundation or fill placement. Loose, unstable or otherwise unsuitable soils shall be removed, where present, in order to provide a stable platform prior to placement of fill. The existing soils should be scarified, moisture conditioned, and recompacted to a depth of 12 inches prior to the placement of newl fill or structures

Compaction Requirements: A representative of the geotechnical engineer should observe fill placement operations on a full-time basis. We recommend the following minimum compaction criteria be used on the project.

Area	Percentage of Proctor Maximum Dry Density	
	Standard Proctor (ASTM D698)	Modified Proctor (ASTM D1557)
Fill beneath foundations	98%	--
Foundation wall backfill	95%	--
Slab Subgrade	95%	--
Beneath pavement Areas/ Flatwork/Utility Trenches	95%	--
Aggregate Pavements	--	95%
Landscape and Other Misc. Overlot Fill Areas	95%	--
For compaction of suitable granular soils, a moisture content within 2 percent of optimum should be maintained. For the compaction of cohesive soils, a moisture content within 0 to 4 percent above optimum should be maintained. A moisture content sufficient to achieve adequate compaction may be used for materials with few fines, such as the aggregate base course used for aggregate pavements.		

PAVEMENT DESIGN

A pavement section is a layered system designed to distribute concentrated traffic loads to the subgrade. Performance of the pavement structure is directly related to the physical properties of the subgrade soils, pavement section, and traffic loadings. We anticipate that gravel surfaced pavements will be used for access drives at both these sites.

Subgrade Materials: Based on the American Association of State Highway Transportation Officials (AASHTO) classification system the soils tested near the proposed subgrade elevation consisted of A-6 soils with a group index of 8. These soils are rated as poor for use as subgrade material.

Design Traffic: We have assumed that after construction, the roads will only receive occasional truck traffic. For our pavement thickness design calculations, we assumed an equivalent 18-kip daily load application (EDLA) of 10 . If it is determined that actual traffic is significantly different from that estimated, we should be contacted to reevaluate the pavement thickness design.

Pavement Sections: The pavement section presented in the following table is recommended for gravel drives constructed for this project.

Pavement Section Thickness (in.)	
Area	Aggregate Base Course
Access Drives	8

Subgrade Preparation: For general subgrade preparation, we recommend the pavement subgrade be thoroughly scarified and well-mixed to a minimum depth of 12 inches, moisture conditioned, and compacted to the specifications presented in the “Site Grading and Earthwork” Section.

Proof Roll: Before paving, the subgrade should be proof rolled with a heavily loaded, pneumatic-tired vehicle. The vehicle should have a gross weight of at least 50,000 pounds, with a single loaded axle weight of 18,000 pounds, and a tire pressure of 100 psi. Areas that deform excessively under heavy wheel loads are not stable and should be removed and replaced with suitable material to achieve a stable subgrade prior to paving.

Subgrade Stabilization: Although not anticipated, areas of unstable subgrade soils may be encountered during subgrade preparation for construction of the new pavement. Unstable foundation soils may be stabilized by overexcavation and replacement of the subgrade with suitable, imported, angular, well-graded materials. Other alternatives include the use of Type 2 biaxial geogrid reinforcement in combination with a layer of Class 6 aggregate base course. It has been our experience that the use of a crushed concrete product meeting a Class 6 gradation can perform well when trying to achieve stabilization. Specific stabilization requirements should be evaluated at the time of construction.

Drainage: The collection and diversion of surface drainage away from paved areas is extremely important to the satisfactory performance of the pavement. Drainage design should provide for the removal of water from paved areas and reduce wetting of the subgrade soils.

Maintenance: Periodic maintenance will be required in paved areas, consisting of grading to remove ruts and potholes created by the environment and traffic, and to replace material that has been washed away or contaminated. During the lifetime of the pavement, the aggregate surfacing may need to be scarified, with additional aggregate added to restore the thickness to the design depth. The subgrade soils should be prepared according to the “Site Grading” section of this report.

Pavement Materials: Aggregate Base Course should conform to the requirements of AASHTO M147 and to Section 703.03 of the Colorado Department of Transportation (CDOT) Standard Specifications for Road and Bridge Construction and should meet Class 5 or 6 grading and quality as defined by the CDOT specifications. Crushed concrete meeting these requirements may also be used, and may be more resistant to rutting.

EXCAVATION CONSIDERATIONS

In our opinion, the overburden soils encountered in the exploratory borings drilled for this study can be excavated with conventional construction equipment. In accordance with OSHA criteria, the on-site clays will classify as a Type B material, and the sands will classify as an OSHA Type C material. Per OSHA criteria, unless excavations are shored, temporary unretained excavations should have slopes no steeper than the following for each soil type encountered.

Type A…………………...3/4:1 (H:V)
Type B………………….1:1 (H:V)
Type C……………….1½:1 (H:V)

A properly braced excavation or the use of a trench box should be used where the indicated unretained slopes cannot be accommodated. Flatter slopes will be required where groundwater seepage is encountered. OSHA regulations require that excavations greater than 20 feet in depth be designed by a professional engineer. If subsurface conditions vary from those indicated in this report are encountered, the OSHA soil type may vary, and the required cut slopes may need to be adjusted. The contractor's "competent person" should make all decisions regarding excavation slopes.

As noted in this report, groundwater was encountered at a depth of about 15.3 feet during the subsurface investigation, and shallow perched water may also be present within the site soils. If groundwater is present above the depth of excavation, flatter slopes will be required. It is assumed site dewatering would occur in advance of the excavation and be maintained the entire duration that the excavation is open. Surface drainage should be diverted away from all temporary cut slopes in order to reduce the potential for slope erosion and instability. OSHA regulations require that excavations greater than 20 feet in depth and excavations that extend below the ground water level be designed by a professional engineer.

DESIGN AND SUPPORT SERVICES

Kumar & Associates, Inc. should be retained to review the project plans and specifications for conformance with the recommendations provided in this report. We are also available to assist the design team in preparing specifications for geotechnical aspects of the project and, if necessary, perform additional studies to accommodate any changes in the proposed construction.

We recommend that Kumar & Associates, Inc. be retained to provide construction observation and testing services to document that the intent of this report and the requirements of the plans and specifications are being followed during construction. This will allow us to identify possible variations in subsurface conditions from those encountered during this study and to allow us to re-evaluate our recommendations, if needed. We will not be responsible for implementation of the recommendations presented in this report by others, if we are not retained to provide construction observation and testing services.

LIMITATIONS

This study has been conducted for exclusive use by the client for geotechnical related design and construction criteria for the project. The conclusions and recommendations submitted in this report are based upon the data obtained from the exploratory borings at the locations indicated on Fig. 1 or as described in the report, and the proposed type of construction. This report may not reflect subsurface variations that occur between the exploratory borings, and the nature and extent of variations across the site may not become evident until site grading and excavations are performed. If during construction, fill, soil, rock or water conditions appear to be different from those described herein, Kumar & Associates, Inc. should be advised at once so that a re-evaluation of the recommendations presented in this report can be made. Kumar & Associates, Inc. is not responsible for liability associated with interpretation of subsurface data by others.

Swelling soils occur on this site. Such soils are stable at a fixed moisture content but will undergo high volume changes with changes in moisture content. The extent and amount of perched water beneath the building site as a result of area irrigation and inadequate surface drainage is difficult, if not impossible, to foresee.

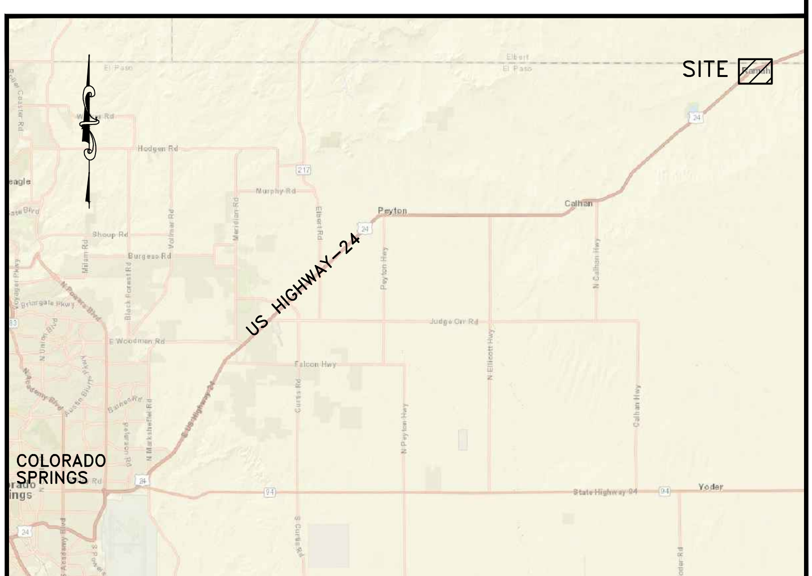
The recommendations presented in this report are based on current theories and experience of our engineers on the behavior of swelling soil in this area. Standards of practice in this area evolve over time. The owner should be aware that there is a risk in constructing a building in an expansive soil area. Following the recommendations given by a geotechnical engineer, careful construction practice and prudent maintenance by the owner can, however, decrease the risk of foundation movement due to expansive soils.

The scope of services for this project does not include any environmental assessment of the site or identification of contaminated or hazardous materials or conditions. If the owner is concerned about the potential for such contamination, other studies should be undertaken.

AFK:th

250 0 250 500

APPROXIMATE SCALE-FEET



VICINITY MAP
NOT TO SCALE

22-2-102	Kumar & Associates	LOCATION OF EXPLORATORY BORINGS	Fig. 1

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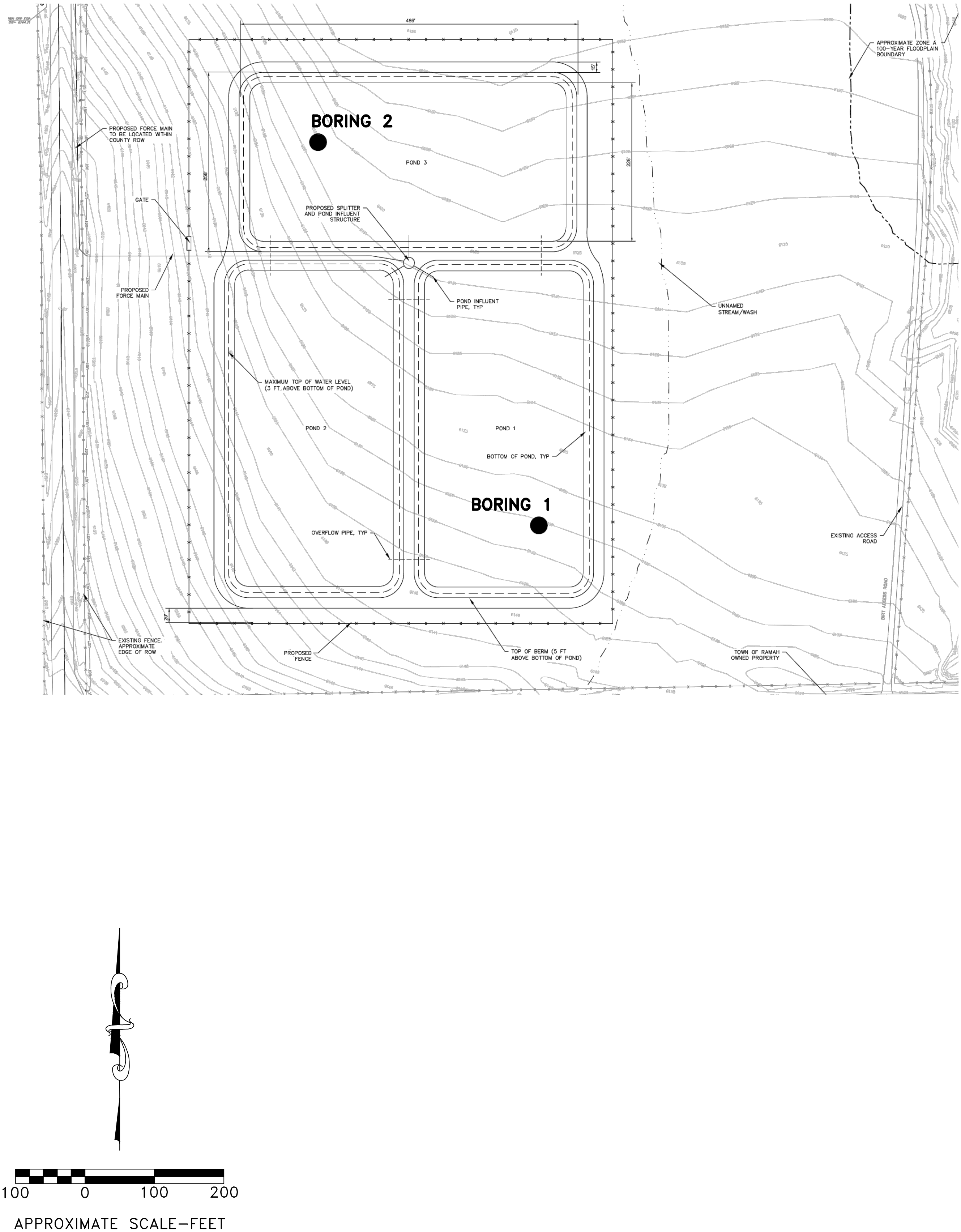

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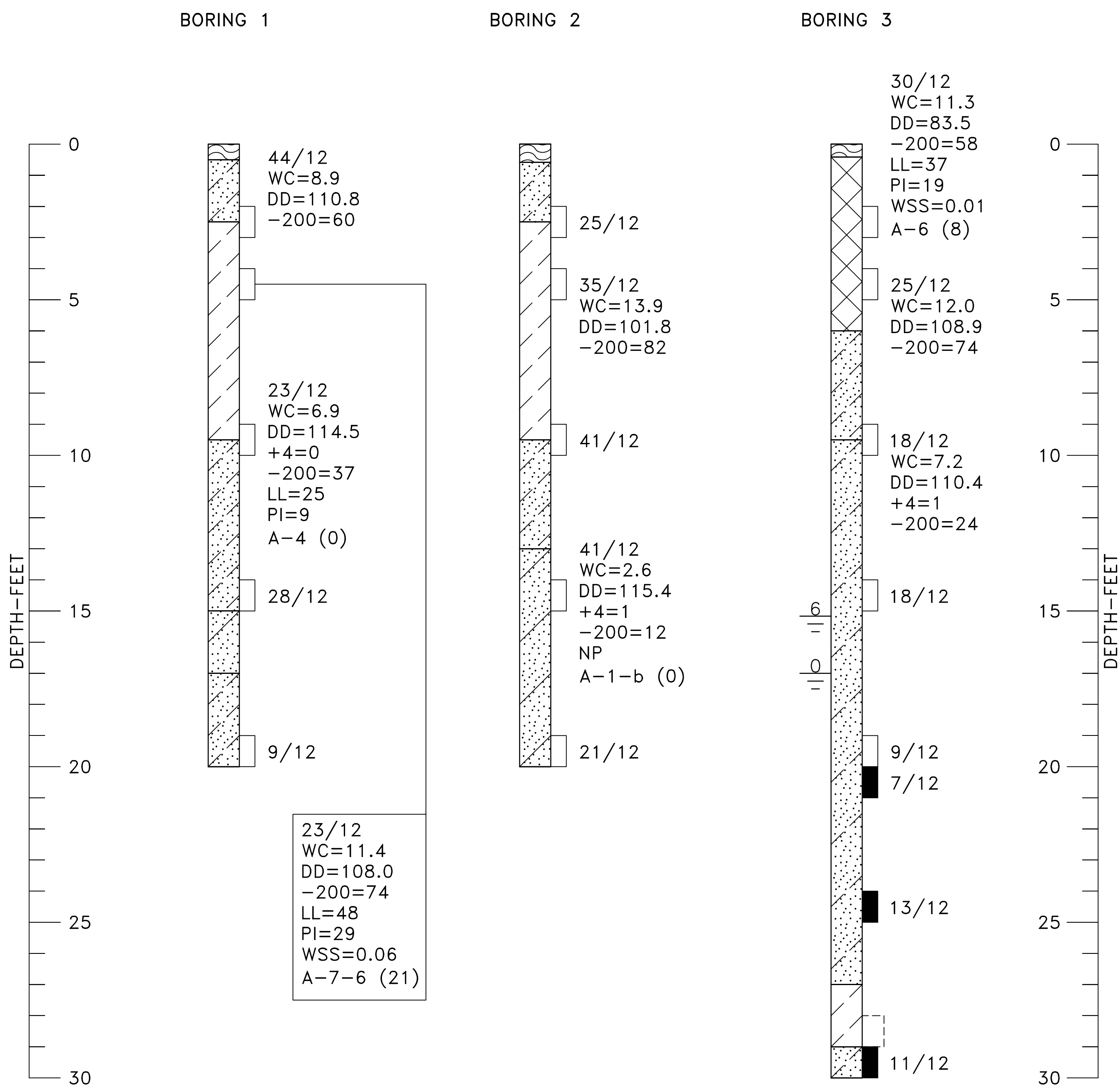
22-2-102	Kumar & Associates	LOCATION OF EXPLORATORY BORINGS	Fig. 1



22-2-102	Kumar & Associates	LOGS OF EXPLORATORY BORINGS	Fig. 2

LEGEND

TOPSOIL.

FILL: SANDY LEAN CLAY AND LEAN CLAY WITH SAND (CL), FINE TO COARSE GRAINED SAND FRACTION, MOIST, MOTTLED BROWNS.

CLAYEY SAND (SC), FINE TO COARSE GRAINED, MEDIUM DENSE TO DENSE, SLIGHTLY MOIST TO VERY MOIST, LIGHT BROWN TO GRAY–BROWN.

SANDY LEAN CLAY TO LEAN CLAY WITH SAND (CL), FINE TO COARSE GRAINED SAND FRACTION, VERY STIFF TO HARD, SLIGHTLY MOIST TO MOIST, LIGHT TO DARK BROWN.

WELL GRADED SAND WITH SILT (SW–SM), FINE TO COARSE GRAINED, MEDIUM DENSE TO DENSE, MOIST, LIGHT BROWN.

WELL GRADED SAND WITH CLAY (SW–SC), FINE TO COARSE GRAINED, LOOSE TO MEDIUM DENSE, MOIST TO WET, LIGHT BROWN TO GRAY–BROWN.

DRIVE SAMPLE, 2–INCH I.D. CALIFORNIA LINER SAMPLE.

DRIVE SAMPLE, 1 3/8–INCH I.D. SPLIT SPOON STANDARD PENETRATION TEST.

DISTURBED BULK SAMPLE.

44/12 DRIVE SAMPLE BLOW COUNT. INDICATES THAT 44 BLOWS OF A 140–POUND HAMMER FALLING 30 INCHES WERE REQUIRED TO DRIVE THE SAMPLER 12 INCHES.

6 DEPTH TO WATER LEVEL AND NUMBER OF DAYS AFTER DRILLING MEASUREMENT WAS MADE.

NOTES

1. THE EXPLORATORY BORINGS WERE DRILLED ON MARCH 15, 2022 WITH A 4–INCH–DIAMETER CONTINUOUS–FLIGHT POWER AUGER.

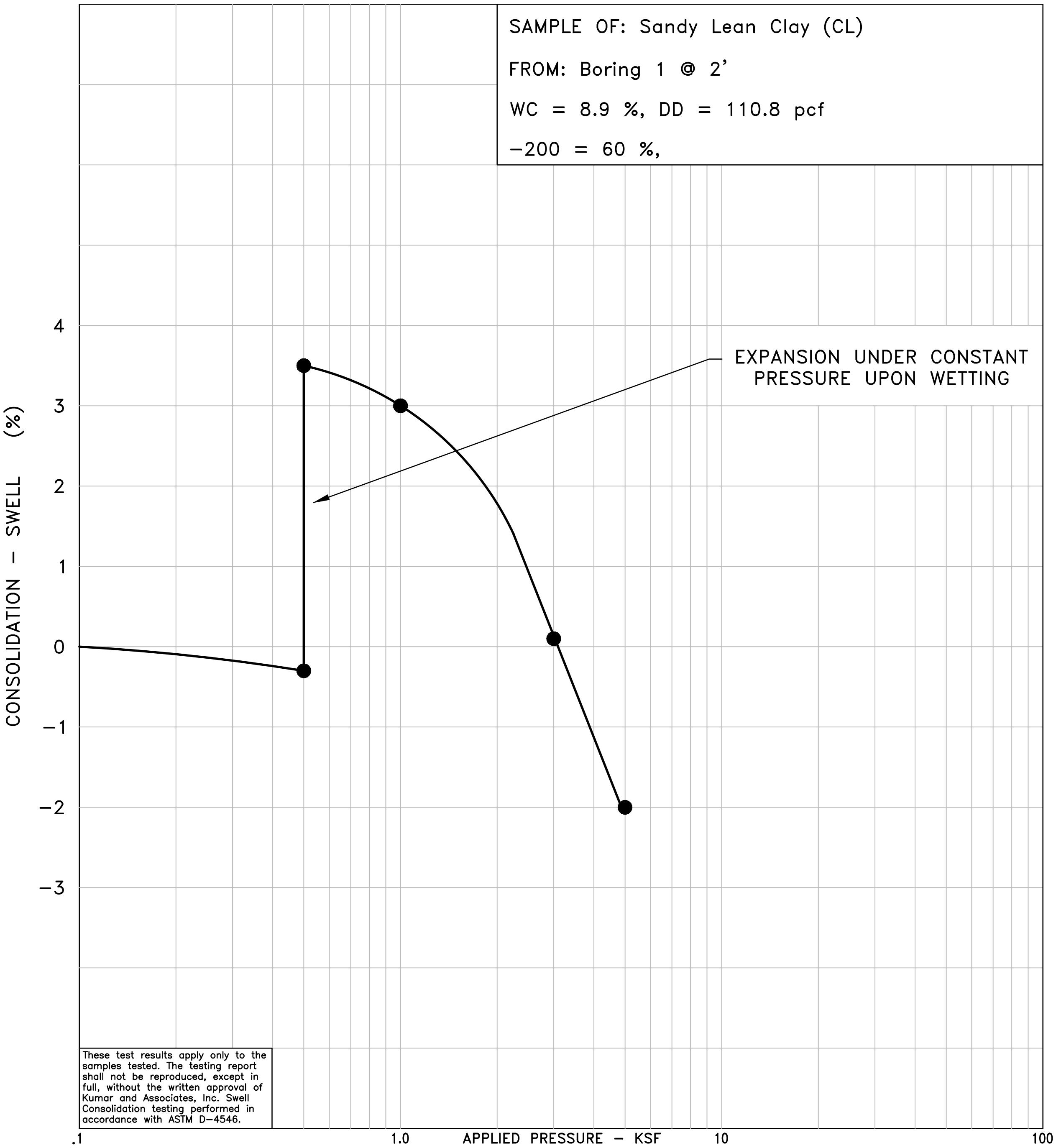
2. THE LOCATIONS OF THE EXPLORATORY BORINGS WERE MEASURED APPROXIMATELY BY PACING FROM FEATURES SHOWN ON THE SITE PLAN PROVIDED AND SHOULD BE CONSIDERED ACCURATE ONLY TO THE DEGREE IMPLIED BY THE METHOD USED.

3. THE ELEVATIONS OF THE EXPLORATORY BORINGS WERE NOT MEASURED AND THE LOGS OF THE EXPLORATORY BORINGS ARE PLOTTED TO DEPTH.

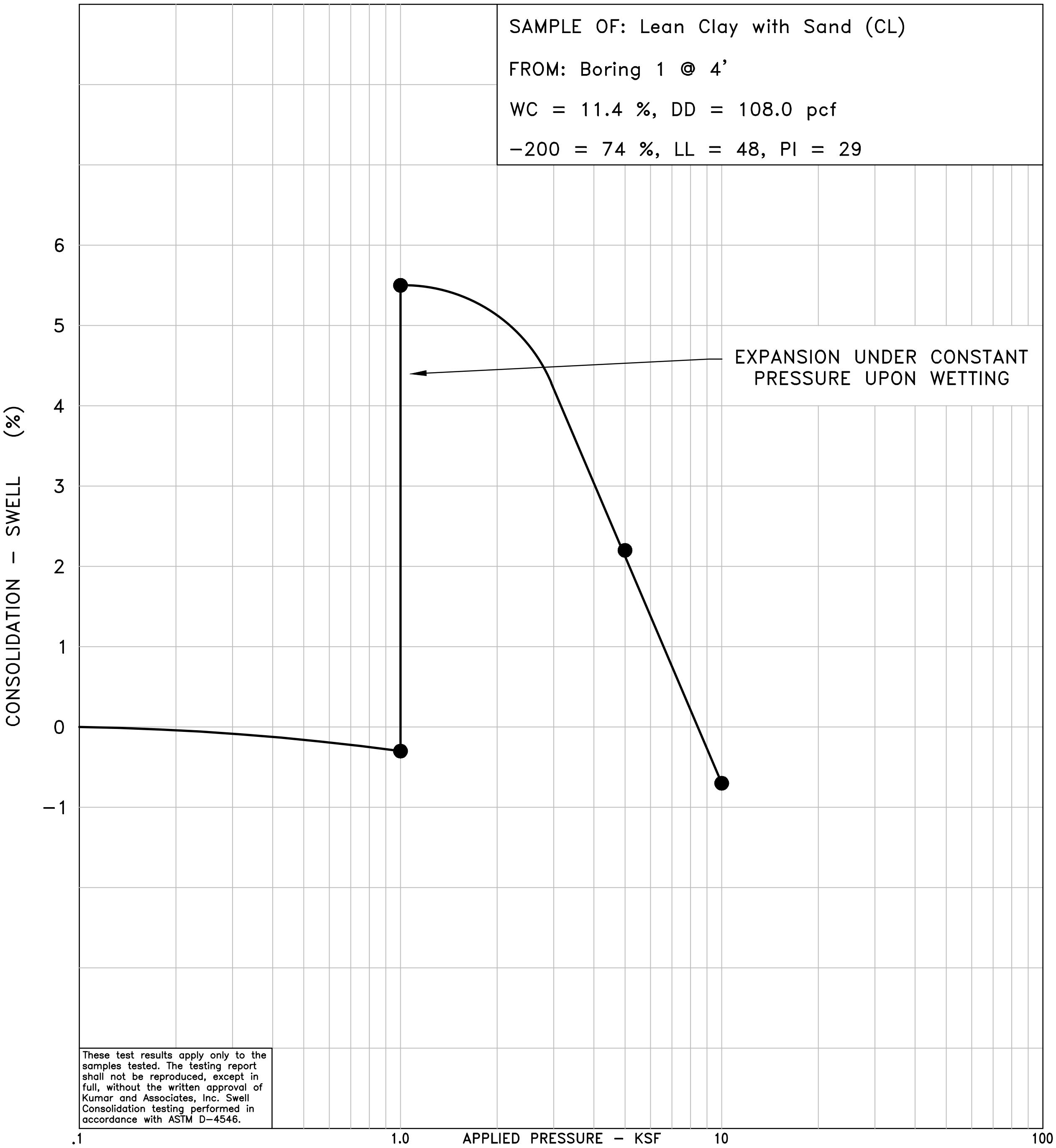
4. THE LINES BETWEEN MATERIALS SHOWN ON THE EXPLORATORY BORING LOGS REPRESENT THE APPROXIMATE BOUNDARIES BETWEEN MATERIAL TYPES AND THE TRANSITIONS MAY BE GRADUAL.

5. GROUNDWATER LEVELS SHOWN ON THE LOGS WERE MEASURED AT THE TIME AND UNDER CONDITIONS INDICATED. FLUCTUATIONS IN THE WATER LEVEL MAY OCCUR WITH TIME.

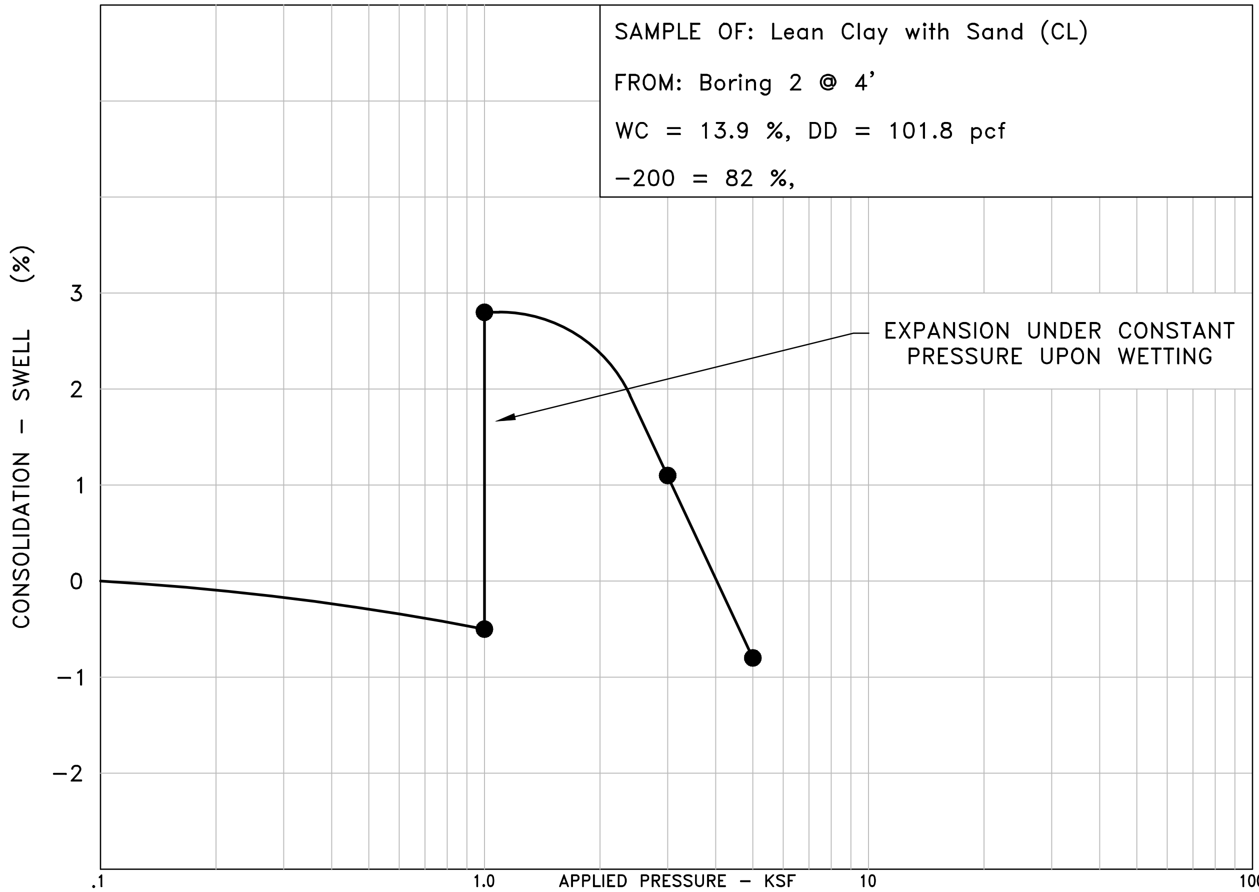
6. LABORATORY TEST RESULTS:
   WC = WATER CONTENT (%) (ASTM D2216);
   DD = DRY DENSITY (pcf) (ASTM D2216);
   +4 = PERCENTAGE RETAINED ON NO. 4 SIEVE (ASTM D6913);
   –200= PERCENTAGE PASSING NO. 200 SIEVE (ASTM D1140);
   LL = LIQUID LIMIT (ASTM D4318);
   PI = PLASTICITY INDEX (ASTM D4318);
   NP = NON–PLASTIC (ASTM D4318);
   WSS = WATER SOLUBLE SULFATES (%) (CP–L 2103);
   A–7–6 (21) = AASHTO CLASSIFICATION (GROUP INDEX) (AASHTO M 145).

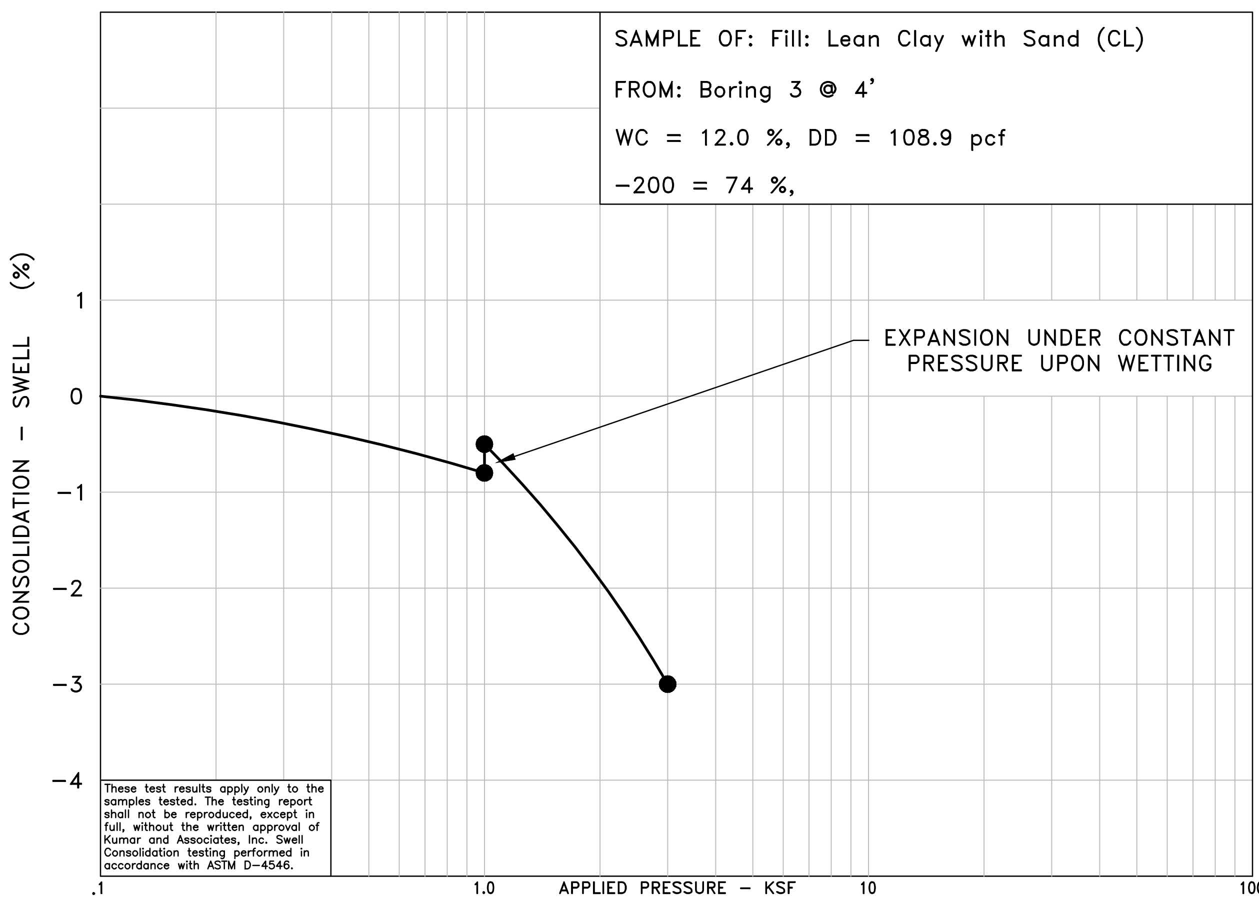


22−2−102	Kumar & Associates	SWELL−CONSOLIDATION TEST RESULTS	Fig. 4

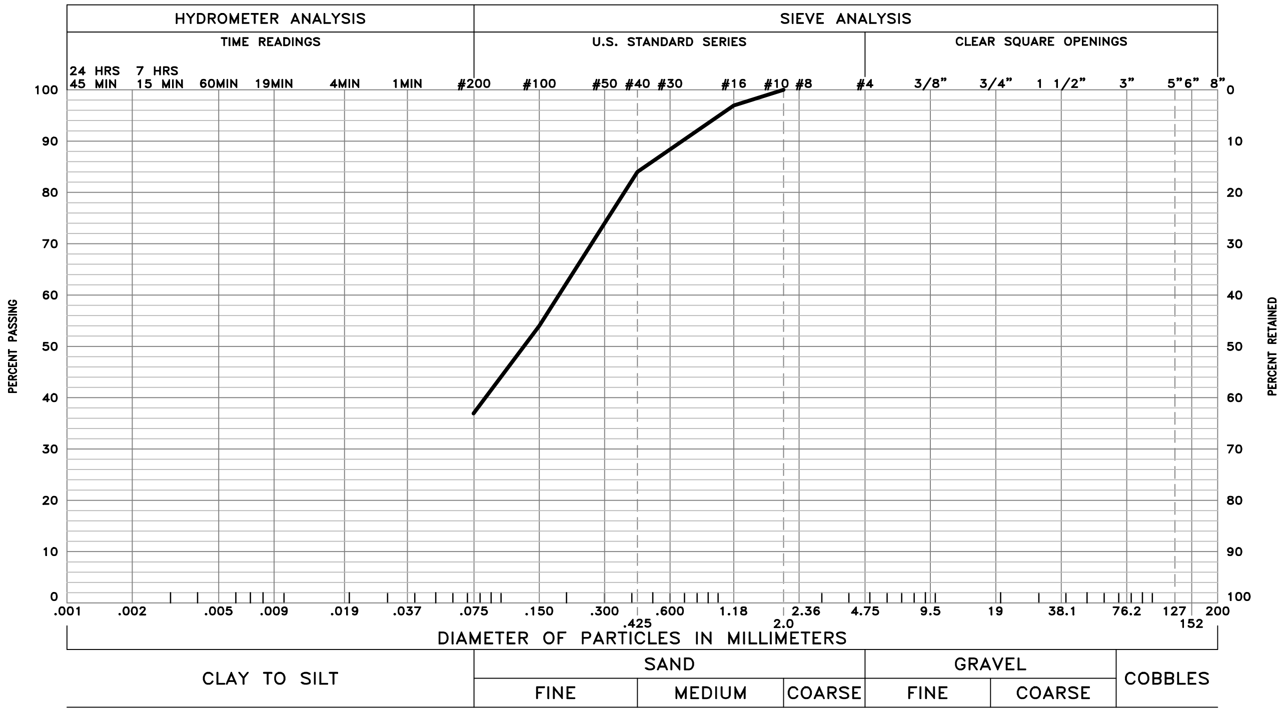


22-2-102	Kumar & Associates	SWELL-CONSOLIDATION TEST RESULTS	Fig. 5





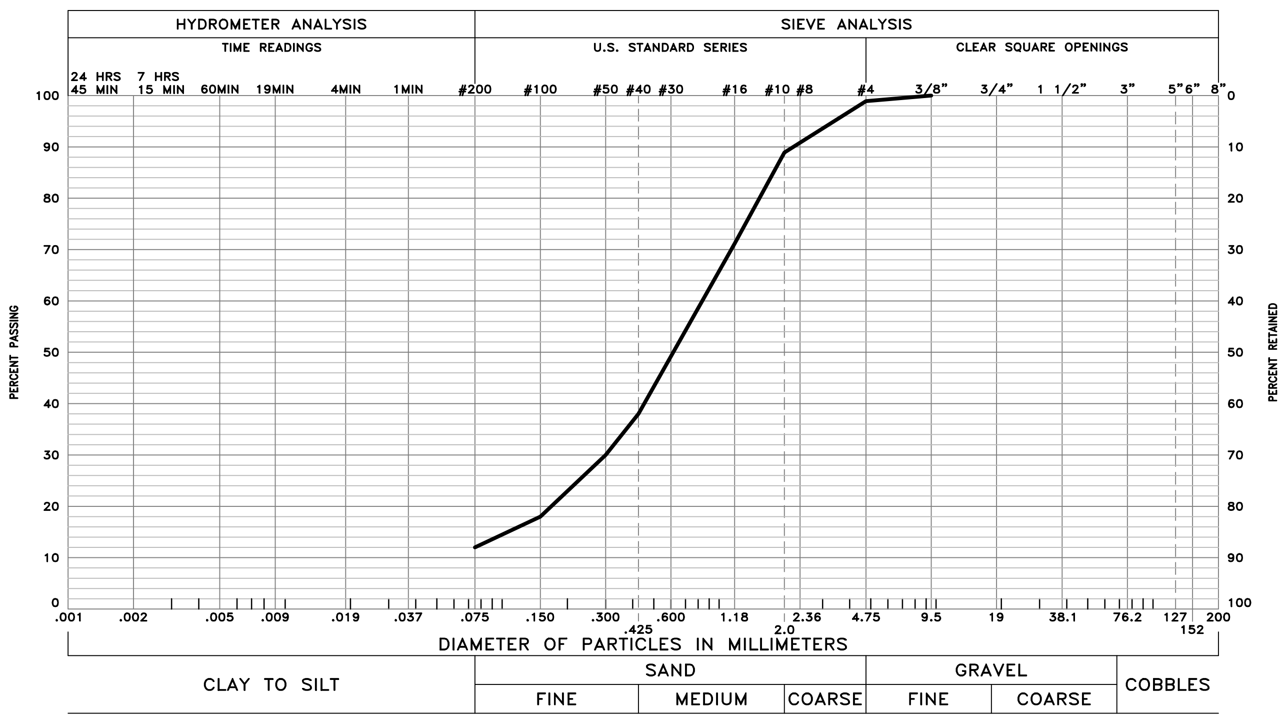
22−2−102	Kumar & Associates	SWELL−CONSOLIDATION TEST RESULTS	Fig. 6



GRAVEL 0 % SAND 63 % SILT AND CLAY 37 %

LIQUID LIMIT 25 PLASTICITY INDEX 9

SAMPLE OF: Clayey Sand (SC) FROM: Boring 1 @ 9'



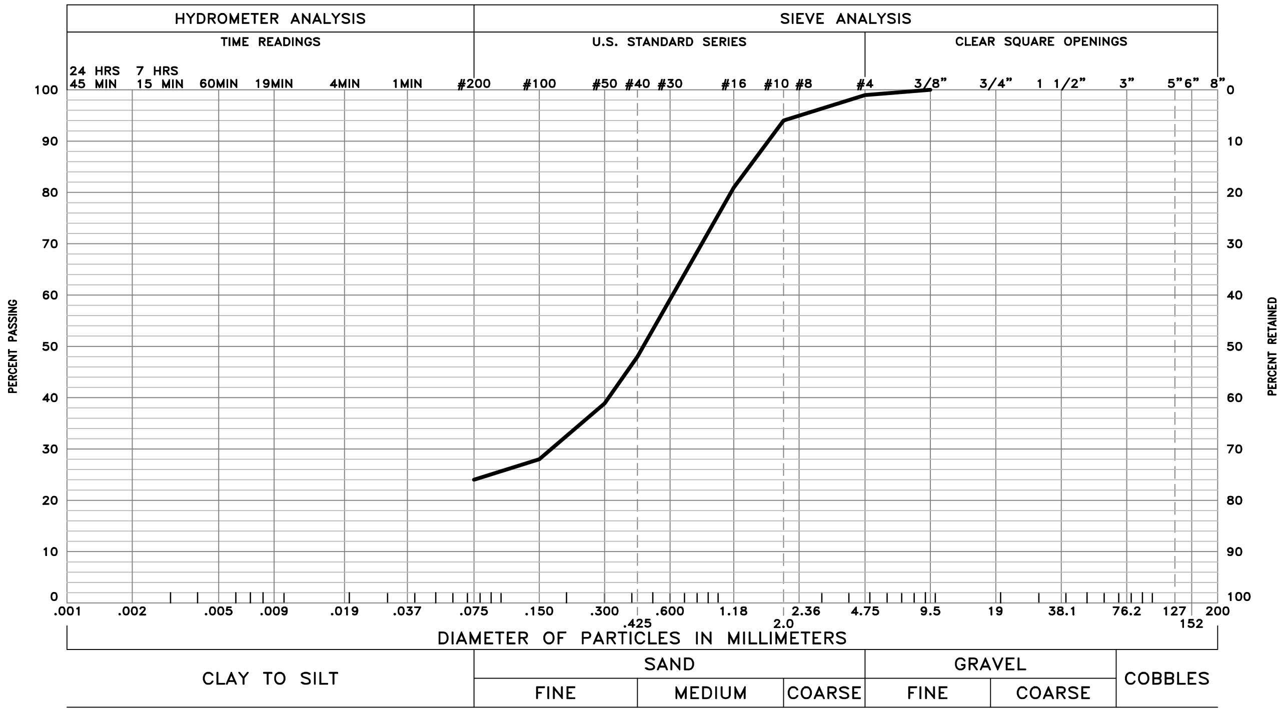
GRAVEL 1 % SAND 87 % SILT AND CLAY 12 %

LIQUID LIMIT – PLASTICITY INDEX NP

SAMPLE OF: Well Graded Sand with Silt (SW–SM) FROM: Boring 2 @ 14'

These test results apply only to the samples which were tested. The testing report shall not be reproduced, except in full, without the written approval of Kumar & Associates, Inc. Sieve analysis testing is performed in accordance with ASTM D6913, ASTM D7928, ASTM C136 and/or ASTM D1140.

22–2–102	Kumar & Associates	GRADATION TEST RESULTS	Fig. 7



GRAVEL 1 % SAND 75 % SILT AND CLAY 24 %

LIQUID LIMIT – PLASTICITY INDEX –

SAMPLE OF: Clayey Sand (SC) FROM: Boring 3 @ 9'

These test results apply only to the samples which were tested. The testing report shall not be reproduced, except in full, without the written approval of Kumar & Associates, Inc. Sieve analysis testing is performed in accordance with ASTM D6913, ASTM D7928, ASTM C136 and/or ASTM D1140.

22–2–102	Kumar & Associates	GRADATION TEST RESULTS	Fig. 8

Kumar and Associates, Inc.

TABLE I

SUMMARY OF LABORATORY TEST RESULTS

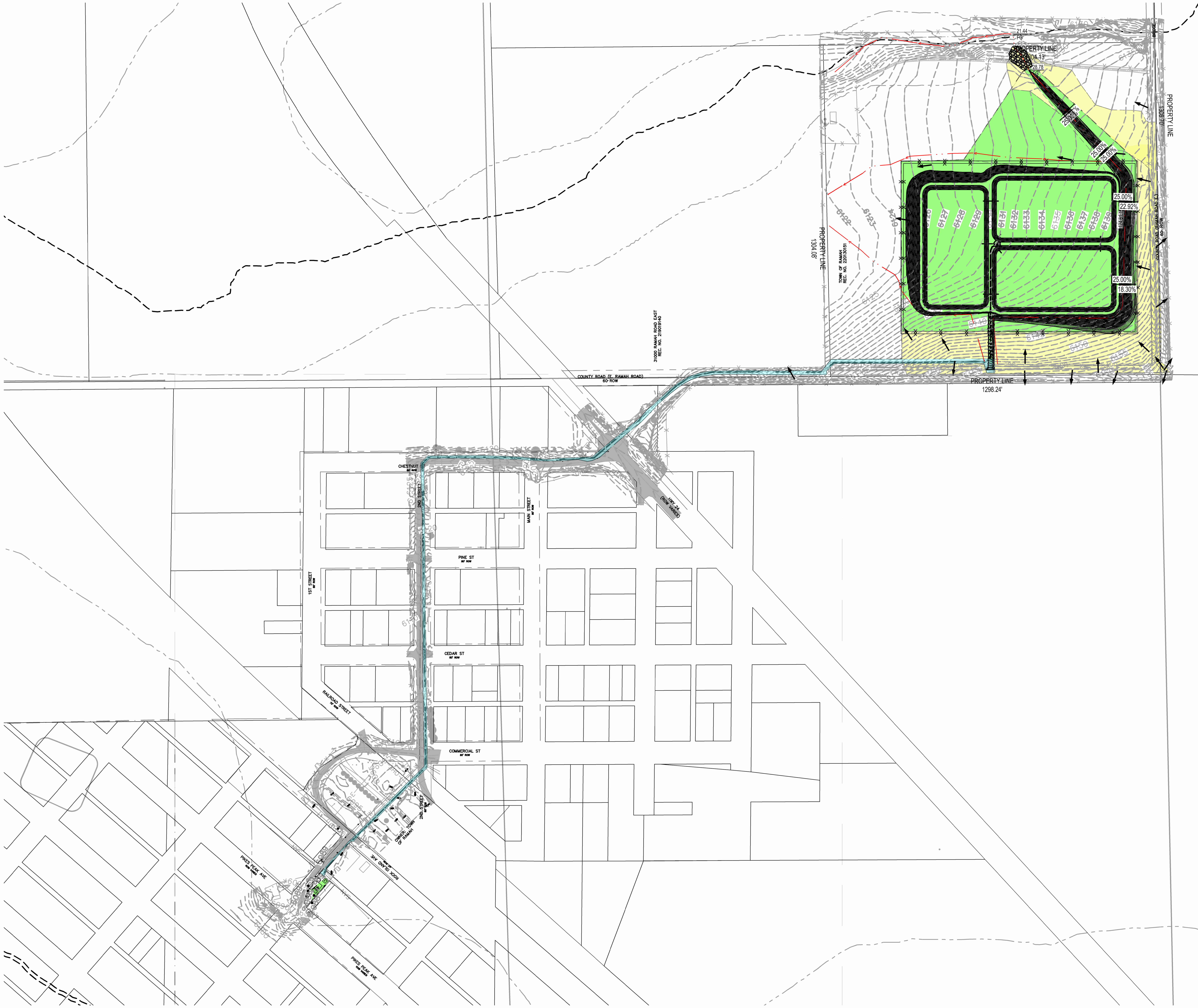
Project No.: 22-2-102
Project Name: Ramah Water Treatment Plant
Date Sampled: 03/15/2022
Date Received: 3/15/22

SAMPLE LOCATION		DATE TESTED	NATURAL MOISTURE CONTENT (%)	NATURAL DRY DENSITY (pcf)	GRADATION		PERCENT PASSING NO. 200 SIEVE	ATTERBERG LIMITS		WATER SOLUBLE SULFATES (%)	AASHTO CLASSIFICATION (Group Index)	SOIL OR BEDROCK TYPE (Unified Soil Classification)
BORING	DEPTH (ft)				GRAVEL (%)	SAND (%)		LIQUID LIMIT	PLASTICITY INDEX			
1	2	3/30/22	8.9	110.8			60					Sandy Lean Clay (CL)
1	4	3/30/22	11.4	108.0			74	48	29	0.06	A-7-6 (21)	Lean Clay with Sand (CL)
1	9	3/30/22	6.9	114.5	0	63	37	25	9		A-4 (0)	Clayey Sand (SC)
2	4	3/30/22	13.9	101.8			82					Lean Clay with Sand (CL)
2	14	3/30/22	2.6	115.4	1	87	12		NP		A-1-b (0)	Well Graded Sand with Silt (SW-SM)
3	2	3/30/22	11.3	83.5			58	37	19	0.01	A-6 (8)	Fill: Sandy Lean Clay (CL)
3	4	3/30/22	12.0	108.9			74					Fill: Lean Clay with Sand (CL)
3	9	3/30/22	7.2	110.4	1	75	24					Clayey Sand (SC)

APPENDIX D: DRAINAGE PLAN

TOWN OF RAMAH WASTE WATER SYSTEM IMPROVMENTS

DRAINAGE PLANS



N S W

200' 0 100' 200' 400'

ORIGINAL SCALE: 1" = 200'

UTILITY EXCEPTION I.7.1.B.4

RUNOFF REDUCTION

NON-DISTURBED AREA

EVstudio

Denver, CO
Evergreen, CO

303.670.7242
design@evstudio.com
inspections@evstudio.com
www.evstudio.com

Contact:
Brian Welch, PE
brian.welch@evstudio.com
303.670.7242 x50

UTILITY NOTIFICATION CENTER

The 811 Promise.com

FOR MARKING OF UNDERGROUND MEMBER UTILITIES.

EVSTUDIO ASSUMES NO RESPONSIBILITY FOR UTILITY LOCATIONS. THE UTILITIES SHOWN ON THIS DRAWING HAVE BEEN PLOTTED FROM THE BEST AVAILABLE INFORMATION. IT IS, HOWEVER, THE CONTRACTOR'S RESPONSIBILITY TO FIELD VERIFY THE LOCATION OF ALL UTILITIES PRIOR TO CONSTRUCTION.

WW SYSTEM IMPROVEMENTS

RAMAH RD, RAMAH, CO 80832

22A267

REVISION:

DATE: 3/15/2024
DRAWN BY: TRO
CHECKED BY: BMW

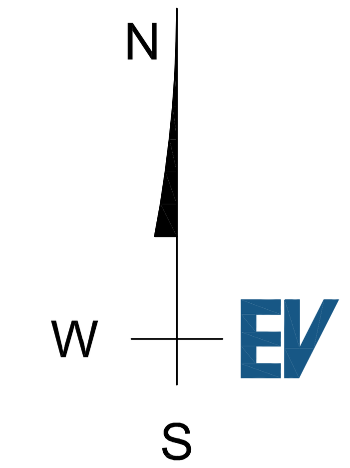
GENERAL LOCATION

C01

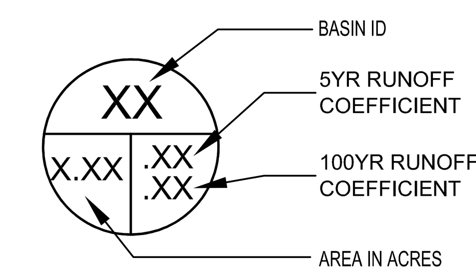
PCD FILE NO. PPR2325

TOWN OF RAMAH WASTE WATER SYSTEM IMPROVMENTS

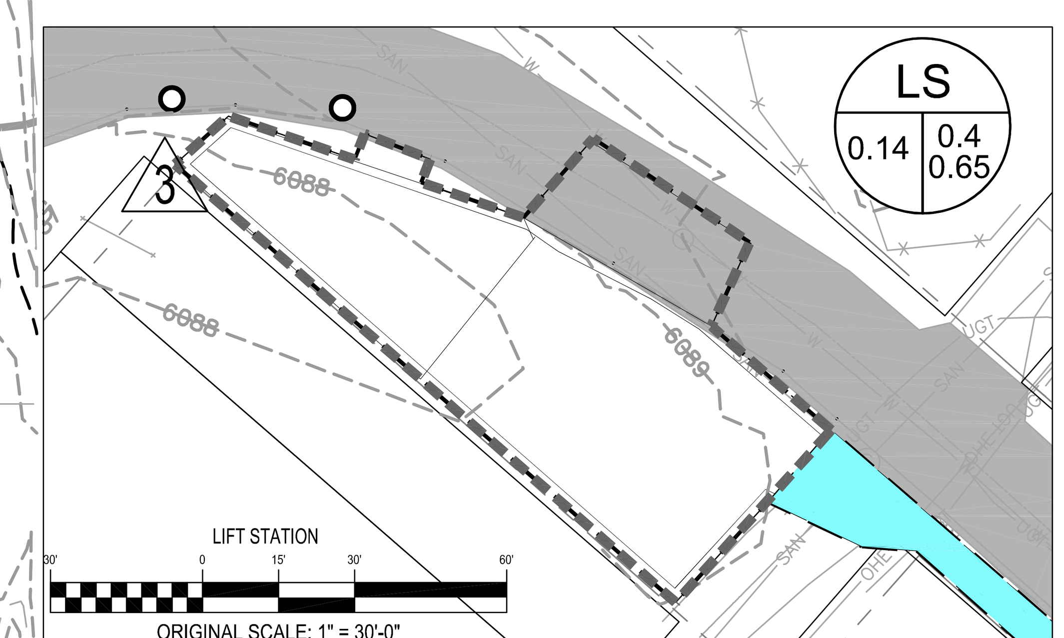
DRAINAGE PLANS



BASIN/DESIGN BOUNDARIES



Design Point



Basin runoff						
Basin ID		C Values		Runoff (cfs)		Comments
		5-year	100-year	5-year	100-year	
Existing						
EX-1	1	0.13	0.40	7.17	51.3	
EX-2	2	0.37	1.06	3.37	22.0	
EX-LS	3	0.40	0.65	0.19	0.7	
EX Total		0.16	0.48	10.73	73.95	

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WW SYSTEM IMPROVEMENTS
RAMAH RD, RAMAH, CO 80832

22A267

REVISION:

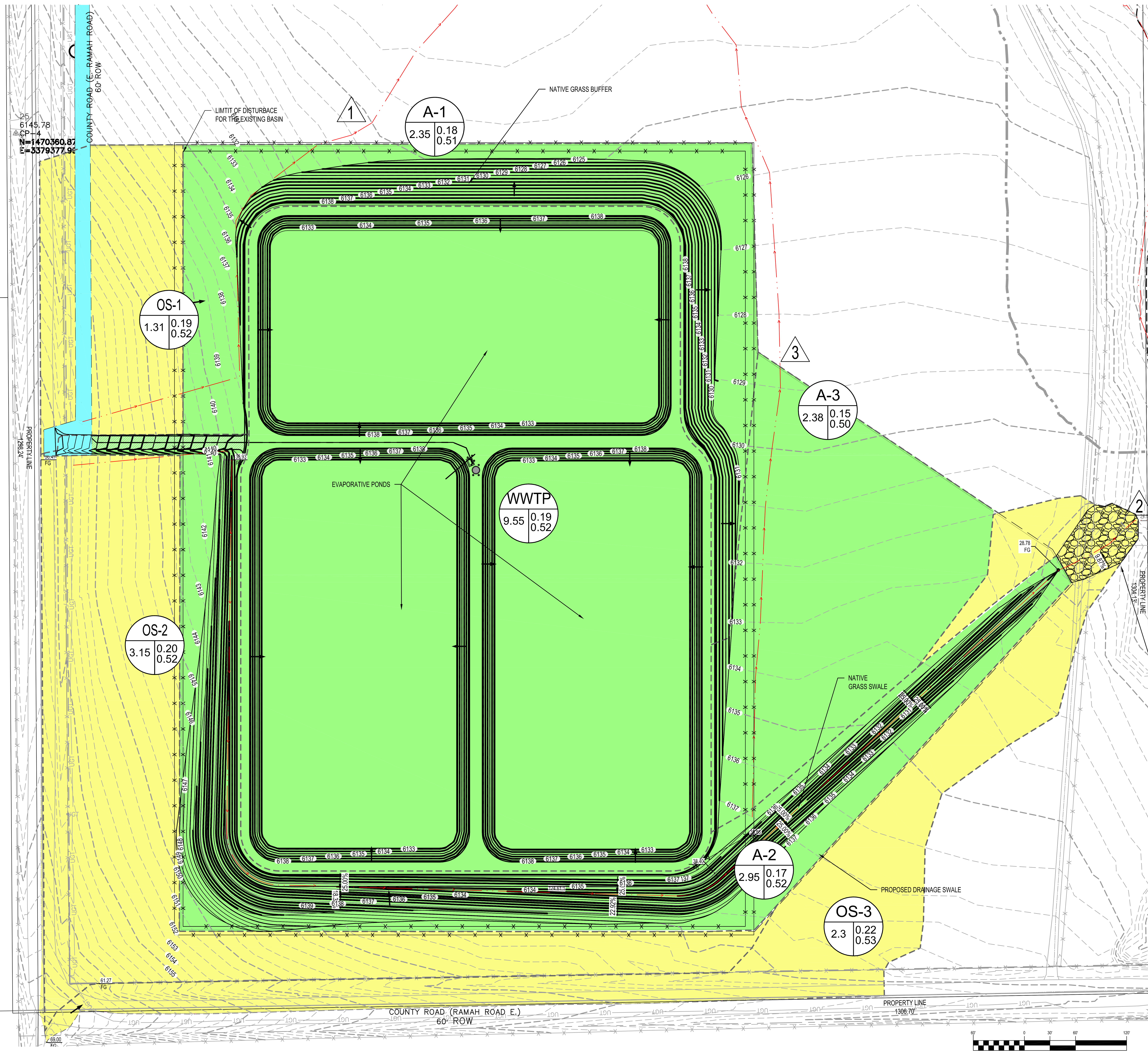
DATE: 3/15/2024
DRAWN BY: TRO
CHECKED BY: BMW

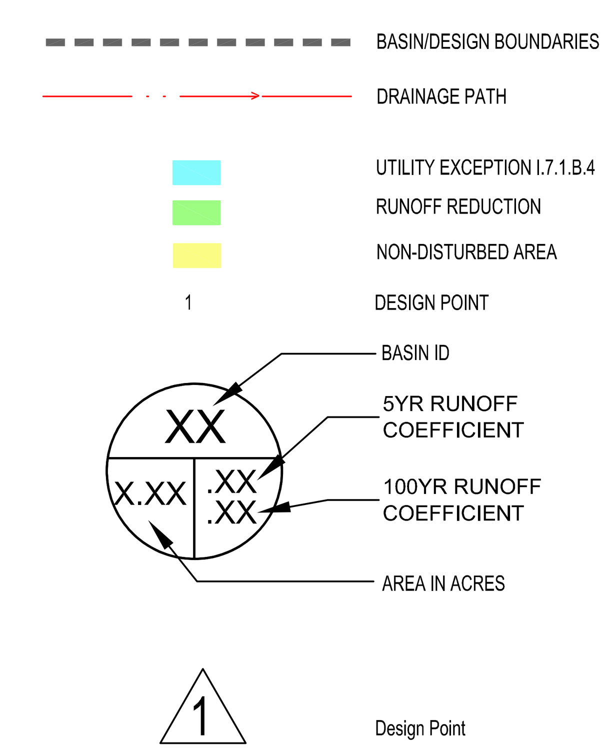
EXISTING DRAINAGE

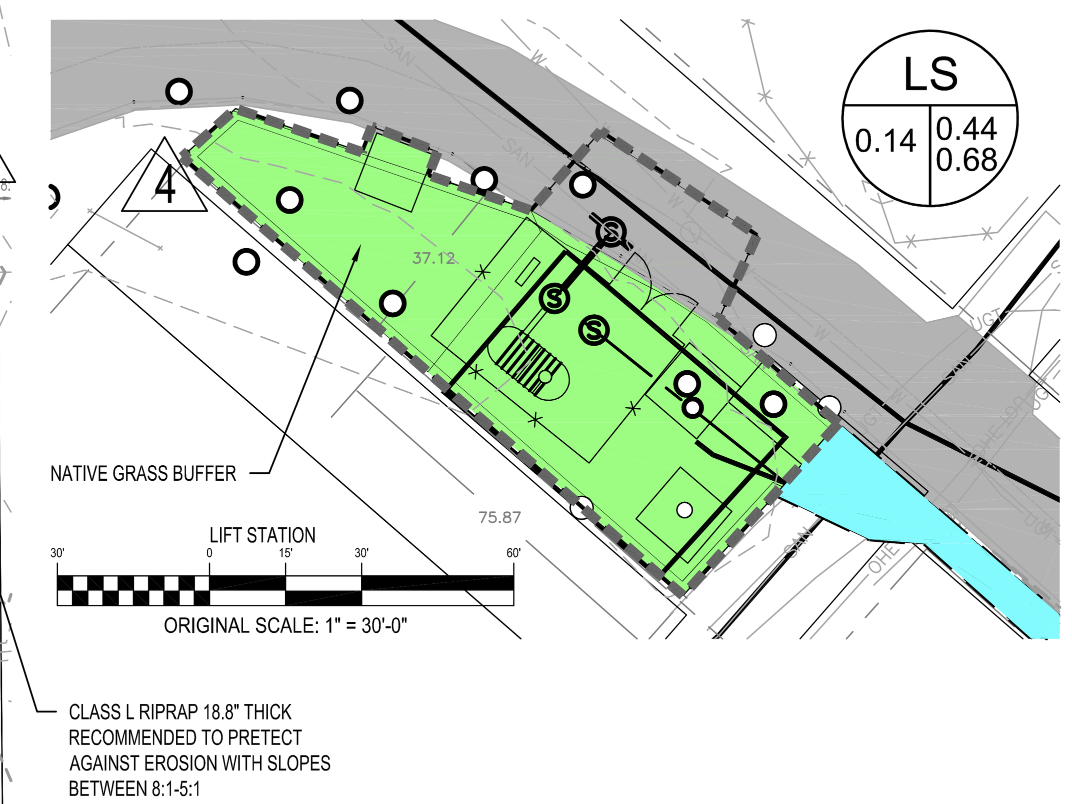
C02

TOWN OF RAMAH WASTE WATER SYSTEM IMPROVMENTS

DRAINAGE PLANS







Basin ID	POINT	C Values		Runoff (cfs)		Comments
		5-year	100-year	5-year	100-year	
Proposed						
WWTP		0.19	0.52	6.50	40.0	Contained on site
A-1	1	0.18	0.51	1.29	8.5	
OS-1	1	0.19	0.52	0.81	4.9	
A-2	2	0.17	0.52	2.02	12.1	
OS-2	2	0.20	0.52	1.86	11.3	
OS-3	2	0.22	0.53	1.77	9.6	
A-3	3	0.15	0.50	1.03	7.6	28.2 CFS < 51 CFS
LS	4	0.44	0.68	0.21	0.7	
Detained Runoff					40.0	CFS
Site Runoff					28.2	CFS
Bypassed Runoff					25.8	CFS
Total Site Runoff					54.7	CFS
Design Point 1 Sum					13.4	CFS
Design Point 2 Sum					33.0	CFS

Basin Runoff

EVstudio
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Contact:
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WASTEWATER SYSTEM IMPROVEMENTS

RAMAH RD, RAMAH, CO 80832

22A267

REVISION:

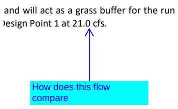
DATE: 5/28/2024
DRAWN BY: TRO
CHECKED BY: BMW

PROPOSED DRAINAGE PLAN

C03

V6_Drainage Report - Final.pdf Markup Summary

Callout (1)



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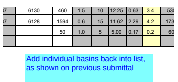
How does this flow compare

Text Box (4)



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Add individual basins back into list, as shown on previous submittal

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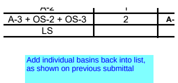

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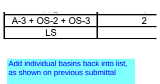

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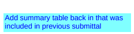

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Add summary table back in that was included in previous submittal