Storm Water Design Report

P1FCU Ponderay

Highway 95
Ponderay, ID 83852

Prepared for:
Mercier Architecture and Planning

Prepared by:

DCI ENGINEERS

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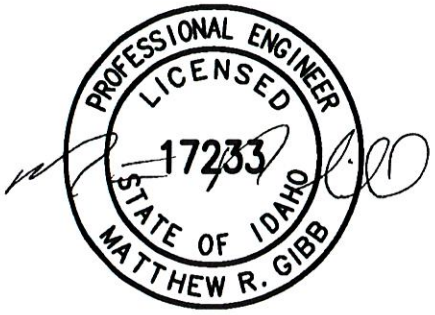
This report has been prepared by the staff of DCI Engineers under the direction of the professional engineer whose stamp and signature appears hereon.



DCI Job No.: #20042-0005
Date: March 4, 2020

The methods, descriptions, and design calculations shown in this design report conform to the City of Ponderay design standards, unless noted otherwise, and are under the jurisdiction of the City of Ponderay relative to the collection, treatment, and disposal of stormwater runoff.

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Basis of Design of Storm Water System

1.0 Introduction

The intent of this design brief is to determine the size of storm water swales and drywells required to handle storm water runoff from the development of a new P1FCU Credit Union (3,800 SF ± footprint) with associated parking, sidewalk, curbing and landscaping constructed in the City of Ponderay, Idaho. The project is located on the west side of Highway 95, just north of the Ponderay police department. See Appendix I for a Project Location Map.

This storm water report has utilized the rational method to design on-site facilities. This method calculates the pre- and post-development runoff based on anticipated lot coverage of buildings, parking and landscape. A storm drainage system (grassed detention area) will be installed to accommodate the storm water runoff generated by the impervious areas.

2.0 Existing Site Conditions

The site consists of a 1.4-acre vacant lot located on the west side of Highway 95, just north of the Ponderay police department. An existing swale drains the west side of Highway 95 along the property frontage. The site is mainly flat with a slight downhill slope running to the southeast corner of the site at ~1.00%.

A geotechnical investigation was performed for this project by ALLWEST Testing & Engineering dated December 23, 2019. The geotechnical report indicates that the onsite soil consists of 'Misson Silt Loam', a rather poorly drained mantle of material derived from volcanic ash and loess underlain by glaciolacustrine deposits. The report also states that the permeability of these soils is very low, and the water erosion hazard is slight. See Appendix II for the Geotechnical Report.

Based on the soil types present on site, infiltration is not an acceptable method of stormwater management.

3.0 Proposed Storm Water Design Concept

The proposed long-range storm water management concept for this project is a grassed detention area that receives and treats storm water runoff, and a catch basin with a metered outlet structure for offsite discharge.

Water Quality Treatment: The area of pollutant-generating impervious surface (PGIS) will be approximately 27,584 square feet, which includes the parking lot and all hydraulically connected hardscape. The City of Ponderay requires the project to provide water quality treatment volume equal to the first one-half inch of runoff over the PGIS area (1,149 cubic feet). 1,169 cubic feet of swale volume has been set aside as shown to provide 6" of treatment depth. A Type 1 catch basin will be installed with its rim set to 6" above the swale bottom and will take overflow from larger storm events.

Flow Control: The new building will be approximately 3,780 square feet. The drive-through canopy is not included in the building area but is included in the PGIS area, since pollutants from under the canopy will likely make their way to the swale. The City of Ponderay requires the project to detain the difference between the pre- and post-developed 25-year design storm. All roof (non-pollutant generating) runoff will be piped directly to the proposed Type 1 catch basin as shown. The remainder of the site will reach the catch basin via overland flow across the parking lot and through the proposed swale. The catch basin outlet will be a 6" pipe that restricts outflow to the pre-development runoff rate (0.61 cfs). The outlet pipe will discharge runoff to a roadside

swale located within the US Highway 95 storm system. Using the Modified Rational Method, it was determined that 403 cubic feet of swale volume is required to detain the 25-year post-development storm event. Since the proposed swale already contains 1,169 cubic feet for water quality treatment, no upsizing is required. Design calculations are attached. The east end of the swale will be constructed as a permeable earthen berm, so that stormwater will not sit in the bottom 6" of the swale indefinitely but will slowly drain out through the berm into the roadside swale.

4.0 Proposed Drainage System

The proposed project includes the construction of a new P1FCU Credit Union (3,800 sqft footprint) with associated parking, sidewalk, curbing and landscaping. The asphalt areas will drain to an appropriately sized swale. The roof will be a non-pollution generating surface (NPGIS), therefore no treatment is required. Storm water runoff from the roof will be piped directly to a proposed catch basin.

The entire site has been modeled as one drainage basin. See Appendix III for a Drainage Area Basin Map.

Basin A is comprised of the entire building and parking lot. Stormwater runoff from the parking lot will drain to a swale with associated catch basin in the southern portion of the lot. Downspouts on the building will pipe stormwater from the roof directly to the catch basin located within Swale A.

Storm water calculations were completed for each individual basin. The drainage areas are based on post development conditions of the site. The slopes within the drainage areas generally range from approximately 1% to 3% in the areas of construction.

The Rational Method was used for all pre- and post-developed storm water calculations. The drainage areas, which are less than 10 acres in size, were analyzed using the Rational Method (Q=CIA) to determine the peak storm water runoff. See Appendix IV for the Storm Water Calculation Spreadsheet.

5.0 Emergency Overflow

In the case of a storm event that is larger than the design storm, overflow runoff will flow southeast into the Highway 95 storm water system. The site is graded in such a way that the overflow will not reach the finished floor elevation.

6.0 Erosion Control Considerations

The Contractor is responsible for ensuring the use of proper erosion control BMP's on the project site and shall maintain such measures throughout construction, until all pertinent landscaping and permanent erosion control measures (i.e. grassed areas, paved surfaces) have been established. Maintenance shall include daily inspections and repair of the silt fencing, hay bales, or other. The Contractor will also inspect all erosion control measures following each storm water event during construction or until the permanent measures are established.

Specific temporary measures, which will be used during construction, include the installation of silt fences and construction entrances. The measures will be installed along the down gradient property lines, parallel with the existing ground contours or perpendicular to the storm water runoff direction.

A construction entrance will be required in order to clean the tires of trucks and vehicles exiting the construction area. Periodically, the temporary erosion control measures must be cleaned of debris and siltation. The contractor shall dispose of the materials so as not to damage any reclaimed areas or create other erosion problem areas. Upon direction by the City of Ponderay, Owner, or Engineer, the Contractor may also be required to clean the roadway of siltation or other debris, which may occur along or at the construction entrance.

7.0 Operation and Maintenance

The property owner is responsible for maintaining the proposed storm drain system. The owner shall inspect the facilities after any major storm event to ensure it is draining properly. The owner shall also conduct yearly cleaning of all catch basins.

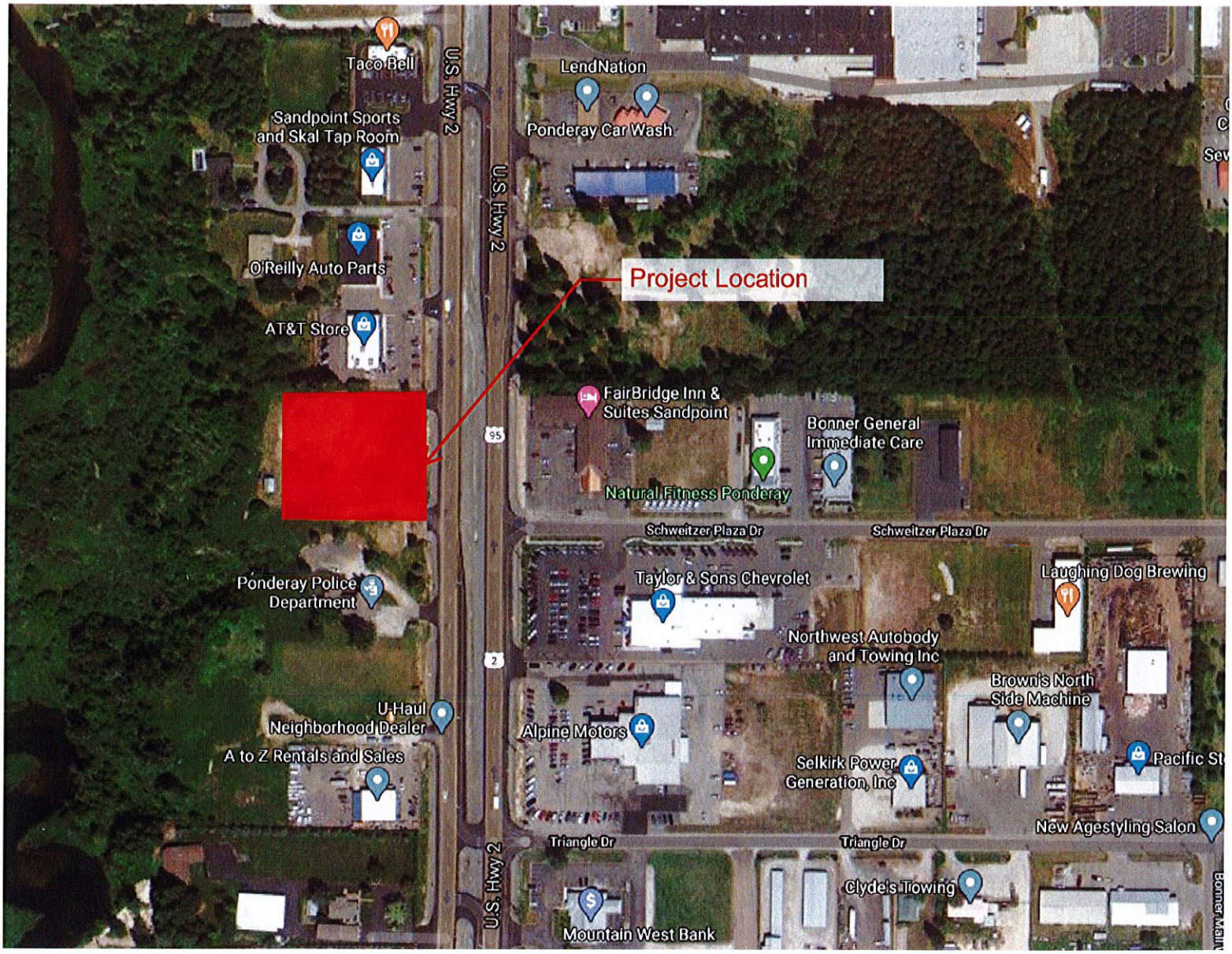
8.0 Summary/Conclusions

The proposed storm water system for this project provides treatment and discharge of runoff as required by the City of Ponderay.

The Contractor shall be responsible for the proper installation and maintenance of all temporary erosion control measures necessary to protect down gradient areas from siltation. The Contractor shall also protect against siltation of the storm water system throughout construction.

Appendix I

Vicinity Map



Appendix II

December 23, 2019

Kenaston Corporation
2517 Main Street
Lewiston, Idaho 83501

Attention: Mr. Reese Hewett

RE: Geotechnical Evaluation
P1FCU Ponderay Branch
Parcel #RPP00000101810A
Ponderay, Idaho
ALLWEST Project No. 119-544P

Mr. Hewett,

ALLWEST has completed the authorized geotechnical evaluation for the proposed P1FCU building, and related site improvements located at Parcel #RPP00000101810A in Ponderay, Idaho. The purpose of this evaluation was to characterize the soil and geologic conditions on the property. The attached report presents the results of the field evaluation and our recommendations to assist with design and construction of the proposed project.

We appreciate the opportunity to work with you on this project. If you have any questions or need additional information, please do not hesitate to call us at (208) 762-4721.

Sincerely,
ALLWEST

Prepared by:

Scott A. Marshall

Scott A. Marshall, P.G.
Engineering Geologist

Reviewed by:

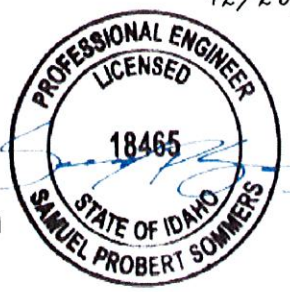
Samuel P Sommers

Samuel Sommers, P.E.
Hayden Engineering Manager

GEOTECHNICAL EVALUATION
P1FCU PONDERAY BRANCH
PARCEL #RPP00000101810A
PONDERAY, IDAHO
ALLWEST PROJECT NO. 119-544P

12/23/2019

December 23, 2019



Prepared for:
Kenaston Corporation
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ALLWEST

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Geotechnical Evaluation
P1FCU Ponderay Branch
Parcel #RPP00000101810A
Ponderay, Idaho

EXECUTIVE SUMMARY

ALLWEST has completed the authorized geotechnical evaluation for the P1FCU Ponderay Branch located at Parcel #RPP00000101810A in Ponderay, Idaho. The general location of the project is shown on the Vicinity Map, Figure A-1, in Appendix A of this report. The purpose of the evaluation was to assess the subsurface conditions on the property with respect to the proposed design and construction. This report details the results of the field evaluation and laboratory testing and presents our recommendations to assist the design and construction of the proposed project. The following geotechnical considerations were identified:

- Frost susceptible soil should be over-excavated to a depth of 2 feet below building footing grade and replaced with imported structural fill.
- All structural elements will require over-excavation of undocumented fill where encountered. This unit was up to 2 feet thick in our explorations.
- An allowable bearing pressure of 2,000 pounds per square foot (psf) can be used for shallow footings bearing on a minimum of 24 inches of structural fill.
- The on-site soil is unsuitable for reuse as structural fill.
- For light-duty pavement: A pavement section of 2½-inches asphaltic concrete over a minimum of 4-inches crushed aggregate base over 12-inches of structural fill is recommended.
- For heavy-duty pavement: A pavement section of 3-inches asphaltic concrete over a minimum of 4-inches of crushed aggregate base over 12-inches of structural fill is recommended.
- Site conditions are not suitable for stormwater infiltration.

Our services were provided in general accordance with our proposal 119-544P dated November 26, 2019. Close monitoring of the construction operations discussed herein will be critical in achieving the design subgrade support. If we are not retained to provide required construction observation and materials testing services, we cannot be responsible for soil engineering related construction errors or omissions. This summary should be used in conjunction with the entire report for design purposes. It should be recognized that details were not included or fully developed in this section, and the report must be read in its entirety for a comprehensive understanding of the items contained herein. The section titled **8.0 EVALUATION LIMITATIONS** should be read for an understanding of the report limitations.

Geotechnical Evaluation
P1FCU Ponderay Branch
Parcel #RPP00000101810A
Ponderay, Idaho

1.0 PROJECT DESCRIPTION

Specific construction and grading details were not available at the time this report was prepared. We anticipate the building will be a single-story building consisting of masonry construction with slab-on-grade floors. We have assumed cut and fill required to grade most of the site will be three feet or less. The site resides just east of Sand Creek. Historical photographs from Google Earth indicate a house previously existed on the site, so there is a possibility of encountering undocumented fill. We have assumed continuous footing loads of approximately 2 to 3 kips per linear foot with column loads up to 50 kips. An asphalt parking lot and miscellaneous landscaping improvements will also be constructed. **If the proposed design or loads vary from those stated, we should be notified to review our recommendations.**

2.0 EVALUATION PROCEDURES

To complete this evaluation, we reviewed soil and geologic literature for the project area. We evaluated the subsurface conditions at the site by excavating seven test pits throughout the project site. The approximate locations of the test pits are shown on Figure A-2, Site and Exploration Plan included in Appendix A. Information obtained from the field evaluation, laboratory testing, and geotechnical analyses was utilized to develop the recommendations presented in this report.

3.0 SITE CONDITIONS

The project site is comprised of a single undeveloped parcel, approximately 2.09 acres in total size. The site is predominantly fairly flat and level, with an existing small structure in the southwest corner, and a well in the northwest corner of the site. The west edge of the property slopes down to Sand Creek. The ground cover consists of grass and dirt. The property is bordered by US Highway 95 to the east, existing commercial property to the north, vacant wooded parcel to the west, and a City of Ponderay building to the south.

3.1 General Geologic Conditions

The preliminary Geologic Map of the Sandpoint 30 x 60 Minute Quadrangle, Idaho and Montana, and the Idaho part of the Chewelah 30 x 60 Minute Quadrangle indicated the mapped geology on the site is Quaternary Glaciolacustrine (Qgl)

deposits. The deposits are described as massively to finely laminated clay, silt, and sand deposited in ice marginal and post glacial lakes in the Purcell Trench. Exhibits well developed rhythmites and beds of sand and silt. This unit includes deposits in the Selle Lowlands and discontinuous terraces in tributary valleys at about 2400 feet and as high as 2600 feet. They are described as mostly well-sorted and finely laminated. Beds are contorted and loading structures are common. These deposits are overlain by glaciofluvial outwash deposits on terraces and in tributary valleys. The soils are silt loam and silty sandy loams of the Mission-Cabinet-Odenson series. The soils classify as CL to ML and SM using the Unified Soil Classification System (USCS) and vary in thickness from tens of meters to hundreds of meters in the Selle Lowlands.

3.2 General Soil Conditions

The USDA Natural Resources Conservation Service (NRCS) has mapped the soils on and around the property as Misson silt loam. The Mission soils consist of a rather poorly drained mantle of material derived from volcanic ash and loess underlain by glaciolacustrine deposits. This soil formed in a glacial lake environment and is typically soft or loose. The permeability is described as very low. Runoff is typically slow and the hazard of water erosion is slight.

4.0 SUBSURFACE CONDITIONS

Seven test pits were excavated at the site on December 13, 2019. The test pits were excavated with a mini-excavator with a standard bucket. Representative soil samples were collected. The approximate locations of the test pits are shown on Figure A-2, Site and Exploration Plan in Appendix A. The soil conditions observed in the test pits were visually described and classified in general accordance with ASTM D2487 and D2488 and the subsurface profiles were logged.

Detailed descriptions of the soil observed in the test pits are presented on the Test pit Logs in Appendix B of this report. The descriptive soil terms used on the test pit logs and in this report can be referenced by the Unified Soil Classification System (USCS). A summary of the USCS is included in Appendix B. The subsurface conditions may vary between test pit locations. Such changes in conditions would not be apparent until construction.

4.1 Subsurface Soil Conditions

The near surface geologic profile appears to consist of topsoil or undocumented fill overlying silt overlying lean clay. Descriptions of the soil types observed follow:

Topsoil – The topsoil layer was encountered at the ground surface. This unit consists of sandy silt with organics. The color appeared dark brown, and the unit was moist. This unit is unsuitable for re-use as structural fill due to the organic content. The topsoil depth extended down 1 to 1½ feet in the test pits.

Undocumented fill – Test pits TP-2 and TP-7 encountered undocumented fill at the ground surface. This unit consists of sandy silt with organics, and minor gravels. The color appeared dark brown to gray, and the unit was moist. This unit is unsuitable for re-use as structural fill due to the organic content. The topsoil depth extended down 1½ feet in test pit TP-2 and 2 feet in test pit TP-7.

Silt – Below the topsoil layer, we encountered native silt. The soil layer was observed to be brown to gray with depth, moist, and stiff. This unit extended to approximately 8 to 9 feet below existing grade in the test pits.

Lean clay – Lean clay with sand lenses was encountered below the silt. The unit was brown, moist to wet with depth, and medium stiff to stiff. The lean clay extended to the maximum extend of our test pit exploration of approximately 10 feet.

4.2 Groundwater Conditions

We did not encounter groundwater in the test pits. We did not observe surface water on the property during our evaluation. Changes in precipitation, irrigation, construction, or other factors may impact depth to groundwater and the surface water flow on the property and therefore, conditions may be different during construction. Seasonally it is common to encounter perched groundwater between sand and clay contacts.

5.0 LABORATORY TESTING

Laboratory testing was performed to supplement field classifications and to assess some of the soil engineering parameters. The laboratory testing included a particle size distribution/gradation test (ASTM D6913) and Liquid and Plastic Limit Tests (ASTM D4318). The laboratory test results are in Appendix C of this report and presented on the test pit logs in Appendix B. The laboratory testing was performed by ALLWEST.

6.0 CONCLUSIONS AND RECOMMENDATIONS

The following conclusions and recommendations are presented to assist the planning and design of the proposed development. The recommendations are based on our understanding of the proposed construction, the conditions observed in the test pits, and engineering analyses. **If the construction scope changes, or if conditions**

are encountered during construction which are different than those described in this report, we should be notified so we can review our recommendations and provide revisions, if necessary.

6.1 Site Preparation

Clearing and Stripping: Once temporary erosion and sediment control (TESC) measures are installed, we expect site preparation to continue with clearing and grubbing brush and stripping of organic-rich topsoil and undocumented fill. Based on our explorations, the stripping depth for topsoil removal is estimated to be approximately 1 to 1½ feet. Clearing and stripping debris should be wasted off site or used for topsoil in landscape areas.

Over-Excavation: Once clearing and stripping are complete, earthwork should continue with over-excavation of the undocumented fill underlying structural elements such as the building, concrete flatworks, and asphalt pavements to approximately 2 feet below the existing ground surface and replacement with structural fill.

Subgrade Preparation: Once over-excavation is complete, all areas that are at design subgrade elevation or areas that will receive new structural fill as recommended in the Subgrade Stabilization section (6.2) of this report.

In the event the exposed subgrade becomes unstable, yielding, or unable to be compacted due to high moisture conditions or construction traffic, we recommend that the materials be removed to a sufficient depth in order to develop stable subgrade soils that can be compacted to the minimum recommended levels. The severity of construction problems will be dependent, in part, on the precautions that are taken by the contractor to protect the subgrade soils.

6.2 SUBGRADE STABILIZATION

We recommend the subgrade be stabilized using geosynthetic reinforcement in conjunction with imported structural fill. The required thicknesses of structural fill (used in conjunction with geosynthetic reinforcement) will be dependent on the construction traffic loading which is unknown at this time. Revisions to the stabilization method may be necessary depending on the anticipated construction traffic.

Geosynthetic reinforcement should consist of a stiff polypropylene bi-axial or tri-axial geogrid such as Tensar BX1200, TX140 or equivalent. Alternative geotextiles should be approved by the geotechnical engineer prior to use on site. The following recommendations are provided for subgrade stabilization using geosynthetic reinforcement.

- Geosynthetic reinforcement materials should be placed on a properly prepared subgrade with smooth surface. Loose and disturbed soil should be removed prior to placement of geosynthetic reinforcement materials.

- A minimum 4-ounce, non-woven filter fabric should be placed on the properly prepared subgrade. The geosynthetic reinforcement should be placed directly on top of the filter fabric. The filter fabric and geosynthetic reinforcement should be unrolled in the primary direction of fill placement and should be over-lapped at least three (3) feet.

- The geosynthetic materials should be pulled taut to remove slack. If the material does not remain taut during fill placement its effectiveness will be reduced.

- Construction equipment should not be operated directly on the geosynthetic materials. Fill should be placed from outside the excavation to create a pad to operate equipment on. We recommend a minimum of 12 inches of structural fill be placed over the geosynthetic reinforcement before operating construction equipment on it. Low pressure, track-mounted equipment should be used to place fill over the geosynthetic reinforcement.

- Fill placed directly over the geosynthetic reinforcement should be properly moisture conditioned prior to placement and should meet the following gradation.

Sieve Size	% Passing
1 ½ inch	100
¾ inch	50 - 100
#4	25 - 50
#40	10 - 20
#100	5 - 15
#200	less than 10

- The fill material should be properly compacted. Care should be taken with the use of vibratory compaction equipment. Vibration should be discontinued if it reduces the subgrade stability.

An ALLWEST representative should be on-site during subgrade stabilization activities to verify our recommendations are followed and to provide additional recommendations as appropriate.

6.3 Excavation

Based on the conditions observed in our explorations, we anticipate excavation of the on-site soil can be achieved with typical excavation equipment. Temporary excavation slope stability is a function of many factors, including:

- The presence and abundance of groundwater;
- The type and density of the various soil strata;
- The depth of cut;
- Surcharge loadings adjacent to the excavation; and
- The length of time the excavation remains open.

It is exceedingly difficult under the variable circumstances to pre-establish a safe and "maintenance-free" temporary cut slope angle. Therefore, it is the responsibility of the contractor to maintain safe temporary slope configurations since the contractor is continuously at the job site, able to observe the nature and condition of the cut slopes, and able to monitor the subsurface materials and groundwater conditions encountered. Unsupported vertical slopes or cuts deeper than 4 feet are not recommended if worker access is necessary. The cuts should be adequately sloped, shored, or supported to prevent injury to personnel from local sloughing and spalling. The excavation should conform to applicable Federal, State, and Local regulations. Regarding trench wall support, the site soil is considered Type C soil according to OSHA guidelines and therefore should not exceed a 1.5H:1V temporary slope.

We recommend that all permanent cut or fill slopes constructed in native soils be designed at a 2H:1V (Horizontal:Vertical) inclination or flatter. All permanent cut and fill slopes should be adequately protected from erosion both temporarily and permanently.

6.4 Materials

The on-site native soil is considered moisture sensitive and frost susceptible based on the percent of fine grains (passing the #200 sieve). Therefore, this material is unsuitable for re-use as structural fill.

Import materials should be well-graded granular soil, free of organics, debris, and other deleterious material and meet the following recommendations. Import materials should be approved by the Geotechnical Engineer prior to delivery to the site.

Fill Type	Recommendations
Structural Fill	Maximum size ≤ 3 inches; Retained on ¾-inch sieve <30% Passing No. 200 Sieve ≤ 10%; Non-plastic
Utility Trench Backfill	Maximum size ≤ 2 inches; Passing No. 200 Sieve ≤ 15%; Non-plastic

6.5 Fill Placement and Compaction

Fill should be placed in lift thicknesses which are appropriate for the compaction equipment used. Typically, eight-inch loose lifts are appropriate for typical rubber tire and steel drum compaction equipment. Lift thicknesses should be reduced to four inches for hand operated compaction equipment. Fill should be moisture conditioned to within two percentage points of the optimum moisture content prior to placement to facilitate compaction. Structural fill and utility trench backfill should be compacted to a minimum of 95 percent of the maximum dry density established by ASTM D1557 (modified Proctor).

6.6 Wet Weather Construction

Due to the climatic effects in this region during late fall, winter, and spring (generally wet conditions), we recommend construction (especially site grading) take place during the summer and early fall season, if possible. If construction occurs during or immediately after excessive precipitation, it may be necessary to over-excavate and replace wet subgrade soil which might otherwise be suitable.

We recommend earthwork for this site be scheduled for the drier seasons of the year. If construction is undertaken in wet periods of the year, it will be important to slope the ground surface to provide drainage away from construction.

6.7 Cold Weather Construction

Foundations should be embedded adequately to protect against frost action as recommended in the Foundation Recommendations section of this report. We recommend removal of frost susceptible soils (soil with fines contents greater than 10 percent) within the frost-depth zone below concrete flatwork (sidewalks, patios, etc.) to reduce the potential detrimental effects of frost heave.

If site grading and construction are anticipated during cold weather, we recommend good winter construction practices be observed. Snow and ice should be removed from excavated and fill areas prior to additional earthwork or construction. Footings, floor slabs or structural portions of the construction should not be placed on frozen

ground; nor should the supporting soils for buildings be permitted to freeze during or after construction. Frozen soils should not be used as backfill or fill.

6.8 Foundation Recommendations

The proposed building may be supported on conventional spread footings bearing on a minimum of 24 inches of structural fill if prepared as recommended in the Site Preparation and Subgrade Stabilization sections of this report. The following recommendations are provided for foundations based on the subsurface conditions observed and the stated assumptions:

- Footings bearing on a minimum of 24 inches of properly prepared structural fill may be designed for an allowable bearing pressure of 2,500 pounds per square foot (psf). The allowable bearing pressure value may be increased by one-third to account for transient loads such as wind and seismic.

- Unless specified by project engineer or governing codes, continuous footings should be a minimum of 18 inches in width and column footings should be a minimum of 24 inches in width.

- An ultimate value for coefficient of friction between cast-in-place concrete and gravel of 0.4 may be used for design.

- Foundation bearing surfaces should be free of loose soil and debris.

- Footings should be embedded at least 24 inches below finished exterior ground surface to help protect against frost action.

- We recommend backfill placed adjacent to foundation walls be brought up uniformly on both sides of the foundation walls to reduce displacement of the foundation walls.

- If the previous recommendations are implemented, it is our opinion the total settlement will be less than one inch and differential settlement will be less than ½-inch in 30-feet.

6.9 Concrete Slabs-on-Grade

Concrete slabs-on-grade should be underlain by at least 6 inches of crushed base course. The crushed base course below the slabs should be compacted to at least 95 percent of the maximum dry density established by modified Proctor (ASTM D1557). The slab subgrade should be prepared as previously recommended which includes over-excavation of the native soil to 4 feet and replacement with structural fill.

From a geotechnical perspective, a vapor barrier is not considered necessary beneath the slab-on-grade floor unless moisture sensitive floor coverings and/or adhesives are used. If a vapor barrier is used, we recommend using a 15-mil, puncture-resistant proprietary product such as Stego Wrap, or an approved equivalent that is classified as a Class A vapor barrier in accordance with ASTM E 1745. Overlap lengths and the appropriate tape used to seal the laps should be in accordance with the vapor retarder manufacturer's recommendations. To avoid puncturing of the vapor barrier, a thin sand layer placed over the crushed gravel is recommended. When conditions warrant the use of a vapor retarder, the slab designer and slab contractor should refer to ACI 302 and ACI 360 for procedures and cautions regarding the use and placement of a vapor retarder/barrier.

6.10 Lateral Earth Pressures

Below-grade walls should be designed to resist lateral earth pressures. The lateral earth pressures for approved structural fill derived from on-site soil should be calculated using the following equivalent fluid pressures:

Condition	Equivalent Fluid Pressure Structural Fill (pcf)
At-rest	55
Active	35
Passive	350

The above values are for level backfill only and do not account for hydrostatic forces. Walls should be provided with adequate drainage so hydrostatic forces do not adversely affect the walls. We recommend placement of gravel behind walls and/or weep holes to assist with drainage and reduce the potential for the buildup of hydrostatic pressures. Walls that are braced in a manner that does not allow any rotational movement (rigid) (e.g. basement walls) should be designed using the given "at-rest" equivalent fluid pressure. The active and at-rest pressures should be increased by an equivalent fluid weight of 10 pounds per cubic foot (pcf) and the passive pressure should be reduced by 10 pcf for seismic design. The dynamic component of the active pressure acts at a height of approximately 0.6 times the height of the wall.

6.11 Seismicity

We anticipate the 2015 International Building Code (IBC) will be used as the basis for design of the proposed structures. The soil at the site can be characterized as Site Class D for seismic design.

The following seismic parameters were calculated using USGS U.S. Seismic Design Maps for use with the 2015 IBC. The latitude and longitude for the site were used to specify the location of the subject property. The following Site Class D seismic parameters may be used for design.

Latitude (degrees)	Longitude (degrees)	Spectral Accelerations		Site Coefficients	
		S_s	S_1	F_a	F_v
48.3052	-116.5470	0.339g	0.112g	1.529	2.353

6.12 Flexible (Hot Mix Asphalt) Pavement

SUBGRADE

We recommend that the moisture content and density of the top 12 inches of the subgrade be evaluated and that the pavement subgrades be proof-rolled within two days prior to commencement of actual paving operations. Areas not in compliance with the required ranges of moisture or density should be moisture conditioned and recompacted. Particular attention should be paid to high traffic areas that were rutted and disturbed earlier and to areas where backfilled trenches are located. Areas where unsuitable conditions are located should be repaired by removing and replacing the materials with properly compacted structural fills. If a significant precipitation event occurs after the evaluation or if the surface becomes disturbed, the subgrade should be reviewed by qualified personnel immediately prior to paving. The subgrade should be in its finished form at the time of the final review.

DESIGN PARAMETERS

Design Parameter	Value
Assumed: Subgrade California Bearing Ratio (CBR)	3%
Estimated: Equivalent Single Axle Load (ESAL)	75,000
Assumed: Pavement reliability	85%
Assumed: Pavement design life	20-year

PAVEMENT SECTION

MINIMUM LIGHT-DUTY PAVEMENT SECTION (CARS ONLY)	
Layer	**Thickness (inches)**
Asphalt Surface	2.5
Crushed Aggregate Base	4.0
Compacted Structural Fill Subgrade	12
Total Pavement Section	6.5

MINIMUM HEAVY-DUTY PAVEMENT SECTION	
Layer	**Thickness (inches)**
Asphalt Surface Course	3.0
Crushed Aggregate Base	4.0
Compacted Structural Fill Subgrade	12
Total Pavement Section	7.0

We also recommend a concrete apron in areas where you expect frequent truck loading, unloading, turning, starting and stopping such as around loading docks and dumpster pads.

MATERIALS

We recommend specifying crushed aggregate base meeting the requirements of the Idaho Standards for Public Works Construction (ISPWC) Section 802, Type I for crushed aggregate for base gradations. We recommend the asphalt concrete pavement meet the requirements of ITD Standard Specification 405 for plant mix asphalt concrete pavements.

We recommend the crushed aggregate base be compacted to a minimum of 95 percent of the maximum dry density established by ASTM D1557 (modified Proctor). We recommend the asphaltic concrete surface be compacted to minimum of 92 percent of the Rice density.

DRAINAGE

Pavements should be sloped to provide rapid drainage of surface water. Water allowed to pond on or adjacent to the pavements could saturate the subgrade and contribute to premature pavement deterioration. In addition, the pavement subgrade

should be graded to provide positive drainage within the crushed aggregate base section.

We recommend drainage be included at the bottom of the crushed aggregate base layer at the storm structures to aid in removing water that may enter this layer. Drainage could consist of small diameter weep holes excavated around the perimeter of the storm structures. The weep holes should be excavated at the elevation of the crushed aggregate base and soil interface. The excavation should be covered with crushed aggregate which is encompassed in Mirafi 140NL or approved equivalent which will aid in reducing fines from entering the storm system.

MAINTENANCE

The pavement sections provided in this report represent minimum recommended thicknesses. Therefore preventive maintenance should be planned and provided for through an on-going pavement management program. Preventive maintenance activities are intended to slow the rate of pavement deterioration, and to preserve the pavement investment. Preventive maintenance consists of both localized maintenance (e.g., crack and joint sealing and patching) and global maintenance (e.g., surface sealing). Preventive maintenance is usually the first priority when implementing a planned pavement maintenance program and provides the highest return on investment for pavements. Prior to implementing any maintenance, additional engineering observation is recommended to determine the type and extent of preventive maintenance. Even with periodic maintenance, some movements and related cracking may still occur and repairs may be required.

6.13 Stormwater and Drainage

We recommend a permanent foundation drainage system be designed and constructed around the perimeter of the structure. The drainage system should consist of a four-inch diameter, Schedule 40 or ADS, perforated pipe surrounded with a free draining aggregate. The pipe should be located at the lowest elevation of the footing trench excavation such that gravity drainage may be achieved. Water collected in the drains should be discharged down-gradient of the home.

We recommend the grading plan include slopes such that storm water run-off is directed away from the building and pavement areas to a storm water management system. We recommend ground surface adjacent to foundations be sloped a minimum of five percent within ten feet of the building. If the adjoining ground surface consists of hardscapes it may be sloped a minimum of two percent in the first ten feet. Water should not be allowed to infiltrate or pond adjacent to the foundations.

Based on the subsurface conditions encountered in our explorations, the site is not suitable for stormwater infiltration.

7.0 ADDITIONAL RECOMMENDED SERVICES

We recommend ALLWEST be retained to provide construction materials testing and observation to verify the soil and geologic conditions and the report recommendations are incorporated into the actual construction. The design engineer of record should determine applicable testing and special inspection requirements in accordance with the governing code documents. If we are not retained to provide required construction observation and materials testing services, we cannot be responsible for soil engineering related construction errors or omissions.

8.0 EVALUATION LIMITATIONS

This report has been prepared to assist the planning and design for P1FCU Ponderay Branch located at Parcel #RPP00000101810A in Ponderay, Idaho. Reliance by any other party is prohibited without the written authorization of ALLWEST. Our services consist of professional opinions and conclusions made in accordance with generally accepted geotechnical engineering principles and practices in the local area at the time this report was prepared. This acknowledgement is in lieu of all warranties, express or implied.

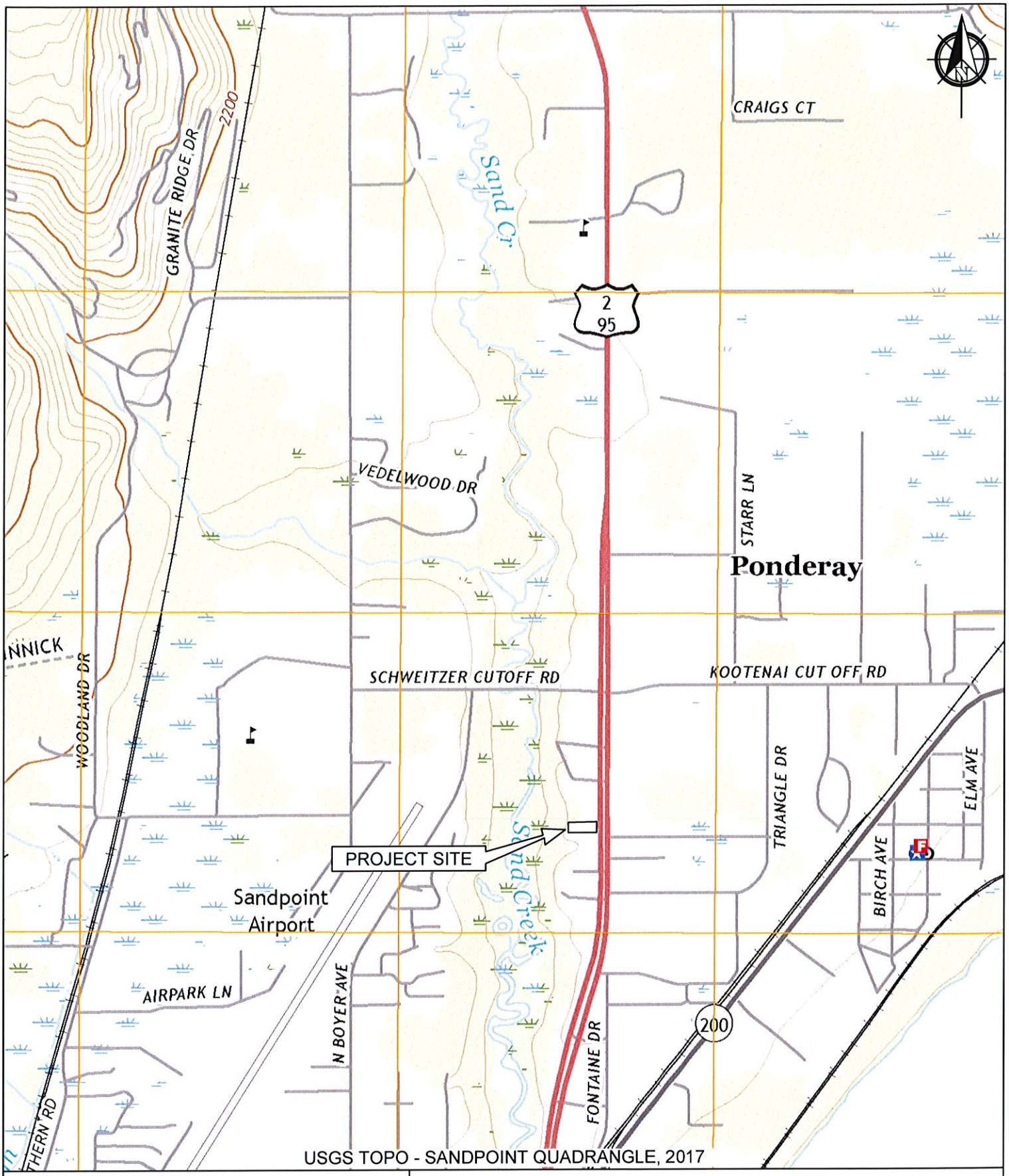
The following appendices complete this report:

Appendix A – Vicinity Map, Site and Exploration Plan
Appendix B – Test pit Logs, Unified Soil Classification System
Appendix C – Laboratory Test Results

Appendix A

Vicinity Map
Site and Exploration Plan

ALLWEST



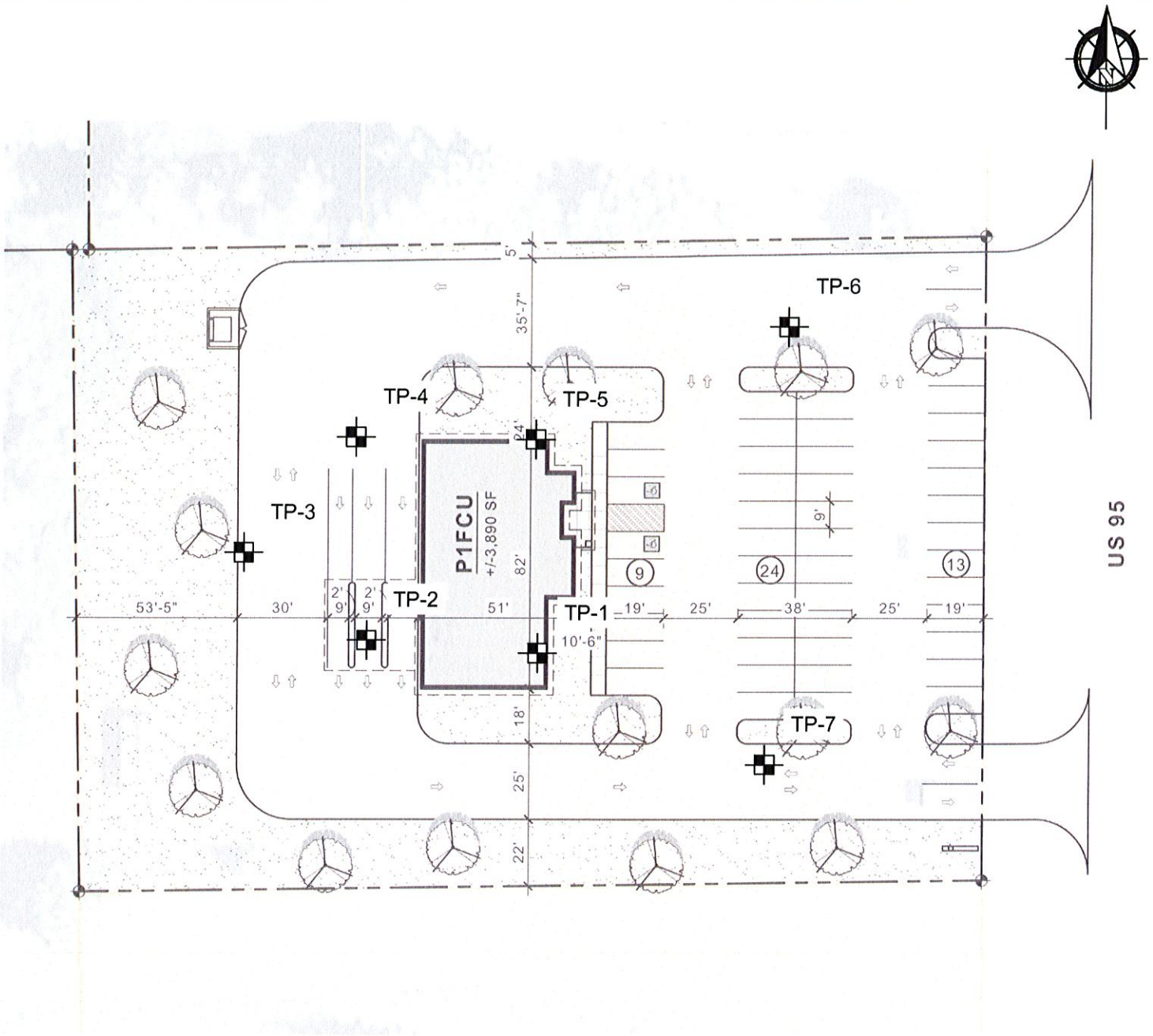
USGS TOPO - SANDPOINT QUADRANGLE, 2017

ALLWEST

690 W Capstone Court
Hayden, Idaho 83835
(208) 762-4721
www.allwesttesting.com

VICINITY MAP

PROJECT:	119-544G P1FCU PONDERAY BRANCH		
LOCATION:	PARCEL #RPP00000101810A, PONDERAY, IDAHO		
CLIENT NAME:	KENASTON CORPERATION		
DATE:	DECEMBER, 2019	SCALE:	NOT TO SCALE



50 0 25 50
SCALE IN FEET

LEGEND:

TP-1 TEST PIT NUMBER AND APPROXIMATE LOCATION

BASEMAP SOURCE: CONCEPTUAL SITE PLAN PREPARED BY MERCIER ARCHITECTURE & PLANNING, DATED DECEMBER 04, 2019.

ALLWEST
690 W Capstone Court
Hayden, Idaho 83835
(208) 762-4721
www.allwesttesting.com

FIGURE A-2: SITE AND EXPLORATION PLAN			
PROJECT:	119-544G P1FCU PONDERAY BRANCH		
LOCATION:	PARCEL #RPP00000101810A, PONDERAY, IDAHO		
CLIENT NAME:	KENASTON CORPERATION		
DATE:	DECEMBER, 2019	SCALE:	AS SHOWN

Appendix B

Test pit Logs
Unified Soil Classification System

ALLWEST

ALLWEST TESTING & ENGINEERING, INC.
HAYDEN, IDAHO
GEOTECHNICAL SECTION

TEST PIT LOG

DATE STARTED: 12/13/2019
DATE FINISHED: 12/13/2019
OPERATOR: Rick Marcus
COMPANY:
LOGGER: Kenny Rukavina
WEATHER: Cloudy

TEST PIT TP-1

EXCAVATOR: Mini Excavator
EXCAVATION METHOD: Soil Excavation Bucket

PROJECT: 119-544G P1FCU Ponderay Branch

NOTES:

TOTAL DEPTH: 10'

DEPTH (ft)	USCS	DESCRIPTION	GRAPHIC LOG	SAMPLE #	NOTES
0	ML	Topsoil: Sandy SILT with organics, dark brown, moist.			
1	ML	SILT, light brown, moist, stiff.			
7	ML	SILT, gray, moist, medium stiff.			
8	CL	Lean CLAY with sand lenses, brown, moist, medium stiff to stiff.			
10		Test pit TP-1 terminated at 10 feet. No groundwater observed. No caving observed.			
12					

WATER LEVELS
▽ WHILE EXCAVATING
▼ AT COMPLETION
▼ AFTER EXCAVATING

ALLWEST TESTING & ENGINEERING, INC.
HAYDEN, IDAHO
GEOTECHNICAL SECTION

TEST PIT LOG

DATE STARTED: 12/13/2019
DATE FINISHED: 12/13/2019
OPERATOR: Rick Marcus
COMPANY:
LOGGER: Kenny Rukavina
WEATHER: Cloudy

TEST PIT TP-2

EXCAVATOR: Mini Excavator
EXCAVATION METHOD: Soil Excavation Bucket

PROJECT: 119-544G P1FCU Ponderay Branch

NOTES:

TOTAL DEPTH: 10'

DEPTH (ft)	USCS	DESCRIPTION	GRAPHIC LOG	SAMPLE #	NOTES
0	FILL	Undocumented Fill: Sandy SILT with organics and small inclusions of gravel and sand, dark brown to gray, moist.			
1					
2	ML	SILT, light brown, moist, stiff.			
3					Plasticity Limits at 3 feet. LL = 31 PI = 8
4					
5					
6					
7	ML	SILT, gray, moist, medium stiff.			
8					
9	CL	Lean CLAY with sand lenses, brown, moist, medium stiff to stiff.			
10		Test pit TP-2 terminated at 10 feet. No groundwater observed. No caving observed.			
11					
12					

WATER LEVELS
▽ WHILE EXCAVATING
▼ AT COMPLETION
▼ AFTER EXCAVATING

ALLWEST TESTING & ENGINEERING, INC.
HAYDEN, IDAHO
GEOTECHNICAL SECTION

TEST PIT LOG

DATE STARTED: 12/13/2019
DATE FINISHED: 12/13/2019
OPERATOR: Rick Marcus
COMPANY:
LOGGER: Kenny Rukavina
WEATHER: Cloudy

TEST PIT TP-3

EXCAVATOR: Mini Excavator
EXCAVATION METHOD: Soil Excavation Bucket

PROJECT: 119-544G P1FCU Ponderay Branch

NOTES:

TOTAL DEPTH: 10'

DEPTH (ft)	USCS	DESCRIPTION	GRAPHIC LOG	SAMPLE #	NOTES
0	ML	Topsoil: Sandy SILT with organics, dark brown, moist.			
1					
2	ML	SILT with sand lenses, light brown, moist, stiff.			
3					
4					
5					
6					
7	ML	SILT, gray, moist, medium stiff.			
8					
9	CL	Lean CLAY with sand lenses, brown, moist, medium stiff to stiff.			
10		Test pit TP-3 terminated at 10 feet. No groundwater observed. No caving observed.			
11					
12					

WATER LEVELS
▽ WHILE EXCAVATING
▼ AT COMPLETION
▼ AFTER EXCAVATING

ALLWEST TESTING & ENGINEERING, INC.
HAYDEN, IDAHO
GEOTECHNICAL SECTION

TEST PIT LOG

DATE STARTED: 12/13/2019
DATE FINISHED: 12/13/2019
OPERATOR: Rick Marcus
COMPANY:
LOGGER: Kenny Rukavina
WEATHER: Cloudy

TEST PIT TP-4

EXCAVATOR: Mini Excavator
EXCAVATION METHOD: Soil Excavation Bucket

PROJECT: 119-544G P1FCU Ponderay Branch

NOTES:

TOTAL DEPTH: 10'

DEPTH (ft)	USCS	DESCRIPTION	GRAPHIC LOG	SAMPLE #	NOTES
0	ML	Topsoil: Sandy SILT with organics, dark brown, moist.			
1					
2	ML	SILT with sand lenses, light brown, moist, stiff. Contained remnant tree roots.			
3					
4					
5					
6	ML	SILT, gray, moist, medium stiff.			
7					
8	CL	Lean CLAY with sand lenses, brown, moist, medium stiff to stiff.			
9					
10		Test pit TP-4 terminated at 10 feet. No groundwater observed. No caving observed.			
11					
12					

WATER LEVELS
▽ WHILE EXCAVATING
▼ AT COMPLETION
▼ AFTER EXCAVATING

ALLWEST TESTING & ENGINEERING, INC.
HAYDEN, IDAHO
GEOTECHNICAL SECTION

TEST PIT LOG

DATE STARTED: 12/13/2019
DATE FINISHED: 12/13/2019
OPERATOR: Rick Marcus
COMPANY:
LOGGER: Kenny Rukavina
WEATHER: Cloudy

TEST PIT TP-5

EXCAVATOR: Mini Excavator
EXCAVATION METHOD: Soil Excavation Bucket

PROJECT: 119-544G P1FCU Ponderay Branch

NOTES:

TOTAL DEPTH: 9'

DEPTH (ft)	USCS	DESCRIPTION	GRAPHIC LOG	SAMPLE #	NOTES
0	ML	Topsoil: Sandy SILT with organics, dark brown, moist.			
1	ML	SILT with sand lenses, light brown, moist, stiff. Contained remnant tree roots.			
6	ML	SILT, gray, moist, medium stiff.			Grain Size Distribution at 7 feet: Sand = 10% Silt/clay = 90%
8	CL	Lean CLAY with sand lenses, brown, moist, medium stiff to stiff.			
9		Test pit TP-5 terminated at 9 feet. No groundwater observed. No caving observed.			

WATER LEVELS
▽ WHILE EXCAVATING
▼ AT COMPLETION
▼ AFTER EXCAVATING

ALLWEST TESTING & ENGINEERING, INC.
HAYDEN, IDAHO
GEOTECHNICAL SECTION

TEST PIT LOG

DATE STARTED: 12/13/2019
DATE FINISHED: 12/13/2019
OPERATOR: Rick Marcus
COMPANY:
LOGGER: Kenny Rukavina
WEATHER: Cloudy

TEST PIT TP-6

EXCAVATOR: Mini Excavator
EXCAVATION METHOD: Soil Excavation Bucket

PROJECT: 119-544G P1FCU Ponderay Branch

NOTES:

TOTAL DEPTH: 10'

DEPTH (ft)	USCS	DESCRIPTION	GRAPHIC LOG	SAMPLE #	NOTES
0	ML	Topsoil: Sandy SILT with organics, dark brown, moist.			
1					
2	ML	SILT with sand lenses, light brown, moist, stiff.			
3					
4					
5					
6	ML	SILT, gray, moist, medium stiff.			
7					
8	CL	Lean CLAY with sand lenses, brown, moist, medium stiff to stiff.			
9					
10		Test pit TP-6 terminated at 10 feet. No groundwater observed. No caving observed.			
11					
12					

WATER LEVELS
▽ WHILE EXCAVATING
▼ AT COMPLETION
▼ AFTER EXCAVATING

ALLWEST TESTING & ENGINEERING, INC.
HAYDEN, IDAHO
GEOTECHNICAL SECTION

TEST PIT LOG

DATE STARTED: 12/13/2019
DATE FINISHED: 12/13/2019
OPERATOR: Rick Marcus
COMPANY:
LOGGER: Kenny Rukavina
WEATHER: Cloudy

TEST PIT TP-7

EXCAVATOR: Mini Excavator
EXCAVATION METHOD: Soil Excavation Bucket

PROJECT: 119-544G P1FCU Ponderay Branch

NOTES:

TOTAL DEPTH: 10'

DEPTH (ft)	USCS	DESCRIPTION	GRAPHIC LOG	SAMPLE #	NOTES
0	FILL	Undocumented Fill: Interbedded layers of topsoil and silt, dark brown to light brown, moist.			
2	ML	SILT with sand lenses, light brown, moist, stiff.			
6	ML	SILT, gray, moist, medium stiff.			
8	CL	Lean CLAY with sand lenses, brown, moist, medium stiff to stiff.			Plasticity limits at 8.5 feet: LL = 27 PI = 5
10		Test pit TP-7 terminated at 10 feet. No groundwater observed. No caving observed.			
12					

WATER LEVELS
▽ WHILE EXCAVATING
▼ AT COMPLETION
▼ AFTER EXCAVATING

Unified Soil Classification System

MAJOR DIVISIONS			SYMBOL	TYPICAL NAMES
COARSE GRAINED SOILS	GRAVELS	CLEAN GRAVELS	GW	Well-Graded Gravel, Gravel-Sand Mixtures.
			GP	Poorly-Graded Gravel, Gravel-Sand Mixtures.
		GRAVELS WITH FINES	GM	Silty Gravel, Gravel-Sand-Silt Mixtures.
			GC	Clayey Gravel, Gravel-Sand-Clay Mixtures.
	SANDS	CLEAN SANDS	SW	Well-Graded Sand, Gravelly Sand.
			SP	Poorly-Graded Sand, Gravelly Sand.
		SANDS WITH FINES	SM	Silty Sand, Sand-Silt Mixtures.
			SC	Clayey Sand, Sand-Clay Mixtures.
FINE GRAINED SOILS	SILTS AND CLAYS LIQUID LIMIT LESS THAN 50%		ML	Inorganic Silt, Silty or Clayey Fine Sand.
			CL	Inorganic Clay of Low to Medium Plasticity, Sandy or Silty Clay.
			OL	Organic Silt and Clay of Low Plasticity.
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50%		MH	Inorganic Silt, Elastic Silt, Micaceous Silt, Fine Sand or Silt.
			CH	Inorganic Clay of High Plasticity, Fat Clay.
			OH	Organic Clay of Medium to High Plasticity.
Highly Organic Soils			PT	Peat, Muck and Other Highly Organic Soils.

ALLWEST

Appendix C

Laboratory Test Results

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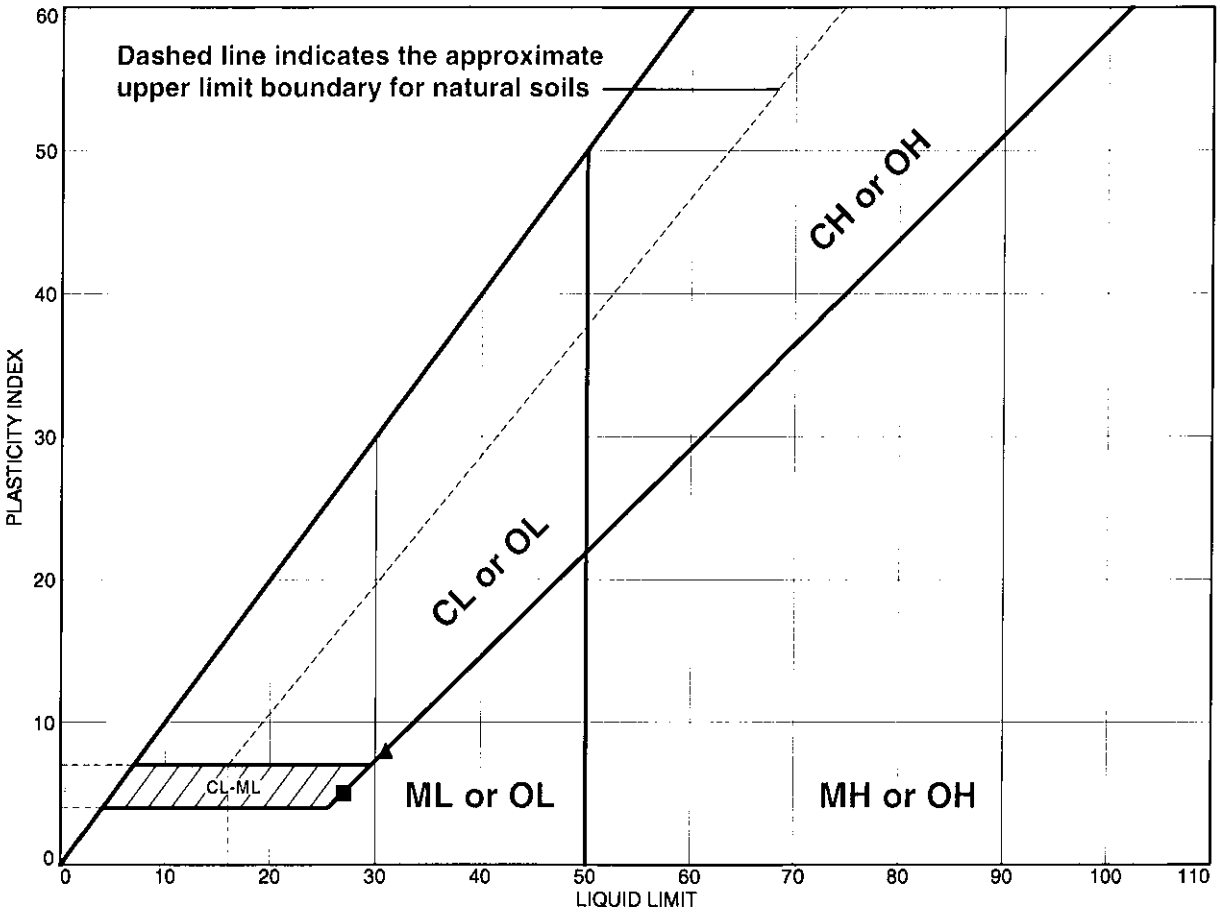
LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	silt	NV	NP	NP	100	90	ML
■	Lean Clay	27	22	5			
▲	Silt	31	23	8			

Project No. 119-544G **Client:** Kenaston Corp.

Project: P1FCU Ponderay Branch

●**Location:** TP-5 **Depth:** -7' **Sample Number:** S119-0967

■**Location:** TP-7 **Depth:** -8.5' **Sample Number:** S119-0966

▲**Location:** TP-2 **Depth:** -3' **Sample Number:** S119-0965

Remarks:

■Sampled By: K.Rukavina
Sample Date: 12/13/19

▲Sampled By: K.Rukavina
Sample Date: 12/13/19

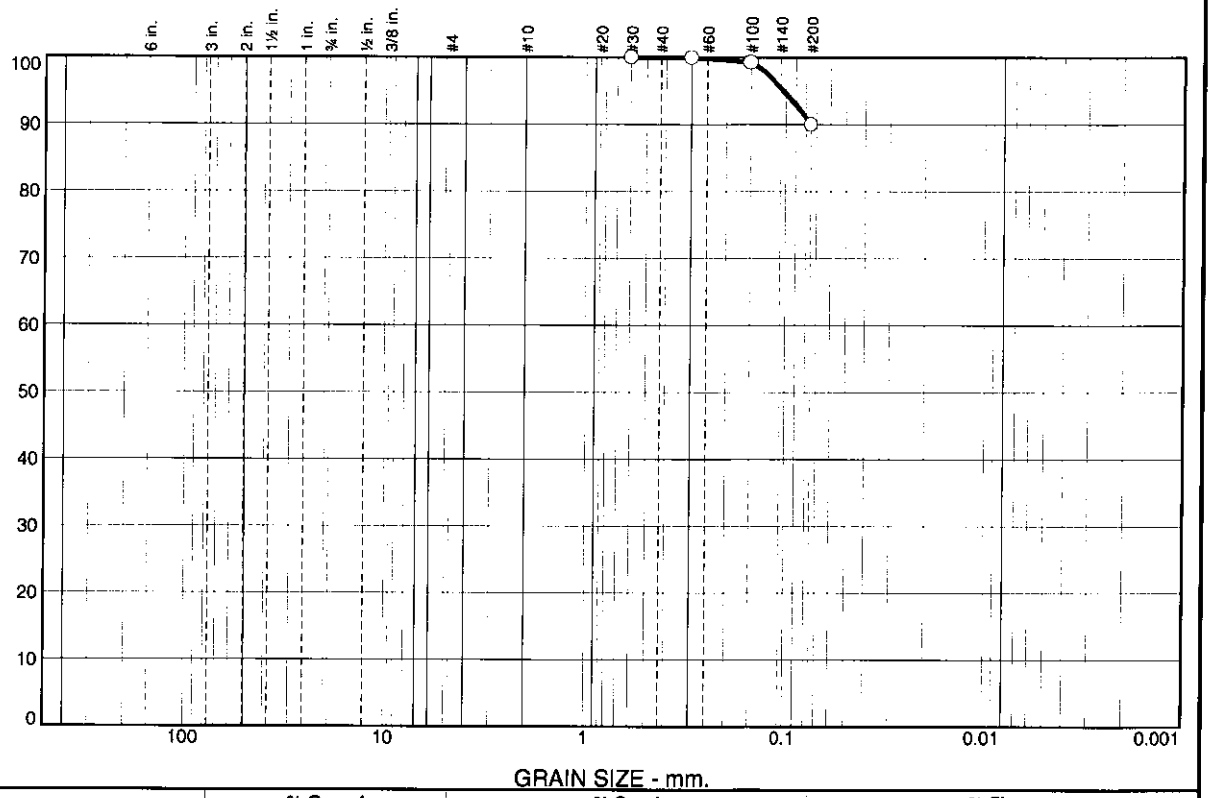
ALLWEST TESTING & ENGINEERING

Hayden, ID

Figure

Tested By: Noah White **Checked By:** Chris McKissen

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	0	0	0	0	10	90	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#30	100		
#50	100		
#100	99		
#200	90		

* (no specification provided)

Material Description

silt

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{90}= 0.0753 D_{85}= D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= ML AASHTO= A-4(0)

Remarks

Sampled By: K.Rukavina
Sample Date: 12/13/19

Location: TP-5
Sample Number: S119-0967 **Depth:** -7' **Date:** 12/16/19

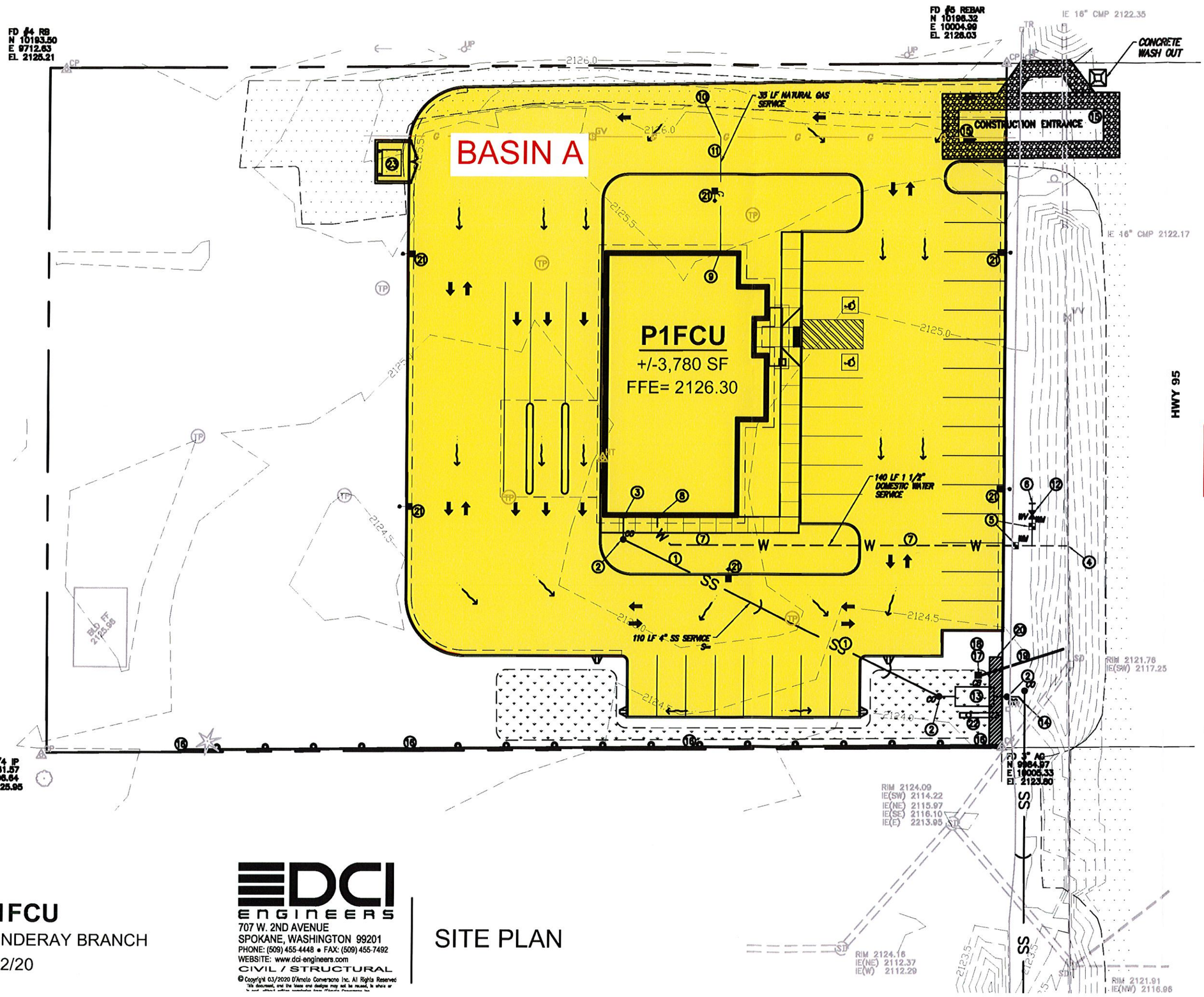
ALLWEST TESTING & ENGINEERING Hayden, ID

Client: Kenaston Corp.
Project: P1FCU Ponderay Branch

Project No: 119-544G **Figure** C-2

Tested By: Noah White **Checked By:** Chris McKissen

Appendix III



REFERENCE NOTES:

1. PROVIDE NEW 4" SDR35 PVC SANITARY SEWER SERVICE TO BUILDING. LENGTH AND SLOPE PER PLAN.
2. PROVIDE NEW CLEANOUT TO GRADE PER.
3. CONNECT TO BUILDING SANITARY SEWER SYSTEM. COORDINATE WITH PLUMBING PLANS.
4. TAP INTO EXISTING WATER LINE. FIELD VERIFY SIZE AND LOCATION PRIOR TO CONSTRUCTION.
5. PROVIDE NEW 1 1/2" WATER METER.
6. PROVIDE NEW 1 1/2" HDPE IRRIGATION SERVICE POINT OF CONNECTION. COORDINATE WITH LANDSCAPE PLAN.
7. PROVIDE NEW 1 1/2" HDPE DOMESTIC WATER SERVICE.
8. CONNECT TO BUILDING DOMESTIC WATER SYSTEM.
9. PROPOSED NATURAL GAS METER. COORDINATE WITH AVISTA UTILITIES AND PLUMBING PLANS.
10. TIE INTO EXISTING NATURAL GAS LINE. COORDINATE WITH AVISTA UTILITIES. FIELD VERIFY LOCATION PRIOR TO CONSTRUCTION.
11. PROPOSED NATURAL GAS SERVICE. COORDINATE WITH AVISTA UTILITIES.
12. PROPOSED NEW 1 1/2" DOUBLE CHECK VALVE ASSEMBLY.
13. PROVIDE NEW SEPTIC TANK WITH PUMP CHAMBER.
14. TIE INTO PROPOSED SANITARY SEWER FORCE MAIN EXTENSION.
15. PROVIDE NEW TEMPORARY CONSTRUCTION ENTRANCE AT ALL PROJECT ENTRY/EXIT POINTS.
16. PROVIDE NEW TEMPORARY SILT FENCE.
17. PROVIDE NEW INLET PROTECTION.
18. PROVIDE NEW TYPE 1 CATCH BASIN.
19. PROVIDE NEW 6" SDR35 PVC STORM DRAIN LINE. DAYLIGHT TO EXISTING SWALE.
20. PROVIDE PERMEABLE EARTHEN BERM.
21. PROPOSED SITE LIGHTING.
22. PROPOSED SIGN.
23. PROPOSED TRASH ENCLOSURE.

ENGINEER'S NOTES:

1. CONTRACTOR TO DETERMINE ELEVATION OF EXISTING SANITARY SEWE STUBS. CONTACT ENGINEER OF RECORD IF MINIMUM SLOPE OF 2% IS NOT POSSIBLE.
2. CONTRACTOR TO VERIFY LOCATION OF ALL EXISTING UTILITY STUBS PRIOR TO CONSTRUCTION.
3. BACKFLOW PREVENTION FOR DOMESTIC AND IRRIGATION SERVICES SHALL BE PER CITY OF ATHOL STANDARDS.
4. DUST CONTROL: AN APPROVED METHOD OF DUST CONTROL, MEETIN THE REQUIREMENTS OF LOCAL REGULATIONS, SHALL BE UTILIZED DURIN CONSTRUCTION OF THIS PROJECT.

SITE DATA:

TOTAL OPEN SPACE AREA – 31,506 SQ. FT.

TOTAL IMPERVIOUS SURFACE AREA – 31,364 SQ. FT.

TOTAL BUILDING FLOOR AREA – 3,780 SQ. FT.

TOTAL LOT SIZE – 62,870 SQ. FT.

N

GRAPHIC SCALE

30 0 15 30

(IN FEET)

1 inch = 30 ft.

P1FCU

PONDERAY BRANCH

3/02/20

DCI ENGINEERS
707 W. 2ND AVENUE
SPOKANE, WASHINGTON 99201
PHONE: (509) 455-4448 • FAX: (509) 455-7492
WEBSITE: www.dci-engineers.com
CIVIL / STRUCTURAL

SITE PLAN

Appendix IV

Stormwater Facilities and Detention Basin Design

PRE-DEVELOPED

Date: 2/19/2020
Job No.: 20042-0005
Arch. Name:
Project: P1FCU - Ponderay

Description and Assumptions:
City of Ponderay
(See Project Location Map)
Design Frequency: 25 year
Basin Area < 10 acres, therefore use Rational Formula

$Q = C \times I \times A$ where Q = Runoff in cubic feet per second
C = Runoff Coefficient
I = Rainfall Intensity in inches per hour
A = Contributing Area in acres

1 Determine Weighted Runoff Coefficient, C and Area, A

Sub-Area: Basin A

Total Drainage Area (A):	39,327	s.f.	0.9028	ac.
Total PGIS:	-	s.f.	-	ac.

Surface Type	Area (s.f.)	Area (ac.)	C	C*Area in acres
Parking/Driveway (PGIS)		-	0.90	-
Sidewalk (PGIS)		-	0.90	-
Landscaping	39,327	0.90	0.30	0.27
Roof (NPGIS)		-	0.90	-
	-	-		-
TOTAL	39,327	0.90		0.27

Weighted Runoff Coefficient (C) = (sum CA)/(sumA) = 0.30

2 Determine Rainfall Intensity, I

Duration (in minutes) is based on the Time of Concentration, Tc

Reach 1: Overland Flow

$$Tc = \frac{L}{K * \sqrt{(S)}}$$

where Tc = Time of Concentration in minutes of the longest route that the flow will take
L = Length in feet
K = Ground Cover Coefficient
S = Average Slope in ft/ft

L (ft)	K	S
281	420	0.007

Time of Concentration: 8.00 minutes

Tc shall not be less than 5 minutes, therefore:

Time of Concentration: 8.00 minutes

Intensity is calculated as: $I = \frac{m}{Tc^n}$

m	n
6.8	0.53

I = 2.26 in./hr.

3 Determine Peak Discharge, Q

Peak 25 yr Discharge = Q25 = CIA = 0.61 c.f.s.

707 W. Second Avenue
Spokane, WA 99201
(509) 455-4448

Stormwater Facilities and Detention Basin Design

POST-DEVELOPED

Date: 2/19/2020
Job No.: 20042-0005
Arch. Name:
Project: P1FCU - Ponderay

Description and Assumptions:
City of Ponderay
(See Project Location Map)
Design Frequency: 25 year
Basin Area < 10 acres, therefore use Rational Formula

$Q = C \times I \times A$ where Q = Runoff in cubic feet per second
C = Runoff Coefficient
I = Rainfall Intensity in inches per hour
A = Contributing Area in acres

1 Determine Weighted Runoff Coefficient, C and Area, A

Sub-Area: Basin A

Total Drainage Area (A):	39,327 s.f.	0.9028 ac.
Total PGIS:	27,584 s.f.	0.63 ac.

Surface Type	Area (s.f.)	Area (ac.)	C	C*Area in acres
Parking/Driveway (PGIS)	27,584	0.63	0.90	0.57
Sidewalk (PGIS)		-	0.90	-
Landscaping	7,963	0.18	0.30	0.05
Roof (NPGIS)	3,780	0.09	0.90	0.08
		-		-
TOTAL	39,327	0.90		0.70

Weighted Runoff Coefficient (C) = (sum CA)/(sumA) = 0.78

2 Determine Rainfall Intensity, I

Duration (in minutes) is based on the Time of Concentration, Tc

Reach 1: Overland Flow

$$Tc = \frac{L}{K * \sqrt{(S)}}$$

where Tc = Time of Concentration in minutes of the longest route that the flow will take
L = Length in feet
K = Ground Cover Coefficient
S = Average Slope in ft/ft

L (ft)	K	S
281	1200	0.01

Time of Concentration: 2.34 minutes

Tc shall not be less than 5 minutes, therefore:

Time of Concentration: 5.00 minutes

Intensity is calculated as: $I = \frac{m}{Tc^n}$

m	n
6.8	0.53

I = 2.90 in./hr.

3 Determine Peak Discharge, Q

Peak 25 yr Discharge = Q25 = CIA = 2.04 c.f.s.

4 Detention Basin Design using the Bowstring Method

Time Increment: 5 minutes
Time of Concentration: 5.00 minutes

Single Depth Drywell(s)

707 W. Second Avenue
Spokane, WA 99201
(509) 455-4448

Desired Outflow: 0.61 cfs
Runoff Coefficient: 0.779
Area: 0.90 acres

0 Double Depth Drywell(s)

Time (minutes)	Time (seconds)	Intensity (in/hr)	Q (cfs)	Volume In (cu.ft.)	Volume Out (cu.ft.)	Storage (cu.ft.)
0	0	-	-	-	-	-
2	120	2.90	2.04	270	93	176
5	300	2.90	2.04	636	234	**403**
10	600	2.01	1.41	864	467	397
15	900	1.62	1.14	1,038	701	337
20	1200	1.39	0.98	1,184	934	250
25	1500	1.23	0.87	1,313	1,168	145
30	1800	1.12	0.79	1,428	1,401	27
35	2100	1.03	0.73	1,534	1,635	(101)
40	2400	0.96	0.68	1,632	1,868	(236)
45	2700	0.90	0.64	1,724	2,102	(378)
50	3000	0.86	0.60	1,811	2,336	(525)
55	3300	0.81	0.57	1,893	2,569	(676)
60	3600	0.78	0.55	1,971	2,803	(831)
65	3900	0.74	0.52	2,046	3,036	(990)
70	4200	0.72	0.50	2,118	3,270	(1,151)
75	4500	0.69	0.48	2,188	3,503	(1,316)
80	4800	0.67	0.47	2,255	3,737	(1,482)
85	5100	0.65	0.45	2,320	3,970	(1,651)
90	5400	0.63	0.44	2,382	4,204	(1,822)
95	5700	0.61	0.43	2,443	4,438	(1,994)
100	6000	0.59	0.42	2,503	4,671	(2,168)

*Check formula depending on t < or > Tc

5 Determine Treatment Volume Required by Basin

Total PGIS Area: 27,584 sf

Required Volume Vs. Provided Volume

Treatment Method: ○ 1133A ● 1815A Equal to first 1/2" of runoff
Required Treatment Volume from Basin: 1,149 cu. ft.
Bioinfiltration Treatment Volume Provided: 1,169 cu. ft.

Bioinfiltration Area Bottom Width: 6.0 ft. Width at treatment depth: 9.0 ft
Bioinfiltration Area Bottom Length: 310 ft. Length at treatment depth: 313.0 ft
Bioinfiltration Treatment Depth: 0.5 ft. (Bottom Elevation to Rim Elevation)
Side Slope (X:1): 3

Bioinfiltration Bottom Area: 1,860 sf
Bioinfiltration Top Area: 2,817 sf

Is the Bioinfiltration area large enough to hold the required treatment volume? OK

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