WENDY'S- PONDERAY, ID

DRAINAGE REPORT

September 2020

The staff of Storhaug Engineering under the direction of the undersigned professional engineer whose seal and signature appear hereon has prepared this report.

Storhaug Engineering
Project No. 20-190

STORHAUG ENGINEERING
510 East Third, Spokane, Washington
Phone 509-242-1000 Fax 509-242-1001

Table of Contents

Chapter 1

DRAINAGE SUMMARY

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Wendy's – Ponderay, Idaho
Drainage Summary

PROJECT LOCATION AND DESCRIPTION

This drainage report and all associated documents are intended to provide design details for the drainage infrastructure proposed for the Wendy's site. It is the purpose of this report to demonstrate there is no negative impact to the adjacent properties with the proposed improvements to the project site with associated paving, utilities, and drainage infrastructure. Refer to the grading and drainage plans for further information on project location and layout. The project is located on Hwy 95, near the NW corner of Hwy 95 and Schweitzer Cutoff Road in the southeast quarter of Section 3, Township 57 North, Range 2 West, B.M., City of Ponderay, Idaho.

SITE CHARACTERISTICS

The site is mostly open with the ground surface sloping down east to west at approximately 1%. The west end of the site slopes steeply towards Sand Creek. The site is currently vacant, except for the shared driveway onto Hwy 95.

SUBSURFACE CHARACTERISTICS

The soils in this region of this property have been identified by the USDA Natural Resources Conservation Service (NRCS) as Mission Silt Loam, which has a published Hydrologic soil group rating of hydrologic group "D".

The following table summarizes the site soil information from Pre-development Conditions to Post-Development Conditions. This table summarizes the entirety of this project in Pre-development vs. Post-Development conditions. The values as listed below show the existing surfaces as well as the proposed construction.

ONSITE PRE-DEVELOPMENT VS. POST-DEVELOPMENT

	Pre-Development Condition	**Post-Development Condition**
Total Basin Area (acres)	**.73**	**.73**
Pollutant Generating Impervious Surface (acres)	**0**	**0.41**
Surface Cover	Existing Asphalt	Asphalt/Driveways
NRCS Soil Type	Type D	Type D
Runoff Curve Number	98	98
Non-pollutant Generating Impervious Surface (acres)	**0**	**0.08**
Surface Cover	Rooftops	Rooftops
NRCS Soil Type	Type D	Type D
Runoff Curve Number	98	98
Pervious Surfaces (acres)	**0.73**	**0.24**
Surface Cover	Brush/weed/grass	Landscaping/Lawn
NRCS Soil Type	Type D	Type D
Runoff Curve Number	73	80
Composite RCN	**73**	**92**

storhäug

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METHODOLOGY
The basin areas associated with the project were analyzed for runoff flows utilizing the NRCS TR-55 Method. Swales were sized to treat the first ½ inch of runoff of the paved impervious surfaces. Precipitation values where derived from NOAA Atlas 2 Isopluvial Maps. The swale/ detention pond was analyzed with the Hydrographs Extension for Civil 3D.

The Post-developed storm hydrograph was routed through the swale and outlet control structure. The outlet contains a 4-inch orifice 6 inches above the pond bottom to attenuate the storm. It also contains an overflow grate if the stormwater exceeds 2 feet of depth. The 6-inch treatment zone of the swale contains an underdrain, which transports treated stormwater to the outlet control structure.

POST DEVELOPMENT – OFFSITE
No upstream inflows were analyzed as part of this project.

PERPETUAL MAINTENANCE OF FACILITIES
The property owner will be responsible for maintaining proposed swale and storm system improvements.

REGIONAL FACILITIES
The drainage design provides for the treatment and proper distribution of runoff from all proposed onsite improvements. The improvements do not require the need for a regional facility nor is it designed to act as a regional facility for other neighboring properties.

RESULTS AND CONCLUSIONS
The table below shows the peak flows for the predeveloped and post-developed conditions, as well as the resultant outflow after the post-developed storm is routed through the swale. This information and the backup data are provided in the Drainage Calculations section of the report

Hydrograph	Peak flows (cfs)			
Return Period	**2-yr**	**10-yr**	**50-yr**	**100-yr**
Predeveloped	0.26	0.60	1.14	1.43
Post-developed	1.50	2.13	2.98	3.39
Outflow	0.24	0.37	0.55	1.39

The swale and outlet control structure effectively attenuate the storm, so that the rate of runoff leaving the site is no more than the predeveloped rate.

The stormwater piping was verified by analyzing each sub-basin with the Rational Method and applying each flow to it respective catch basin. A hydraulic analysis of the pipes is included in the calculations. Sub-basin information is as follows:

Sub-basin	Area (sf)	Runoff Coefficient	25-year Peak Flow (cfs)	Flow Assigned to Structure
A1	4,328	0.9	0.25	CB #1
A2	4,546	0.9	0.26	CB#2
A3	3,399	0.9	0.20	CB#3
A4	4,022	0.9	0.23	CB#5
A5	2,910	0.9	0.17	CB#6
A6	3,690	0.9	0.21	CB#4
A7	2,592	0.9	0.15	CB#5

Additional calculations and documentation supporting this design concept follow.

Chapter 2

BASIN MAP

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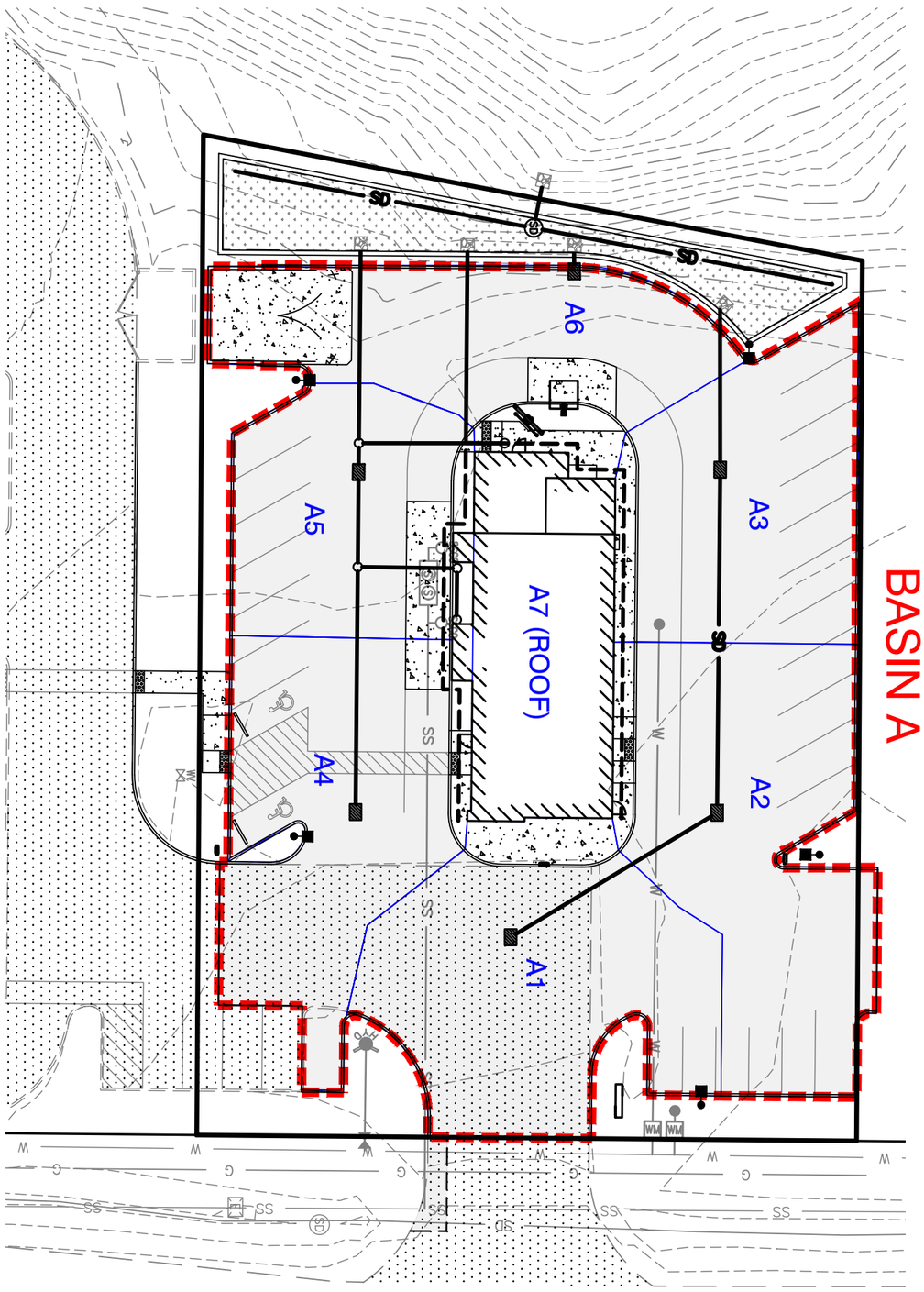
02.01 20–190 BASIN MAP.dwg 11x17



GRAPHIC SCALE

(IN FEET)
1 inch = 30 ft.

30 0 15 30 60

CALL BEFORE YOU DIG 811

DRAWN	JEW	DATE 09/15/20	
CHECKED	SPS	SCALE 1" = 30'	
1 OF 1			
PROJECT NUMBER	20–190		

BASIN MAP
WENDYS
PONDERAY, ID

storhäug
civil engineering | planning
landscape architecture | surveying
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509.242.1000 | www.storhauginc.com

Chapter 3

CALCULATIONS

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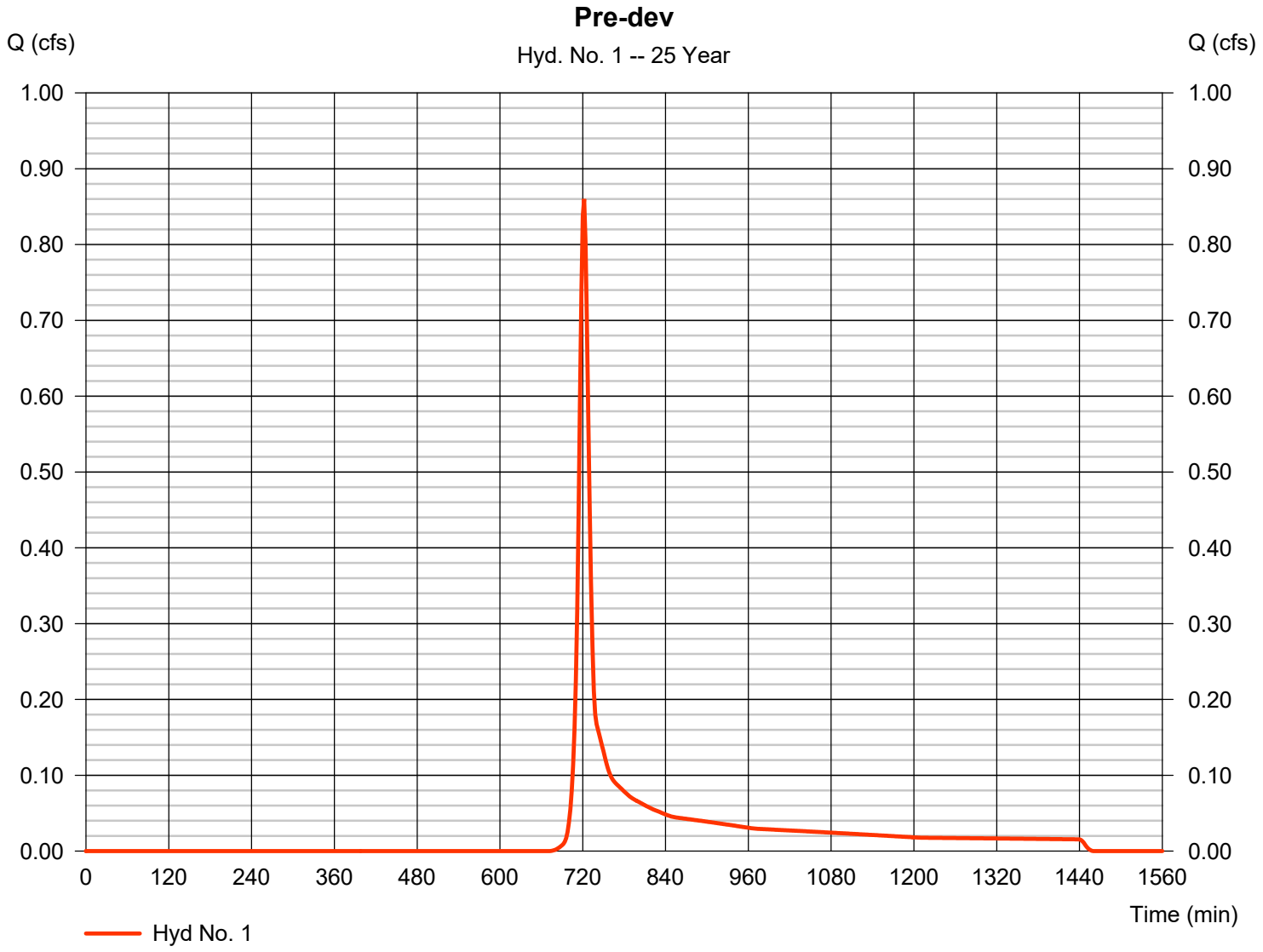
Hydrograph Report

Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2021 Thursday, 09 / 17 / 2020

Hyd. No. 1

Pre-dev

Hydrograph type	= SCS Runoff	Peak discharge	= 0.860 cfs
Storm frequency	= 25 yrs	Time to peak	= 722 min
Time interval	= 2 min	Hyd. volume	= 2,343 cuft
Drainage area	= 0.730 ac	Curve number	= 73
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 10.00 min
Total precip.	= 3.00 in	Distribution	= Type II
Storm duration	= 24 hrs	Shape factor	= 484



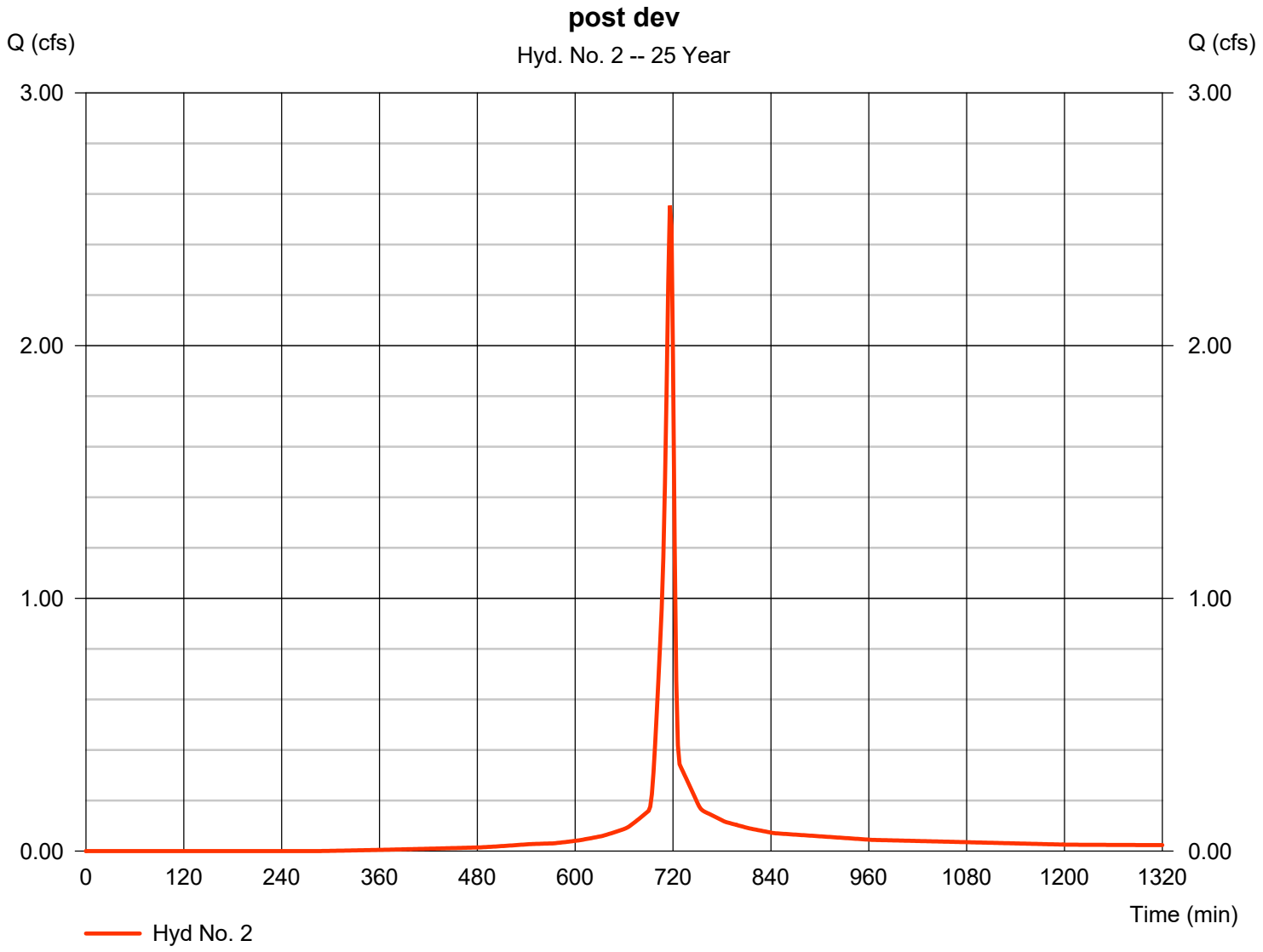
Hydrograph Report

Hyd. No. 2

post dev

Hydrograph type	=	SCS Runoff	Peak discharge	=	2.554 cfs
Storm frequency	=	25 yrs	Time to peak	=	716 min
Time interval	=	2 min	Hyd. volume	=	5,369 cuft
Drainage area	=	0.730 ac	Curve number	=	92*
Basin Slope	=	0.0 %	Hydraulic length	=	0 ft
Tc method	=	User	Time of conc. (Tc)	=	5.00 min
Total precip.	=	3.00 in	Distribution	=	Type II
Storm duration	=	24 hrs	Shape factor	=	484

* Composite (Area/CN) = [(0.490 x 98) + (0.240 x 80)] / 0.730



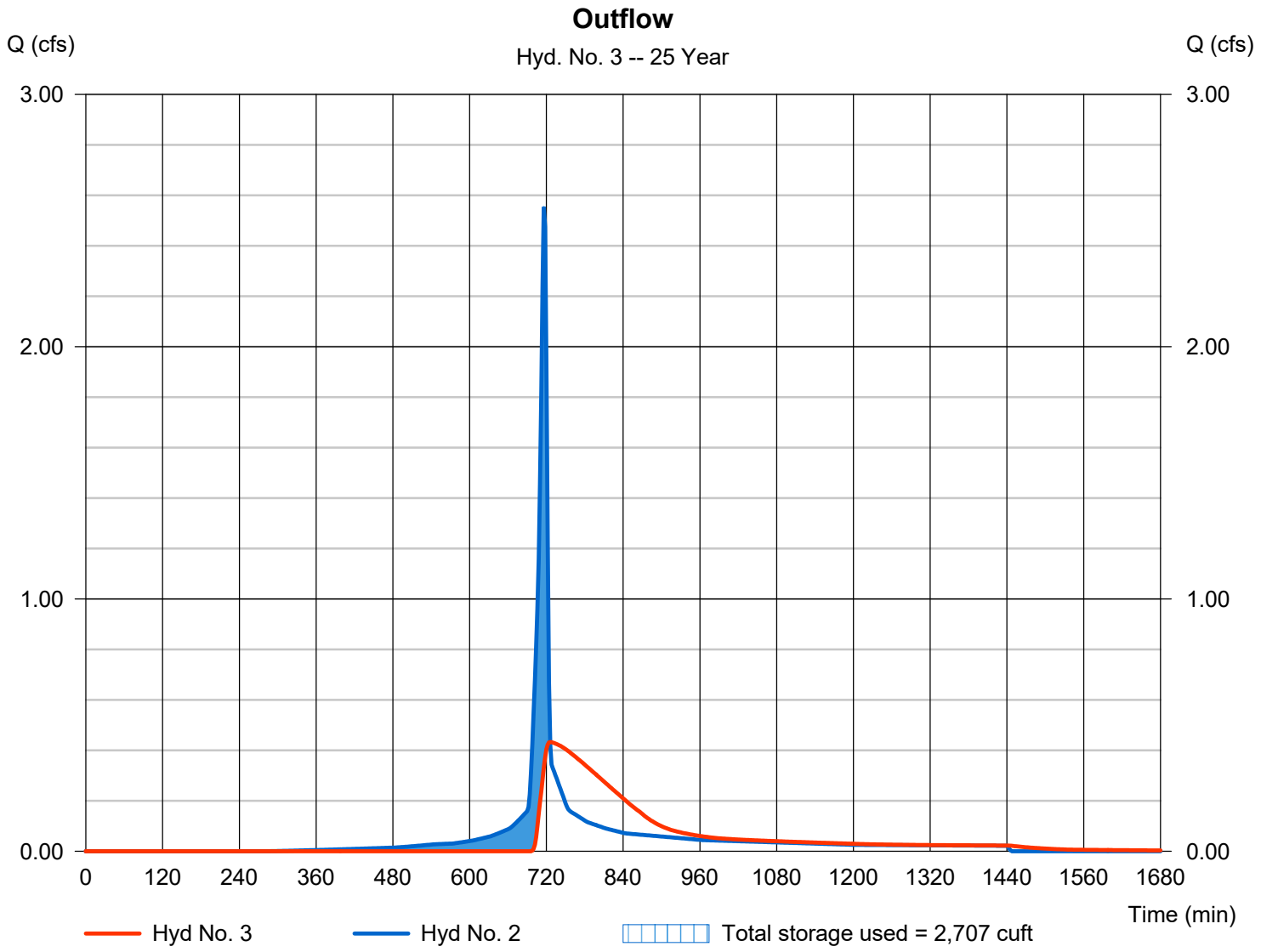
Hydrograph Report

Hyd. No. 3

Outflow

Hydrograph type	=	Reservoir	Peak discharge	=	0.433 cfs
Storm frequency	=	25 yrs	Time to peak	=	726 min
Time interval	=	2 min	Hyd. volume	=	4,574 cuft
Inflow hyd. No.	=	2 - post dev	Max. Elevation	=	2123.73 ft
Reservoir name	=	Pond	Max. Storage	=	2,707 cuft

Storage Indication method used.



Pond Report

Pond No. 1 - Pond

Pond Data

Contours -User-defined contour areas. Average end area method used for volume calculation. Begining Elevation = 2122.00 ft

Stage / Storage Table

Stage (ft)	Elevation (ft)	Contour area (sqft)	Incr. Storage (cuft)	Total storage (cuft)
0.00	2122.00	1,566	0	0
0.50	2122.50	1,566	783	783
1.00	2123.00	1,566	783	1,566
1.50	2123.50	1,566	783	2,349
2.00	2124.00	1,566	783	3,132
2.50	2124.50	1,566	783	3,915
3.00	2125.00	1,566	783	4,698

Culvert / Orifice Structures

	[A]	[B]	[C]	[PrfRsr]
Rise (in)	= 6.00	4.00	0.00	0.00
Span (in)	= 6.00	4.00	0.00	0.00
No. Barrels	= 1	1	0	0
Invert El. (ft)	= 2120.00	2122.50	0.00	0.00
Length (ft)	= 10.00	1.00	0.00	0.00
Slope (%)	= 2.00	1.00	0.00	n/a
N-Value	= .013	.013	.013	n/a
Orifice Coeff.	= 0.60	0.60	0.60	0.60
Multi-Stage	= n/a	Yes	No	No

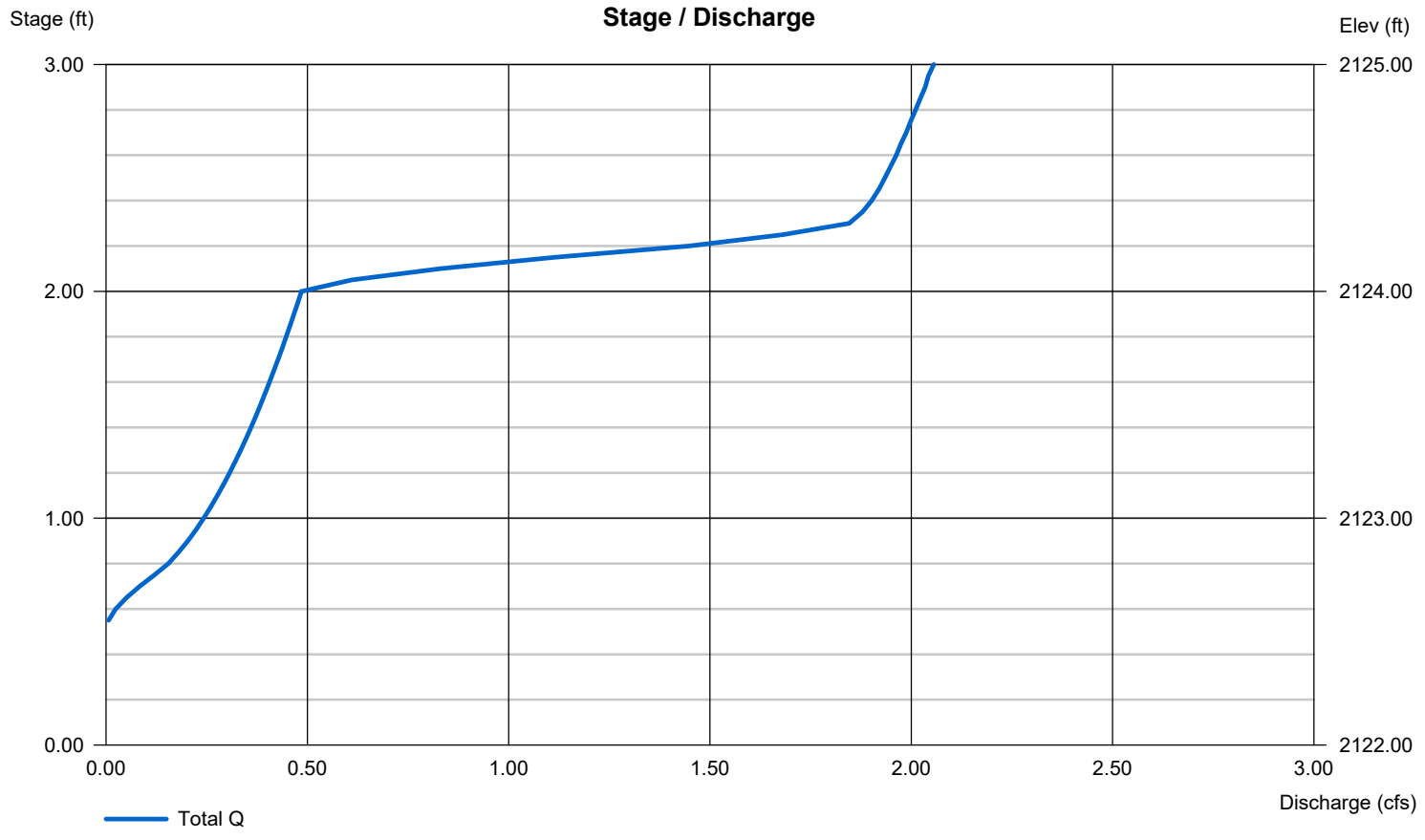
Weir Structures

	[A]	[B]	[C]	[D]
Crest Len (ft)	= 3.14	0.00	0.00	0.00
Crest El. (ft)	= 2124.00	0.00	0.00	0.00
Weir Coeff.	= 3.30	3.33	3.33	3.33
Weir Type	= 1	---	---	---
Multi-Stage	= Yes	No	No	No

Exfil.(in/hr) = 0.000 (by Contour)

TW Elev. (ft) = 0.00

Note: Culvert/Orifice outflows are analyzed under inlet (ic) and outlet (oc) control. Weir risers checked for orifice conditions (ic) and submergence (s).

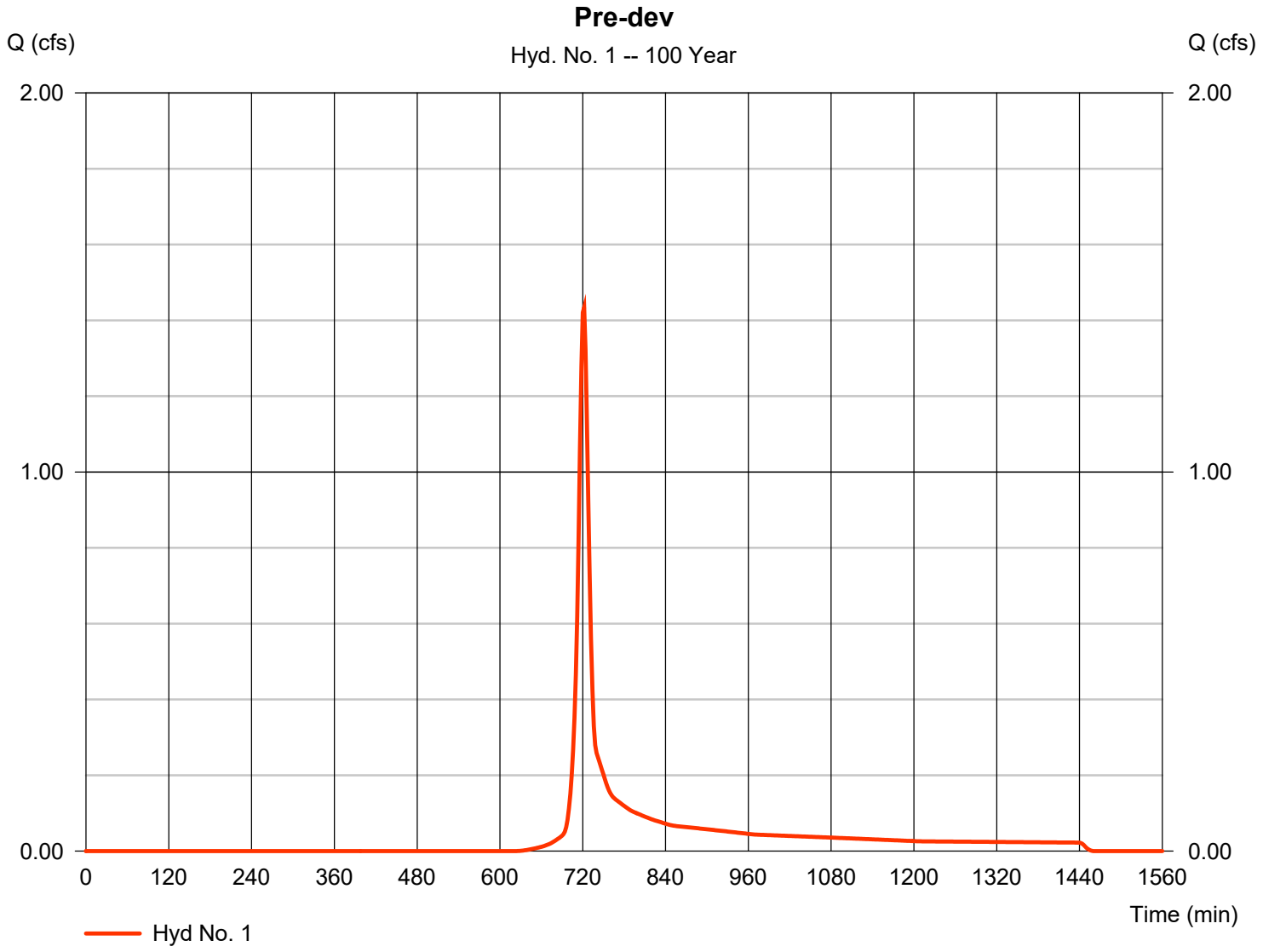


Hydrograph Report

Hyd. No. 1

Pre-dev

Hydrograph type	=	SCS Runoff	Peak discharge	=	1.434 cfs
Storm frequency	=	100 yrs	Time to peak	=	722 min
Time interval	=	2 min	Hyd. volume	=	3,786 cuft
Drainage area	=	0.730 ac	Curve number	=	73
Basin Slope	=	0.0 %	Hydraulic length	=	0 ft
Tc method	=	User	Time of conc. (Tc)	=	10.00 min
Total precip.	=	3.80 in	Distribution	=	Type II
Storm duration	=	24 hrs	Shape factor	=	484



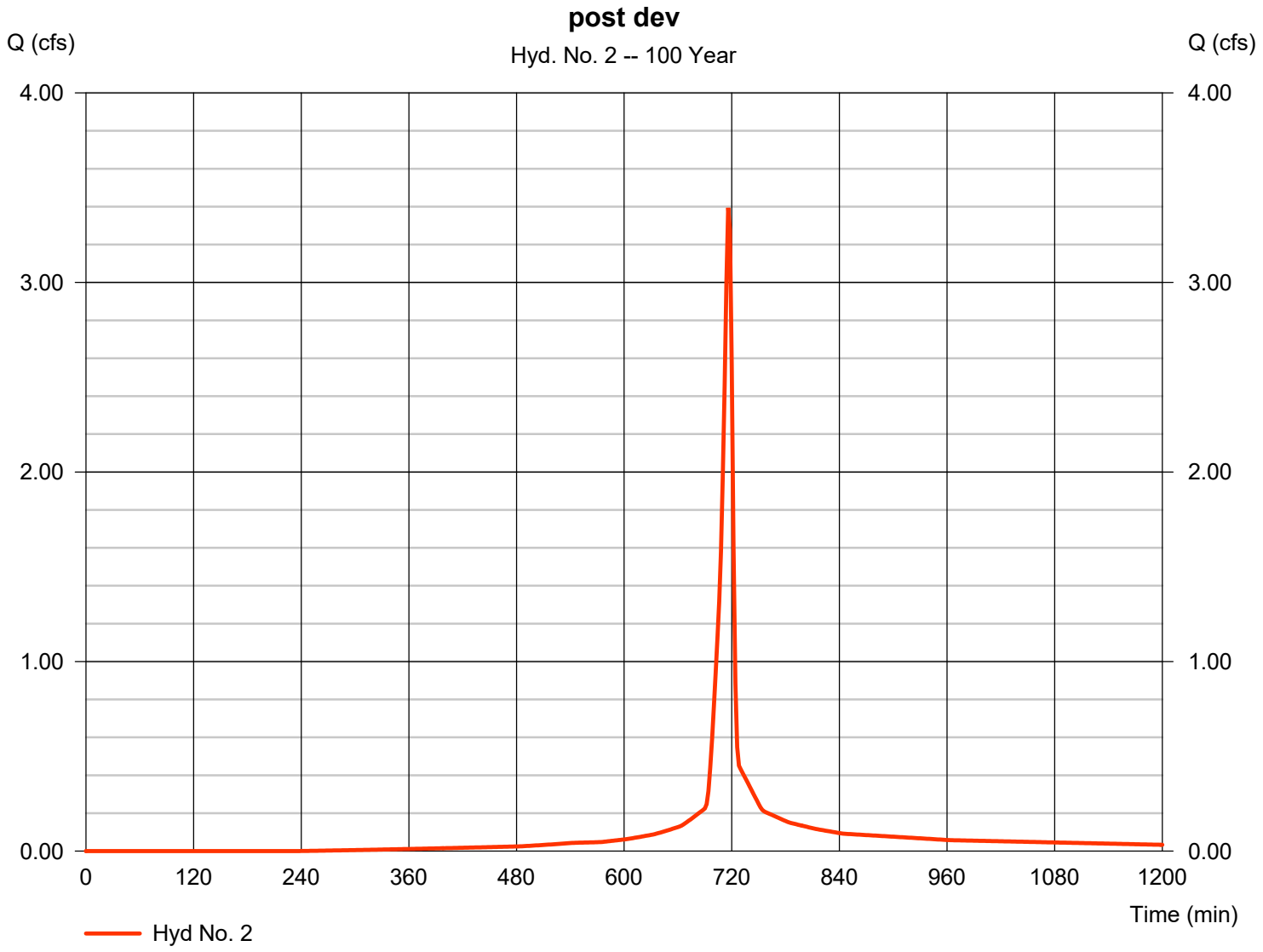
Hydrograph Report

Hyd. No. 2

post dev

Hydrograph type	=	SCS Runoff	Peak discharge	=	3.394 cfs
Storm frequency	=	100 yrs	Time to peak	=	716 min
Time interval	=	2 min	Hyd. volume	=	7,266 cuft
Drainage area	=	0.730 ac	Curve number	=	92*
Basin Slope	=	0.0 %	Hydraulic length	=	0 ft
Tc method	=	User	Time of conc. (Tc)	=	5.00 min
Total precip.	=	3.80 in	Distribution	=	Type II
Storm duration	=	24 hrs	Shape factor	=	484

* Composite (Area/CN) = [(0.490 x 98) + (0.240 x 80)] / 0.730



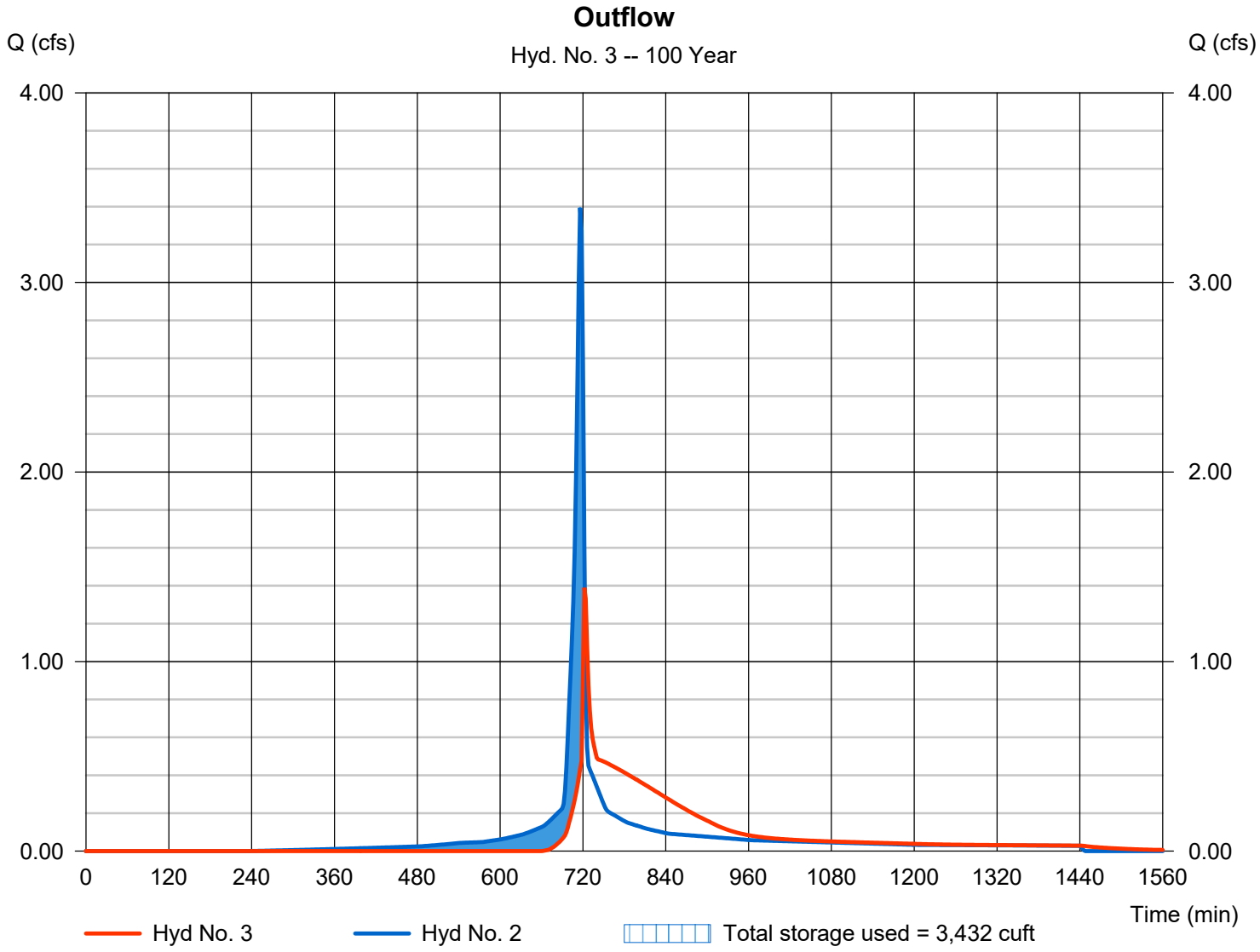
Hydrograph Report

Hyd. No. 3

Outflow

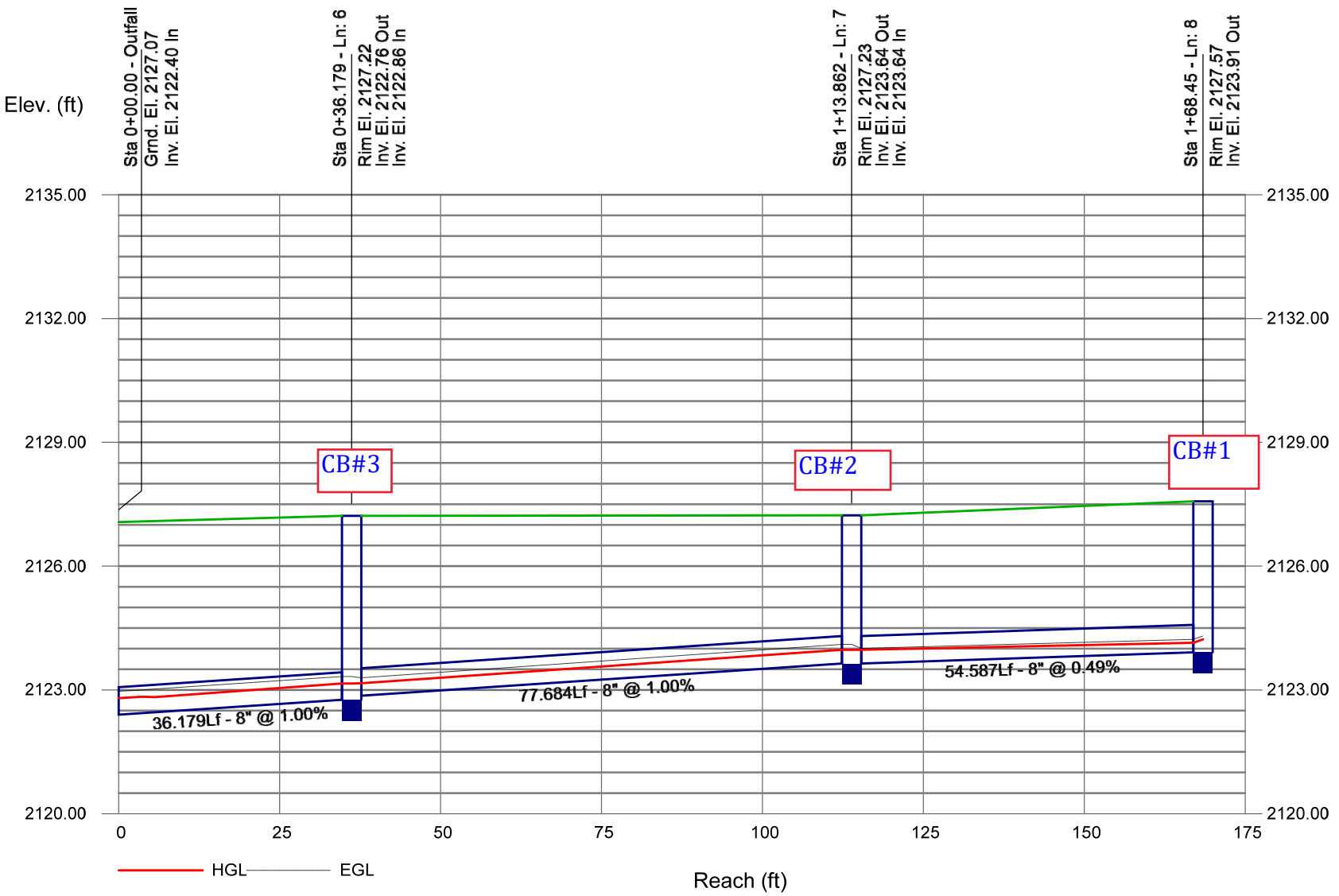
Hydrograph type	=	Reservoir	Peak discharge	=	1.390 cfs
Storm frequency	=	100 yrs	Time to peak	=	722 min
Time interval	=	2 min	Hyd. volume	=	6,471 cuft
Inflow hyd. No.	=	2 - post dev	Max. Elevation	=	2124.19 ft
Reservoir name	=	Pond	Max. Storage	=	3,432 cuft

Storage Indication method used.



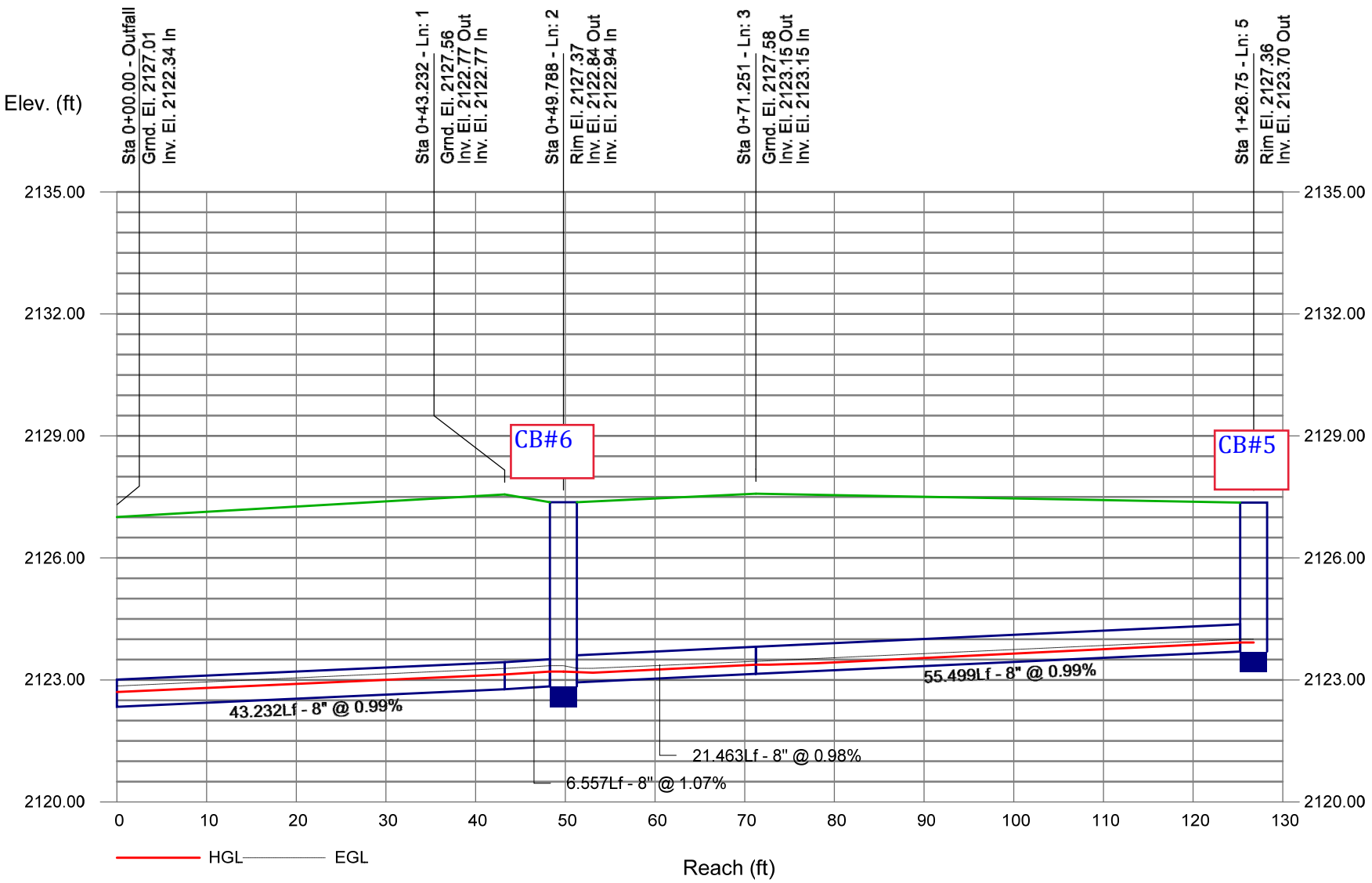
Storm Sewer Profile

Proj. file: storm sewer analysis2.stm



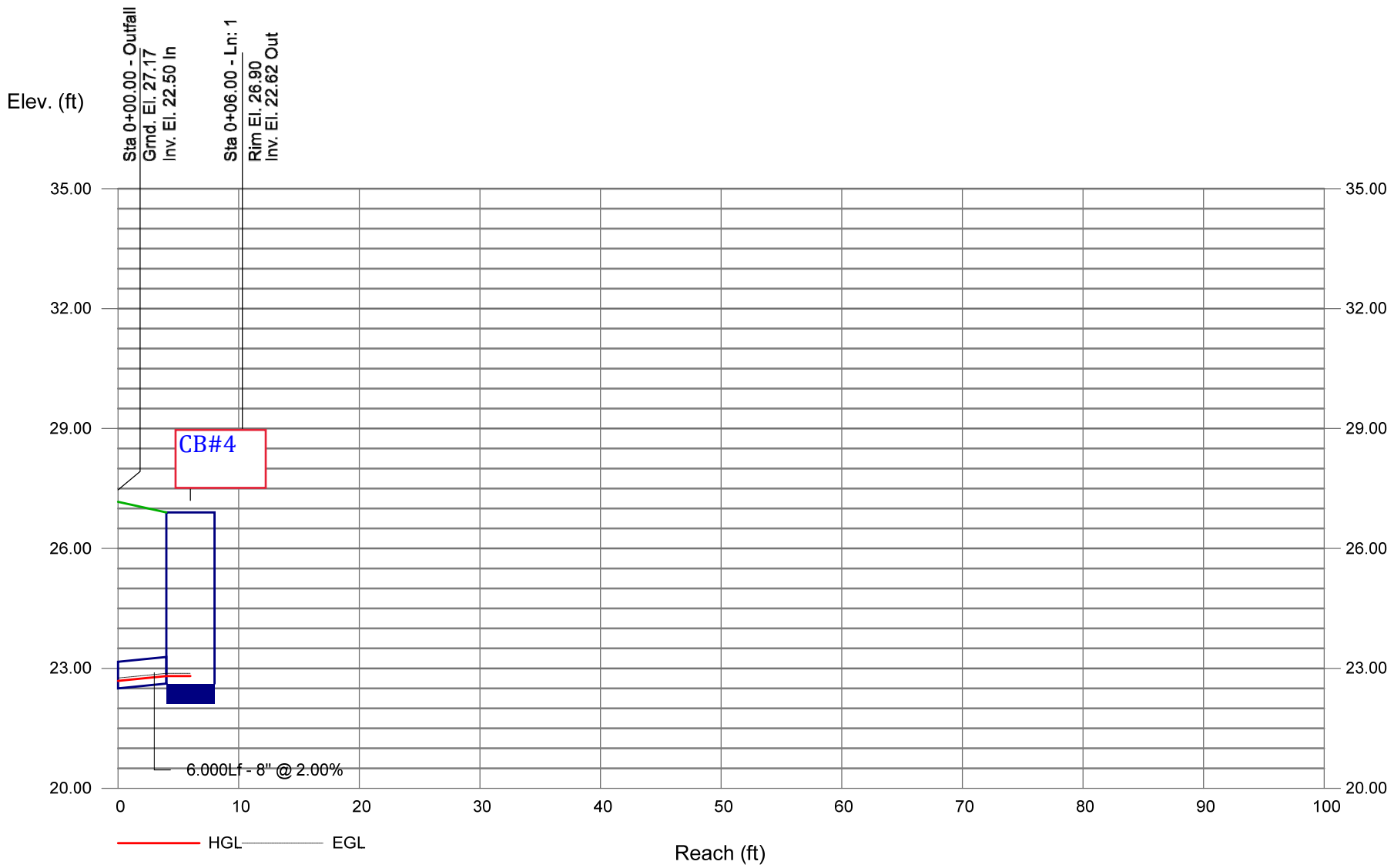
Storm Sewer Profile

Proj. file: storm sewer analysis2.stm



Storm Sewer Profile

Proj. file: New.stm



Chapter 4

GEOTECHNICAL REPORT

STORHAUG ENGINEERING
510 East Third, Spokane, Washington
Phone 509-242-1000 Fax 509-242-1001

June 22, 2020

Wenspok Companies
503 East 2nd Avenue, Suite B
Spokane, Washington 99202

Attention: Mr. Jennifer Robson

RE: Geotechnical Evaluation
Wendy's Ponderay Branch
Parcel #: RPP00000039520A
Ponderay, Idaho
ALLWEST Project No. 120-058G

Ms. Robson,

ALLWEST has completed the authorized geotechnical evaluation for the proposed restaurant building located at Parcel #: RPP00000039520A 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, P.G.
Senior Engineering Geologist

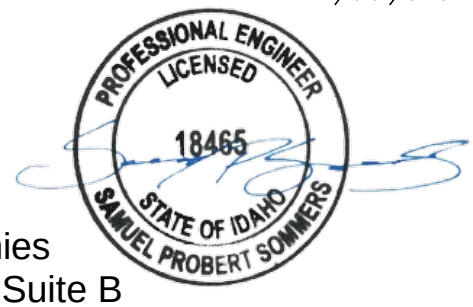
Reviewed by:

Samuel P. Sommers, P.E.
Engineering Services Manager

GEOTECHNICAL EVALUATION
WENDY'S PONDERAY BRANCH
PARCEL #: RPP00000039520A
PONDERAY, IDAHO
ALLWEST PROJECT NO. 120-058G

06/22/2020

June 22, 2020



Prepared for:
Wenspok Companies
503 East 2nd Avenue, Suite B
Spokane, Washington 99202

Prepared by:
ALLWEST
690 West Capstone Court
Hayden, Idaho 83835

WWW.ALLWESTTESTING.COM

TABLE OF CONTENTS
Geotechnical Evaluation
Wendy's Ponderay Branch
Parcel #: RPP00000039520A
Ponderay, Idaho

EXECUTIVE SUMMARY

ALLWEST has completed the authorized geotechnical evaluation for the Wendy's Ponderay Branch located at Parcel #: RPP00000039520A in Ponderay, Idaho. 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:

- Undocumented fill and topsoil were encountered on the site to depths of up to 2 feet and should be over-excavated from underneath all structural elements to the site.
- The undocumented fill and topsoil are not suitable for re-use as structural fill due to the organic contents. The native silt is not suitable for re-use as structural fill due to it being moisture sensitive and frost susceptible.
- An allowable bearing pressure of 2,500 pounds per square foot (psf) can be used for shallow footings bearing on structural fill overlying properly prepared subgrade.
- For light-duty pavement: A pavement section of 2½-inches asphaltic concrete over a minimum of 4-inches crushed aggregate base over at least 18 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 at least 18 inches of structural fill is recommended.
- We recommend a permanent foundation drainage system be designed and constructed around the perimeter of the structure.
- Based on the subsurface conditions encountered in our explorations, the site is not suitable for stormwater infiltration.

Our services were provided in general accordance with our proposal 120-058P dated February 24, 2020. 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
Wendy's Ponderay Branch
Parcel #: RPP00000039520A
Ponderay, Idaho

1.0 PROJECT DESCRIPTION

We understand the proposed project will consist of constructing a restaurant building. We estimate the structure will have continuous footing loads of 3 to 4 kips/lineal foot. An asphalt parking lot and miscellaneous landscaping improvements may 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 six test pits throughout the project site. The approximate locations of the test pits are shown on Figure A-1, Site and Exploration Plan. 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 one parcel, approximately 0.7 acres in total size. Topographically, the property is relatively flat. There is an existing asphalt driveway in the southeast section of the property that leads into the adjacent southern lot. The property is bordered to the South by a retail complex, to the east by State Highway 95, and the North and East by undeveloped parcels. The ground coverage consists of mostly grass and dirt.

4.0 SUBSURFACE CONDITIONS

4.1 Published Geologic Information

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.

The USDA Natural Resources Conservation Service (NRCS) has mapped the soils on and around the property as Mission 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.

4.2 Subsurface Investigation

Six test pits were excavated at the site on June 4, 2020. The test pits were excavated with a mini-excavator with a 18-inch wide toothed excavation bucket. The approximate locations of the test pits are shown on Figure A-1, Site and Exploration Plan. 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.

The near surface geologic profile appears to consist of undocumented fill or topsoil overlying silt overlying silty sand 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 damp. This unit is unsuitable for re-use as structural fill due to the organic content. The topsoil was approximately 1 foot thick, where encountered.

Undocumented fill – Many of the test pits encountered undocumented fill at the ground surface. This unit consists of sandy silt with organics, and minor debris. The color appeared dark brown, and the unit was damp. This unit is unsuitable for re-use as structural fill due to the organic content. The unit was approximately 1 to 2 feet thick.

Silt – Below the topsoil or undocumented fill layers, we encountered native silt. The soil layer was observed to be brown, moist, and stiff. This unit extended to approximately 6 to 7 feet below existing grade in the test pits.

Silty sand – Below the silt layer, we encountered native silty sand. The soil layer was observed to be gray-brown, very moist, and medium dense. This unit extended to approximately 8 to 9 feet below existing grade in the test pits.

Lean clay – Lean clay was encountered below the silty sand. The unit was light brown, moist, and medium stiff . The lean clay extended beyond the maximum depth of our test pit exploration of approximately 10 feet.

4.3 Groundwater Conditions

We did not encounter groundwater in our explorations. 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 particle size distribution by Wash 200 tests (ASTM D1140) and Liquid Limit 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 and stripping of organic-rich topsoil and undocumented fill. Based on our explorations, the stripping depth for topsoil and undocumented fill removal is estimated to be approximately 1 to 2 feet. Clearing and stripping debris should be wasted off site or used for topsoil in landscape areas.

Subgrade Preparation: Once clearing and stripping is complete, all areas that are at design subgrade elevation or areas that will receive new structural fill as

recommended in the **6.2 SUBGRADE STABILIZATION** section of this report should be evaluated by the Geotechnical Engineer before proceeding with Subgrade Stabilization.

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 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 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 4 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 undocumented fill and topsoil.

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 structural fill 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
48.3100	-116.5470	0.338g	0.112g	1.529	2.354

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 Loads (ESALs) Light / Heavy Duty	30,000 / 75,000
Assumed: Pavement reliability	85%
Assumed: Pavement design life	20-year

PAVEMENT SECTION 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	18
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	18
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. Concrete aprons should be underlain by a minimum of 4 inches of crushed aggregate base.

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

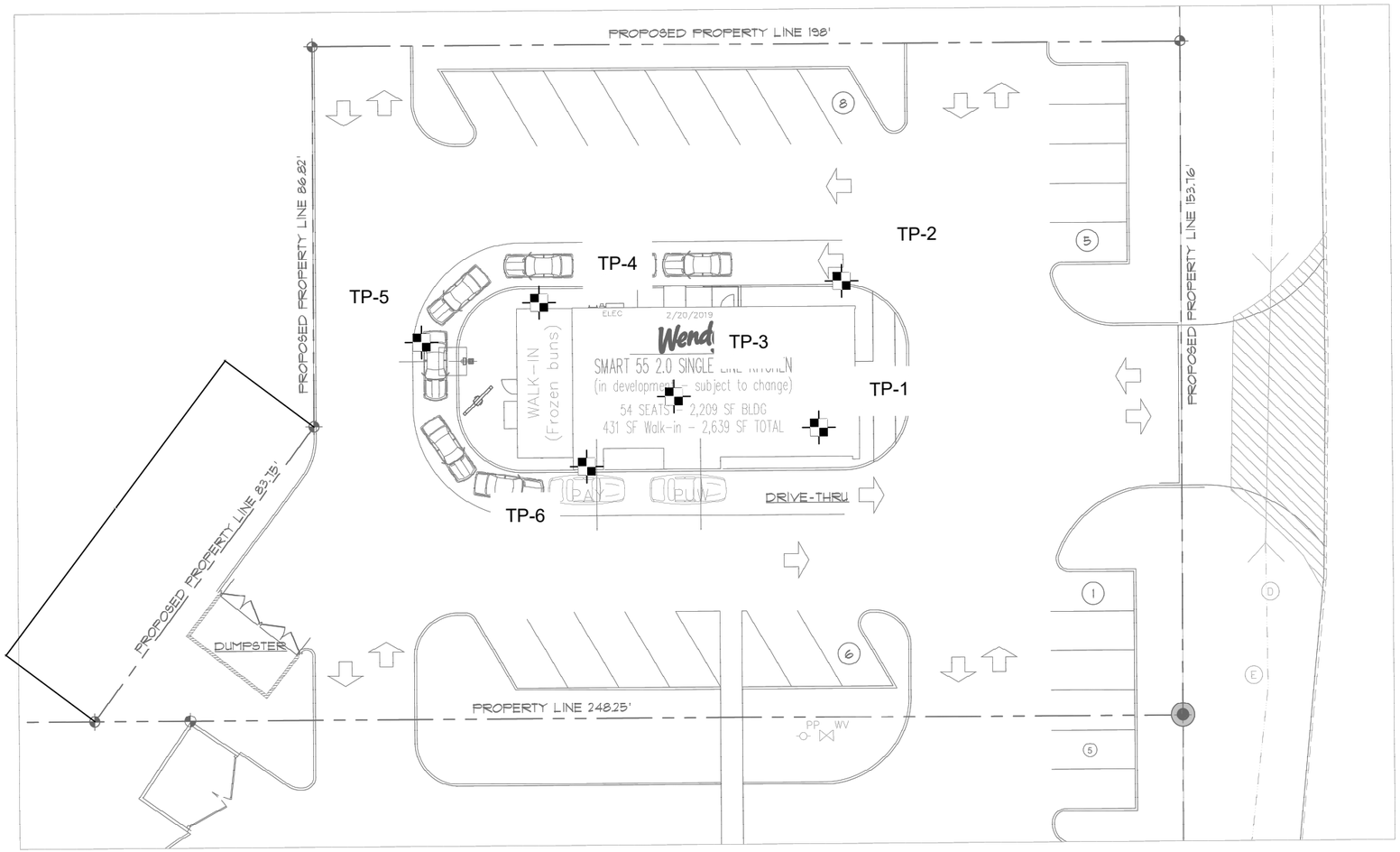
This report has been prepared to assist the planning and design for the Wendy's Ponderay Branch located at Parcel #: RPP00000039520A in Ponderay, Idaho. Reliance by any other party is prohibited without the written authorization of ALLWEST. Our services consist of professional opinions and conclusions 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 – Site and Exploration Plan
Appendix B – Test Pit Logs, Unified Soil Classification System
Appendix C – Laboratory Test Results

Appendix A

Site and Exploration Plan



Basemap Source: Site Plan prepared by Russell C. Page Architects, dated February 5, 2020.

LEGEND:

TP-1 TEST PIT NUMBER AND APPROXIMATE LOCATION

ALLWEST

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

30 0 15 30
SCALE IN FEET

FIGURE A-1: SITE AND EXPLORATION PLAN			
PROJECT:	120-058G Wendy's Ponderay Branch		
LOCATION:	Parcel #: RPP00000039520A, Ponderay, Idaho		
CLIENT NAME:	Wenspok Companies		
DATE:	June 2020	SCALE:	As Shown

Appendix B

Test Pit Logs
Unified Soil Classification System

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

TEST PIT LOG

DATE STARTED: 6/4/2020
DATE FINISHED: 6/4/2020
OPERATOR: Rick Marcus
COMPANY:
LOGGER: Kenny Rukavina
WEATHER: Clear

TEST PIT TP-1

EXCAVATOR: Mini Excavator
EXCAVATION METHOD: Soil Excavation Bucket

PROJECT: 120-058G Wendy's Ponderay Branch

NOTES:

TOTAL DEPTH: 10'

DEPTH (ft)	USCS	DESCRIPTION	GRAPHIC LOG	NOTES
0	TOPSOIL	Topsoil: Sandy SILT with organics, dark brown, damp.		
1	ML	SILT with sand, light brown to brown, moist, stiff.		@ 1ft: LL = 29 PI = 7 Fines Content = 78%
7	SM	Silty SAND, gray-brown, very moist, medium dense.		
8	CL	Lean CLAY, light brown, moist, medium stiff.		
10		Test pit TP-1 terminated at 10 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: 6/4/2020
DATE FINISHED: 6/4/2020
OPERATOR: Rick Marcus
COMPANY:
LOGGER: Kenny Rukavina
WEATHER: Clear

TEST PIT TP-2

EXCAVATOR: Mini Excavator
EXCAVATION METHOD: Soil Excavation Bucket

PROJECT: 120-058G Wendy's Ponderay Branch

NOTES:

TOTAL DEPTH: 10'

DEPTH (ft)	USCS	DESCRIPTION	GRAPHIC LOG	NOTES
0	FILL	Undocumented FIll: Sandy SILT with organics and gravel, dark brown, damp. Contained asphalt debris.		
1				
2	ML	SILT with sand, light brown to brown, moist, stiff.		
3				@ 3ft: LL = 31 PI = 8 Fines Content = 84%
4				
5				
6				
7	SM	Silty SAND, gray-brown, moist, medium dense.		
8				
9	CL	Lean CLAY, light brown, moist, medium 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: 6/4/2020
DATE FINISHED: 6/4/2020
OPERATOR: Rick Marcus
COMPANY:
LOGGER: Kenny Rukavina
WEATHER: Clear

TEST PIT TP-3

EXCAVATOR: Mini Excavator
EXCAVATION METHOD: Soil Excavation Bucket

PROJECT: 120-058G Wendy's Ponderay Branch

NOTES:

TOTAL DEPTH: 10'

DEPTH (ft)	USCS	DESCRIPTION	GRAPHIC LOG	NOTES
0	FILL	Undocumented Flll: Sandy SILT with organics and gravel, dark brown, damp. Contained garbage.		
2	ML	SILT, light brown to brown, moist, stiff.		
7	SM	Silty SAND, gray-brown, very moist, medium dense.		
8	CL-ML	CLAYEY SILT, light brown, moist, medium stiff.		@ 9ft: LL = 26 PI = 5 Fines Content = 97%
10		Test pit TP-3 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: 6/4/2020
DATE FINISHED: 6/4/2020
OPERATOR: Rick Marcus
COMPANY:
LOGGER: Kenny Rukavina
WEATHER: Clear

TEST PIT TP-4

EXCAVATOR: Mini Excavator
EXCAVATION METHOD: Soil Excavation Bucket

PROJECT: 120-058G Wendy's Ponderay Branch

NOTES:

TOTAL DEPTH: 9.5'

DEPTH (ft)	USCS	DESCRIPTION	GRAPHIC LOG	NOTES
0	FILL	Undocumented FIll: Sandy SILT with organics and gravel, dark brown, damp. Contained PVC piping.		
1				
2	ML	SILT, light brown to brown, moist, stiff.		
3				
4				
5				
6				
7	SM	Silty SAND, gray-brown, very moist, medium dense.		
8				
9	CL	Lean CLAY, light brown, moist, medium stiff.		
10		Test pit TP-4 terminated at 9.5 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: 6/4/2020
DATE FINISHED: 6/4/2020
OPERATOR: Rick Marcus
COMPANY:
LOGGER: Kenny Rukavina
WEATHER: Clear

TEST PIT TP-5

EXCAVATOR: Mini Excavator
EXCAVATION METHOD: Soil Excavation Bucket

PROJECT: 120-058G Wendy's Ponderay Branch

NOTES:

TOTAL DEPTH: 10'

DEPTH (ft)	USCS	DESCRIPTION	GRAPHIC LOG	NOTES
0	TOPSOIL	Topsoil: Sandy SILT with organics, dark brown, damp.		
1	ML	SILT, light brown to brown, moist, stiff.		
6	SM	Silty SAND, gray-brown, very moist, medium dense.		
8	CL	Lean CLAY, light brown, moist, medium stiff.		
10		Test pit TP-5 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: 6/4/2020
DATE FINISHED: 6/4/2020
OPERATOR: Rick Marcus
COMPANY:
LOGGER: Kenny Rukavina
WEATHER: Clear

TEST PIT TP-6

EXCAVATOR: Mini Excavator
EXCAVATION METHOD: Soil Excavation Bucket

PROJECT: 120-058G Wendy's Ponderay Branch

NOTES:

TOTAL DEPTH: 10'

DEPTH (ft)	USCS	DESCRIPTION	GRAPHIC LOG	NOTES
0	FILL	Undocumented Fill: Sandy SILT with organics, dark brown, damp. Contains metal debris and garbage.		
1	ML	SILT, light brown to brown, moist, stiff.		@ 2ft: LL = 29 PI = 7 Fines Content = 78%
7	SM	Silty SAND, gray-brown, very moist, medium dense.		
8	CL	Lean CLAY, light brown, moist, medium stiff.		
10		Test pit TP-6 terminated at 10 feet. No groundwater observed. No caving observed.		
12				

WATER LEVELS
▽ WHILE EXCAVATING
▼ AT COMPLETION
▼ AFTER EXCAVATING

Unified Soil Classification System

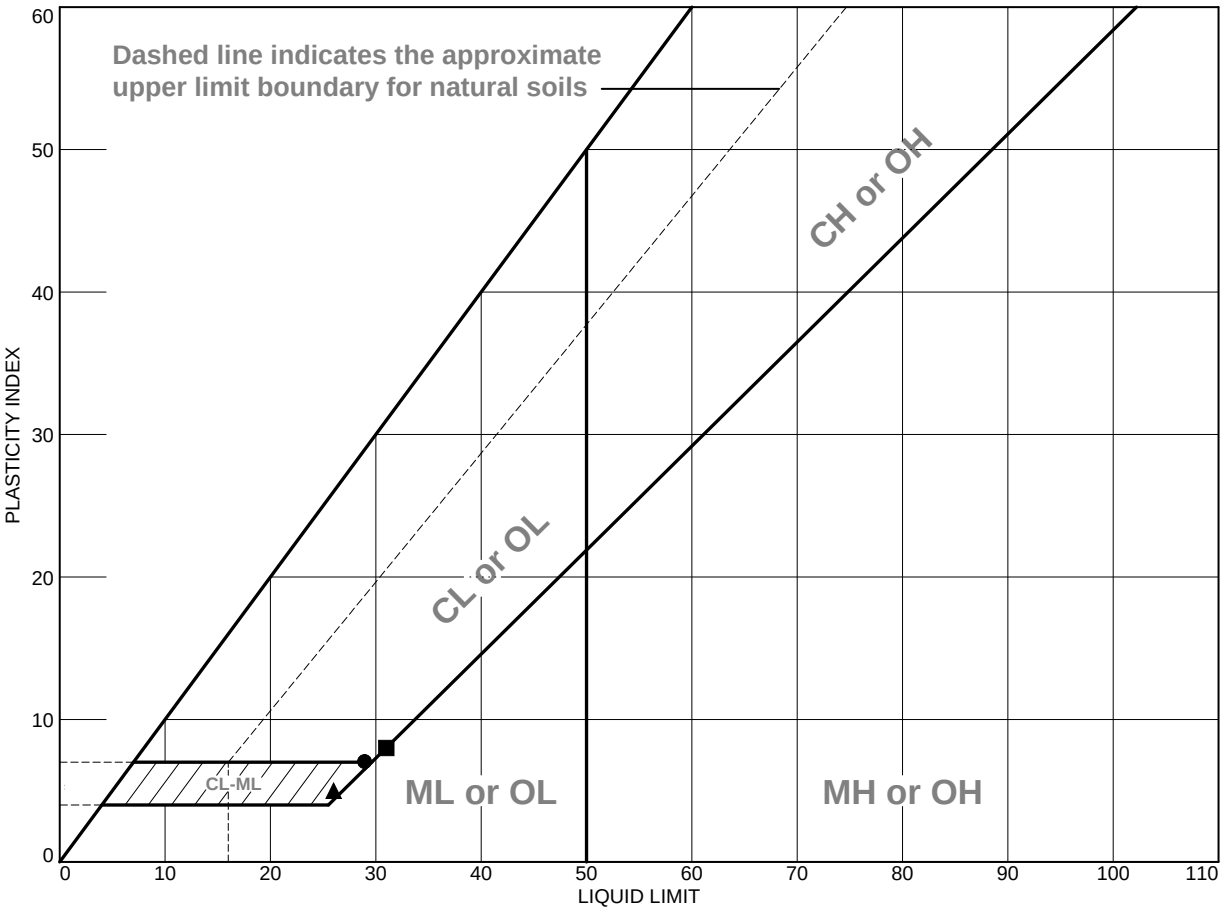
MAJOR DIVISIONS			SYMBOL	TYPICAL NAMES
COARSE GRAINED SOILS	GRAVELS	CLEAN GRAVELS	GW	Well-Graded Gravel, Gravel-Sand Mixtures.
			GP	Poorly-Graded Gravel, Gravel-Sand Mixtures.
		GRAVELS WITH FINES	GM	Silty Gravel, Gravel-Sand-Silt Mixtures.
			GC	Clayey Gravel, Gravel-Sand-Clay Mixtures.
	SANDS	CLEAN 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

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Light Brown to Brown Silt	29	22	7		78	
■	Light Brown to Brown Silt	31	23	8		84	
▲	Light Brown Lean Clay	26	21	5		97	
◆	Light Brown to Brown Silt	29	22	7		78	

Project No. 120-058G **Client:** Wenspok Companies

Project: Wendy's Ponderay Branch

●**Location:** TP1 **Depth:** -1' **Sample Number:** S120-0437
■**Location:** TP2 **Depth:** -3' **Sample Number:** S120-0438
▲**Location:** TP3 **Depth:** -9' **Sample Number:** S120-0440
◆**Location:** TP6 **Depth:** -2' **Sample Number:** S120-0442

ALLWEST TESTING & ENGINEERING

Hayden, ID

Remarks:

●Sampled By: Kenneth Rukavina
Sample Date: 6/8/20
■Sampled By: Kenneth Rukavina
Sample Date: 6/8/20
▲Sampled By: Kenneth Rukavina
Sample Date: 6/8/20
◆Sampled By: Kenneth Rukavina
Sample Date: 6/8/20

Figure C-1

Tested By: Christian Kreiger **Checked By:** Chris McKissen