EXCEL
ARCHITECTS • ENGINEERS • SURVEYORS

STORM WATER & EROSION CONTROL CALCULATIONS
FOR:
Montessori School
CITY OF WAUKESHA, WI
Excel Job #: 1818660

BASED ON SCS TR-55 METHOD, MANNINGS EQUATION, AND SLAMM
JULY 9, 2018
REV: AUGUST 13, 2018

ERIC
DRAZKOWSKI
0026356-006
FOND DU LAC
WISCONSIN

Prepared By:
Eric Drazkowski, P.E.
Excel Engineering Inc.
100 Camelot Drive
Fond du Lac, WI 54935
920-926-9800

Table of Contents

OVERVIEW:

The proposed project is located on the northeast corner of University Avenue and Summit Avenue within the City of Waukesha, WI. This development involves the placement of a 18,500 sf school addition, parking lot reconfiguration and improvement to the existing parking lot on the east side of the site. The addition and parking area will be constructed over previously developed property. The project will combine the 3 existing lots into 1 overall property. The stormwater plan has accounted for all development shown on the plan. The existing site is being reconfigured and will meet city setback requirements. The existing zoning is I-1 Institutional – as part of City zoning code. The combined site is 8.42 acres and approximately 6 acres will be disturbed as part of the project.

The existing site generally drains from north to south into Summit Avenue's storm system. The post developed site will be routed with 100yr storm pipe to drain to the dry stormwater detention pond on the south side of the property and into the City storm system. A stormwater filter will be installed to treat the redeveloped site for TSS removal. The area consists of a portion of new and redevelopment, therefore the storm filter treats for the prorated amount. See the attached proposed stormwater calculations in Appendix E. The City has made the client aware that flooding exists in the surround neighborhood. A flood analysis for the 100yr storm was provided. Ponding does not occur on the Montessori site during the 100yr flood. All post development flows are less than predevelopment flows for the 2, 10, and 100yr, 24 hour storms.

SOIL INFORMATION:

Existing Soils data: Soil Type: BsA: Brookston silt loam, 0-2% slopes, Hydro. Soil Rating C/D.
HmB: Hochheim loam, 2-6% slopes, Hydro Soil Rating D.
HmC2: Hochheim loam, 6-12% slopes, Hydro Soil Rating D.

Soil classifications for the proposed property were taken off of the USDA Web Soil Survey. Please see attached hydrologic soil group map showing the soils within the drainage areas in Appendix C.

DRAINAGE CALCULATIONS:

Rainfall depths used for the runoff calculations were referenced from The City of Waukesha Stormwater Ordinance Chapter 32.11(a)2. Calculations use Type II distribution.

1-year: 2.3 inches
2-year: 2.7 inches
10-year: 4.0 inches
100-year: 5.6 inches

Curve Numbers:
Impervious – 98

Lawn (B) – 61
Lawn (C) – 74
Lawn (D) – 80
Woods (C) – 70
Woods (D) – 77

WATER QUANTITY

City of Waukesha Requirements – New development shall maintain or reduce the 2, 10, & 100-Year/24 hour post construction peak runoff discharge rates to the 2, 10, & 100-Year /24 hour predevelopment peak runoff discharge rates respectively. Peak discharge rates as required will be maintained or reduced as part of the project. Analyze with tailwater.

Wisconsin Department of Natural Resources – Maintain or reduce the 1-Year/24 hour and the 2-Year/24 hour post construction peak runoff discharge rates to the 1-Year/24 hour and the 2-Year/24 hour predevelopment peak runoff discharge rates respectively.

The analysis has detained the 1, 2, 10, and 100year flows to predevelopment levels with the use of a dry detention pond. The stormwater pond(s) will hold and treat the 100 year storm in the pond and discharge to the storm system prior to Summit Avenue storm system. The pond will tie into the existing stormwater catchbasin for connection to this system (See Appendix E for information). The peak discharge from the site to the existing square grate is maintained or reduced. See below for modeled data. Tailwater was to be analyzed for the ponds. Based on the City's flooding analysis for the 100yr storm the elevation would be to top of Rim (elevation 132.96) in Summit Ave (pond's connection point). Since the tailwater is 3' below the pond's outlet this cannot be modeled and will not have an effect on how the current pond is modeled. North Basin modeled to show 100yr elevation is 150.04 which is below the rims.

SITE RUNOFF SUMMARY:

Runoff Summary Chart for Detention Pond and Offsite flows (in cfs)

Storm (24-hour)	Pre-Development Basin A Runoff (cfs)	Total Post Development Runoff (cfs)	Pond Elevation
1- yr.	3.37	3.26	137.88
2-yr	4.78	4.71	138.15
10yr	9.98	7.99	138.97
100yr	17.03	16.86	139.27

WATER QUALITY

City of Waukesha & Wisconsin DNR Requirements
– Reduce total suspended solids load by 80 percent for new impervious and 40 percent for redevelopment as compared to no controls.

Since a portion of the site was previously developed, a prorated quality requirement has been calculated for the site. In the proposed condition, 5.73 acres will be disturbed. 2.63 acres will be redeveloped and 3.10 acres will be new development. Based on this information, a prorated requirement of 61.6% of the TSS will be required to be removed on site. See SLAMM map table in Appendix B.

	Particulate Solids For drainage Area (lbs)	Particulate Solids Yield after drainage and Controls (lbs)	Particulate Removed
Total Filter w/ offsite	1,595	573.20	1,021.80

SLAMM calculations show that the proposed development with offsite flows meets the quality requirement with a 15 filter UpfloFilter system. See Appendix F for calculations.
Results: 1,021.8/1,595 = 0.6406 => 64.06% Removed, therefore stormwater quality requirements are met.

INFILTRATION:

City of Waukesha, Wisconsin and DNR Requirements (Redevelopment) – site is exempt due to it being a redevelopment per NR 151.124(3)(b)3.

STORM SEWER PIPE DESIGN & 100-YEAR CONVEYANCE:

All storm pipes bringing water to the proposed pond were sized to convey the 100-year storm. See Appendix A, B, D, and E for calculations and basin map. The calculated 100-year storm event will be contained within the proposed stormwater management pond berm and will discharge over the banks after the 100year storm event to the south. Emergency overflows routes are provided on the north side of the site to convey runoff to the east and west of the building and ultimately south overland to the pond. Curb cuts have been provided on site to allow overflow conveyance.

EROSION AND SEDIMENT CONTROL:

The following are practices that will be used to control sediment during construction:
Silt Fence – Silt fence will be placed around the perimeter of the site for perimeter control as well as downhill of any disturbed areas where sheet flow will exist.
Tracking Pads – Stone tracking pads will be placed at all construction entrances to the site to ensure dirt and soil tracked onto public roads is limited.
Ditch Checks – Ditch checks will be provided to reduce the velocity of water flowing in ditch bottoms.
Erosion Matting – Erosion matting will be placed on any steep slopes as well as ditch bottoms to ensure that these areas are permanently stabilized over time.

The erosion control locations, specifications, construction sequence, site stabilization notes, and seeding notes can be seen on civil sheets C1.0 and C1.3.

POST CONSTRUCTION OPERATION AND MAINTENANCE PLAN

FOR:

MONTESSORI SCHOOL OF WAUKESHA – 2600 SUMMIT AVENUE

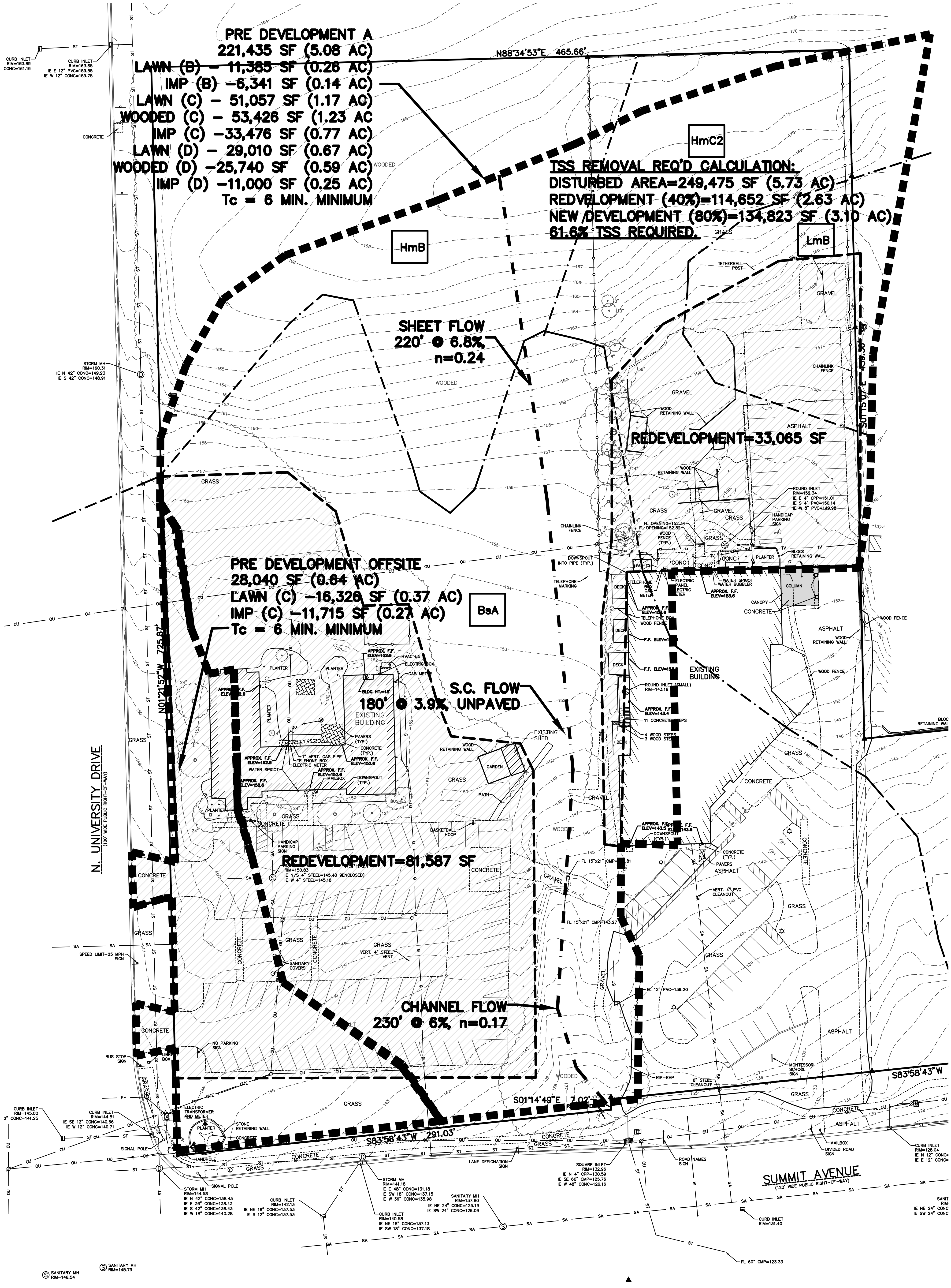
The owner of the property affected shall inspect and maintain the following stormwater management systems frequently, especially after heavy rainfalls, but at least on an annual basis unless otherwise specified.

STORMWATER FACILITY	**TYPE OF ACTION**
1. Lawn and Landscaped Areas	All lawn areas shall be kept clear of any materials that block the flow of stormwater. Rills and small gullies shall immediately be filled and seeded or have sod placed in them. The lawn shall be kept mowed, tree seedlings shall be removed, and litter shall be removed from landscaped areas.
2. Swales	All grassed swales showing signs of erosion, scour, or channelization shall be repaired, reinforced, and revegetated immediately. All swales shall be repaired to the original plan requirements. Mowing shall take place no less than twice per year at a height of no less than three inches. Grasses shall not be allowed to grow to a height that permits branching or bending. Mowing shall only take place when the ground is dry and able to support machinery.
3. Catch Basin Grates	The grate openings to these structures must be cleared of any clogging or the blocking of stormwater flow from getting into the stormwater conveyance system of any kind.
4. Catch Basin Sumps	Sumps shall visually be inspected every 3 months. Siltation shall be removed and disposed of offsite when the sump depth is within 3" of the outlet pipe invert elevation. The removal of siltation should occur a minimum of once per year.

5. Detention Basin	Outlet structures, inlet and outlet pipes shall be kept clear of debris. Non-structurally sound devices shall be replaced. Floating litter and algae shall be removed monthly. All grassed areas, embankments, and flow control devices showing signs of erosion shall be repaired, reinforced, and revegetated immediately to the original plan requirements. Grasses shall not be allowed to grow to a height that permits branching or bending. Mowing shall only take place when the ground is dry and able to support machinery. Every 5 years, beginning in the first summer following completion of the basin (to be completed after detention basin is constructed), the elevations of the pond bottom shall be surveyed to determine the permanent pool depth and sediment depth in the pond. Cleaning, removal, and deposit of silt from the detention pond shall be done by means and methods acceptable to the Wisconsin Department of Natural Resources.
6. Hydro International Up-Flo Filter Quality Structures	Inspection of the structure shall be completed annually at a minimum by qualified maintenance personnel. Sediment in the bottom of the structure shall be inspected to verify sediment is less than 16” deep. If sediment is greater than 16” deep, the sediment shall be removed per Hydro International requirements. Qualified maintenance personnel shall enter structure to remove a Media Bag to be weighed. Media Bags weighing more than 40 lbs are an indication that the bag is full and need to be replaced. Replace per manufacturer specifications. Qualified maintenance personnel shall inspect the oil layer on the water surface to oil being entrained in the Media Bags. If the oil accumulation is greater than 1.5”, the structure shall be pumped per manufacturer’s specifications. After storm events of greater than 1” of rainfall, the structure shall be inspected 48 hours after the rainfall even to verify the water level inside the structure has dropped to below the base of the filter modules. If the water level has not dropped, the filters are considered to be clogged and shall be replaced per manufacturer’s specifications. For further information, obtain Hydro International’s Up-Flo Filter Operation and Maintenance Manual for details.
7. Record of Maintenance	The operation and maintenance plan shall remain onsite and be available for inspection when requested by WDNR and the City of Waukesha. When requested, the owner shall make available for inspection all maintenance records to the department or agent for the life of the system.

Appendix A

Pre-Development Area(s):



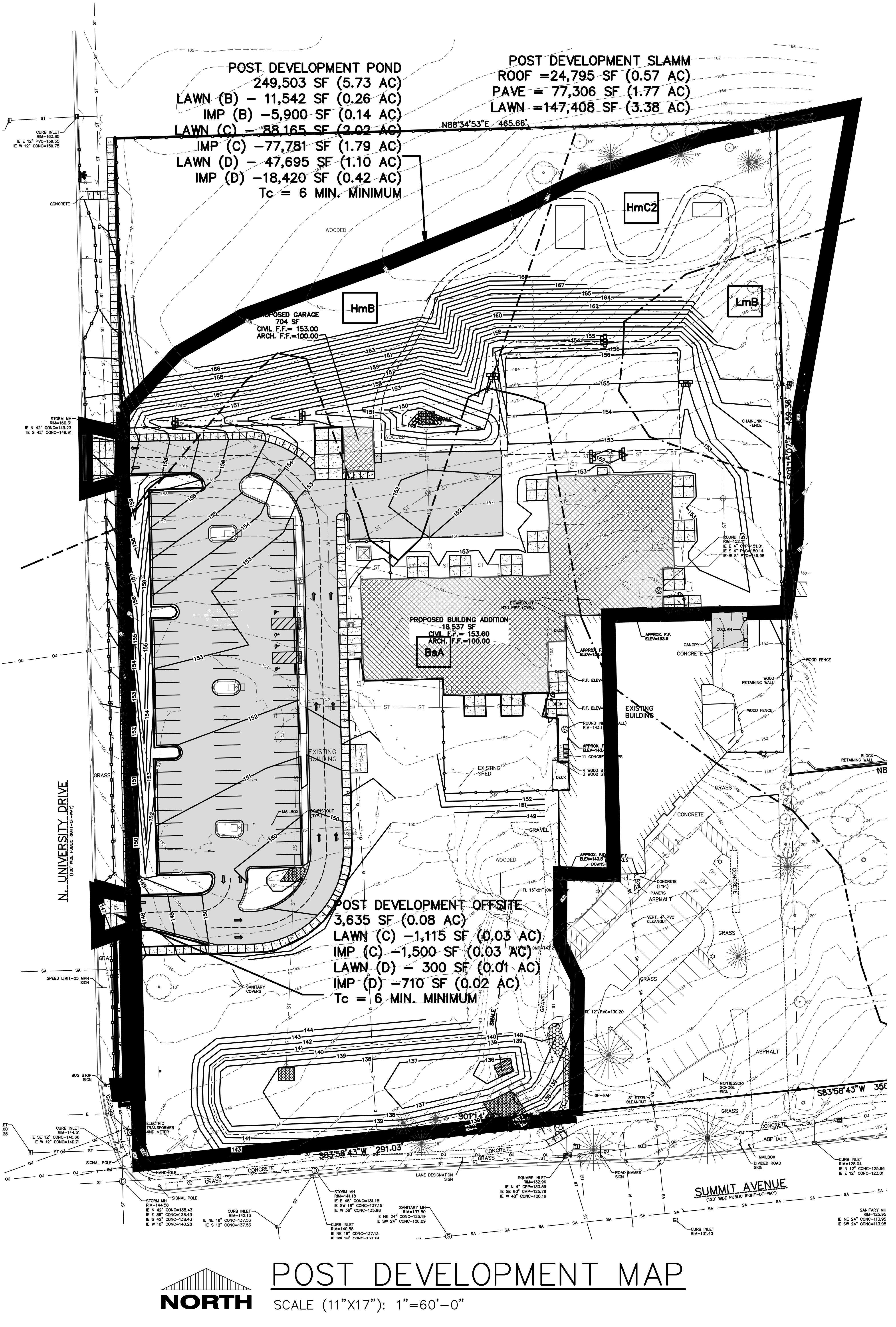
NORTH

PREDEVELOPMENT MAP

SCALE (11"X17"): 1"=60'-0"

Appendix B

Post Development Area(s):



POST DEVELOPMENT MAP

NORTH

SCALE (11"X17"): 1"=60'–0"

Appendix C
Soil Maps & Boring Data

Hydrologic Soil Group—Milwaukee and Waukesha Counties, Wisconsin

Map Scale: 1:1,830 if printed on A portrait (8.5" x 11") sheet.

Map projection: Web Mercator Corner coordinates: WGS84 Edge tics: UTM Zone 16N WGS84

MAP LEGEND

Area of Interest (AOI)

- ⬜ Area of Interest (AOI)

Soils

Soil Rating Polygons

- 🟪 A
- 🟩 A/D
- 🟦 B
- 🟫 B/D
- 🟦 C
- 🟨 C/D
- 🟥 D
- ⬜ Not rated or not available

Soil Rating Lines

- 〰 A
- 〰 A/D
- 〰 B
- 〰 B/D
- 〰 C
- 〰 C/D
- 〰 D
- 〰 Not rated or not available

Soil Rating Points

- ■ A
- ■ A/D
- ■ B
- ■ B/D
- ■ C
- ■ C/D
- ■ D
- □ Not rated or not available

Water Features

- 〰 Streams and Canals

Transportation

- ┼┼┼ Rails
- 〰 Interstate Highways
- 〰 US Routes
- 〰 Major Roads
- 〰 Local Roads

Background

- 🟩 Aerial Photography

MAP INFORMATION

The soil surveys that comprise your AOI were mapped at 1:15,800.

Warning: Soil Map may not be valid at this scale.

Enlargement of maps beyond the scale of mapping can cause misunderstanding of the detail of mapping and accuracy of soil line placement. The maps do not show the small areas of contrasting soils that could have been shown at a more detailed scale.

Please rely on the bar scale on each map sheet for map measurements.

Source of Map: Natural Resources Conservation Service
Web Soil Survey URL:
Coordinate System: Web Mercator (EPSG:3857)

Maps from the Web Soil Survey are based on the Web Mercator projection, which preserves direction and shape but distorts distance and area. A projection that preserves area, such as the Albers equal-area conic projection, should be used if more accurate calculations of distance or area are required.

This product is generated from the USDA-NRCS certified data as of the version date(s) listed below.

Soil Survey Area: Milwaukee and Waukesha Counties, Wisconsin
Survey Area Data: Version 13, Oct 6, 2017

Soil map units are labeled (as space allows) for map scales 1:50,000 or larger.

Date(s) aerial images were photographed: Sep 7, 2014—Sep 22, 2014

The orthophoto or other base map on which the soil lines were compiled and digitized probably differs from the background imagery displayed on these maps. As a result, some minor shifting of map unit boundaries may be evident.

Hydrologic Soil Group

Map unit symbol	Map unit name	Rating	Acres in AOI	Percent of AOI
BsA	Brookston silt loam, 0 to 2 percent slopes	C/D	5.9	41.0%
HmB	Hochheim loam, 2 to 6 percent slopes	D	2.7	18.8%
HmC2	Hochheim loam, 6 to 12 percent slopes, eroded	D	3.8	26.6%
KlA	Kendall silt loam, 1 to 3 percent slopes	C	0.6	4.4%
LmB	Lamartine silt loam, 0 to 3 percent slopes	B/D	1.3	9.2%
Totals for Area of Interest			**14.4**	**100.0%**

Description

Hydrologic soil groups are based on estimates of runoff potential. Soils are assigned to one of four groups according to the rate of water infiltration when the soils are not protected by vegetation, are thoroughly wet, and receive precipitation from long-duration storms.

The soils in the United States are assigned to four groups (A, B, C, and D) and three dual classes (A/D, B/D, and C/D). The groups are defined as follows:

Group A. Soils having a high infiltration rate (low runoff potential) when thoroughly wet. These consist mainly of deep, well drained to excessively drained sands or gravelly sands. These soils have a high rate of water transmission.

Group B. Soils having a moderate infiltration rate when thoroughly wet. These consist chiefly of moderately deep or deep, moderately well drained or well drained soils that have moderately fine texture to moderately coarse texture. These soils have a moderate rate of water transmission.

Group C. Soils having a slow infiltration rate when thoroughly wet. These consist chiefly of soils having a layer that impedes the downward movement of water or soils of moderately fine texture or fine texture. These soils have a slow rate of water transmission.

Group D. Soils having a very slow infiltration rate (high runoff potential) when thoroughly wet. These consist chiefly of clays that have a high shrink-swell potential, soils that have a high water table, soils that have a claypan or clay layer at or near the surface, and soils that are shallow over nearly impervious material. These soils have a very slow rate of water transmission.

If a soil is assigned to a dual hydrologic group (A/D, B/D, or C/D), the first letter is for drained areas and the second is for undrained areas. Only the soils that in their natural condition are in group D are assigned to dual classes.

Rating Options

Aggregation Method: Dominant Condition

Component Percent Cutoff: None Specified

Tie-break Rule: Higher

Geotechnical Engineering Exploration and Analysis

Proposed Improvements
Montessori School of Waukesha
2600 Summit Avenue
Waukesha, Wisconsin

Prepared for:

Excel Engineering, Inc.
Fond du Lac, Wisconsin

July 25, 2018
Project No. 1G-1806024

GILES
ENGINEERING ASSOCIATES, INC.

GILES

ENGINEERING ASSOCIATES, INC.

GEOTECHNICAL, ENVIRONMENTAL & CONSTRUCTION MATERIALS CONSULTANTS

- Atlanta, GA
- Baltimore, MD
- Dallas, TX
- Los Angeles, CA
- Manassas, VA
- Milwaukee, WI

July 25, 2018

Excel Engineering, Inc.
100 Camelot Drive
Fond du Lac, WI 54935

Attention: Mr. Dean Schulz
Project Manager

Subject: Geotechnical Engineering Exploration and Analysis
Proposed Improvements
Montessori School of Waukesha
2600 Summit Avenue
Waukesha, Wisconsin
Project No. 1G-1806024

Dear Mr. Schulz:

As requested, Giles Engineering Associates, Inc. conducted a *Geotechnical Engineering Exploration and Analysis* for the proposed project. The accompanying report describes the services that were performed, and it provides geotechnical-related findings, conclusions, and recommendations that were derived from those services.

We sincerely appreciate the opportunity to provide geotechnical services for the proposed project. Please contact the undersigned if there are questions concerning the report, or if we may be of further service.

Very truly yours,

GILES ENGINEERING ASSOCIATES, INC.

Andrew J. Globig, E.I.T.
Staff Professional I

Anthony C. Giles, P.E.
Vice President

WISCONSIN
ANTHONY C.
GILES
29561-6
OCONOMOWOC
WI
PROFESSIONAL ENGINEER

7-25-18

Distribution: Excel Engineering, Inc.
Attn: Mr. Dean Schulz (2 via USPS, 1 via email: dean.s@excelengineer.com)

N8 W22350 Johnson Drive • Suite A1 • Waukesha, WI 53186
262/544-0118 • Fax 262/549-5868 • E-Mail milwaukee@gilesengr.com

TABLE OF CONTENTS

GEOTECHNICAL ENGINEERING EXPLORATION AND ANALYSIS

PROPOSED IMPROVEMENTS
MONTESSORI SCHOOL OF WAUKESHA
2600 SUMMIT AVENUE
WAUKESHA, WISCONSIN
PROJECT NO. 1G-1806024

GEOTECHNICAL ENGINEERING EXPLORATION AND ANALYSIS

PROPOSED IMPROVEMENTS
MONTESSORI SCHOOL OF WAUKESHA
2600 SUMMIT AVENUE
WAUKESHA, WISCONSIN
PROJECT NO. 1G-1806024

1.0 SCOPE OF SERVICES

This report provides the results of the *Geotechnical Engineering Exploration and Analysis* that Giles Engineering Associates, Inc. (“Giles”) conducted for the proposed development. The *Geotechnical Engineering Exploration and Analysis* included a Geotechnical Subsurface Exploration Program, Geotechnical Laboratory Services, and Geotechnical Engineering Services. The scope of each service area was narrow and limited, as directed by our client, and based on our understanding and assumptions about the proposed project. Service areas are briefly described later.

Geotechnical-related recommendations for design and construction of the foundations and at-grade floor slab for the proposed building addition are provided in this report. Geotechnical-related recommendations are also provided for the planned pavement areas. Furthermore, preliminary information is provided regarding stormwater infiltration at the site. Site preparation recommendations are given, but are only preliminary, as the means and methods of site preparation will depend on factors that were unknown when this report was prepared. Those factors include, but are not limited to, weather before and during construction, subsurface conditions that are exposed during construction, and finalized details of the proposed development. Environmental consulting was beyond our authorized scope of services for this project.

2.0 SITE DESCRIPTION

The subject site is located at the northeast corner of the intersection of N. University Drive and Summit Avenue in Waukesha, Wisconsin. The site address is 2600 Summit Avenue. When the test borings (described later) were performed, the site was occupied by the Montessori School of Waukesha, and included two structures along with areas of asphalt-concrete pavement. Undeveloped areas of the site were generally wooded or grass-covered. The site area is depicted on the *Test Boring Location Plan* (Figure 1 in Appendix A), which was prepared using the *Concept Grading Plan* (dated June 12, 2018) by Excel Engineering, Inc. The site is relatively hilly. Based on topographic contour lines shown on the *Concept Grading Plan*, ground elevations at the site generally range between ±El. 127 and ±El. 172. Neighboring features include Summit Avenue to the south, N. University Drive to the west, an athletic facility to the north, and a residential area to the east.

3.0 PROJECT DESCRIPTION

Building Addition and Pavement Areas

The building addition will be constructed at the northwest corner of the existing Montessori School, as shown on the *Test Boring Location Plan*. According to information that was provided to us, the addition will be an 18,537 square-foot, single-story, wood-frame structure with a wood-truss roof system. The addition will not have a basement or other below-grade spaces. Bearing walls and columns will support the addition. Maximum foundation loads are understood to be 6,000 pounds per lineal foot (plf) from bearing walls, and 75 kips per column. The at-grade floor is planned to be a ground-bearing concrete slab. The maximum floor load is expected to be 100 pounds per square foot (psf).

New parking areas and drives will be constructed at the site, and are shown on the *Test Boring Location Plan*. It is assumed that traffic within the new pavement areas will consist of passenger vehicles with very limited heavy-truck traffic from occasional deliveries and refuse removal. It is also assumed that new pavement is planned to be hot-mix asphalt-concrete, possibly with Portland cement concrete in high-stress areas.

The *Concept Grading Plan* shows that the finish floor of the proposed addition is planned to be at El. 153.47. Topographic contour lines shown on the *Concept Grading Plan* indicate that ground grades within the proposed addition area range between ±El. 152.5 and ±El. 157.5. Therefore, up to about five feet of cut is expected in the addition area, with only minor filling, if any. Based on the *Concept Grading Plan*, significant cutting is also expected in proposed pavement areas.

Preliminary Stormwater Management Device

A dry detention basin is planned to be constructed at the southwest corner of the site, as shown on the *Test Boring Location Plan*. Contour lines on the *Concept Grading Plan* show that the bottom of the basin will slope down to El. 136, and the highest ground grade at the perimeter of the basin will be about El. 144. Up to about seven feet of excavation is expected to be necessary to construct the proposed basin.

4.0 GEOTECHNICAL SUBSURFACE EXPLORATION PROGRAM

To explore subsurface conditions, seven test borings were conducted at the site using a mechanical drill-rig. Test Borings 1 and 2 were in future pavement areas along N. University Drive, and were ±11 feet deep. Test Borings 3 through 6 were in the proposed addition area, and were ±21 feet deep. Test Boring 7 was in the proposed stormwater management area, near Summit Avenue, and was ±16 feet deep. Test-boring locations were positioned (staked) at the site based on measurements from existing site features and apparent property lines, and by approximating right angles. Approximate locations of the test borings are shown on the *Test Boring Location Plan*.

Samples were collected from each test boring, at certain depths, using the Standard Penetration Test (SPT), conducted with the drill rig. A brief description of the SPT is given in Appendix B, along with descriptions of other field procedures. Immediately after sampling, select portions of the SPT samples were placed in containers that were labeled at the site for identification. A Standard Penetration Resistance value (N-value) was determined from each SPT. N-values are reported on the *Test Boring Logs* (in Appendix A), which are records of the test borings. Retained samples were transported to Giles' geotechnical laboratory.

The boreholes were backfilled upon completion; however, backfill material will likely settle and/or heave, possibly creating a hazard that can injure people and animals. Borehole areas should, therefore, be carefully and routinely monitored by the property owner; settlement and/or heave of backfill materials should be repaired immediately. Giles will not monitor or repair boreholes.

The ground elevation at each test boring was estimated using topographic contour lines on the *Concept Grading Plan*. Test-boring elevations are noted on the *Test Boring Logs*, and are considered accurate within about one foot.

5.0 GEOTECHNICAL LABORATORY SERVICES

The retained samples were classified using the descriptive terms and particle-size criteria shown on the *General Notes* in Appendix D, and by using the Unified Soil Classification System (ASTM D 2488-75) as a general guide. Classifications are shown on the *Test Boring Logs*, along with horizontal lines that show estimated depths of material change. Field-related information pertaining to the test borings is also shown on the *Test Boring Logs*. For simplicity and abbreviation, terms and symbols are used on the *Test Boring Logs;* the terms and symbols are defined on the *General Notes*.

Soil samples obtained from Test Boring 7 (conducted in the proposed stormwater management area) were also visually classified using the USDA textural classification system in general accordance with the guidelines provided in the *Field Book for Describing and Sampling Soils* (USDA, Sept. 2012). USDA classifications of the retained samples are shown on the Wisconsin DSPS *Soil Evaluation – Storm* log, enclosed in Appendix A. Supplemental information regarding soil classifications, including the USDA and USCS soil classification systems, is included in the *Soil Classification Notes* enclosure within Appendix D.

Calibrated penetrometer resistance and moisture content tests were performed on select soil samples to evaluate their general engineering properties. Also, a P270 test (percent of material passing the No. 270 sieve) was performed on a soil sample obtained from Test Boring 7. Results of the laboratory tests are on the *Test Boring Logs*. Because SPT samples were used, which are categorized as disturbed samples, results of the calibrated penetrometer tests are considered to be approximate and were used as supplemental information. Test results are on the *Test Boring Logs*. Laboratory procedures are briefly described in Appendix C.

6.0 MATERIAL CONDITIONS

Because material sampling at the test borings was discontinuous, it was necessary to estimate conditions between sample intervals. Estimated conditions at the test borings are briefly discussed in this section, and are described in more detail on the *Test Boring Logs*. The conclusions and recommendations in this report are based only on the estimated conditions.

6.1. Surface Materials

Topsoil was at the surface of the test borings, except at Test Boring 6. The topsoil, which generally consisted of dark brown clayey silt with variable amounts of sand and organic matter, was measured to be between ±6 and ±24 inches thick, depending on the test boring. At Test Boring 6, about 8 inches of crushed-limestone gravel was at the ground surface, and was underlain by about 14 inches of buried topsoil.

6.2. Fill Material

Material classified as fill was beneath the surface materials at Test Boring 7, and extended to about 6½ feet below-ground. The fill material generally consisted of silty clay with estimated little amounts of sand and gravel. Fill material had relatively low to moderate strength characteristics, based on SPT N-values.

6.3. Native Soil

Native soil was below the materials described above, and extended to the termination depth at each test boring. The native soil varied, but generally consisted of sandy silt, silt, and silty fine sand. Based on SPT N-values, these granular soils typically exhibited variable strength characteristics ranging between loose and very dense; however, some N-values are likely not representative of relative density due to interference (during sampling/testing) from gravel, cobbles, and/or boulders, which were typically encountered within the native soil. Cobbles and boulders could be numerous and nested.

7.0 GROUNDWATER CONDITIONS

It is estimated that the water table was about 11 to 16 feet below-ground at the test-boring locations, when the Geotechnical Subsurface Exploration Program was conducted. However, because groundwater was encountered at about 8.2 and 5 feet below-ground at Test Borings 2 and 3, respectively, the site appears to be subject to shallower perched-groundwater conditions, where groundwater perches above the water table. Perched groundwater could be relatively significant, considering the variable subsurface conditions and the topographic relief of the site. Groundwater conditions will likely fluctuate depending on precipitation, surface run-off, and other factors.

The estimated water-table depth discussed above is only an approximation based on the (gray) colors and relative moisture conditions of the retained soil samples. The water table could be higher or lower than estimated. If needed, groundwater observation wells could be installed and observed at the site to more precisely evaluate the water-table depth/elevation. Giles could install and monitor groundwater observation wells, if it is determined that a more precise determination of the water-table depth/elevation is needed.

8.0 CONCLUSIONS AND RECOMMENDATIONS

8.1. Seismic Design Considerations

A soil Site Class C is recommended for seismic design. By definition, Site Class is based on the average properties of subsurface materials to 100 feet below-ground. Because 100-foot test borings were not requested or authorized for the project, it was necessary to estimate the Site Class based on the test borings, presumed area geology, and the International Building Code.

8.2. Building Addition Foundation Recommendations

A spread-footing foundation is recommended for the proposed addition. However, existing fill is unsuitable for direct and/or indirect support of foundations. All footings must bear on suitable-bearing native soil, and/or on new engineered fill that is placed on suitable-bearing native soil. The foundations are recommended to be designed using a 3,000 pound per square foot (psf) maximum, net, allowable soil bearing capacity. For geotechnical considerations, strip-footing pads are recommended to be at least 16 inches wide, and isolated pads are recommended to be at least 24 inches wide, regardless of the calculated foundation-bearing stress. Also, from a geotechnical perspective, foundation walls could be built of cast-in-place concrete or concrete masonry units. It is recommended and assumed that a structural engineer will provide specific foundation details, including footing dimensions, reinforcing, etc.

A minimum 48-inch foundation-embedment depth is required by the local building code. Footings for perimeter walls and other exterior elements of the proposed addition are, therefore, recommended to bear at least 48 inches below the finished ground-grade adjacent to the addition, or to the depth required by the governing building code. Interior footings could be directly below the floor slab since the addition will be heated and it is assumed that support soil will not freeze. The foundation analysis was conducted assuming that perimeter and interior foundations will bear about 4 feet and 1½ feet below the at-grade floor surface, respectively. Using those depths, and the proposed floor elevation (El. 153.47), it is expected that perimeter and interior footings will bear at El. 149.47 and El. 151.97, respectively, referenced to the *Concept Grading Plan*.

The following table provides the estimated depths and elevations of suitable-bearing native soil at Test Borings 3 through 6, which were conducted in the proposed addition area. However, suitable-bearing native soil might be at variable and deeper depths between the test borings, especially near the existing building.

TABLE 1
ESTIMATED DEPTH/ELEVATION OF SUITABLE BEARING NATIVE SOIL (a)

Test Boring	Depth Below Surface (b)	Elevation (c)
3	±4 feet	El. ±149.3
4	±2 feet	El. ±151.5
5	±4 feet	El. ±150.0
6	±4 feet	El. ±151.5

(a) For direct foundation support and/or for placement of engineered fill, based on a 3,000 psf maximum, net, allowable soil bearing capacity.
(b) Referenced to the site grades during the Geotechnical Subsurface Exploration Program.
(c) Referenced to the *Concept Grading Plan* provided by the client.

Considering the foundation-bearing elevations given above, and the depths/elevations of suitable-bearing native soil shown in the previous table, suitable-bearing native soil is expected to be near and below the assumed foundation-bearing elevations, but some relatively minor over-excavation should be expected for interior footings. More extensive over-excavation will likely be necessary along the existing building. Considering the likelihood that unsuitable soil is along the existing building, and also considering the possible presence of lower-strength soil due to shallow perched-groundwater, testing and approval of foundation-support soil by a geotechnical engineer during construction is critical. Without testing and approval of foundation-support soil, by a geotechnical engineer, the addition could be improperly supported.

Foundation excavations are recommended to be dug with a smooth-edge backhoe bucket to develop a relatively undisturbed bearing grade. A toothed bucket will likely disturb foundation-bearing soil more than a smooth-edge bucket, thereby making soil at the excavation base more susceptible to saturation and instability, especially during adverse weather. It is critical that contractors protect foundation-support soil and foundation construction materials (concrete and reinforcing). In addition, engineered fill is recommended to be placed and compacted in benched excavations along foundation walls, immediately after the foundation walls are capable of supporting lateral pressures from backfill, compaction, and compaction equipment. Earth-formed footing construction techniques will likely not be feasible due to caving of granular soil.

Foundation Support Soil Requirements

Existing fill is unsuitable for direct and/or indirect support of foundations. All footings must bear on suitable-bearing native soil, and/or on new engineered fill that is placed on suitable-bearing native soil. Based on the recommended 3,000 psf maximum, net, allowable soil bearing capacity, the in-situ unconfined compressive strength of native cohesive soil, such as silty clay or clayey silt, within foundation influence zones is recommended to be at least 1.5 tons per square foot (tsf). Native granular soil, such as sand, within foundation influence zones is recommended to have a corrected N-value (determined from SPTs and correlated from other in-situ tests) of at least 9, based on the recommended bearing capacity. It is further recommended that the strength characteristics of soil within all foundation influence zones (determined by a geotechnical engineer during construction) meet or exceed the recommended values, unless Giles approves other values based on depth and footing dimensions.

Evaluation of foundation-support soil by a geotechnical engineer during foundation excavation and foundation construction is critical, especially considering the likelihood that unsuitable soil is along the existing building, and also considering the possible presence of lower-strength soil due to shallow perched-groundwater. The purpose of the recommended evaluation is (1) to confirm that the foundations will be properly supported by suitable native soil, (2) to determine over-excavation depths and locations, and (3) to confirm that the support materials are similar to those described on the *Test Boring Logs*. If a firm other than Giles performs the recommended support-soil evaluation, Giles must be notified if the composition or strength characteristics of foundation-support soils differ from those shown on the *Test Boring Logs*, thereby allowing us the opportunity to revise this report, if needed.

Unsuitable materials beneath foundation areas could be replaced with engineered fill, such as well-graded aggregate that has low water-sensitivity. If engineered fill is used as backfill, lateral over-excavation of the unsuitable materials will also be required. The amount of lateral over-excavation will depend on the vertical over-excavation. For budgeting purposes, the minimum lateral over-excavation could be determined by extending an imaginary line outward and downward at a ratio of 1(horizontal):2(vertical) from the bottom edges of a footing pad, but the actual lateral extents of over-excavation are recommended to be approved by a geotechnical engineer during construction.

Lean Portland cement concrete (minimum 28-day compressive strength of 500 psi) could also be used to replace unsuitable materials beneath foundation areas. Where it is used, footing construction must not begin until the lean concrete has gained sufficient strength. Also, over-excavations that are filled with lean concrete are recommended to be at least three inches wider (on all sides) than the footing pad that will be supported by the concrete, and excavation sidewalls are recommended to be plumb and parallel. To help control sloughing and caving, especially due to the granular soil, lean-concrete backfill is recommended to be placed immediately after excavation. This “trench and pour” method requires close communication and scheduling between the general contractor, foundation contractor, geotechnical engineer, and concrete supply company. With a “trench and pour” method, it is critical that a geotechnical engineer observes excavations as they are made.

Existing Building Considerations

Precautions must be taken to protect the existing building during construction, and to ensure that excavations do not undermine or otherwise compromise the existing building or other existing site improvements. If a void develops below existing footings or floor slabs, a geotechnical engineer should immediately observe the conditions and provide repair recommendations. In general, voids should be immediately filled with a concrete dry-pack, or a non-shrink, expansive sand-and-cement slurry should be injected into the void, under

appropriate pressure, to redevelop contact between the foundation and supporting soils.
Within a close proximity of the existing building, it is recommended that foundations for the addition bear at the same elevation as the adjacent (existing) foundations, assuming that the required 48-inch embedment depth will be met, where required. If the new and existing footings will bear at different elevations, a structural engineer should evaluate the stresses to be imposed on the lower foundation, and confirm that the structural integrity of the existing building and addition will be maintained. Control joints should separate the existing building and the addition since some differential movement is expected to occur at these junctures. Excavations must not be performed within the zone of influence (determined by a geotechnical engineer) of an existing footing; otherwise, existing footings could be undermined, possibly causing significant (and catastrophic) damage.

Where new foundations are perpendicular to the existing foundation, it may be necessary to cantilever new foundations a certain distance away from the outside face of the existing building to help reduce potential settlement of the existing building due to overlapping stress from the new construction. When the existing and proposed foundation systems and depths can be confirmed, Giles should be contacted to evaluate whether our recommendations need to be updated. Care must be taken to protect the existing building during construction of the addition. The existing building should be underpinned and braced, where needed. Extra care should be exercised not to undermine existing footings during removal of unsuitable materials, and during construction of the new footings.

It is assumed that the proposed addition will be a self-supporting structure, and that no structural load will be imposed on the existing building due to the addition. If load is added to the existing building, it will likely undergo some settlement. The amount and location of settlement will partly depend on the magnitude and location of the load increase. Differential settlement should be expected between the existing building and the addition, even if additional load is not imposed on the existing building.

Estimated Foundation Settlement

The post-construction total and differential settlements of a spread-footing foundation designed and constructed based on this report are estimated to be less than about 1.0 inch and 0.5 inch, respectively. The post-construction angular distortion is estimated to be less than about 0.002 inch per inch across a distance of 20 feet or more. Estimated settlements are based on the assumption that foundation-support soil will be thoroughly tested and approved by a geotechnical engineer during foundation excavation and foundation construction.

8.3. At-Grade Floor Slab Recommendations

Based on the proposed floor elevation, and with proper subgrade preparation, it is expected that site soil (including existing fill) will be suitable for support of an at-grade floor slab for the proposed addition. Over-excavation and/or improvement of unsuitable soil might, however, be necessary to develop a suitable subgrade, considering the likelihood that unsuitable soil is along the existing building, and due to the possible presence of lower-strength soil associated with shallow perched-groundwater. Engineered fill that is selected, placed, and compacted according to this report could also support a concrete floor slab.

Assuming a maximum 100 psf floor load, and with regard to geotechnical considerations, the floor slab is recommended to be at least 4 inches thick. The recommended thickness assumes that the 28-day compressive strength of concrete will be at least 3,500 pounds per square inch (psi). At-grade floor slabs could be designed based on a *Modulus of Subgrade Reaction* (K_{V1}) value of 100 pounds per square inch per inch (psi/in). It is recommended and assumed that a structural engineer will specify the actual floor slab thickness, reinforcing, joint details, and other parameters.

A minimum 4-inch-thick base course is recommended to be below the floor slab to serve as a capillary break and for support considerations. It is recommended that the base course consist of free-draining aggregate that has been tested and approved by a geotechnical engineer. Depending on aggregate gradation, a geotextile might need to be below the base course to serve as a separator. The need for a geotextile should be determined during construction, with the assistance of a geotechnical engineer.

A minimum 10-mil vapor retarder is recommended to be directly above or below the base course throughout the entire floor area. The location (above or below the base course) of the vapor retarder should be specified by the project structural engineer or architect. Abutting vapor retarder sheets are recommended to be overlapped and taped, and must extend to all foundation walls. Vapor retarders are recommended to be in accordance with ASTM E 1745, entitled: *Standard Specification for Plastic Water Vapor Retarders Used in Contact with Soil or Granular Fill under Concrete Slabs*, and/or other relevant documents. If the base-course material has sharp, angular aggregate, protecting the retarder with a geotextile (or by other means) is recommended.

Due to the frost-susceptible site soil, and shallow perched-groundwater, areas of the floor slab (such as near exterior doors and entrance/exit vestibules), and sidewalks, will likely be susceptible to freeze-thaw related movement. Installation of insulation (or other protective measures against freeze-thaw movement) should be considered for these areas. Pavement and ground grades are recommended to be sloped away from the building and sidewalks to reduce water infiltration and potential freeze-thaw problems.

Estimated Floor Slab Settlement

The post-construction total and differential settlements of an isolated floor slab constructed in accordance with this report are estimated to be less than about 0.5 inch and 0.3 inch, respectively, over a distance of about 20 feet. Estimated settlements are based on the assumption that floor slab support materials will be thoroughly tested and approved by a geotechnical engineer during the earthwork operations (including subgrade preparation and fill placement), and immediately before floor slab construction.

8.4. Pavement Recommendations

Giles was not given information regarding the expected traffic conditions within new pavement areas. Therefore, to provide pavement recommendations, it was necessary to use an arbitrarily selected traffic condition. The pavement sections given below are for a maximum daily traffic condition consisting of five 18,000-pound equivalent single axle loads (ESALs). The pavement sections are only for light-duty areas, such as areas that are subject to passenger vehicles along with occasional, infrequent heavy vehicles. Giles could provide recommendations for a heavier traffic condition after specific details regarding the expected traffic are provided to us.

It is recommended that the project owner, developer, civil engineer, and other design professionals involved with the project confirm that the arbitrarily selected traffic condition is appropriate. If requested, Giles can provide supplemental pavement recommendations based on other traffic conditions. If the pavement sections are subject to a traffic condition greater than assumed, increased maintenance and premature failure could occur.

It was not within Giles' scope to conduct California Bearing Ratio (CBR) testing (used to determine soil support parameters for pavement design) on pavement support materials; therefore, to give pavement recommendations, it was necessary to assume a CBR value. Based on the test borings, it is assumed that the pavement subgrade will consist of native granular soil (sandy silt, silt, and silty fine sand) with an estimated field CBR value of at least 10. Engineered fill that is placed in proposed pavement areas is recommended to have a CBR value equal to or greater than 10, and the fill is recommended to be placed and compacted per this report.

The recommended asphalt-concrete pavement section is shown in the following table. The pavement section is based on the assumed traffic condition and the assumed CBR value. Due to shallow-perched groundwater, and the expected cutting depths in future pavement areas, the subgrade might need to be improved, especially if construction is during or after adverse weather. There are various methods of subgrade improvement, including the use of geogrid, coarse-aggregate modification, and soil stabilization with hydrated lime or Portland cement. The need for subgrade improvement should be determined during construction with the assistance of a geotechnical engineer.

Considering that the site is subject to shallow perched-groundwater, a geotextile fabric is recommended to be directly below the base course to serve as a separator; geotextile is recommended to be placed on a properly prepared subgrade in accordance with the geotextile manufacturer's recommendations.

TABLE 2
RECOMMENDED ASPHALT-CONCRETE PAVEMENT SECTION

Material	Pavement Section Thickness	Wisconsin DOT Standard Specifications
Hot-Mix Asphalt Surface Course	1.5 inches	Section 460
Hot-Mix Asphalt Binder Course	2.0 inches	Section 460
Dense-Graded Aggregate Base Course	8.0 inches	Section 305 1¼-inch Crushed Stone
Geotextile Fabric	Mirafi® 150N (or similar geotextile approved by Giles)	Section 645
EASL = 18-kip equivalent single axle loads (per day)		

For light-duty conditions, a minimum 6-inch-thick Portland cement concrete (PCC) pavement with a minimum 4-inch-thick compacted aggregate base course is recommended for high-stress areas, such as at entrance/exit aprons, at a trash/recyclables enclosure, and in areas where trucks will turn or will be parked. The concrete should have a minimum 28-day compressive strength of 4,000 psi with 4 to 7 percent air entrainment. Control-joint spacing should be determined in accordance with the current ACI code. Expansion joints should be provided where pavement abuts fixed objects, such as light poles. Materials and construction procedures for concrete pavement are recommended to be per Wisconsin DOT Standard Specifications Section 415 for concrete and Section 305 for base course. The geotextile fabric discussed above is recommended to be directly below the base course of PCC pavement to serve as a separator; geotextile is recommended to be placed on a properly prepared subgrade in accordance with the geotextile manufacturer's recommendations

Pavement Drainage Considerations

Due to shallow perched-groundwater, a drain system is recommended to be below the new pavement areas to collect and remove water. Installing an under-pavement drain system could increase the service life of the new pavement, it could help preserve the condition of the pavement, and it could reduce the need for non-routine maintenance and repair of the pavement. However, even with an under-pavement drainage system, pavement damage and other problems should be expected due to freeze-thaw of the frost-susceptible subgrade materials.

It is recommended that a civil engineer design the under-pavement drainage system based on details of the site. If possible, the under-drain system should include finger drains, along with a sloped subgrade, that discharges water to catch basins with weep holes. While the primary purpose of the drainage system is to collect groundwater from pavement areas, the drainage system is recommended to be configured to collect (intercept) water from beneath sidewalks and other flatwork.

Even with the recommended drainage system, frequent maintenance and repair of the pavement may be necessary due to shallow perched-groundwater and frost-susceptible site soil. Pavement damage and other problems should be expected due to frost-heave and subsequent thaw-related strength loss of subgrade soil. Frost-heave could be significant.

General Pavement Considerations

The pavement recommendations assume that the subgrade will be prepared per report, the base course will be properly drained, and a geotechnical engineer will observe pavement construction. Pavement was designed based on AASHTO parameters for a twenty-year design period. Pavement maintenance along with a major rehabilitation after about 8 to 10 years should be expected. Local codes may require specific testing to determine soil support characteristics and/or a minimum pavement section thickness might be required.

8.5. Initial Stormwater Infiltration Screening

Test Boring 7 was performed in the proposed stormwater management area to initially screen for the possibility of infiltrating stormwater. At that test boring, silty clay (classified as fill) was present to about 6½ feet below-ground (±El. 135), and was underlain by native soil that generally consisted of silty fine sand. Considering that the bottom of the basin will slope down to El. 136, and the subsoil conditions at Test Boring 7, it is expected that an infiltration rate of at least 0.5 inch per hour could be used for design of stormwater management devices; however, the actual infiltration rate will depend on the textural characteristics of soils beneath the stormwater management area, including the “percent fines” of the soil. The Wisconsin Department of Natural Resources document titled *Site Evaluation for Stormwater Infiltration* defines percent fines as the percentage of soil that passes the No. 200 sieve. The actual infiltration rate will also depend on the in-place density, or compactness, or site soil. According to the *State of Wisconsin Administrative Code*, a “filtering layer” consisting of at least a 5-foot-thick soil layer with at least 10% fines, or a 3-foot-thick layer with at least 20% fines, must be between the bottom of the infiltration system and seasonal-high groundwater. Based on Test Boring 7, it is expected that this condition will be met. Per Wisconsin requirements, design infiltration rates must be determined from test pits conducted within proposed infiltration areas. Giles could conduct the required test-pit evaluation, upon request and authorization. Over-excavation of unsuitable soil (such as silty clay) from the proposed basin area is expected to be necessary, based on Test Boring 7 and the planned basin elevations.

8.6. Generalized Site Preparation Recommendations

This section deals with site preparation, including preparation of floor slab, pavement, and engineered fill areas. The means and methods of site preparation will greatly depend on the weather conditions before and during construction, the subsurface conditions that are exposed during earthwork operations, and the finalized details of the proposed development. Therefore, only generalized site preparation recommendations are given.

In addition to being generalized, the following site preparation recommendations are abbreviated; the *Guide Specifications* in Appendix D gives further recommendations. The *Guide Specifications* should be read along with this section. Also, the *Guide Specifications* are recommended to be used as an aid to develop the project specifications.

Demolition, Removal, and Stripping

It is understood that the structure within the western portion of the site will be razed. All components of the existing building are recommended to be removed from the proposed building area. Disposal of rubble and debris is recommended to be in accordance with local, state, and federal regulations for the material type. Outside the proposed building area, it may be feasible for existing foundations to remain, provided the foundations are stable, are cut off at least 3 feet below the planned subgrade, and hollow cores are grouted solid. Remaining floor slabs that are outside the proposed building area could also stay in-place, provided that the slabs are at least 3 feet below the planned finished grade, are perforated (broken) on a maximum 2-foot grid, are “seated” into the subgrade for stability, and are covered with a minimum 12-inch-thick layer of well-graded, free-draining, granular material for drainage. It is important to note that building remnants that are left in-place may cause excavation difficulties for new utilities and landscape plantings, and for future construction. Excavations created during removal of construction components must be backfilled with engineered fill, which might need to be benched into the surrounding soil, as noted in Item No. 3 of the *Guide Specifications* enclosed in Appendix D.

Existing pavement, surface vegetation, trees and bushes (including root-balls), topsoil with adverse organic content, and otherwise unsuitable bearing materials are recommended to be removed from the proposed addition footprint, pavement areas, and other structural areas. Stripping should extend at least several feet beyond proposed development area, where feasible.

Proof-Rolling and Fill Placement

After the recommended demolition, removal, and stripping, and once the development areas are cut (lowered) as needed, subgrades are recommended to be proof-rolled with a fully-loaded, tandem-axle dump truck, or other suitable construction equipment, to help locate unstable areas based on subgrade deflection caused by the wheel loads of the proof-roll equipment. However, proof-roll equipment must be kept a sufficient distance from the existing building, and other existing construction, as existing construction could be damaged during proof-rolling. For safety, proof-roll equipment must also be kept a sufficient distance from excavations. It is recommended that a geotechnical engineer observe proof-roll operations, and evaluate subgrade stability based on those observations. Areas that cannot be proof-rolled (such as near the existing building and existing pavement) are recommended to be evaluated (and approved) by a geotechnical engineer using appropriate means and methods.

Due to lower-strength soil, shallow perched-groundwater, and soil with a high moisture-content, it is expected that unsuitable soil will be encountered during proof-rolling/testing. Unsuitable materials are recommended to be removed and replaced with engineered fill, or otherwise improved. Recommendations for subgrade improvement should, however, be made by a geotechnical engineer based on the site conditions during construction. Depending on the conditions that are encountered, areas requiring soil improvement might be large, and improvement methods might need to extend up to several feet below the planned subgrade. Extensive subgrade improvement should be expected in some areas, based on the test borings. Areas requiring subgrade improvement should be defined during construction with the assistance of a geotechnical engineer. Also, specific improvement methods should be determined during construction on an area-by-area basis. Where subgrade improvement is needed, it might be necessary/beneficial to construct “test strips” to determine the most cost-effective and appropriate means of developing a suitable subgrade.

Construction areas are recommended to be raised, where necessary, to the planned finished grade with engineered fill immediately after the subgrade is confirmed to be stable and suitable to support the proposed site improvements. Engineered fill is recommended to be placed in uniform, relatively thin layers (lifts). And each layer of engineered fill is recommended to be compacted to at least 95 percent of the fill material’s maximum dry density determined from the Standard Proctor compaction test (ASTM D698). As an exception, the in-place dry density of engineered fill within one foot of the pavement subgrade is recommended to be compacted to at least 100 percent of the fill’s maximum dry density. The water content of fill material is recommended to be uniform and within a narrow range of the optimum moisture content, also determined by the Standard Proctor compaction test. Item Nos. 4 and 5 of the *Guide Specifications* give move information pertaining to selection and compaction of engineered fill.

Care must be taken not to damage the existing building (or other existing construction) during compaction of engineered fill. In some areas (such as along the existing building and along existing pavement), it will likely be necessary to use walk-behind vibratory compaction equipment, possibly along with imported aggregate fill materials. Also, vibratory compaction equipment should not be used near groundwater (including perched groundwater), since vibratory compaction near groundwater could cause soil to become unstable.

Engineered fill that does not meet the density and water content requirements is recommended to be replaced with new fill, or scarified to a sufficient depth (likely 6 to 12 inches, or more), moisture-conditioned, and compacted to the required density. A subsequent lift of fill should only be placed after a geotechnical engineer confirms that the previous lift was properly placed and compacted. Subgrade soil will likely need to be recompacted immediately before construction since equipment traffic and adverse weather may reduce soil stability.

Use of Site Soil as Engineered Fill

Site soil that does not contain adverse organic content or other deleterious materials, as noted in the *Guide Specifications,* could be used as engineered fill. However, site soil will likely need to be moisture conditioned (uniformly moistened or dried) prior to being used as engineered fill. If construction is during adverse weather (discussed in the following section), drying site soil will likely not be feasible. In that case, aggregate fill (or other fill material with a low water-sensitivity) will likely need to be imported to the site. Additional recommendations regarding fill selection, placement and compaction are given in the *Guide Specifications*.

8.7. Generalized Construction Considerations

Adverse Weather

Site soil is moisture sensitive and will become unstable when exposed to adverse weather, such as rain, snow, and freezing temperatures. Therefore, it might be necessary to remove or stabilize the upper 6 to 12 inches (or more) of soil due to adverse weather, which commonly occurs during late fall, winter, and early spring. At least some over-excavation and/or stabilization of unstable soil should be expected if construction is during or after adverse weather. Because site preparation is weather dependent, bids for site preparation, and other earthwork activities, should consider the time of year that construction will be conducted.

In an effort to protect soil from adverse weather, the site surface is recommended to be smoothly graded and contoured during construction to divert surface water away from construction areas. Contoured subgrades are recommended to be rolled with a smooth-drum compactor, before precipitation, to “seal” the surface. Furthermore, construction traffic should be restricted to certain aggregate-covered areas in an effort to reduce traffic-related soil disturbance. Foundation, floor slab, and pavement construction should begin immediately after suitable support is confirmed.

Dewatering

Based on the assumed elevations, excavations are expected to be above the water table. However, dewatering might be necessary during construction due to perched groundwater and due to precipitation. Filtered sump pumps, drawing water from sump pits, will likely be adequate to remove water that collects in shallow excavations. Excavated sump pits should be lined with a geotextile and filled with open-graded, free-draining aggregate.

Existing Fill Considerations

The site has been developed and existing fill was encountered at Test Boring 7. Unsuitable materials may have been buried during previous site grading and fill placement. Potentially unsuitable materials, where encountered, are recommended to be evaluated by a geotechnical engineer to determine if removal and replacement with engineered fill is necessary. Disposal of unsuitable materials should be in accordance with local, state and federal regulations. This report might need to be revised if conditions encountered during construction differ from those shown on the *Test Boring Logs*.

Excavation Stability

Excavations are recommended to be made in accordance with current OSHA excavation and trench safety standards, and other applicable requirements. Sides of excavations might need to be benched, sloped, and/or braced to maintain or develop a safe work environment. Temporary shoring must be designed according to applicable regulatory requirements. Contractors are responsible for excavation safety.

Existing Utilities

Existing utilities are recommended to be located, and any that are planned to be maintained should be relocated outside the addition area, if possible. Utilities that are not reused should be capped-off and removed, or properly abandoned in-place in accordance with local codes and ordinances. Utility-removal excavations are recommended to be backfilled with engineered fill placed under engineering controlled conditions. Grading operations must be done carefully so that existing utilities are not damaged or disturbed. Utility elevations, depths, and sizes should be checked relative to the planned construction, including the planned foundation elevations.

8.8. Recommended Construction Materials Testing Services

This report was prepared assuming that a geotechnical engineer will perform Construction Materials Testing ("CMT") services during construction of the proposed development. Supplemental geotechnical recommendations may be needed based on the results of CMT services and specific details of the project not known at this time.

9.0 BASIS OF REPORT

This report is strictly based on the project description given earlier in this report. Giles must be notified if any part of the project description or our assumptions are not accurate so that this report can be amended, if needed. This report is based on the assumption that the facility will be designed and constructed according to the codes that govern construction at the site.

The conclusions and recommendations in this report are based on estimated subsurface conditions as shown on the *Test Boring Logs*. Giles must be notified if the subsurface conditions that are encountered during construction of the proposed development differ from those shown on the *Test Boring Logs* because this report will likely need to be revised. General comments and limitations of this report are given in the appendix.

The conclusions and recommendations presented in this report have been promulgated in accordance with generally accepted professional engineering practices in the field of geotechnical engineering. No other warranty is either expressed or implied.

1G-1806024-report/18Geo02/ajg/

APPENDIX A

FIGURES AND TEST BORING LOGS

The Test Boring Location Plan contained herein was prepared based upon information supplied by *Giles'* client, or others, along with *Giles'* field measurements and observations. The diagram is presented for conceptual purposes only and is intended to assist the reader in report interpretation.

The Test Boring Logs and related information enclosed herein depict the subsurface (soil and water) conditions encountered at the specific boring locations on the date that the exploration was performed. Subsurface conditions may differ between boring locations and within areas of the site that were not explored with test borings. The subsurface conditions may also change at the boring locations over the passage of time.

LEGEND:

①	GEOTECHNICAL TEST BORING

NOTES:

1.) TEST BORING LOCATIONS ARE APPROXIMATE.

2.) BASE MAP DEVELOPED FROM THE "CONCEPT GRADING PLAN" (SHEET C1.3), DATED 6-12-18, PREPARED BY EXCEL ENGINEERING.

3.) EXISTING STRUCTURES (IN BLUE) ARE APPROXIMATE BASED ON THE "EXISTING SITE AND DEMOLITION PLAN (SHEET C1.1), DATED 6-12-18, PREPARED BY EXCEL ENGINEERING.

GILES ENGINEERING ASSOCIATES, INC.
N8 W22350 JOHNSON DRIVE, SUITE A1
WAUKESHA, WI 53186 (262)544-0118
www.gilesengr.com

FIGURE 1
TEST BORING LOCATION PLAN
PROPOSED IMPROVEMENTS
MONTESSORI SCHOOL OF WAUKESHA
2600 SUMMIT AVENUE AND 601 N. UNIVERSITY DRIVE
WAUKESHA, WISCONSIN

DESIGNED	DRAWN	SCALE	DATE	REVISED
BMS/AJG		approx. 1"=80'	07-25-18	--
PROJECT NO.: 1G-1806024			CAD No. 1g1806024-blp	

BORING NO. & LOCATION:
1

SURFACE ELEVATION:
152 feet

COMPLETION DATE:
07/05/18

FIELD REP:
CHARLES RENS

TEST BORING LOG

PROPOSED BUILDING & PARKING LOT ADDITIONS

MONTESSORI SCHOOL OF WAUKESHA
2600 SUMMIT AVENUE & 601 N. UNIVERSITY DRIVE
WAUKESHA, WISCONSIN

PROJECT NO: 1G-1806024

GILES ENGINEERING ASSOCIATES, INC.

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
± 24" Topsoil: Dark Brown Clayey Silt, little Sand and Organic Matter-Moist			1-SS	5						
		150								
Brown and Gray Sandy Silt, little Gravel-Very Moist to Wet			2-SS	5				18		
Light Brown Silty fine Sand, little Gravel-Moist (contains Cobbles and Boulders)	5		3-SS	42						
		145								
			4-SS	15				9		
	10		5-SS	46				8		

Boring Terminated at about 11 feet (EL. 141')

	Water Observation Data	Remarks:
▽	Water Encountered During Drilling:	
▼	Water Level At End of Drilling:	
░	Cave Depth At End of Drilling: 7 ft.	
▼	Water Level After Drilling:	
▒	Cave Depth After Drilling:	

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

BORING NO. & LOCATION: 2	
SURFACE ELEVATION: 157.3 feet	
COMPLETION DATE: 07/05/18	
FIELD REP: CHARLES RENS	

TEST BORING LOG

PROPOSED BUILDING & PARKING LOT ADDITIONS

MONTESSORI SCHOOL OF WAUKESHA
2600 SUMMIT AVENUE & 601 N. UNIVERSITY DRIVE
WAUKESHA, WISCONSIN

PROJECT NO: 1G-1806024

GILES ENGINEERING ASSOCIATES, INC.

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
± 6" Topsoil: Dark Brown Clayey Silt, little Sand and Organic Matter-Moist			1-SS	6				8		
Gray and Brown Sandy Silt, little Gravel-Very Moist		155	2-SS	15				10		
Light Brown Silty fine Sand, little Gravel-Very Moist (contains Cobbles and Boulders)	5		3-SS	10				9		
	▽	150	4-SS	17				10		
	10		5-SS	25						(a)

Boring Terminated at about 11 feet (EL. 146.3')

	Water Observation Data
▽	Water Encountered During Drilling: 8.2 ft.
▼	Water Level At End of Drilling:
░	Cave Depth At End of Drilling: 7.5 ft.
▼	Water Level After Drilling:
▒	Cave Depth After Drilling:

Remarks:

(a) No sample recovery

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

BORING NO. & LOCATION: 3	
SURFACE ELEVATION: 153.3 feet	
COMPLETION DATE: 07/05/18	
FIELD REP: CHARLES RENS	

TEST BORING LOG

PROPOSED BUILDING & PARKING LOT ADDITIONS

MONTESSORI SCHOOL OF WAUKESHA
2600 SUMMIT AVENUE & 601 N. UNIVERSITY DRIVE
WAUKESHA, WISCONSIN

PROJECT NO: 1G-1806024

GILES ENGINEERING ASSOCIATES, INC.

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
± 12" Topsoil: Dark Brown Clayey Silt, trace Sand and Organic Matter-Moist			1-SS	5		2.5		25		
Brown and Gray Silt, little fine Sand-Moist			2-SS	8				18		
		150								
	▼ 5		3-SS	17				15		
Light Brown Silty fine Sand, little Gravel-Moist (contains Cobbles and Boulders)			4-SS	21				9		
		145								
	10		5-SS	47				7		
Gray Silty fine Sand, little Gravel-Moist (contains Cobbles and Boulders)		140								
	15		6-SS	63				7		
		135								
	20		7-SS	74						

Boring Terminated at about 21 feet (EL. 132.3')

	Water Observation Data	Remarks:
▽	Water Encountered During Drilling:	
▼	Water Level At End of Drilling: 5 ft.	
░	Cave Depth At End of Drilling: 5.5 ft.	
▼	Water Level After Drilling:	
▒	Cave Depth After Drilling:	

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

BORING NO. & LOCATION: 4

SURFACE ELEVATION: 153.5 feet

COMPLETION DATE: 07/05/18

FIELD REP: CHARLES RENS

TEST BORING LOG

PROPOSED BUILDING & PARKING LOT ADDITIONS

MONTESSORI SCHOOL OF WAUKESHA
2600 SUMMIT AVENUE & 601 N. UNIVERSITY DRIVE
WAUKESHA, WISCONSIN

PROJECT NO: 1G-1806024

GILES ENGINEERING ASSOCIATES, INC.

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
± 13" Topsoil: Dark Brown Silty Clay, trace Sand and Organic Matter-Moist			1-SS	8		2.4		27		
Light Brown Sandy Silt, little Gravel-Very Moist			2-SS	11						
		150								
	5		3-SS	9				9		
			4-SS	12				8		
		145								
Brown Silty fine Sand with Gravel-Moist (contains Cobbles and Boulders)	10		5-SS	43						
Gray Silty fine Sand, little Gravel-Very Moist to Wet (contains Cobbles and Boulders)		140								
	▒									
	▼ 15		6-SS	42						
		135								
	▽ 20		7-SS	50/5"						(a)
Boring Terminated at about 21 feet (EL. 132.5')										

	Water Observation Data
▽	Water Encountered During Drilling: 19.5 ft.
▼	Water Level At End of Drilling: 15.5 ft.
▒	Cave Depth At End of Drilling: 14 ft.
▼	Water Level After Drilling:
▓	Cave Depth After Drilling:

Remarks:

(a) Poor sample recovery

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

BORING NO. & LOCATION: 5

SURFACE ELEVATION: 154 feet

COMPLETION DATE: 07/05/18

FIELD REP: CHARLES RENS

TEST BORING LOG

PROPOSED BUILDING & PARKING LOT ADDITIONS

MONTESSORI SCHOOL OF WAUKESHA
2600 SUMMIT AVENUE & 601 N. UNIVERSITY DRIVE
WAUKESHA, WISCONSIN

PROJECT NO: 1G-1806024

GILES ENGINEERING ASSOCIATES, INC.

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
± 16" Topsoil: Dark Brown Clayey Silt, trace Sand and Organic Matter-Moist			1-SS	6		1.5		27		
Brown and Gray Silt, little fine Sand-Moist to Very Moist			2-SS	8		3.0		15		
		150								
Light Brown Silty fine Sand, little Gravel-Moist to Very Moist (contains Cobbles and Boulders)	5		3-SS	12				8		
			4-SS	28				9		
		145								
	10		5-SS	19				8		
Gray Silty fine Sand, little Gravel-Moist (contains Cobbles and Boulders)		140								
	15		6-SS	59				7		
		135								
	20		7-SS	55						(a)

Boring Terminated at about 21 feet (EL. 133')

	Water Observation Data	Remarks:
▽	Water Encountered During Drilling:	(a) Poor sample recovery
▼	Water Level At End of Drilling:	
░	Cave Depth At End of Drilling: 18 ft.	
▼	Water Level After Drilling:	
▒	Cave Depth After Drilling:	

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

BORING NO. & LOCATION: 6	
SURFACE ELEVATION: 155.5 feet	
COMPLETION DATE: 07/05/18	
FIELD REP: CHARLES RENS	

TEST BORING LOG

PROPOSED BUILDING & PARKING LOT ADDITIONS

MONTESSORI SCHOOL OF WAUKESHA
2600 SUMMIT AVENUE & 601 N. UNIVERSITY DRIVE
WAUKESHA, WISCONSIN

PROJECT NO: 1G-1806024

GILES ENGINEERING ASSOCIATES, INC.

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
± 8" Crushed Limestone Gravel **± 14" Buried Topsoil:** Dark Brown Clayey Silt, little Sand and Organic Matter-Moist		155	1-SS	4						
Brown and Gray Silt, trace Sand-Moist			2-SS	8		1.5		22		
	5	150	3-SS	10		2.3		19		
Brown Silty fine to medium Sand-Wet			4-SS	12						
Light Brown Silty fine Sand, trace Gravel-Very Moist to Wet	10	145	5-SS	8				10		
Gray Silty fine Sand, trace to little Gravel-Very Moist	15	140	6-SS	14				9		
	20	135	7-SS	15				11		(a)

Boring Terminated at about 21 feet (EL. 134.5')

	Water Observation Data	Remarks:
▽	Water Encountered During Drilling:	(a) Poor sample recovery
▼	Water Level At End of Drilling:	
░	Cave Depth At End of Drilling: 8 ft.	
▼	Water Level After Drilling:	
▒	Cave Depth After Drilling:	

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

BORING NO. & LOCATION:
7

SURFACE ELEVATION:
141.5 feet

COMPLETION DATE:
07/05/18

FIELD REP:
CHARLES RENS

TEST BORING LOG

PROPOSED BUILDING & PARKING LOT ADDITIONS

MONTESSORI SCHOOL OF WAUKESHA
2600 SUMMIT AVENUE & 601 N. UNIVERSITY DRIVE
WAUKESHA, WISCONSIN

PROJECT NO: 1G-1806024

GILES ENGINEERING ASSOCIATES, INC.

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
± 12" Topsoil: Dark Brown Clayey Silt, little Sand and Organic Matter-Moist		140	1-SS	6						
Fill: Brown and Gray Silty Clay, little Sand and Gravel-Moist (contains Cobbles)			2-SS	16						(a)
	5		3-SS	8				20		
Light Brown Silty fine Sand, little Gravel-Very Moist (contains Cobbles and Boulders)		135	4-SS	18				9		P270=32.5%
	10		5-SS	13				11		
		130	6-SS	8						
	15		7-SS	27				11		(a)

Boring Terminated at about 16 feet (EL. 125.5')

	Water Observation Data	Remarks:
▽	Water Encountered During Drilling:	(a) No sample recovery
▼	Water Level At End of Drilling:	
░	Cave Depth At End of Drilling: 13.5 ft.	
▼	Water Level After Drilling:	
▒	Cave Depth After Drilling:	

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

WISCONSIN DEPARTMENT OF SAFETY AND PROFESSIONAL SERVICES
DSPS

SOIL AND SITE EVALUATION – STORM

Division of Industry Services
P. O. Box 2658
Madison, Wisconsin 53701
Scott Walker, Governor
Laura Gutierrez, Secretary

In accordance with SPS 382.365, 385, Wis. Adm. Code, and WDNR Standard 1002

Attach a complete site plan on paper not less than 8 ½ x 11 inches in size. Plan must include, but not limited to: vertical and horizontal reference point (BM), direction and percent of slope, scale or dimensions, north arrow, and BM referenced to nearest road

Please print all information

Personal information you provide may be used for secondary purposes [Privacy Law, s. 15.04(1)(m)]

County

Parcel I.D.

Reviewed by:
Date:

Property Owner
Montessori School of Waukesha, Inc.

Property Location
Govt. Lot SE ¼ Section 32, T7N, R19E

Property Owner' Mail Address
2600 Summit Avenue

Lot # Block # Subd. Name or CSM #

City: Waukesha State: WI Zip Code: 53188 Phone Number:

☒ City ☐ Village ☐ Town: Waukesha Nearest Road: Summit Avenue

Drainage area sq. ft. acres

Test site suitable for (check all that apply) ☐ Site not suitable:

☐ Bioretention ☐ Subsurface Dispersal System:

☐ Reuse: ☐ Irrigation: ☐ Other:

Hydraulic Application Test Method
x Morphological Evaluation
__ Double Ring Infiltrometer
__ Other: (specify) ____________ .

Soil Moisture
Date of soil borings:
USDA-NRCS WETS Value:
__ Dry = 1;
X Normal = 2;
__ Wet = 3

7 #OBS. ☐ Pit ☒ Boring Ground surface elevation 141.5 ft. Elevation of limiting factor ______________ ft.

Horizon	Depth in.	Dominant Color Munsell	Redox Description Qu. Sz. Cont. Color	Texture	Structure Gr. Sz. Sh.	Consistence	Boundary	% Rock Frags.	% Fines	Hydraulic App Rate inches/Hr
A	0-12	10 YR 3/3	--	SIL	1, F, GR	M, FR	A, S	< 5%		--
FILL	12-78	10 YR 5/3	--	SICL	MA	M, FI	A, S	20%		--
B	78-192	10 YR 6/4	--	SL	MA	M, FI	--	20%		0.50

Comments:

Name (Please Print): David M. Cornale, P.E.
Signature:
Credential Number: 43336-6

Address: N8 W22350 Johnson Drive, Suite A1 Waukesha, WI 53186
Date Evaluation Conducted: July 5, 2018
Telephone Number: (262) 544 0118

Overall Site Comments:

APPENDIX B

FIELD PROCEDURES

The field operations were conducted in general accordance with the procedures recommended by the American Society for Testing and Materials (ASTM) designation D 420 entitled “Standard Guide for Sampling Rock and Rock” and/or other relevant specifications. Soil samples were preserved and transported to *Giles’* laboratory in general accordance with the procedures recommended by ASTM designation D 4220 entitled “Standard Practice for Preserving and Transporting Soil Samples.” Brief descriptions of the sampling, testing and field procedures commonly performed by *Giles* are provided herein.

GENERAL FIELD PROCEDURES

Test Boring Elevations

The ground surface elevations reported on the Test Boring Logs are referenced to the assumed benchmark shown on the Boring Location Plan (Figure 1). Unless otherwise noted, the elevations were determined with a conventional hand-level and are accurate to within about 1 foot.

Test Boring Locations

The test borings were located on-site based on the existing site features and/or apparent property lines. Dimensions illustrating the approximate boring locations are reported on the Boring Location Plan (Figure 1).

Water Level Measurement

The water levels reported on the Test Boring Logs represent the depth of “free” water encountered during drilling and/or after the drilling tools were removed from the borehole. Water levels measured within a granular (sand and gravel) soil profile are typically indicative of the water table elevation. It is usually not possible to accurately identify the water table elevation with cohesive (clayey) soils, since the rate of seepage is slow. The water table elevation within cohesive soils must therefore be determined over a period of time with groundwater observation wells.

It must be recognized that the water table may fluctuate seasonally and during periods of heavy precipitation. Depending on the subsurface conditions, water may also become perched above the water table, especially during wet periods.

Borehole Backfilling Procedures

Each borehole was backfilled upon completion of the field operations. If potential contamination was encountered, and/or if required by state or local regulations, boreholes were backfilled with an “impervious” material (such as bentonite slurry). Borings that penetrated pavements, sidewalks, etc. were “capped” with Portland Cement concrete, asphaltic concrete, or a similar surface material. It must, however, be recognized that the backfill material may settle, and the surface cap may subside, over a period of time. Further backfilling and/or re-surfacing by *Giles’* client or the property owner may be required.

FIELD SAMPLING AND TESTING PROCEDURES

Auger Sampling (AU)

Soil samples are removed from the auger flights as an auger is withdrawn above the ground surface. Such samples are used to determine general soil types and identify approximate soil stratifications. Auger samples are highly disturbed and are therefore not typically used for geotechnical strength testing.

Split-Barrel Sampling (SS) – (ASTM D-1586)

A split-barrel sampler with a 2-inch outside diameter is driven into the subsoil with a 140-pound hammer free-falling a vertical distance of 30 inches. The summation of hammer-blows required to drive the sampler the final 12-inches of an 18-inch sample interval is defined as the “Standard Penetration Resistance” or N-value is an index of the relative density of granular soils and the comparative consistency of cohesive soils. A soil sample is collected from each SPT interval.

Shelby Tube Sampling (ST) – (ASTM D-1587)

A relatively undisturbed soil sample is collected by hydraulically advancing a thin-walled Shelby Tube sampler into a soil mass. Shelby Tubes have a sharp cutting edge and are commonly 2 to 5 inches in diameter.

Bulk Sample (BS)

A relatively large volume of soils is collected with a shovel or other manually-operated tool. The sample is typically transported to *Giles’* materials laboratory in a sealed bag or bucket.

Dynamic Cone Penetration Test (DC) – (ASTM STP 399)

This test is conducted by driving a 1.5-inch-diameter cone into the subsoil using a 15-pound steel ring (hammer), free-falling a vertical distance of 20 inches. The number of hammer-blows required to drive the cone 1¾ inches is an indication of the soil strength and density, and is defined as “N”. The Dynamic Cone Penetration test is commonly conducted in hand auger borings, test pits and within excavated trenches.

- Continued -

Ring-Lined Barrel Sampling – (ASTM D 3550)

In this procedure, a ring-lined barrel sampler is used to collect soil samples for classification and laboratory testing. This method provides samples that fit directly into laboratory test instruments without additional handling/disturbance.

Sampling and Testing Procedures

The field testing and sampling operations were conducted in general accordance with the procedures recommended by the American Society for Testing and Materials (ASTM) and/or other relevant specifications. Results of the field testing (i.e. N-values) are reported on the Test Boring Logs. Explanations of the terms and symbols shown on the logs are provided on the appendix enclosure entitled “General Notes”.

APPENDIX C

LABORATORY TESTING AND CLASSIFICATION

The laboratory testing was conducted under the supervision of a geotechnical engineer in accordance with the procedures recommended by the American Society for Testing and Materials (ASTM) and/or other relevant specifications. Brief descriptions of laboratory tests commonly performed by *Giles* are provided herein.

LABORATORY TESTING AND CLASSIFICATION

Photoionization Detector (PID)

In this procedure, soil samples are “scanned” in *Giles’* analytical laboratory using a Photoionization Detector (PID). The instrument is equipped with an 11.7 eV lamp calibrated to a Benzene Standard and is capable of detecting a minute concentration of **certain** Volatile Organic Compound (VOC) vapors, such as those commonly associated with petroleum products and some solvents. Results of the PID analysis are expressed in HNu (manufacturer’s) units rather than actual concentration.

Moisture Content (w) (ASTM D 2216)

Moisture content is defined as the ratio of the weight of water contained within a soil sample to the weight of the dry solids within the sample. Moisture content is expressed as a percentage.

Unconfined Compressive Strength (qu) (ASTM D 2166)

An axial load is applied at a uniform rate to a cylindrical soil sample. The unconfined compressive strength is the maximum stress obtained or the stress when 15% axial strain is reached, whichever occurs first.

Calibrated Penetrometer Resistance (qp)

The small, cylindrical tip of a hand-held penetrometer is pressed into a soil sample to a prescribed depth to measure the soils capacity to resist penetration. This test is used to evaluate unconfined compressive strength.

Vane-Shear Strength (qs)

The blades of a vane are inserted into the flat surface of a soil sample and the vane is rotated until failure occurs. The maximum shear resistance measured immediately prior to failure is taken as the vane-shear strength.

Loss-on-Ignition (ASTM D 2974; Method C)

The Loss-on-Ignition (L.O.I.) test is used to determine the organic content of a soil sample. The procedure is conducted by heating a dry soil sample to 440°C in order to burn-off or “ash” organic matter present within the sample. The L.O.I. value is the ratio of the weight loss due to ignition compared to the initial weight of the dry sample. L.O.I. is expressed as a percentage.

Particle Size Distribution (ASTB D 421, D 422, and D 1140)

This test is performed to determine the distribution of specific particle sizes (diameters) within a soil sample. The distribution of coarse-grained soil particles (sand and gravel) is determined from a “sieve analysis,” which is conducted by passing the sample through a series of nested sieves. The distribution of fine-grained soil particles (silt and clay) is determined from a “hydrometer analysis” which is based on the sedimentation of particles suspended in water.

Consolidation Test (ASTM D 2435)

In this procedure, a series of cumulative vertical loads are applied to a small, laterally confined soil sample. During each load increment, vertical compression (consolidation) of the sample is measured over a period of time. Results of this test are used to estimate settlement and time rate of settlement.

Classification of Samples

Each soil sample was visually-manually classified, based on texture and plasticity, in general accordance with the Unified Soil Classification System (ASTM D-2488-75). The classifications are reported on the Test Boring Logs.

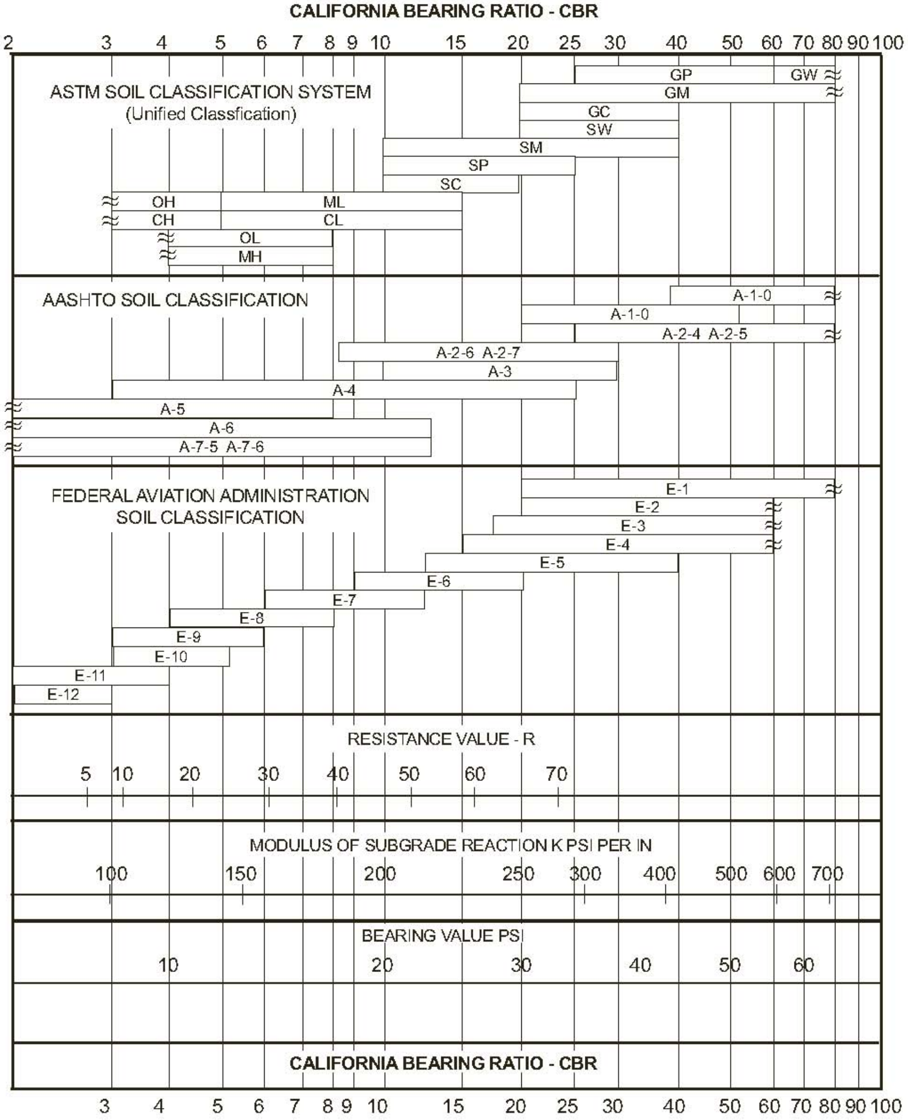
Laboratory Testing

The laboratory testing operations were conducted in general accordance with the procedures recommended by the American Society for Testing and Materials (ASTM) and/or other relevant specifications. Results of the laboratory tests are provided on the Test Boring Logs or other appendix enclosures. Explanation of the terms and symbols used on the logs is provided on the appendix enclosure entitled “General Notes.”

California Bearing Ratio (CBR) Test ASTM D-1833

The CBR test is used for evaluation of a soil subgrade for pavement design. The test consists of measuring the force required for a 3-square-inch cylindrical piston to penetrate 0.1 or 0.2 inch into a compacted soil sample. The result is expressed as a percent of force required to penetrate a standard compacted crushed stone.

Unless a CBR test has been specifically requested by the client, the CBR is estimated from published charts, based on soil classification and strength characteristics. A typical correlation chart is below.



APPENDIX D

GENERAL INFORMATION

AND
IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL REPORT

GENERAL COMMENTS

The soil samples obtained during the subsurface exploration will be retained for a period of thirty days. If no instructions are received, they will be disposed of at that time.

This report has been prepared exclusively for the client in order to aid in the evaluation of this property and to assist the architects and engineers in the design and preparation of the project plans and specifications. Copies of this report may be provided to contractor(s), with contract documents, to disclose information relative to this project. The report, however, has not been prepared to serve as the plans and specifications for actual construction without the appropriate interpretation by the project architect, structural engineer, and/or civil engineer. Reproduction and distribution of this report must be authorized by the client and *Giles*.

This report has been based on assumed conditions/characteristics of the proposed development where specific information was not available. It is recommended that the architect, civil engineer and structural engineer along with any other design professionals involved in this project carefully review these assumptions to ensure they are consistent with the actual planned development. When discrepancies exist, they should be brought to our attention to ensure they do not affect the conclusions and recommendations provided herein. The project plans and specifications may also be submitted to *Giles* for review to ensure that the geotechnical related conclusions and recommendations provided herein have been correctly interpreted.

The analysis of this site was based on a subsoil profile interpolated from a limited subsurface exploration. If the actual conditions encountered during construction vary from those indicated by the borings, *Giles* must be contacted immediately to determine if the conditions alter the recommendations contained herein.

The conclusions and recommendations presented in this report have been promulgated in accordance with generally accepted professional engineering practices in the field of geotechnical engineering. No other warranty is either expressed or implied.

GUIDE SPECIFICATIONS FOR SUBGRADE AND GRADE PREPARATION FOR FILL, FOUNDATION, FLOOR SLAB AND PAVEMENT SUPPORT; AND SELECTION, PLACEMENT AND COMPACTION OF FILL SOILS USING STANDARD PROCTOR PROCEDURES

1. Construction monitoring and testing of subgrades and grades for fill, foundation, floor slab and pavement; and fill selection, placement and compaction shall be performed by an experienced soils engineer and/or his representatives.

2. All compaction fill, subgrades and grades shall be (a) underlain by suitable bearing material; (b) free of all organic, frozen, or other deleterious material, and (c) observed, tested and approved by qualified engineering personnel representing an experienced soils engineer. Preparation of subgrades after stripping vegetation, organic or other unsuitable materials shall consist of (a) proof-rolling to detect soil, wet yielding soils or other unstable materials that must be undercut, (b) scarifying top 6 to 8 inches, (c) moisture conditioning the soils as required, and (d) recompaction to same minimum in-situ density required for similar materials indicated under Item 5. Note: compaction requirements for pavement subgrade are higher than other areas. Weather and construction equipment may damage compacted fill surface and reworking and retesting may be necessary to assure proper performance.

3. In overexcavation and fill areas, the compacted fill must extend (a) a minimum 1 foot lateral distance beyond the exterior edge of the foundation at bearing grade or pavement subgrade and down to compacted fill subgrade on a maximum 0.5(H):1(V) slope, (b) 1 foot above footing grade outside the building, and (c) to floor subgrade inside the building. Fill shall be placed and compacted on a 5(H):1(V) slope or must be stepped or benched as required to flatten if not specifically approved by qualified personnel under the direction of an experienced soil engineer.

4. The compacted fill materials shall be free of deleterious, organic, or frozen matter, shall contain no chemicals that may result in the material being classified as “contaminated”, and shall be low-expansive with a maximum Liquid Limit (ASTM D-423) and Plasticity Index (ASTM D-424) of 30 and 15, respectively, unless specifically tested and found to have low expansive properties and approved by an experienced soils engineer. The top 12 inches of compacted fill should have a maximum 3-inch-particle diameter and all underlying compacted fill a maximum 6-inch-diameter unless specifically approved by an experienced soils engineer. All fill materials must be tested and approved under the direction of an experienced soils engineer prior to placement. If the fill is to provide non-frost susceptible characteristics, it must be classified as a clean GW, GP, SW or SP per the Unified Soil Classification System (ASTM D-2487).

5. For structural fill depths less than 20 feet, the density of the structural compacted fill and scarified subgrade and grades shall not be less than 95 percent of the maximum dry density as determined by Standard Proctor (ASTM-698) with the exception of the top 12 inches of pavement subgrade which shall have a minimum in-situ density of 100 percent of maximum dry density, or 5 percent higher than underlying fill materials. Where the structural fill depth is greater than 20 feet, the portions below 20 feet should have a minimum in-place density of 100 percent of its maximum dry density of 5 percent greater than the top 20 feet. The moisture content of cohesive soil shall not vary by more than -1 to +3 percent and granular soil ±3 percent of the optimum when placed and compacted or recompacted, unless specifically recommended/approved by the soils engineer monitoring the placement and compaction. Cohesive soils with moderate to high expansion potentials (PI>15) should, however, be placed, compacted and maintained prior to construction at a moisture content 3±1 percent above optimum moisture content to limit further heave. The fill shall be placed in layers with a maximum loose thickness of 8 inches for foundations and 10 inches for floor slabs and pavement, unless specifically approved by the soils engineer taking into consideration the type of materials and compaction equipment being used. The compaction equipment should consist of suitable mechanical equipment specifically designed for soil compaction. Bulldozers or similar tracked vehicles are typically not suitable for compaction.

6. Excavation, filling, subgrade and grade preparation shall be performed in a manner and sequence that will provide drainage at all times and proper control of erosion. Precipitation, springs and seepage water encountered shall be pumped or drained to provide a suitable working platform. Springs or water seepage encountered during grading/foundation construction must be called to the soil engineer’s attention immediately for possible construction procedure revision or inclusion of an underdrain system.

7. Non-structural fill adjacent to structural fill should typically be placed in unison to provide lateral support. Backfill along walls must be placed and compacted with care to ensure excessive unbalanced lateral pressures do not develop. The type of fill material placed adjacent to below-grade walls (i.e. basement walls and retaining walls) must be properly tested and approved by an experienced soils engineer with consideration for the lateral pressure used in the wall design.

8. Whenever, in the opinion of the soils engineer or the Owner’s Representatives, an unstable condition is being created either by cutting or filling, the work shall not proceed into that area until an appropriate geotechnical exploration and analysis has been performed and the grading plan revised, if found necessary.

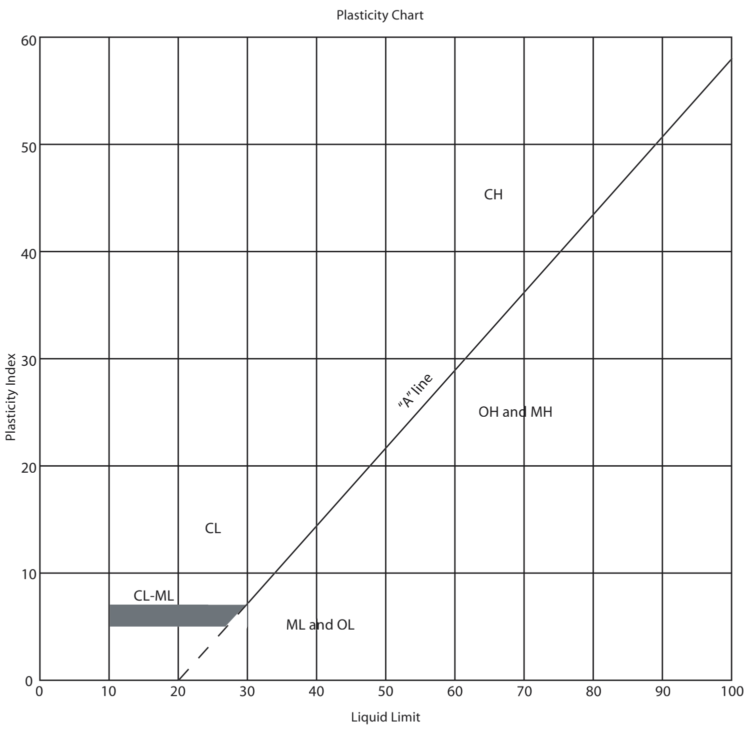
CHARACTERISTICS AND RATINGS OF UNIFIED SOIL SYSTEM CLASSES FOR SOIL CONSTRUCTION *

Class	Compaction Characteristics	Max. Dry Density Standard Proctor (pcf)	Compressibility and Expansion	Drainage and Permeability	Value as an Embankment Material	Value as Subgrade When Not Subject to Frost	Value as Base Course	Value as Temporary Pavement	
								With Dust Palliative	With Bituminous Treatment
GW	Good: tractor, rubber-tired, steel wheel or vibratory roller	125-135	Almost none	Good drainage, pervious	Very stable	Excellent	Good	Fair to poor	Excellent
GP	Good: tractor, rubber-tired, steel wheel or vibratory roller	115-125	Almost none	Good drainage, pervious	Reasonably stable	Excellent to good	Poor to fair	Poor	
GM	Good: rubber-tired or light sheepsfoot roller	120-135	Slight	Poor drainage, semipervious	Reasonably stable	Excellent to good	Fair to poor	Poor	Poor to fair
GC	Good to fair: rubber-tired or sheepsfoot roller	115-130	Slight	Poor drainage, impervious	Reasonably stable	Good	Good to fair **	Excellent	Excellent
SW	Good: tractor, rubber-tired or vibratory roller	110-130	Almost none	Good drainage, pervious	Very stable	Good	Fair to poor	Fair to poor	Good
SP	Good: tractor, rubber-tired or vibratory roller	100-120	Almost none	Good drainage, pervious	Reasonably stable when dense	Good to fair	Poor	Poor	Poor to fair
SM	Good: rubber-tired or sheepsfoot roller	110-125	Slight	Poor drainage, impervious	Reasonably stable when dense	Good to fair	Poor	Poor	Poor to fair
SC	Good to fair: rubber-tired or sheepsfoot roller	105-125	Slight to medium	Poor drainage, impervious	Reasonably stable	Good to fair	Fair to poor	Excellent	Excellent
ML	Good to poor: rubber-tired or sheepsfoot roller	95-120	Slight to medium	Poor drainage, impervious	Poor stability, high density required	Fair to poor	Not suitable	Poor	Poor
CL	Good to fair: sheepsfoot or rubber-tired roller	95-120	Medium	No drainage, impervious	Good stability	Fair to poor	Not suitable	Poor	Poor
OL	Fair to poor: sheepsfoot or rubber-tired roller	80-100	Medium to high	Poor drainage, impervious	Unstable, should not be used	Poor	Not suitable	Not suitable	Not suitable
MH	Fair to poor: sheepsfoot or rubber-tired roller	70-95	High	Poor drainage, impervious	Poor stability, should not be used	Poor	Not suitable	Very poor	Not suitable
CH	Fair to poor: sheepsfoot roller	80-105	Very high	No drainage, impervious	Fair stability, may soften on expansion	Poor to very poor	Not suitable	Very poor	Not suitable
OH	Fair to poor: sheepsfoot roller	65-100	High	No drainage, impervious	Unstable, should not be used	Very poor	Not suitable	Not suitable	Not suitable
Pt	Not suitable		Very high	Fair to poor drainage	Should not be used	Not suitable	Not suitable	Not suitable	Not suitable

* "The Unified Classification: Appendix A - Characteristics of Soil, Groups Pertaining to Roads and Airfields, and Appendix B - Characteristics of Soil Groups Pertaining to Embankments and Foundations," Technical Memorandum 357, U.S. Waterways Ixperiment Station, Vicksburg, 1953.

** Not suitable if subject to frost.

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D-2487)

Major Divisions			*Group Symbols*		*Typical Names*	*Laboratory Classification Criteria*		
Coarse-grained soils (more than half of material is larger than No. 200 sieve size)	Gravels (More than half of coarse fraction is larger than No. 4 sieve size)	Clean gravels (little or no fines)	GW		Well-graded gravels, gravel-sand mixtures, little or no fines	Determine percentages of sand and gravel from grain-size curve. Depending on percentage of fines (fraction smaller than No. 200 sieve size), coarse-grained soils are classified as follows: Less than 5 percent: GW, GP, SW, SP More than 12 percent: GM, GC, SM, SC 5 to 12 percent: *Borderline* cases requiring dual symbols[b]	$C_u = \frac{D_{60}}{D_{10}}$ greater than 4; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3	
			GP		Poorly graded gravels, gravel-sand mixtrues, little or no fines		Not meeting all gradation requirements for GW	
		Gravels with fines (appreciable amount of fines)	GM[a]	d	Silty gravels, gravel-sand-silt mixtures		Atterberg limits below "A" line or P.I. less than 4	Limits plotting within shaded area, above "A" line with P.I. between 4 and 7 are *borderline* cases requiring use of dual symbols
				u				
			GC		Clayey gravels, gravel-sand-clay mixtures		Atterberg limits above "A" line or P.I. greater than 7	
	Sands (More than half of coarse fraction is smaller than No. 4 sieve size)	Clean sands (Little or no fines)	SW		Well-graded sands, gravelly sands, little or no fines		$C_u = \frac{D_{60}}{D_{10}}$ greater than 4; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3	
			SP		Poorly graded sands, gravelly sands, little or no fines		Not meeting all gradation requirements for SW	
		Sands with fines (Appreciable amount of fines)	SM[a]	d	Silty sands, sand-silt mixtures		Atterberg limits below "A" line or P.I. less than 4	Limits plotting within shaded area, above "A" line with P.I. between 4 and 7 are *borderline* cases requiring use of dual symbols
				u				
			SC		Clayey sands, sand-clay mixtures		Atterberg limits above "A" line or P.I. greater than 7	
Fine-grained soils (More than half material is smaller than No. 200 sieve size)	Silts and clays (Liquid limit less than 50)		ML		Inorganic silts and very fine sands, rock flour, silty or clayey fine sands, or clayey silts with slight plasticity	 Plasticity Chart		
			CL		Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays			
			OL		Organic silts and organic silty clays of low plasticity			
	Silts and clays (Liquid limit greater than 50)		MH		Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts			
			CH		Inorganic clays of high plasticity, fat clays			
			OH		Organic clays of medium to high plasticity, organic silts			
	Highly organic soils		Pt		Peat and other highly organic soils			

[a] Division of GM and SM groups into subdivisions of d and u are for roads and airfields only. Subdivision is based on Atterberg limits, suffix d used when L.L. is 28 or less and the P.I. is 6 or less; the suffix u is used when L.L. is greater than 28.

[b] Borderline classifications, used for soils possessing characteristics of two groups, are designated by combinations of group sympols. For example GW-GC, well-graded gravel-sand mixture with clay binder.

GENERAL NOTES

SAMPLE IDENTIFICATION

All samples are visually classified in general accordance with the Unified Soil Classification System (ASTM D-2487-75 or D-2488-75)

DESCRIPTIVE TERM (% BY DRY WEIGHT)

Trace:	1-10%
Little:	11-20%
Some:	21-35%
And/Adjective	36-50%

PARTICLE SIZE (DIAMETER)

Boulders:	8 inch and larger
Cobbles:	3 inch to 8 inch
Gravel:	coarse - ¾ to 3 inch
	fine – No. 4 (4.76 mm) to ¾ inch
Sand:	coarse – No. 4 (4.76 mm) to No. 10 (2.0 mm)
	medium – No. 10 (2.0 mm) to No. 40 (0.42 mm)
	fine – No. 40 (0.42 mm) to No. 200 (0.074 mm)
Silt:	No. 200 (0.074 mm) and smaller (non-plastic)
Clay:	No 200 (0.074 mm) and smaller (plastic)

SOIL PROPERTY SYMBOLS

Dd:	Dry Density (pcf)
LL:	Liquid Limit, percent
PL:	Plastic Limit, percent
PI:	Plasticity Index (LL-PL)
LOI:	Loss on Ignition, percent
Gs:	Specific Gravity
K:	Coefficient of Permeability
w:	Moisture content, percent
qp:	Calibrated Penetrometer Resistance, tsf
qs:	Vane-Shear Strength, tsf
qu:	Unconfined Compressive Strength, tsf
qc:	Static Cone Penetrometer Resistance (correlated to Unconfined Compressive Strength, tsf)
PID:	Results of vapor analysis conducted on representative samples utilizing a Photoionization Detector calibrated to a benzene standard. Results expressed in HNU-Units. (BDL=Below Detection Limit)
N:	Penetration Resistance per 12 inch interval, or fraction thereof, for a standard 2 inch O.D. (1⅜ inch I.D.) split spoon sampler driven with a 140 pound weight free-falling 30 inches. Performed in general accordance with Standard Penetration Test Specifications (ASTM D-1586). N in blows per foot equals sum of N-Values where plus sign (+) is shown.
Nc:	Penetration Resistance per 1¾ inches of Dynamic Cone Penetrometer. Approximately equivalent to Standard Penetration Test N-Value in blows per foot.
Nr:	Penetration Resistance per 12 inch interval, or fraction thereof, for California Ring Sampler driven with a 140 pound weight free-falling 30 inches per ASTM D-3550. Not equivalent to Standard Penetration Test N-Value.

DRILLING AND SAMPLING SYMBOLS

SS:	Split-Spoon
ST:	Shelby Tube – 3 inch O.D. (except where noted)
CS:	3 inch O.D. California Ring Sampler
DC:	Dynamic Cone Penetrometer per ASTM Special Technical Publication No. 399
AU:	Auger Sample
DB:	Diamond Bit
CB:	Carbide Bit
WS:	Wash Sample
RB:	Rock-Roller Bit
BS:	Bulk Sample
Note:	Depth intervals for sampling shown on Record of Subsurface Exploration are not indicative of sample recovery, but position where sampling initiated

SOIL STRENGTH CHARACTERISTICS

COHESIVE (CLAYEY) SOILS

COMPARATIVE CONSISTENCY	**BLOWS PER FOOT (N)**	**UNCONFINED COMPRESSIVE STRENGTH (TSF)**
Very Soft	0 - 2	0 - 0.25
Soft	3 - 4	0.25 - 0.50
Medium Stiff	5 – 8	0.50 - 1.00
Stiff	9 – 15	1.00 - 2.00
Very Stiff	16 – 30	2.00 - 4.00
Hard	31+	4.00+

NON-COHESIVE (GRANULAR) SOILS

RELATIVE DENSITY	**BLOWS PER FOOT (N)**
Very Loose	0 - 4
Loose	5 - 10
Firm	11 - 30
Dense	31 - 50
Very Dense	51+

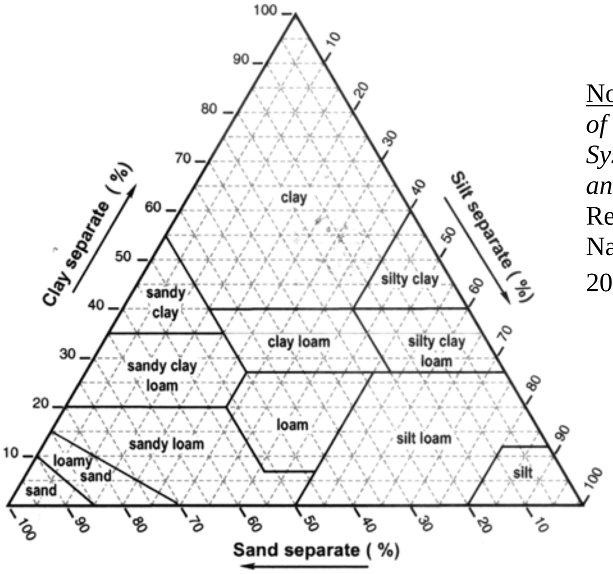
DEGREE OF PLASTICITY	**PI**
None to Slight	0 - 4
Slight	5 - 10
Medium	11 - 30
High to Very High	31+

DEGREE OF EXPANSIVE POTENTIAL	**PI**
Low	0 - 15
Medium	15 - 25
High	25+

SOIL CLASSIFICATION NOTES

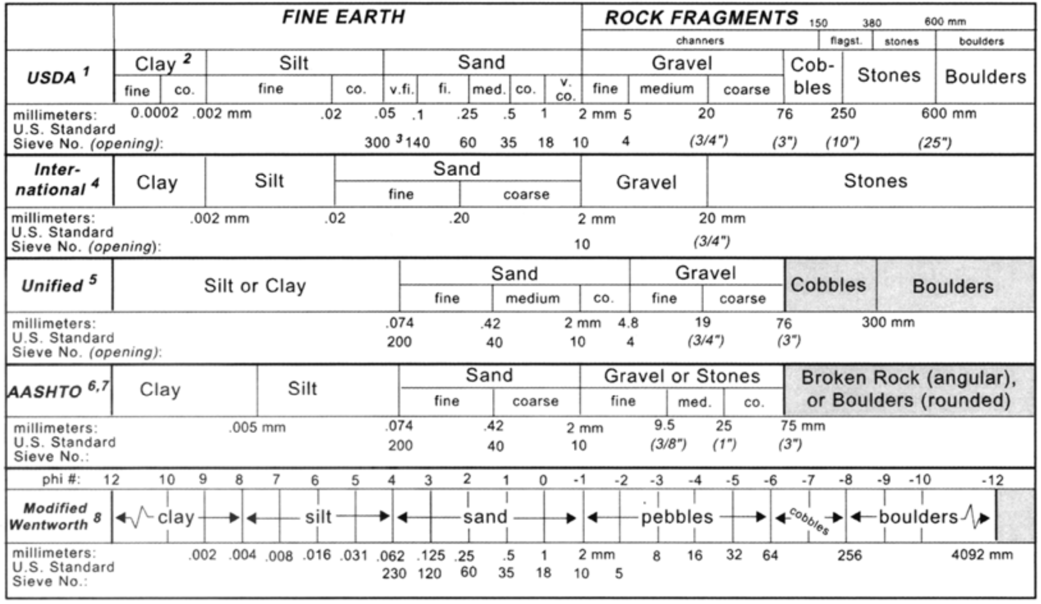
Texture Triangle:

Fine Earth Texture Classes (▬▬)



Note: *Texture Triangle* and *Comparison of Particle Size Classes in Different Systems* from *Field Book for Describing and Sampling Soil*, USDA Natural Resources Conservation Service National Soil Survey Center (September 2002).

Comparison of Particle Size Classes in Different Systems



	FINE EARTH									***ROCK FRAGMENTS*** 150 380 600 mm					
										channers			flagst.	stones	boulders
USDA [1]	Clay [2]		Silt		Sand					Gravel			Cob-bles	Stones	Boulders
	fine	co.	fine	co.	v.fi.	fi.	med.	co.	v. co.	fine	medium	coarse			

millimeters: 0.0002 .002 mm .02 .05 .1 .25 .5 1 2 mm 5 20 76 250 600 mm
U.S. Standard
Sieve No. *(opening)*: 300 [3] 140 60 35 18 10 4 *(3/4")* *(3")* *(10")* *(25")*

Inter-national [4]	Clay	Silt	Sand		Gravel	Stones
			fine	coarse		

millimeters: .002 mm .02 .20 2 mm 20 mm
U.S. Standard
Sieve No. *(opening)*: 10 *(3/4")*

Unified [5]	Silt or Clay	Sand			Gravel		Cobbles	Boulders
		fine	medium	co.	fine	coarse		

millimeters: .074 .42 2 mm 4.8 19 76 300 mm
U.S. Standard
Sieve No. *(opening)*: 200 40 10 4 *(3/4")* *(3")*

AASHTO [6,7]	Clay	Silt	Sand		Gravel or Stones			Broken Rock (angular), or Boulders (rounded)
			fine	coarse	fine	med.	co.	

millimeters: .005 mm .074 .42 2 mm 9.5 25 75 mm
U.S. Standard
Sieve No.: 200 40 10 *(3/8")* *(1")* *(3")*

phi #: 12 10 9 8 7 6 5 4 3 2 1 0 -1 -2 -3 -4 -5 -6 -7 -8 -9 -10 -12

Modified Wentworth [8]	clay	silt	sand	pebbles	cobbles	boulders

millimeters: .002 .004 .008 .016 .031 .062 .125 .25 .5 1 2 mm 8 16 32 64 256 4092 mm
U.S. Standard
Sieve No.: 230 120 60 35 18 10 5

1. Soil Survey Staff. 1995. Soil survey Laboratory information manual. USDA, Natural Resources Conservation Service, Soil Survey Investigations Report No. 45, Version 1.0, National Soil Survey Center, Lincoln, NE. 305 p.
2. Soil Survey Staff. 1995. Soil Survey Lab information manual. USDA-NRCS, Soil Survey Investigation Report #45, version 1.0, National Soil Survey Center, Lincoln, NE. Note: Mineralogy studies may subdivide clay into three size ranges; fine (<0.08µm), medium (0.08-0.2µm), and coarse (0.2-2µm); Jackson, 1969.
3. The Soil Survey Lab (Lincoln, NE) uses a no. 300 sieve (0.047 mm opening) for the USDA-sand/silt measurement. A no. 270 sieve (0.053 mm opening) is more readily available and widely used.
4. International Soil Science Society. 1951. *In*: Soil Survey Manual. Soil Survey Staff, USDA-Soil Conservation Service, Agricultural Handbook No. 18, U.S. Gov. Print. Office, Washington, D.C. 214 p.
5. ASTM. 1993. Standard classification of soils for engineering purposes (Unified Soil Classification System). ASTM designation D2487-92. *In*: Soil and rock; dimension stone; geosynthetics. Annual book of ASTM standards-Vol. 04.08.
6. AASHTO. 1986a. Recommended practice for the classification of soils and soil-aggregate mixtures for highway construction purposes. AASHTO designation M145-82. *In*: Standard specifications for transportation materials and methods of sampling and testing; Part 1: Specifications (14th ed.). American Association of State Highway and Transportation Officials, Washington, D.C.
7. AASHTO. 1986b. Standard definitions of terms relating to subgrade, soil-aggregate, and fill materials. AASHTO designation M146-70 (1980). *In*: sampling and testing; Part 1: Specifications (14th ed.). American Association of State Highway and Transportation Officials, Washington, D.C.
8. Ingram, R.L. 1982. Modified Wentworth scale. *In*: Grain-size scales. AGI Date Sheet 29.1. *In*: Dutro, J.T., Dietrich, R.V., and Foose, R.M. 1989. AGI data sheets for geology in the field, laboratory, and office, 3rd edition. American Geological Institute, Washington, D.C.

Important Information About Your
Geotechnical Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

The following information is provided to help you manage your risks.

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical engineering study conducted for a civil engineer may not fulfill the needs of a construction contractor or even another civil engineer. Because each geotechnical engineering study is unique, each geotechnical engineering report is unique, prepared *solely* for the client. No one except you should rely on your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you* — should apply the report for any purpose or project except the one originally contemplated.

Read the Full Report

Serious problems have occurred because those relying on a geotechnical engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

A Geotechnical Engineering Report Is Based on A Unique Set of Project-Specific Factors

Geotechnical engineers consider a number of unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical engineering report that was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light industrial plant to a refrigerated warehouse,
- elevation, configuration, location, orientation, or weight of the proposed structure,
- composition of the design team, or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical engineering report is based on conditions that existed at the time the study was performed. *Do not rely on a geotechnical engineering report* whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, earthquakes, or groundwater fluctuations. *Always* contact the geotechnical engineer before applying the report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ—sometimes significantly—from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final,* because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.*

A Geotechnical Engineering Report Is Subject to Misinterpretation

Other design team members' misinterpretation of geotechnical engineering reports has resulted in costly problems. Lower that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering report. Reduce that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

Give Contractors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure contractors have sufficient time* to perform additional study. Only then might you be in a position to give contractors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and contractors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations" many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform a *geoenvironmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical engineering report does not usually relate any geoenvironmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own geoenvironmental information, ask your geotechnical consultant for risk management guidance. *Do not rely on an environmental report prepared for someone else.*

Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the *express purpose* of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, a number of mold prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold prevention consultant; ***none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.***

Rely, on Your ASFE-Member Geotechncial Engineer for Additional Assistance

Membership in ASFE/The Best People on Earth exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with you ASFE-member geotechnical engineer for more information.

ASFE
The Best People on Earth

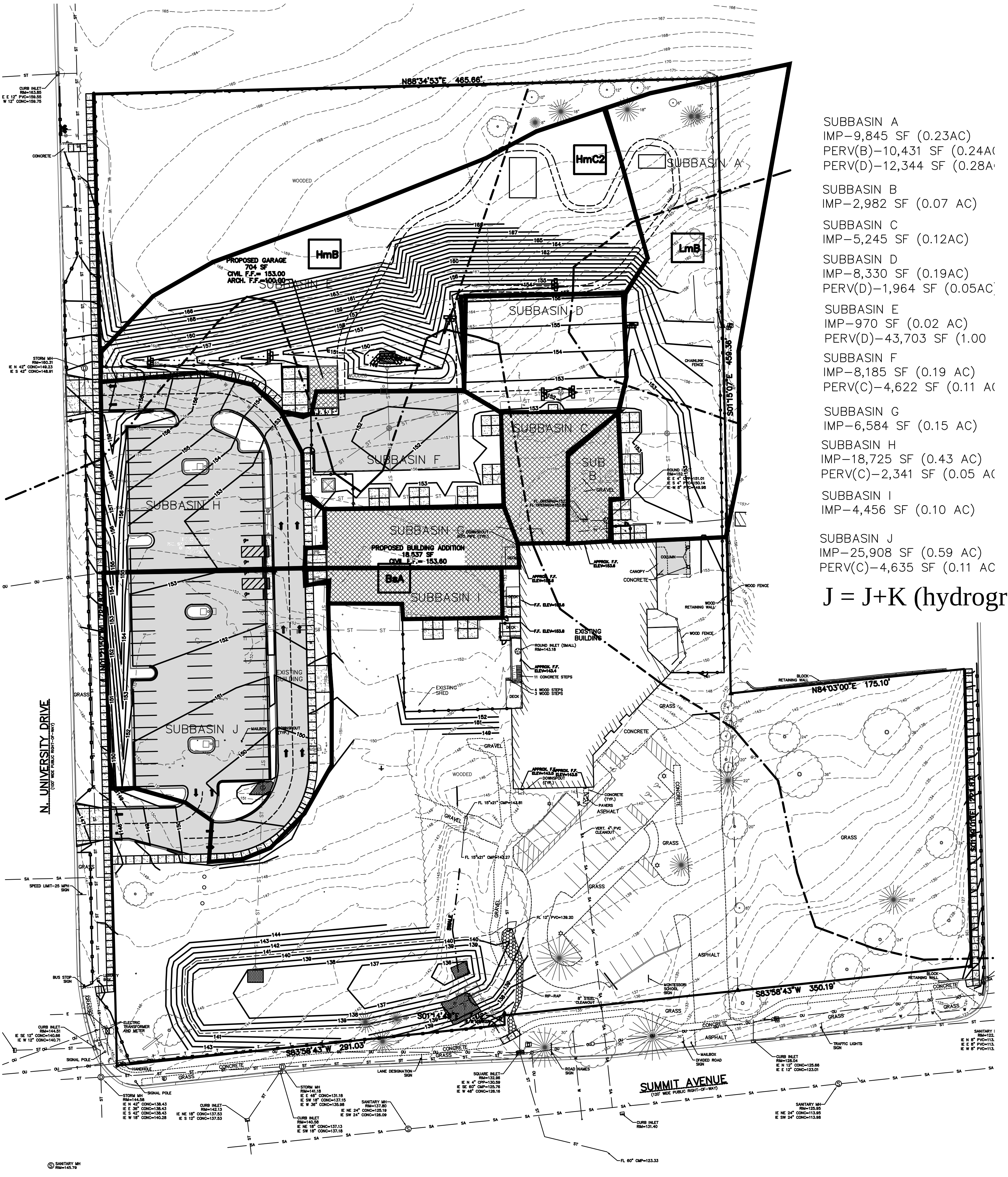
8811 Colesville Road/Suite G106, Silver Spring, MD 20910
Telephone: 301/565-2733 Facsimile: 301/589-2017
e-mail: info@asfe.org www.asfe.org

Geotechnical, Environmental & Construction Materials Consultants

GILES
ENGINEERING ASSOCIATES, INC.

www.gilesengr.com

Appendix D
<u>Pipe Capacity Calculations</u>



SUBBASIN A
IMP–9,845 SF (0.23AC)
PERV(B)–10,431 SF (0.24A
PERV(D)–12,344 SF (0.28A

SUBBASIN B
IMP–2,982 SF (0.07 AC)

SUBBASIN C
IMP–5,245 SF (0.12AC)

SUBBASIN D
IMP–8,330 SF (0.19AC)
PERV(D)–1,964 SF (0.05AC

SUBBASIN E
IMP–970 SF (0.02 AC)
PERV(D)–43,703 SF (1.00

SUBBASIN F
IMP–8,185 SF (0.19 AC)
PERV(C)–4,622 SF (0.11 A

SUBBASIN G
IMP–6,584 SF (0.15 AC)

SUBBASIN H
IMP–18,725 SF (0.43 AC)
PERV(C)–2,341 SF (0.05 A

SUBBASIN I
IMP–4,456 SF (0.10 AC)

SUBBASIN J
IMP–25,908 SF (0.59 AC)
PERV(C)–4,635 SF (0.11 AC

J = J+K (hydrograph)

NORTH

PIPE SIZING MAP

SCALE (11"X17"): 1"=80'–0"

EXCEL
ARCHITECTS • ENGINEERS • SURVEYORS

Excel Engineering Project No. 1818660

Project Name Montessori School - Waukesha

Pipe Data				Pipe Capacity (100-yr)				
Pipe No.	Diameter (FT)	Slope (FT/FT)	Manning's n	Basin No.	Total Flow (cfs)	Total Flow (gpm)	Full Flow Capacity (cfs)	Full Flow Capacity (gpm)
1	0.67	0.005	0.012	B	0.49	220	0.94	422
2	1.25	0.005	0.012	A, B	4.23	1898	4.96	2227
3	0.67	0.005	0.012	C	0.83	373	0.94	422
4	1.5	0.005	0.012	A, B, C,D	6.67	2993	8.07	3621
5	0.67	0.010	0.012	G	1.04	467	1.33	597
6	1.25	0.015	0.012	E	5.23	2347	8.59	3857
7	2	0.005	0.012	A,B,C,D,E,F,G	14.82	6651	17.38	7798
8	0.67	0.010	0.012	I	0.69	310	1.33	597
9	2	0.008	0.012	A,B,C,D,E,F,G,I,H	18.80	8437	21.98	9864
~~10~~	~~0.83~~	~~0.014~~	~~0.012~~	~~J~~	~~1.46~~	~~655~~	~~2.79~~	~~1250~~
11	2	0.055	0.012	A,B,C,D,E,F,G,I,H,J,K	23.51	10551	57.63	25864

NA - Removed; flow drains to pipe no. 11

Full Flow Capacity based off Manning's Equation

$$Q = \frac{1.49}{n} R^{2/3} S^{1/2} a$$

Q = Full Flow Capacity of Pipe (cfs)
n = manning's roughness coefficient
R = hydraulic radius (ft) (D/4)
s = hydraulic gradient, slope (ft/ft)
a = flow area (sq. ft.)

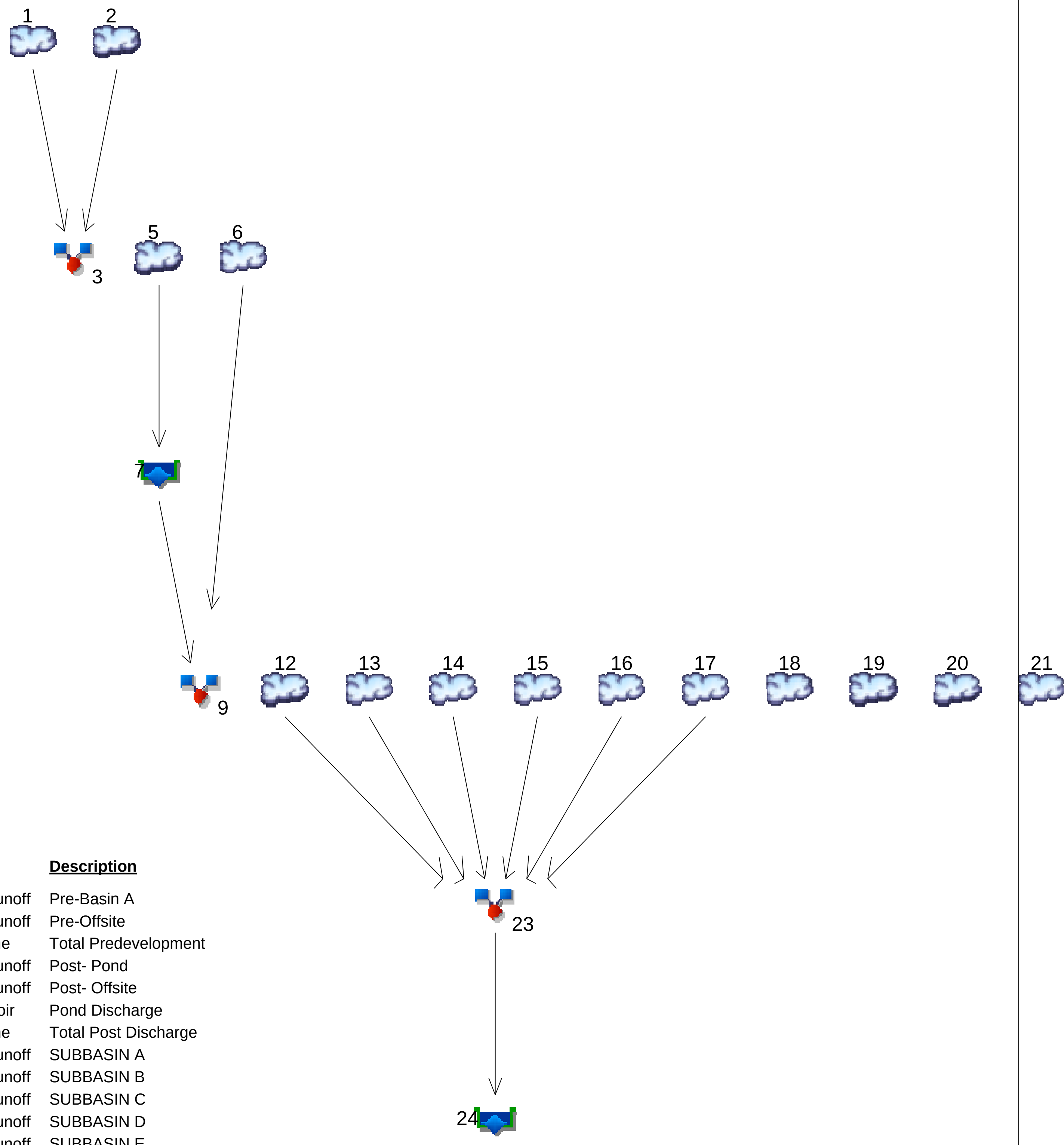
Typical Manning's n	
HDPE	0.012
PVC	0.012
Concrete	0.013
CMP	0.024

*Total Flow calculated via TR-55 hydrologic calculations. Reference Storm Pipe Basin Map & TR-55 Calculations

Appendix E
SCS TR55
Stormwater Management Calculations:

- Hydrograph Return Period Recap
- Hydrograph Summary Reports
- Hydrograph Plots
- Hydrograph Tc Worksheets

Watershed Model Schematic



Legend

Hyd.	Origin	Description
1	SCS Runoff	Pre-Basin A
2	SCS Runoff	Pre-Offsite
3	Combine	Total Predevelopment
5	SCS Runoff	Post- Pond
6	SCS Runoff	Post- Offsite
7	Reservoir	Pond Discharge
9	Combine	Total Post Discharge
12	SCS Runoff	SUBBASIN A
13	SCS Runoff	SUBBASIN B
14	SCS Runoff	SUBBASIN C
15	SCS Runoff	SUBBASIN D
16	SCS Runoff	SUBBASIN E
17	SCS Runoff	SUBBASIN F
18	SCS Runoff	SUBBASIN G
19	SCS Runoff	SUBBASIN H
20	SCS Runoff	SUBBASIN I
21	SCS Runoff	SUBBASIN J
23	Combine	North Basin Flows
24	Reservoir	North Discharge

Hydrograph Return Period Recap

Hyd. No.	Hydrograph type (origin)	Inflow hyd(s)	Peak Outflow (cfs)								Hydrograph Description
			1-yr	2-yr	3-yr	5-yr	10-yr	25-yr	50-yr	100-yr	
1	SCS Runoff	------	3.370	4.778	-------	-------	9.980	-------	-------	17.03	Pre-Basin A
2	SCS Runoff	------	0.921	1.226	-------	-------	2.277	-------	-------	3.617	Pre-Offsite
3	Combine	1, 2	3.676	5.219	-------	-------	10.82	-------	-------	18.32	Total Predevelopment
5	SCS Runoff	------	8.247	10.98	-------	-------	20.39	-------	-------	32.38	Post- Pond
6	SCS Runoff	------	0.130	0.172	-------	-------	0.320	-------	-------	0.509	Post- Offsite
7	Reservoir	5	3.204	4.642	-------	-------	7.874	-------	-------	15.91	Pond Discharge
9	Combine	6, 7,	3.256	4.709	-------	-------	7.993	-------	-------	16.09	Total Post Discharge
12	SCS Runoff	------	0.780	1.083	-------	-------	2.218	-------	-------	3.742	SUBBASIN A
13	SCS Runoff	------	0.196	0.231	-------	-------	0.346	-------	-------	0.486	SUBBASIN B
14	SCS Runoff	------	0.336	0.396	-------	-------	0.592	-------	-------	0.833	SUBBASIN C
15	SCS Runoff	------	0.588	0.714	-------	-------	1.120	-------	-------	1.613	SUBBASIN D
16	SCS Runoff	------	1.135	1.558	-------	-------	3.138	-------	-------	5.227	SUBBASIN E
17	SCS Runoff	------	0.579	0.733	-------	-------	1.245	-------	-------	1.875	SUBBASIN F
18	SCS Runoff	------	0.419	0.495	-------	-------	0.741	-------	-------	1.041	SUBBASIN G
19	SCS Runoff	------	1.269	1.517	-------	-------	2.316	-------	-------	3.289	SUBBASIN H
20	SCS Runoff	------	0.280	0.330	-------	-------	0.494	-------	-------	0.694	SUBBASIN I
21	SCS Runoff	------	0.587	0.693	-------	-------	1.037	-------	-------	1.458	SUBBASIN J
22	SCS Runoff	------	1.150	1.408	-------	-------	2.241	-------	-------	3.254	SUBBASIN K

Hydrograph Summary Report

Hydraflow Hydrographs Extension for AutoCAD® Civil 3D® 2016 by Autodesk, Inc. v11

Hyd. No.	Hydrograph type (origin)	Peak flow (cfs)	Time interval (min)	Time to Peak (min)	Hyd. volume (cuft)	Inflow hyd(s)	Maximum elevation (ft)	Total strge used (cuft)	Hydrograph Description
1	SCS Runoff	3.370	3	729	12,701	------	------	------	Pre-Basin A
2	SCS Runoff	0.921	3	717	2,098	------	------	------	Pre-Offsite
3	Combine	3.676	3	726	14,799	1, 2	------	------	Total Predevelopment
5	SCS Runoff	8.247	3	717	18,781	------	------	------	Post- Pond
6	SCS Runoff	0.130	3	717	295	------	------	------	Post- Offsite
7	Reservoir	3.204	3	726	18,780	5	137.88	4,778	Pond Discharge
9	Combine	3.256	3	726	19,075	6, 7,	------	------	Total Post Discharge
12	SCS Runoff	0.780	3	720	1,803	------	------	------	SUBBASIN A
13	SCS Runoff	0.196	3	717	494	------	------	------	SUBBASIN B
14	SCS Runoff	0.336	3	717	846	------	------	------	SUBBASIN C
15	SCS Runoff	0.588	3	717	1,371	------	------	------	SUBBASIN D
16	SCS Runoff	1.135	3	720	2,615	------	------	------	SUBBASIN E
17	SCS Runoff	0.579	3	717	1,308	------	------	------	SUBBASIN F
18	SCS Runoff	0.419	3	717	1,058	------	------	------	SUBBASIN G
19	SCS Runoff	1.269	3	717	3,048	------	------	------	SUBBASIN H
20	SCS Runoff	0.280	3	717	705	------	------	------	SUBBASIN I
21	SCS Runoff	0.587	3	717	1,481	------	------	------	SUBBASIN J
22	SCS Runoff	1.150	3	717	2,655	------	------	------	SUBBASIN K

1818660STORM.gpw	Return Period: 1 Year	Monday, 07 / 9 / 2018

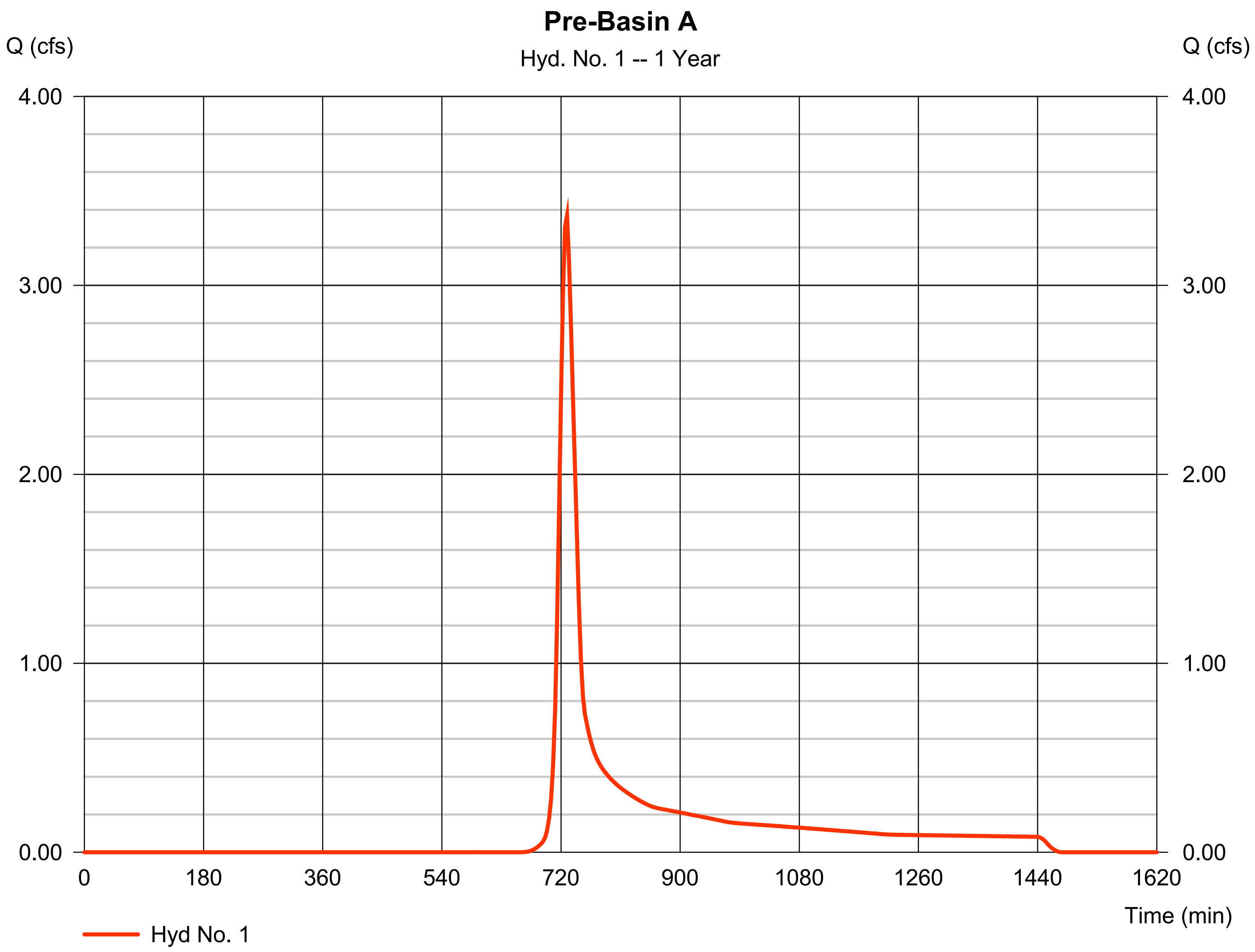
Hydrograph Report

Hyd. No. 1

Pre-Basin A

Hydrograph type	= SCS Runoff	Peak discharge	= 3.370 cfs
Storm frequency	= 1 yrs	Time to peak	= 729 min
Time interval	= 3 min	Hyd. volume	= 12,701 cuft
Drainage area	= 5.080 ac	Curve number	= 79*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= TR55	Time of conc. (Tc)	= 21.40 min
Total precip.	= 2.30 in	Distribution	= Type II
Storm duration	= 24 hrs	Shape factor	= 484

* Composite (Area/CN) = [(0.260 x 61) + (1.160 x 98) + (1.170 x 74) + (1.230 x 70) + (0.670 x 80) + (0.590 x 77)] / 5.080



TR55 Tc Worksheet

Hyd. No. 1

Pre-Basin A

Description	A	B	C	Totals
Sheet Flow				
Manning's n-value	= 0.240	0.011	0.011	
Flow length (ft)	= 220.0	0.0	0.0	
Two-year 24-hr precip. (in)	= 2.54	0.00	0.00	
Land slope (%)	= 6.80	0.00	0.00	
Travel Time (min)	**= 18.45**	**+ 0.00**	**+ 0.00**	**= 18.45**
Shallow Concentrated Flow				
Flow length (ft)	= 180.00	0.00	0.00	
Watercourse slope (%)	= 3.90	0.00	0.00	
Surface description	= Unpaved	Paved	Paved	
Average velocity (ft/s)	=3.19	0.00	0.00	
Travel Time (min)	**= 0.94**	**+ 0.00**	**+ 0.00**	**= 0.94**
Channel Flow				
X sectional flow area (sqft)	= 20.00	0.00	0.00	
Wetted perimeter (ft)	= 24.00	0.00	0.00	
Channel slope (%)	= 6.00	0.00	0.00	
Manning's n-value	= 0.170	0.015	0.015	
Velocity (ft/s)	=1.90	0.00	0.00	
Flow length (ft)	({0})230.0	0.0	0.0	
Travel Time (min)	**= 2.02**	**+ 0.00**	**+ 0.00**	**= 2.02**

Total Travel Time, Tc .... 21.40 min

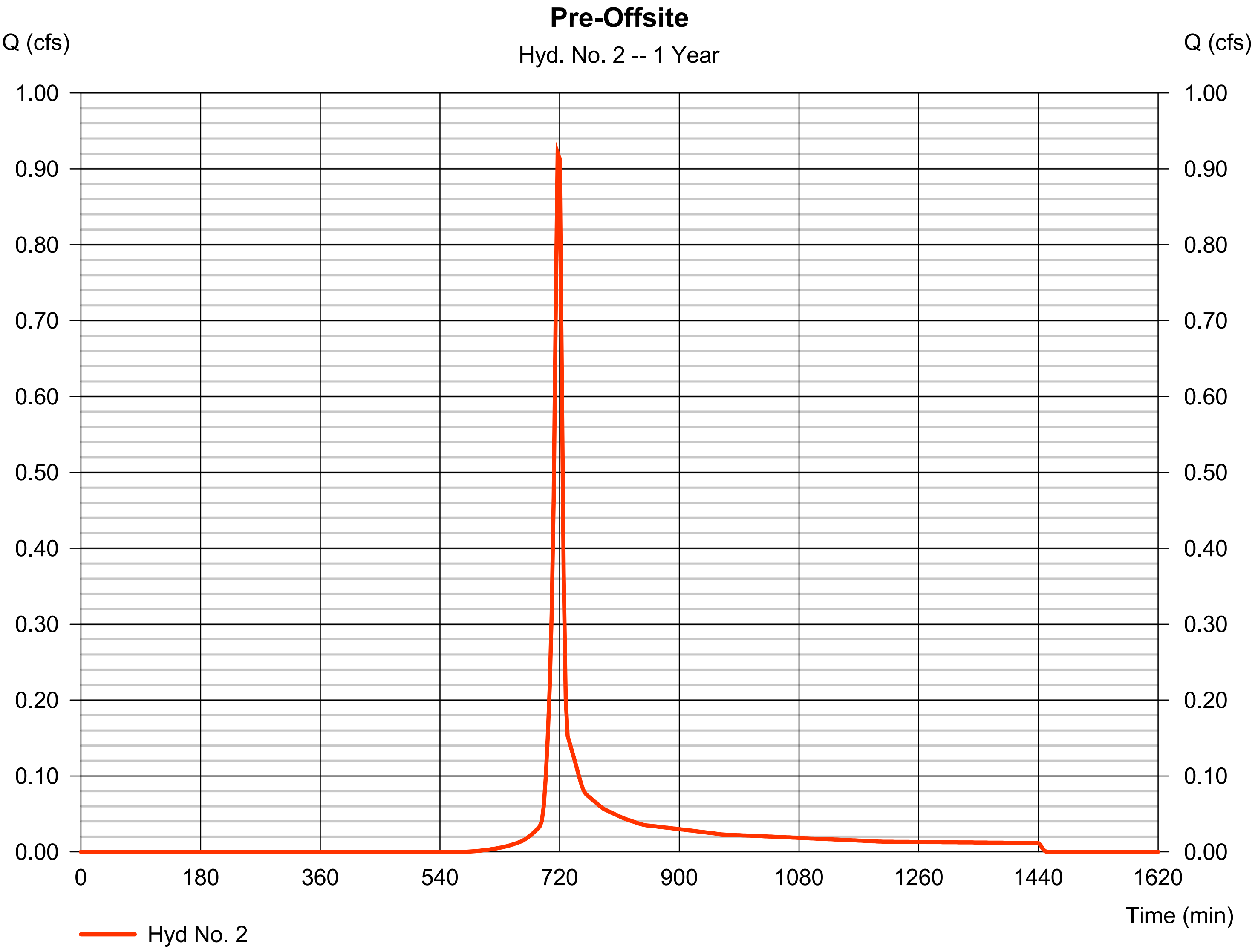
Hydrograph Report

Hyd. No. 2

Pre-Offsite

Hydrograph type	= SCS Runoff	Peak discharge	= 0.921 cfs
Storm frequency	= 1 yrs	Time to peak	= 717 min
Time interval	= 3 min	Hyd. volume	= 2,098 cuft
Drainage area	= 0.640 ac	Curve number	= 84*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 6.00 min
Total precip.	= 2.30 in	Distribution	= Type II
Storm duration	= 24 hrs	Shape factor	= 484

* Composite (Area/CN) = [(0.370 x 74) + (0.270 x 98)] / 0.640



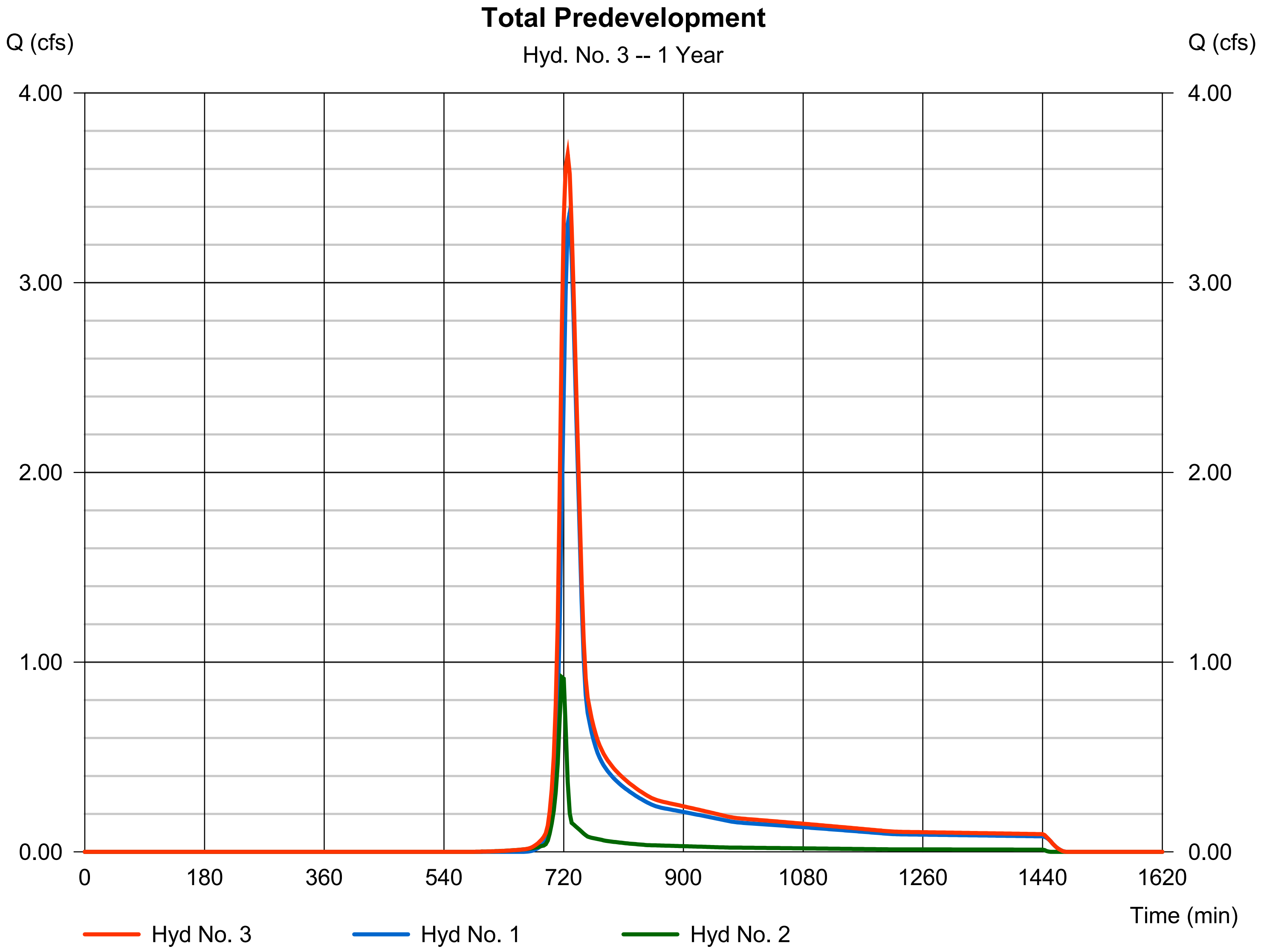
Hydrograph Report

Hyd. No. 3

Total Predevelopment

Hydrograph type = Combine
Storm frequency = 1 yrs
Time interval = 3 min
Inflow hyds. = 1, 2

Peak discharge = 3.676 cfs
Time to peak = 726 min
Hyd. volume = 14,799 cuft
Contrib. drain. area = 5.720 ac



Hydrograph Report

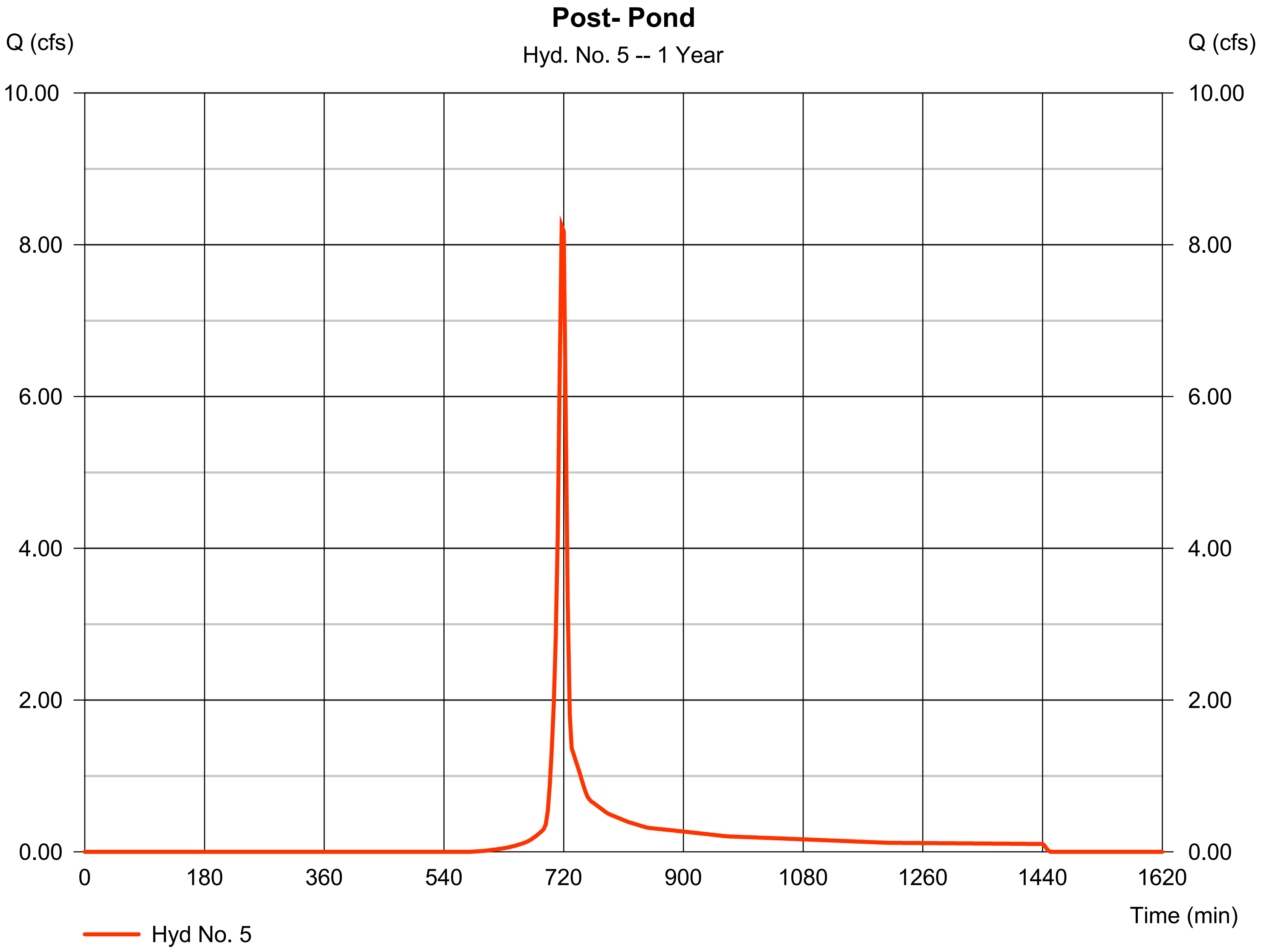
Hydraflow Hydrographs Extension for AutoCAD® Civil 3D® 2016 by Autodesk, Inc. v11 Monday, 07 / 9 / 2018

Hyd. No. 5

Post- Pond

Hydrograph type	= SCS Runoff	Peak discharge	= 8.247 cfs
Storm frequency	= 1 yrs	Time to peak	= 717 min
Time interval	= 3 min	Hyd. volume	= 18,781 cuft
Drainage area	= 5.730 ac	Curve number	= 84*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 6.00 min
Total precip.	= 2.30 in	Distribution	= Type II
Storm duration	= 24 hrs	Shape factor	= 484

* Composite (Area/CN) = [(0.260 x 61) + (2.350 x 98) + (2.020 x 74) + (1.100 x 80)] / 5.730



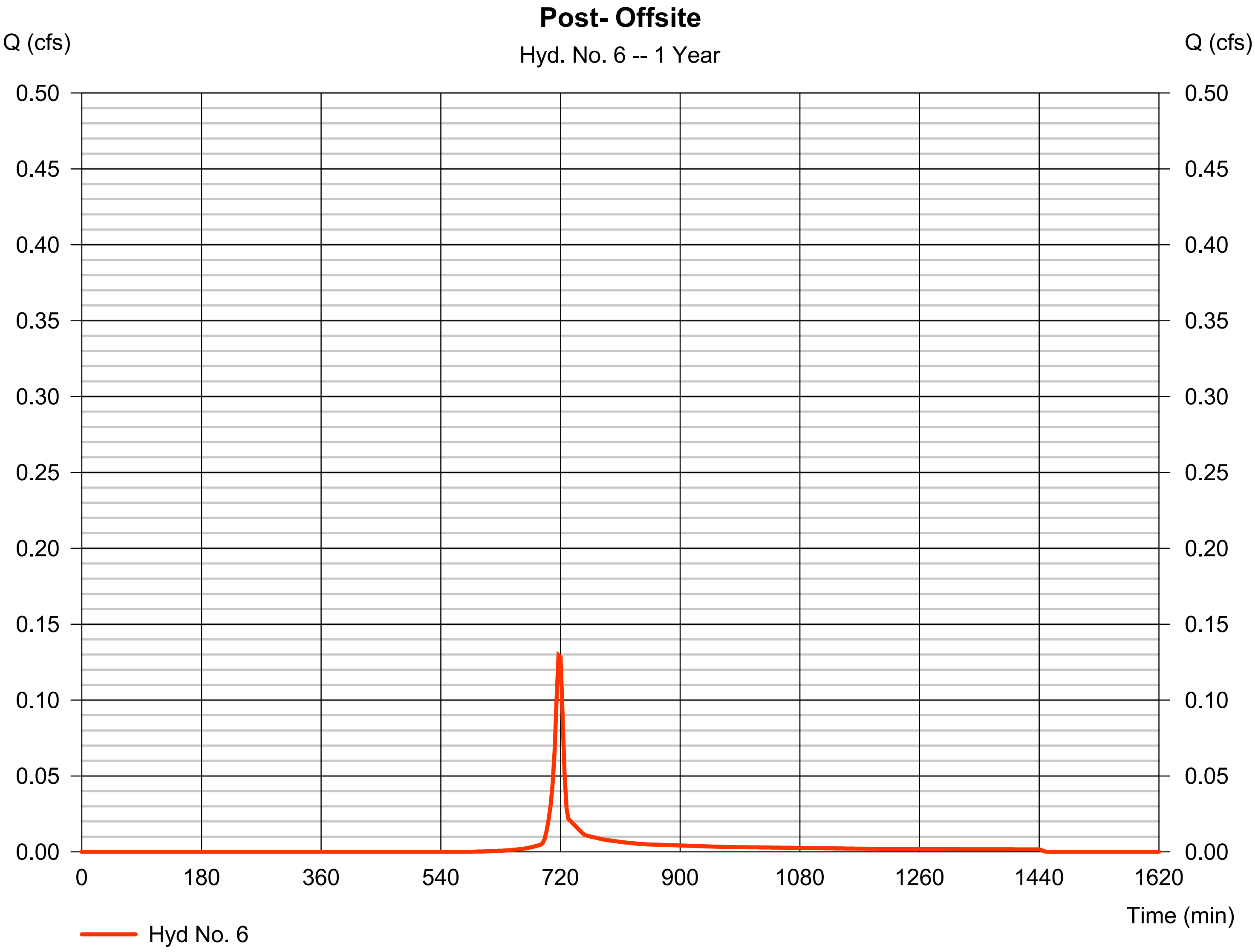
Hydrograph Report

Hyd. No. 6

Post- Offsite

Hydrograph type	= SCS Runoff	Peak discharge	= 0.130 cfs
Storm frequency	= 1 yrs	Time to peak	= 717 min
Time interval	= 3 min	Hyd. volume	= 295 cuft
Drainage area	= 0.090 ac	Curve number	= 84*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 6.00 min
Total precip.	= 2.30 in	Distribution	= Type II
Storm duration	= 24 hrs	Shape factor	= 484

* Composite (Area/CN) = [(0.030 x 61) + (0.050 x 98) + (0.010 x 80)] / 0.090



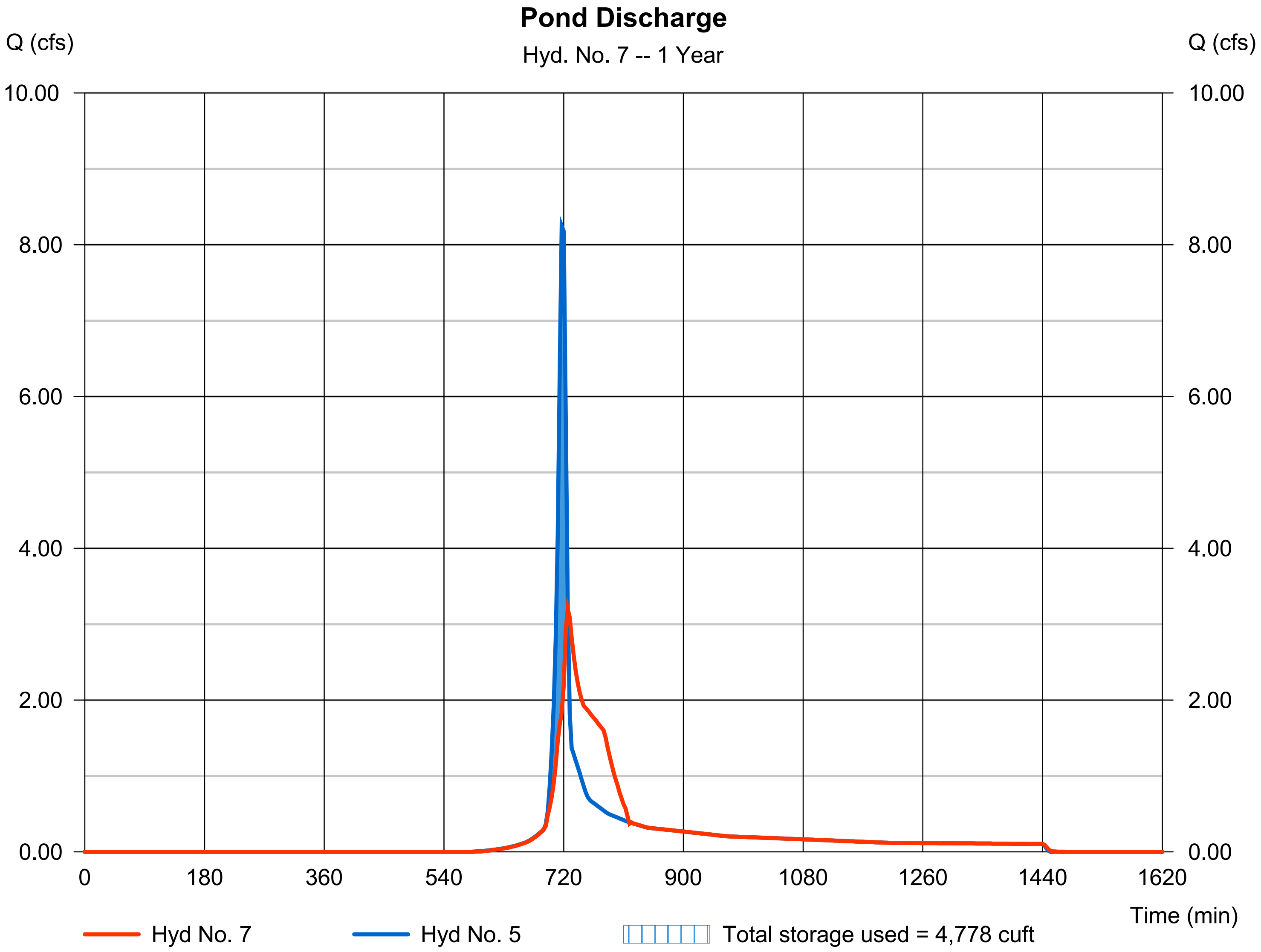
Hydrograph Report

Hyd. No. 7

Pond Discharge

Hydrograph type	= Reservoir	Peak discharge	= 3.204 cfs
Storm frequency	= 1 yrs	Time to peak	= 726 min
Time interval	= 3 min	Hyd. volume	= 18,780 cuft
Inflow hyd. No.	= 5 - Post- Pond	Max. Elevation	= 137.88 ft
Reservoir name	= South Detention	Max. Storage	= 4,778 cuft

Storage Indication method used.



Pond Report

Pond No. 1 - South Detention

Pond Data

Contours -User-defined contour areas. Conic method used for volume calculation. Begining Elevation = 135.50 ft

Stage / Storage Table

Stage (ft)	Elevation (ft)	Contour area (sqft)	Incr. Storage (cuft)	Total storage (cuft)
0.00	135.50	10	0	0
0.50	136.00	188	40	40
1.50	137.00	2,340	1,064	1,104
2.50	138.00	6,390	4,199	5,302
3.50	139.00	9,411	7,851	13,154
4.50	140.00	11,707	10,537	23,691
5.00	140.50	13,070	6,191	29,881

Culvert / Orifice Structures

	[A]	[B]	[C]	[PrfRsr]
Rise (in)	= 15.00	8.00	10.00	0.00
Span (in)	= 15.00	8.00	18.00	0.00
No. Barrels	= 1	1	1	0
Invert El. (ft)	= 135.50	135.50	137.50	0.00
Length (ft)	= 55.00	0.00	0.00	0.00
Slope (%)	= 11.00	0.00	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	Yes	No

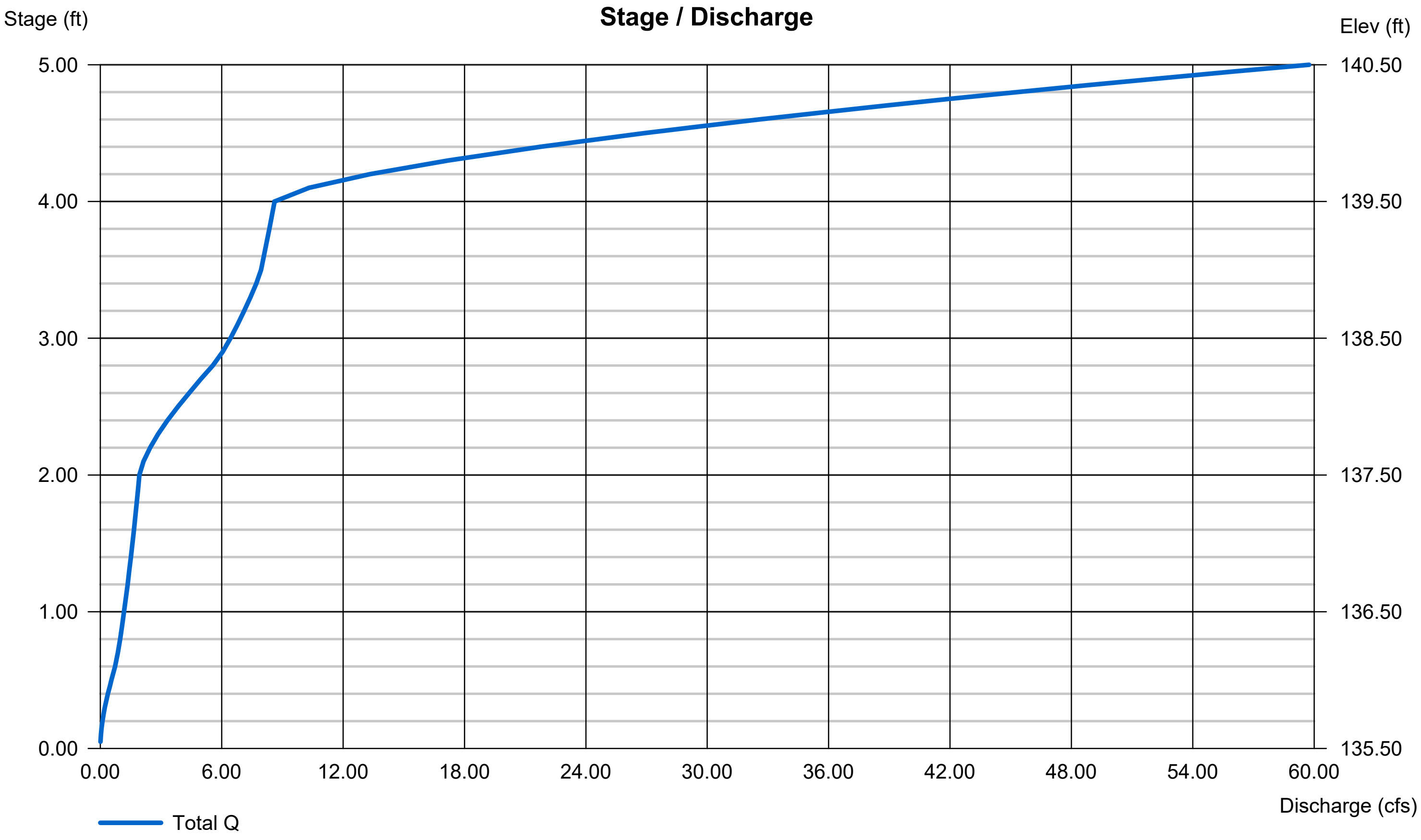
Weir Structures

	[A]	[B]	[C]	[D]
Crest Len (ft)	= 15.00	0.00	0.00	0.00
Crest El. (ft)	= 139.50	0.00	0.00	0.00
Weir Coeff.	= 3.33	3.33	3.33	3.33
Weir Type	= Rect	---	---	---
Multi-Stage	= No	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).

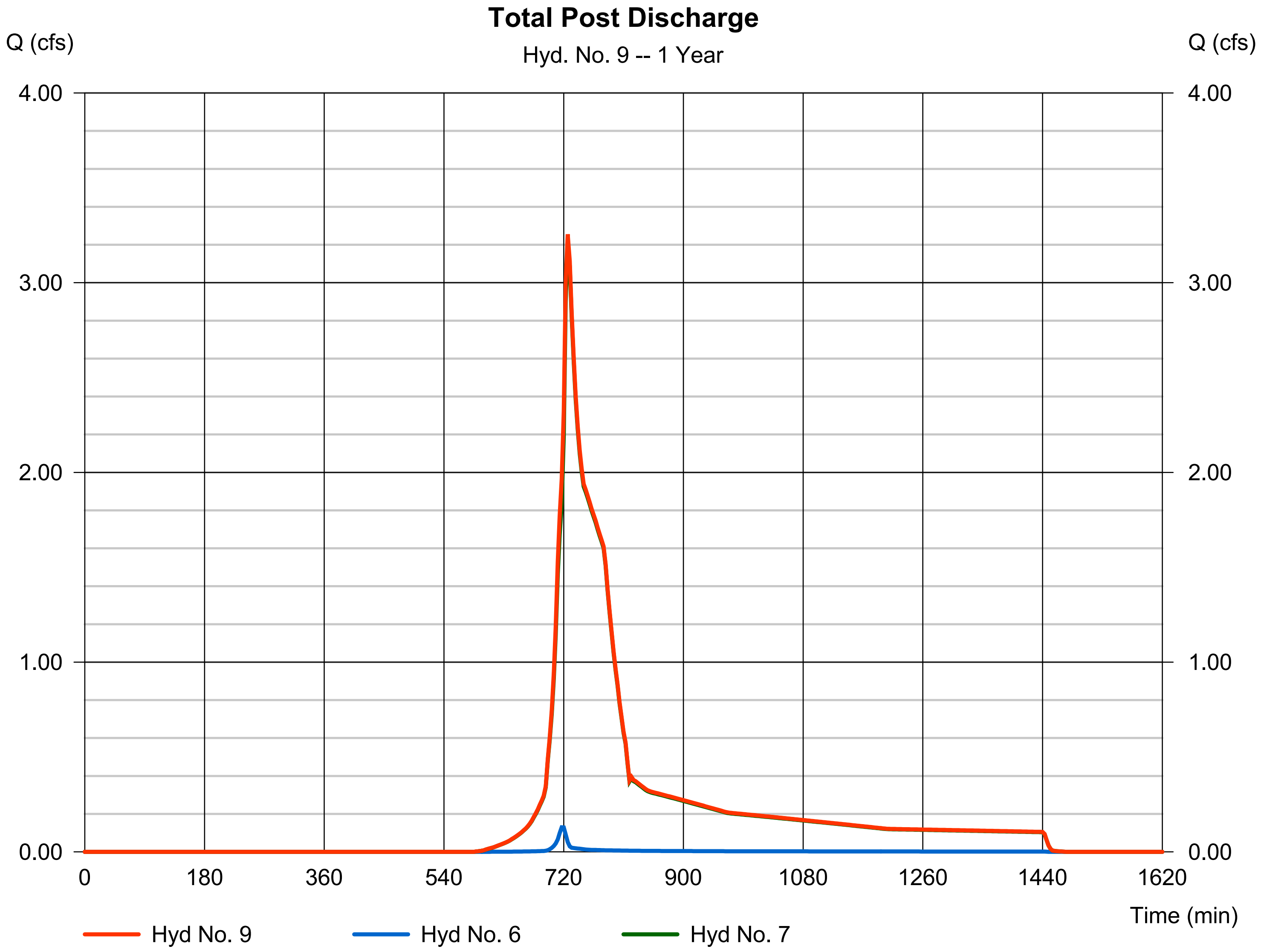


Hydrograph Report

Hyd. No. 9

Total Post Discharge

Hydrograph type	= Combine	Peak discharge	= 3.256 cfs
Storm frequency	= 1 yrs	Time to peak	= 726 min
Time interval	= 3 min	Hyd. volume	= 19,075 cuft
Inflow hyds.	= 6, 7	Contrib. drain. area	= 0.090 ac



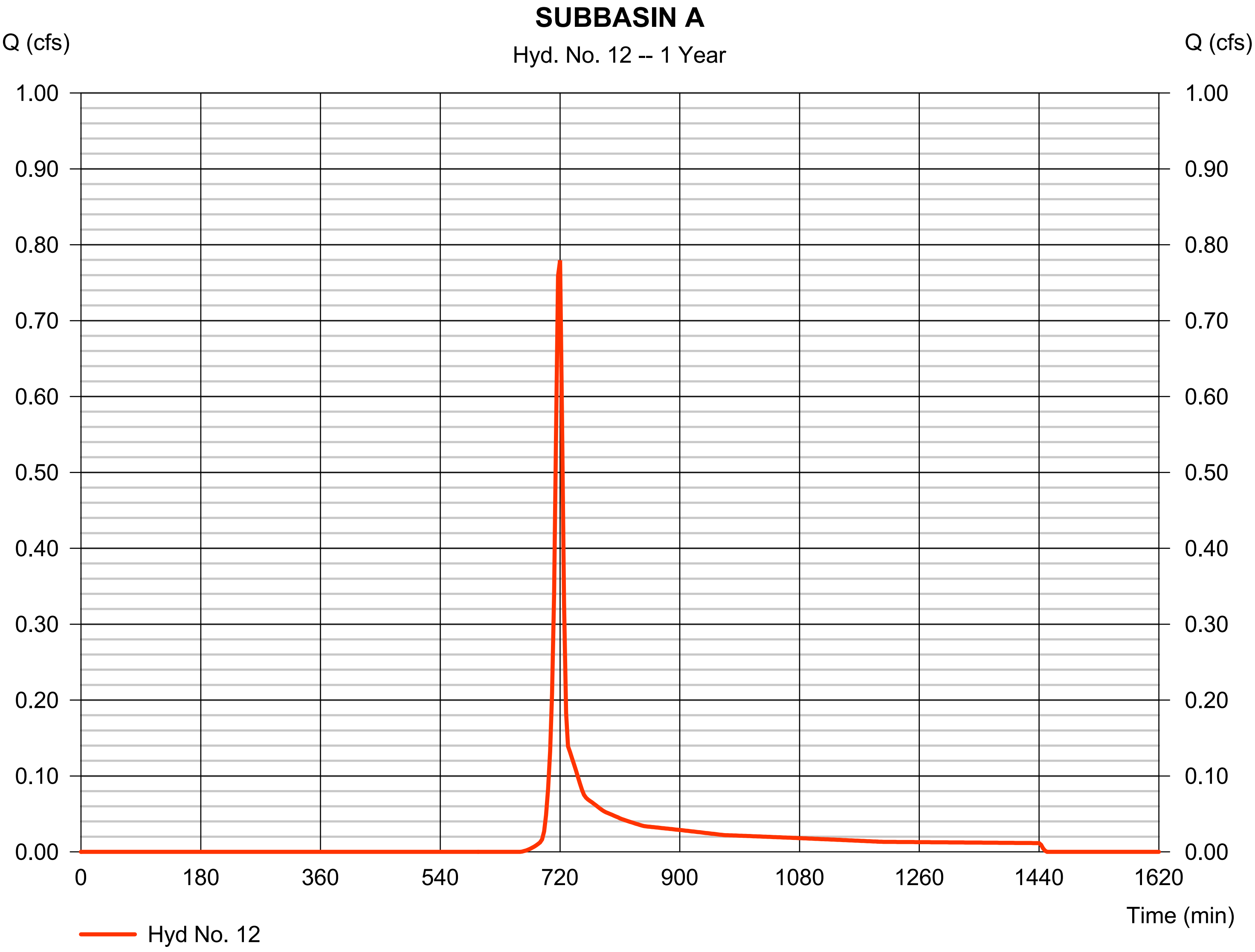
Hydrograph Report

Hyd. No. 12

SUBBASIN A

Hydrograph type	= SCS Runoff	Peak discharge	= 0.780 cfs
Storm frequency	= 1 yrs	Time to peak	= 720 min
Time interval	= 3 min	Hyd. volume	= 1,803 cuft
Drainage area	= 0.750 ac	Curve number	= 79*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 6.00 min
Total precip.	= 2.30 in	Distribution	= Type II
Storm duration	= 24 hrs	Shape factor	= 484

* Composite (Area/CN) = [(0.230 x 98) + (0.240 x 61) + (0.280 x 80)] / 0.750



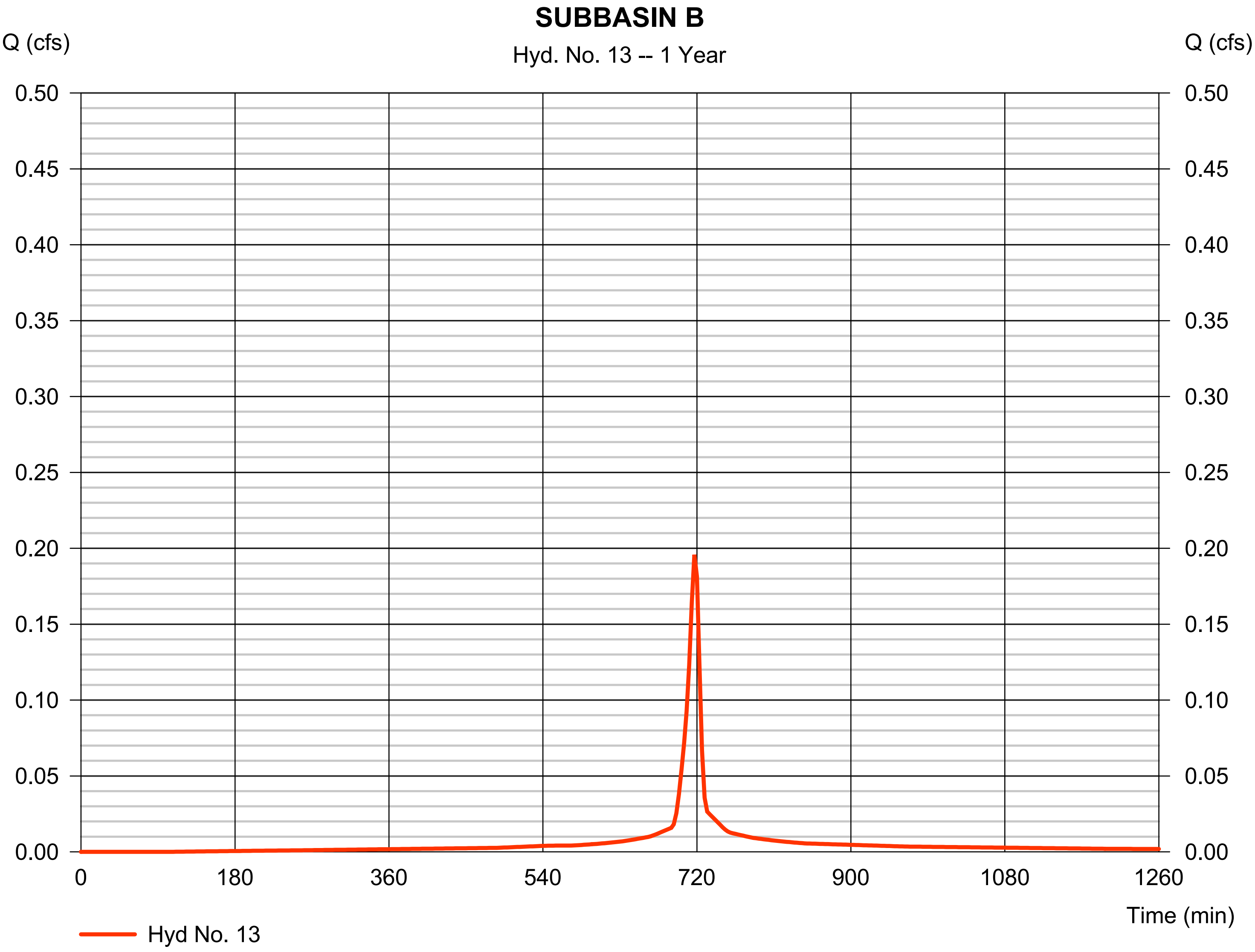
Hydrograph Report

Hyd. No. 13

SUBBASIN B

Hydrograph type	= SCS Runoff	Peak discharge	= 0.196 cfs
Storm frequency	= 1 yrs	Time to peak	= 717 min
Time interval	= 3 min	Hyd. volume	= 494 cuft
Drainage area	= 0.070 ac	Curve number	= 98*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 6.00 min
Total precip.	= 2.30 in	Distribution	= Type II
Storm duration	= 24 hrs	Shape factor	= 484

* Composite (Area/CN) = [(0.070 x 98)] / 0.070



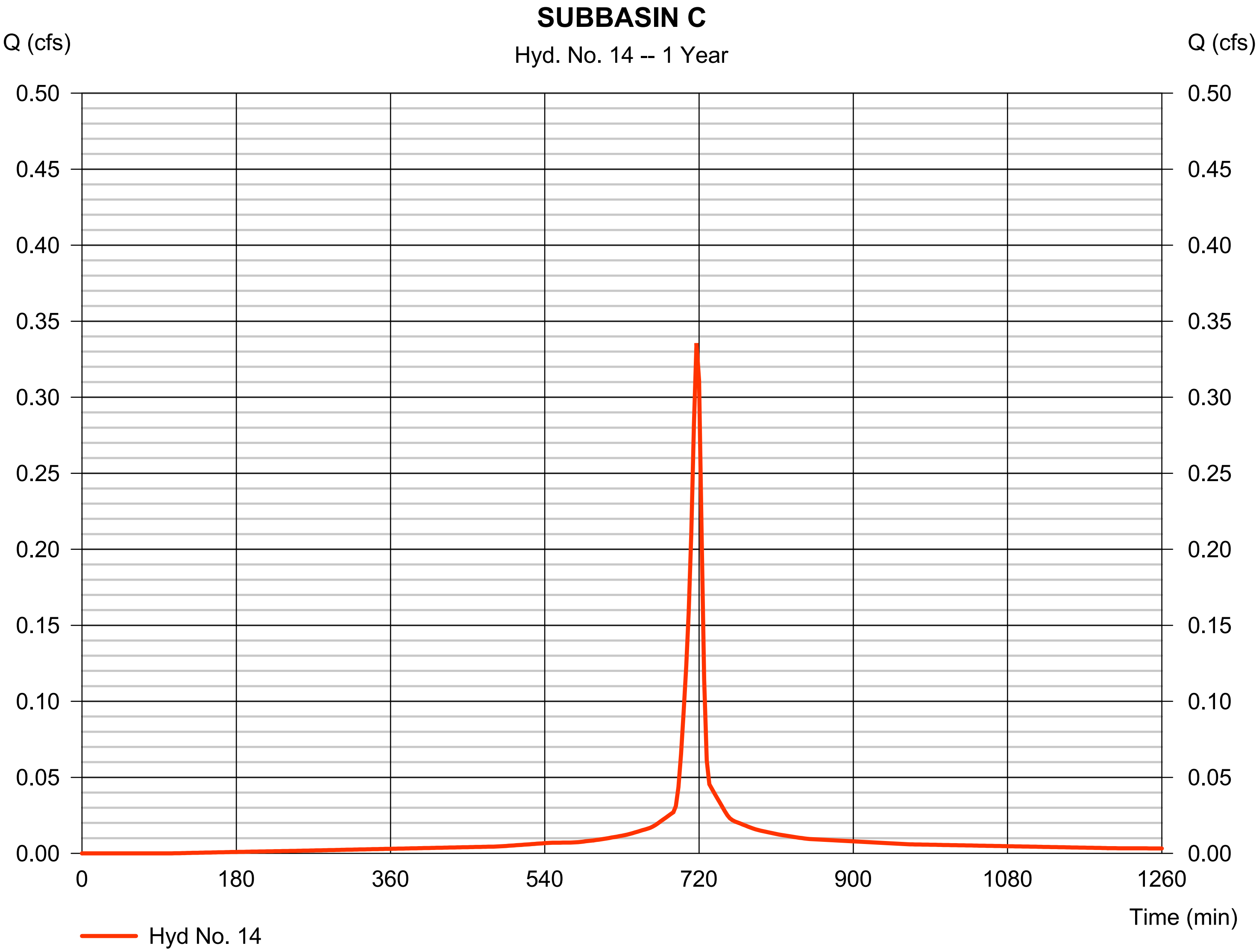
Hydrograph Report

Hyd. No. 14

SUBBASIN C

Hydrograph type	= SCS Runoff	Peak discharge	= 0.336 cfs
Storm frequency	= 1 yrs	Time to peak	= 717 min
Time interval	= 3 min	Hyd. volume	= 846 cuft
Drainage area	= 0.120 ac	Curve number	= 98*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 6.00 min
Total precip.	= 2.30 in	Distribution	= Type II
Storm duration	= 24 hrs	Shape factor	= 484

* Composite (Area/CN) = [(0.120 x 98)] / 0.120



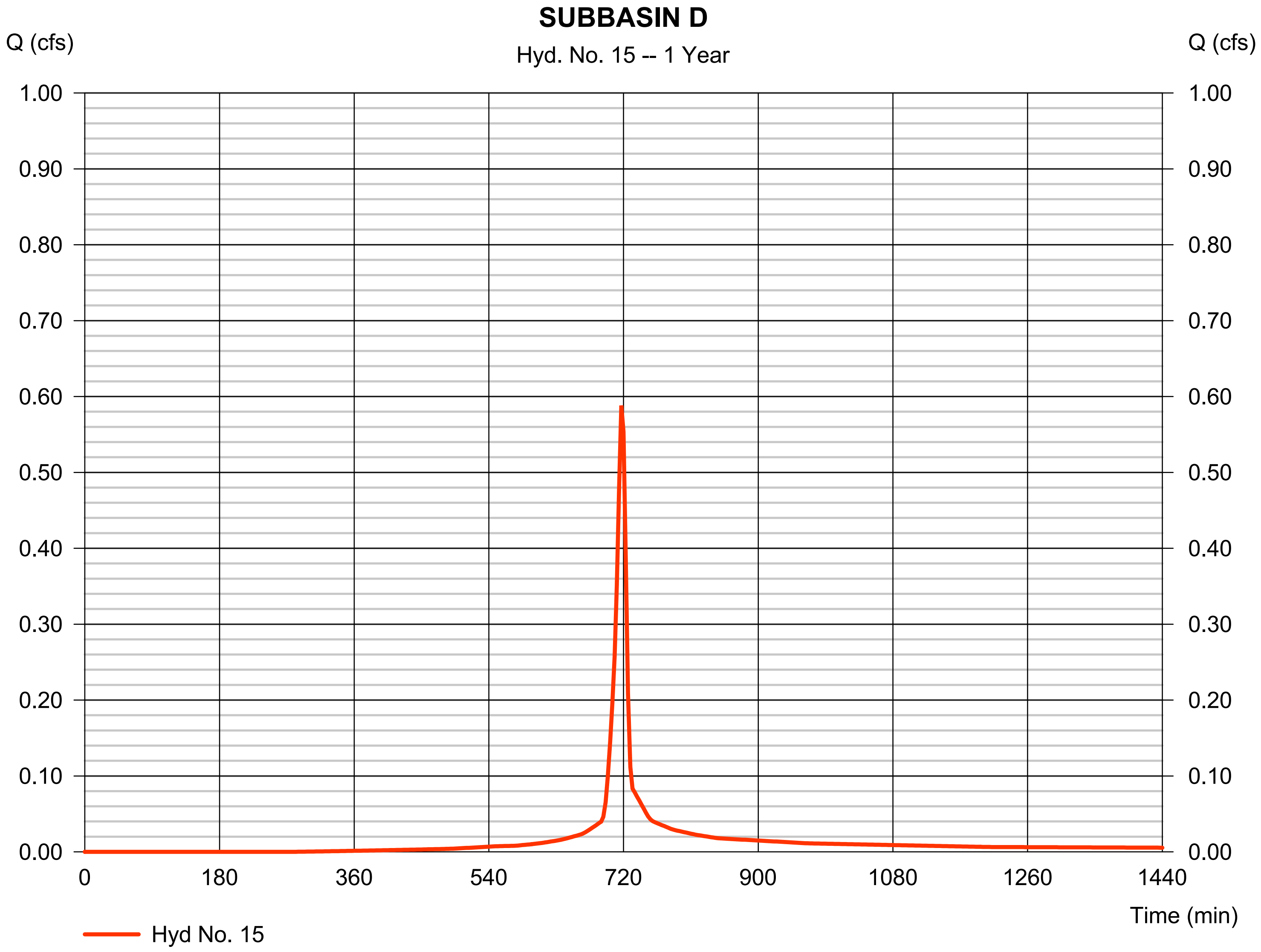
Hydrograph Report

Hyd. No. 15

SUBBASIN D

Hydrograph type	= SCS Runoff	Peak discharge	= 0.588 cfs
Storm frequency	= 1 yrs	Time to peak	= 717 min
Time interval	= 3 min	Hyd. volume	= 1,371 cuft
Drainage area	= 0.240 ac	Curve number	= 94*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 6.00 min
Total precip.	= 2.30 in	Distribution	= Type II
Storm duration	= 24 hrs	Shape factor	= 484

* Composite (Area/CN) = [(0.190 x 98) + (0.050 x 80)] / 0.240



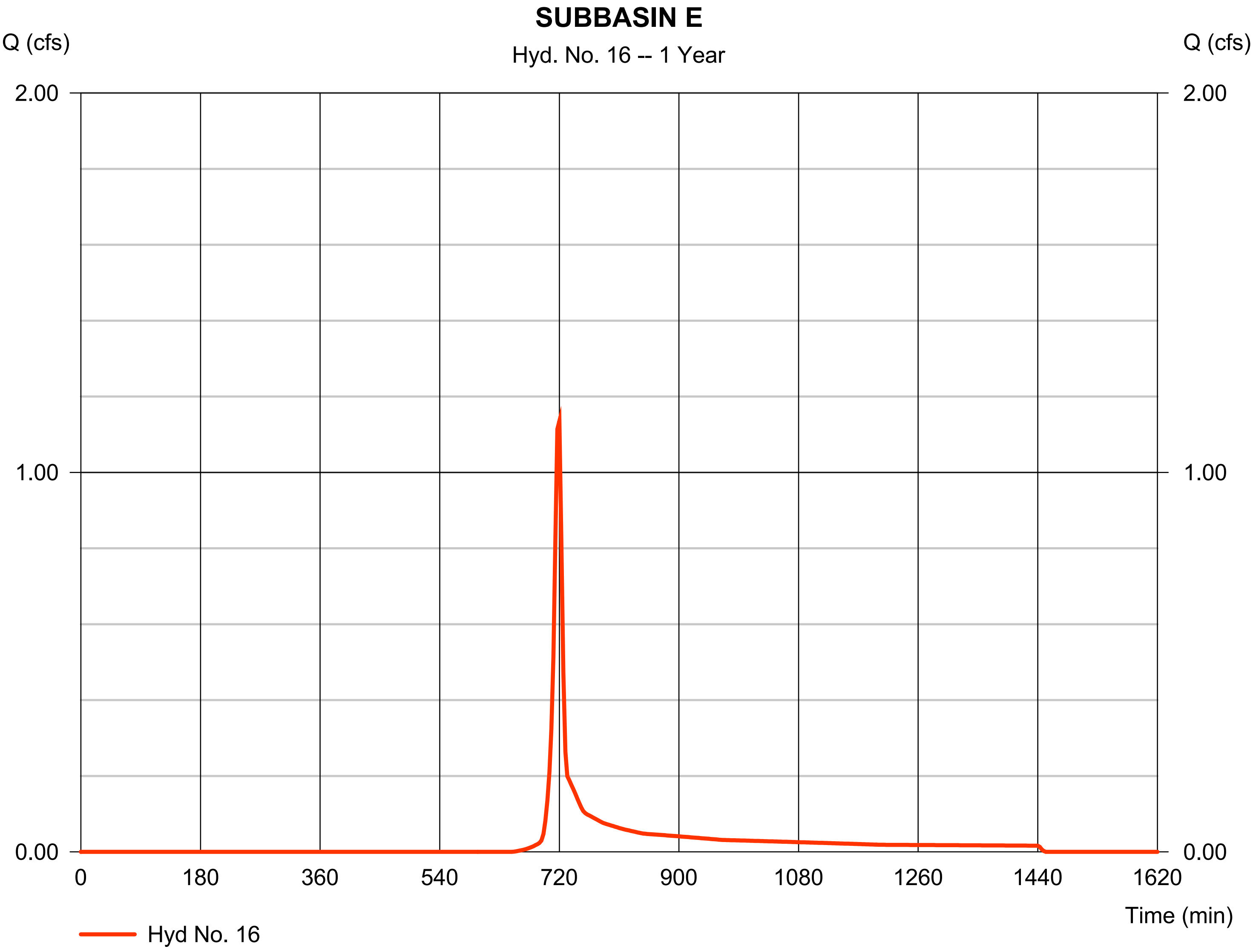
Hydrograph Report

Hyd. No. 16

SUBBASIN E

Hydrograph type	= SCS Runoff	Peak discharge	= 1.135 cfs
Storm frequency	= 1 yrs	Time to peak	= 720 min
Time interval	= 3 min	Hyd. volume	= 2,615 cuft
Drainage area	= 1.020 ac	Curve number	= 80*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 6.00 min
Total precip.	= 2.30 in	Distribution	= Type II
Storm duration	= 24 hrs	Shape factor	= 484

* Composite (Area/CN) = [(0.020 x 98) + (1.000 x 80)] / 1.020



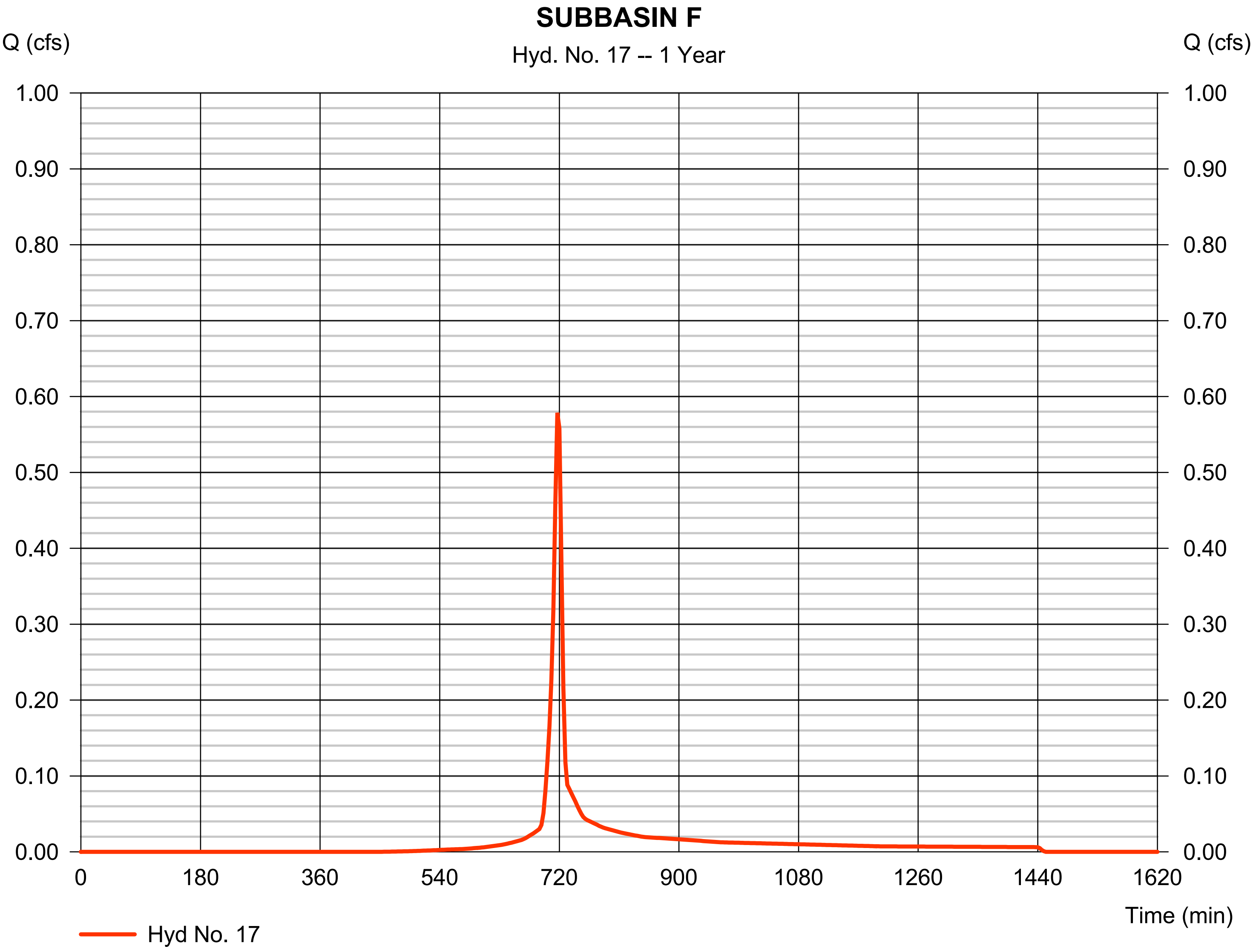
Hydrograph Report

Hyd. No. 17

SUBBASIN F

Hydrograph type	= SCS Runoff	Peak discharge	= 0.579 cfs
Storm frequency	= 1 yrs	Time to peak	= 717 min
Time interval	= 3 min	Hyd. volume	= 1,308 cuft
Drainage area	= 0.300 ac	Curve number	= 89*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 6.00 min
Total precip.	= 2.30 in	Distribution	= Type II
Storm duration	= 24 hrs	Shape factor	= 484

* Composite (Area/CN) = [(0.190 x 98) + (0.110 x 74)] / 0.300



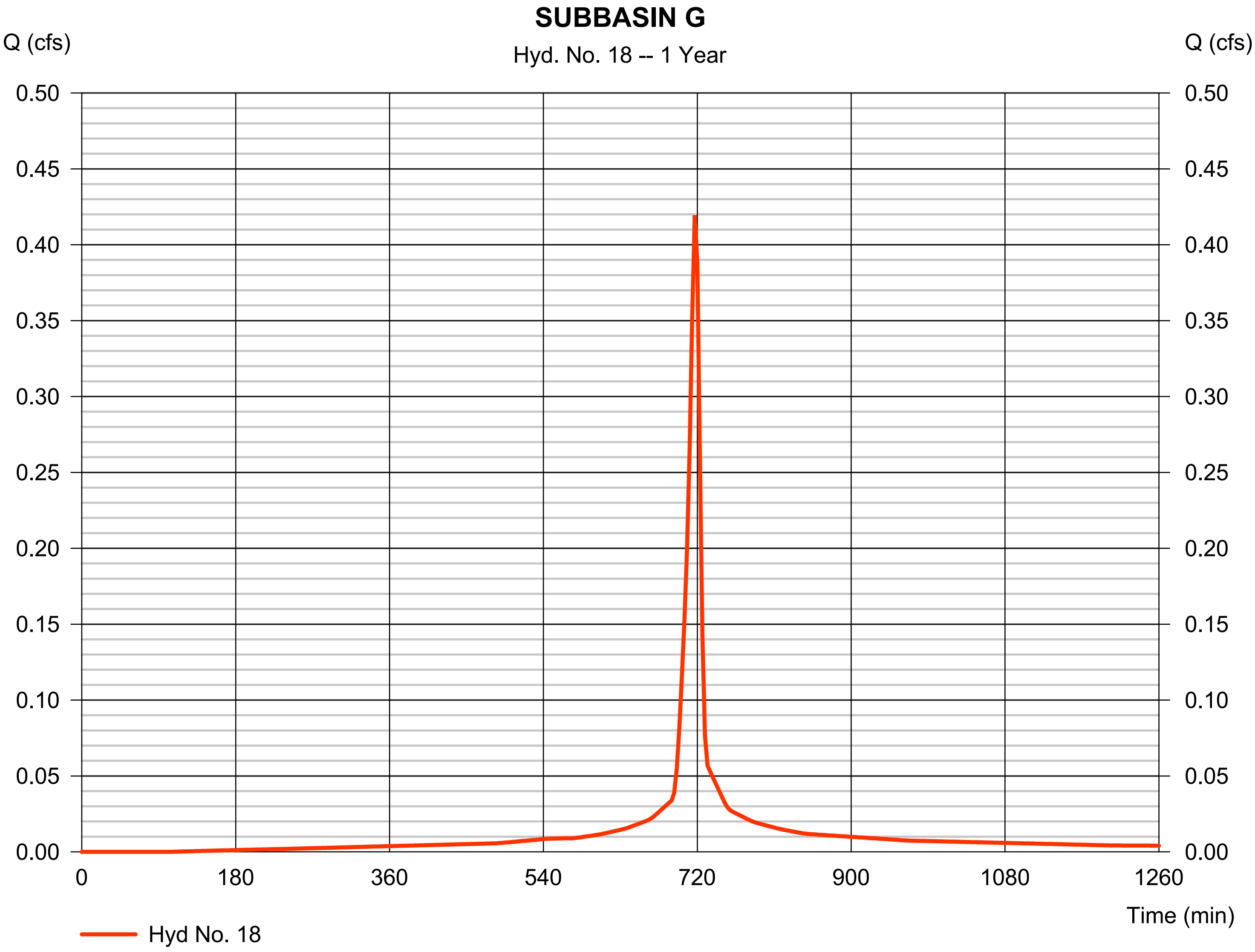
Hydrograph Report

Hyd. No. 18

SUBBASIN G

Hydrograph type	= SCS Runoff	Peak discharge	= 0.419 cfs
Storm frequency	= 1 yrs	Time to peak	= 717 min
Time interval	= 3 min	Hyd. volume	= 1,058 cuft
Drainage area	= 0.150 ac	Curve number	= 98*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 6.00 min
Total precip.	= 2.30 in	Distribution	= Type II
Storm duration	= 24 hrs	Shape factor	= 484

* Composite (Area/CN) = [(0.150 x 98)] / 0.150



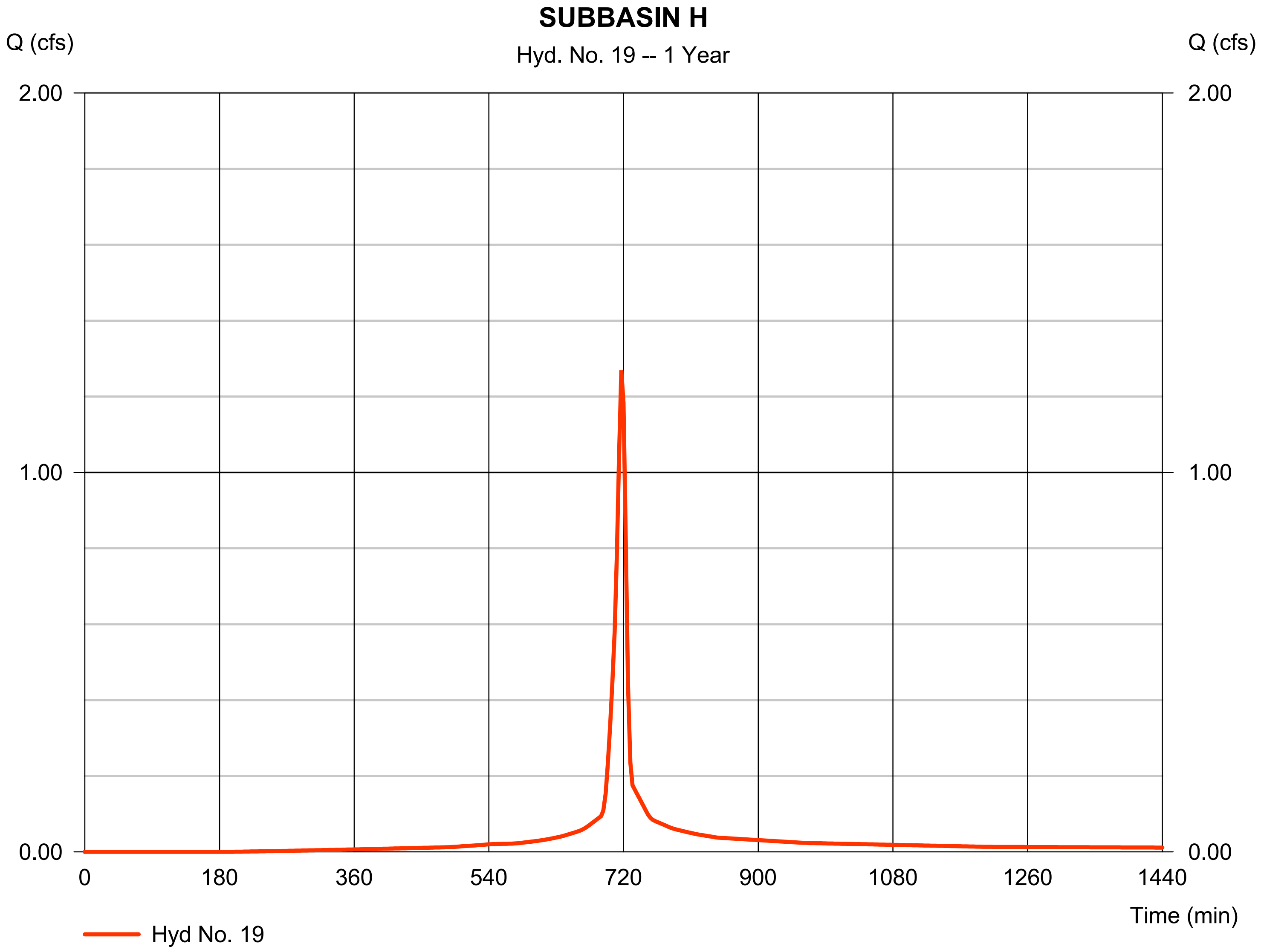
Hydrograph Report

Hyd. No. 19

SUBBASIN H

Hydrograph type	= SCS Runoff	Peak discharge	= 1.269 cfs
Storm frequency	= 1 yrs	Time to peak	= 717 min
Time interval	= 3 min	Hyd. volume	= 3,048 cuft
Drainage area	= 0.480 ac	Curve number	= 96*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 6.00 min
Total precip.	= 2.30 in	Distribution	= Type II
Storm duration	= 24 hrs	Shape factor	= 484

* Composite (Area/CN) = [(0.430 x 98) + (0.050 x 74)] / 0.480



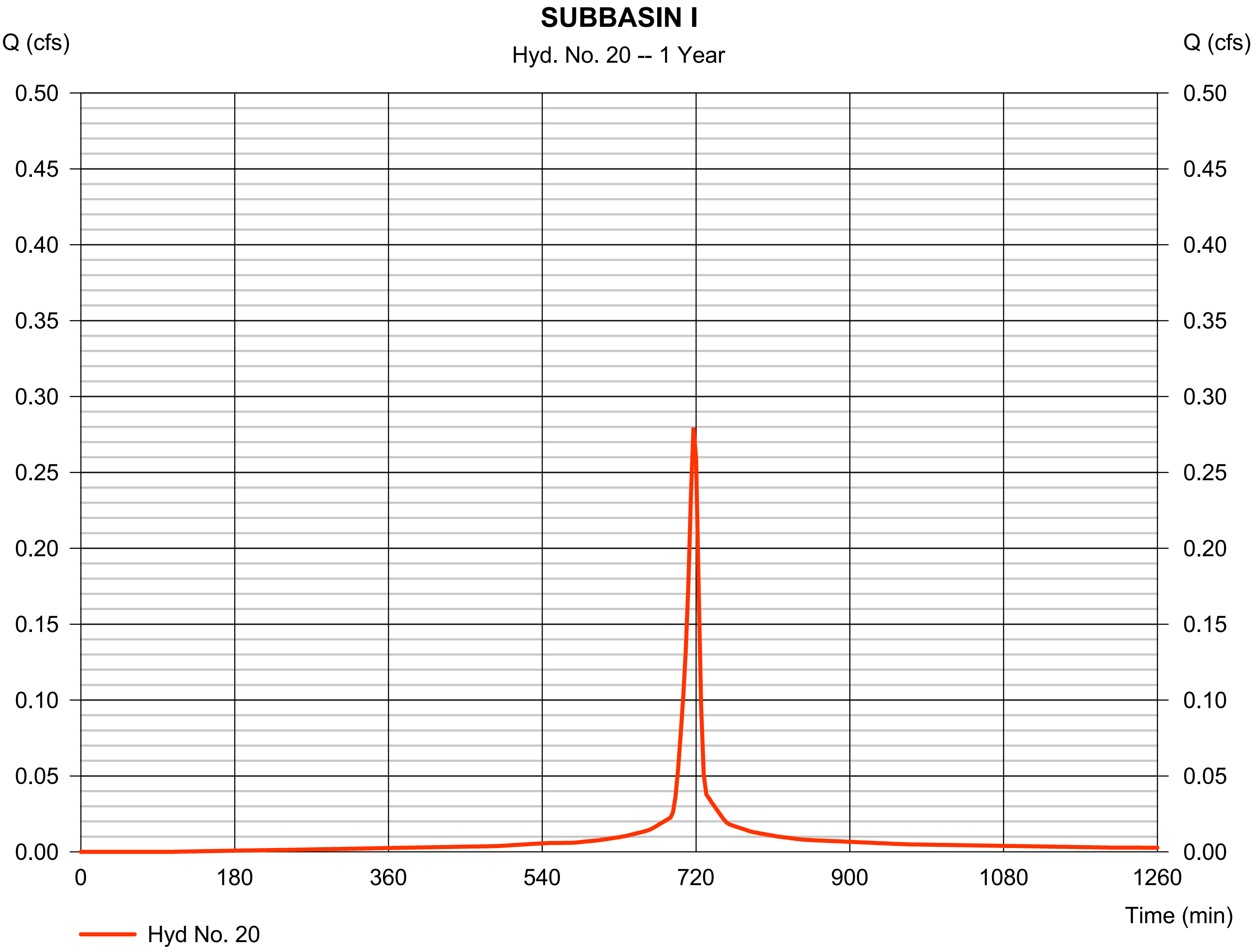
Hydrograph Report

Hyd. No. 20

SUBBASIN I

Hydrograph type	= SCS Runoff	Peak discharge	= 0.280 cfs
Storm frequency	= 1 yrs	Time to peak	= 717 min
Time interval	= 3 min	Hyd. volume	= 705 cuft
Drainage area	= 0.100 ac	Curve number	= 98*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 6.00 min
Total precip.	= 2.30 in	Distribution	= Type II
Storm duration	= 24 hrs	Shape factor	= 484

* Composite (Area/CN) = [(0.100 x 98)] / 0.100



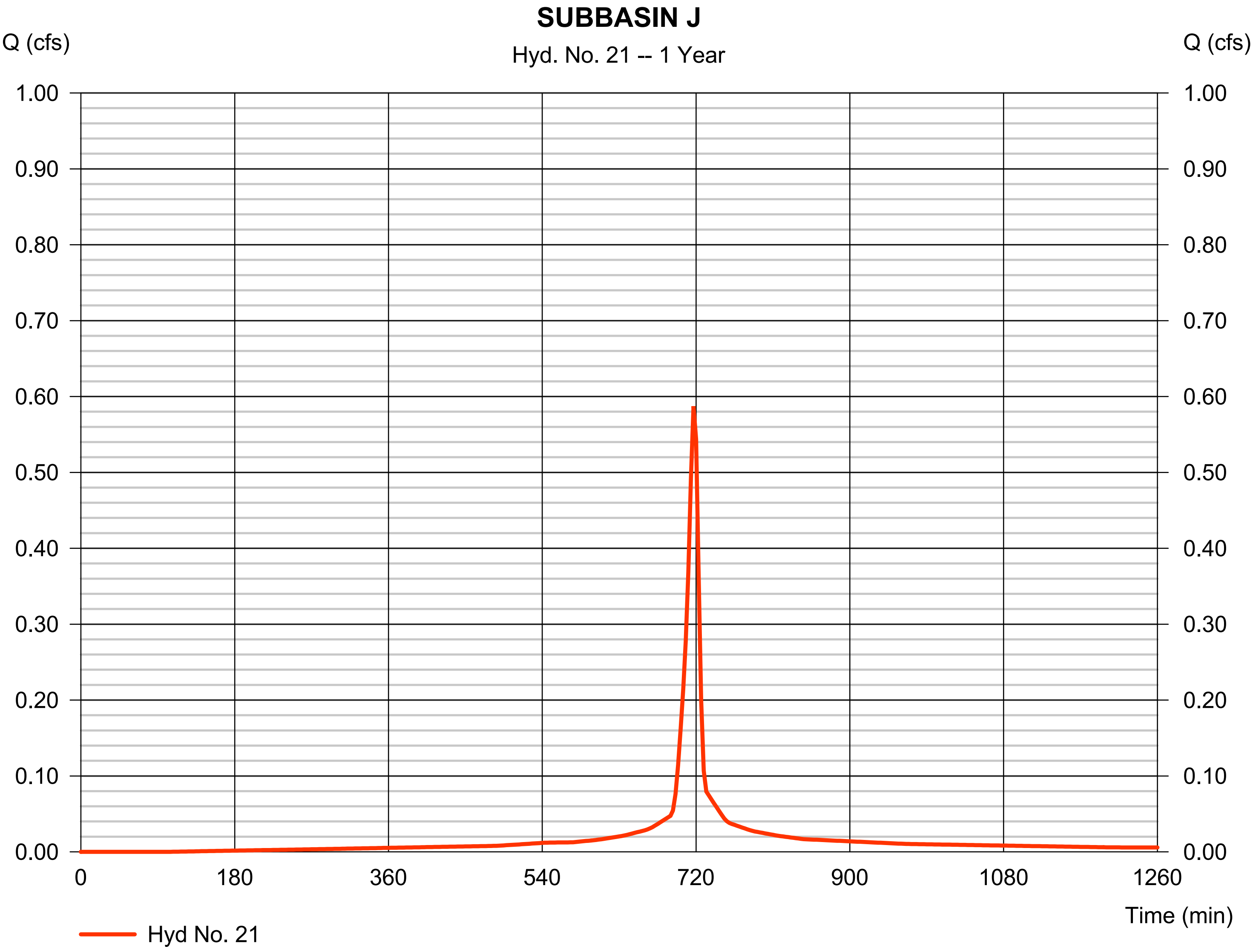
Hydrograph Report

Hyd. No. 21

SUBBASIN J

Hydrograph type	=	SCS Runoff	Peak discharge	=	0.587 cfs
Storm frequency	=	1 yrs	Time to peak	=	717 min
Time interval	=	3 min	Hyd. volume	=	1,481 cuft
Drainage area	=	0.210 ac	Curve number	=	98*
Basin Slope	=	0.0 %	Hydraulic length	=	0 ft
Tc method	=	User	Time of conc. (Tc)	=	6.00 min
Total precip.	=	2.30 in	Distribution	=	Type II
Storm duration	=	24 hrs	Shape factor	=	484

* Composite (Area/CN) = [(0.210 x 98)] / 0.210



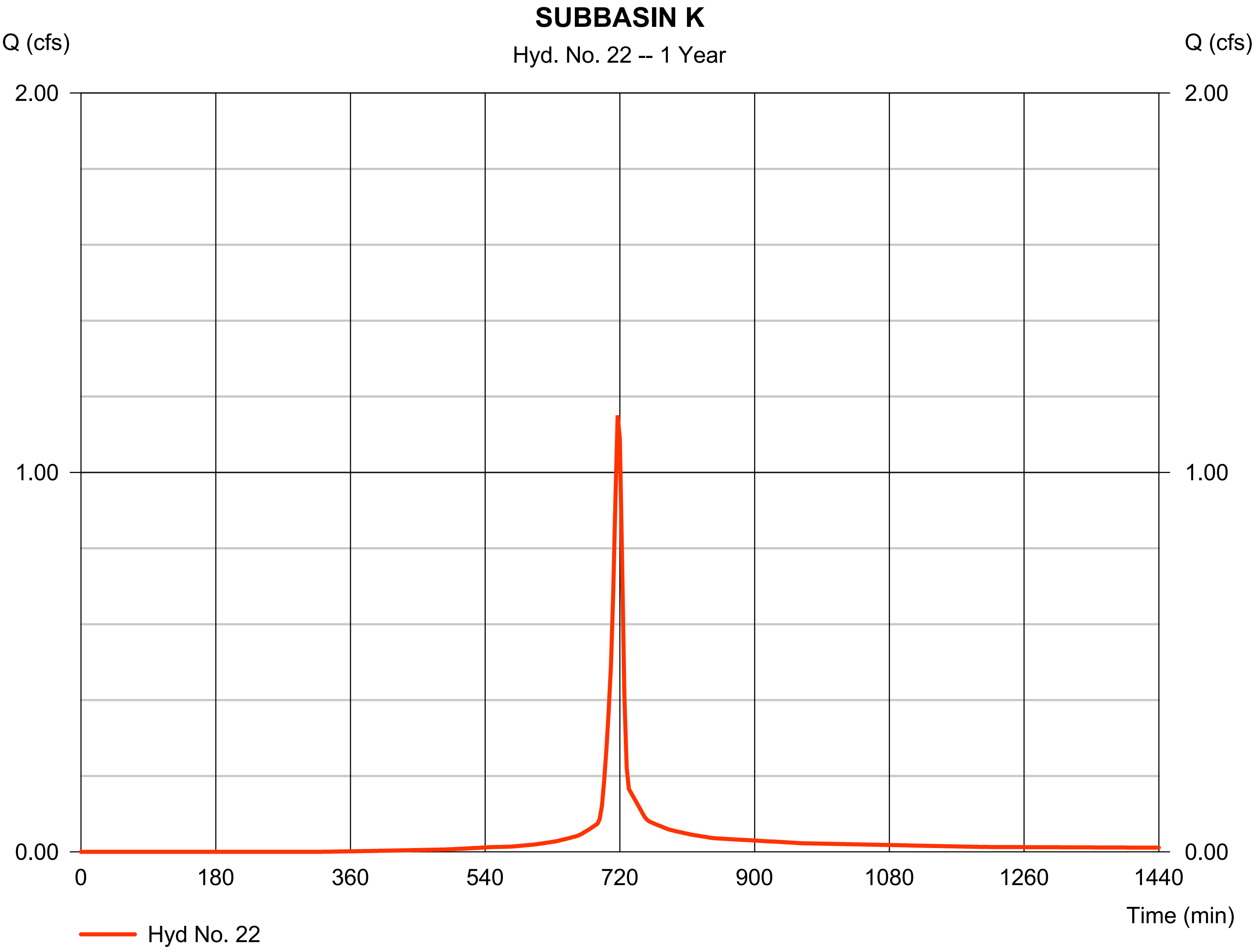
Hydrograph Report

Hyd. No. 22

SUBBASIN K

Hydrograph type	=	SCS Runoff	Peak discharge	=	1.150 cfs
Storm frequency	=	1 yrs	Time to peak	=	717 min
Time interval	=	3 min	Hyd. volume	=	2,655 cuft
Drainage area	=	0.490 ac	Curve number	=	93*
Basin Slope	=	0.0 %	Hydraulic length	=	0 ft
Tc method	=	User	Time of conc. (Tc)	=	6.00 min
Total precip.	=	2.30 in	Distribution	=	Type II
Storm duration	=	24 hrs	Shape factor	=	484

* Composite (Area/CN) = [(0.380 x 98) + (0.110 x 74)] / 0.490



Hydrograph Summary Report

Hydraflow Hydrographs Extension for AutoCAD® Civil 3D® 2016 by Autodesk, Inc. v11

Hyd. No.	Hydrograph type (origin)	Peak flow (cfs)	Time interval (min)	Time to Peak (min)	Hyd. volume (cuft)	Inflow hyd(s)	Maximum elevation (ft)	Total strge used (cuft)	Hydrograph Description
1	SCS Runoff	4.778	3	729	17,514	------	------	------	Pre-Basin A
2	SCS Runoff	1.226	3	717	2,773	------	------	------	Pre-Offsite
3	Combine	5.219	3	726	20,288	1, 2	------	------	Total Predevelopment
5	SCS Runoff	10.98	3	717	24,828	------	------	------	Post- Pond
6	SCS Runoff	0.172	3	717	390	------	------	------	Post- Offsite
7	Reservoir	4.642	3	726	24,828	5	138.15	6,438	Pond Discharge
9	Combine	4.709	3	726	25,218	6, 7,	------	------	Total Post Discharge
12	SCS Runoff	1.083	3	720	2,486	------	------	------	SUBBASIN A
13	SCS Runoff	0.231	3	717	588	------	------	------	SUBBASIN B
14	SCS Runoff	0.396	3	717	1,009	------	------	------	SUBBASIN C
15	SCS Runoff	0.714	3	717	1,683	------	------	------	SUBBASIN D
16	SCS Runoff	1.558	3	720	3,575	------	------	------	SUBBASIN E
17	SCS Runoff	0.733	3	717	1,665	------	------	------	SUBBASIN F
18	SCS Runoff	0.495	3	717	1,261	------	------	------	SUBBASIN G
19	SCS Runoff	1.517	3	717	3,687	------	------	------	SUBBASIN H
20	SCS Runoff	0.330	3	717	840	------	------	------	SUBBASIN I
21	SCS Runoff	0.693	3	717	1,765	------	------	------	SUBBASIN J
22	SCS Runoff	1.408	3	717	3,282	------	------	------	SUBBASIN K

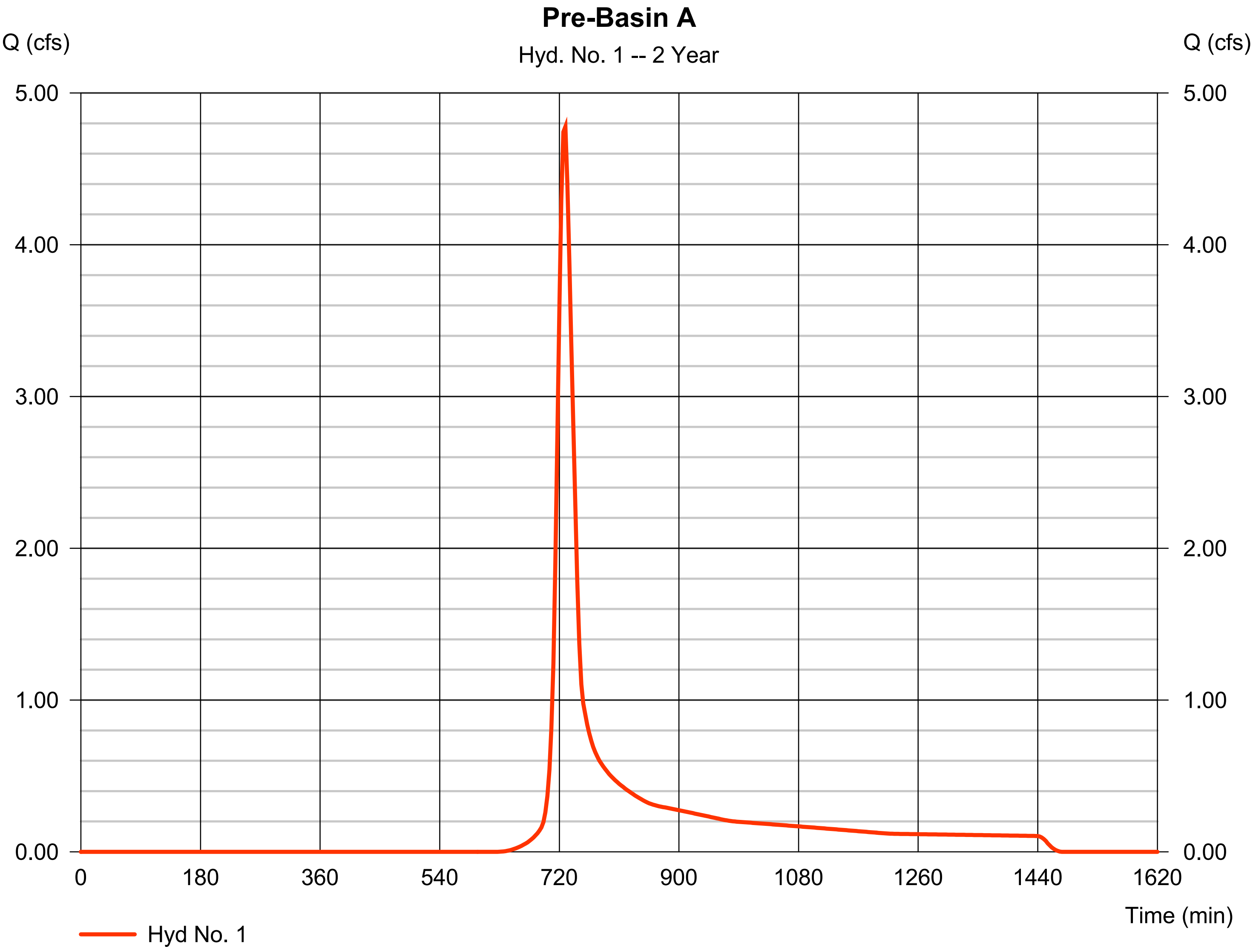
Hydrograph Report

Hyd. No. 1

Pre-Basin A

Hydrograph type	= SCS Runoff	Peak discharge	= 4.778 cfs
Storm frequency	= 2 yrs	Time to peak	= 729 min
Time interval	= 3 min	Hyd. volume	= 17,514 cuft
Drainage area	= 5.080 ac	Curve number	= 79*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= TR55	Time of conc. (Tc)	= 21.40 min
Total precip.	= 2.70 in	Distribution	= Type II
Storm duration	= 24 hrs	Shape factor	= 484

* Composite (Area/CN) = [(0.260 x 61) + (1.160 x 98) + (1.170 x 74) + (1.230 x 70) + (0.670 x 80) + (0.590 x 77)] / 5.080



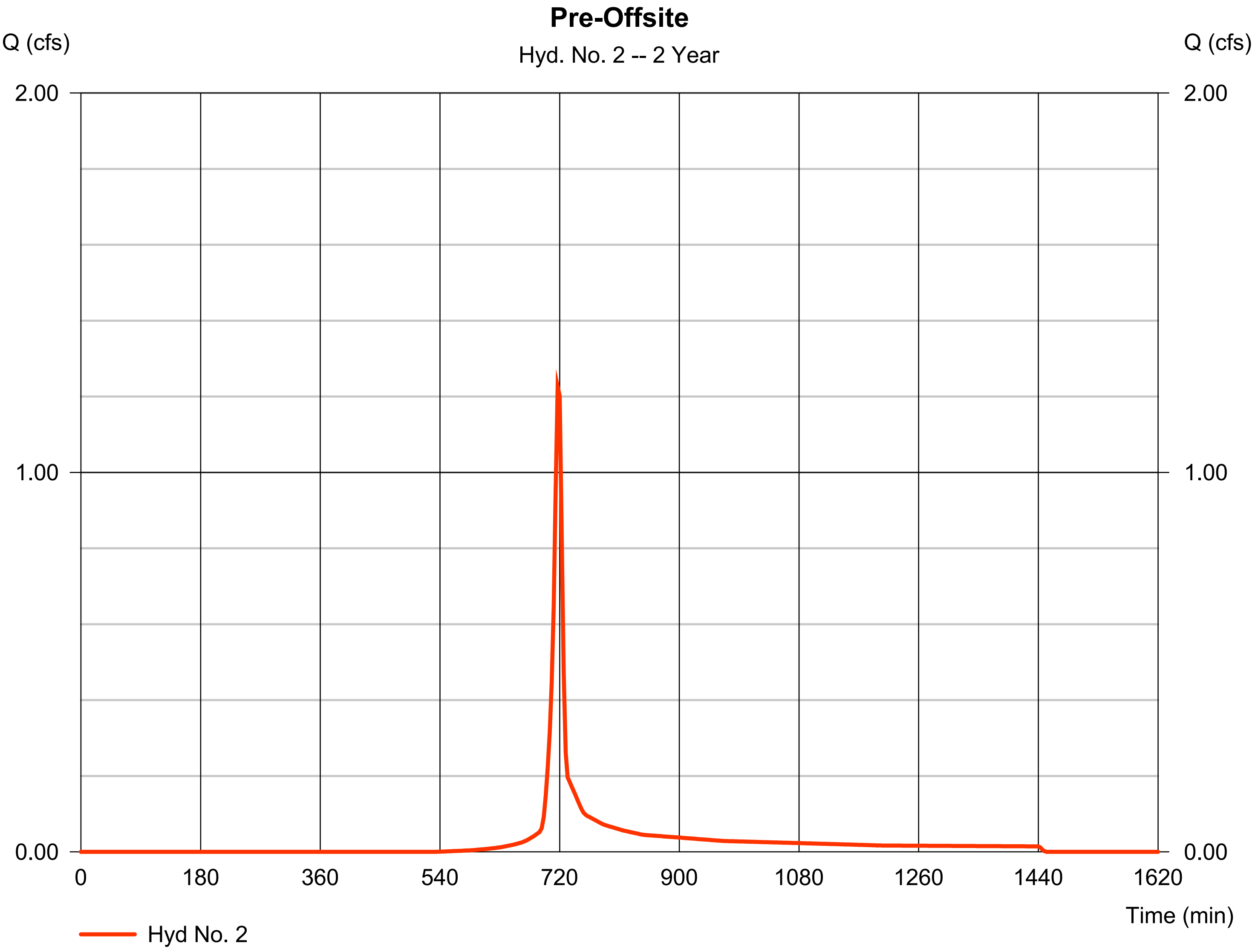
Hydrograph Report

Hyd. No. 2

Pre-Offsite

Hydrograph type	= SCS Runoff	Peak discharge	= 1.226 cfs
Storm frequency	= 2 yrs	Time to peak	= 717 min
Time interval	= 3 min	Hyd. volume	= 2,773 cuft
Drainage area	= 0.640 ac	Curve number	= 84*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 6.00 min
Total precip.	= 2.70 in	Distribution	= Type II
Storm duration	= 24 hrs	Shape factor	= 484

* Composite (Area/CN) = [(0.370 x 74) + (0.270 x 98)] / 0.640

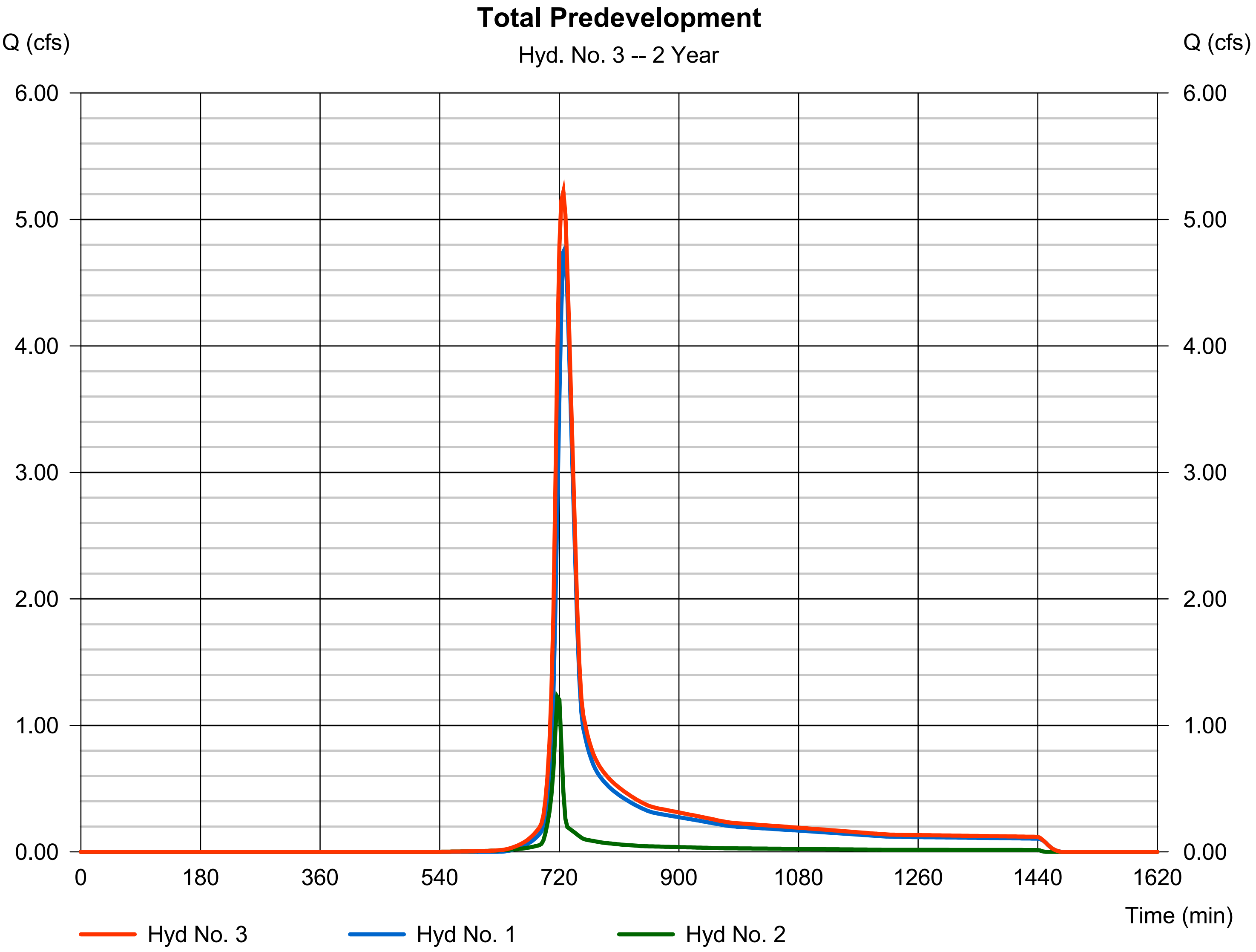


Hydrograph Report

Hyd. No. 3

Total Predevelopment

Hydrograph type	= Combine	Peak discharge	= 5.219 cfs
Storm frequency	= 2 yrs	Time to peak	= 726 min
Time interval	= 3 min	Hyd. volume	= 20,288 cuft
Inflow hyds.	= 1, 2	Contrib. drain. area	= 5.720 ac



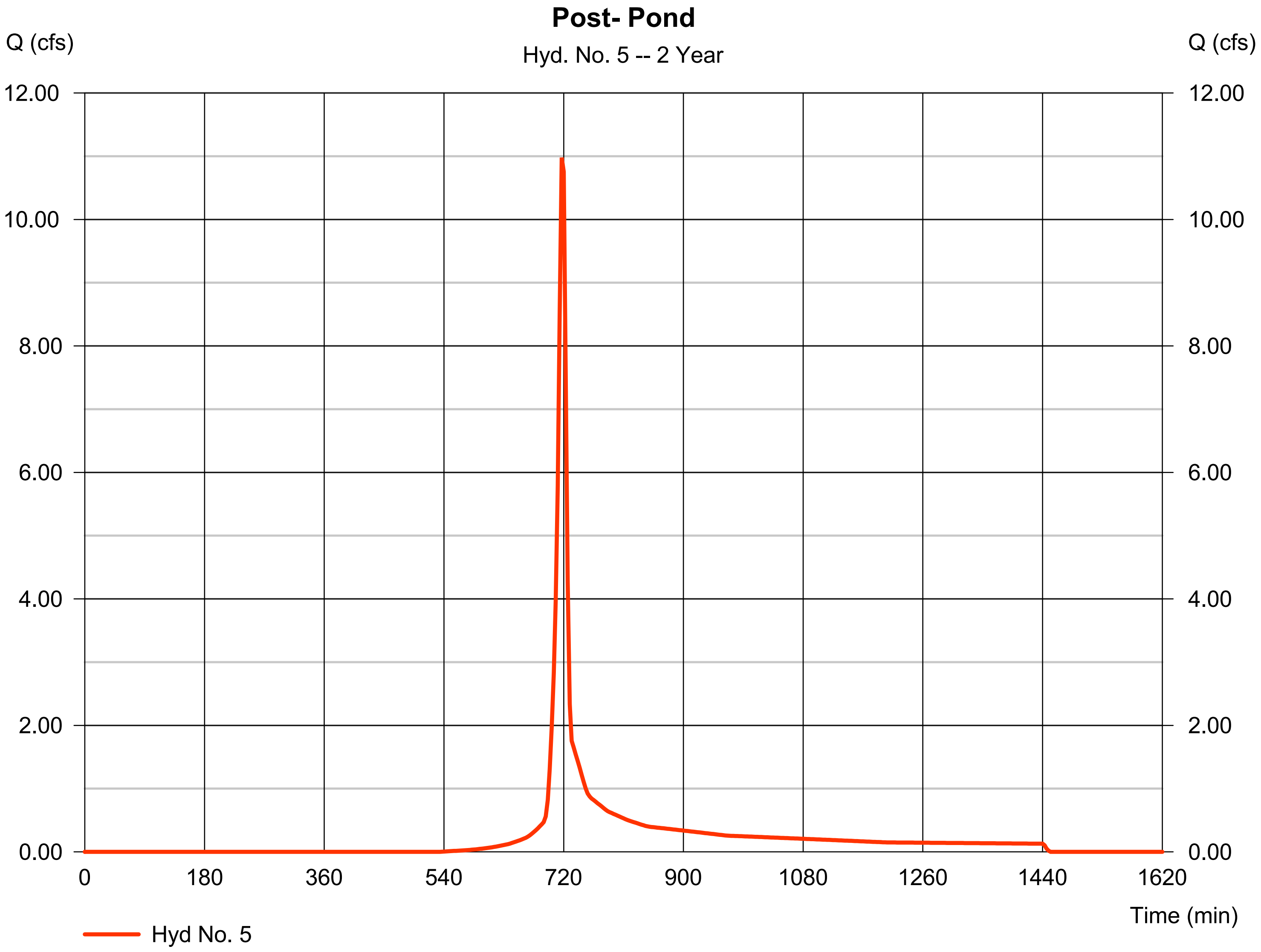
Hydrograph Report

Hyd. No. 5

Post- Pond

Hydrograph type	= SCS Runoff	Peak discharge	= 10.98 cfs
Storm frequency	= 2 yrs	Time to peak	= 717 min
Time interval	= 3 min	Hyd. volume	= 24,828 cuft
Drainage area	= 5.730 ac	Curve number	= 84*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 6.00 min
Total precip.	= 2.70 in	Distribution	= Type II
Storm duration	= 24 hrs	Shape factor	= 484

* Composite (Area/CN) = [(0.260 x 61) + (2.350 x 98) + (2.020 x 74) + (1.100 x 80)] / 5.730



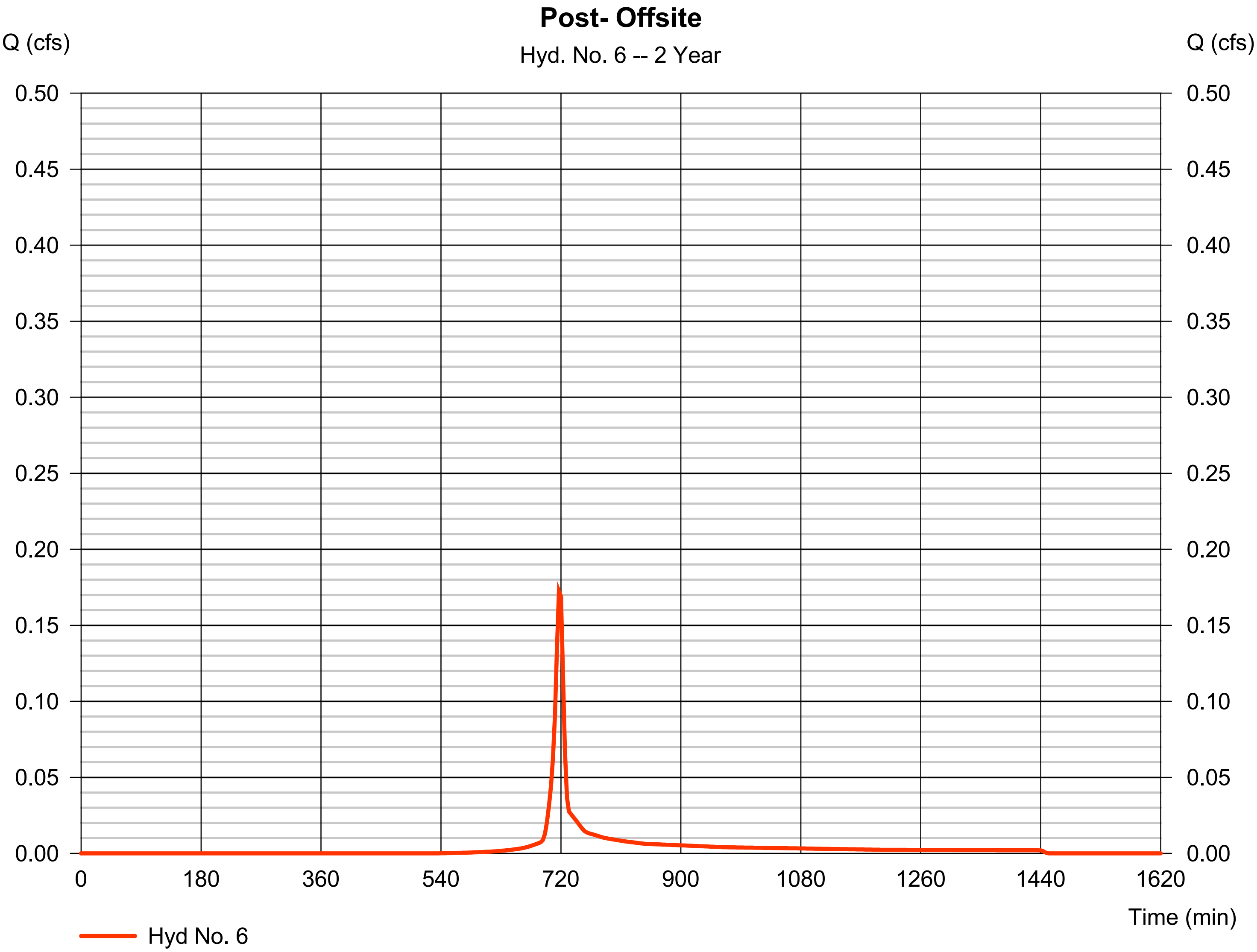
Hydrograph Report

Hyd. No. 6

Post- Offsite

Hydrograph type	= SCS Runoff	Peak discharge	= 0.172 cfs
Storm frequency	= 2 yrs	Time to peak	= 717 min
Time interval	= 3 min	Hyd. volume	= 390 cuft
Drainage area	= 0.090 ac	Curve number	= 84*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 6.00 min
Total precip.	= 2.70 in	Distribution	= Type II
Storm duration	= 24 hrs	Shape factor	= 484

* Composite (Area/CN) = [(0.030 x 61) + (0.050 x 98) + (0.010 x 80)] / 0.090



Hydrograph Report

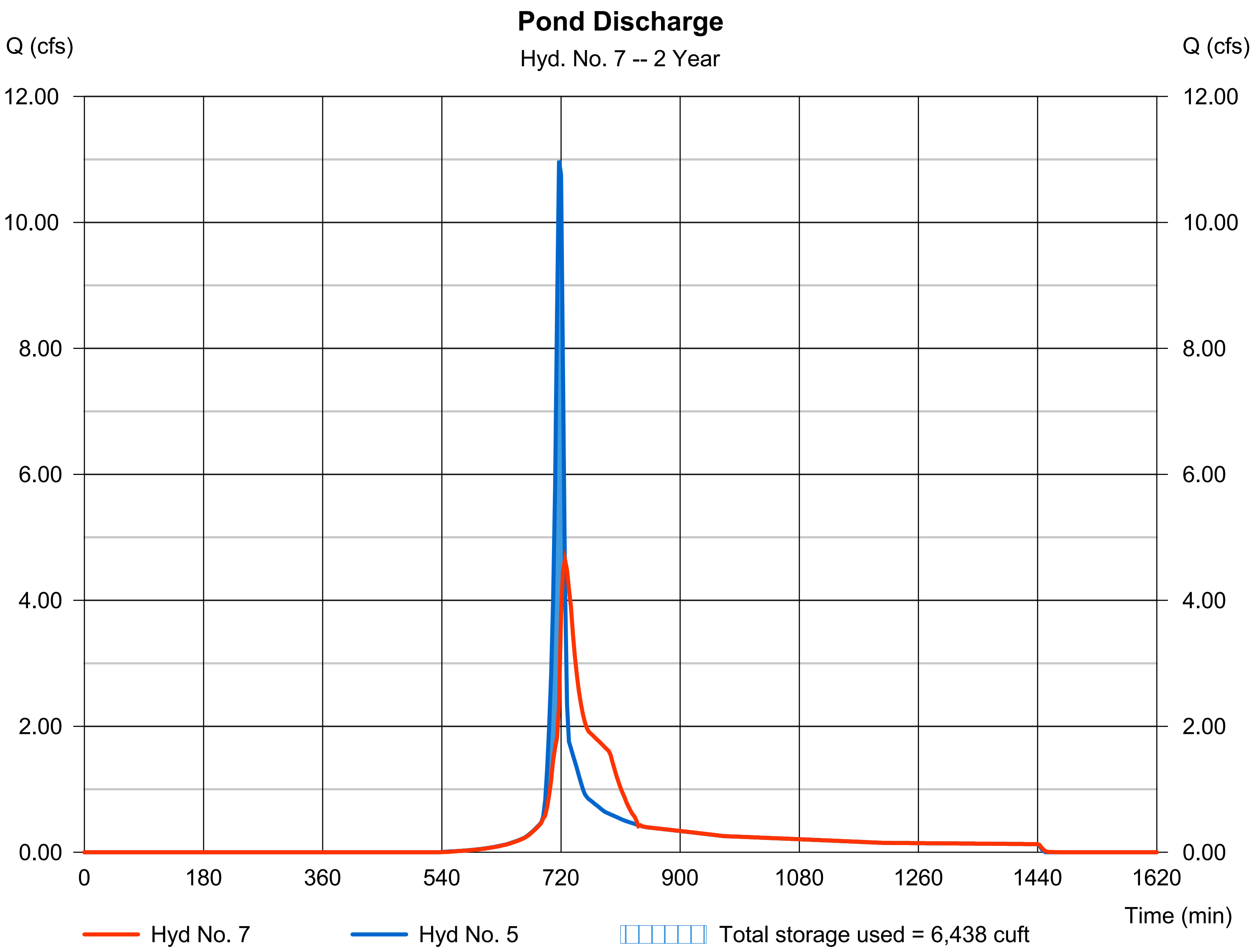
Hyd. No. 7

Pond Discharge

Hydrograph type = Reservoir
Storm frequency = 2 yrs
Time interval = 3 min
Inflow hyd. No. = 5 - Post- Pond
Reservoir name = South Detention

Peak discharge = 4.642 cfs
Time to peak = 726 min
Hyd. volume = 24,828 cuft
Max. Elevation = 138.15 ft
Max. Storage = 6,438 cuft

Storage Indication method used.



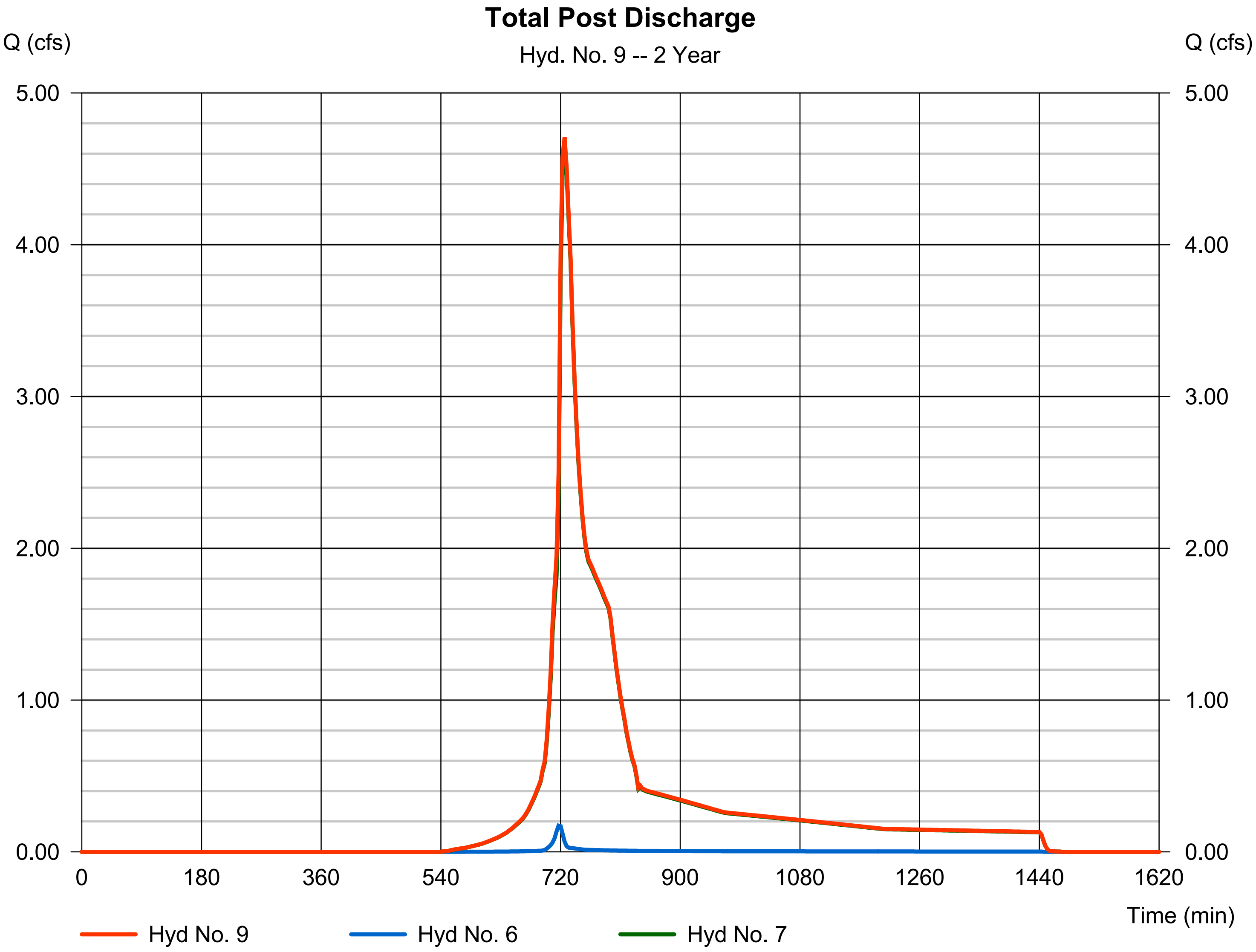
Hydrograph Report

Hyd. No. 9

Total Post Discharge

Hydrograph type = Combine
Storm frequency = 2 yrs
Time interval = 3 min
Inflow hyds. = 6, 7

Peak discharge = 4.709 cfs
Time to peak = 726 min
Hyd. volume = 25,218 cuft
Contrib. drain. area = 0.090 ac



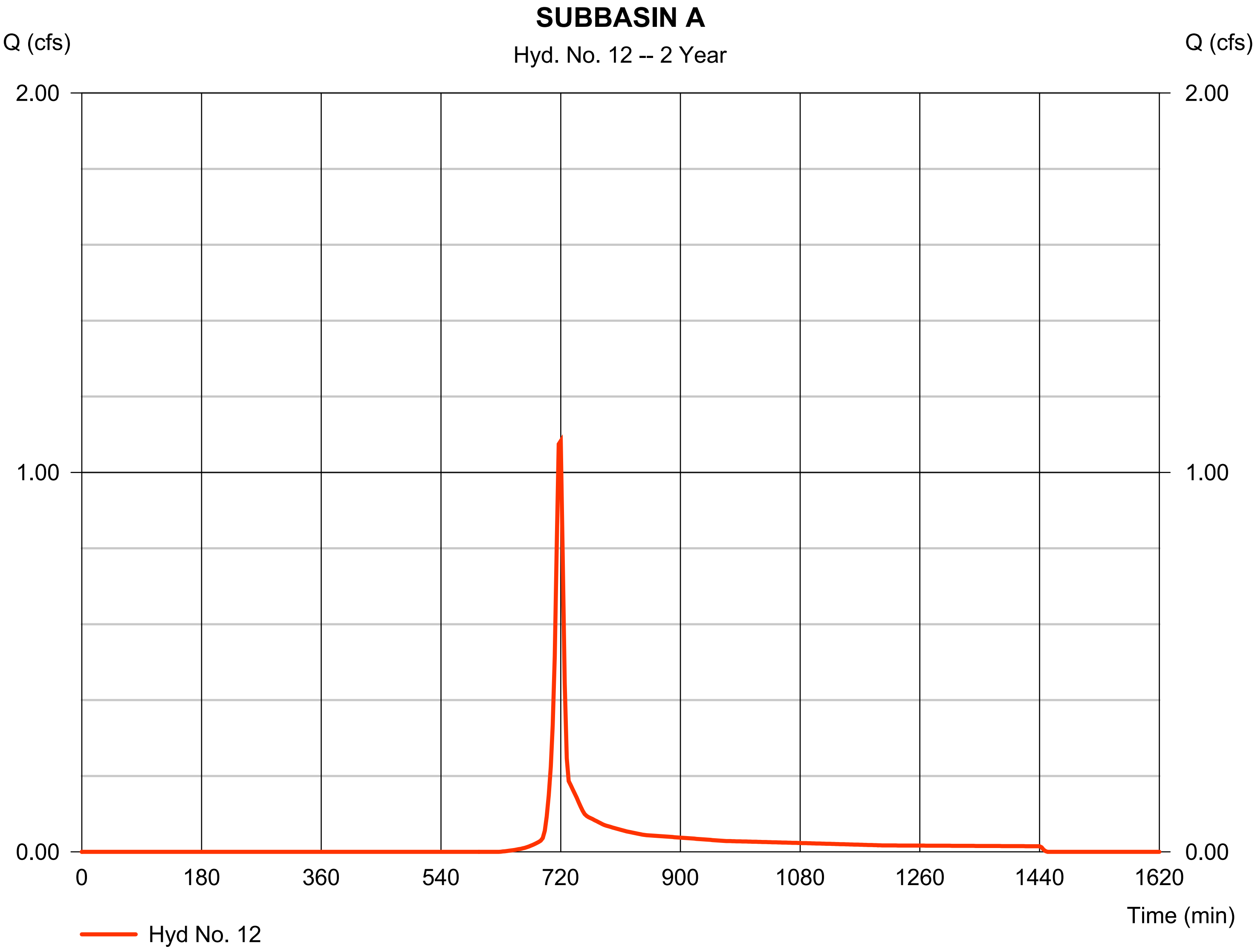
Hydrograph Report

Hyd. No. 12

SUBBASIN A

Hydrograph type	= SCS Runoff	Peak discharge	= 1.083 cfs
Storm frequency	= 2 yrs	Time to peak	= 720 min
Time interval	= 3 min	Hyd. volume	= 2,486 cuft
Drainage area	= 0.750 ac	Curve number	= 79*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 6.00 min
Total precip.	= 2.70 in	Distribution	= Type II
Storm duration	= 24 hrs	Shape factor	= 484

* Composite (Area/CN) = [(0.230 x 98) + (0.240 x 61) + (0.280 x 80)] / 0.750



Hydrograph Report

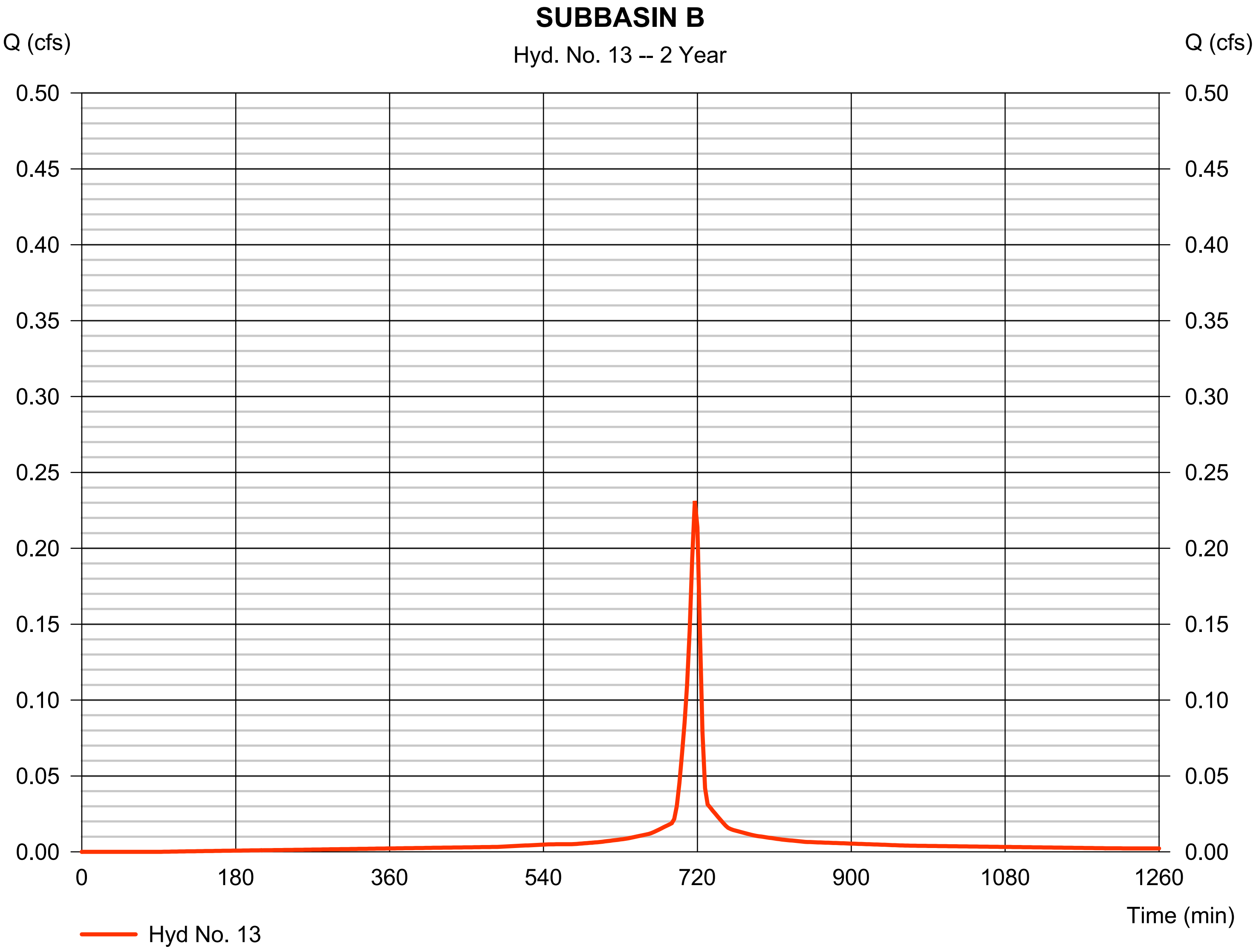
Hyd. No. 13

SUBBASIN B

Hydrograph type = SCS Runoff
Storm frequency = 2 yrs
Time interval = 3 min
Drainage area = 0.070 ac
Basin Slope = 0.0 %
Tc method = User
Total precip. = 2.70 in
Storm duration = 24 hrs

Peak discharge = 0.231 cfs
Time to peak = 717 min
Hyd. volume = 588 cuft
Curve number = 98*
Hydraulic length = 0 ft
Time of conc. (Tc) = 6.00 min
Distribution = Type II
Shape factor = 484

* Composite (Area/CN) = [(0.070 x 98)] / 0.070



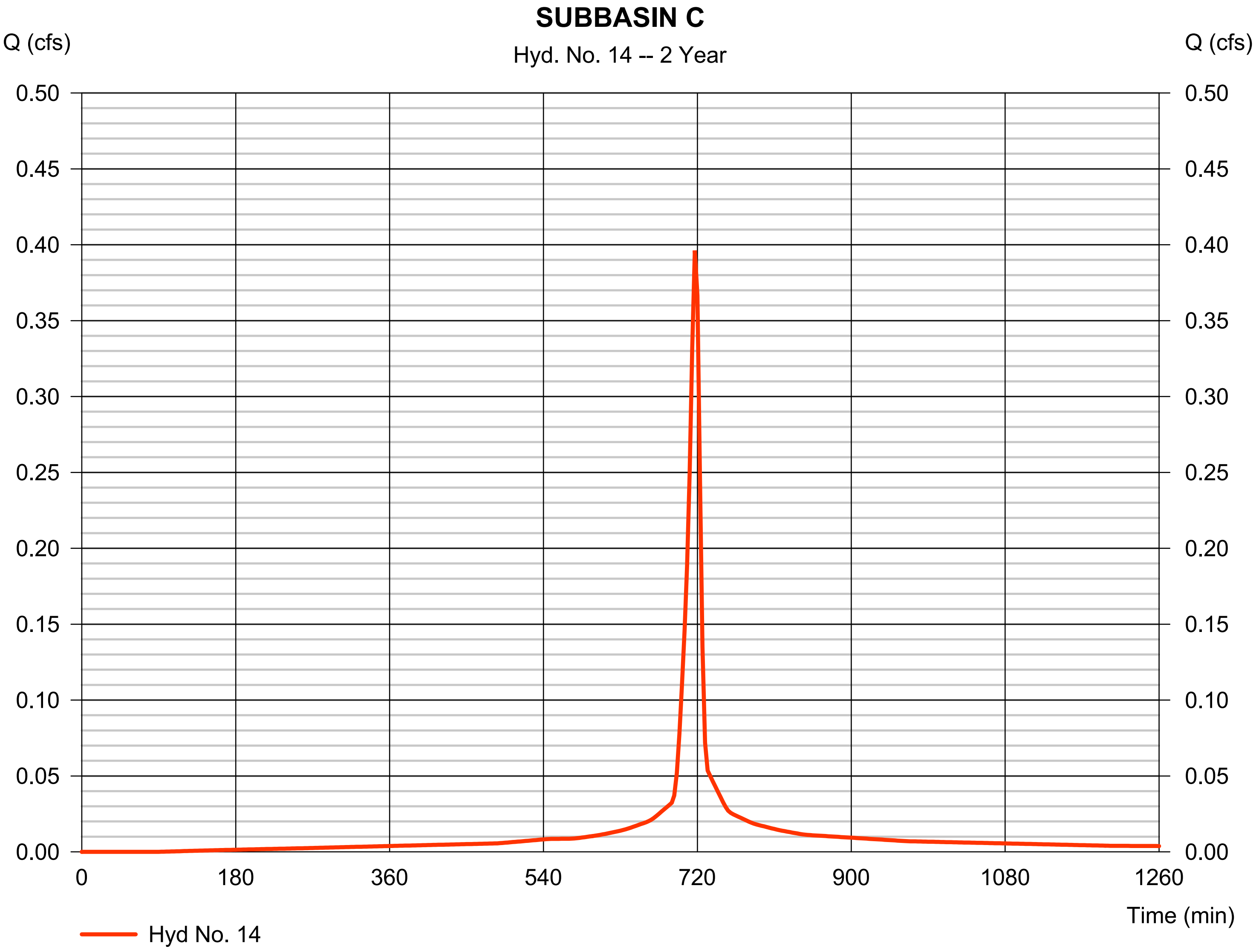
Hydrograph Report

Hyd. No. 14

SUBBASIN C

Hydrograph type	= SCS Runoff	Peak discharge	= 0.396 cfs
Storm frequency	= 2 yrs	Time to peak	= 717 min
Time interval	= 3 min	Hyd. volume	= 1,009 cuft
Drainage area	= 0.120 ac	Curve number	= 98*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 6.00 min
Total precip.	= 2.70 in	Distribution	= Type II
Storm duration	= 24 hrs	Shape factor	= 484

* Composite (Area/CN) = [(0.120 x 98)] / 0.120



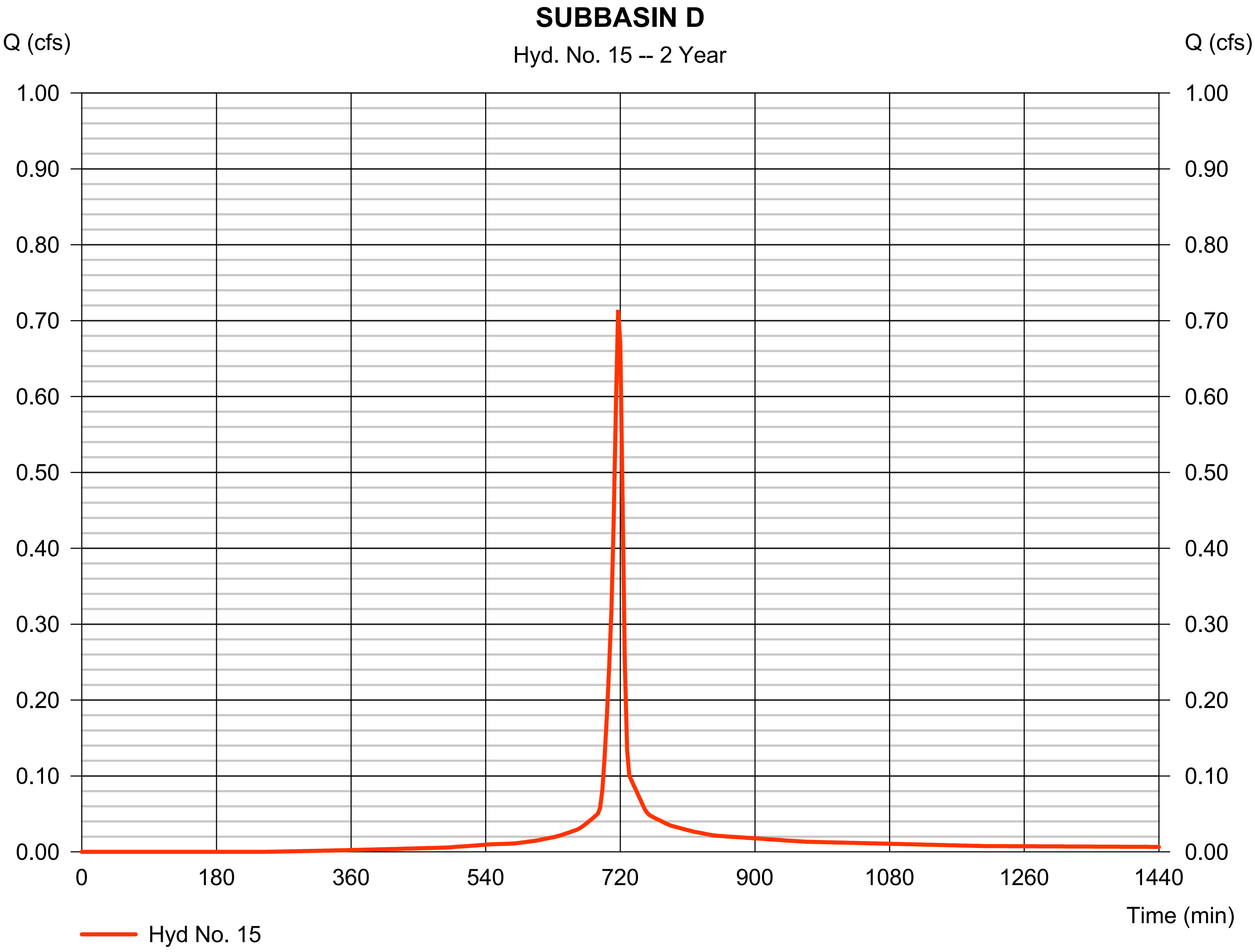
Hydrograph Report

Hyd. No. 15

SUBBASIN D

Hydrograph type	= SCS Runoff	Peak discharge	= 0.714 cfs
Storm frequency	= 2 yrs	Time to peak	= 717 min
Time interval	= 3 min	Hyd. volume	= 1,683 cuft
Drainage area	= 0.240 ac	Curve number	= 94*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 6.00 min
Total precip.	= 2.70 in	Distribution	= Type II
Storm duration	= 24 hrs	Shape factor	= 484

* Composite (Area/CN) = [(0.190 x 98) + (0.050 x 80)] / 0.240



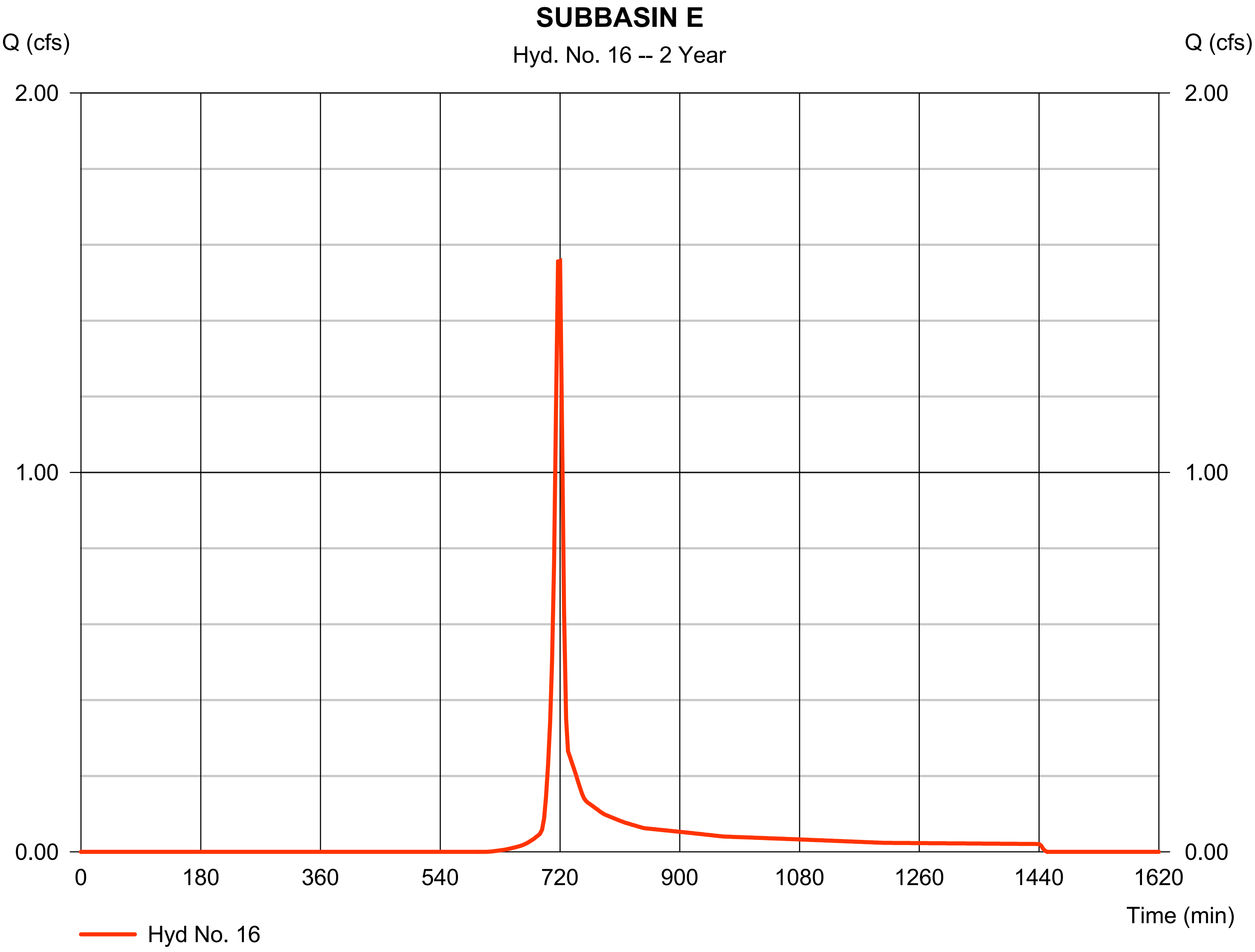
Hydrograph Report

Hyd. No. 16

SUBBASIN E

Hydrograph type	= SCS Runoff	Peak discharge	= 1.558 cfs
Storm frequency	= 2 yrs	Time to peak	= 720 min
Time interval	= 3 min	Hyd. volume	= 3,575 cuft
Drainage area	= 1.020 ac	Curve number	= 80*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 6.00 min
Total precip.	= 2.70 in	Distribution	= Type II
Storm duration	= 24 hrs	Shape factor	= 484

* Composite (Area/CN) = [(0.020 x 98) + (1.000 x 80)] / 1.020



Hydrograph Report

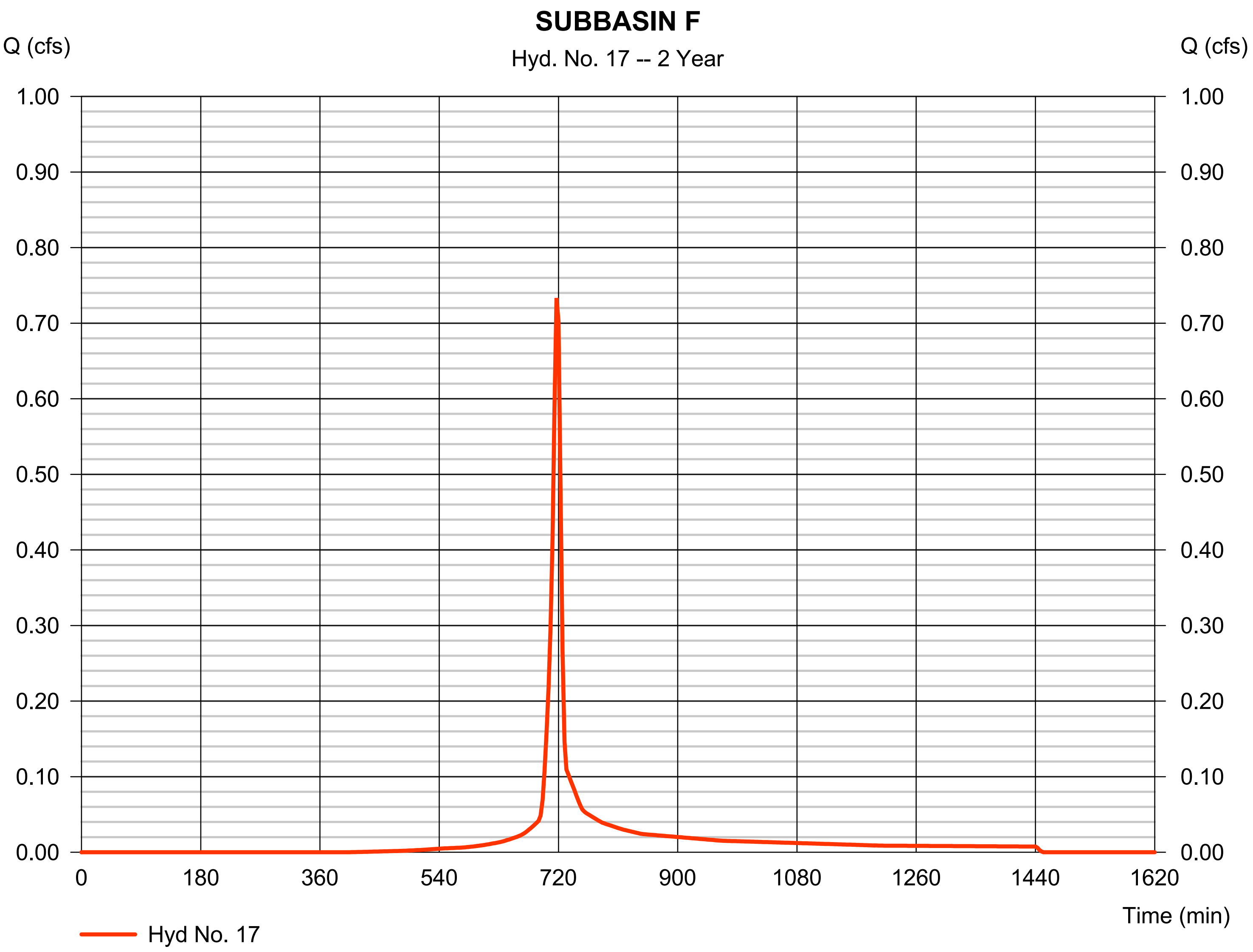
Hyd. No. 17

SUBBASIN F

Hydrograph type = SCS Runoff
Storm frequency = 2 yrs
Time interval = 3 min
Drainage area = 0.300 ac
Basin Slope = 0.0 %
Tc method = User
Total precip. = 2.70 in
Storm duration = 24 hrs

Peak discharge = 0.733 cfs
Time to peak = 717 min
Hyd. volume = 1,665 cuft
Curve number = 89*
Hydraulic length = 0 ft
Time of conc. (Tc) = 6.00 min
Distribution = Type II
Shape factor = 484

* Composite (Area/CN) = [(0.190 x 98) + (0.110 x 74)] / 0.300



Hydrograph Report

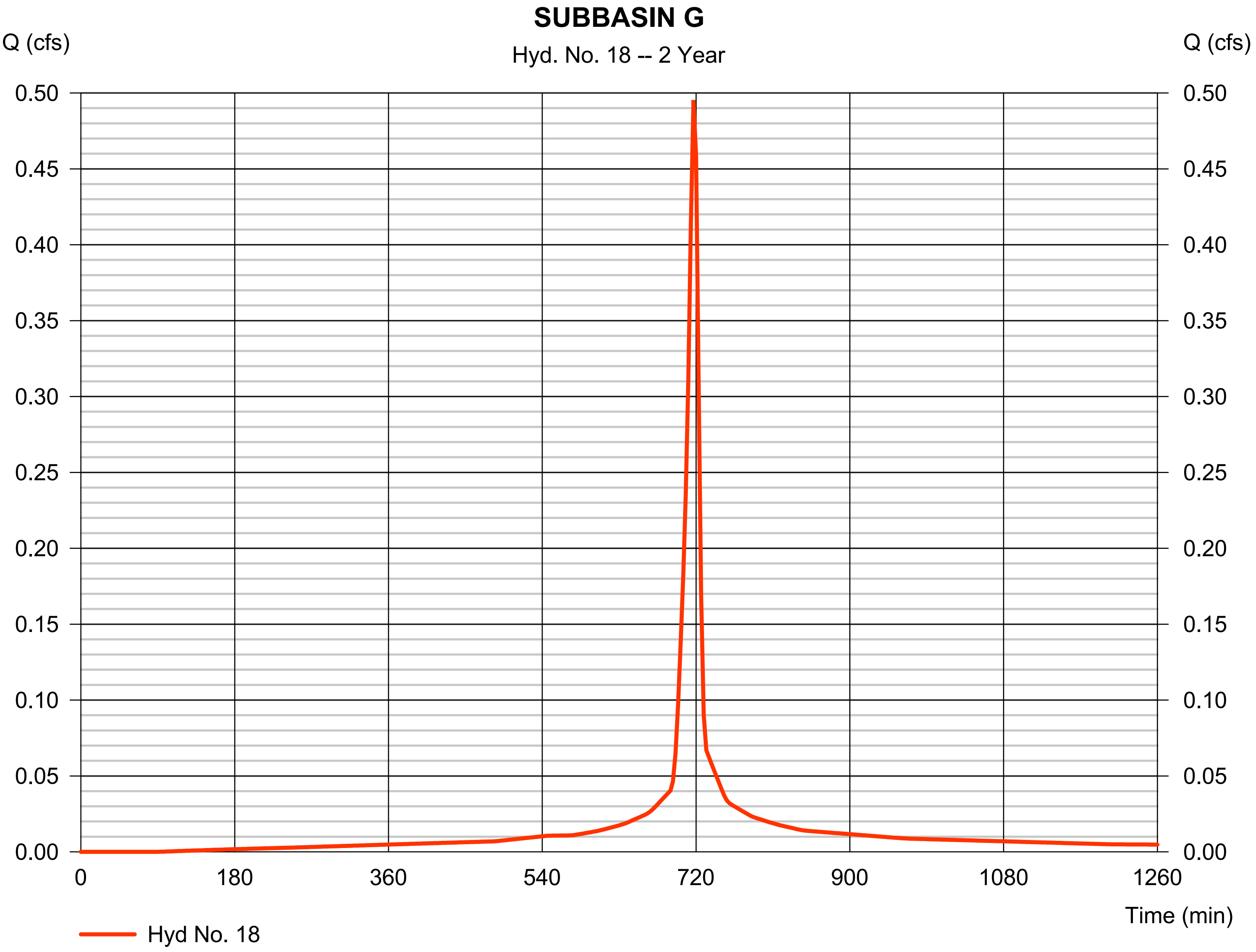
Hyd. No. 18

SUBBASIN G

Hydrograph type = SCS Runoff
Storm frequency = 2 yrs
Time interval = 3 min
Drainage area = 0.150 ac
Basin Slope = 0.0 %
Tc method = User
Total precip. = 2.70 in
Storm duration = 24 hrs

Peak discharge = 0.495 cfs
Time to peak = 717 min
Hyd. volume = 1,261 cuft
Curve number = 98*
Hydraulic length = 0 ft
Time of conc. (Tc) = 6.00 min
Distribution = Type II
Shape factor = 484

* Composite (Area/CN) = [(0.150 x 98)] / 0.150



Hydrograph Report

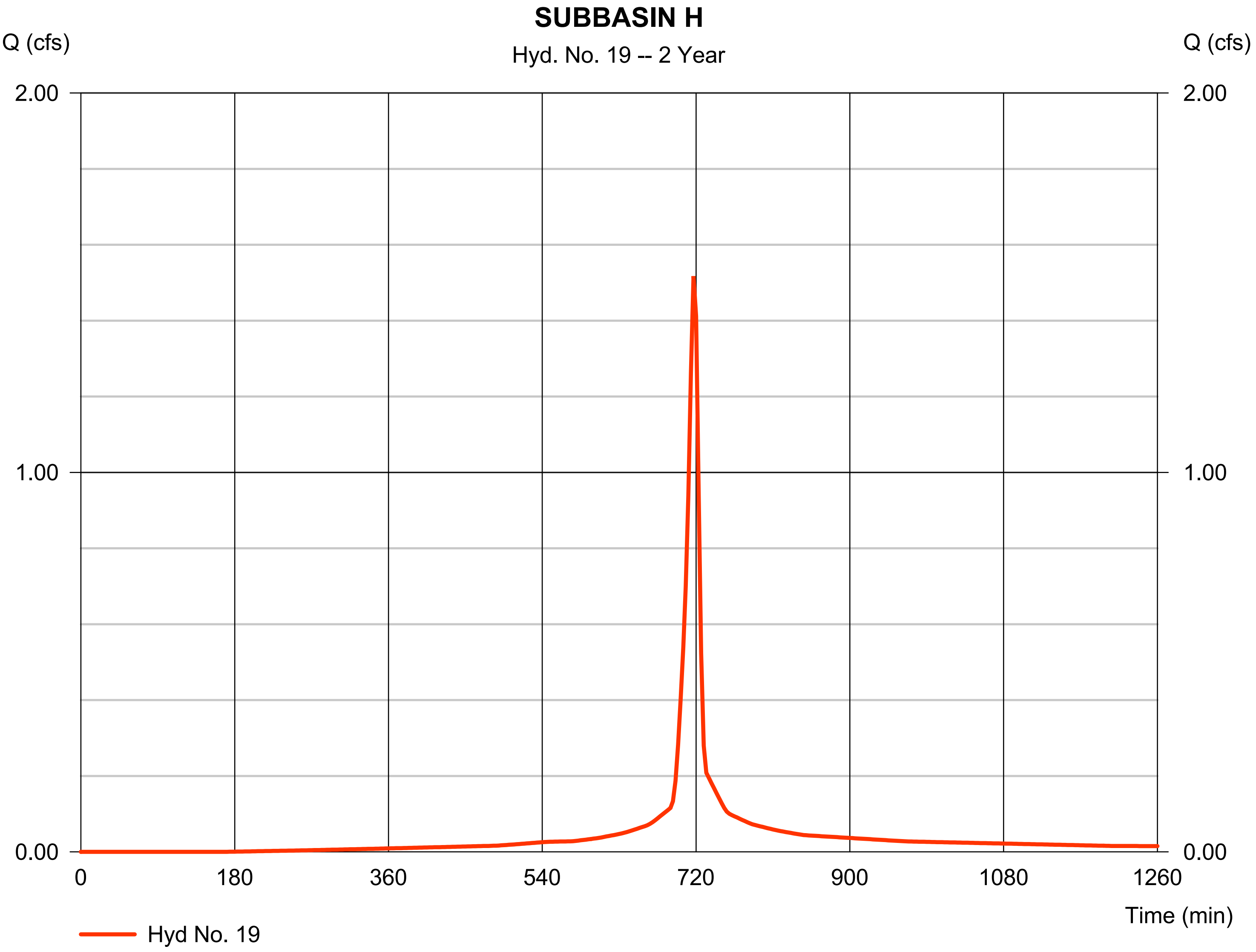
Hyd. No. 19

SUBBASIN H

Hydrograph type = SCS Runoff
Storm frequency = 2 yrs
Time interval = 3 min
Drainage area = 0.480 ac
Basin Slope = 0.0 %
Tc method = User
Total precip. = 2.70 in
Storm duration = 24 hrs

Peak discharge = 1.517 cfs
Time to peak = 717 min
Hyd. volume = 3,687 cuft
Curve number = 96*
Hydraulic length = 0 ft
Time of conc. (Tc) = 6.00 min
Distribution = Type II
Shape factor = 484

* Composite (Area/CN) = [(0.430 x 98) + (0.050 x 74)] / 0.480



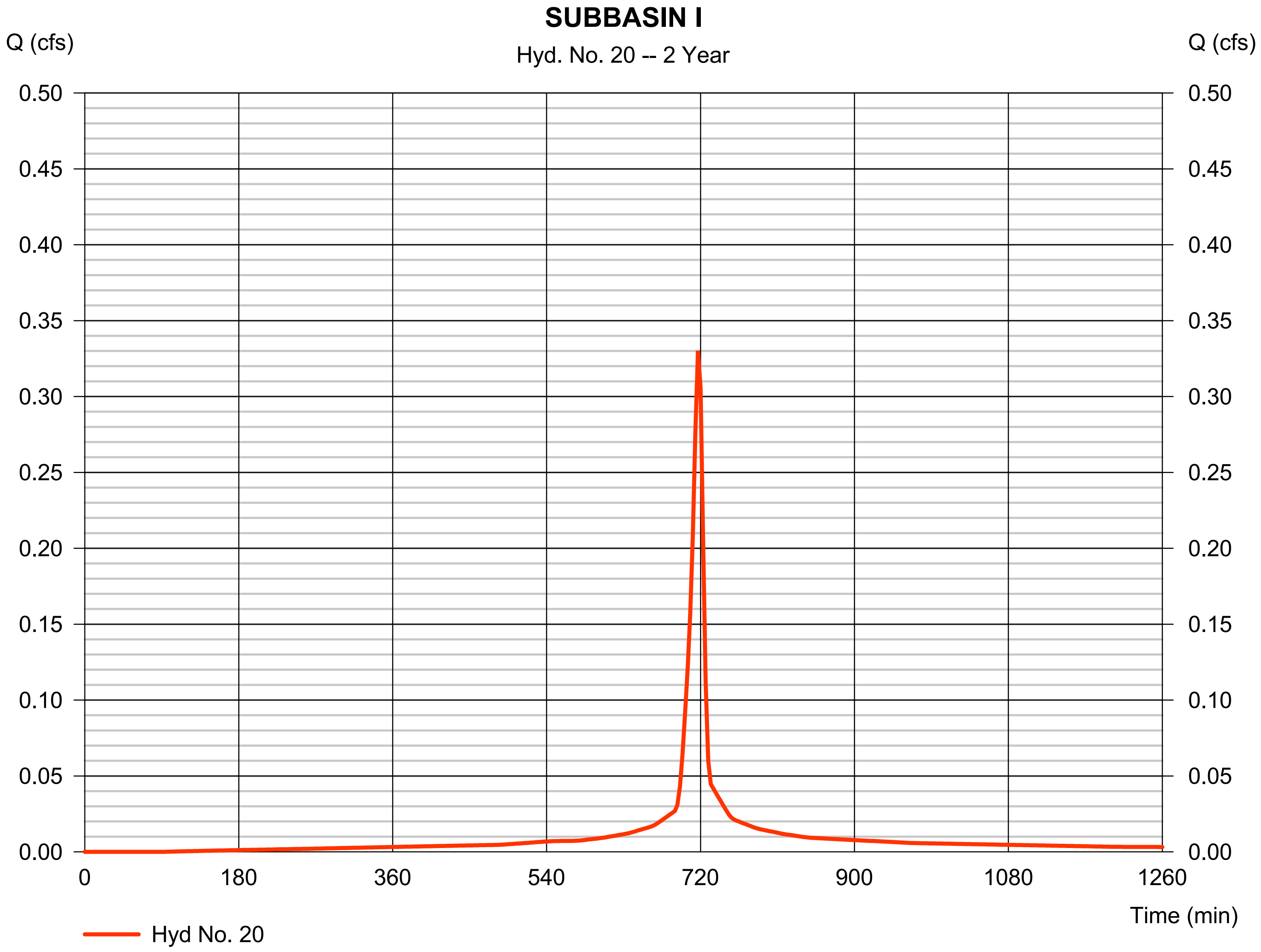
Hydrograph Report

Hyd. No. 20

SUBBASIN I

Hydrograph type	= SCS Runoff	Peak discharge	= 0.330 cfs
Storm frequency	= 2 yrs	Time to peak	= 717 min
Time interval	= 3 min	Hyd. volume	= 840 cuft
Drainage area	= 0.100 ac	Curve number	= 98*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 6.00 min
Total precip.	= 2.70 in	Distribution	= Type II
Storm duration	= 24 hrs	Shape factor	= 484

* Composite (Area/CN) = [(0.100 x 98)] / 0.100



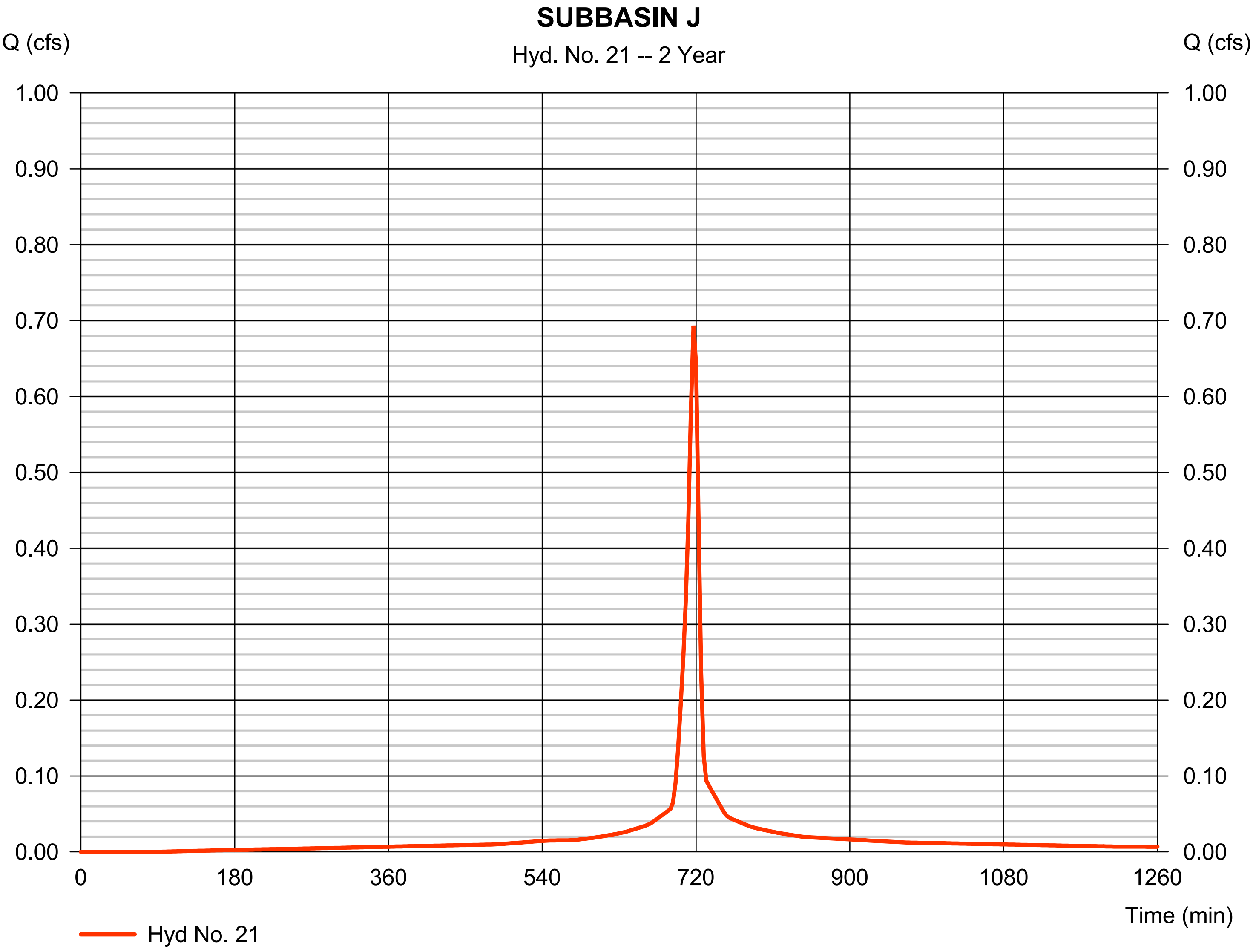
Hydrograph Report

Hyd. No. 21

SUBBASIN J

Hydrograph type	= SCS Runoff	Peak discharge	= 0.693 cfs
Storm frequency	= 2 yrs	Time to peak	= 717 min
Time interval	= 3 min	Hyd. volume	= 1,765 cuft
Drainage area	= 0.210 ac	Curve number	= 98*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 6.00 min
Total precip.	= 2.70 in	Distribution	= Type II
Storm duration	= 24 hrs	Shape factor	= 484

* Composite (Area/CN) = [(0.210 x 98)] / 0.210



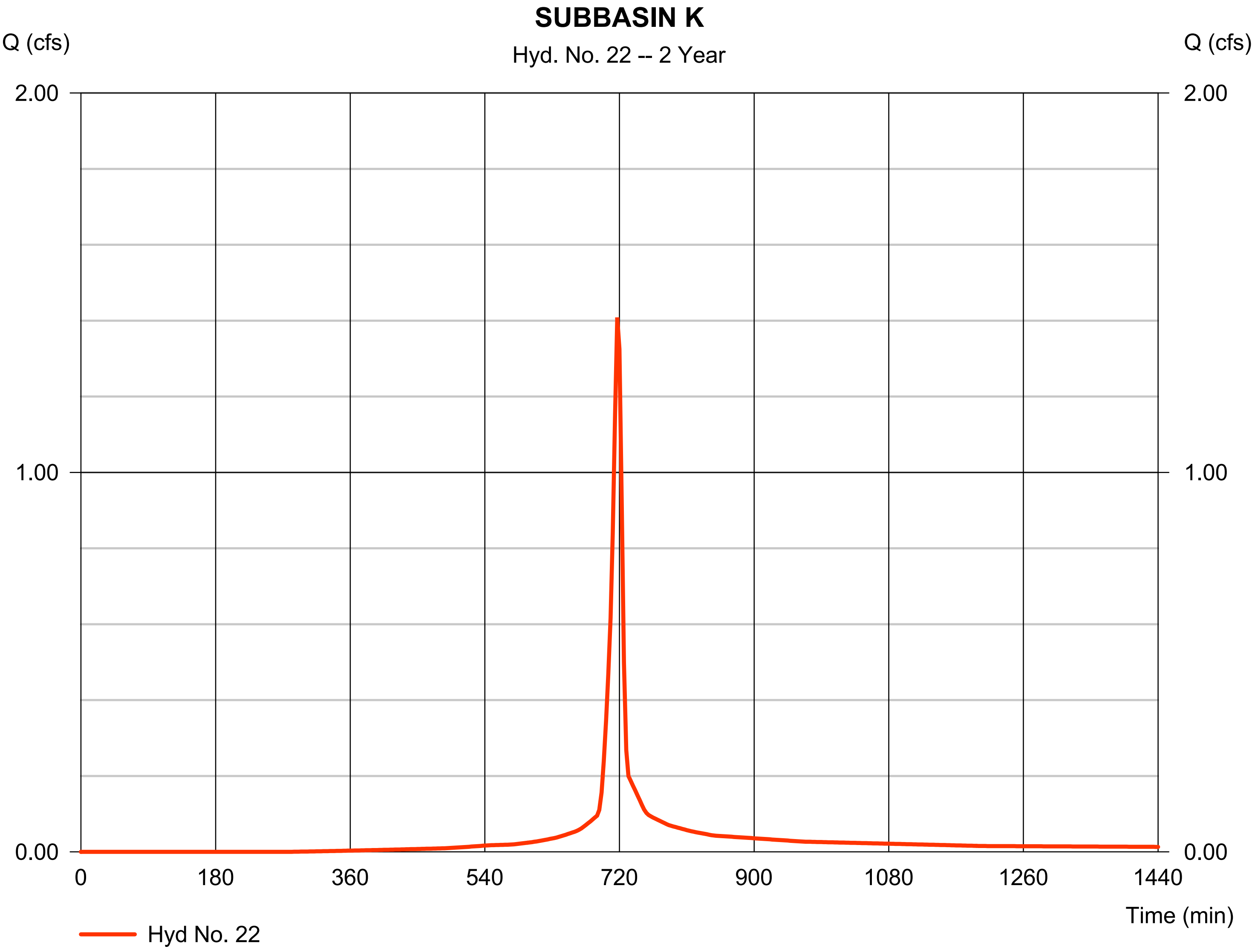
Hydrograph Report

Hyd. No. 22

SUBBASIN K

Hydrograph type	= SCS Runoff	Peak discharge	= 1.408 cfs
Storm frequency	= 2 yrs	Time to peak	= 717 min
Time interval	= 3 min	Hyd. volume	= 3,282 cuft
Drainage area	= 0.490 ac	Curve number	= 93*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 6.00 min
Total precip.	= 2.70 in	Distribution	= Type II
Storm duration	= 24 hrs	Shape factor	= 484

* Composite (Area/CN) = [(0.380 x 98) + (0.110 x 74)] / 0.490



Hydrograph Summary Report

Hydraflow Hydrographs Extension for AutoCAD® Civil 3D® 2016 by Autodesk, Inc. v11

Hyd. No.	Hydrograph type (origin)	Peak flow (cfs)	Time interval (min)	Time to Peak (min)	Hyd. volume (cuft)	Inflow hyd(s)	Maximum elevation (ft)	Total strge used (cuft)	Hydrograph Description
1	SCS Runoff	9.980	3	726	35,302	------	------	------	Pre-Basin A
2	SCS Runoff	2.277	3	717	5,164	------	------	------	Pre-Offsite
3	Combine	10.82	3	726	40,467	1, 2	------	------	Total Predevelopment
5	SCS Runoff	20.39	3	717	46,236	------	------	------	Post- Pond
6	SCS Runoff	0.320	3	717	726	------	------	------	Post- Offsite
7	Reservoir	7.874	3	726	46,236	5	138.97	12,912	Pond Discharge
9	Combine	7.993	3	726	46,962	6, 7,	------	------	Total Post Discharge
12	SCS Runoff	2.218	3	717	5,011	------	------	------	SUBBASIN A
13	SCS Runoff	0.346	3	717	897	------	------	------	SUBBASIN B
14	SCS Runoff	0.592	3	717	1,538	------	------	------	SUBBASIN C
15	SCS Runoff	1.120	3	717	2,715	------	------	------	SUBBASIN D
16	SCS Runoff	3.138	3	717	7,087	------	------	------	SUBBASIN E
17	SCS Runoff	1.245	3	717	2,882	------	------	------	SUBBASIN F
18	SCS Runoff	0.741	3	717	1,922	------	------	------	SUBBASIN G
19	SCS Runoff	2.316	3	717	5,783	------	------	------	SUBBASIN H
20	SCS Runoff	0.494	3	717	1,281	------	------	------	SUBBASIN I
21	SCS Runoff	1.037	3	717	2,691	------	------	------	SUBBASIN J
22	SCS Runoff	2.241	3	717	5,369	------	------	------	SUBBASIN K

1818660STORM.gpw	Return Period: 10 Year	Monday, 07 / 9 / 2018

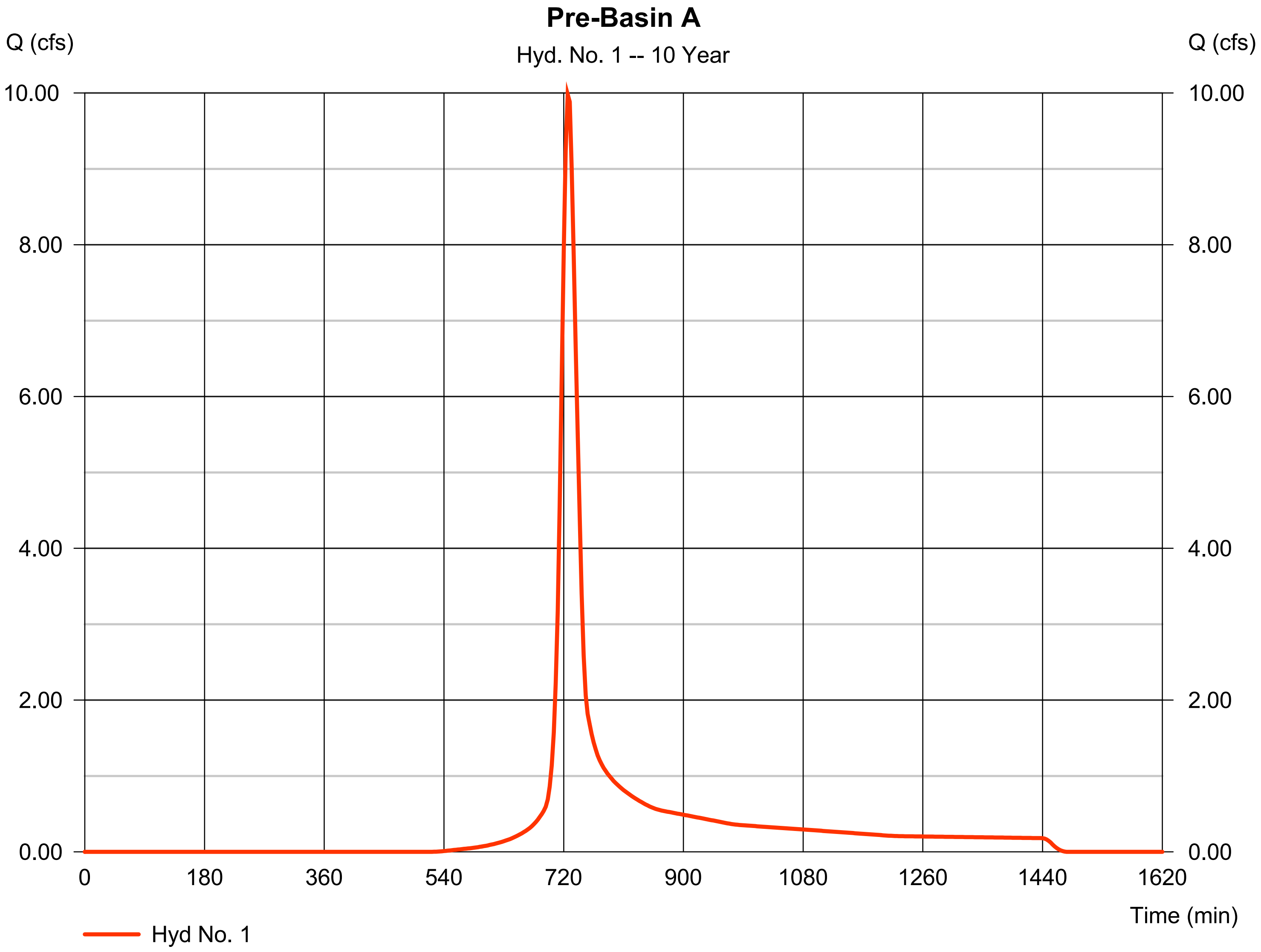
Hydrograph Report

Hyd. No. 1

Pre-Basin A

Hydrograph type	= SCS Runoff	Peak discharge	= 9.980 cfs
Storm frequency	= 10 yrs	Time to peak	= 726 min
Time interval	= 3 min	Hyd. volume	= 35,302 cuft
Drainage area	= 5.080 ac	Curve number	= 79*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= TR55	Time of conc. (Tc)	= 21.40 min
Total precip.	= 4.00 in	Distribution	= Type II
Storm duration	= 24 hrs	Shape factor	= 484

* Composite (Area/CN) = [(0.260 x 61) + (1.160 x 98) + (1.170 x 74) + (1.230 x 70) + (0.670 x 80) + (0.590 x 77)] / 5.080



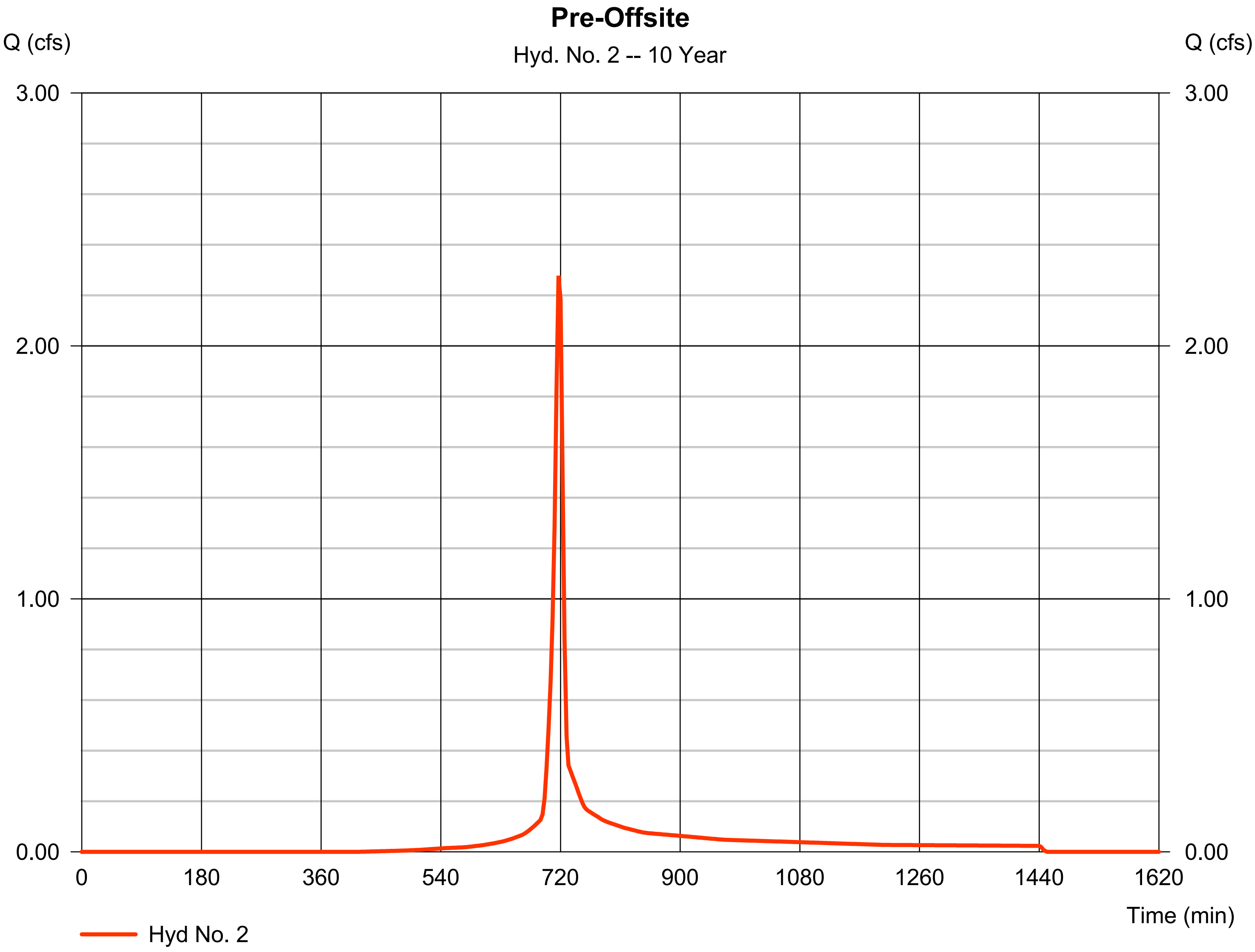
Hydrograph Report

Hyd. No. 2

Pre-Offsite

Hydrograph type	= SCS Runoff	Peak discharge	= 2.277 cfs
Storm frequency	= 10 yrs	Time to peak	= 717 min
Time interval	= 3 min	Hyd. volume	= 5,164 cuft
Drainage area	= 0.640 ac	Curve number	= 84*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 6.00 min
Total precip.	= 4.00 in	Distribution	= Type II
Storm duration	= 24 hrs	Shape factor	= 484

* Composite (Area/CN) = [(0.370 x 74) + (0.270 x 98)] / 0.640

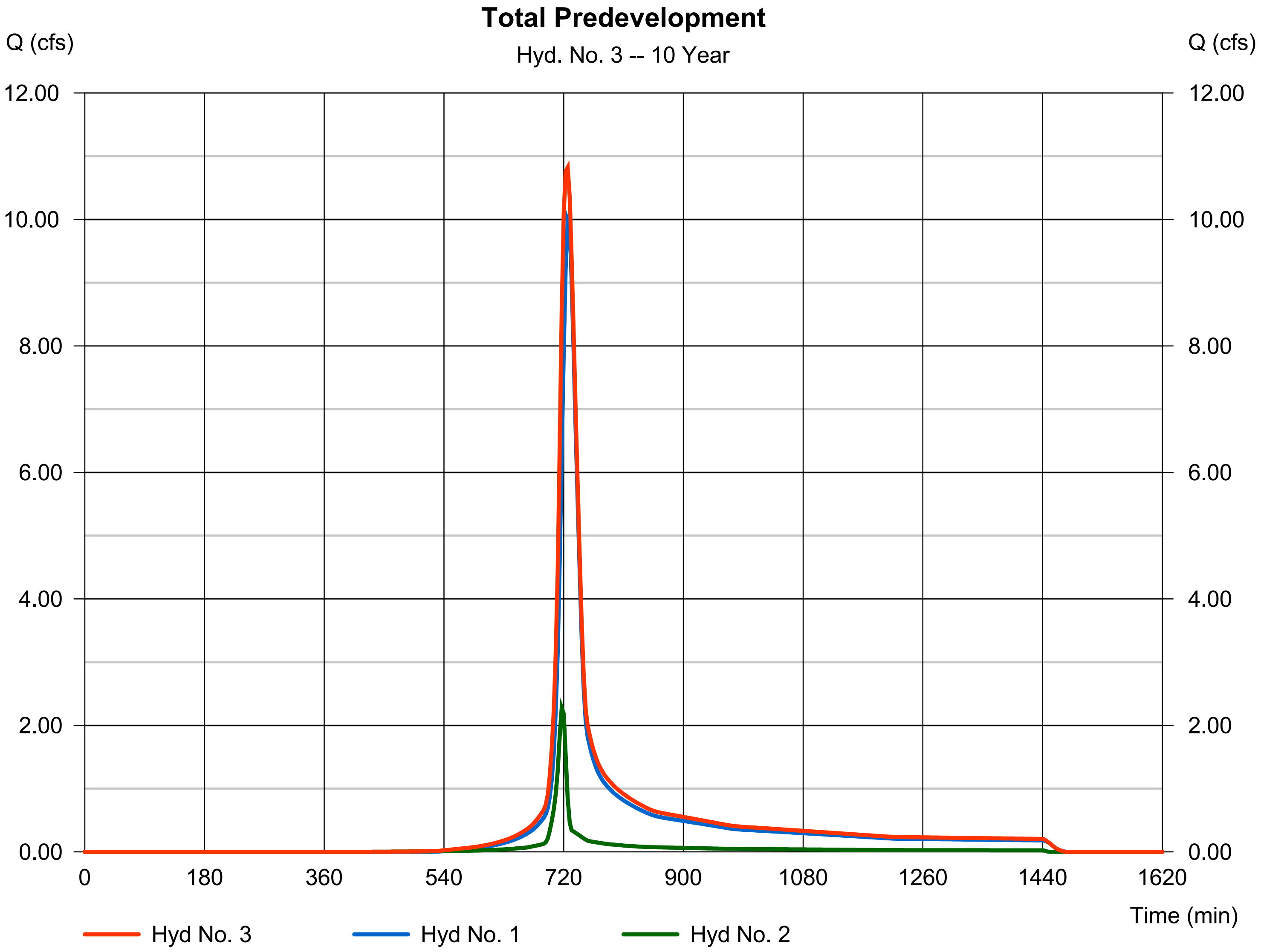


Hydrograph Report

Hyd. No. 3

Total Predevelopment

Hydrograph type	= Combine	Peak discharge	= 10.82 cfs
Storm frequency	= 10 yrs	Time to peak	= 726 min
Time interval	= 3 min	Hyd. volume	= 40,467 cuft
Inflow hyds.	= 1, 2	Contrib. drain. area	= 5.720 ac



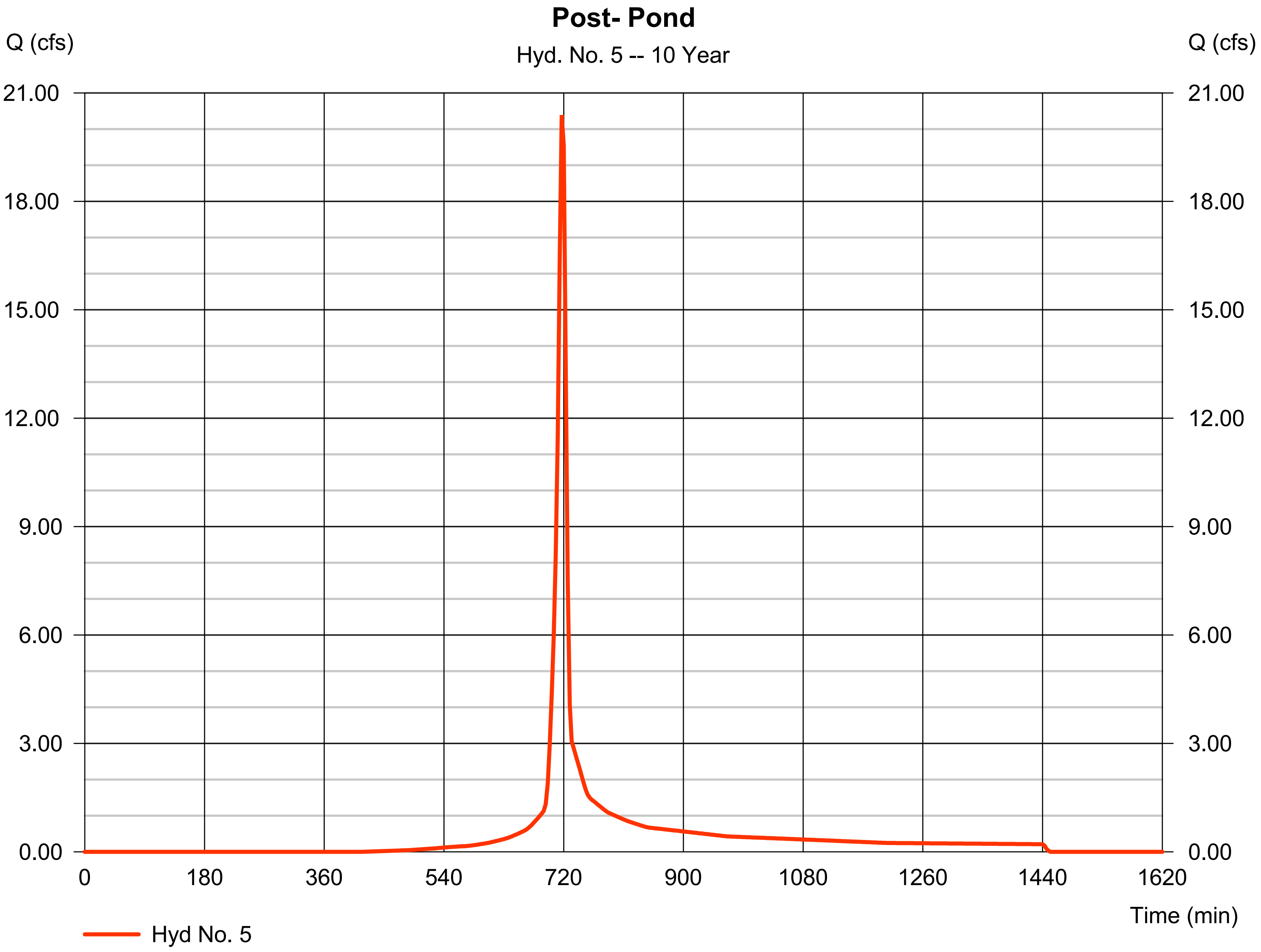
Hydrograph Report

Hyd. No. 5

Post- Pond

Hydrograph type	= SCS Runoff	Peak discharge	= 20.39 cfs
Storm frequency	= 10 yrs	Time to peak	= 717 min
Time interval	= 3 min	Hyd. volume	= 46,236 cuft
Drainage area	= 5.730 ac	Curve number	= 84*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 6.00 min
Total precip.	= 4.00 in	Distribution	= Type II
Storm duration	= 24 hrs	Shape factor	= 484

* Composite (Area/CN) = [(0.260 x 61) + (2.350 x 98) + (2.020 x 74) + (1.100 x 80)] / 5.730



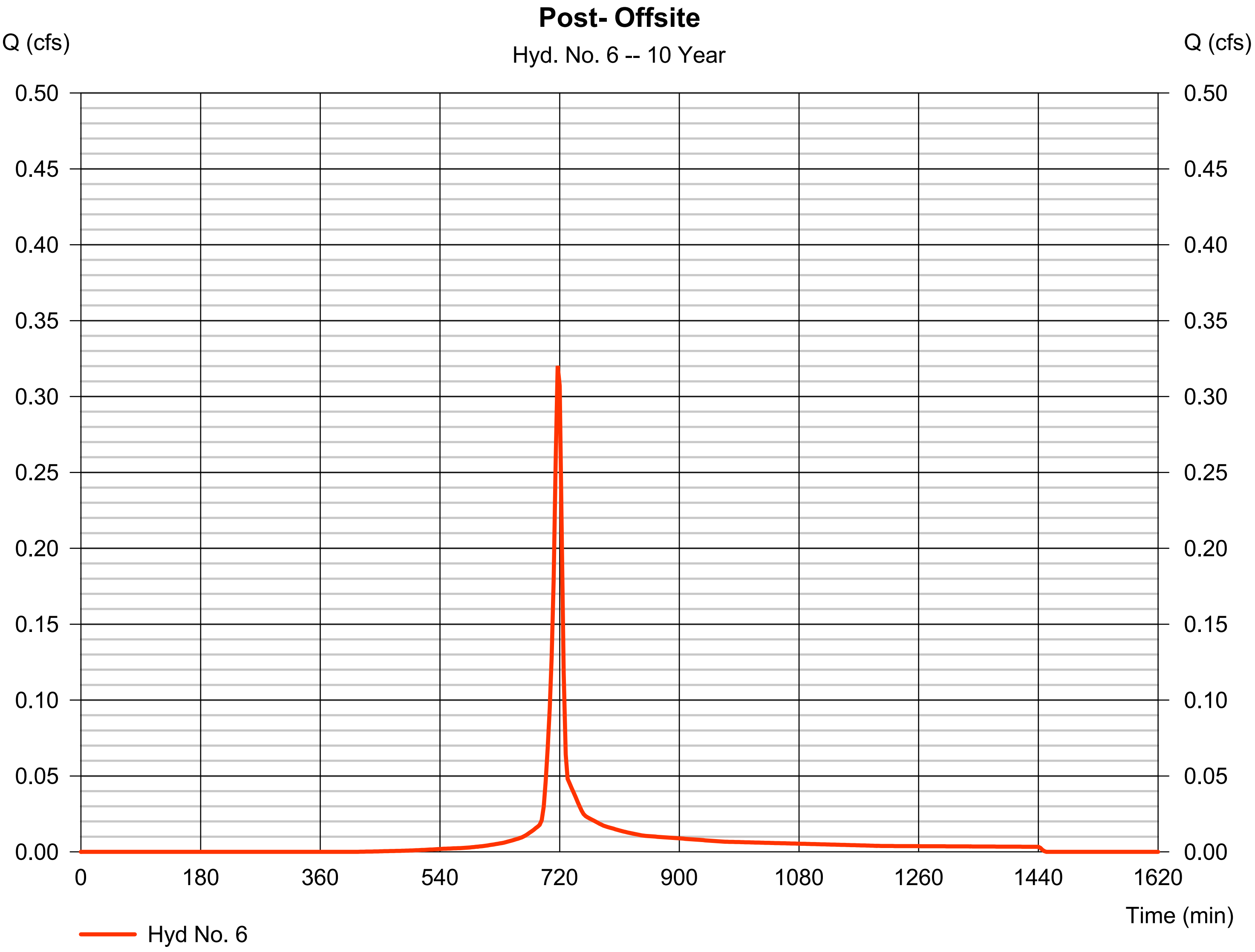
Hydrograph Report

Hyd. No. 6

Post- Offsite

Hydrograph type	= SCS Runoff	Peak discharge	= 0.320 cfs
Storm frequency	= 10 yrs	Time to peak	= 717 min
Time interval	= 3 min	Hyd. volume	= 726 cuft
Drainage area	= 0.090 ac	Curve number	= 84*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 6.00 min
Total precip.	= 4.00 in	Distribution	= Type II
Storm duration	= 24 hrs	Shape factor	= 484

* Composite (Area/CN) = [(0.030 x 61) + (0.050 x 98) + (0.010 x 80)] / 0.090



Hydrograph Report

Hydraflow Hydrographs Extension for AutoCAD® Civil 3D® 2016 by Autodesk, Inc. v11 Monday, 07 / 9 / 2018

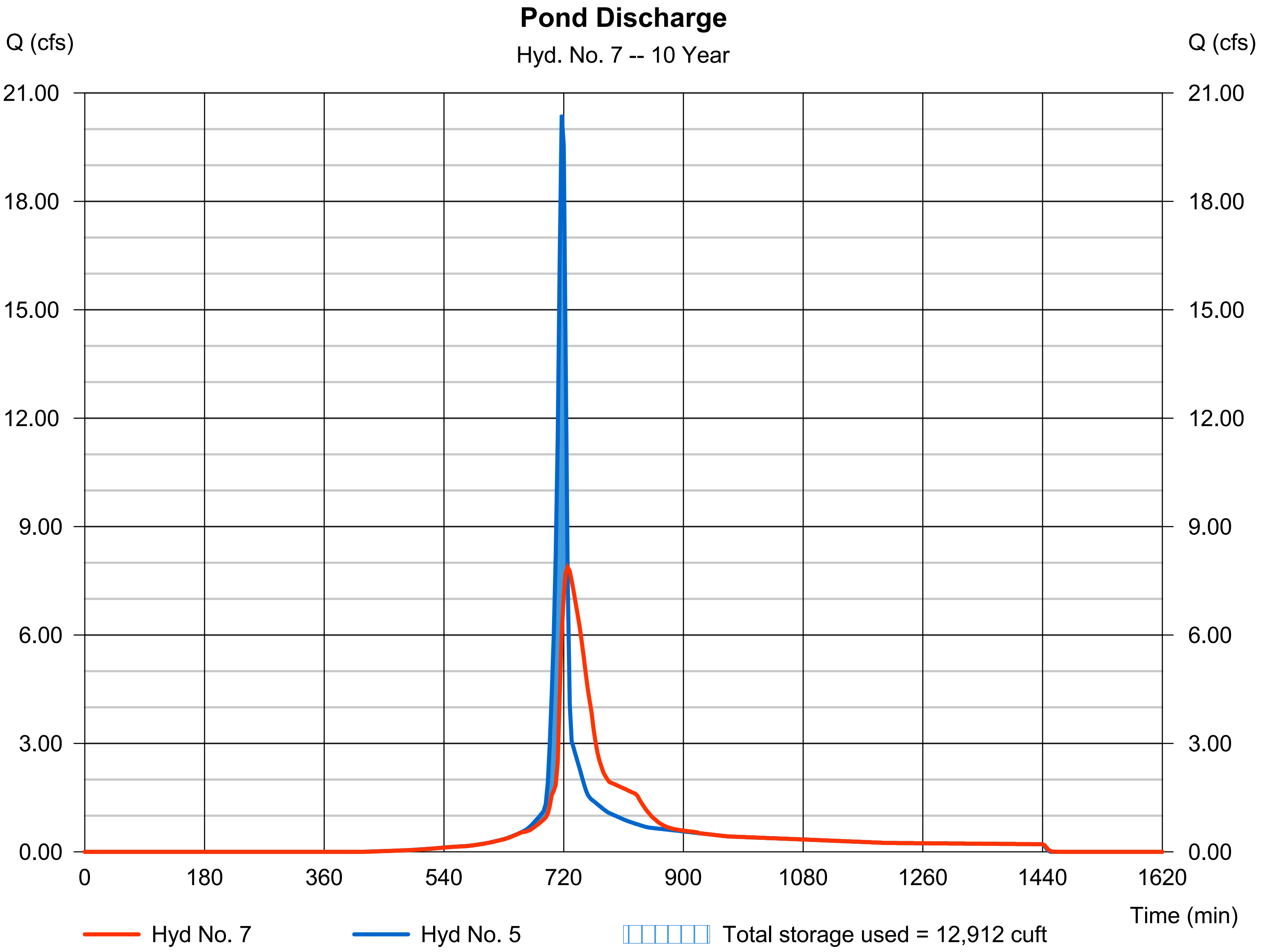
Hyd. No. 7

Pond Discharge

Hydrograph type = Reservoir
Storm frequency = 10 yrs
Time interval = 3 min
Inflow hyd. No. = 5 - Post- Pond
Reservoir name = South Detention

Peak discharge = 7.874 cfs
Time to peak = 726 min
Hyd. volume = 46,236 cuft
Max. Elevation = 138.97 ft
Max. Storage = 12,912 cuft

Storage Indication method used.

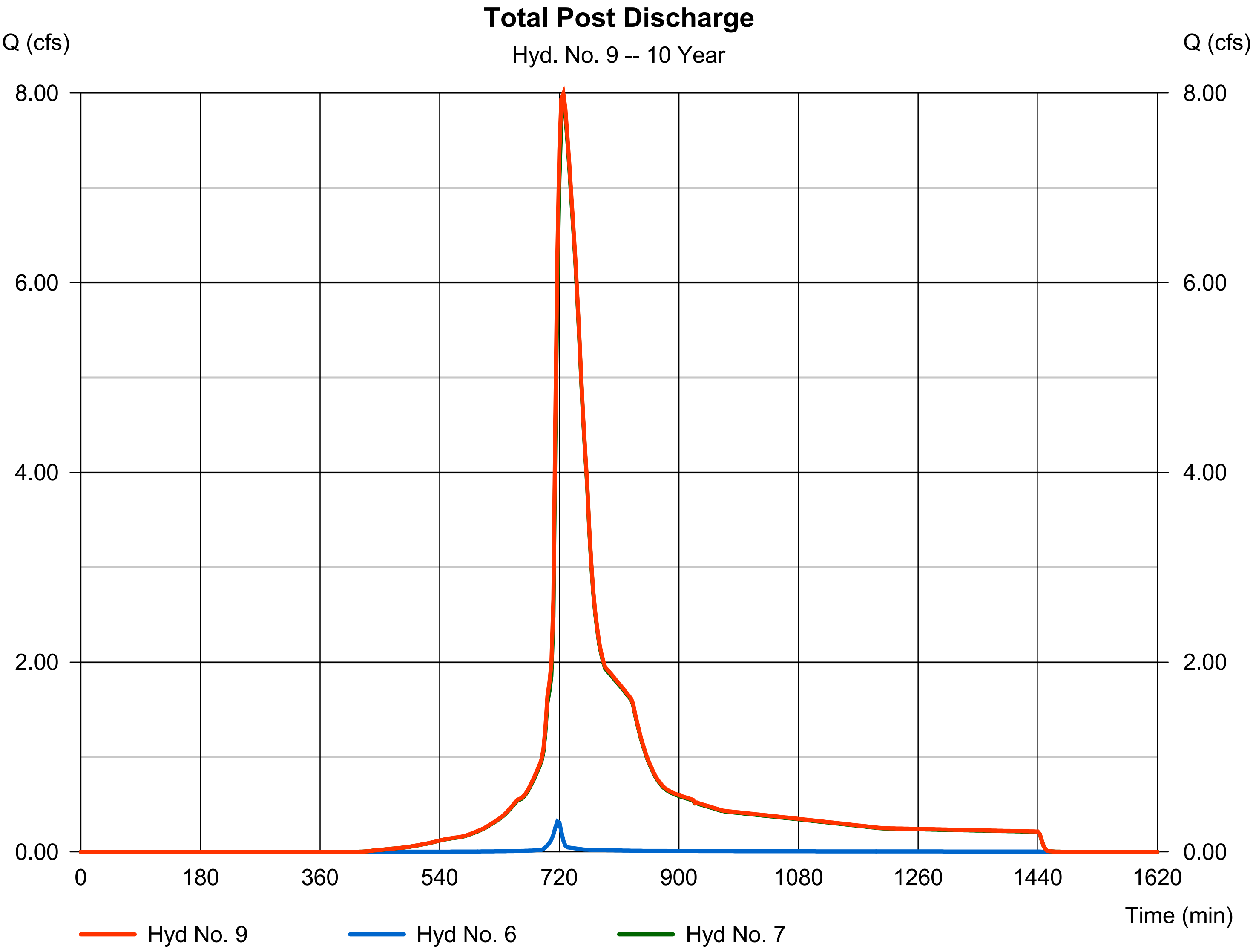


Hydrograph Report

Hyd. No. 9

Total Post Discharge

Hydrograph type	= Combine	Peak discharge	= 7.993 cfs
Storm frequency	= 10 yrs	Time to peak	= 726 min
Time interval	= 3 min	Hyd. volume	= 46,962 cuft
Inflow hyds.	= 6, 7	Contrib. drain. area	= 0.090 ac



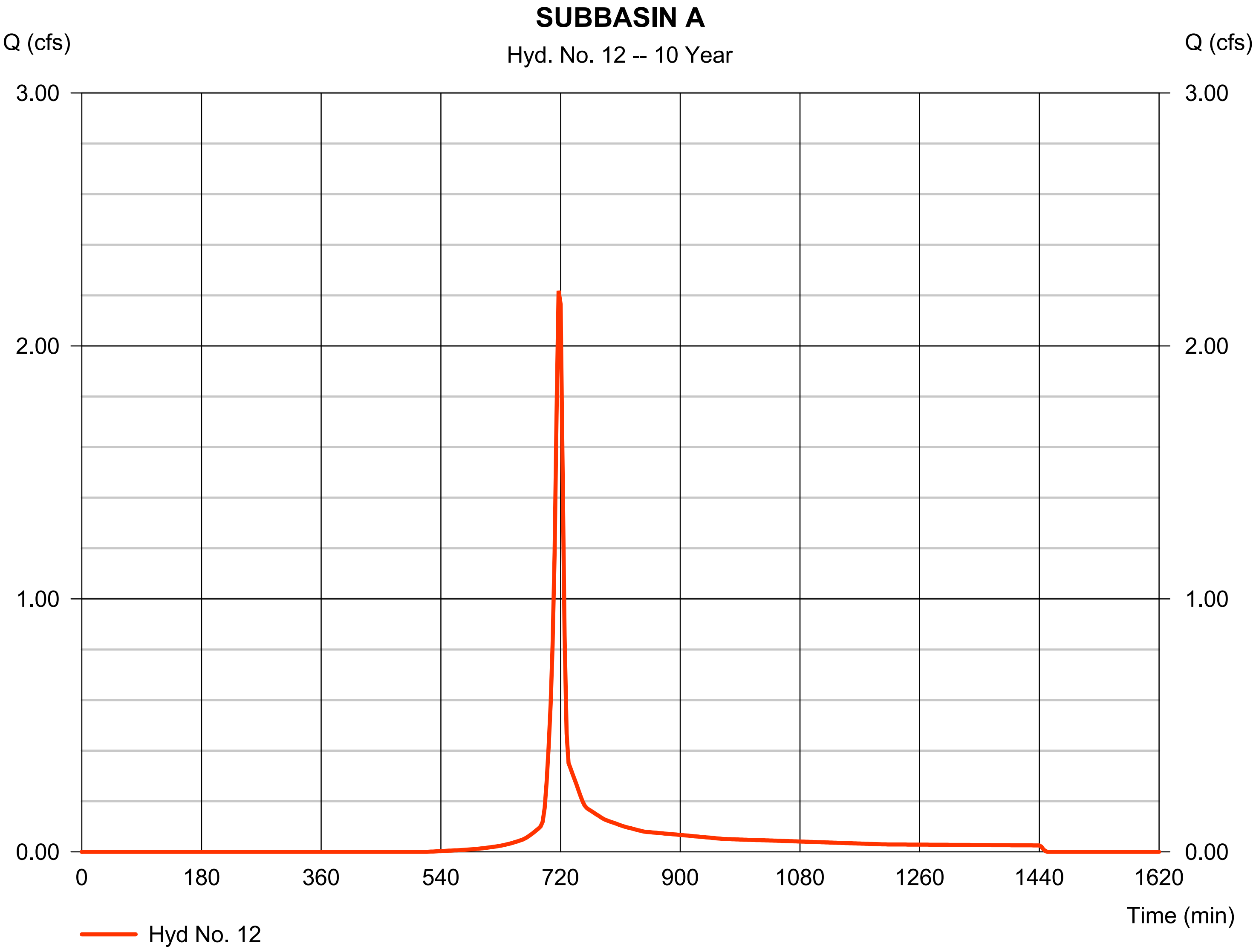
Hydrograph Report

Hyd. No. 12

SUBBASIN A

Hydrograph type	= SCS Runoff	Peak discharge	= 2.218 cfs
Storm frequency	= 10 yrs	Time to peak	= 717 min
Time interval	= 3 min	Hyd. volume	= 5,011 cuft
Drainage area	= 0.750 ac	Curve number	= 79*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 6.00 min
Total precip.	= 4.00 in	Distribution	= Type II
Storm duration	= 24 hrs	Shape factor	= 484

* Composite (Area/CN) = [(0.230 x 98) + (0.240 x 61) + (0.280 x 80)] / 0.750



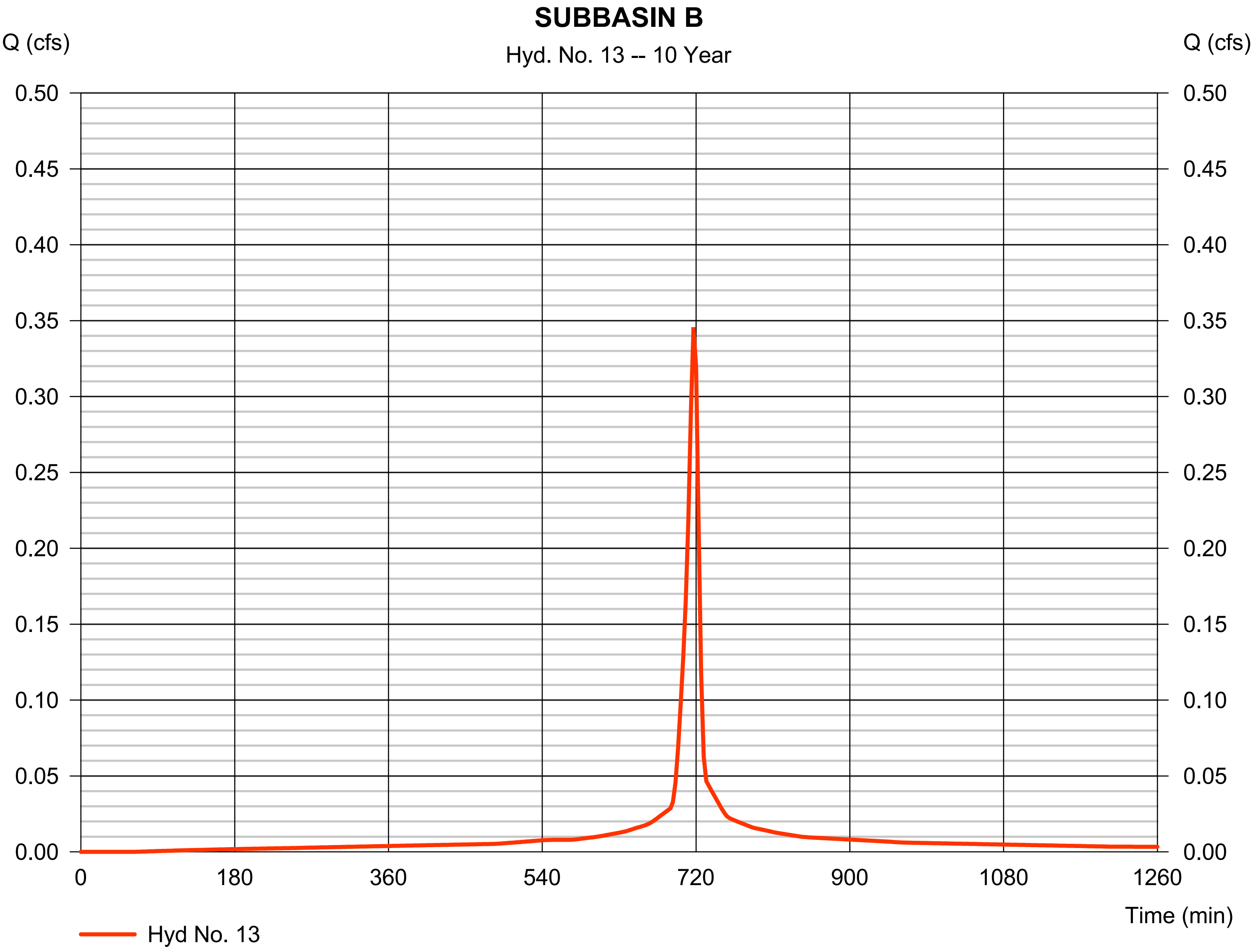
Hydrograph Report

Hyd. No. 13

SUBBASIN B

Hydrograph type	= SCS Runoff	Peak discharge	= 0.346 cfs
Storm frequency	= 10 yrs	Time to peak	= 717 min
Time interval	= 3 min	Hyd. volume	= 897 cuft
Drainage area	= 0.070 ac	Curve number	= 98*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 6.00 min
Total precip.	= 4.00 in	Distribution	= Type II
Storm duration	= 24 hrs	Shape factor	= 484

* Composite (Area/CN) = [(0.070 x 98)] / 0.070



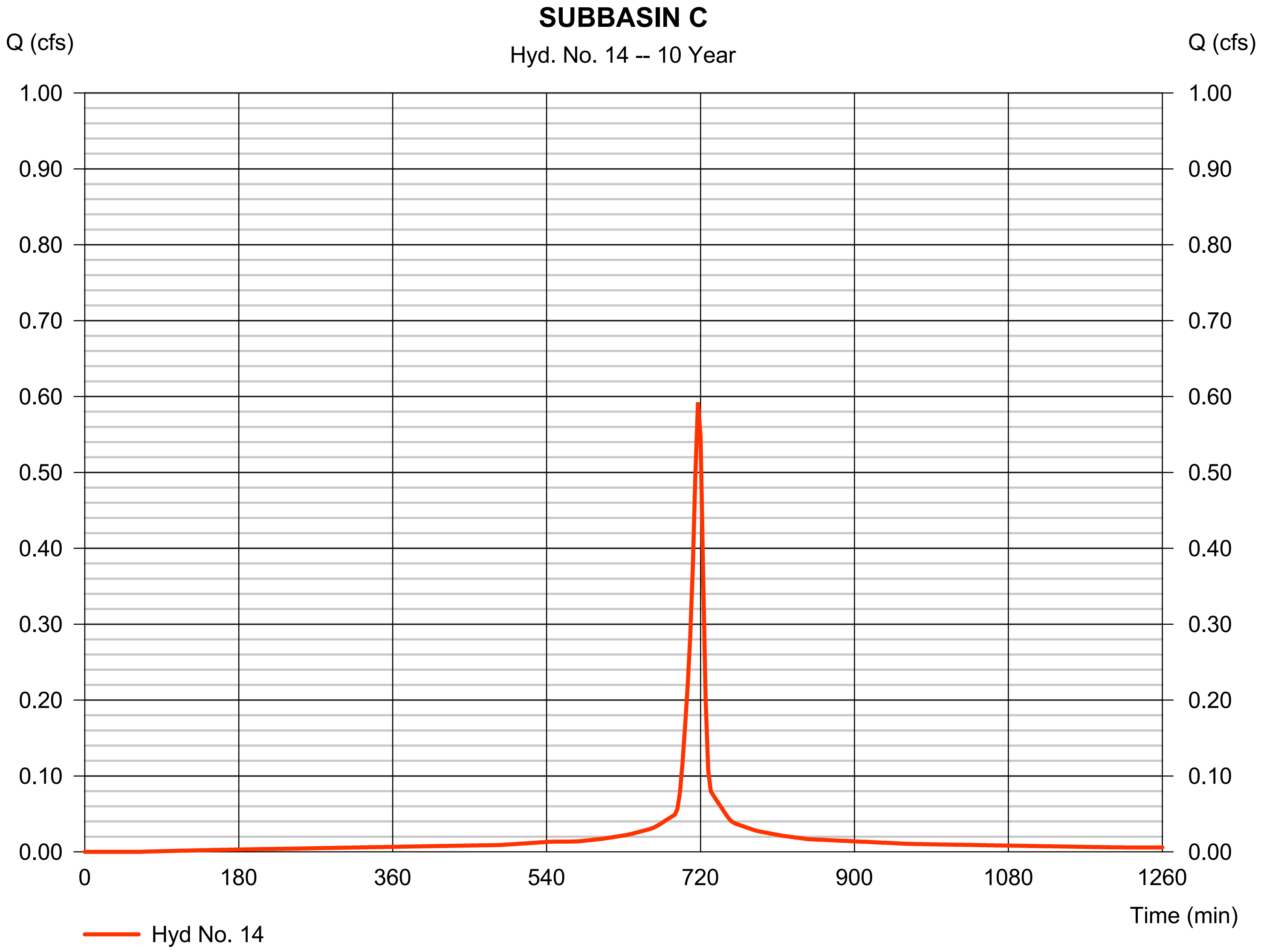
Hydrograph Report

Hyd. No. 14

SUBBASIN C

Hydrograph type	= SCS Runoff	Peak discharge	= 0.592 cfs
Storm frequency	= 10 yrs	Time to peak	= 717 min
Time interval	= 3 min	Hyd. volume	= 1,538 cuft
Drainage area	= 0.120 ac	Curve number	= 98*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 6.00 min
Total precip.	= 4.00 in	Distribution	= Type II
Storm duration	= 24 hrs	Shape factor	= 484

* Composite (Area/CN) = [(0.120 x 98)] / 0.120



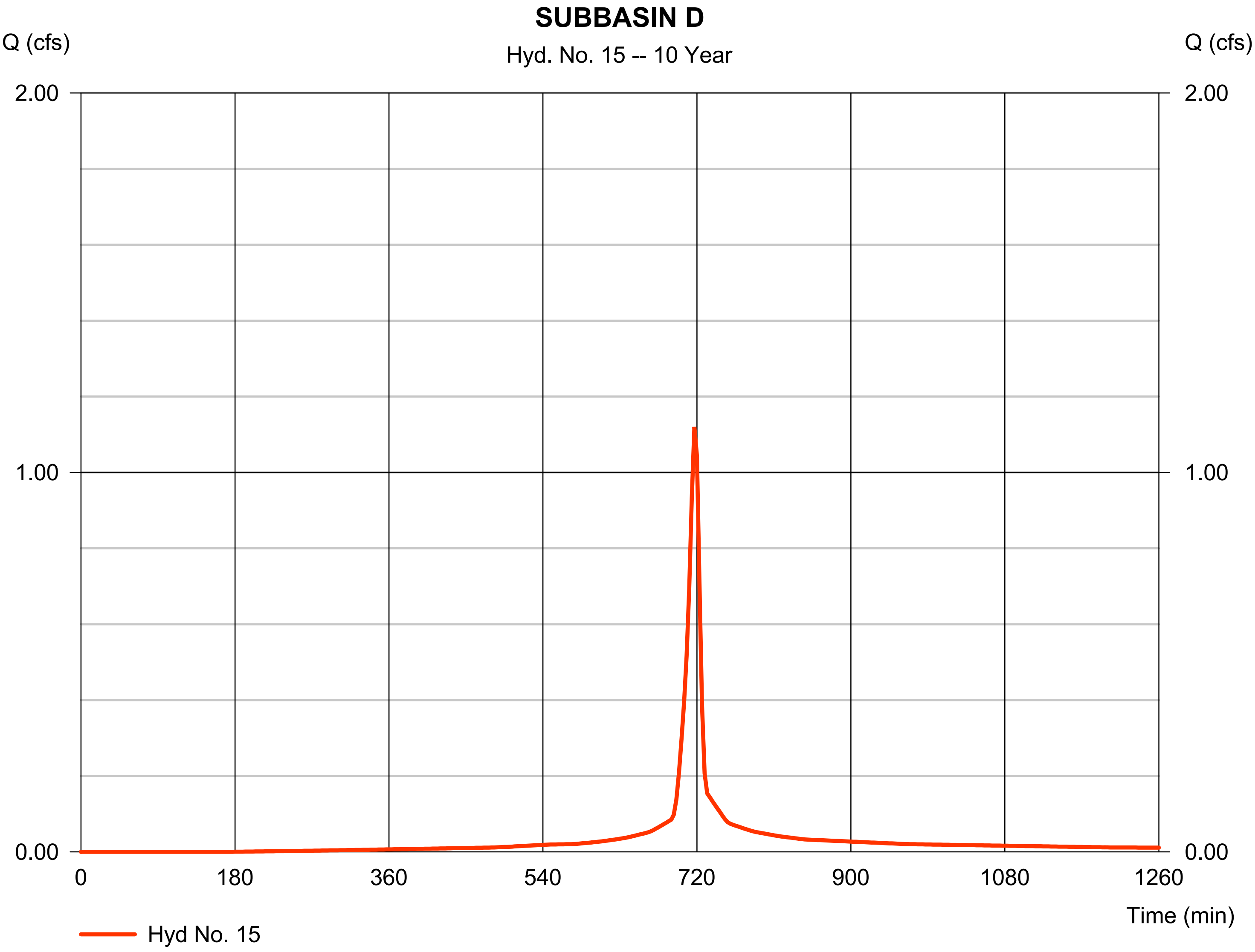
Hydrograph Report

Hyd. No. 15

SUBBASIN D

Hydrograph type	= SCS Runoff	Peak discharge	= 1.120 cfs
Storm frequency	= 10 yrs	Time to peak	= 717 min
Time interval	= 3 min	Hyd. volume	= 2,715 cuft
Drainage area	= 0.240 ac	Curve number	= 94*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 6.00 min
Total precip.	= 4.00 in	Distribution	= Type II
Storm duration	= 24 hrs	Shape factor	= 484

* Composite (Area/CN) = [(0.190 x 98) + (0.050 x 80)] / 0.240



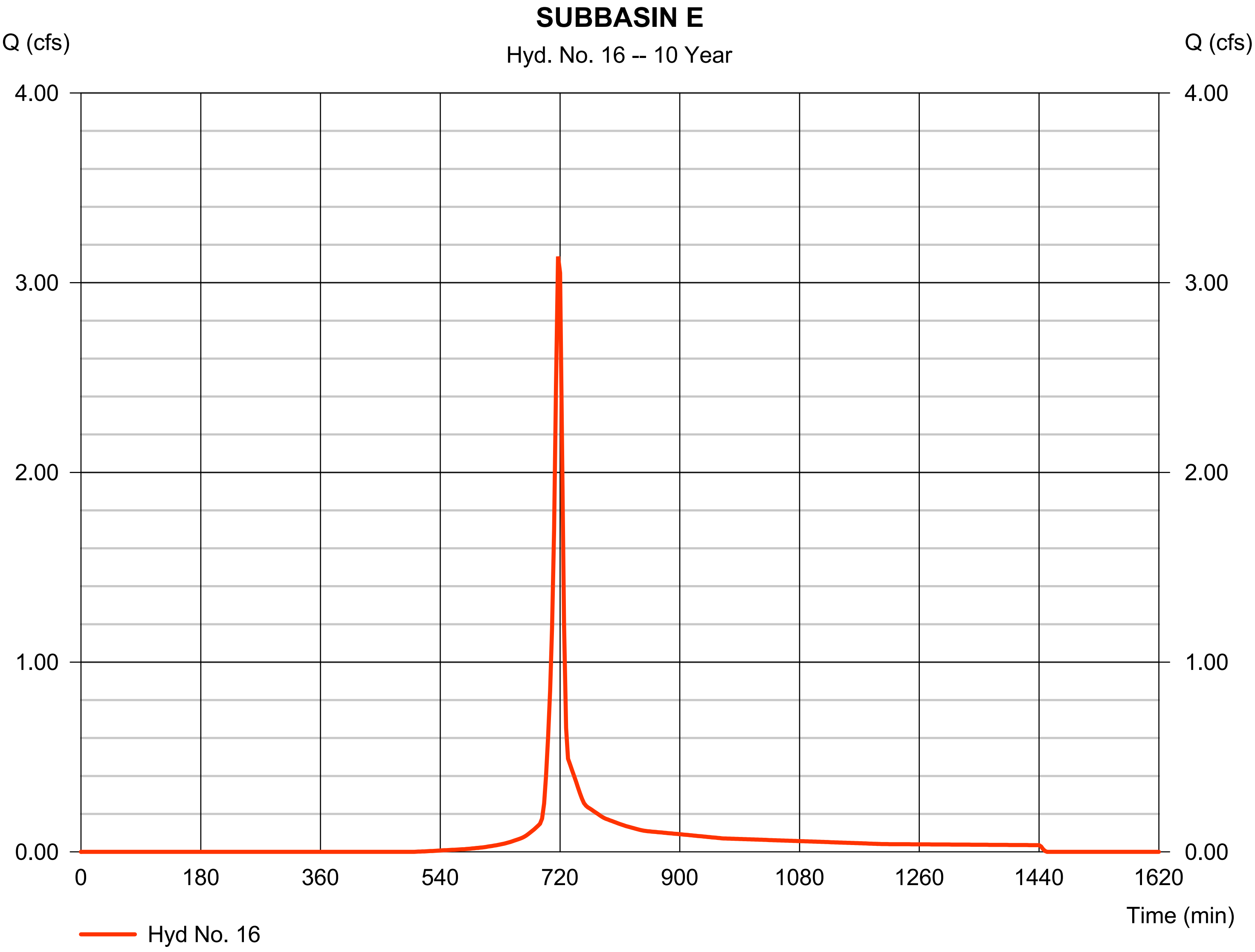
Hydrograph Report

Hyd. No. 16

SUBBASIN E

Hydrograph type	= SCS Runoff	Peak discharge	= 3.138 cfs
Storm frequency	= 10 yrs	Time to peak	= 717 min
Time interval	= 3 min	Hyd. volume	= 7,087 cuft
Drainage area	= 1.020 ac	Curve number	= 80*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 6.00 min
Total precip.	= 4.00 in	Distribution	= Type II
Storm duration	= 24 hrs	Shape factor	= 484

* Composite (Area/CN) = [(0.020 x 98) + (1.000 x 80)] / 1.020



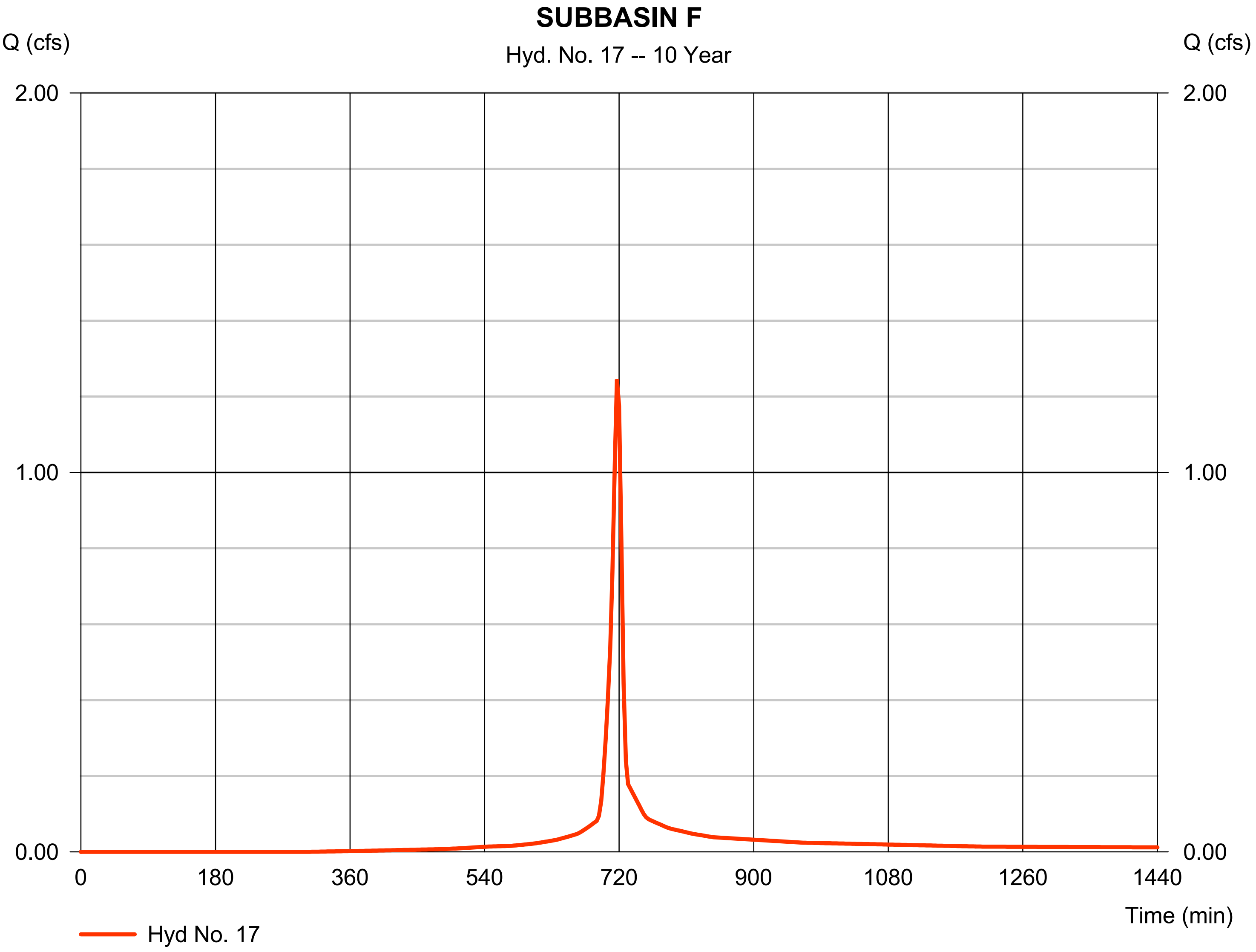
Hydrograph Report

Hyd. No. 17

SUBBASIN F

Hydrograph type	= SCS Runoff	Peak discharge	= 1.245 cfs
Storm frequency	= 10 yrs	Time to peak	= 717 min
Time interval	= 3 min	Hyd. volume	= 2,882 cuft
Drainage area	= 0.300 ac	Curve number	= 89*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 6.00 min
Total precip.	= 4.00 in	Distribution	= Type II
Storm duration	= 24 hrs	Shape factor	= 484

* Composite (Area/CN) = [(0.190 x 98) + (0.110 x 74)] / 0.300



Hydrograph Report

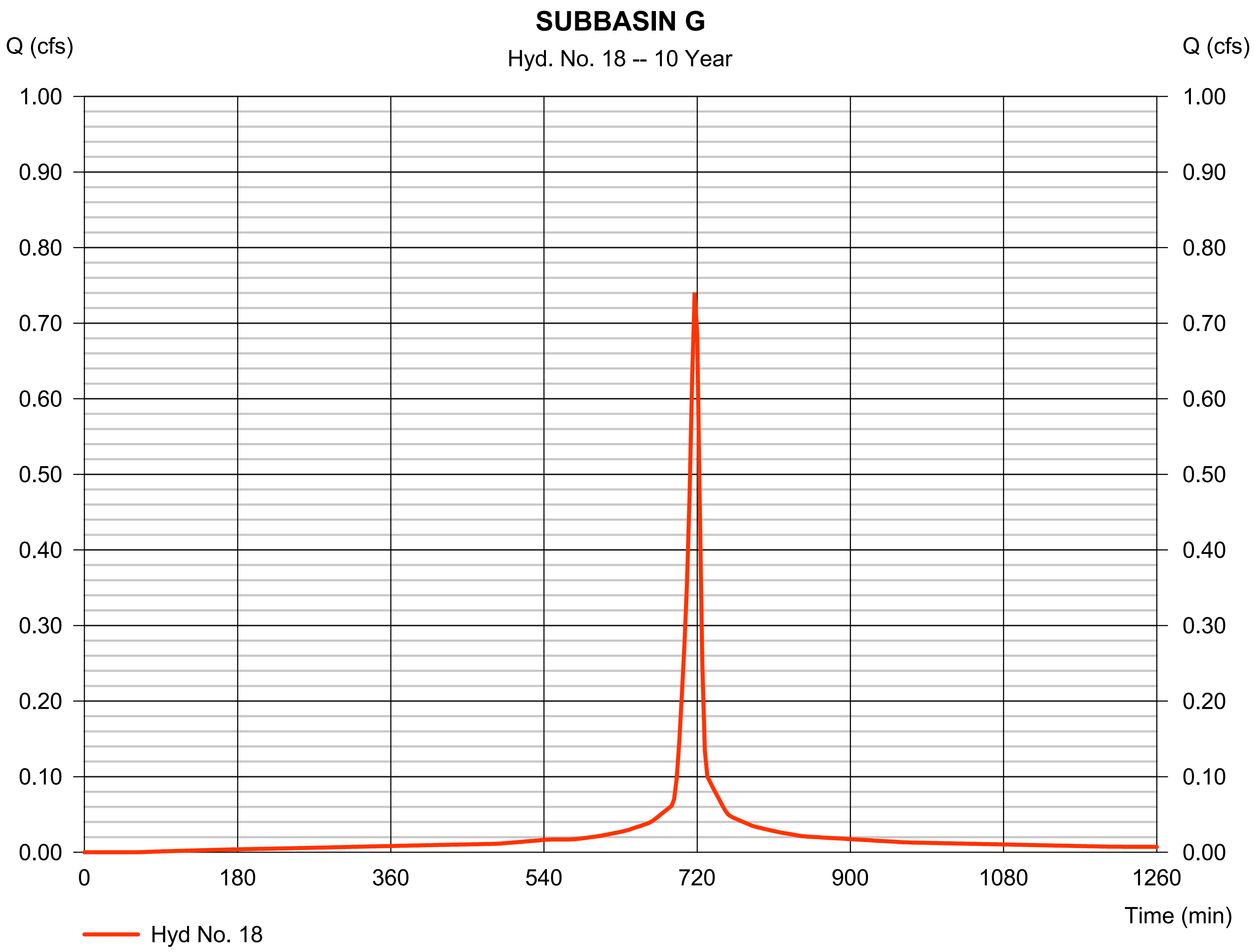
Hydraflow Hydrographs Extension for AutoCAD® Civil 3D® 2016 by Autodesk, Inc. v11 Monday, 07 / 9 / 2018

Hyd. No. 18

SUBBASIN G

Hydrograph type	= SCS Runoff	Peak discharge	= 0.741 cfs
Storm frequency	= 10 yrs	Time to peak	= 717 min
Time interval	= 3 min	Hyd. volume	= 1,922 cuft
Drainage area	= 0.150 ac	Curve number	= 98*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 6.00 min
Total precip.	= 4.00 in	Distribution	= Type II
Storm duration	= 24 hrs	Shape factor	= 484

* Composite (Area/CN) = [(0.150 x 98)] / 0.150



Hydrograph Report

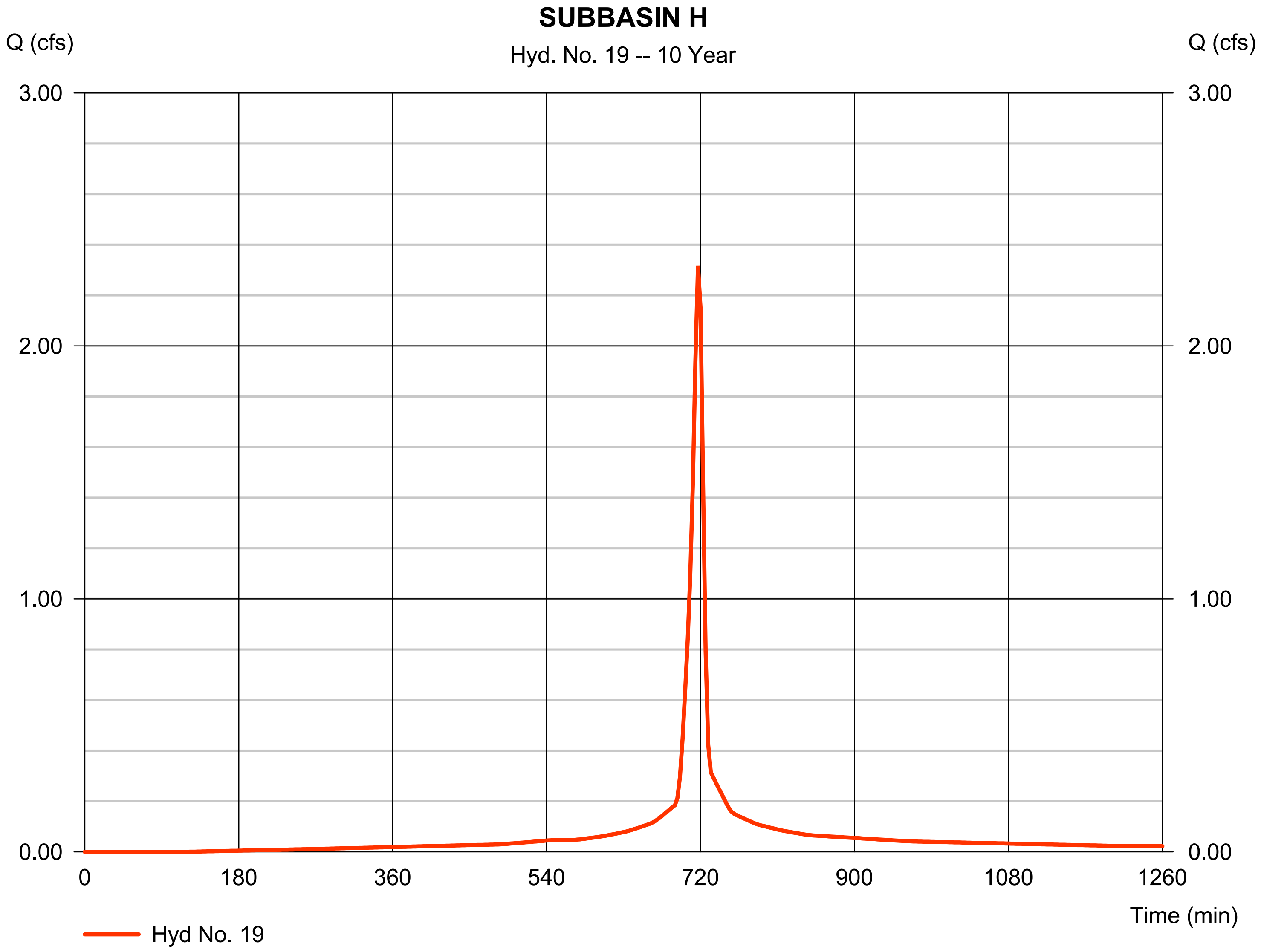
Hyd. No. 19

SUBBASIN H

Hydrograph type = SCS Runoff
Storm frequency = 10 yrs
Time interval = 3 min
Drainage area = 0.480 ac
Basin Slope = 0.0 %
Tc method = User
Total precip. = 4.00 in
Storm duration = 24 hrs

Peak discharge = 2.316 cfs
Time to peak = 717 min
Hyd. volume = 5,783 cuft
Curve number = 96*
Hydraulic length = 0 ft
Time of conc. (Tc) = 6.00 min
Distribution = Type II
Shape factor = 484

* Composite (Area/CN) = [(0.430 x 98) + (0.050 x 74)] / 0.480



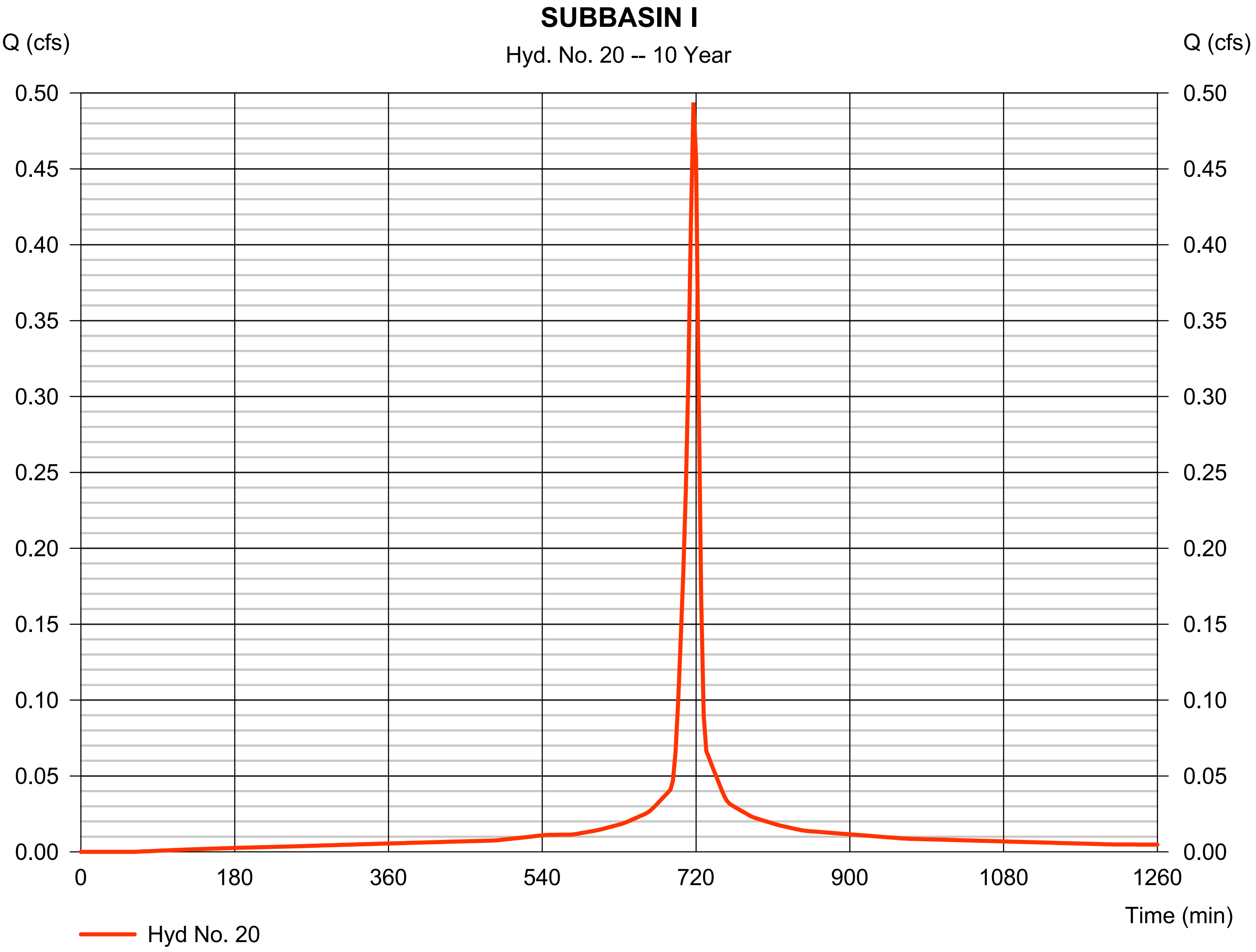
Hydrograph Report

Hyd. No. 20

SUBBASIN I

Hydrograph type	= SCS Runoff	Peak discharge	= 0.494 cfs
Storm frequency	= 10 yrs	Time to peak	= 717 min
Time interval	= 3 min	Hyd. volume	= 1,281 cuft
Drainage area	= 0.100 ac	Curve number	= 98*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 6.00 min
Total precip.	= 4.00 in	Distribution	= Type II
Storm duration	= 24 hrs	Shape factor	= 484

* Composite (Area/CN) = [(0.100 x 98)] / 0.100



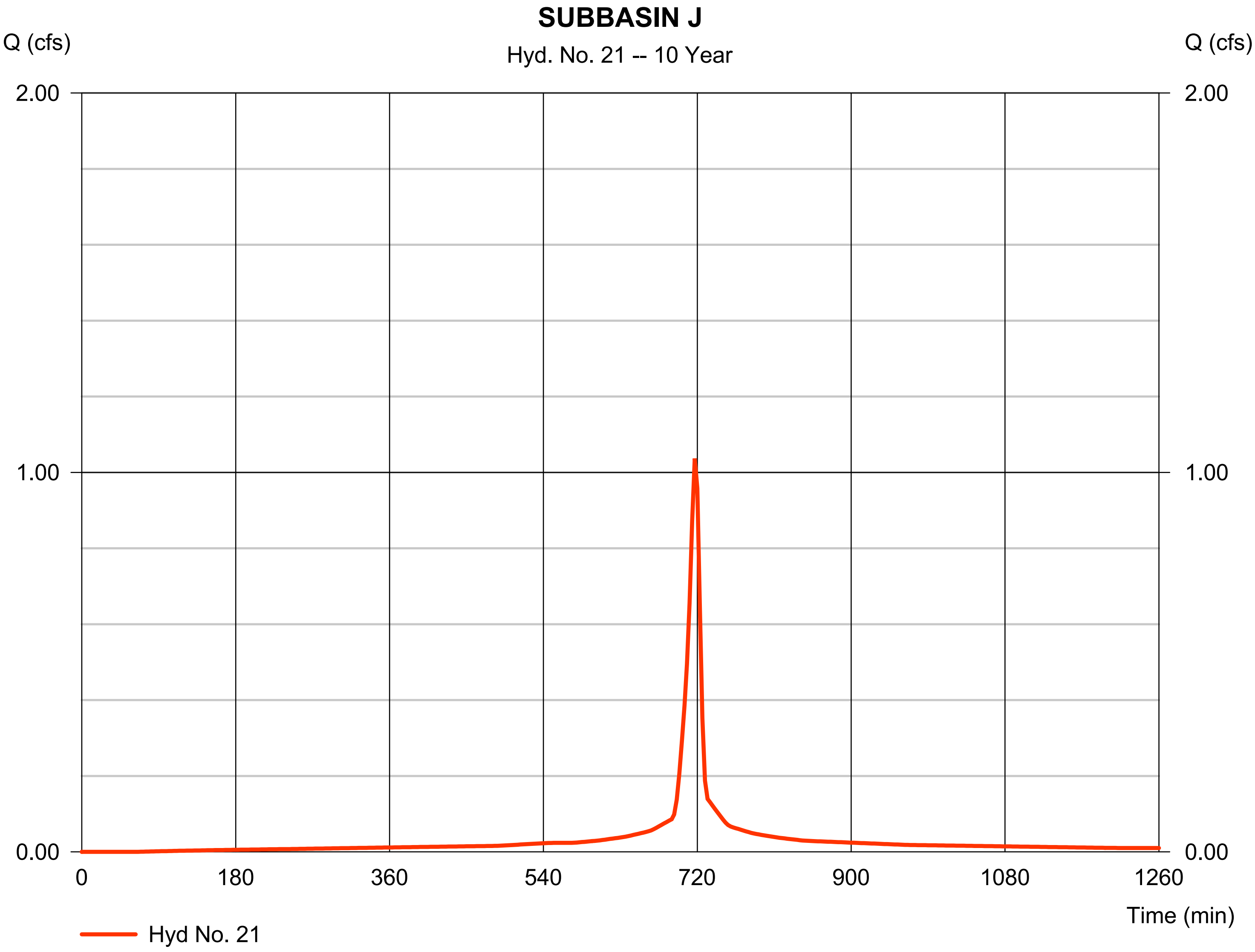
Hydrograph Report

Hyd. No. 21

SUBBASIN J

Hydrograph type	= SCS Runoff	Peak discharge	= 1.037 cfs
Storm frequency	= 10 yrs	Time to peak	= 717 min
Time interval	= 3 min	Hyd. volume	= 2,691 cuft
Drainage area	= 0.210 ac	Curve number	= 98*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 6.00 min
Total precip.	= 4.00 in	Distribution	= Type II
Storm duration	= 24 hrs	Shape factor	= 484

* Composite (Area/CN) = [(0.210 x 98)] / 0.210



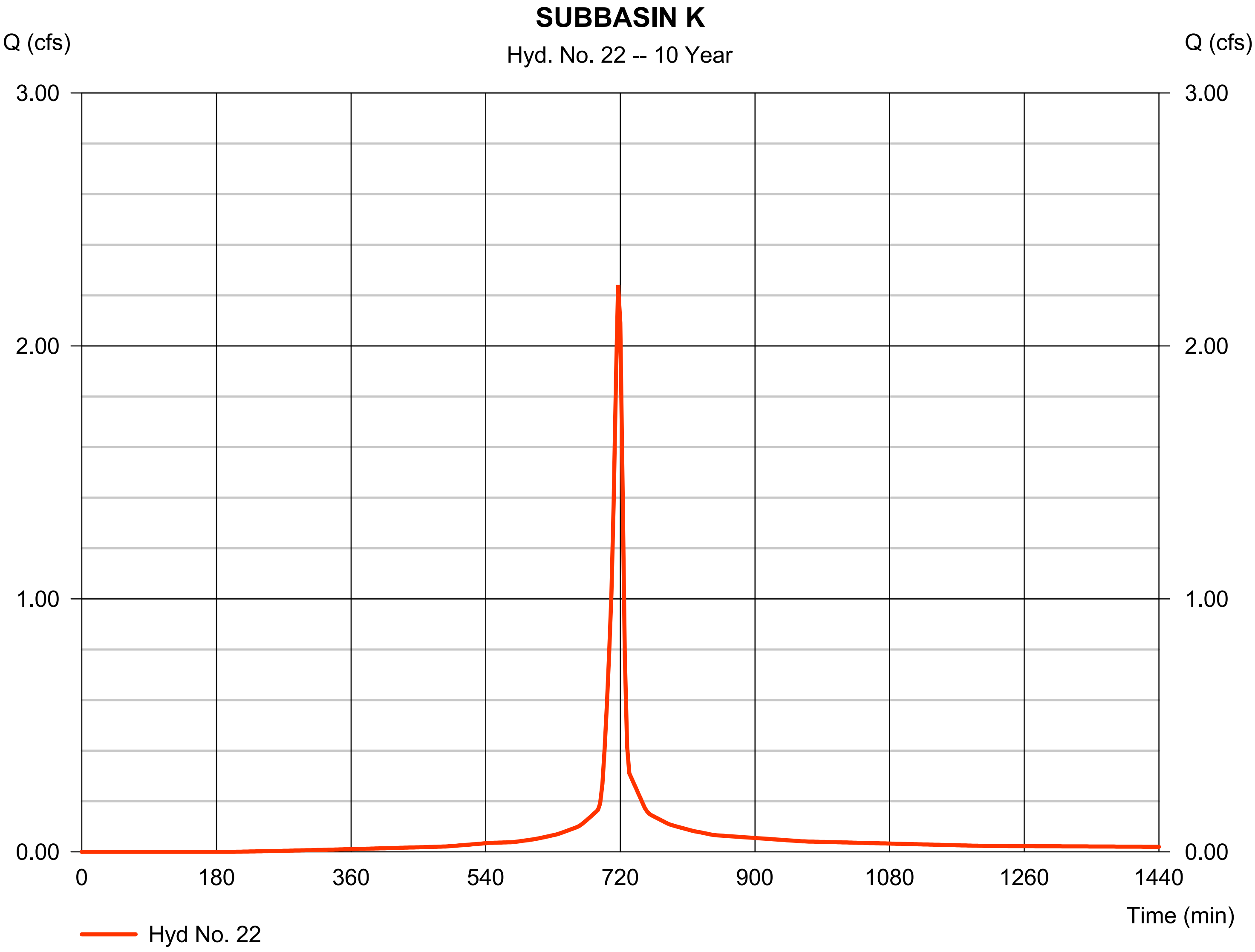
Hydrograph Report

Hyd. No. 22

SUBBASIN K

Hydrograph type	= SCS Runoff	Peak discharge	= 2.241 cfs
Storm frequency	= 10 yrs	Time to peak	= 717 min
Time interval	= 3 min	Hyd. volume	= 5,369 cuft
Drainage area	= 0.490 ac	Curve number	= 93*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 6.00 min
Total precip.	= 4.00 in	Distribution	= Type II
Storm duration	= 24 hrs	Shape factor	= 484

* Composite (Area/CN) = [(0.380 x 98) + (0.110 x 74)] / 0.490



Hydrograph Summary Report

Hyd. No.	Hydrograph type (origin)	Peak flow (cfs)	Time interval (min)	Time to Peak (min)	Hyd. volume (cuft)	Inflow hyd(s)	Maximum elevation (ft)	Total strge used (cuft)	Hydrograph Description
1	SCS Runoff	17.03	3	726	59,775	------	------	------	Pre-Basin A
2	SCS Runoff	3.617	3	717	8,328	------	------	------	Pre-Offsite
3	Combine	18.32	3	726	68,103	1, 2	------	------	Total Predevelopment
5	SCS Runoff	32.38	3	717	74,559	------	------	------	Post- Pond
6	SCS Runoff	0.509	3	717	1,171	------	------	------	Post- Offsite
7	Reservoir	15.91	3	726	74,559	5	139.80	21,233	Pond Discharge
9	Combine	16.09	3	726	75,730	6, 7,	------	------	Total Post Discharge
12	SCS Runoff	3.742	3	717	8,486	------	------	------	SUBBASIN A
13	SCS Runoff	0.486	3	717	1,277	------	------	------	SUBBASIN B
14	SCS Runoff	0.833	3	717	2,190	------	------	------	SUBBASIN C
15	SCS Runoff	1.613	3	717	4,003	------	------	------	SUBBASIN D
16	SCS Runoff	5.227	3	717	11,880	------	------	------	SUBBASIN E
17	SCS Runoff	1.875	3	717	4,440	------	------	------	SUBBASIN F
18	SCS Runoff	1.041	3	717	2,737	------	------	------	SUBBASIN G
19	SCS Runoff	3.289	3	717	8,379	------	------	------	SUBBASIN H
20	SCS Runoff	0.694	3	717	1,825	------	------	------	SUBBASIN I
21	SCS Runoff	1.458	3	717	3,832	------	------	------	SUBBASIN J
22	SCS Runoff	3.254	3	717	7,984	------	------	------	SUBBASIN K

1818660STORM.gpw	Return Period: 100 Year	Monday, 07 / 9 / 2018

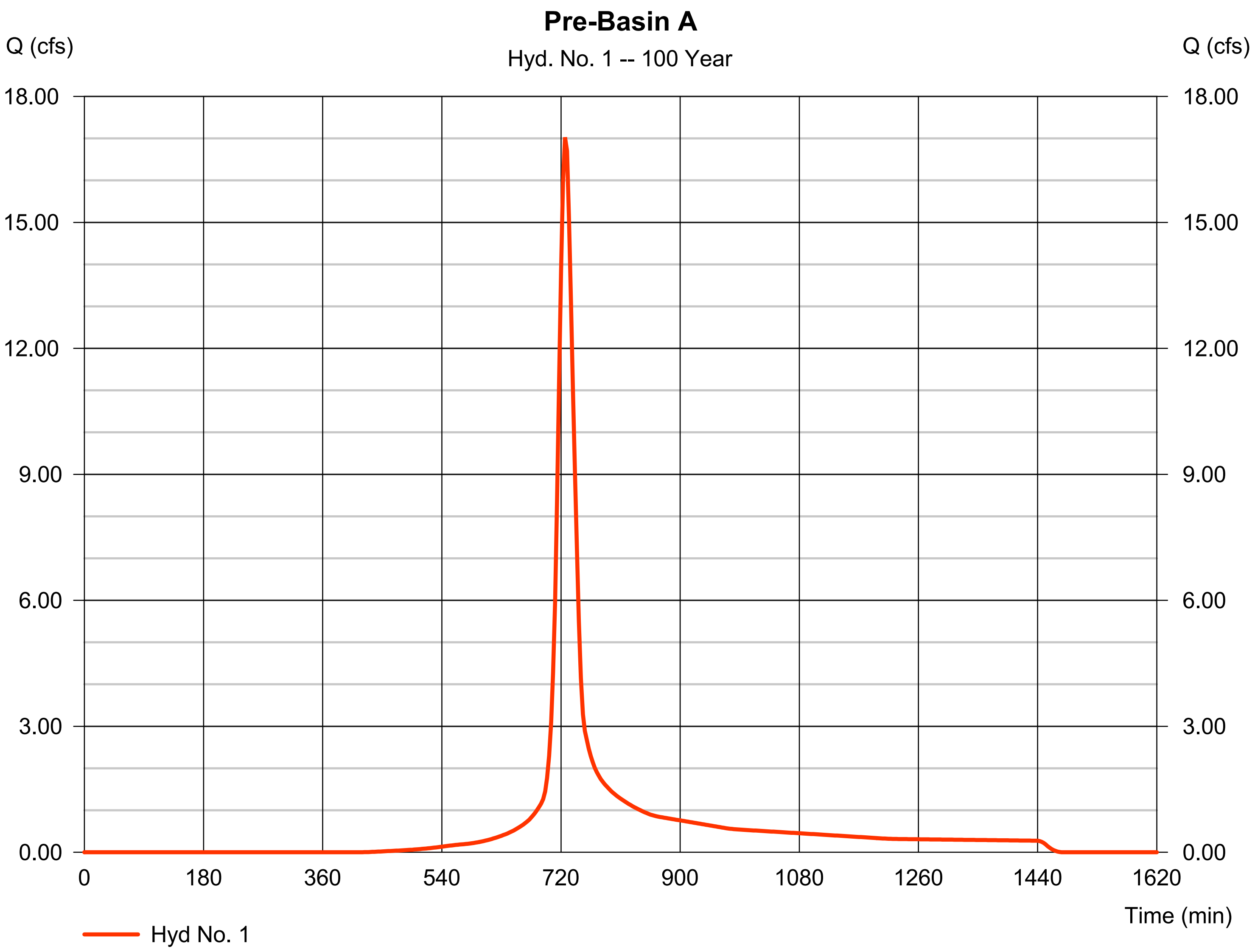
Hydrograph Report

Hyd. No. 1

Pre-Basin A

Hydrograph type	= SCS Runoff	Peak discharge	= 17.03 cfs
Storm frequency	= 100 yrs	Time to peak	= 726 min
Time interval	= 3 min	Hyd. volume	= 59,775 cuft
Drainage area	= 5.080 ac	Curve number	= 79*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= TR55	Time of conc. (Tc)	= 21.40 min
Total precip.	= 5.60 in	Distribution	= Type II
Storm duration	= 24 hrs	Shape factor	= 484

* Composite (Area/CN) = [(0.260 x 61) + (1.160 x 98) + (1.170 x 74) + (1.230 x 70) + (0.670 x 80) + (0.590 x 77)] / 5.080



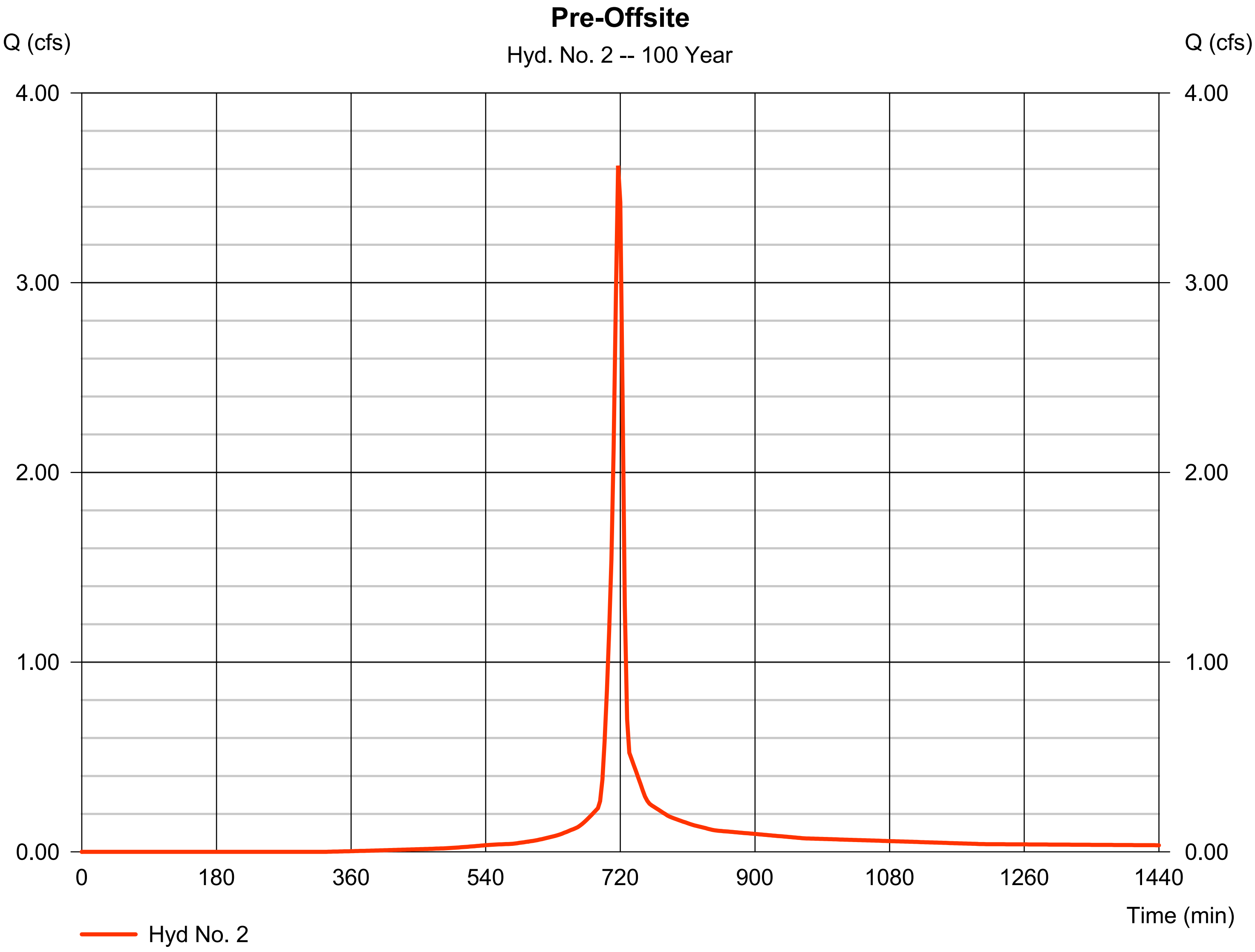
Hydrograph Report

Hyd. No. 2

Pre-Offsite

Hydrograph type	= SCS Runoff	Peak discharge	= 3.617 cfs
Storm frequency	= 100 yrs	Time to peak	= 717 min
Time interval	= 3 min	Hyd. volume	= 8,328 cuft
Drainage area	= 0.640 ac	Curve number	= 84*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 6.00 min
Total precip.	= 5.60 in	Distribution	= Type II
Storm duration	= 24 hrs	Shape factor	= 484

* Composite (Area/CN) = [(0.370 x 74) + (0.270 x 98)] / 0.640

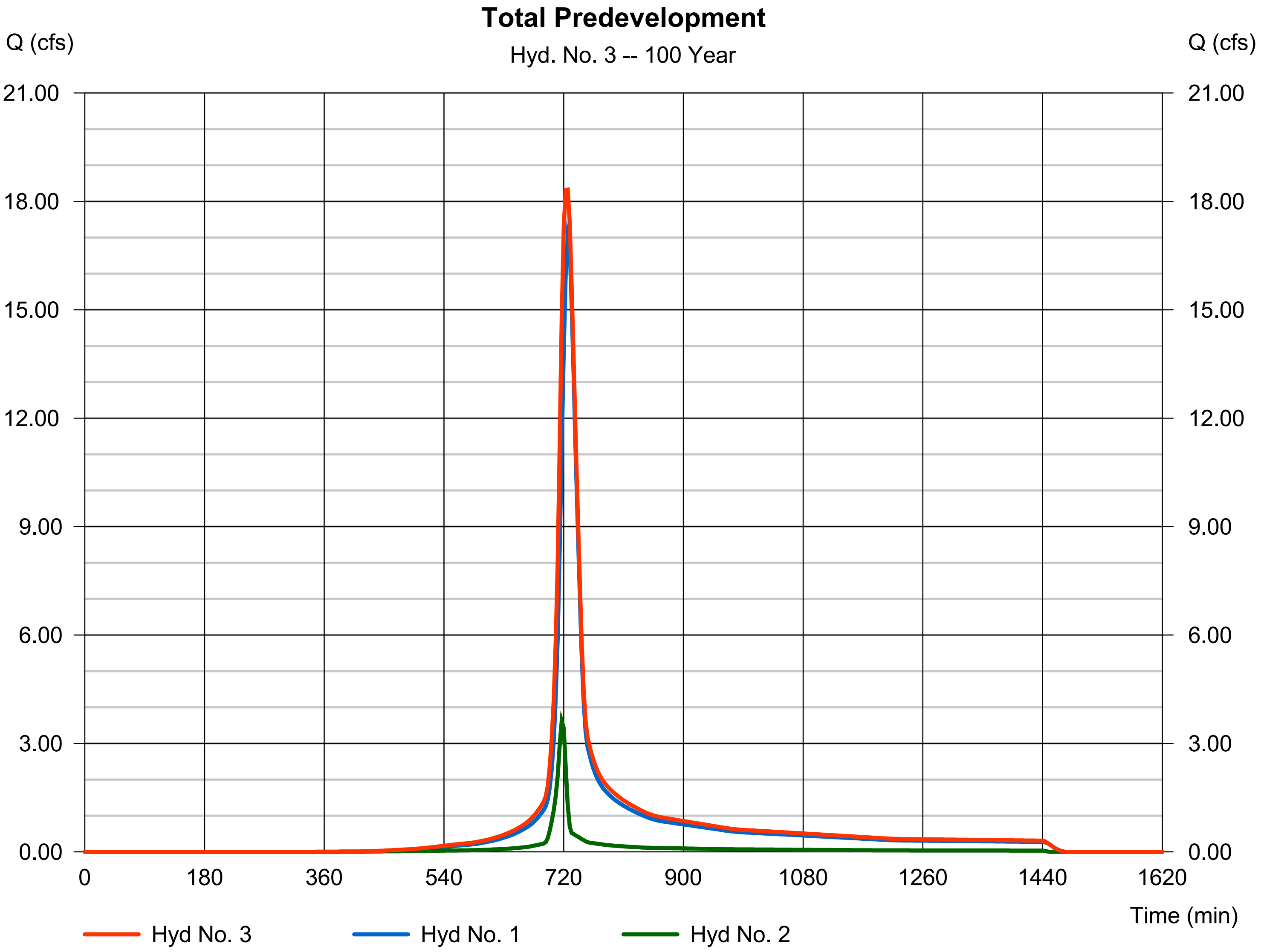


Hydrograph Report

Hyd. No. 3

Total Predevelopment

Hydrograph type	= Combine	Peak discharge	= 18.32 cfs
Storm frequency	= 100 yrs	Time to peak	= 726 min
Time interval	= 3 min	Hyd. volume	= 68,103 cuft
Inflow hyds.	= 1, 2	Contrib. drain. area	= 5.720 ac



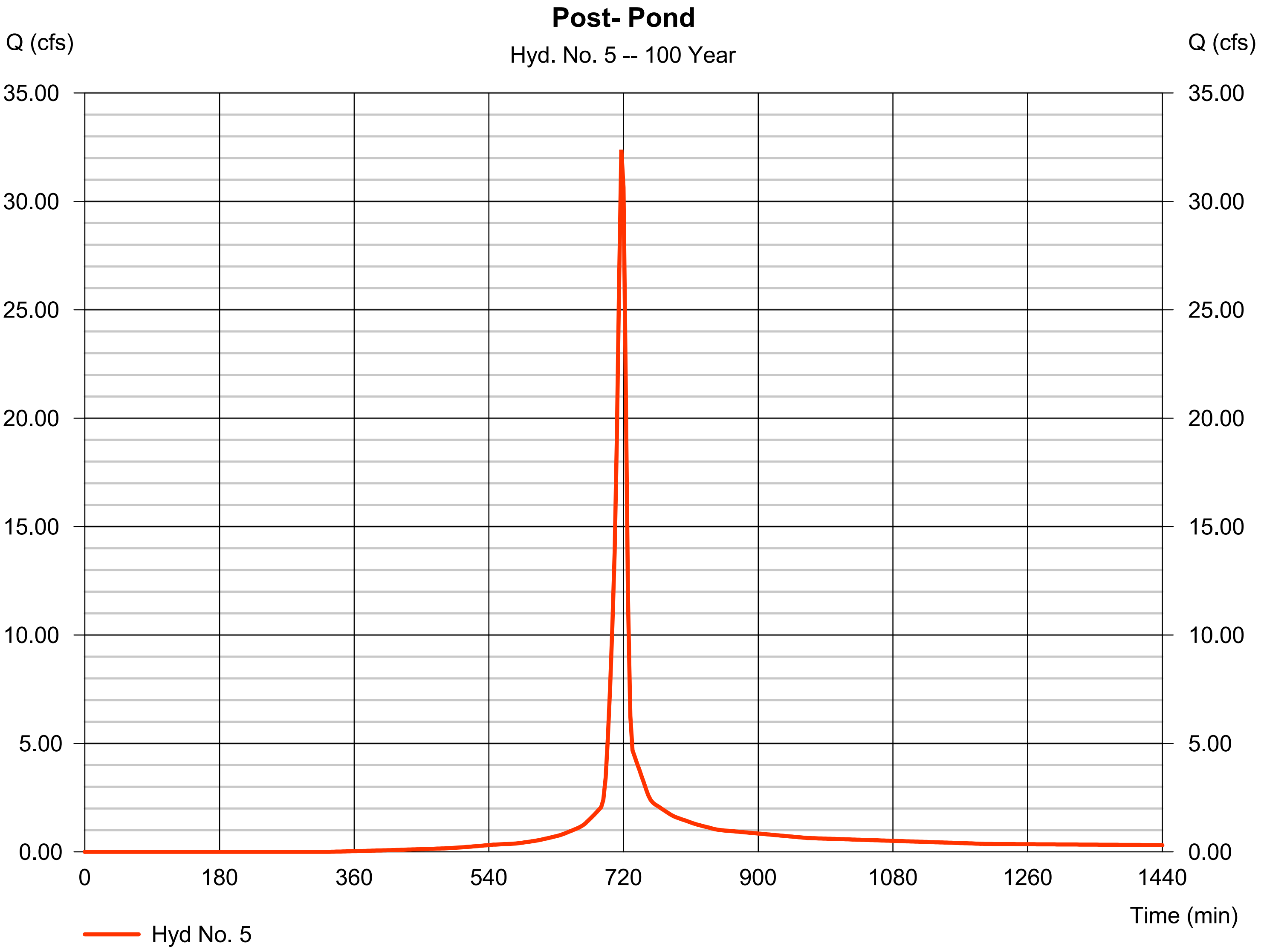
Hydrograph Report

Hyd. No. 5

Post- Pond

Hydrograph type	= SCS Runoff	Peak discharge	= 32.38 cfs
Storm frequency	= 100 yrs	Time to peak	= 717 min
Time interval	= 3 min	Hyd. volume	= 74,559 cuft
Drainage area	= 5.730 ac	Curve number	= 84*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 6.00 min
Total precip.	= 5.60 in	Distribution	= Type II
Storm duration	= 24 hrs	Shape factor	= 484

* Composite (Area/CN) = [(0.260 x 61) + (2.350 x 98) + (2.020 x 74) + (1.100 x 80)] / 5.730



Hydrograph Report

Hydraflow Hydrographs Extension for AutoCAD® Civil 3D® 2016 by Autodesk, Inc. v11 Monday, 07 / 9 / 2018

Hyd. No. 6

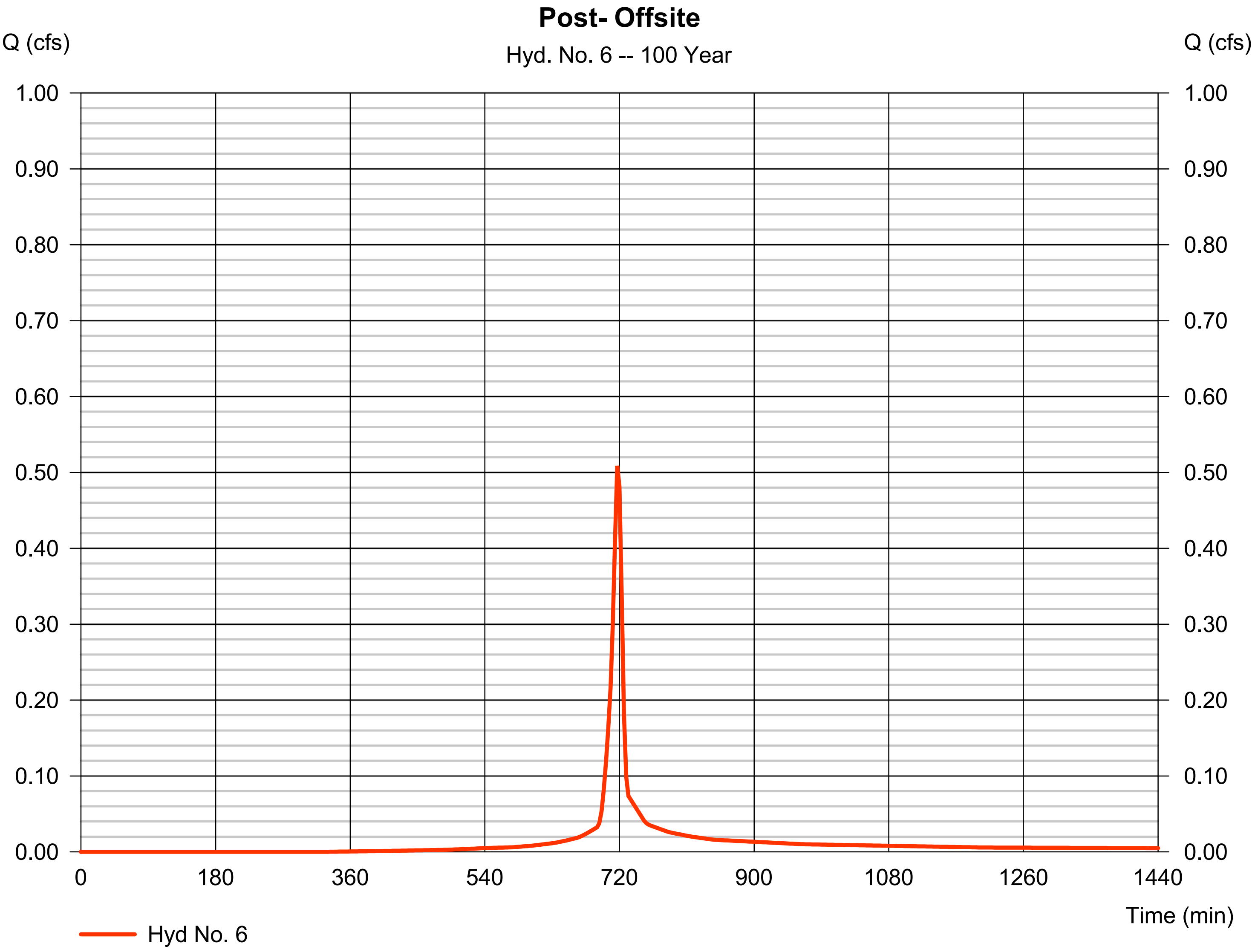
Post- Offsite

Hydrograph type = SCS Runoff
Storm frequency = 100 yrs
Time interval = 3 min
Drainage area = 0.090 ac
Basin Slope = 0.0 %
Tc method = User
Total precip. = 5.60 in
Storm duration = 24 hrs

Peak discharge = 0.509 cfs
Time to peak = 717 min
Hyd. volume = 1,171 cuft
Curve number = 84*
Hydraulic length = 0 ft
Time of conc. (Tc) = 6.00 min
Distribution = Type II
Shape factor = 484

---

* Composite (Area/CN) = [(0.030 x 61) + (0.050 x 98) + (0.010 x 80)] / 0.090



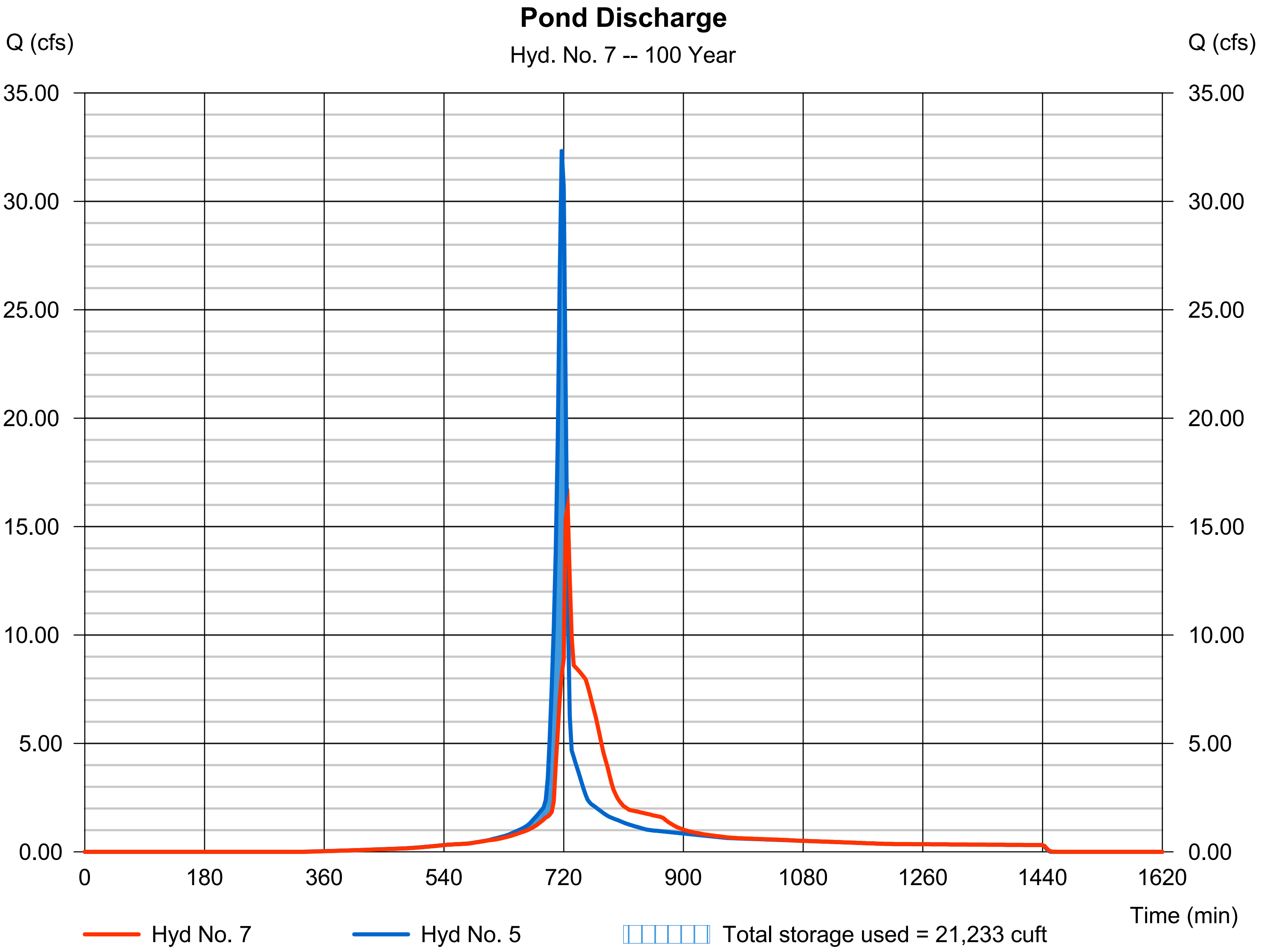
Hydrograph Report

Hyd. No. 7

Pond Discharge

Hydrograph type	= Reservoir	Peak discharge	= 15.91 cfs
Storm frequency	= 100 yrs	Time to peak	= 726 min
Time interval	= 3 min	Hyd. volume	= 74,559 cuft
Inflow hyd. No.	= 5 - Post- Pond	Max. Elevation	= 139.80 ft
Reservoir name	= South Detention	Max. Storage	= 21,233 cuft

Storage Indication method used.

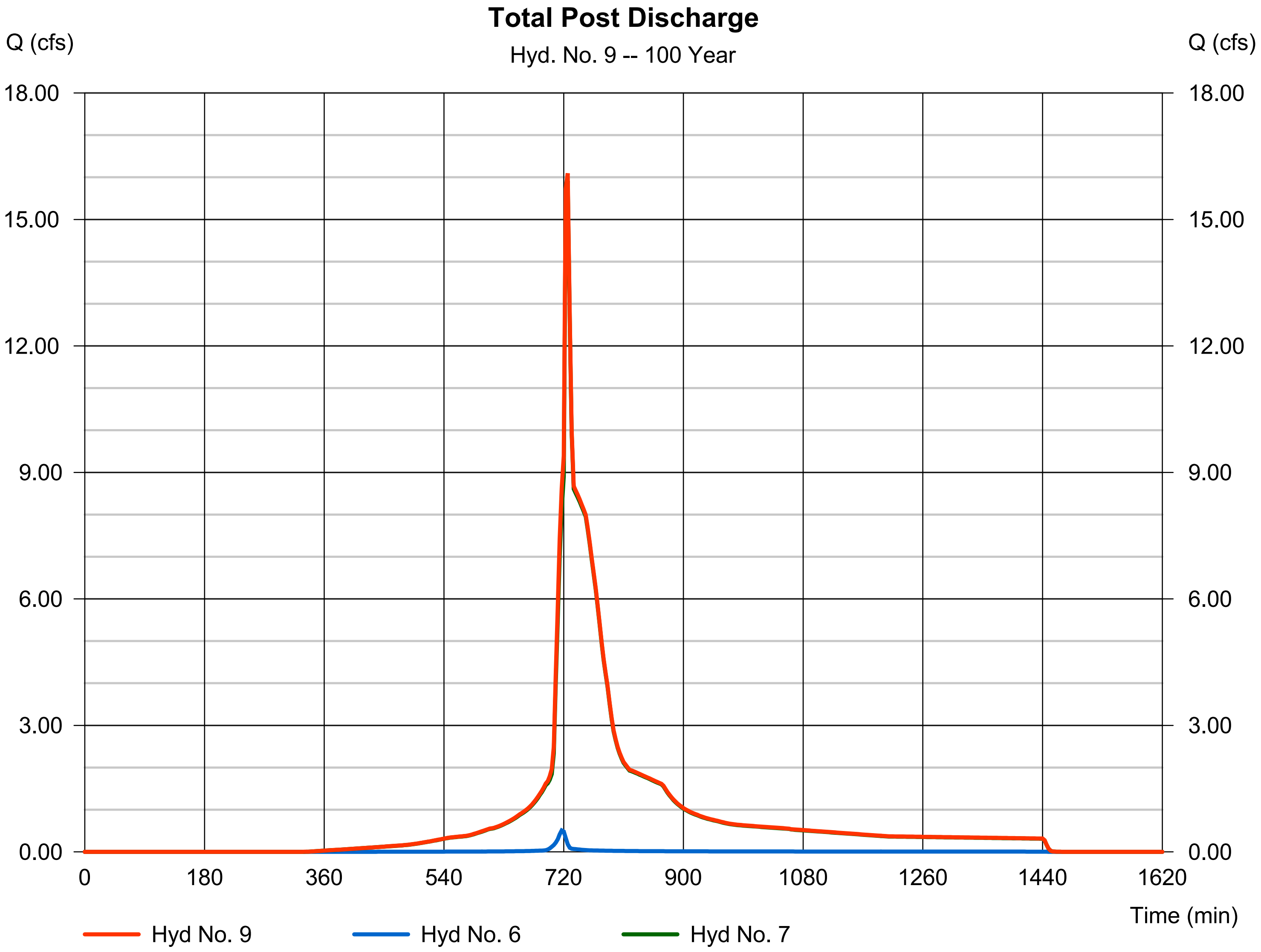


Hydrograph Report

Hyd. No. 9

Total Post Discharge

Hydrograph type	= Combine	Peak discharge	= 16.09 cfs
Storm frequency	= 100 yrs	Time to peak	= 726 min
Time interval	= 3 min	Hyd. volume	= 75,730 cuft
Inflow hyds.	= 6, 7	Contrib. drain. area	= 0.090 ac



Hydrograph Report

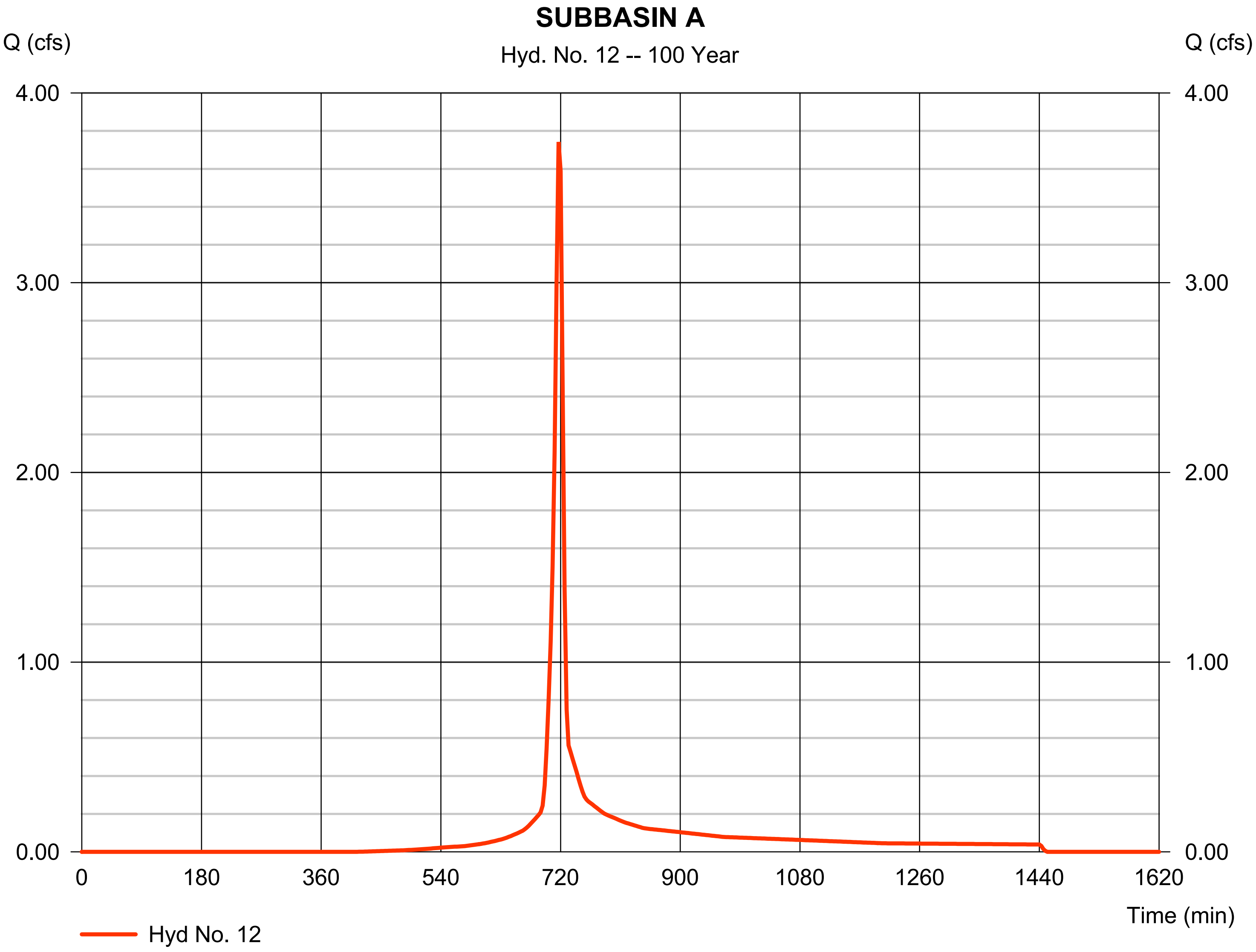
Hyd. No. 12

SUBBASIN A

Hydrograph type = SCS Runoff
Storm frequency = 100 yrs
Time interval = 3 min
Drainage area = 0.750 ac
Basin Slope = 0.0 %
Tc method = User
Total precip. = 5.60 in
Storm duration = 24 hrs

Peak discharge = 3.742 cfs
Time to peak = 717 min
Hyd. volume = 8,486 cuft
Curve number = 79*
Hydraulic length = 0 ft
Time of conc. (Tc) = 6.00 min
Distribution = Type II
Shape factor = 484

* Composite (Area/CN) = [(0.230 x 98) + (0.240 x 61) + (0.280 x 80)] / 0.750



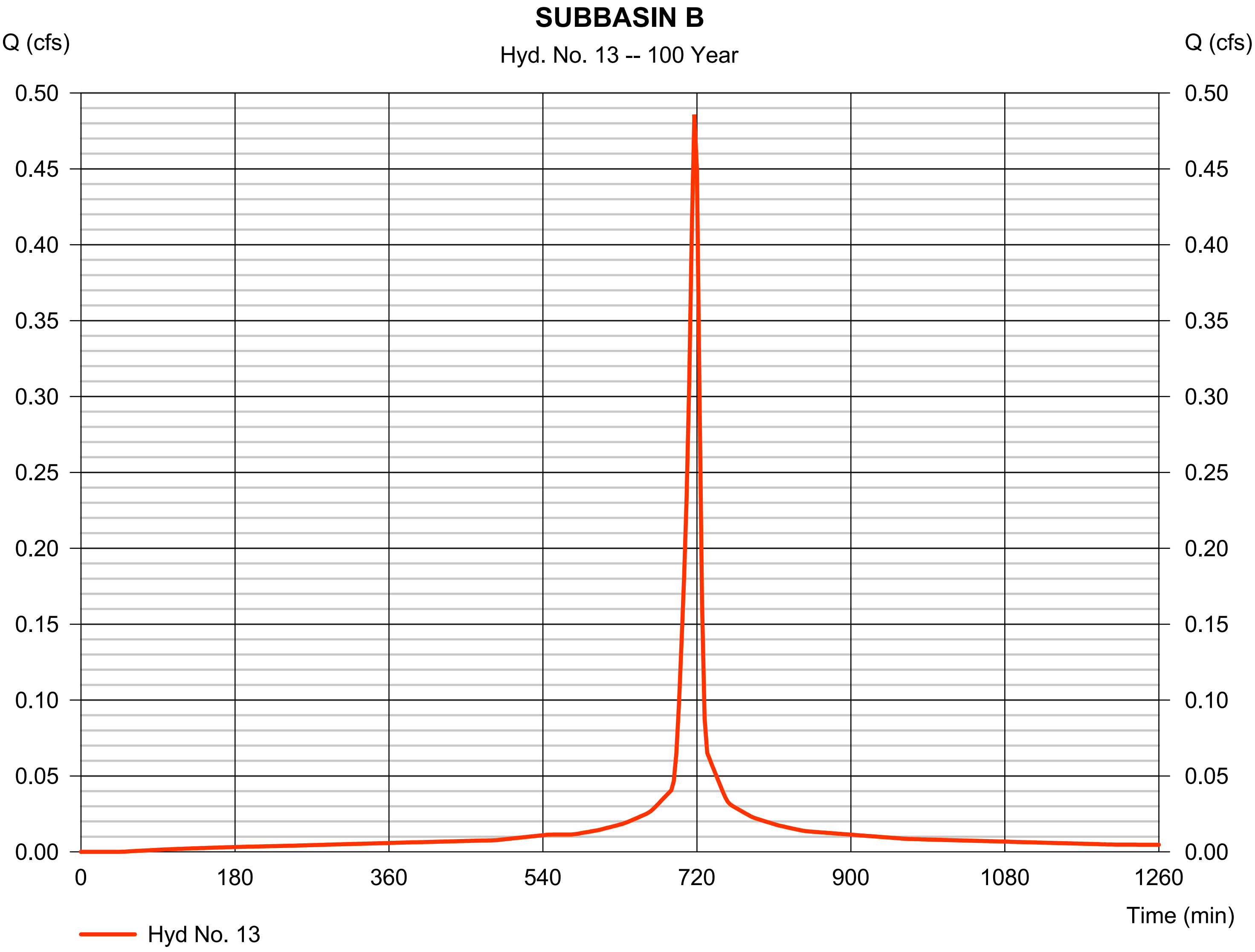
Hydrograph Report

Hyd. No. 13

SUBBASIN B

Hydrograph type	= SCS Runoff	Peak discharge	= 0.486 cfs
Storm frequency	= 100 yrs	Time to peak	= 717 min
Time interval	= 3 min	Hyd. volume	= 1,277 cuft
Drainage area	= 0.070 ac	Curve number	= 98*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 6.00 min
Total precip.	= 5.60 in	Distribution	= Type II
Storm duration	= 24 hrs	Shape factor	= 484

* Composite (Area/CN) = [(0.070 x 98)] / 0.070



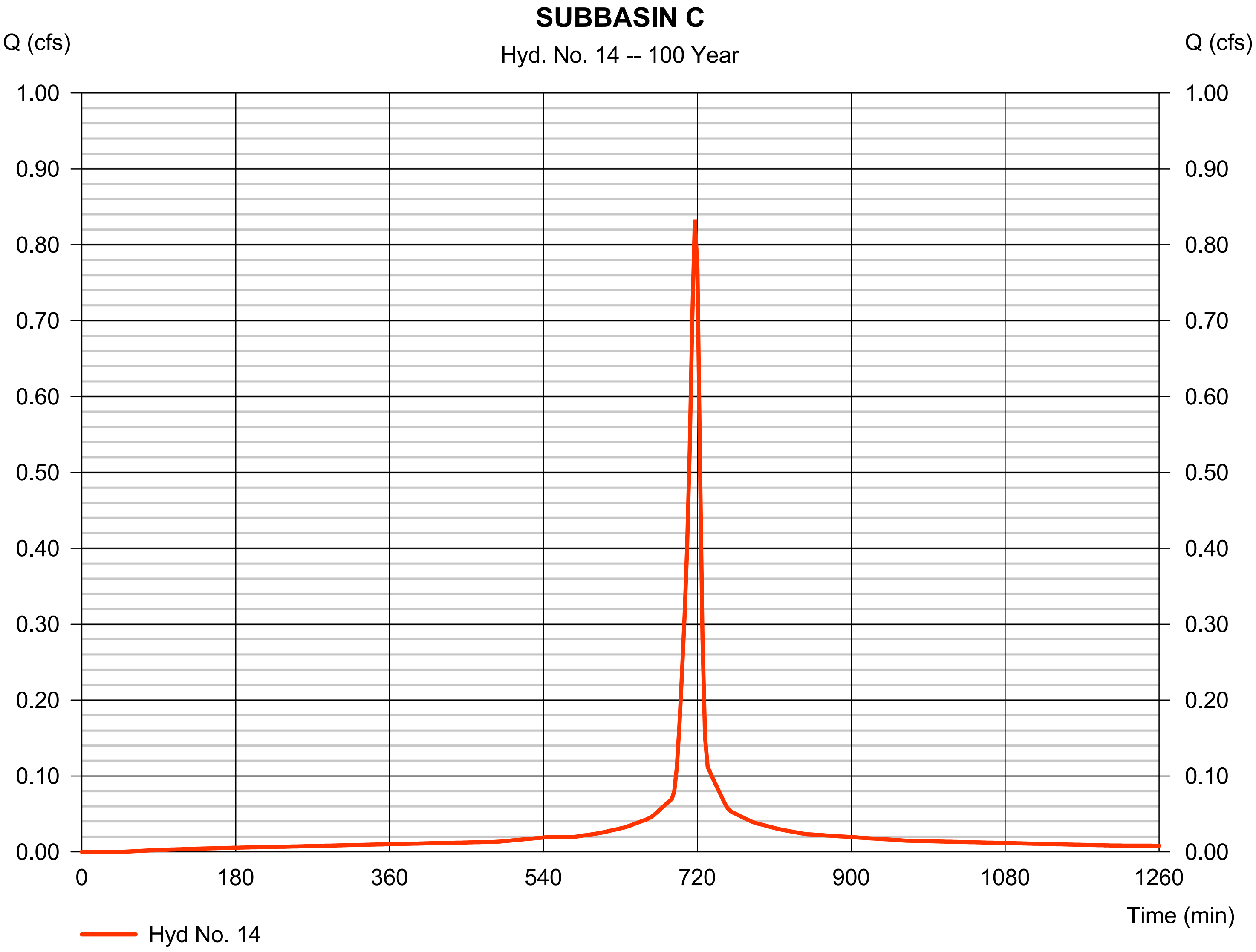
Hydrograph Report

Hyd. No. 14

SUBBASIN C

Hydrograph type	= SCS Runoff	Peak discharge	= 0.833 cfs
Storm frequency	= 100 yrs	Time to peak	= 717 min
Time interval	= 3 min	Hyd. volume	= 2,190 cuft
Drainage area	= 0.120 ac	Curve number	= 98*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 6.00 min
Total precip.	= 5.60 in	Distribution	= Type II
Storm duration	= 24 hrs	Shape factor	= 484

* Composite (Area/CN) = [(0.120 x 98)] / 0.120



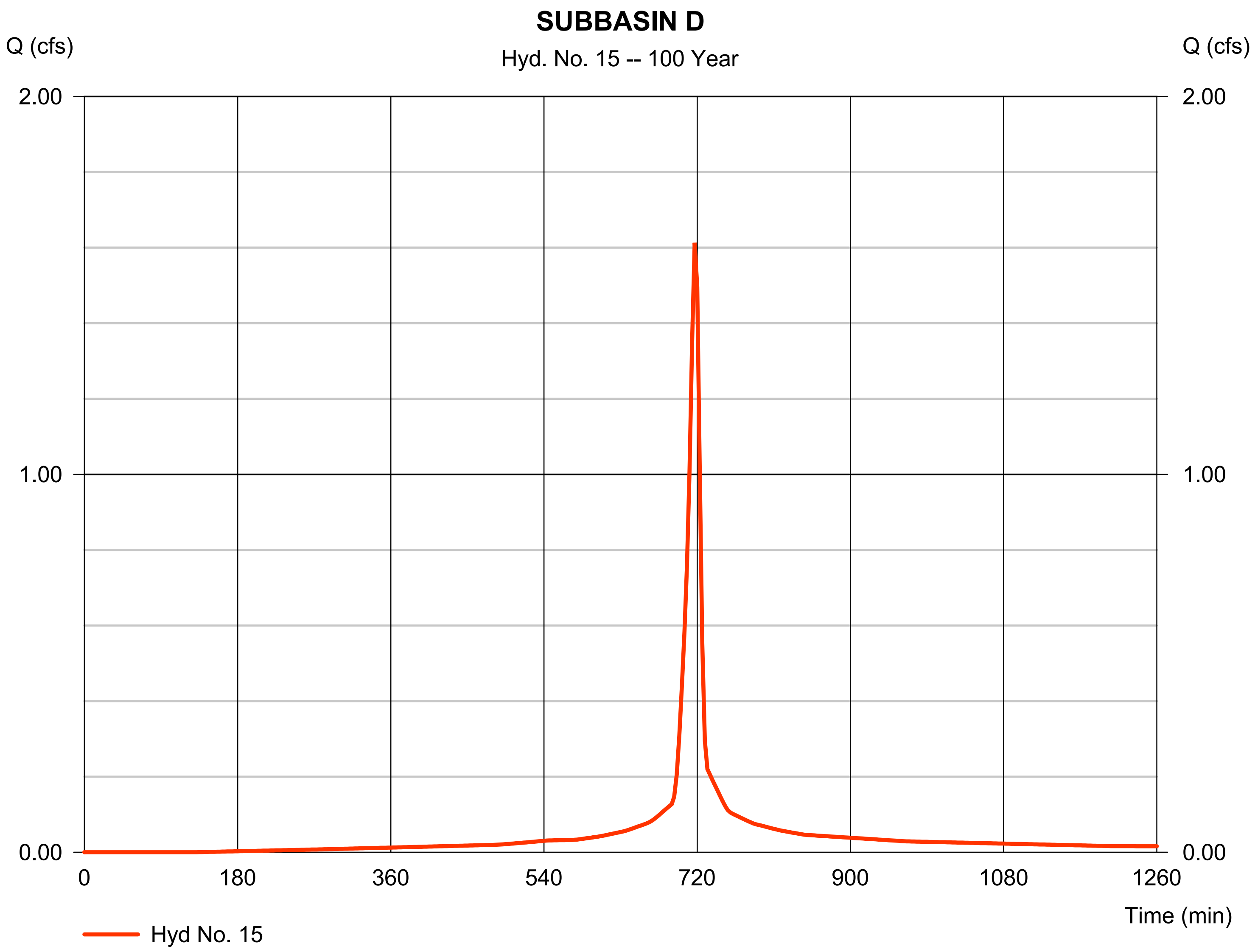
Hydrograph Report

Hyd. No. 15

SUBBASIN D

Hydrograph type	= SCS Runoff	Peak discharge	= 1.613 cfs
Storm frequency	= 100 yrs	Time to peak	= 717 min
Time interval	= 3 min	Hyd. volume	= 4,003 cuft
Drainage area	= 0.240 ac	Curve number	= 94*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 6.00 min
Total precip.	= 5.60 in	Distribution	= Type II
Storm duration	= 24 hrs	Shape factor	= 484

* Composite (Area/CN) = [(0.190 x 98) + (0.050 x 80)] / 0.240



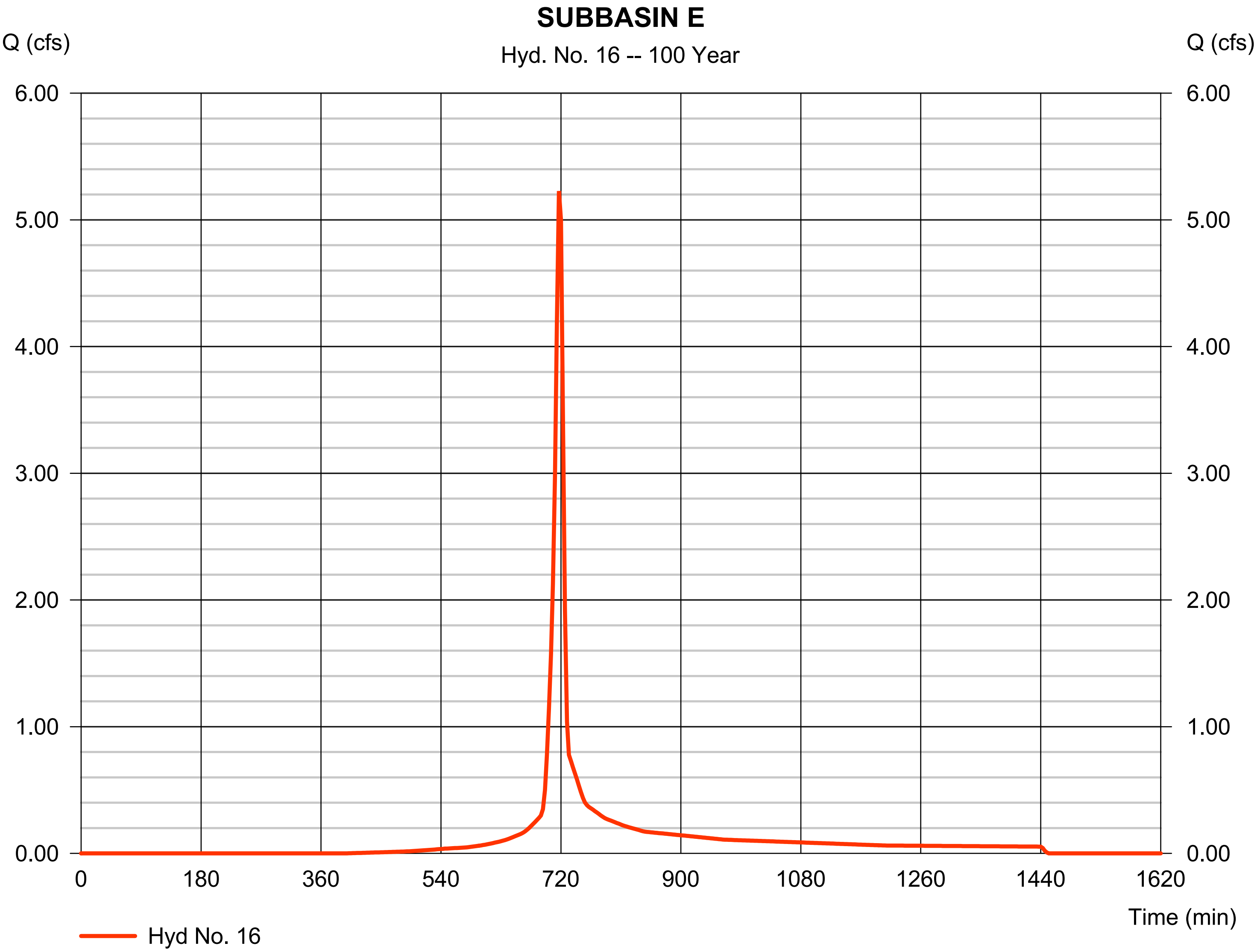
Hydrograph Report

Hyd. No. 16

SUBBASIN E

Hydrograph type	= SCS Runoff	Peak discharge	= 5.227 cfs
Storm frequency	= 100 yrs	Time to peak	= 717 min
Time interval	= 3 min	Hyd. volume	= 11,880 cuft
Drainage area	= 1.020 ac	Curve number	= 80*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 6.00 min
Total precip.	= 5.60 in	Distribution	= Type II
Storm duration	= 24 hrs	Shape factor	= 484

* Composite (Area/CN) = [(0.020 x 98) + (1.000 x 80)] / 1.020



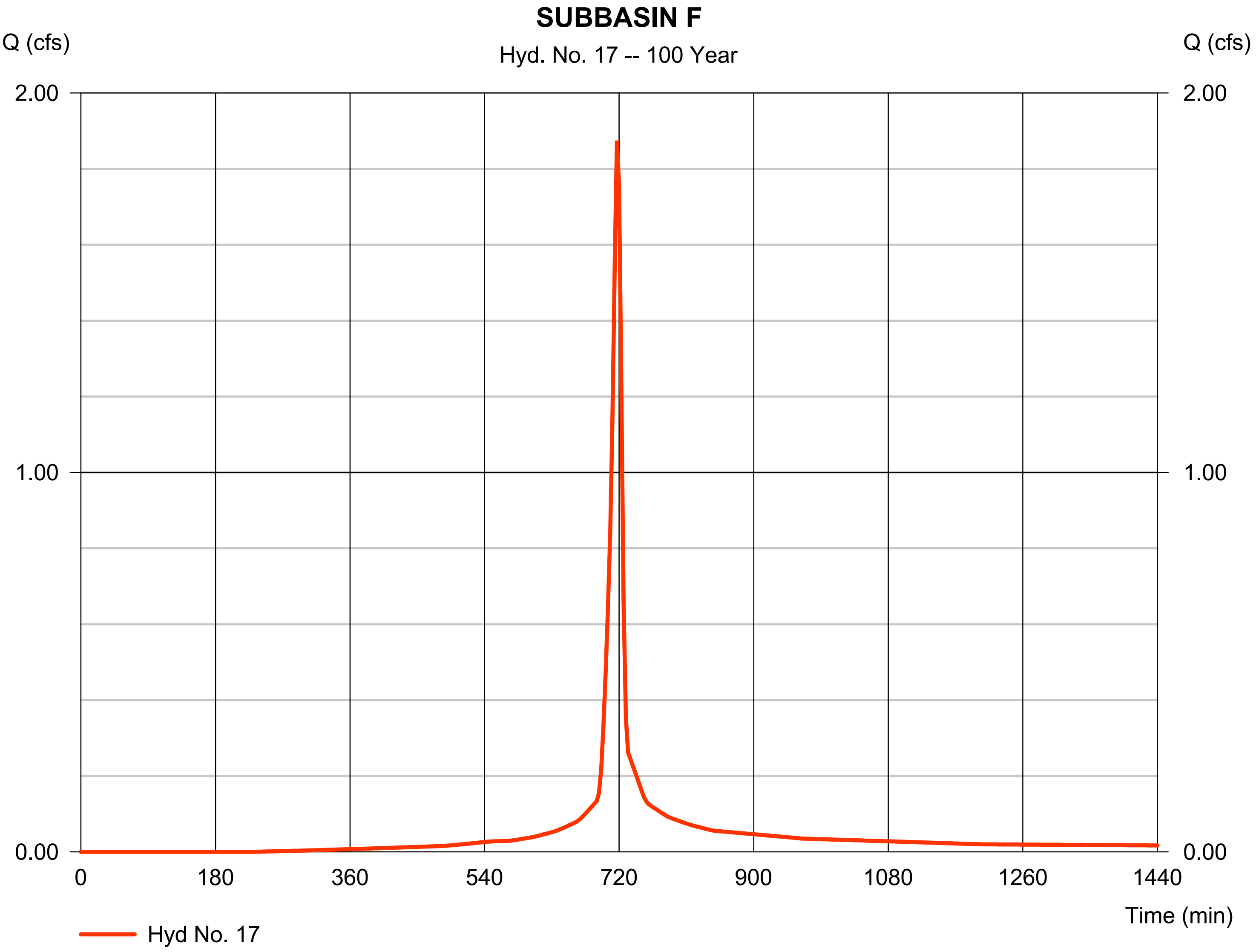
Hydrograph Report

Hyd. No. 17

SUBBASIN F

Hydrograph type	= SCS Runoff	Peak discharge	= 1.875 cfs
Storm frequency	= 100 yrs	Time to peak	= 717 min
Time interval	= 3 min	Hyd. volume	= 4,440 cuft
Drainage area	= 0.300 ac	Curve number	= 89*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 6.00 min
Total precip.	= 5.60 in	Distribution	= Type II
Storm duration	= 24 hrs	Shape factor	= 484

* Composite (Area/CN) = [(0.190 x 98) + (0.110 x 74)] / 0.300



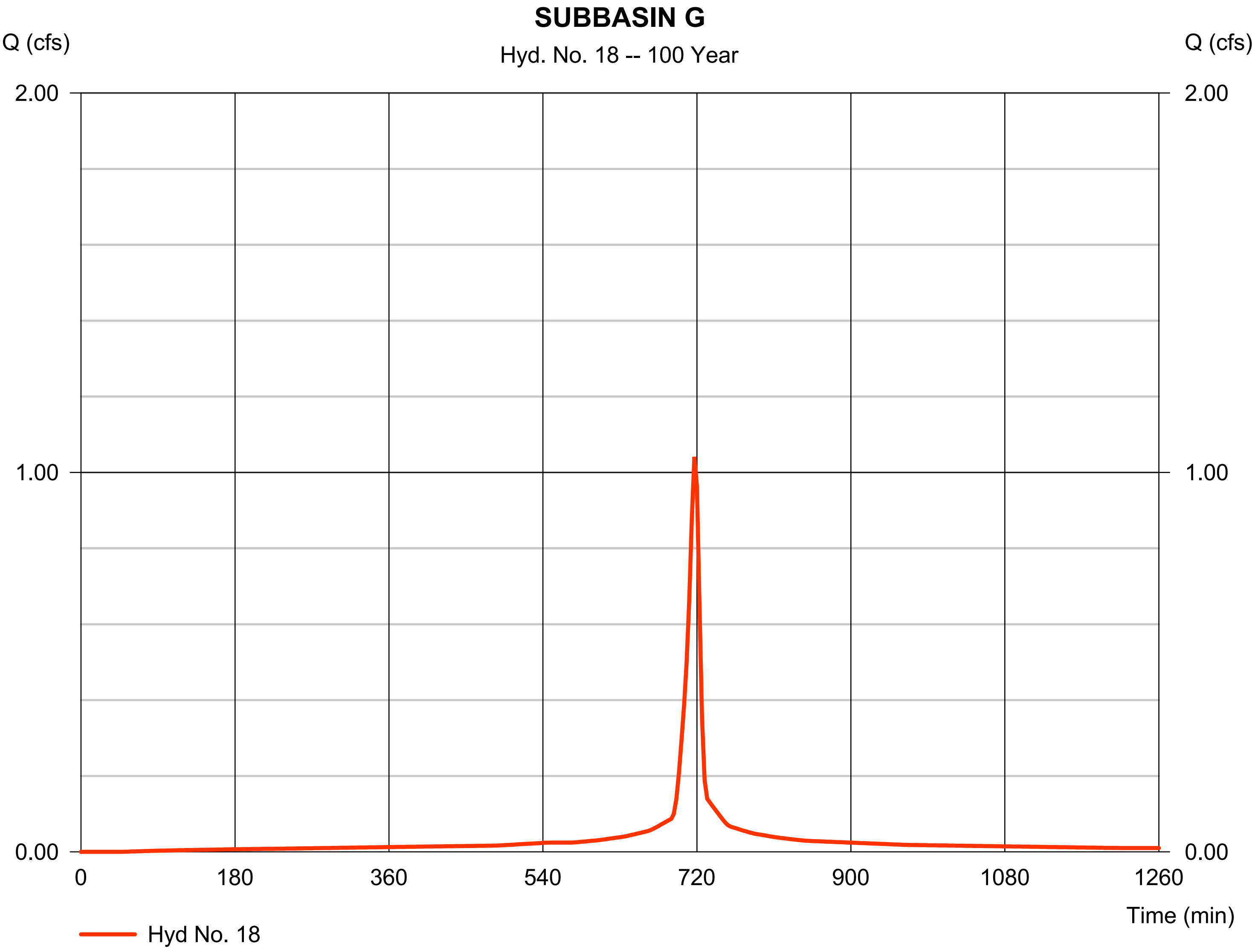
Hydrograph Report

Hyd. No. 18

SUBBASIN G

Hydrograph type	= SCS Runoff	Peak discharge	= 1.041 cfs
Storm frequency	= 100 yrs	Time to peak	= 717 min
Time interval	= 3 min	Hyd. volume	= 2,737 cuft
Drainage area	= 0.150 ac	Curve number	= 98*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 6.00 min
Total precip.	= 5.60 in	Distribution	= Type II
Storm duration	= 24 hrs	Shape factor	= 484

* Composite (Area/CN) = [(0.150 x 98)] / 0.150



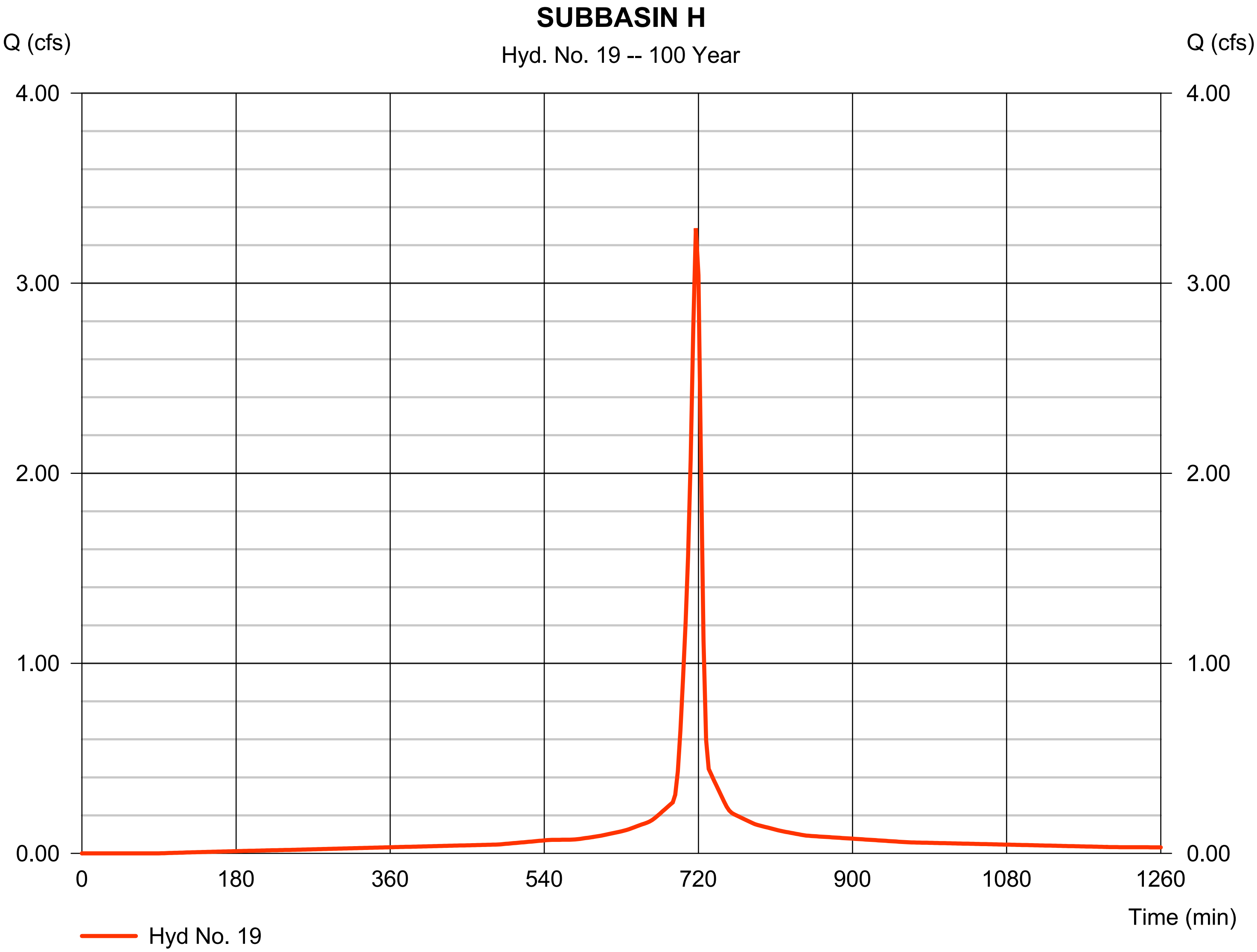
Hydrograph Report

Hyd. No. 19

SUBBASIN H

Hydrograph type	= SCS Runoff	Peak discharge	= 3.289 cfs
Storm frequency	= 100 yrs	Time to peak	= 717 min
Time interval	= 3 min	Hyd. volume	= 8,379 cuft
Drainage area	= 0.480 ac	Curve number	= 96*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 6.00 min
Total precip.	= 5.60 in	Distribution	= Type II
Storm duration	= 24 hrs	Shape factor	= 484

* Composite (Area/CN) = [(0.430 x 98) + (0.050 x 74)] / 0.480



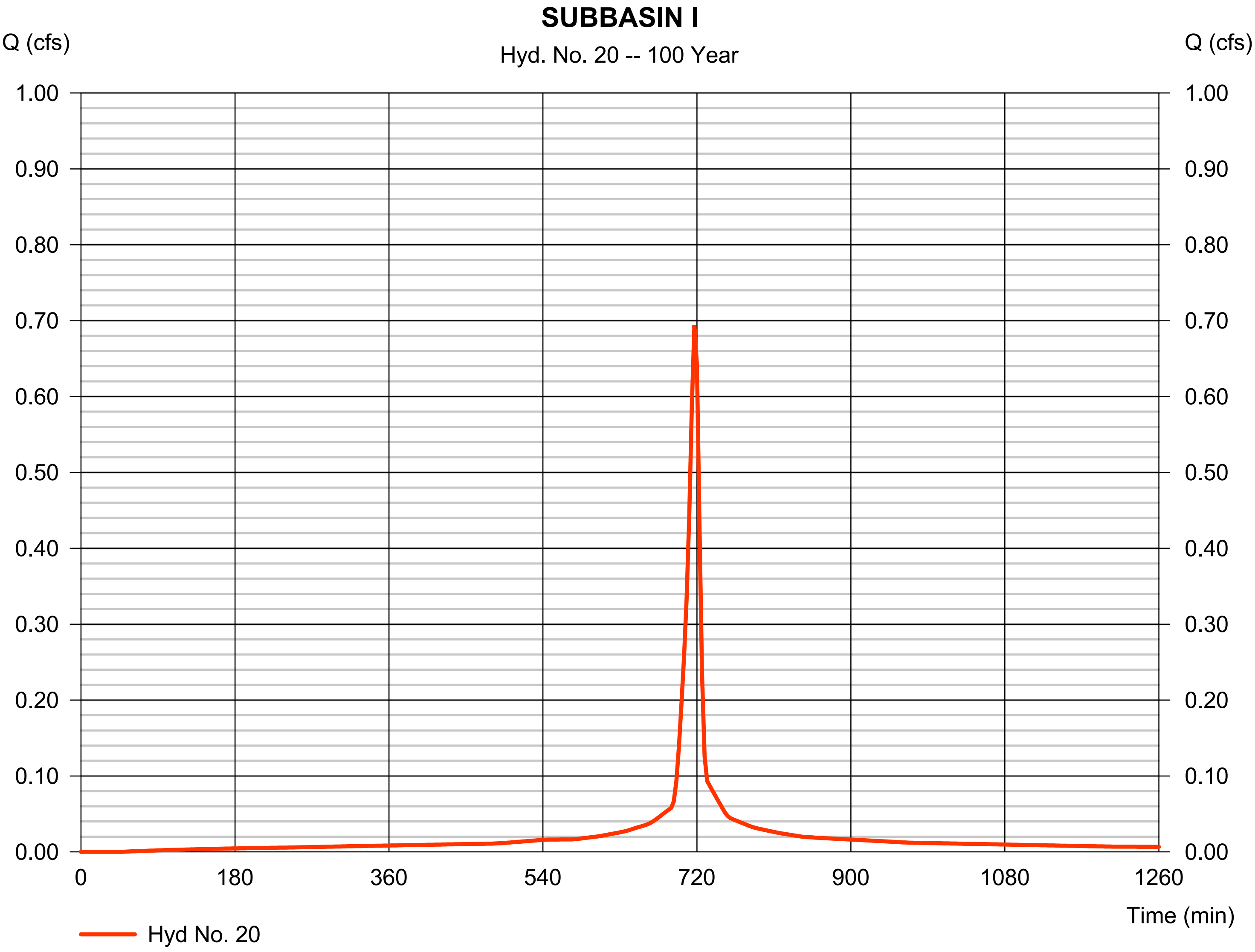
Hydrograph Report

Hyd. No. 20

SUBBASIN I

Hydrograph type	= SCS Runoff	Peak discharge	= 0.694 cfs
Storm frequency	= 100 yrs	Time to peak	= 717 min
Time interval	= 3 min	Hyd. volume	= 1,825 cuft
Drainage area	= 0.100 ac	Curve number	= 98*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 6.00 min
Total precip.	= 5.60 in	Distribution	= Type II
Storm duration	= 24 hrs	Shape factor	= 484

* Composite (Area/CN) = [(0.100 x 98)] / 0.100



Hydrograph Report

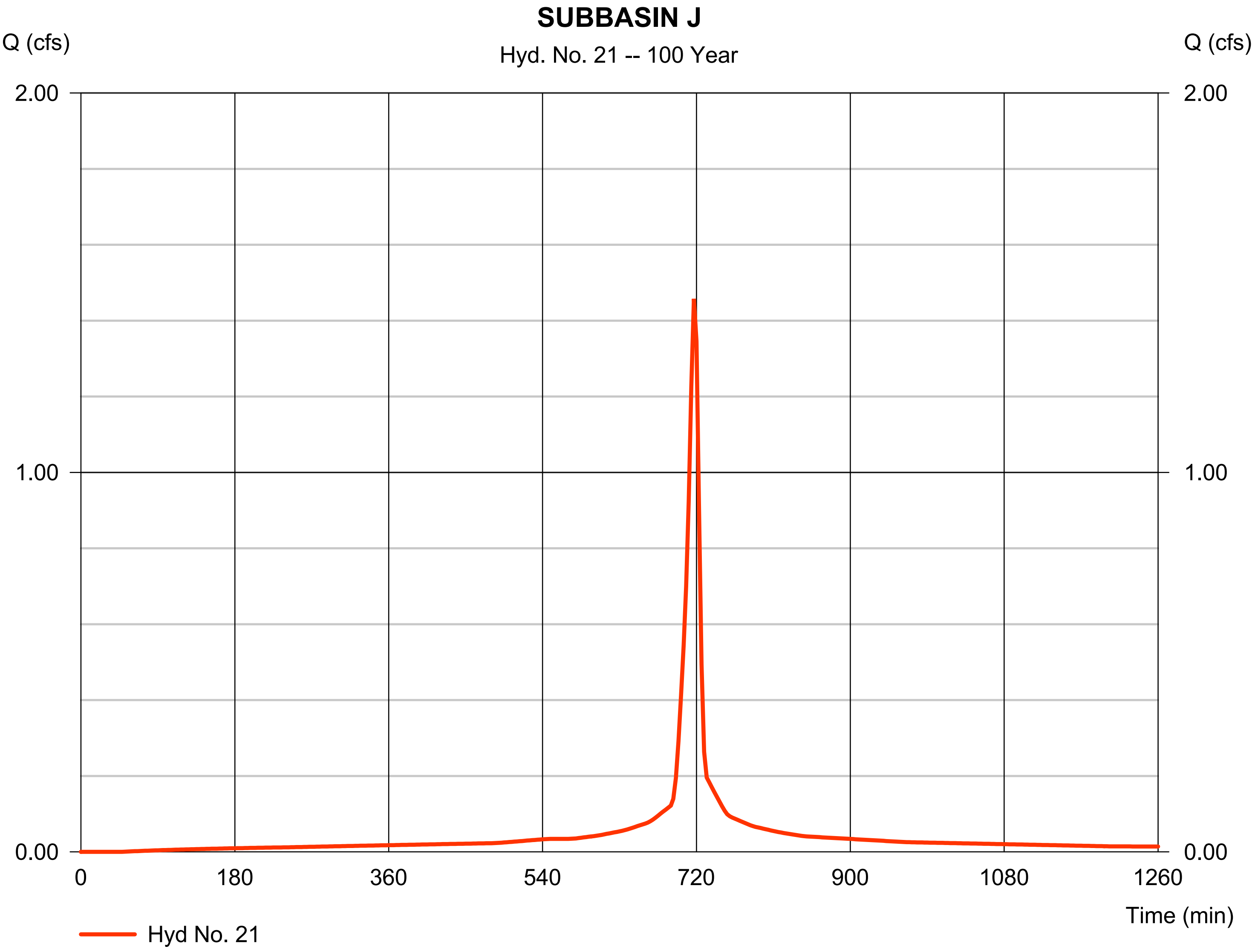
Hyd. No. 21

SUBBASIN J

Hydrograph type = SCS Runoff
Storm frequency = 100 yrs
Time interval = 3 min
Drainage area = 0.210 ac
Basin Slope = 0.0 %
Tc method = User
Total precip. = 5.60 in
Storm duration = 24 hrs

Peak discharge = 1.458 cfs
Time to peak = 717 min
Hyd. volume = 3,832 cuft
Curve number = 98*
Hydraulic length = 0 ft
Time of conc. (Tc) = 6.00 min
Distribution = Type II
Shape factor = 484

* Composite (Area/CN) = [(0.210 x 98)] / 0.210



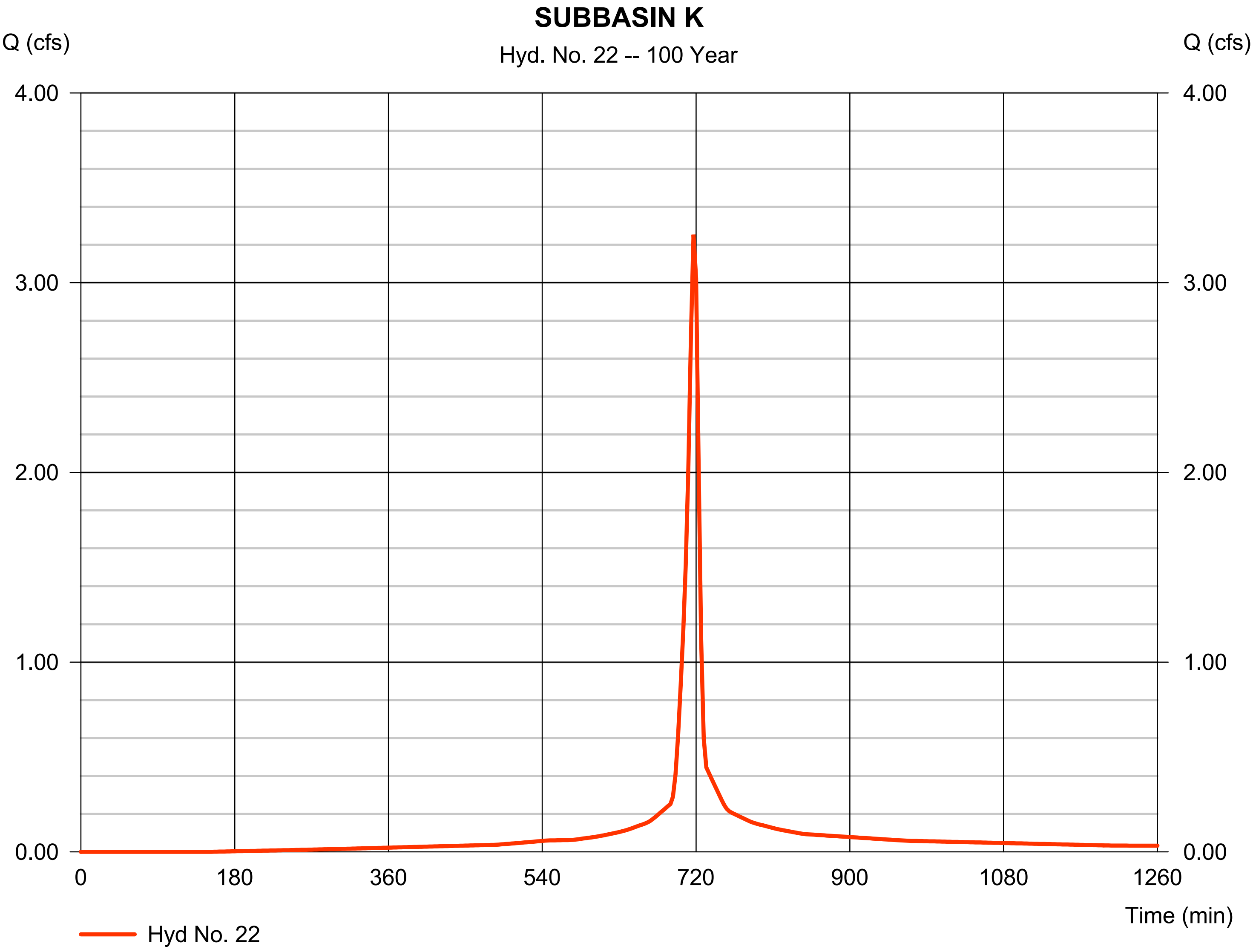
Hydrograph Report

Hyd. No. 22

SUBBASIN K

Hydrograph type	=	SCS Runoff	Peak discharge	=	3.254 cfs
Storm frequency	=	100 yrs	Time to peak	=	717 min
Time interval	=	3 min	Hyd. volume	=	7,984 cuft
Drainage area	=	0.490 ac	Curve number	=	93*
Basin Slope	=	0.0 %	Hydraulic length	=	0 ft
Tc method	=	User	Time of conc. (Tc)	=	6.00 min
Total precip.	=	5.60 in	Distribution	=	Type II
Storm duration	=	24 hrs	Shape factor	=	484

* Composite (Area/CN) = [(0.380 x 98) + (0.110 x 74)] / 0.490



Appendix F
<u>SLAMM Input / output Information:</u>

```
Data file name:  F:\Job Files\1818660 Montessori School of Waukesha-Waukesha,
WI\1818664 Civil\storm water report and calculations\slamm.mdb
WinSLAMM Version 10.3.2
Rain file name:  F:\Programs\civil\WinSLAMM\v10.3.2\Parameter Files\WisReg -
Milwaukee WI 1969.RAN
Particulate Solids Concentration file name:
F:\Programs\Civil\WinSLAMM\v10.3.2\Parameter Files\v10.1 WI_AVG01.pscx
Runoff Coefficient file name:  F:\Programs\Civil\WinSLAMM\v10.3.2\Parameter
Files\WI_SL06 Dec06.rsvx
Residential Street Delivery file name:
F:\Programs\Civil\WinSLAMM\v10.3.2\Parameter Files\WI_Res and Other Urban Dec06.std
Institutional Street Delivery file name:
F:\Programs\Civil\WinSLAMM\v10.3.2\Parameter Files\WI_Com Inst Indust Dec06.std
Commercial Street Delivery file name:  F:\Programs\Civil\WinSLAMM\v10.3.2\Parameter
Files\WI_Com Inst Indust Dec06.std
Industrial Street Delivery file name:  F:\Programs\Civil\WinSLAMM\v10.3.2\Parameter
Files\WI_Com Inst Indust Dec06.std
Other Urban Street Delivery file name:
F:\Programs\Civil\WinSLAMM\v10.3.2\Parameter Files\WI_Res and Other Urban Dec06.std
Freeway Street Delivery file name:  F:\Programs\Civil\WinSLAMM\v10.3.2\Parameter
Files\Freeway Dec06.std
Apply Street Delivery Files to Adjust the After Event Load Street Dirt Mass
Balance:  False
Pollutant Relative Concentration file name:
F:\Programs\civil\WinSLAMM\v10.3.2\Parameter Files\WI_GEO03.ppdx
Source Area PSD and Peak to Average Flow Ratio File:
F:\Programs\Civil\WinSLAMM\v10.3.2\Parameter Files\NURP Source Area PSD Files.csv
Cost Data file name:
Seed for random number generator:  -42
Study period starting date:  01/05/69       Study period ending date:  12/31/69
Start of Winter Season:  12/06              End of Winter Season:  03/28
Date:  07-05-2018                           Time:  15:19:50
Site information:

LU# 1 - Institutional:  Filter     Total area (ac):  5.720
     1 - Roofs 1:  0.570 ac.    Flat    Connected    Source Area PSD File:
C:\WinSLAMM Files\NURP.cpz
     13 - Paved Parking 1:  1.770 ac.    Connected    Source Area PSD File:
C:\WinSLAMM Files\NURP.cpz
     45 - Large Landscaped Areas 1:  3.380 ac.    Normal Clayey    Low Density
Source Area PSD File: C:\WinSLAMM Files\NURP.cpz


LU# 2 - Institutional:  Offsite     Total area (ac):  0.090
     13 - Paved Parking 1:  0.050 ac.    Connected    Source Area PSD File:
C:\WinSLAMM Files\NURP.cpz
     45 - Large Landscaped Areas 1:  0.040 ac.    Normal Clayey    Low Density
Source Area PSD File: C:\WinSLAMM Files\NURP.cpz
```

```
      Control Practice 1:  Upflo Filter CP# 1 (DS) - DS UpfloFilter # 1
         Media Type:  CPZ
         Fraction of Area Served by Upflo Filters (0-1):  1.0
         Height from Outlet Invert to Structure Top (ft):  4.0
         Sump Depth (ft):  2.00
         Sump Cleaning/Filter Replacement is not considered during the model run
         Solve for Given Conditions
         Number of filters:  15
         Upflo Filter particle size distribution file name:  Not needed -
calculated by program
```

```
SLAMM for Windows Version 10.3.2
(c) Copyright Robert Pitt and John Voorhees 2012
All Rights Reserved

Data file name:  F:\Job Files\1818660 Montessori School of Waukesha-Waukesha,
WI\1818664 Civil\storm water report and calculations\slamm.mdb
Data file description:
Rain file name:  F:\Programs\civil\WinSLAMM\v10.3.2\Parameter Files\WisReg -
Milwaukee WI 1969.RAN
Particulate Solids Concentration file name:
F:\Programs\Civil\WinSLAMM\v10.3.2\Parameter Files\v10.1 WI_AVG01.pscx
Runoff Coefficient file name:  F:\Programs\Civil\WinSLAMM\v10.3.2\Parameter
Files\WI_SL06 Dec06.rsvx
Residential Street Delivery file name:
F:\Programs\Civil\WinSLAMM\v10.3.2\Parameter Files\WI_Res and Other Urban Dec06.std
Institutional Street Delivery file name:
F:\Programs\Civil\WinSLAMM\v10.3.2\Parameter Files\WI_Com Inst Indust Dec06.std
Commercial Street Delivery file name:  F:\Programs\Civil\WinSLAMM\v10.3.2\Parameter
Files\WI_Com Inst Indust Dec06.std
Industrial Street Delivery file name:  F:\Programs\Civil\WinSLAMM\v10.3.2\Parameter
Files\WI_Com Inst Indust Dec06.std
Other Urban Street Delivery file name:
F:\Programs\Civil\WinSLAMM\v10.3.2\Parameter Files\WI_Res and Other Urban Dec06.std
Freeway Street Delivery file name:  F:\Programs\Civil\WinSLAMM\v10.3.2\Parameter
Files\Freeway Dec06.std
Pollutant Relative Concentration file name:
F:\Programs\civil\WinSLAMM\v10.3.2\Parameter Files\WI_GEO03.ppdx
Start of Winter Season:  12/06              End of Winter Season:  03/28
Model Run Start Date:  01/05/69    Model Run End Date:  12/31/69
Date of run:  07-05-2018    Time of run:  15:19:36
Total Area Modeled (acres):  5.810
Years in Model Run:  0.99

                                                      Runoff     Percent
Particulate Particulate     Percent
                                                      Volume      Runoff
Solids      Solids Particulate
                                                     (cu ft)      Volume
Conc.       Yield      Solids
                                                               Reduction
(mg/L)       (lbs)   Reduction

Total of all Land Uses without Controls:              211986          -
120.5        1595          -
Outfall Total with Controls:                          212211      -0.11%
43.27       573.2      64.06%
Annualized Total After Outfall Controls:              215158
       581.2
```