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Designing Flood Resilient Road-Stream Crossings: a Case Study from the July 2016 Northwest Wisconsin Flood

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Citation

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Abstract

The July 2016 flood in northwest Wisconsin caused the failure of over 40 road-stream crossings and damaged 300 miles of roads on the Chequamegon-Nicolet National Forest. Peak flood flow estimates were obtained at 26 road-stream crossing sites including 17 sites designed using stream simulation (SS). Peak flood magnitudes ranged from 9 to 2,629 cubic feet/second/square mile (cfsm). Floods at 21 sites were over 100 cfsm which were 1.4 to 14.4 times greater than the estimated 1 percent annual exceedance probability flood. Only 1 of 17 SS sites failed, although 13 overtopped. Infrastructure survival was attributed to the large capacity associated with SS, ability to pass debris, and the ability to pass large flows over the road, usually to one or both sides of the structure. Substantial differences in flood flow rates among the 26 sites were correlated to differences in watershed characteristics, including slope, storage, soils, and drainage density, which are related with landtype association (LTAs). Watersheds with more rapid runoff are anticipated to be more vulnerable to increasing flood flows from more-intense rainfall in the future. Using this concept, LTAs in Wisconsin were placed into four vulnerability classes as an example of how to anticipate future flood flow increases during road-stream crossing design. These classes were combined with site characteristics to illustrate how culvert and bridge design can be adapted to accommodate a changing climate. Based on our findings, we propose SS design as a primary design criterion, and suggest adding supplemental design features as watershed and site vulnerability to flooding increase. This approach has the potential to help road managers meet infrastructure and environmental objectives in the short term and long term.

Keywords: stream simulation, culvert design, flood resilience, flood vulnerability, stream restoration

Cover Photos

Upper left: Unnamed tributary to Morgan Falls Creek at Forest Trail 209 on July 20, 2016. The maximum water depth over trail was 0.6 feet on July 12, 2016. Upper right: Preemption Creek at Forest Road 377 on July 16, 2016. The maximum water depth over the road was 0.5 feet on July 12, 2016. Lower left: Mineral Lake Inlet at Forest Road 187 on July 17, 2016. Site failed on July 12, 2016. Lower right: Unnamed tributary to Hawkins Creek at Forest Road 383 on September 7, 2016. Site failed on July 12, 2016. USDA Forest Service photos by Dale Higgins.

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INTRODUCTION

Background

On July 11–12, 2016, an area of the Great Divide Ranger District of the Chequamegon-Nicolet National Forest (herein referred to as the Forest) in northwest Wisconsin received up to 9 inches of rainfall in 6 hours. Extreme flooding associated with this storm caused tremendous impacts to 300 miles of road infrastructure. Over 40 road-stream crossings failed and many more were impacted, requiring replacement construction of 3 new bridges, 14 large culverts, and many smaller culverts.

Over the 15 years preceding this flooding event, Forest personnel replaced 17 culverts using the stream simulation (SS) design approach (Cenderelli et al. 2011). The benefits included preventing structure failure; reducing maintenance; and restoring stream ecology including aquatic organism passage, channel morphology, sediment transport, and movement of woody material. Essentially, SS designs allow a stream channel through the crossing, then a culvert or bridge-type structure is built around it to ensure that stream channel dimensions are consistent above, within, and below the crossing. Of the 17 SS structures, 16 survived the 2016 flood even though 13 overtopped. Those and other road-stream crossings in the flood area provided an opportunity to evaluate the flood resiliency of SS culverts and gain valuable insights for designing future road-stream crossings that will provide a resilient transportation network in response to a changing climate.

While there is a history of post-flood studies to assist with recovery efforts and future flood prevention actions (Fitzpatrick et al. 2008, 2017; Furniss et al. 1998; Northwest Regional Planning Commission 2018; Yochum 2015), there is little literature focusing on the flood resiliency of road-stream crossing structures. This study addresses that gap by providing estimates of flood peaks that occurred at crossings, comparing those flows to flood frequency estimates at each site, providing estimates of flow through the structure and over the road where feasible, and demonstrating widely different flood responses associated with watershed characteristics. The latter has important implications for identifying the vulnerability or resilience of watersheds to increases in future flood peaks from more extreme rainfall, and for designing flood resilient road-stream crossings.

The flood resiliency of culverts, bridges and other structures at road-stream crossings is an important design consideration, given the increased frequency and magnitude of extreme floods projected by climate change scenarios. There is a growing consensus that climate change is increasing the frequency and intensity of extreme rainfall events which will lead to more extreme flooding (Easterling et al. 2017, Prein et al. 2016, U.S. Global Change Research Program 2018). Coupled with this concern are the large number of road-stream crossings on the landscape. The Forest has over 1,000 main gravel road crossings on perennial and intermittent streams (Tables 3–6 in USDA Forest Service 2004) and Wisconsin is estimated to have over 62,000 road-stream crossings (M. A. Miller, Wisconsin Department of Natural Resources, pers. comm., 2020). Starting in 1998, Forest managers pursued an ambitious program that replaced 257 road-stream crossings through 2017 for an average of just under 13 crossings per year. Nationwide, the Forest Service has over 40,000 road-stream crossings and is estimated to replace less than 1 percent of those structures each year (N. Gillespie, USDA Forest Service, pers. comm., 2023). To maintain existing stream crossing infrastructure, structures installed today will need to last 75 to 100 years based on these replacement cycles, and consequently, must be capable of surviving flood flows associated with a future climate.

Traditional Culvert Design

Traditional culvert design relies on a method known as hydrology and hydraulics (H&H). The hydrology portion of H&H involves the determination of a design flood discharge based primarily on the tolerance for risk of overtopping and economics, usually with a desire to minimize construction cost. Criteria for determining design discharges are most often associated with the annual flood series (e.g., largest instantaneous flood each year) and have typically been in the range of 1 to 10 percent annual exceedance probabilities (AEPs). These are also frequently referred to as 100-year to 10-year average recurrence intervals (RIs), respectively. Culverts on lower standard roads are typically designed to pass lower magnitude, more frequent floods; those on major roads are designed to pass higher magnitude, less frequent floods. Many road management agencies have standards or criteria that define the AEPs used, which often vary by road class. The hydraulic analysis portion of H&H utilizes site-specific characteristics of the road crossing and proposed structure to model the water surface elevations associated with each modeled discharge, with a focus on the headwater elevation at the inlet. Agencies typically define the allowable headwater at the inlet of the structure as top of road or a headwater-to-depth ratio (HW/D) (i.e., inlet depth of water divided by depth or height of culvert). For an HW/D = 1, the headwater depth would be at the top of the culvert at the inlet. The smallest culvert that will meet the allowable HW/D ratio for the design discharge is selected, which tends to minimize construction costs. This approach also allows the use of multiple culverts within the channel.

Traditional H&H culvert design has numerous ecological and infrastructure effects since it typically results in a constricted channel from structure widths much less than bankfull channel width. Many natural resource professionals describe this method as squeezing as much “clear water” through the pipe as possible. Clear water is referenced because the method typically does not account for the transport of sediment, woody material or ice, or the movement of aquatic organisms. Traditional H&H culvert design does not include geomorphic considerations of the natural channel such as bankfull width (Leopold et al. 1964), channel gradient, scour depth, and sediment transport. These factors often cause an elevated headwater, especially during frequent floods. The elevated headwater results in upstream deposition of sediments, high water velocities in the structure, culvert abrasion, and downstream scour. These factors increase risk of failure, disrupt aquatic organism passage, and result in high maintenance from debris plugging. Structures narrower than bankfull width are also more prone to plugging from beaver activity. Debris plugging is exacerbated where multiple culverts occur at one crossing.

While progress has been made in recent years to incorporate more geomorphic and ecological standards into culvert design (Bates 2003; Kilgore et al. 2010; USDA Forest Service 2008, 2014; USDA Natural Resources Conservation Service [USDA NRCS] 2011; Wisconsin Department of Transportation [WisDOT] 2017), many road management agencies still utilize the traditional H&H approach. For example, the NRCS lists minimum design storm frequencies (i.e., average RIs) for forest access roads, farm driveways, and public campgrounds as 2-year, 10-year, and 25-year average RIs, respectively (Table 1 in USDA NRCS 2010). These standards apply to all culverts and bridges installed in a natural drainageway and the structure must pass the design storm runoff without causing erosion or road overtopping. For low-standard roads, which include sites covered in this analysis, the Federal Highway Administration specifies a 25-year flood average RI (i.e., 4 percent AEP) with a HW/D <1.2 for culverts larger than 48 inches, and HW/D <1.5 for smaller culverts (Federal Highway Administration 2018)). WisDOT allows an HW/D = 1.5 (WisDOT 2017) and specifies use of a 25-year average RI design frequency for structures in main and primary channels on rural county and state trunk highways with average daily traffic less than 4,000 and 1,500 vehicles, respectively (WisDOT 1997).

Stream Simulation Culvert Design

In comparison, stream simulation (SS) culvert design integrates the sciences of fluvial geomorphology, hydrology, civil engineering, and biology to create a natural, dynamic stream channel through a culvert or bridge that passes flood flows, sediment, wood, and aquatic organisms (Cenderelli et al. 2011). The basic principle is to design the channel based on a nearby reference reach, then build a structure around it. The structure may have a full invert or an open bottom. Key requirements of SS design are:

1. sizing a single span structure wider than or equal to bankfull channel width
2. constructing or developing a natural streambed and banks
3. creating a stream profile that maintains or restores the natural slope and bedforms
4. performing scour analysis based on stream geomorphology
5. aligning the structure with the stream, especially at the inlet
6. checking hydraulic capacity to ensure a HW/D <0.8 at the 1 percent AEP (i.e., 100-year average RI)
7. verifying that stream banks and key pieces composed of large rock are stable while the bed is mobile at the appropriate flows
8. and ensuring that streamflow stays within the channel and associated floodplain if the structure is overtopped, rather than diverting down the road.

Although SS was originally developed to provide passage of all species and life stages of aquatic organisms by incorporating natural geomorphic processes at road-stream crossings, it also provides substantial infrastructure benefits by reducing maintenance and extending culvert lifespan in the following ways.

- Hydraulic capacity: Structures greater than bankfull channel width are extremely flood resilient and often pass the 1 percent AEP flood with a HW/D substantially less than the design requirement of 0.8, thereby greatly reducing failures and increasing lifespan.

- Stream continuity: SS maintains natural sediment transport, thereby maintaining inlet capacity. Traditional H&H culvert design frequently constricts the channel, which can cause sediment to aggrade upstream and reduce inlet capacity. Constricting the channel can also produce a large plunge pool that undermines the outlet of the culvert.

- Debris passage: SS minimizes structure plugging with wood, which is a common failure mechanism at stream crossings in forested landscapes (Furniss et al. 1998). Large wood transported during floods is either less than bankfull width or moving parallel to flow (Flanagan 2004). Aligning the inlet with the stream allows wood longer than bankfull width to enter the structure without turning and catching. HW/D <0.8 allows freeboard for that wood to move through the structure. Most wood in transport is shorter than bankfull width so a structure greater than bankfull channel width allows such wood to move through it (Flanagan 2004). Only a single structure is allowed to span the bankfull channel, rather than the potential of multiple smaller structures allowed under H&H, which would increase this hazard. Sediment and earthen debris flows can also be a major cause of crossing failures. SS structures are more likely to pass such debris and are somewhat less vulnerable to such events.

- Abrasion protection: Many traditionally designed culverts have a short lifespan because sediment transport causes abrasion and corrosion along the invert. SS culverts are set at the appropriate slope, with the invert sufficiently below the thalweg of the stream to account for scour over time and with a natural stream bed and banks that protect the invert from abrasion.

- Reduced hydraulic forces: If the SS structure experiences an extreme flood and overtops, it is more likely to survive, especially with low road fills, because the tailwater rises in concert with the headwater up to the top of the structure. This process will reduce the difference in water surfaces from upstream to downstream, which can reduce water velocity and erosional forces.

- Diversion potential: Water that overtops a road crossing and diverts down the road can cause tremendous erosion. SS design provides a much larger structure that minimizes the potential for diversion and includes steps to ensure that diversion does not occur during an extreme flood that overtops the road. These factors reduce road maintenance and lifespan costs at the crossing.

Costs

To provide these benefits, however, SS culverts are typically much larger than the existing culverts they replace, resulting in higher initial costs that can be a key concern for road management agencies. Very few studies have compared installation costs for new SS structures to the estimated costs of culverts replaced-in-kind. This type of comparison can help managers anticipate and account for the costs and benefits of SS design on road infrastructure, especially during changing conditions, as part of an overall ecological and economic assessment.

Objectives

During the early response to the July 11–12, 2016 flood, high-water marks were surveyed at the 17 SS sites and 9 additional road-stream crossings, and then used to estimate the peak flood flow that occurred at each crossing (Table 1). The flood flow estimates quantified the extreme nature of the flood, characterized how flood magnitudes varied across the flood area due to variability in watershed characteristics, and for the 17 SS sites, allowed their design to be evaluated against the 2016 flood flows.

The objectives of this study were to (1) estimate the peak flood flows from the July 2016 storm, (2) assess factors that affected the survival or failure of road-stream crossings to the flood, (3) determine factors that influenced the magnitude of peak flows in different watersheds, (4) compare the infrastructure costs and benefits of SS structures to their predecessors, and (5) provide new perspectives on designing road-stream crossings to improve their flood resiliency. We provide estimates of flood peaks at each crossing, compare those flows to flood frequency estimates at each site, provide estimates of flow through the structure and over the road where feasible, and describe contrasting flood responses associated with watershed characteristics. The latter has important implications for identifying the vulnerability or resilience of watersheds to increases in future flood peaks from more extreme rainfall, and for designing flood resilient road-stream crossings.

Table 1.1.—Peak flow estimates and other site data for 26 road-stream crossings, Chequamegon-Nicolet National Forest, July 2016 flood

Site #	Stream	Road or trail	Site ID	Design method	Rainfall (in)	Drainage area (sq mi)	Peak flood flow (cfsm)	0.2% Annual exceedance probability (AEP) peak flow (cfsm)	1% AEP peak flow (cfsm)	Peak flood flow (cfs)	0.2% AEP peak flow (cfs)	1% AEP peak flow (cfs)
1	Unnamed tributary to (UNT) Morgan	FR 199	21731	Other	8.98	0.51	2,629	420	290	1,341	214	148
2	20 Mi	FR 378	21613	SS	8.26	0.78	1,712	163	119	1,335	127	93
3	UNT Hawkins	FR 383B	21776	SS	8.80	0.94	1,223	383	299	1,150	360	281
4	UNT Whisky, East	FR 198	21711	Other	9.20	1.03	1,103	146	107	1,136	150	110
5	Hawkins	FR 383	21792	SS	8.94	3.37	1,019	212	150	3,435	713	506
6	UNT Whisky, West	FR 198	21709	SS	9.10	0.21	762	190	143	160	40	30
7	UNT Morgan Falls	FT 209	21770	SS	9.02	0.29	759	397	300	220	115	87
8	UNT Whisky, Middle	FR 198	21710	SS	9.20	0.65	692	142	106	450	92	69
9	20 Mi	FR 202	21622	SS	8.27	0.31	677	190	139	210	59	43
10	UNT Marengo	FR 377	21636	SS	8.55	0.21	657	595	405	138	125	85
11	UNT Trout, East	FR 390	21841	SS	8.02	0.61	598	192	141	365	117	86
12	UNT Marengo, South	FR 194	22419	SS	8.11	0.37	581	197	146	215	73	54
13	McCarthy	FR 184	218MC3	SS	7.64	1.35	348	115	86	470	155	116
14	Preemption	FR 377	21607	SS	7.39	2.67	262	164	118	700	437	315
15	UNT Marengo, North	FR 194	22409	SS	8.11	1.43	245	131	97	350	188	138
16	Trout	FR 390	21845	SS	8.08	5.09	221	126	92	1,125	640	466
17	Mineral Lake Inlet	FR 187	21814	Other	8.15	2.09	215	104	78	450	217	162
18	UNT Brunsweiler	FR 387	21801	Other	8.32	1.28	195	82	59	250	105	75
19	UNT Marengo	FR 196	22412	Other	8.22	0.69	145	136	101	99	93	69
20	18 Mi	FR 371	215181	Other	7.83	20.47	125	61	45	2,550	1,240	929
21	UNT WF Chippewa	FR 191	22407	SS	7.16	2.16	102	62	48	220	133	103
22	Brush	FR 354	226BC2	SS	5.38	6.84	60	47	36	410	319	246
23	Porcupine	FR 213	215187	Other	8.01	5.83	60	65	49	350	377	286
24	18 Mi	FR 213	215182	SS	8.00	10.4	27	22	17	280	223	171
25	WF Chippewa	FR 191	22406	Other	7.48	11.5	25	45	35	290	517	399
26	Red Ike	FR 208	22401	Other	6.57	5.02	9	40	32	45	203	160

Notes: Horizontal dashed lines delineate groups of similar peak flow response; 0.2 percent and 1 percent flood estimates were derived from regression equations for Wisconsin watersheds (Walker and Krug 2003) using drainage area, storage, main channel slope, soil permeability, and snow.

AEP = annual exceedance probability, RI = recurrence interval, BFW = bankfull width, SS = stream simulation.

Table 1.2.—Peak flow estimates and other site data for 26 road-stream crossings, Chequamegon-Nicolet National Forest, July 2016 flood

Site #	Stream	Ratio of 2016 peak flood flow to 0.2% annual exceedance probability (AEP) peak flow	Ratio of 2016 flood peak flow to 1% AEP peak flow	Flood impact: Site overtopped?	Flood impact: result	Storage (%)	Main channel slope (ft/mi)	Mean annual snowfall (in)
1	Unnamed tributary to (UNT) Morgan	6.3	9.1	Yes	Failed	3.5	218	80
2	20 Mi	10.5	14.4	Yes	Survived	12.6	56	80
3	UNT Hawkins	3.2	4.1	Yes	Survived	1.3	172	80
4	UNT Whisky, East	7.6	10.3	Yes	Failed	17.2	58	80
5	Hawkins	4.8	6.8	Yes	Failed	6.2	109	80
6	UNT Whisky, West	4.0	5.3	Yes	Survived	14.7	59	80
7	UNT Morgan Falls	1.9	2.5	Yes	Survived	22.4	36	80
8	UNT Whisky, Middle	4.9	6.5	Yes	Survived	23.7	59	80
9	20 Mi	3.6	4.9	Yes	Survived	17.1	75	80
10	UNT Marengo	1.1	1.6	No	Survived	0.4	247	70
11	UNT Trout, East	3.1	4.2	Yes	Survived	10.0	57	100
12	UNT Marengo, South	2.9	4.0	Yes	Survived	9.2	57	80
13	McCarthy	3.0	4.1	Yes	Survived	23.0	38	100
14	Preemption	1.6	2.2	Yes	Survived	11.6	85	75
15	UNT Marengo, North	1.9	2.5	Yes	Survived	21.8	60	80
16	Trout	1.8	2.4	Yes	Survived	26.0	82	100
17	Mineral Lake Inlet	2.1	2.8	Yes	Failed	18.4	34	80
18	UNT Brunsweiler	2.4	3.3	Yes	Survived	43.0	31	75
19	UNT Marengo	1.1	1.4	No	Survived	24.1	55	80
20	18 Mi	2.1	2.7	Yes	Failed	24.2	25	75
21	UNT WF Chippewa	1.7	2.1	Yes	Survived	34.0	15	75
22	Brush	1.3	1.7	No	Survived	50.0	16	70
23	Porcupine	0.9	1.2	No	Survived	32.1	23	75
24	18 Mi	1.3	1.6	No	Survived	23.9	15	75
25	WF Chippewa	0.6	0.7	No	Survived	36.6	13	75
26	Red Ike	0.2	0.3	No	Survived	35.6	8	75

Notes: Horizontal dashed lines delineate groups of similar peak flow response. "Site overtopped" signifies flow over the road.

DESCRIPTION OF THE AREA

The portion of the Forest that received more than 5.0 inches of rainfall during the July 2016 storm event covered 480 square miles and included parts of the Great Divide and Washburn Ranger Districts (Fig. 1). The area includes portions of the Wisconsin counties of Bayfield, Ashland, and Sawyer.

The area has a humid continental climate with warm summers and cold winters. Average low and high temperatures are 1 degree and 22° F in January, and 55 and 80° F in July. Precipitation averages 32 inches/year with 10 inches falling from October through March and 22 inches from April through September. Precipitation frequency estimates for the area for 6-hour and 24-hour durations were 1.68 inches and 2.38 inches, respectively, for a 100 percent AEP (i.e., 1-year average RI); and 5.16 inches and 7.43 inches, respectively, for a 1 percent AEP (National Oceanic and Atmospheric Administration [NOAA] 2017). From 1981 to 2010, the area has experienced 5 days/year with more than 1 inch of rainfall, but that total is predicted to increase to 8 days/year by 2041 to 2060 (Wisconsin Initiative on Climate Change Impacts 2021a). The area also experienced 1–2 days/100 years with rainfall greater than 5 inches, which is expected to increase to 3–4 days/100 years in the next 20 to 40 years (Wisconsin Initiative on Climate Change Impacts 2021b).

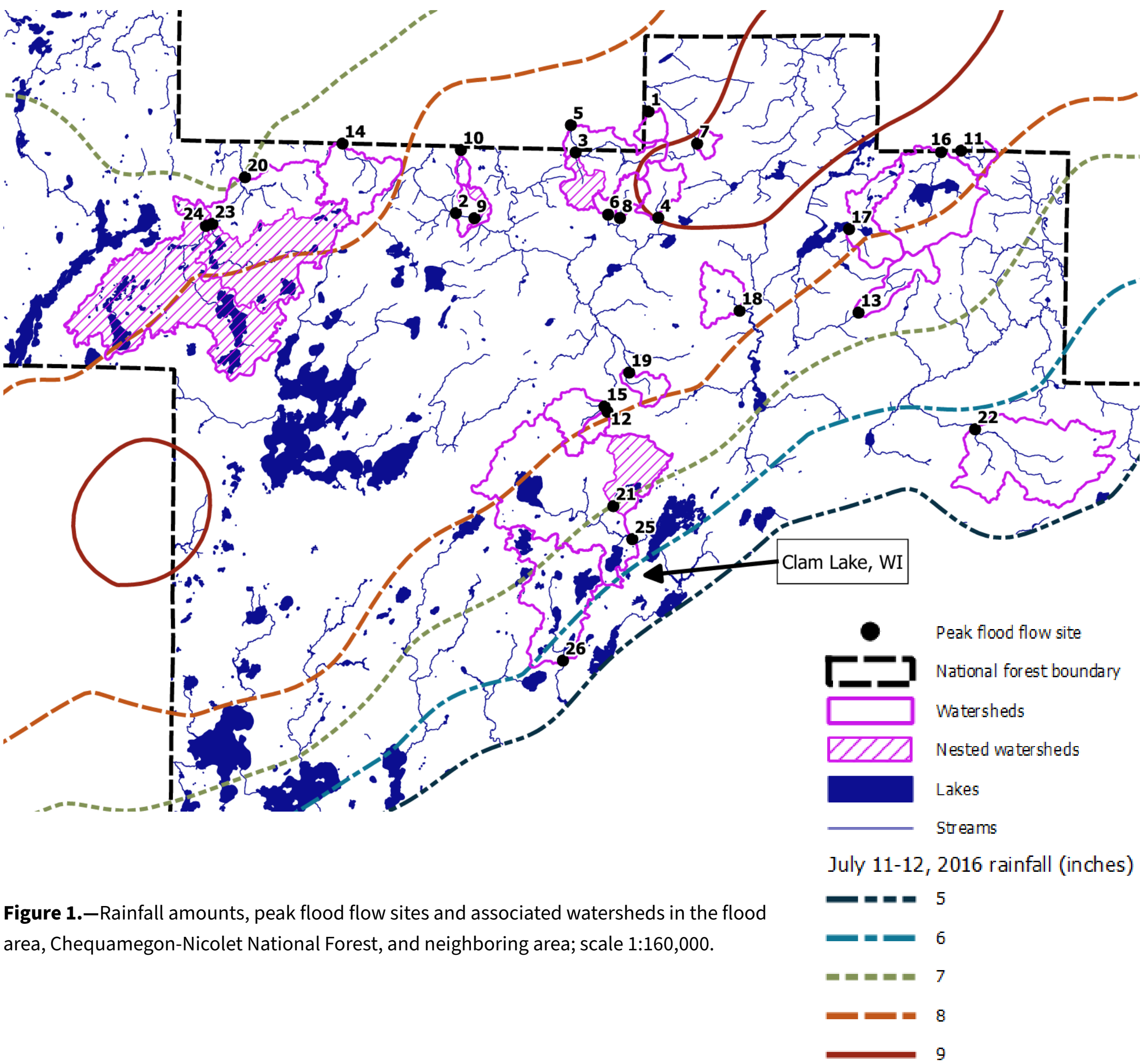

Figure 1.—Rainfall amounts, peak flood flow sites and associated watersheds in the flood area, Chequamegon-Nicolet National Forest, and neighboring area; scale 1:160,000.

Cumulative snowfall ranges from 60 to 100 inches per year. Average annual runoff is about 12 inches. Depending on the year, the annual floods in northern Wisconsin can be produced by snowmelt or by rainfall.

The geology of northern Wisconsin consists of glacial deposits from the Pleistocene era underlain by Precambrian rocks. The glacial deposits typically range in depth from 50 to 100 feet within the flood area except for the Penokee Range, where bedrock can occur at the surface or just a few feet below (Clayton 1984). The most recent glacial advance reached its maximum extent well south of the area about 30,000 years ago. It retreated and readvanced several times until finally leaving Wisconsin about 11,000 years ago (Wisconsin Geological and Natural History Survey 2011). The flood area is located within the Copper Falls Formation, which contains deposits of glacial till and meltwater-stream sediment. Wetlands also contain postglacial deposits of organic sediment (Clayton 1984).

Landtype associations (LTAs) are ecological units based on geomorphic process, general topography, geology, surficial geology, soil families, local climate, and potential natural vegetation communities (Cleland et al. 1997). They are mapped at a scale of 1:60,000 to 1:250,000 and typically are thousands to tens of thousands of acres in size. LTAs are a useful land unit for evaluating hydrologic processes because factors used in their delineation correlate closely with the routing of water through a landscape, especially the factors of topographic relief, soil permeability and storage. These factors are also integrated into the U.S. Geological Survey (USGS) flood frequency prediction equations.

Six LTAs occur within the flood area (Fig. 2). The Penokee/Gogebic Iron Range (PG) is a hilly bedrock-controlled moraine with predominantly sandy loam soils over sandy loam till or bedrock. While depth to bedrock can range up to 50 feet, this LTA contains the shallowest soils in the area and bedrock outcrops are common. The Valhalla/Marenisco Moraines (VMM) consist of rolling collapsed moraine with predominantly fine sandy loam over sandy loam till or outwash. Chequamegon Washed Till and Outwash (CWTO) is a complex of rolling collapsed moraine and outwash plain with predominantly fine sandy loam over loamy sand till. Glidden Drumlins (GD) contain outwash plains and swamps between drumlins with predominantly fine sandy loam over loamy sand till. Cable Rolling Outwash (CRO) consists of collapsed and uncollapsed outwash with loamy sand over outwash. Telemark Washed End Moraine (TWEM) consists of hilly collapsed moraine with sandy loam over outwash or loamy sand till. Only one study watershed contains a small area of TWEM.

Elevation ranges from 1,050 to 1,800 feet above mean sea level, but local relief seldom exceeds 400 feet. The greatest relief and steepest slopes occur in the PG and VMM LTAs where slopes up to 35 percent exist and slopes of 5 to 15 percent are more common. Other LTAs typically have slopes of less than 5 percent.

Flooding from the 2016 event occurred on both sides of the Continental Divide. The predominant 10-digit hydrologic units affected included the Marengo River (0401030204), White River (0401030206), and Headwaters Bad River (0401030203), which flow north to Lake Superior; and the West Fork Chippewa River (0705000101) and Upper Namekagon River (0703000201), which flow south to the Mississippi River.

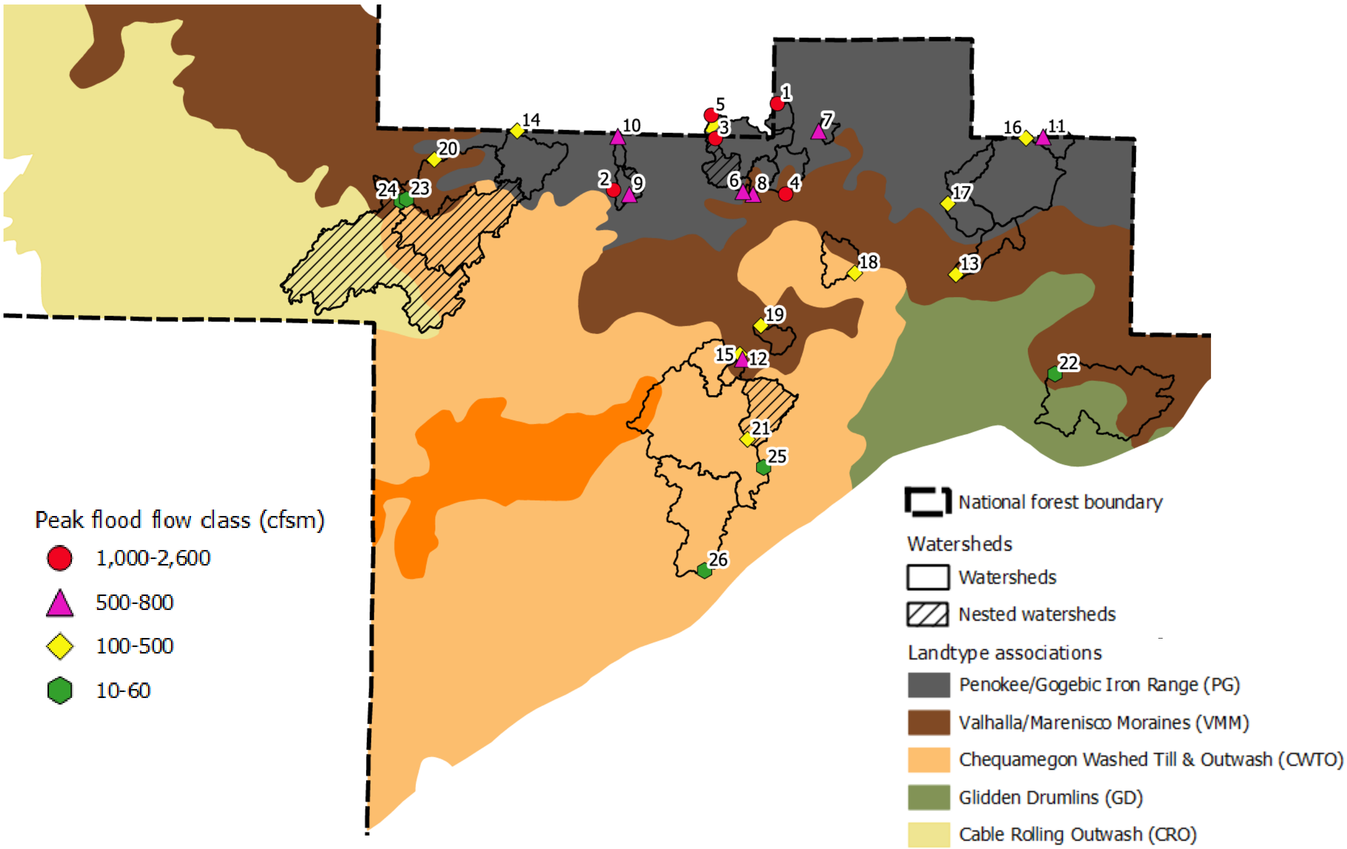

Figure 2.—Landtype associations, and peak flood flow classes for each site in cubic feet per second per square mile (cfsm), Chequamegon-Nicolet National Forest; scale 1:200,000. Numbers highlighted in white indicate individual site numbers.

Given the climate and glacial topography, the 2016 flood area has an abundance of water resources. Lakes and wetlands are common and have the effect of attenuating flood peaks through the storage they provide. Wetland occupies 20 percent of the area but varies from 3.5 to 39 percent depending on LTA. Lakes and ponds cover another 5.3 percent but vary from a few percent to over 10 percent depending on LTA. The density of streams reflects how rapidly runoff drains from the landscape. The area contains 374 miles of stream mapped at a scale of 1:24,000 for an average drainage density of 0.78 mile/square mile. But drainage density is highly variable depending on landform and ranges from 0.0 mile/square mile in highly porous areas such as TWEM to over 1.5 miles/square mile where drainage is more rapid such as PG.

The morphological characteristics of streams in the flood area are typical of northern Wisconsin where most streams have slight entrenchment, entrenchment being the ratio of flood-prone width divided by bankfull width (Rosgen 1994). They also exhibit low gradient, low to moderate width-to-depth ratios , low to moderate sinuosity, and sand or gravel channel materials (Savery et al. 2001). Compared to upland areas, streams located in wetlands are slightly entrenched and have mostly lower gradients, low width-to-depth ratios, and sand channel materials. Wetland stream channels are Rosgen Level I, C, and E types occurring in roughly equal proportion (Rosgen 1994). Upland entrenchments tend to be lower than in wetlands on average and include the range from entrenched to slightly entrenched. Stream channels in uplands also tend to have higher gradients and more gravel and cobble as the dominant channel materials. Rosgen Level I stream types occurring in uplands, listed in their approximate frequency of occurrence, include C, B, E, and F types. The PG and VMM LTAs tend to have more upland stream channels than other LTAs in the flood area.

The area is mostly forested with a mixture of species. Northern hardwoods and aspen (*Populus tremuloides)* occur throughout the area, especially on finer soils, along with eastern white pine (*Pinus strobus*), hemlock (*Tsuga*), and balsam fir (*Abies balsamea*). Red and jack pine (*Pinus resinosa* and *Pinus banksiana*) are more common on sandier soils. Forested wetlands contain either black ash (*Fraxinus nigra*) and red maple (*Acer rubrum*) or black spruce (*Picea mariana*) and tamarack (*Larix laricina*). Nonforested wetlands are composed mostly of alder (*Alnus*) and other shrubs, sedge meadows, or open bogs.

METHODS

Rainfall

Rainfall amounts from the July 11–12, 2016 storm were determined from precipitation data made available by the National Weather Service in a downloadable geographic information system (GIS) matrix (NOAA 2017). The matrix consisted of hourly precipitation amounts at a grid spacing of 4,400 meters (2.734 miles) based on a combination of multisensor inputs from radar, gauge, and satellites. Rainfall values from the 6-hour period with highest amounts of rainfall were summed to obtain total storm rainfall. These data were analyzed in GIS to develop 1.0-inch isohyets. Thiessen polygons were generated for the rainfall points, and used to estimate total storm rainfall for the watershed at each site where peak flow estimates were made from high water marks.

The rainfall amount for a 6-hour duration 1 percent AEP for the flood area was determined using the NOAA Atlas 14 precipitation frequency data server found at: https://hdsc.nws.noaa.gov/pfds/pfds_map_cont.html. The selected representative point was at latitude 46.3066 and longitude -90.9137.

Culvert Design Methods

We assessed design for 26 road-stream crossings where 2016 flood peaks were measured. The 17 crossings designed using SS methods were installed from 2001 through 2016. Since 12 of these streams had very low gradients and were predominantly sand beds, stream banks were not constructed through the culvert—the culvert sides act as banks for the sand bed channel in the culvert. Three of the 17 SS culverts were not quite bankfull width and 8 did not meet the strict criterion for HW/D <0.8 at the 1 percent AEP. However, these sites were considered SS designed because structure width was only 5 to 16 percent less than bankfull width, and seven of the eight sites had HW/D of 0.88 to 1.08; the other was 1.30. In this instance, bankfull width is the more critical criterion.

The design criteria are unknown for six of the other nine non-SS sites. Two of these nine were designed more than 30 years ago. These sites, unnamed tributary to (UNT) Brunsweiler at Forest Road (FR) 387 (site 18) and WF Chippewa FR 191 (site 25), were likely designed to pass the 1 percent AEP with available head using the runoff curve number method for estimating flood hydrology. One of the other nine sites, UNT Whisky E at FR 198 (site 4), was constructed in 2005 and designed to pass the 1 percent AEP with HW/D <1.0, but culvert width was only 60 percent of bankfull width.

Peak Flood Flow Estimates

The 26 peak flood flows associated with the July 2016 flood were determined from high-water marks measured or flagged between July 12 and September 1, 2016. In some cases, actual surveys were performed later; however, all surveys were completed by October 31, 2016. High-water marks were identified mostly from debris, seed, and mud lines (Koening et al. 2016). Wash lines of folding grass were also used where roads overtopped to identify the high-water mark from the lateral extent of overtopping. High-water mark elevations were measured with a laser level and rod.

For sites where an existing survey and one-dimensional Hydrologic Engineering Center River Analysis System (HEC-RAS) hydraulic model existed, the peak flood flow was estimated by adjusting the modeled flow until it matched the high-water elevation at the inlet. In all cases, the existing surveys included at least four cross-sections with two upstream and two downstream, culvert size, culvert invert elevations, and the road profile as required by HEC-RAS (Brunner 2016). These sites included the 16 SS sites that survived the flood, 2 older sites (Porcupine at FR 213, site 23; Red Ike at FR 208, site 26) that also survived the flood and 1 older site (Mineral Lake Inlet at FR 187, site 17) that failed. These three older sites had been surveyed in preparation for their replacement. For site 17, high-water marks were noted on the road on each side of the stream, and it was assumed the failure began at that peak elevation. For these sites, the high-water marks were referenced to the benchmark from the original design survey to obtain the high-water elevation. For sites that overtopped, flow over the road including weir flow, width of flow, average depth, and maximum depth were derived from the detailed culvert output in the HEC-RAS model. In five cases where gravel eroded off the road surface requiring replacement and maintenance, it was assumed the erosion began at the peak water level. If the erosion began before the peak water elevation, then there would have been more area with flow over the road at a greater depth and the actual discharge was higher than reported. In these cases, the calculated peak flood discharge estimates are conservative because they underestimate the actual flood peak.

For three older sites that survived, site 25, UNT Marengo at FR 196 (site 19), and site 18, the invert, top of culvert, and high-water mark were surveyed, and the peak flood flow was estimated from the HW/D and assuming inlet control (Schall et al. 2012). Site 18 also overtopped, so the road surface was surveyed and treated as a weir to estimate the portion of the flood flow that passed over the road.

For three older sites that failed, UNT Morgan at FR 199 (site 1), site 4, and 18 Mile at FR 371(site 20), a channel cross-sectional measurement was performed in addition to measurement of high-water marks. The slope-area method was then used to estimate the peak discharge with Manning's equation (Dalrymple and Benson 1984) (Table 2). This method was also used for 20 Mi at FR 378 (site 2) because of concerns with the HEC-RAS model results. Calculations were performed using the Hydraulic Toolbox (channel calculator tool) which allows roughness values to vary from channel to overbank areas (Federal Highway Administration 2021). Roughness values were developed primarily from tabular and, to a lesser extent, photographic guidance (Yochum 2017).

Table 2.—Slope-area flood flow estimates for four sites, Chequamegon-Nicolet National Forest, July 2016 flood

Site #	Stream	Road	Site ID	n Channel	n Overbank	Hydraulic radius (ft)	Slope (ft/ft)	Area (sq ft)	Top width (ft)	Mean depth (ft)	Maximum depth (ft)	Velocity (ft/sec)	Froude (#)	Discharge (cfs)
1	Unnamed tributary to (UNT) Morgan	FR 199	21731	0.060	0.10	3.168	0.032	138	40.9	3.37	5.03	9.74	0.93	1,341
2	20 Mi	FR 378	21613	0.045	0.10	3.552	0.020	131	31.3	4.19	6.86	10.19	0.88	1,335
4	UNT Whisky, East	FR 198	21711	0.050	0.08	2.039	0.015	182	85.2	2.14	5.23	6.24	0.75	1,136
20	18 Mi	FR 371	215181	0.045	-	4.014	0.016	241	55.0	4.38	6.07	10.59	0.89	2,551

The flood flow estimate for Hawkins at FR 383 (site 5) was developed from the estimated flood flow at the upstream site UNT Hawkins at FR 383B (site 3) using the drainage area transfer method (U.S. Army Corps of Engineers 2021). A flood flow estimate using HEC-RAS was not feasible because the structure failed and had water flow down the road from upstream. The drainage area adjustment was made with an exponent of 0.857, which is the corresponding exponent for 2 percent AEP (i.e., 50-year average RI) and 1 percent AEP floods in flood frequency area 4 (Walker and Krug 2003).

Flood Frequency Analyses

Flood frequency curves were developed for each of the 26 sites using regression equations for Wisconsin watersheds (Walker and Krug 2003). All watersheds were located within flood frequency area 4 and the following watershed characteristics were used in the equations: drainage area (square mile), storage (percent), main channel slope (feet/mile), soil permeability (inches/hour), and mean annual snowfall (inches). Drainage area and main channel slope were determined in GIS from 1:24,000 topographic maps. Main channel slope (MCS) is the slope of the stream between points that are 10 percent and 85 percent of the distance along the channel from the outlet of the watershed to the basin divide. Storage (STO) is the percentage of the watershed area in lakes, ponds, and wetlands. It was determined with GIS from 1:24,000 Wisconsin hydrography (Wisconsin Department of Natural Resources [WDNR] 2007) based on U.S. Geological Survey (USGS) topographic maps and the 1:24,000 Wisconsin wetland layer (WDNR 2021). Lake and pond area included shaid_typ polygons coded as lake or pond (LP), backwater area (BA), and convoluted stream (ZZ) (WDNR 2007). Mean annual snowfall was determined from Figure 2 in Walker and Krug (2003) which displayed lines of equal snowfall in increments of 10 inches. Soil permeability was determined from a GIS shapefile provided by the USGS of Plate 2 (Walker and Krug 2003) which had five permeability classes; every watershed had the same median permeability class of 1.65 inches/hour. Most of these curves were developed during culvert design for the site prior to the 2016 flood.

Five of the 26 sites had watersheds that were nested in another site's larger watershed (Fig. 2). In three cases—site 3, 20 Mi at FR 202 (site 9), and UNT WF Chippewa at FR 191 (site 21)—the lower portion of the larger watershed was similar to the nested watershed, so there was no need to separate the two portions when evaluating the effects of watershed characteristics on flood runoff. The other two nested watersheds—site 23 and 18 Mi at FR 213 (site 24) were in the headwaters of site 20, which is the largest of the 26 watersheds and nearly twice the size of the next largest watershed. Site 23 and site 24 also had substantially different watershed characteristics than the lower portion, so for some analyses the lower portion was treated as a separate unit. The lower portion of site 20 had a drainage area of 3.78 square miles or 18 percent of the entire watershed. Flood frequency data for that entire watershed are presented in Table 1, while analyses associated with watershed characteristics for it are based on the lower 18 percent sub-basin. The peak flood flow for the lower sub-basin was determined by subtracting the peak flows at the upper watersheds (sites 23 and 24) from the peak flow in Table 1 for the total area.

For three SS sites, the flood frequency curves were adjusted during the culvert design. Evidence from hydraulic modeling and watershed characteristics showed that the Wisconsin regression equations did not provide reasonable estimates. The adjustments for two sites were in large part based on hydraulic modeling of the 66.7 percent AEP (i.e., 1.5-year average RI, or flood which should correspond to bankfull elevation [Rosgen 2008, Leopold 1994]). In Ohio, for example, the median AEP for the bankfull discharge was 72.4 percent (i.e., 1.38-year average RI) (Sherwood and Huitger 2005). For site 24, the 66.7 percent AEP flood modeled 0.8 feet above the bankfull elevation across 70 feet of floodplain, and the watershed had a substantial area of sand outwash. These factors indicated the regression equations overestimated the flood flows, so they were adjusted downward 60 percent to match the bankfull elevation at the 66.7 percent AEP. These adjustments were substantially below the standard error for the regression equation discharges.

For UNT Morgan Falls at FT 209 (site 7), the opposite was true, and flows were adjusted upward by a factor of three in the same manner. UNT Marengo at FR 377 (site 10) had very high flood flow estimates (1 percent AEP or 474 cfsm) given the minimal storage and steep channel slope. It also had a drainage area of 0.21 square mile, which was 68 percent below the minimum size watershed of 0.66 square mile used to develop the regression equations. Based on those factors, and a comparison with two nearby gauges, the flood flows were reduced by about 15 percent but remained above the flows associated with lower standard error of the regression equations. The adjusted flows for these three sites are used throughout this paper.

The 0.2 percent AEP (i.e., 500-year average RI) flows were compared to the 2016 observed flood peaks at each site to characterize the extreme nature of the flood. The 1 percent AEP flows provided additional perspective and were used during hydraulic modeling to determine the HW/D ratio for SS culverts and to ascertain how well they met hydraulic design standards.

Site Characteristics

Flood-prone width, entrenchment, road fill height, channel slope, and road-stream angle were determined to help characterize factors that could have affected the survival or failure of each site, especially at overtopping. These site characteristics were derived from survey data for the 17 SS sites and 5 other sites. Site features for the other four locations were derived from road-stream crossing inventory data, topographic maps, and aerial photography. Flood-prone width was determined from the surveyed cross-sections (22 sites) giving preference to those immediately up and downstream of the road and to the narrower of the two, or from measurements on topographic maps and aerial photos (four sites) adjacent to the road. Flood-prone width and entrenchment are good indicators of the opportunity to provide for broad overflow areas across the road if overtopping occurs during an extreme flood and also to prevent stream diversion down the road. Fill height was determined from site surveys and represented the distance from the low point of the road surface (not necessarily at the culvert) to the floodplain elevation adjacent to the channel typically on the downstream side (22 sites) or road-stream crossing inventory data (4 sites). In general, the greater the fill height, the greater the potential for failure at overtopping and the greater the consequences. Channel slope was determined from stream profile surveys (22 sites) or topographic maps (4 sites). Steeper channel slopes can increase velocity, turbulence, and the transport of woody material. They can also lead to greater differences in upstream to downstream water surface at overtopping. The angle between road and stream is the deviation from 90°. The angle was determined from survey data (22 sites) or aerial photography (4 sites).

Structure Cost Comparisons

Because SS design method typically results in much larger stream crossing structures, increased structure cost is frequently a concern of road managers. The aforementioned 17 SS stream crossings provide an opportunity to compare costs for replace-in-kind (R-I-K) structures versus SS and its associated benefits, keeping in mind R-I-K structures may not meet current infrastructure or environmental objectives. When agency policy is to use SS design, such as with the USDA Forest Service (2014), these cost comparisons would be unnecessary; the goal would be to produce the most cost-effective SS design. For some other road management agencies, however, this cost comparison may help anticipate and account for SS costs and benefits.

Replacement costs for road-stream crossings can be complicated because each site is unique, and multiple factors affect the final cost. Site-specific factors that affect costs include stream size, fill height, road width, distance required to haul fill or other materials, subsurface conditions, traffic, safety standards, and road surface material (i.e., gravel vs. pavement). Costs can also vary substantially depending on the choices and objectives of the road management agency, including type of culvert material (i.e., corrugated metal with or without polymer coating vs. aluminum vs. concrete), the thickness or gauge of a metal pipe, type of structure (e.g., a culvert vs. box culvert vs. bridge), end treatment (headwall vs. 2:1, 3:1, or 4:1 side slopes), and whether the agency has the capability to perform construction or must contract the work. However, reasonable cost estimates can be calculated using as much site-specific information as possible and typical costs as proxy.

SS structure costs were based on budget records. In all cases, the intent was to estimate construction cost not including survey, design, and contract administration costs. For contracts, the award was used when available, and the engineer's estimate when contract award was not available. For projects constructed by Forest Service Heavy Equipment Operators (HEOs), salary and equipment time were added to the cost of the culvert and other supplies to obtain final construction cost. In some cases, planned costs were used when actual expenditures could not be recovered. A project implemented by HEOs is generally less expensive than a construction contract. Since these projects were implemented between 2001 and 2015, costs were adjusted to 2020 values using the Construction Cost Index (ENR 2021).

Cost estimates for R-I-K structures were developed using methods similar to Diebel et al. (2014) who estimated costs for replacement of all road stream crossings in the Pine-Popple Watershed of Wisconsin using data from a 2008 road-stream crossing survey. Site-specific road data were used to estimate excavation, structure, and reconstruction costs. Excavation and reconstruction costs were based on calculated volumes at $12/cubic yard, with bedding and gravel surfacing at $22/cubic yard. Disposal of unsuitable material was estimated assuming 20 percent of excavated material at $12/cubic yard. Culvert costs were estimated from culvert size, culvert length, and cost per foot of pipe using 2020 costs provided by Contech Engineered Solutions LLC (Headquarters: West Chester, OH) for each culvert size. Culvert costs were for corrugated metal pipe without a step bevel or polymer coating. Where more than one culvert existed, the R-I-K structure consisted of a single pipe with the same total area. In all cases, R-I-K culverts fit into the existing road fill avoiding increased fill height. Several other items were treated as fixed costs and added to the prior amount including pipe disposal ($800), riprap ($1800; 20 cubic yards at $90/cubic yard), erosion control ($1,660; silt fence, seeding, mulch), and dewater pumping ($4,000). Mobilization cost of 10 percent was then added to obtain a final estimate. All these costs were based on 2020 values derived from USDA Forest Service statistics. While the estimates are referred to as R-I-K, it was assumed that 2:1 side-slopes were a minimum requirement to ensure slope stability, so culvert lengths and construction volumes were adjusted accordingly.

Additional HEC-RAS modeling was conducted for the 16 surviving SS structures and their counterpart R-I-K structures to determine the flow at road overtopping. The results were used to estimate whether the R-I-K structures would have failed during the flood.

Effect of Watershed Characteristics on Flood Peaks

Watershed characteristics were investigated to determine their effect on flood response, and to serve as indicators for classifying landform vulnerability to increased flood flows (Furniss et al. 2013). Simple linear regression analysis was used to identify the relation of observed peak flows (cfsm) to main channel slope (MCS, feet/mile), storage (STO, percent), saturated hydraulic conductivity (KSAT, inches/hour), and drainage density (DD, miles/square mile) individually. The interrelation of each watershed characteristic was also investigated using simple linear regression. Peak flows, MCS, and DD received a $\log_{10}$ transformation to meet homogeneity of variance among groups and normality assumptions of residuals for all regression analyses. MCS and STO were determined as described above for flood frequency methods.

Two different KSAT parameters were determined for each of the 26 watersheds using the following procedures. One approach used the procedure described by Walker et al. (2020) which extracted data from Soil Survey Geographic database (SSURGO) datasets using the following criteria options: Aggregation method equals weighted average, Tie Break Rule equals fastest, Layer options equals surface layer. The mean KSAT for each watershed was determined using the Streamstats application (https://streamstats.usgs.gov/ss/). After this approach failed to correlate with the peak flood flows, a second approach was used. This second approach extracted KSAT data for each watershed from the USDA Web Soil Survey site (https://websoilsurvey.sc.egov.usda.gov/app/) (usda.gov) using the following criteria options: Aggregation method equals weighted average, Tie Break Rule equals slowest, Layer options equals all layers. Only upland soil polygons were used for this approach, based on the assumption that wetlands would serve as storage areas that receive runoff, making KSAT in those polygons less relevant to flood peaks. We used this second approach for our KSAT Up parameter. An area weighted mean was then calculated for each watershed. In both approaches, KSAT was converted from micrometers/second to inches/hour to provide a more direct comparison to rainfall intensities.

The highest KSAT Up value of 7.35 inches per hour for UNT Marengo S at FR 194 (site 12) was determined to be an influential outlier during the simple linear regression analysis and was removed from any further statistical analyses. Site 12 had a peak flow of 581 cfsm and the KSAT Up of 7.35 inches/hour while 14 other sites with peak flows from 195 to 1,103 cfsm had lower KSAT Up values of 1.47 to 3.03 inches/hour. The cause for this anomaly is unknown but may be from a soil mapping error. An evaluation of the outlier revealed a substantial discrepancy in the amount of wetland within the watershed when comparing the Wisconsin wetland layer versus the soil layer.

DD (miles/square mile) was determined by intersecting stream and watershed layers in GIS. The stream layer was derived from the WI 24,000 hydrography layer (Wisconsin Department of Natural Resources 2007) by selecting the following linear types: CL (i.e., the centerline of a stream or river mapped as a polygon), ST (i.e., single line stream), and DC (i.e., ditch or canal). Both perennial and intermittent duration were used. DD excluded centerlines through lakes, ponds, and reservoirs. The hydrography layer was obtained from the WDNR (WDNR 2021). One drawback of attempting to use DD as a watershed characteristic was lack of representation of some stream channels at the 1:24,000 scale for some small watersheds, especially in the PG and VMM LTAs. As a result, reasonable DD data were only available for 22 of the 26 watersheds.

Initial evaluation of peak flows indicated spatial variability, and this variability appeared to align with LTA boundaries. Using a state-wide GIS layer of LTAs (WDNR 2021), we intersected the 26 watersheds with the LTA layer to determine the proportions of each LTA in each watershed. The watersheds were then classified according to their dominant LTA. Because there was a limited amount of replication for some LTAs, they were lumped based on similarity into three categories and that classification was applied to each watershed.

A simple one-way analysis of variance (ANOVA) was used to compare the mean differences of each variable (peak flow, and the watershed characteristics of main channel slope, storage, drainage density and KSAT Up) individually among the three LTA groups. Differences among LTA groups for each univariate model were determined using the Tukey-Kramer adjustment test for multiple comparison of means ($p = 0.05$). To meet homogeneity of variance and normality assumptions of the residuals, peak flow, rainfall, MCS, DD, and KSAT Up were $\log_{10}$ transformed. Rainfall also required a reflected transformation prior to the $\log_{10}$ transformation due to data being negatively skewed. While storm rainfall was substantial for all sites, it was not uniformly distributed across the study area. For the peak flow analysis, we used an analysis of covariance (ANCOVA) to account for the potential effect of this uneven distribution by entering rainfall as a covariate. Rainfall was not considered a covariate for the watershed characteristic univariate analyses because it does not influence these landform characteristics.

Landscape Vulnerability to Flood Flow Increases

LTAs were used as building blocks to characterize the vulnerability or resilience of areas to increases in flood flows using the 2016 flood response of the 26 watersheds as a guide. Landforms with high infiltration capacity, high permeability, low relief, and high storage are expected to be more resilient to increases in future flood flows because those characteristics attenuate runoff from rainfall. Landforms with low infiltration capacity, low soil permeability, high relief, and low storage are expected to be vulnerable to increases in future flood flows because those characteristics lead to more rapid runoff from rainfall. DD was used as a surrogate for infiltration capacity, permeability, and relief because it reflects the integration of those factors for a given climate (Carlston 1963). Higher DDs occur where runoff is rapid and lower DDs occur where runoff is attenuated. DDs for each LTA were determined by intersecting stream and LTA layers in GIS using the methods described earlier.

STO was determined for each LTA by combining lake and pond area with wetland area and intersecting these with the LTA layer in GIS. Lake and pond area, and wetland area were determined using methods described earlier under flood frequency with a few exceptions. Reservoir or flowage (i.e., RF) areas were not included, because this portion of the analysis was intended to characterize the general vulnerability of LTAs to flood flow increases from rainfall during culvert design. Most reservoirs are on main rivers that are too large for culverts or small bridges and therefore not relevant. When computing percent STO, the reservoir area was removed from the LTA. Wetland area included all wetland polygons except those coded with a special modifier description of red clay complex. These units occurred on old lake plain adjacent to Lake Superior where wet and dry clay soils are intermingled (WDNR 1992). These wetland units are more likely to contribute to rapid runoff than attenuate runoff as wetlands typically do and were removed from the layer representing STO.

Each LTA was classified as either vulnerable, moderately vulnerable, moderately resilient, or resilient using a combination of DD, STO, and observed flood flows at gaging stations as guides. DD and STO were used in the initial classification process. Based on the range of values, DD and STO were given a rank of 1 to 5 for each LTA, and these were also summed for a total rank to guide the initial classification process. Areas with the highest DDs and lowest STO were considered most vulnerable to flood flow increases, while areas with the lowest DDs and highest STO were considered most resilient, with one exception. Some LTAs have very low DDs because of very high infiltration and permeability, such as outwash sands, but also have very low STO. These areas serve primarily as groundwater recharge areas and would be resilient to increases in future flood flows from more extreme rainfall events. In these cases, DD and the LTA soil description were given priority over STO in the classification process. The 1 percent AEP flood flow expressed at cfsm was determined for USGS gaging stations with drainage areas less than 75 square miles and these points were plotted across the state. During LTA classification, the flood flows within or near the LTA were also used as a guide in the classification process with the highest flow rates representing vulnerable landforms and lowest flow rates representing resilient landforms. Gaging stations with drainage areas less than 75 square miles were used because channel transit time can become a dominant factor affecting flood flows as watershed area increases above that size (Carlston 1963).

The occurrence of Class I and II trout streams (WDNR 2021) was a fourth factor considered during the classification process to serve as a tiebreaker for some LTAs. Trout streams typically have a substantial inflow of groundwater at baseflow. In areas where DD was moderate or low, Class I and II trout streams were reviewed to determine whether streams in the LTA were more likely fed by groundwater or surface water, with groundwater indicating greater resilience to increased flood flows from more intense rainfall. However, many trout streams in Wisconsin, such as those located in the driftless area, Penokee/Gogebic Range, and Lake Superior clay till plain, have both a substantial groundwater-fed baseflow and rapid runoff in response to rainfall. Therefore, trout stream occurrence was not considered in areas with high drainage densities and high relief.

Descriptive statistics were used to characterize the range and variability of DD, STO, and 1 percent AEP flood flows within each of the four vulnerability classes following classification. Simple one-way ANOVAs were used to test ($p = 0.05$) for mean differences of each variable among the four vulnerability classifications. Flood flows were $\log_{10}$ transformed to meet the homogeneity of variance and normality of residuals assumption. Differences among groups were determined using the Tukey-Kramer test for multiple comparison of means.

RESULTS

Storm Magnitude

This rainstorm was a very rare event of extreme magnitude. It began on the evening of July 11, 2016, with the bulk of the rainfall occurring in a 6-hour period (Fig. 3). Total storm rainfall in the area ranged from 5.0 to 9.2 inches (Fig. 1). A 6-hour, 5.16-inch rainstorm in that portion of northern Wisconsin was estimated to have an AEP of 1 percent (NWS 2017). AEPs for the highest rainfall amounts of 6.5 to over 9.0 inches were reported to be in the range of 0.2 percent to 0.1 percent (i.e., 1,000-year average RI) (NWS 2016). The 26 watersheds where flood flows were estimated received 5.4 to 9.2 inches with 19 of the 26 watersheds receiving 8.0 or more inches over 6 hours (Table 1).

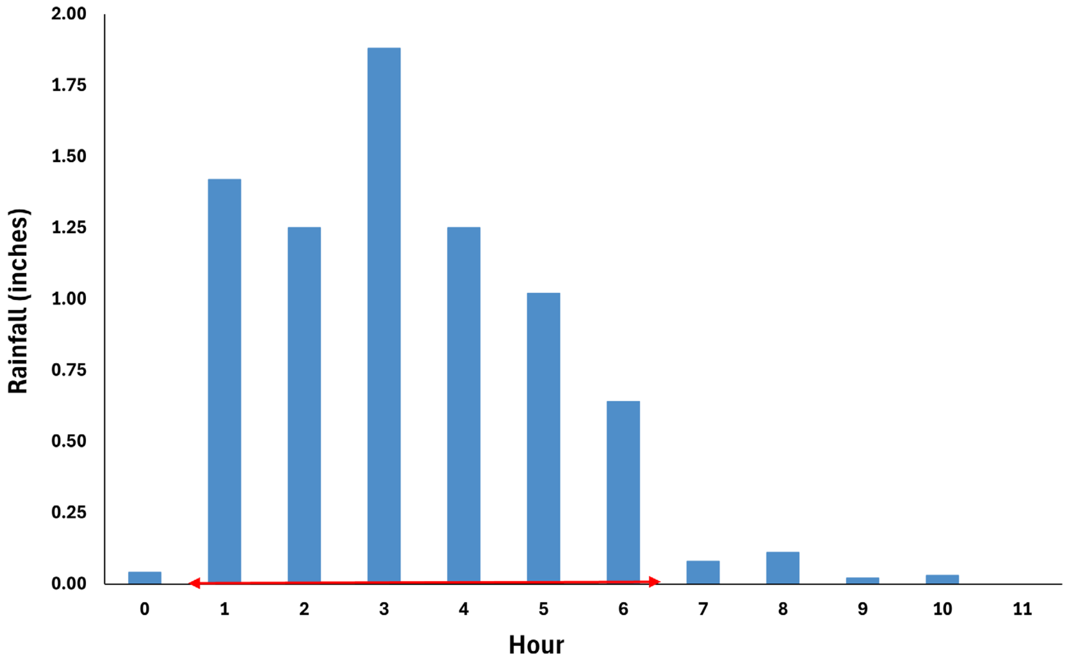

Figure 3.—Rainfall duration and hourly amounts for July 11–12, 2016, latitude 46.2875, longitude 90.9846. The maximum 6-hour duration, denoted by the red line, began at 1900 central daylight time on July 11, 2016.

Flood flows for 21 watersheds ranged from 102 to 2,629 cfsm with the other 5 from 9 to 60 cfsm (Table 1). Presenting flood flows in terms of cfsm adjusts for differences in drainage area which ranged from 0.21 to 20.47 square miles for the 26 watersheds (Table 1). Site 24 was the largest watershed with the next largest almost half its size at 11.5 square miles. Flood magnitudes are also expressed as ratios of the 0.2 percent AEP flood as determined from the USGS regression equations for each watershed (Walker and Krug 2003). These ratios ranged from 1.1 to 10.5 for all but three watersheds, illustrating the extreme nature of the 2016 flood. These ratios are impressive considering that typical increase in flood magnitude from the 1 percent AEP to the 0.2 percent AEP based on the regression equations, is about 35 percent. Although the runoff response among watersheds was highly variable, most produced flood flows that were several times larger than the estimated 0.2 percent AEP.

A flood frequency graph can be produced for each site from the USGS regression equations and would seem to offer the opportunity to estimate the AEP or average RI for the estimated flood peaks. This proved unreasonable in our case, since the 2016 flood peaks plotted several orders of magnitude beyond any rational AEP or RI on the logarithmic scale. This outcome led to the use of the 0.2 percent AEP flood ratio as an expression of the relative magnitude of the flood at each site.

Culvert Survival and Failure at SS Sites

The 17 SS structures had 16 survivors and 1 failure, but flood flows overtopped the road at 13 of the survivors (Table 1). The 2016 flood exceeded the 1 percent AEP at all 17 sites with the estimated flood flows from 1.6 to 14.4 times the discharge for the 1 percent AEP and 1.1 to 10.5 times for the 0.2 percent AEP.

Two key criteria for SS design are that structure width equals or exceeds bankfull width and the 1 percent AEP flow passes with HW/D <0.8. The 17 SS structures were evaluated to determine how well they met these criteria. All but three had constriction ratios (structure width/bankfull width) of 1.0 or greater and therefore met the bankfull width criteria (Table 3). The three narrower structures were 0.84 to 0.95 of bankfull width and were deemed sufficiently close to be included in the SS category. These narrower structure designs most likely occurred from balancing costs, structure availability, road height, and other factors during design.

Nine of 17 SS structures met the criterion for HW/D <0.8 at the 1 percent AEP with values from 0.62 to 0.78 (Table 3). Of the eight sites that exceeded the HW/D criterion, seven were from 0.88 to 1.08 and one was in excess at 1.30. The site with the largest HW/D, site 3 (Table 3), utilized an existing culvert and was the first site in the Forest where stream banks and bed were constructed within the structure. While that culvert marginally met bankfull width, it was hydraulically undersized with the road overtopping at the 1 percent AEP.

Site factors can also influence culvert survival and failure, especially at overtopping. In our study, 13 of the 17 SS sites had very broad flood-prone areas that, with 1 exception, ranged from 110 to 460 feet wide, resulting in slight entrenchments of 6.6 to 77 (Table 3). These sites also had relatively low fill heights. Excluding a 5.8-foot fill to accommodate a large box culvert and reduce a road surface erosion problem at a site that did not overtop, fill heights for these slightly entrenched sites ranged from 1.7 to 4.0 feet with an average of 2.8 feet. In all but two of these cases, the road was aligned perpendicular to the floodplain so flow over the road was at a right angle. These sites also had more gentle channel slopes with only three over 0.4 percent (Table 3). While the 2016 flood overtopped the road at 11 of these 13 sites, the site factors of slight entrenchment, low fill height, and low channel slope, in addition to the extra capacity associated with SS structures, no doubt contributed to their survival. While the portion of total flow that passed over the road was generally related to the magnitude of the flood at the site, there were notable exceptions. Two sites had 2016 flood flows of 581 and 759 cfsm yet passed only 9 and 5 percent of the total flow over the road.

Table 3.1.—Flow over the road and other site data for 26 road-stream crossings, Chequamegon-Nicolet National Forest, July 2016 flood

Site #	Stream	Design method	Flood flow (cfsm)	Constriction ratio	Headwater/Depth (HW/D) 1% annual exceedance probability (AEP)	Bankfull width (ft)	Flood prone width (ft)	Entrench-ment	Road fill height (ft)	Channel slope (%)	Flow angle (from 90°)	Road surface
2	20 Mi	SS	1,712	0.85[a]	1.00[a]	7.0	230	32.9	2.1	0.3	20	Gravel
3	Unnamed tributary to (UNT) Hawkins	SS	1,223	1.13	1.30[a]	7.0	140	20.0	2.9	1.5	0	Gravel
5	Hawkins	SS	1,019	1.54	0.73	16.0	30	1.9	7.1	2.0	30	Gravel
6	UNT Whisky, West	SS	762	0.95[a]	0.68	5.6	170	30.4	3.0	0.3	0	Gravel
7	UNT Morgan Falls	SS	759	1.45	0.88[a]	5.0	33	6.6	2.5	0.9	0	Gravel
8	UNT Whisky, Middle	SS	692	1.45	0.71	5.0	110	22.0	3.8	0.3	0	Gravel
9	20 Mi	SS	677	1.34	0.77	4.4	330	75.0	2.0	0.2	0	Gravel
10	UNT Marengo	SS	657	1.33	0.76	4.0	8	2.0	4.0	2.7	Down ditch	Gravel
11	UNT Trout, East	SS	598	1.21	0.92[a]	6.0	460	76.7	1.7	0.4	0	Gravel
12	UNT Marengo, South	SS	581	1.73	0.65	4.2	190	45.2	3.4	0.3	0	Gravel
13	McCarthy	SS	348	1.03	0.90[a]	9.7	170	17.5	4.0	0.2	0	Gravel
14	Preemption	SS	262	1.14	0.71	10.5	19	1.8	7.8	2.3	0	Paved
15	UNT Marengo, North	SS	245	0.84[a]	0.90[a]	8.6	250	29.1	3.1	0.1	0	Gravel
16	Trout	SS	221	1.89	0.78	15.9	19	1.2	7.0	1.0	20	Paved
21	UNT WF Chippewa	SS	125	1.00	1.02[a]	8.6	390	45.3	2.2	0.1	0	Paved
22	Brush	SS	102	1.05	1.08[a]	15.3	240	15.7	3.3	0.1	0	Gravel
24	18 Mi	SS	27	1.24	0.62	13.0	120	9.2	5.8	0.3	0	Gravel
1	UNT Morgan	Other	2,629	0.40	>1.00	10.0	18	1.8	5.0	3.2	15	Gravel
4	UNT Whisky, East	Other	1,103	0.58	0.70	12.5	45	3.6	9.0	1.5	0	Gravel
17	Mineral Lake Inlet	Other	215	0.67	1.09	12.0	72	6.0	8.0	0.7	35	Gravel
18	UNT Brunsweiler	Other	195	0.42	>2.02	8.4	50	6.0	5.0	1.5	0	Gravel
19	UNT Marengo	Other	145	0.73	0.98	5.5	72	13.1	9.0	0.5	0	Gravel
20	18 Mi	Other	125	0.45	1.25	29.0	50	1.7	12.0	1.6	0	Gravel
23	Porcupine	Other	60	0.53	1.07	16.0	70	4.4	7.0	1.1	20	Gravel
25	WF Chippewa	Other	25	0.39	1.40	18.0	60	3.3	12.0	0.1	5	Paved
26	Red Ike	Other	9	0.32	>1.49	11.0	100	9.1	4.0	0.1	5	Gravel

Notes: Horizontal dashed lines separate the SS design method from other methods. Stream simulation standard is >1.0 for constriction ratio and <0.8 for HW/D at 1 percent AEP. Site 10 is on an alluvial fan. The > sign for HW/D 1% AEP indicates flow would overtop road and the HW/D value is the point of overtopping. Values followed by a superscript "a" did not meet SS standards.

Table 3.2.—Flow over the road and other data for 26 road-stream crossings, Chequamegon-Nicolet National Forest, July 2016 flood

Site #	Stream	Discharge (cfs)	Discharge (%)	Mean depth (ft)	Max depth (ft)	Water width (ft)	Mean velocity (ft/sec)
2	20 Mi	1,257	94	1.41	2.43	270	3.30
3	Unnamed tributary to UNT Hawkins	946	82	1.70	2.94	151	3.67
5	Hawkins	Site failed; flow overtopped road					
6	UNT Whisky, West	124	78	0.34	0.65	230	1.55
7	UNT Morgan Falls	10	5	0.29	0.57	24	1.44
8	UNT Whisky, Middle	319	71	0.74	1.16	186	2.31
9	20 Mi	126	60	0.39	0.60	185	1.75
10	UNT Marengo						
11	UNT Trout, East	262	72	0.47	0.75	300	1.86
12	UNT Marengo, South	19	9	0.13	0.15	149	0.96
13	McCarthy	172	37	0.56	0.67	230	1.32
14	Preemption	66	9	0.32	0.53	133	1.55
15	UNT Marengo, North	137	39	0.38	0.63	222	1.63
16	Trout	600	53	1.41	1.79	258	1.65
21	UNT WF Chippewa	47	21	0.25	0.49	126	1.50
22	Brush						
24	18 Mi						
1	UNT Morgan	Site failed; flow overtopped road					
4	UNT Whisky, East	Site failed; flow overtopped road					
17	Mineral Lake Inlet	Site failed; flow overtopped road					
18	UNT Brunsweiler	194	77	0.80	1.18	80	3.03
19	UNT Marengo						
20	18 Mi	Site failed; flow overtopped road					
23	Porcupine						
25	WF Chippewa						
26	Red Ike						

Note: Horizontal dashed lines separate the SS design method from other methods.

Three SS sites with entrenchment less than 2.0 had flood-prone widths of 30 feet and less, road fill heights of 7.0 to 7.8 feet, and channel slopes of 1.0 to 2.3 percent. The two surviving sites also had a paved road surface which likely helped survival, and one had concrete bridge abutments. Flow over the road was low (9 percent) and moderate (53 percent) and these sites had moderate unit flood flows (221 and 262 cfsm) for the 2016 flood. Water over the road far exceeded the flood-prone width at both sites. The SS site that failed, site 5, had flow cross the road at a 30° angle, along with flow down the road. While the extreme flow (1,019 cfsm), moderate entrenchment, high fill height, and steep slope substantially contributed to the failure of this site, the poor angle at overtopping may have also played a role. Site 5 had an aluminum box culvert which would have much less structural strength if the surrounding fill eroded away, and that likely reduced its survivability compared to a concrete structure. The site did not experience any slumps or landslides.

Site 10 is unique because it is located on an alluvial fan. It has a small, narrow valley with a steep intermittent channel and the valley begins to widen and flatten out at the road crossing. To accommodate this landform, a second culvert was installed in an overflow channel west of the main channel and the road was raised over these culverts because the previous culvert was drastically undersized. The road elevation drops to the west so any flow that exceeded the capacity of the two culverts would travel west down the ditch. During the 2016 flood about 4 cfs or 3 percent of the flow went down the ditch at this site even though it had a high 2016 flood flow of 657 cfsm.

Survival at Overtopping at SS Sites

There appear to be five primary reasons that 13 SS structures (12 culverts and 1 bridge) overtopped but survived. First, the additional capacity provided by the SS approach substantially increased the amount of water that could be conveyed through the structure and thus not passed over the road. It also allowed the headwater to naturally rise with the tailwater until the two water surfaces began to separate when the upstream water level approached and exceeded the top of the culvert. This resulted in a lower difference between the tailwater and headwater elevations (and thus lower pressure differentials) at overtopping, compared to a smaller culvert, which reduced erosional forces on the structure and road. This is illustrated in Fig. 4, where the headwater for the 1 percent and 0.2 percent AEP flood flows are not artificially elevated above the tailwater and the two water levels do not start to separate until the headwater approaches the top of the culvert.

Second, none of these sites plugged with debris. In all but one case, the bankfull width structures were able to pass whatever size woody material was in transport. Site 3 had an approximately 8-inch diameter log wedged at the inlet of the culvert, but it represented a small portion of the waterway opening; the site did not fail even though it passed 82 percent of the flow over the road. The bridge over Trout at FR 390 (site 16) had a log approximately 30 feet long and 12 inches diameter, along with some smaller woody debris, wedged against the downstream bridge railing on the road. There were minor amounts of small woody debris caught in the riprap on the sides of the channel.

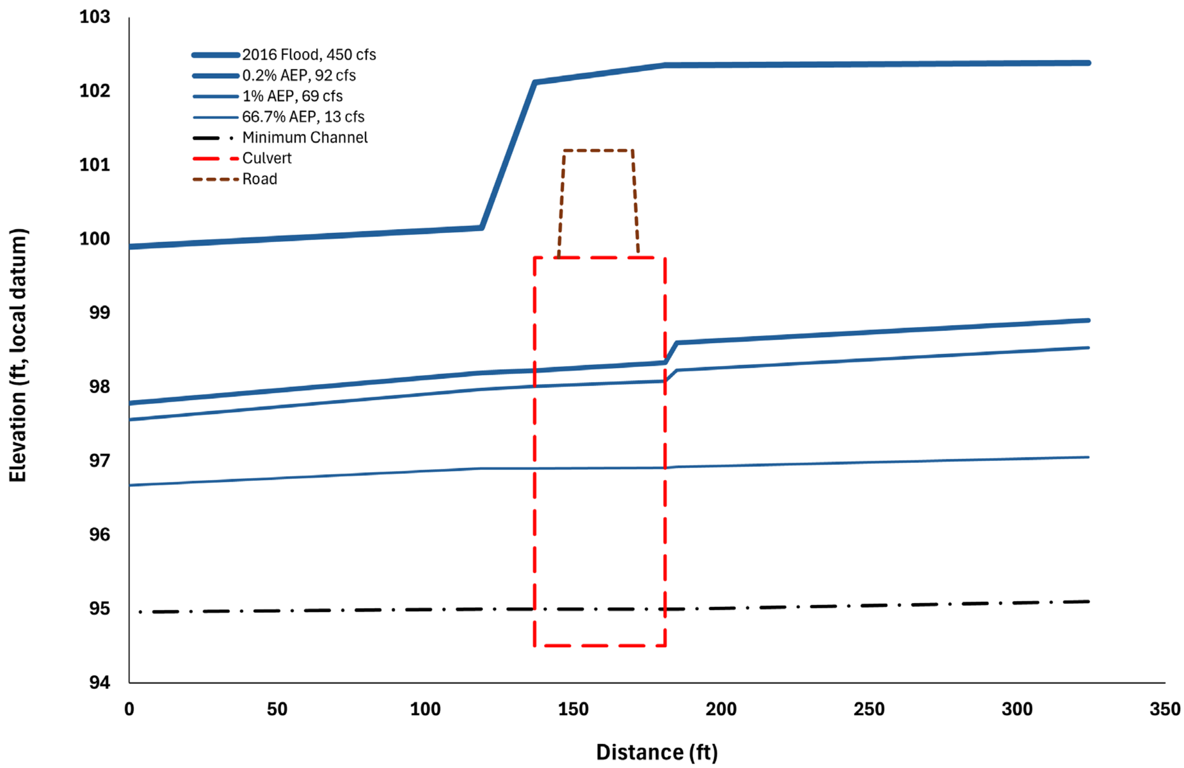

Figure 4.—Water surface profiles for unnamed tributary to Whisky Creek middle at Forest Road 198 (site 8); cfs = cubic feet per second.

Third, crossings which overtopped were able to pass large portions of the flood across the road without completely failing. Table 3 provides the HEC-RAS results for water over the road for these sites along with entrenchment and fill height. 11 of the survivors had very slight entrenchment and low fill heights (i.e., <4.0 feet). The other two survivors with low entrenchment and higher fill heights were paved. Discharges over the road covered a wide range from 10 to 1,270 cfs and from 5 to 95 percent of the total estimated flood flow. Three sites had 5 to 9 percent of the flood pass over the road, five sites had 21 to 60 percent, and five sites had 71 to 95 percent. The typical water width, average depth and average velocity over the road was 150 feet, 0.5 feet and 1.5 feet/second, respectively. There was little or no gravel loss off the roads which passed only 5 to 37 percent of the flood (Fig. 5a), while some sites that passed higher proportions lost more gravel into the road ditch or downstream floodplain (Fig. 5b).

A B

Figure 5.—Comparison of road surface erosion: (A) minimal erosion for McCarthy Creek at Forest Road 184 (site 13) and (B) more substantial erosion for unnamed tributary to Marengo River North at Forest Road 194 (site 15). USDA Forest Service photos by Dale Higgins.

Fourth, in 12 cases, the larger culvert associated with SS required the road surface to ramp gradually up and over the culvert to provide adequate fill over the pipe. This provided overflow areas to one or both sides of the culvert which helped protect the structure itself. Examples of this design feature include UNT Trout E at FR 390 (site 11) with overflow areas on both sides of the culvert (Fig. 6a) and UNT Whisky M at FR 198 (site 8) with an overflow area on one side (Fig. 6b).

Fifth, construction and maintenance practices that provide erosion protection to the road surface and downstream fill slope including pavement, riprap, and vegetative ground cover appeared to help site survival. All the SS sites that overtopped and survived had riprap around the structure and dense vegetative ground cover. Three of those sites (Trout at FR 390, site 16; Preemption at FR 377, site 14; and site 21) had asphalt pavement which helped protect the road surface from erosion (Table 3).

Anecdotal evidence suggested the angle of the road to flow at overtopping may have influenced survivability though it was not identified as a substantial factor. Overtopping at all but one of the SS sites, site 16, occurred at a right angle to the road which reduced turbulence and erosion as water flowed across the road surface. In areas where the road is not aligned with the stream, water flows over the road surface for a longer distance and turns downstream when flowing off the surface which could increase turbulence. At the SS site that failed, site 5, the structure was aligned with the stream, but the road was not, and this may have contributed to the failure even though other factors such as flood magnitude, narrow floodplain width, and channel slope played a substantial role in failure. Some sites where flood flows were not estimated and that failed also had poor alignment between the road and stream at overtopping while some survivors had broad flood-prone widths at a right angle to flow. This general field observation presents a new question for study and should not be confused with the current best practice of aligning the culvert inlet with the stream to reduce eddying and increase debris passing.

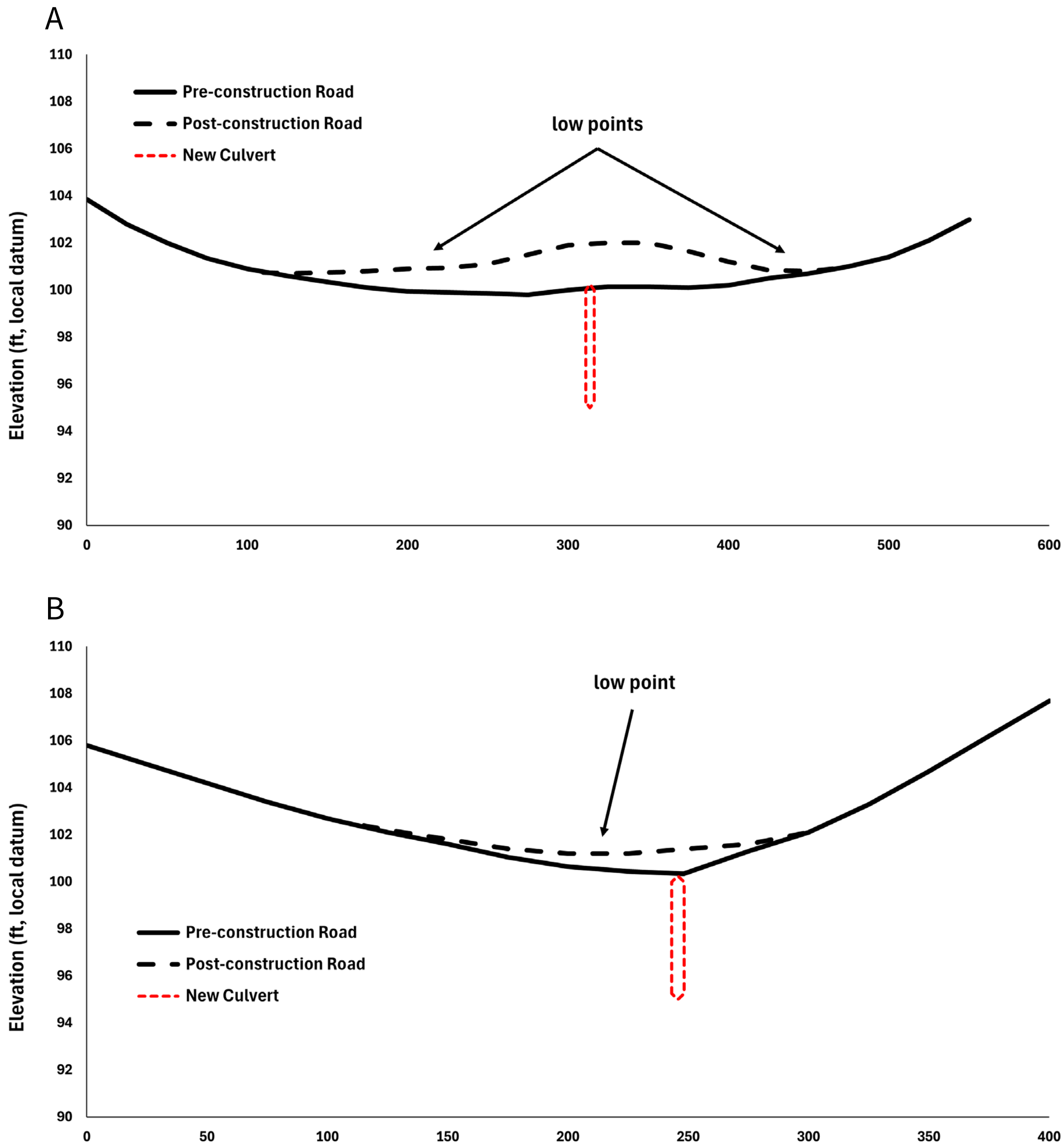

Figure 6.—Example of (A) low points in road profile on either side of culvert at Unnamed Tributary to Trout Creek east at Forest Road 390 (site 11) and (6B) low point in road profile on one side of culvert, Unnamed Tributary to Whisky Creek middle at Forest Road 198 (site 8).

Four of the 17 SS sites had stream banks and beds constructed inside the structure. All four of these sites overtopped, but three survived. One purpose of the SS design criteria for HW/D <0.8 at the 1 percent AEP is to prevent pressurized flow in the culvert and protect the streambed from failure. While these three sites had HW/D ratios from 1.44 to 1.71 at the flood peak, the streambed under the bridge at site 16 appeared to be unaffected. Portions of the other two constructed streambeds (site 3 and site 14) were lost during the flood. Some bank rocks and other streambed material remained in the culverts and those sites were still providing aquatic organism passage.

Culvert Survival and Failure at Other Sites

For the nine other sites not designed using SS techniques and where 2016 flood flows were estimated, five structures survived and four failed (Table 1). These sites had both the highest and lowest flood flows observed; two sites were less than the 1 percent AEP, five sites were 1.2 to 3.3 times the 1 percent AEP, and two sites were over 9 times the 1 percent AEP.

The five surviving sites had the first, second, fourth, eighth, and ninth lowest flood flows of all 26 sites; the flows were 9, 25, 60, 145, and 195 cfsm, respectively (Table 1). These lower flood flows likely contributed to structure survival because four did not overtop. Site 18 had the highest flood flow (i.e., 195 cfsm) of the five survivors, and was the only one to overtop; large riprap, dense side slope vegetation, and flow at a right angle to the road, all helped it survive, though it passed 77 percent of the flood over the road (Table 3).

Of the four other sites that failed, two sites had the highest and third highest flood flows: 2,629 and 1,103 cfsm, respectively (Table 1). The other two sites were much lower but still substantial at 215 and 125, cfsm, respectively. In addition to the extreme nature of the flood, lack of capacity was a primary contributor to failure at these four sites as evidenced by constriction ratios of 0.40 to 0.67 (Table 3). They also had site characteristics that increased their risk of failure at overtopping (Table 3). They had somewhat narrow flood-prone widths (18–72 feet), low entrenchments (1.7–6.0), and steep channels (0.7–3.2 percent); three sites had high road fills (8–12 feet); and two had flow over the road at an angle (15 and 35°). Site 17 had two 4-foot diameter concrete pipes that were separating and a deteriorating rock head wall which, along with extreme flood conditions, likely played a primary role in its failure. Flow over the road at a 35° angle may also have contributed to this failure. There were no slumps or landslides observed at the sites that failed.

Cost Comparisons

The type, size and 2020 construction cost estimates for the 17 SS structures that were installed from 2001 to 2015 are provided in Table 4 along with similar data for R-I-K structures. Table 5 provides supplemental data representing three quantifiable SS benefits including constriction ratio, water way opening area, and cost per square foot of waterway opening. SS costs covered the wide range of $17,230 to $278,710 while estimated R-I-K costs varied from $14,827 to $27,538. The cost for SS structures was from $1,070 to $252,821 higher than R-I-K costs or 7 to 982 percent higher. These wide ranges are explained by many site-specific characteristics, especially stream size and the degree to which the previous culvert was undersized. Data presented in Tables 4 and 5 can be split into two logical groups based on SS structure size: (1) the first group contains 11 pipe-arch culverts less than 10 feet wide referred to as “culverts”, and (2) the second group contains 6 structures 10 feet wide or larger referred to as “bridge-type” structures.

Table 4.1.—Type, size, and cost estimates for 17 replace-in-kind (R-I-K) structures, Chequamegon-Nicolet National Forest, 2016 flood area

Site #	Stream	Bankfull Width (ft)	Type	Width (ft)	Height (ft)	Length (ft)	R-I-K length (ft)	Cost (2020 $)
6	Unnamed tributary to (UNT) Whisky, West	5.6	Pipe arch	3.50	2.42	30	40	$16,159
2	20 Mi	7.0	Circ cmp	3.00	3.00	29	40	$15,794
7	UNT Morgan Falls	5.0	Circ cmp	4.00	4.00	20	36	$17,436
12	UNT Marengo, South	4.2	Pipe arch	4.08	2.75	33	34	$19,307
8	UNT Whiskey, Middle	5.0	Pipe arch	4.08	2.75	28	46	$19,030
10	UNT Marengo	4.0	Pipe arch	1.75	1.25	41	50	$14,827
9	20 Mi	4.4	Pipe arch	2.92	2.00	38	40	$18,762
11	UNT Trout, East	6.0	3-Circ cmp	1.50	1.50	32	42	$17,979
15	UNT Marengo, North	8.6	2- Pipe arch	3.50	2.42	34	56	$22,245
3	UNT Hawkins	7.0	Circ cmp	3.00	3.00	27	48	$19,380
21	UNT WF Chippewa	8.6	Pipe arch	4.75	3.17	34	50	$21,303
13	McCarthy	9.7	2-circ conc	4.00	4.00	28	46	$25,664
22	Brush	15.3	Circ cmp	5.00	5.00	30	50	$24,155
14	Preemption	10.5	Pipe arch	4.75	3.17	36	52	$27,538
24	18 Mi	13.0	Circ cmp	3.50	3.50	30	40	$17,956
16	Trout	15.9	Circ steel	4.00	4.00	31	50	$20,081
5	Hawkins	16.0	2-Circ cmp	4.00	4.00	34	44	$25,889

Notes: Culverts are above the dashed line and bridge-type structures below. Circ cmp = circular corrugated metal pipe, alum = aluminum. Previous culvert diameters at UNT Trout East/FR 390 were 2.0 ft, 1.5 ft, and 1.0 ft.

Table 4.2.—Type, size, and cost estimates for 17 stream simulation structures, Chequamegon-Nicolet National Forest, 2016 flood area

Site #	Stream	Bankfull Width(ft)	Type	Width (ft)	Height (ft)	Length (ft)	Year	Cost (2020 $)
6	Unnamed tributary to (UNT) Whisky, West	5.6	Pipe arch	5.33	3.58	40	2015	$17,230
2	20 Mi	7.0	Pipe arch	5.92	3.92	40	2001	$18,111
7	UNT Morgan Falls	5.0	Pipe arch	7.25	5.25	36	2015	$19,009
12	UNT Marengo, South	4.2	Pipe arch	7.25	5.25	34	2003	$20,844
8	UNT Whisky, Middle	5.0	Pipe arch	7.25	5.25	46	2014	$22,119
10	UNT Marengo	4.0	2-Pipe arch	5.33	3.58	50	2015	$22,466
9	20 Mi	4.4	Pipe arch	5.92	3.92	40	2012	$22,938
11	UNT Trout, East	6.0	Pipe arch	7.25	5.25	42	2012	$23,575
15	UNT Marengo, North	8.6	Pipe arch	7.25	5.25	56	2010	$24,069
3	UNT Hawkins	7.0	Pipe arch	7.92	5.58	48	2006	$27,000
21	UNT WF Chippewa	8.6	Pipe arch	8.58	5.92	50	2013	$27,405
13	McCarthy	9.7	Alum box	10.00	4.83	31.5	2010	$52,708
22	Brush	15.3	Alum box	16.00	4.25	32	2009	$64,050
14	Preemption	10.5	Ellipse	12.00	8.50	52	2007	$68,573
24	18 Mi	13.0	Alum box	16.17	5.08	31.5	2012	$106,602
16	Trout	15.9	Bridge	30.00	6.00	26	2012	$217,326
5	Hawkins	16.0	Alum box	24.58	9.75	44	2014	$278,710

Notes: Culverts are above the dashed line and bridge-type structures below. Circ cmp = circular corrugated metal pipe, alum = aluminum. The second culvert at UNT Marengo/FR 377 was 49 in × 33 in × 40 ft.

Several benefits of SS are difficult to quantify financially. These include flood resiliency, reduced maintenance, increased life-span, improved aquatic organism passage, and restoration of stream morphology. Two parameters that help quantify these benefits are constriction ratio and the open or cross-sectional area of a structure. Constriction ratio is a measure of structure width to bankfull channel width. The objective of stream simulation is to provide a structure that is bankfull channel width or greater (i.e., constriction ratio >1.0) because it provides multiple benefits to both the stream and road crossing as previously described in the introduction. The average constriction ratio was 0.55 (range 0.25–0.97) for all R-I-K culverts and increased to 1.25 (range 0.84–1.89) for all SS structures. For the 11 culvert sites, the average constriction ratio of 0.63 (range 0.43–0.97) for R-I-K culverts increased to an average of 1.21 (range 0.84–1.73) for SS design. For the six bridge-type sites, the increase in constriction ratio was even more dramatic; the average of 0.41 (range 0.25–0.71) for R-I-K increased to an average of 1.31 (range 1.03–1.89) for SS.

Though there is no specific requirement per se for the cross-sectional area of a structure opening in SS design, the requirements for bankfull width, matching a reference reach cross-section and passing the 1 percent AEP flow with HW/D <0.8, together often lead to a large increase in open area. The average open area was 11.0 square feet (range 1.6–25.2) for all R-I-K culverts and increased to 45.1 square feet (range 14.3–128) for all SS structures. For the 11 culvert sites, the R-I-K average open area of 7.9 square feet (range 1.6–12.8) increased to 27.5 square feet (range 14.3–42.4) with SS; an average increase of 340 percent (range 123–1,338 percent). While all sites are unique, site 10 requires special consideration because of a severely undersized pipe and location on an alluvial fan. These conditions required the 1,338 percent increase in open area and substantial amount of additional road fill for the SS structure. Without that site, the average increase in open area was 240 percent (range 123–463 percent). For the six bridge-type sites, the increase in open area was even more dramatic with the R-I-K average of 16.8 square feet (range 9.6–25.2 square feet) increasing to 77.5 square feet (range 40.2–128 square feet) for SS, an average increase of 440 percent (range 60–924 percent). The 17 SS structures increased the open area 1.2–13.5 times with most in the range of 3–5 times. Thus, a large part of the increases in cost for SS structures was that the existing culverts were so substantially undersized.

Cost per square foot of open area can be used to compare cost efficiency of SS versus R-I-K structures since all SS structures have much greater area (Table 5). For all but three sites, SS structures had lower costs per square foot. The SS culverts cost per square foot averaged 66 percent less (range 52–89 percent) than R-I-K (Table 5). For bridge-type structures, three SS had costs 21 to 53 percent less than R-I-K while three SS costs were higher. Costs of two SS structures were modestly higher (6 and 29 percent), and one SS was substantially higher (158 percent). Overall, the substantial increase in open area for SS structures comes at a much lower cost per square foot.

The increased cost for SS structures was substantially less for the 11 culvert sites than the 6 bridge-type sites. For culverts, the R-I-K 2020 average cost was $18,384 (range $14,827–$22,245) and the SS average was $22,251 (range $17,230–$27,405) or only 21 percent higher on average. Site 10 had the largest increase at 52 percent, but the water way opening increased 1,338 percent because the existing structure was so severely undersized. Sites where SS can be accomplished with culverts less than 10 feet wide provide very substantial infrastructure and environmental benefits for the moderately higher cost.

Table 5.1.—Characteristics and for stream simulation and replace-in-kind (R-I-K) structures, Chequamegon-Nicolet National Forest, 2016 flood area

Site #	Stream	In-kind structure type	Stream simulation (SS) structure type	Bankfull width (ft)	R-I-K culvert width (ft)	SS culvert width (ft)	Culvert Width Increase %	R-I-K constriction ratio	SS constriction ratio	R-I-K open area (sq ft)	Open Area Increase %	Increase %
6	Unnamed tributary to (UNT) Whisky, West	Pipe arch	Pipe arch	5.6	3.50	5.33	52	0.63	0.95	6.4	14.3	123
2	20 Mi	Circ cmp	Pipe arch	7.0	3.00	5.92	97	0.43	0.85	7.1	17.6	148
7	UNT Morgan Falls	Circ cmp	Pipe arch	5.0	4.00	7.25	81	0.80	1.45	12.5	32.1	157
12	UNT Marengo, South	Pipe arch	Pipe arch	4.2	4.08	7.25	78	0.97	1.73	9.6	32.1	234
8	UNT Whisky, Middle	Pipe arch	Pipe arch	5.0	4.08	7.25	78	0.82	1.45	8.7	32.1	269
10	UNT Marengo	Pipe arch	Pipe arch	4.0	1.75	5.33	205	0.44	1.33	1.6	23.0	1338
9	20 Mi	Pipe arch	Pipe arch	4.4	2.92	5.92	103	0.66	1.34	4.4	17.6	300
11	UNT Trout East	Pipe arch	Pipe arch	6.0	3.50	7.25	107	0.58	1.21	5.7	32.1	463
15	UNT Marengo, North	Pipe arch	Pipe arch	8.6	5.33	7.25	36	0.62	0.84	12.8	32.1	151
3	UNT Hawkins	Circ cmp	Pipe arch	7.0	3.00	7.92	164	0.43	1.13	7.1	27.0	280
21	UNT WF Chippewa	Pipe arch	Pipe arch	8.6	4.75	8.58	81	0.55	1.00	11.4	42.4	272
13	McCarthy	Pipe arch	Alum box	9.7	6.92	10.00	45	0.71	1.03	25.2	40.2	60
22	Brush	Circ cmp	Alum box	15.3	5.00	16.00	220	0.33	1.05	16.6	59.5	258
14	Preemption	Pipe arch	Ellipse	10.5	4.75	12.00	153	0.45	1.14	11.4	60.0	426
24	18 Mi	Circ cmp	Alum box	13.0	3.50	16.17	362	0.27	1.24	9.6	72.3	653
16	Trout	Circ cmp	Bridge	15.9	4.00	30.00	650	0.25	1.89	12.5	128.0	924
5	Hawkins	Pipe arch	Alum box	16.0	6.92	24.58	255	0.43	1.54	25.2	105.0	317

Notes: Culverts are above the dashed line and bridge-type structures below. Circ cmp = circular corrugated metal pipe, alum = aluminum. Culverts are above the dashed line and bridge-type structures below.

Table 5.2.—Characteristics for 17 stream simulation and replace-in-kind (R-I-K) structures, Chequamegon-Nicolet National Forest, 2016 flood area

Site #	Stream	In-kind structure type	Stream simulation (SS) structure type	2020 R-I-K construction cost estimate (2020 $)	2020 SS construction cost estimate (2020 $)	Increase in estimated total cost (%)	2020 R-I-K cost per sq ft estimate (2020 $)	2020 SS cost per sq ft estimate (2020 $)	Difference in estimated cost per sq ft (%)
6	Unamed tributary (UNT) Whiskey, West	Pipe arch	Pipe arch	$16,159	$17,230	7	$2,525	$1,205	-52
2	20 Mi	Circ cmp	Pipe arch	$15,794	$18,111	15	$2,224	$1,029	-54
7	UNT Morgan Falls	Circ cmp	Pipe arch	$17,436	$19,009	9	$1,395	$592	-58
12	UNT Marengo, South	Pipe arch	Pipe arch	$19,307	$20,844	8	$2,011	$649	-68
8	UNT Whiskey, Middle	Pipe arch	Pipe arch	$19,030	$22,119	16	$2,187	$689	-68
10	UNT Marengo	Pipe arch	2-Pipe arch	$14,827	$22,466	52	$9,267	$977	-89
9	20 Mi	Pipe arch	Pipe arch	$18,762	$22,938	22	$4,264	$1,303	-69
11	UNT Trout, East	Pipe arch	Pipe arch	$17,979	$23,575	31	$3,154	$734	-77
15	UNT Marengo, North	Pipe arch	Pipe arch	$22,245	$24,069	8	$1,738	$750	-57
3	UNT Hawkins	Circ cmp	Pipe arch	$19,380	$27,000	39	$2,730	$1,000	-63
21	UNT WF Chippewa	Pipe arch	Pipe arch	$21,303	$27,405	29	$1,869	$646	-65
13	McCarthy	Pipe arch	Alum box	$25,664	$52,708	105	$1,018	$1,311	29
22	Brush	Circ cmp	Alum box	$24,155	$64,050	165	$1,455	$1,076	-26
14	Preemption	Pipe arch	Ellipse	$27,538	$68,573	149	$2,416	$1,143	-53
24	18 Mi	Circ cmp	Alum box	$17,956	$106,602	494	$1,870	$1,474	-21
16	Trout	Circ cmp	Bridge	$20,081	$217,326	982	$1,606	$1,698	6
5	Hawkins	Pipe arch	Alum box	$25,889	$278,710	977	$1,027	$2,654	158

Notes: Culverts are above the horizontal dashed line and bridge-type structures below. Circ cmp = circular corrugated metal pipe, alum = aluminum.

For the six bridge-type sites, the R-I-K 2020 average cost was $23,547 (range $17,956–$27,538) and the SS average was $131,328 (range $52,708–$278,710) or 479 percent higher on average (range 115–1,205 percent). These higher costs can be attributed to the following factors: undersized existing structures (all but one had constriction ratios <0.5), larger streams that required bridge-type structures in place of undersized culverts, and use of materials that would extend structure life. In addition, construction of stream banks and bed was necessary at three sites to provide aquatic organism passage. Four of the SS bridge-type structures used aluminum box culverts with headwalls and wingwalls which added to cost but shortened structure length. A typical R-I-K corrugated metal pipe has an expected service life of 25 to 50 years; a polymer coating may extend that to 75 years. But R-I-K structures have a high to moderate likelihood of failing before reaching the end of their service life. Aluminum box culverts have an expected service life of 75 years or more which would increase lifespan in the range of 50 to 200 percent over a corrugated metal pipe given their greater flood resiliency.

Given the size and wide range of cost increases for bridge-type structures, it is worthwhile to review the specifics of each site. McCarthy at FR 184 (site 13), Brush at FR 354 (site 22), and site 24 had circular culverts replaced with aluminum box culverts. McCarthy and Brush had cost increases of 105 and 165 percent, respectively, or two to three times the R-I-K cost. The constriction ratio was increased to just over 1.0 and the open area was increased by 160 and 358 percent, respectively. The cost of the SS structure at site 24 was nearly seven times the R-I-K cost, but the constriction ratio was increased from 0.27 to 1.24 and the open area was increased by 653 percent. The road was also lifted four feet at that site to accommodate the structure and reduce slope to the crossing to solve a severe erosion and sedimentation problem. The SS for site 14 required the construction of stream banks and bed through a 12-foot wide elliptical multiplate pipe with an increase of 149 percent over the R-I-K cost. The new structure increased the constriction ratio from 0.45 to 1.14 and open area by 426 percent and restored aquatic organism passage.

The SS for site 16 replaced a 4.0-foot circular culvert with a 30-foot span bridge over a constructed stream channel at a 2020 cost of $217,326, a 982 percent increase over the R-I-K cost. This SS increased the constriction ratio from 0.25 to 1.89, increased open area by 924 percent and restored aquatic organism passage. The site has a bankfull width of 15.9 feet. It was part of a larger project to reduce erosion and sedimentation including road ditching, installation of numerous cross-drain culverts and paving. A unique aspect of this crossing design was to consider the potential for higher flood flows in the future. A short distance upstream from the crossing, the stream and narrow floodplain are filled with large waste rock, several feet in diameter, from a historical quarry. While the stream flows through this material, the rock ponds water upstream at baseflow, attenuating flood flows. This attenuation likely helped the narrow 4.0-foot culvert persist at that site. Hydraulic modeling performed for the new bridge assumed the rock might be removed in the future to restore the stream and this, in part, led to the 30-foot span bridge and high cost. It is also highly likely that a 4.0-foot diameter R-I-K culvert would not have been able to obtain appropriate environmental permits.

The SS for site 5 replaced two 4.0-foot diameter circular culverts with a 24.6-foot span aluminum box culvert with a constructed stream channel at a 2020 cost of $278,710. This was a 977 percent cost increase over a R-I-K structure. SS increased the constriction ratio from 0.25 to 1.89 and open area by 317 percent and restored aquatic organism passage.

The two 4.0-foot culverts had failed nearly every year, were an impediment to fish passage, and it is likely that officials would not have been able to obtain appropriate environmental permits for a R-I-K structure. This SS site failed during the 2016 flood because of the extreme flood at that site (1,019 cfsm) and because of several site conditions contributing to vulnerability. A concrete structure would have been more survivable at this site given its vulnerable watershed and site characteristics.

While a life cycle cost analysis has not been conducted, this analysis provides insight into costs and benefits that can be anticipated when SS design leads to replacement of traditionally undersized structures. When SS can be accomplished with culverts less than 10 feet wide, the costs will be modestly higher for these structures, which increase culvert lifespan, reduce maintenance, and provide desired ecological benefit. When SS requires the use of bridge-type structures 10 feet wide or greater, the costs will be much higher, especially for previously undersized culverts.

Hydraulic modeling indicates that 11 of the 16 surviving SS structures would have failed if they had been R-I-K types, as they would have overtopped at flows that were only 7 to 38 percent of the SS structures. Four R-I-K structures would have overtopped at flows that were 50 to 61 percent of the SS structures. These R-I-K structures, while they had a somewhat better chance to survive, likely would have failed too. One R-I-K structure overtopped at the same flow as the SS structure and likely would have survived. Thus, at least 11 and as many as 15 of the 16 surviving SS sites would have failed during the 2016 flood had they been R-I-K structures, providing substantial replacement cost savings at those sites.

Effect of Watershed Characteristics on Flood Peaks

An initial review of the different flood magnitudes indicated watershed characteristics would explain much of the observed variability. Using LTAs, it was apparent these landform units correlated well with differences in flood magnitude (Fig. 2). In general, the highest peak flood flows occurred in PG, followed by VMM, CWTO, GD, and CRO. These responses help to illustrate how watersheds that are more susceptible to rapid runoff are also more likely to experience greater increases in flood flows than those with more attenuated runoff. This result was not surprising since the PG has always been regarded as most responsive to runoff because of shallow silt soils over bedrock, greater relief, and less storage compared to LTAs consisting of sandy outwash that have high infiltration rates and low runoff potential. What was surprising was the magnitude of difference in flood response. Most watersheds in PG had flood flows in the range of 500 to 1,200 cfsm and higher; those in CWTO and CRO were less than 60 cfsm, with the remainder between these two (Fig. 2).

To better characterize flood response by landform, each watershed was classified according to its dominant LTAs (Table 6). The total number of classes were minimized by lumping LTAs with similar characteristics and flood magnitudes. This resulted in three main classes: PG/VMM (n=16), CWTO/VMM (n=5), and CWTO (n=5). GD is similar to CWTO and was considered so in classification. CRO has the lowest runoff response in the flood area but only occurred in one watershed and was lumped with CWTO.

Though total storm rainfall was substantial at all sites, it was not uniform across sites and differences in rainfall most likely influenced peak flows to some degree. The average storm rainfall for all watersheds was 8.08 inches (SE = 0.17, median = 8.13 inches) with means for each LTA group of 7.44 (SE = 0.27), 7.63 (SE = 0.56), and 8.42 (SE = 0.17) for CWTO, CWTO/VMM, and PG/VMM, respectively. Site 22 in CWTO/VMM had the lowest rainfall of 5.38 inches which had a strong influence on that LTA group mean with only five observations. Analysis of variance confirmed that rainfall differed by LTA group (appendix) with CWTO significantly less then PG/VMM (Fig. 7A, appendix). The median values displayed in Figure 7A were 7.74, 8.17, and 8.27 inches for CWTO, CWTO/VMM, and PG/VMM. The one-way ANOVA was performed using a reflected $\log_{10}$ transformation to meet homogeneity of variance and normality of residual assumptions.

Table 6.1.—Peak flood flow estimates, watershed characteristics, and landtype associations for 26 road-stream crossing sites on the Chequamegon-Nicollet National Forest, July 2016 flood

Site #	Stream	Drainage area (sq mi)	Ratio of 2016 peak flood flow to 0.2% annual exceedance probability peak flow	Peak discharge (cfsm)	Rainfall (in)	Main channel slope (ft/mi)	Storage (%)	Drainage density (mi/sq mi)
1	Unnamed tributary to (UNT) Morgan	0.51	6.3	2,629	8.98	218	3.5	
2	20 Mi	0.78	10.5	1,712	8.26	56	12.6	1.11
3	UNT Hawkins	0.94	3.2	1,223	8.80	172	1.3	0.99
4	UNT Whisky, East	1.03	7.6	1,103	9.20	58	17.2	1.15
5	Hawkins	3.37	4.8	1,019	8.94	109	6.2	1.90
6	UNT Whisky, West	0.21	4.0	762	9.10	59	14.7	
7	UNT Morgan Falls	0.29	1.9	759	9.02	36	22.4	
8	UNT Whisky, Middle	0.65	4.9	692	9.20	59	23.7	1.27
9	20 Mi	0.31	3.6	677	8.27	75	17.1	1.08
10	UNT Marengo	0.21	1.1	657	8.55	247	0.4	3.82
11	UNT Trout, East	0.61	3.1	598	8.02	57	10.0	3.08
20	18 Mi (lower 18%)	3.78	3.0	508	7.06	90	9.8	1.53
13	McCarthy	1.35	3.0	348	7.64	38	23.0	1.28
14	Preemption	2.67	1.6	262	7.39	85	11.6	1.25
16	Trout	5.09	1.8	221	8.08	82	26.0	1.03
17	Mineral Lake Inlet	2.09	2.1	215	8.15	34	18.4	0.68
12	UNT Marengo, South	0.37	2.9	581	8.11	57	9.2	
15	UNT Marengo, North	1.43	1.9	245	8.11	60	21.8	0.56
18	UNT Brunsweiler	1.28	2.4	195	8.32	31	43.0	1.90
19	UNT Marengo	0.69	1.1	145	8.22	55	24.1	0.71
22	Brush	6.84	1.3	60	5.38	16	50.0	0.28
21	UNT WF Chippewa	2.16	1.7	102	7.16	15	34.0	0.97
23	Porcupine	5.83	0.9	60	8.01	23	32.1	0.61
24	18 Mi	10.4	1.3	27	8.00	15	23.9	0.47
25	WF Chippewa	11.5	0.6	25	7.48	13	36.6	0.53
26	Red Ike	5.02	0.2	9	6.57	8	35.6	0.36

Note: Horizontal dashed lines delineate similar LTA groups. Storage = storage in the form of lakes and wetlands.

Simple linear regression of peak flow on rainfall for each LTA group illustrates both the effect of rainfall on each group and differences in runoff among groups (Fig. 8A). The slopes of the three regression lines were not significantly different indicating that within each group the response to differences in rainfall on peak flow was similar. Differences in the intercepts indicate the substantial difference in peak flows among the groups.

Table 6.2.—Peak flood flow estimates, watershed characteristics, and landtype associations for 26 road-stream crossing sites, Chequamegon-Nicolet National Forest, July 2016 flood

Site #	Stream	Saturated hydraulic conductivity (KSAT) upland all layers (in/hr)	KSAT surface layer (in/hr)	Landtype association (LTA)	LTA group
1	Unnamed tributary to (UNT) Morgan	2.3	9.5	PG	PG/VMM
2	20 Mi	2.0	7.9	PG	PG/VMM
3	UNT Hawkins	1.1	7.9	PG	PG/VMM
4	UNT Whisky, East	2.7	9.5	PG/VMM	PG/VMM
5	Hawkins	1.9	8.6	PG	PG/VMM
6	UNT Whisky, West	2.8	8.7	PG	PG/VMM
7	UNT Morgan Falls	2.6	10.2	PG	PG/VMM
8	UNT Whisky, Middle	2.8	8.1	PG/VMM	PG/VMM
9	20 Mi	1.9	8.0	PG	PG/VMM
10	UNT Marengo	2.0	7.9	PG	PG/VMM
11	UNT Trout East	1.5	9.7	PG	PG/VMM
20	18 Mi (lower 18%)	2.6	7.0	VMM/PG	PG/VMM
13	McCarthy	1.9	10.5	PG	PG/VMM
14	Preemption	3.0	5.8	PG	PG/VMM
16	Trout	2.0	9.4	PG	PG/VMM
17	Mineral Lake Inlet	2.0	9.5	VMM/PG	PG/VMM
12	UNT Marengo, South	7.4	5.1	VMM/CWTO	CWTO/VMM
15	UNT Marengo, North	1.5	9.2	CWTO/VMM	CWTO/VMM
18	UNT Brunsweiler	2.6	10.0	CWTO/VMM	CWTO/VMM
19	UNT Marengo	2.5	9.8	VMM	CWTO/VMM
22	Brush	3.3	6.0	VMM/GD	CWTO/VMM
21	UNT WF Chippewa	3.9	6.5	CWTO	CWTO
23	Porcupine	2.7	5.2	CWTO	CWTO
24	18 Mi	7.5	6.3	CRO/CWTO	CWTO
25	WF Chippewa	5.0	6.7	CWTO	CWTO
26	Red Ike	6.6	6.4	CWTO	CWTO

Notes: Horizontal dashed lines delineate similar LTA groups. PG = Penokee Goegebic Iron Range, VMM = Valhalla Marenisco Moraines, CWTO = Chequamegon Washed Till & Outwash. GD = Glidden Drumlins, CRO = Cable Rolling Outwash.

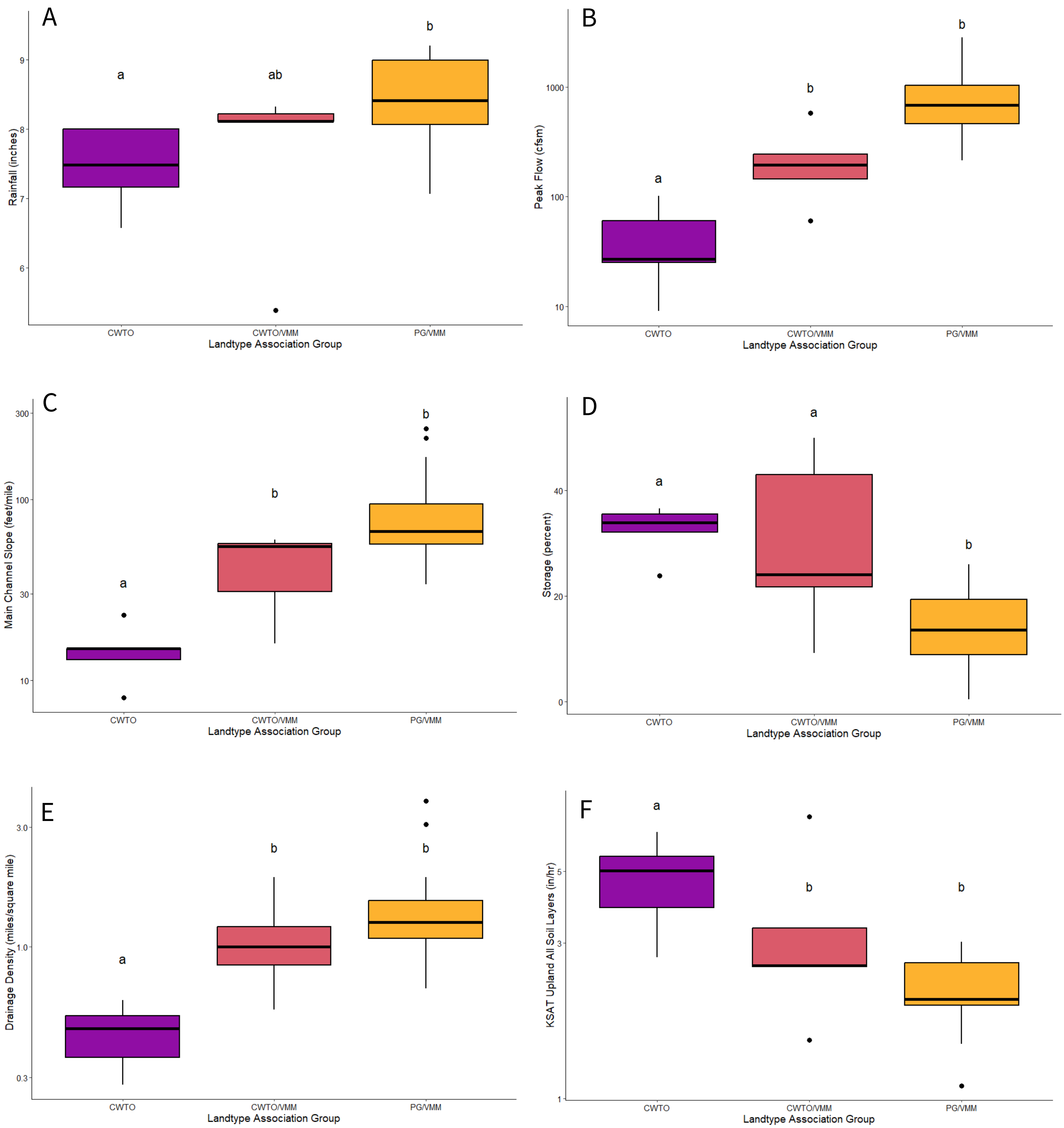

Figure 7.—Differences in precipitation, peak flow, and four watershed characteristics by landtype association group, where CWTO = Chequamegon Washed Till & Outwash, VMM = Valhalla/Marenisco Moraines, PG = Penokee/Gogebic Iron Range, KSAT = saturated hydraulic conductivity, and cfsm = cubic feet per second per square mile. Letters designate statistical differences (p = 0.05) from Tukey-Kramer pairwise means comparisons.

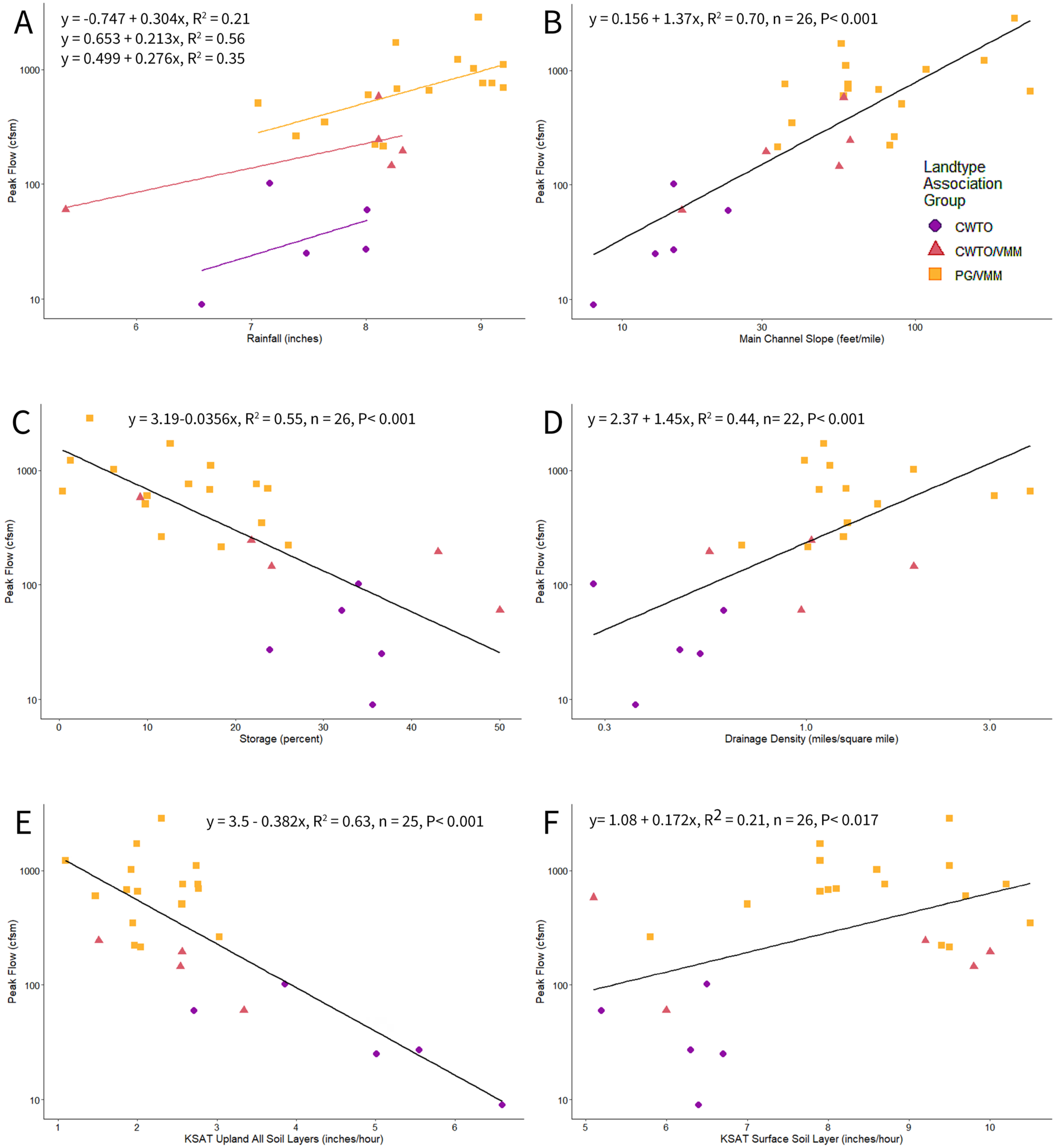

Figure 8.—Relation of peak flow to rainfall and five watershed characteristics, where CWTO = Chequamegon Washed Till & Outwash, VMM = Valhalla/Marenisco Moraines, PG = Penokee/Gogebic Iron Range, KSAT = saturated hydraulic conductivity, and cfsm = cubic feet per second per square mile.

The mean peak flow varied considerably among LTA groups with PG/VMM having the highest runoff response followed by CWTO/VMM and then CWTO (Table 7). Without considering rainfall, all three LTA groups were significantly different ($p = 0.05$). However, when rainfall was used as a covariate, results of a one-way analysis of covariance showed a significant difference among LTA groups but only CWTO was significantly different ($p = 0.05$) from CTWO/VMM and PG/VMM (Table 7, Fig. 7A, appendix). Because CWTO and CTWO/VMM received less rainfall on average and PG/VMM received more, the rainfall covariate adjusted peak flow means accordingly with CWTO increasing 42 percent, CWTO/VMM increasing 30 percent, and PG/VMM decreasing 18 percent (Table 7). Despite these adjustments, there were still substantial differences in mean peak flows with PG/VMM over 11 times higher than CWTO and more than two times higher than CWTO/VMM; CWTO/VMM was more than five times higher than CWTO (Table 7). These analyses helped confirm that landform and associated watershed characteristics were the major cause of observed differences in peak flow and differences in rainfall amount played a more minor role for the 2016 storm.

The best illustration of this point is a comparison of site 24 and site 16. Most of the watershed for site 24 was located predominantly in CRO while the watershed for site 16 was in PG (Fig. 2). They both received nearly identical rainfall amounts (8.00 vs. 8.08 inches) yet site 24 had a peak flow of 27 cfsm versus 221 cfsm for site 16 (Table 6). The difference in watershed characteristics resulted in a peak flow that was more than eight times higher for site 16.

Several watershed characteristics were evaluated to help explain differences in peak flows both between LTA groups and within groups. These differences provide insight into factors that can be used to classify watersheds according to vulnerability to future increases in flood flows. Watershed characteristics that were readily available and evaluated in this analysis were MCS, STO, DD, and two forms of KSAT.

Table 7.—Mean peak flows by landtype association group with and without rainfall as a covariate

Landtype Association Group	Back-calculated peak flow mean (cfsm), no covariate	Back-calculated peak flow mean (cfsm) with rainfall covariate
Penokee Goegebic Iron Range/Valhalla Marenisco Moraines	668 a	549 a
Chequamegon Washed Till & Outwash/Valhalla Marenisco Moraines	189 b	245 a
Chequamegon Washed Till & Outwash	33 c	47 b

Notes: The presented peak flow means were back-calculated from $\log_{10}$ means; cfsm = cubic feet per second per square mile. Letters indicate significant differences ($p = 0.05$) from Tukey-Kramer pairwise means comparisons.

Two other variables that were readily available but not used in the analysis of peak flows were soil permeability (inches/hour) and mean annual snowfall (inches). These variables were used to estimate flood frequencies for each site with the USGS regression equations. Soil permeability was not used because it was 1.65 inches/hour for all watersheds and soil data providing KSAT values were obtained as a replacement. Mean annual snowfall was not used because it was deemed not applicable to runoff from a July rainstorm.

MCS is commonly used in USGS regression equations for predicting flood flows because it provides a measure of relief and how rapidly water can drain from the watershed. MCS explained the most variability in peak flows ($R^2 = 0.70$; Fig. 8B). The overall regression was statistically significant, with peak flows increasing as MCS increased across the range of 8 to 247 feet/mile. MCS significantly varied among LTA groups; it was significantly less for CWTO than for CWTO/VMM and PG/VMM (Fig. 7C; appendix). MCS is determined from specific points on a stream after a watershed is delineated. As such, it is relatively easy to determine MCS for a road-stream crossing. But it is less practical for classifying broader land areas because of the uncertainty of which watersheds to delineate and at what scale.

In turn, MCS was also significantly related with STO ($R^2 = 0.70$; Fig. 9), DD ($R^2 = 0.57$; Fig. 9B), and KSAT Up ($R^2 = 0.57$; Fig.9C). Increasing MCS was associated with increasing DD and decreasing STO and KSAT Up. These relations demonstrate that these factors can account for some of the peak flow variability associated with MCS.

STO is the area of a watershed in lakes and wetlands which can attenuate floods by temporarily storing flood waters. Like MCS, STO explained a substantial amount of the variability in peak flows ($R^2 = 0.55$; Fig. 8C). The overall regression was statistically significant, with peak flows decreasing as STO increased across the range of 0.4 to 50 percent. STO significantly differed among LTA groups with STO for CWTO and CWTO/VMM greater than STO for PG/VMM (Fig. 7D; appendix).

STO was also significantly related with DD ($R^2 = 0.50$; Fig. 9D) and KSAT Up ($R^2 = 0.29$; Fig. 9E). STO increased as DD decreased and as KSAT Up increased. STO is easily determined in GIS from a wetland and hydrography layer and is therefore a valuable parameter for classifying landscape vulnerability to increases in flood flows. Of note is the strong relationship between STO and MCS ($R^2 = 0.70$) indicating STO can partially account for MCS effects.

DD is the density of streams per unit area and represents how quickly surface water can drain from an area. DD also accounted for a substantial amount of the variation in peak flow response among 22 watersheds ($R^2 = 0.44$; Fig. 8D). The overall regression was statistically significant, with peak flows increasing as DD increased across the range of 0.28 to 3.82 miles/square mile. DD was significantly different among LTA groups and was less for CWTO than for CWTO/VMM and PG/VMM (Fig. 7E; appendix). DD is easily determined in GIS from a hydrography layer and was strongly correlated with peak flow response; therefore, it was considered a useful parameter for classifying landscape vulnerability to increases in flood flows.

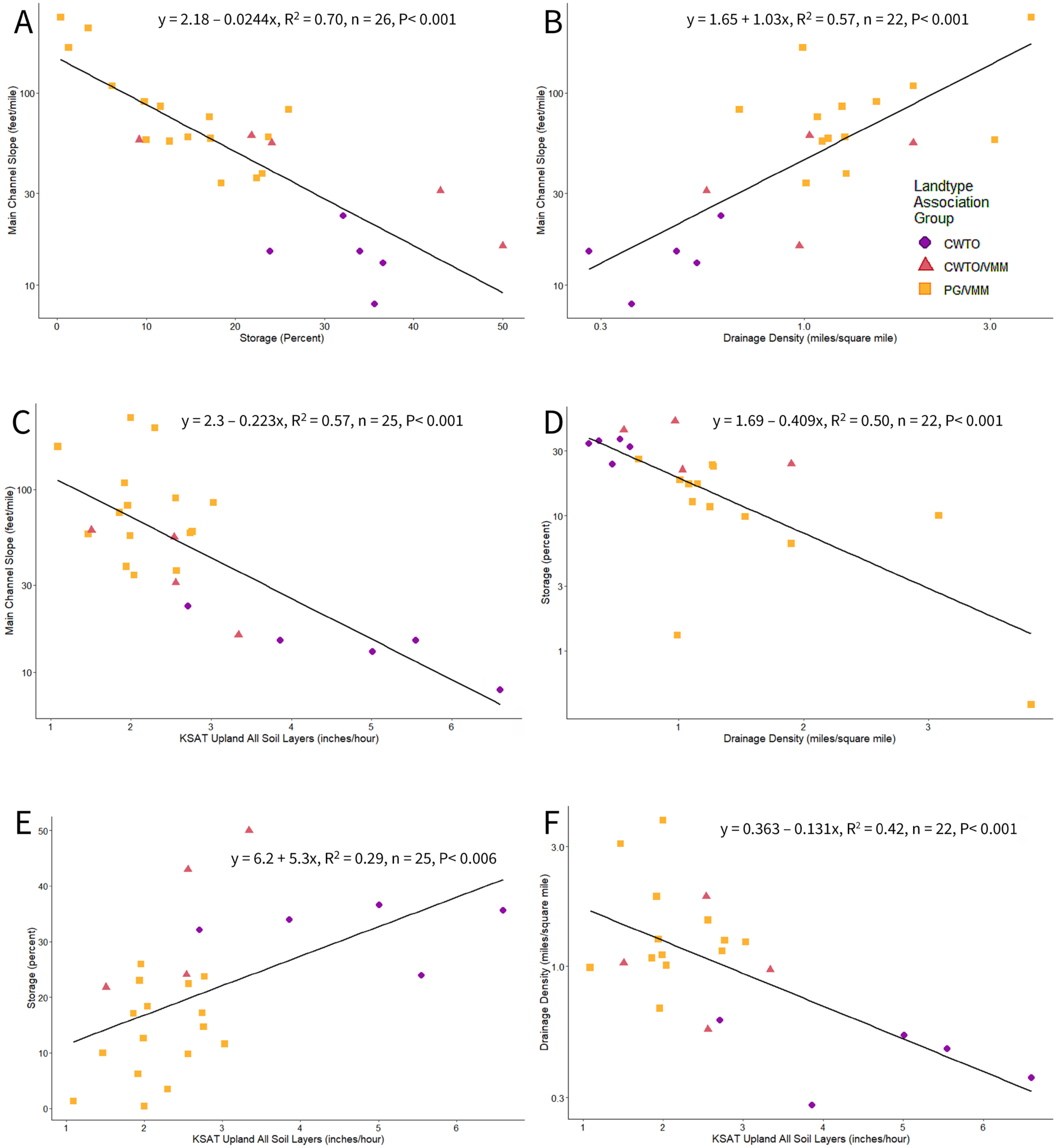

Figure 9.—Linear regression relationships of four watershed characteristics to each other. Where CWTO = Chequamegon Washed Till & Outwash, VMM = Valhalla/Marenisco Moraines, PG = Penokee/Gogebic Iron Range, and KSAT = saturated hydraulic conductivity.

DD was also significantly related with MCS (R^2=0.57; Fig. 9B), STO (R^2=0.50; Fig. 9D), and KSAT Up (R^2 = 0.42; Fig. 9F). Increasing DD was associated with increasing MCS and decreasing STO and KSAT Up. These significant relations indicate DD can partially account for the effects of all three of these watershed characteristics on peak flows.

KSAT is the ease with which water can move through a saturated medium and represents the rate that water can infiltrate the ground surface and move downward. KSAT Up using all soil layers accounted for substantial variation in peak flow response (R^2 = 0.63; Fig. 8E). The overall regression was statistically significant with peak flows decreasing as KSAT Up increased across the range of 1.09 to 6.60 inches/hour. However, 22 of the 25 KSAT Up values were less than 3.86 inches/hour, while the other three were over 5.0 inches/hour (Table 6). KSAT Up significantly differed among LTA groups and was greater for CWTO than for PG/VMM (Fig. 7F; appendix). Only upland soils were used for this KSAT parameter because wetlands were considered to serve primarily as storage areas that receive runoff, overriding the importance of KSAT in those polygons regarding flood peaks. All soil layers were used because restrictive layers below the surface can impede the downward movement of water and lead to more rapid runoff.

The initial KSAT parameter based on the soil surface layer for the entire watershed yielded substantially different results than KSAT Up. The values ranged from 5.1 to 10.5 inches/hour and were much larger than KSAT Up values already described (Table 6). They also had an unexpected relationship with the 2016 flood flows that had no clear physical basis. The relation, while weak (R^2 = 0.21) but statistically significant (p = 0.017), was positive (Fig. 8F). This result defied expectations, since greater saturated hydraulic conductivity should lead to greater infiltration and lower flood flows. Therefore, this parameter was dropped from the analysis. This unexpected outcome may be the result of limiting KSAT to the surface layer only. Some soils may have a porous surface but a more restrictive subsoil that still limits infiltration. In addition, the storage effect of wetlands may override the effect of KSAT in those areas.

In addition to these four watershed characteristics which substantially affect flood response, unique aspects of each watershed also affected flood response. For example, site 26 had the lowest flood flow response in part because it had the lowest MCS, high STO, and high KSAT, but also because the culvert was located at the outlet of Red Ike Lake (38 acres). This caused the lake to act like a flood control reservoir, and while the lake elevation increased 1.4 feet, it very likely attenuated the flood peak more than expected based solely on the low MCS and high STO. Site16 has a partially obstructed channel a short distance above the bridge caused by waste rock from an adjacent abandoned quarry. Most of the rocks are several feet in diameter and completely cover the channel and floodplain. While the stream flows through this obstruction with some ponding during baseflows, the waste rock probably caused some upstream storage of water and attenuated the flood peak. Consequently, the peak flow was lower than expected for a PG watershed.

Using LTA to Characterize Landscape Vulnerability to Flood Increases in Wisconsin

Given the strong correlation of differences in observed 2016 flood peaks to LTAs and the correlation of many LTA delineation criteria with hydrologic processes, the Wisconsin LTA map was used as a framework to identify differences in vulnerability to future increases in flood flows. Each LTA in the state was placed into one of four vulnerability classes using the first author's professional knowledge while guided by three key attributes: DD, STO, and flood flows for the 1 percent AEP. These attributes were used to classify each LTA as either vulnerable, moderately vulnerable, moderately resilient, and resilient (Fig. 10). A close-up of the vulnerability map for northwest Wisconsin shows the substantial differences in DD and STO (lakes, ponds, wetlands) between the four vulnerability classes (Fig. 11). The purpose of this classification was to provide an example of how landscapes can be classified to assist those working on climate change adaptations for road-stream crossings.

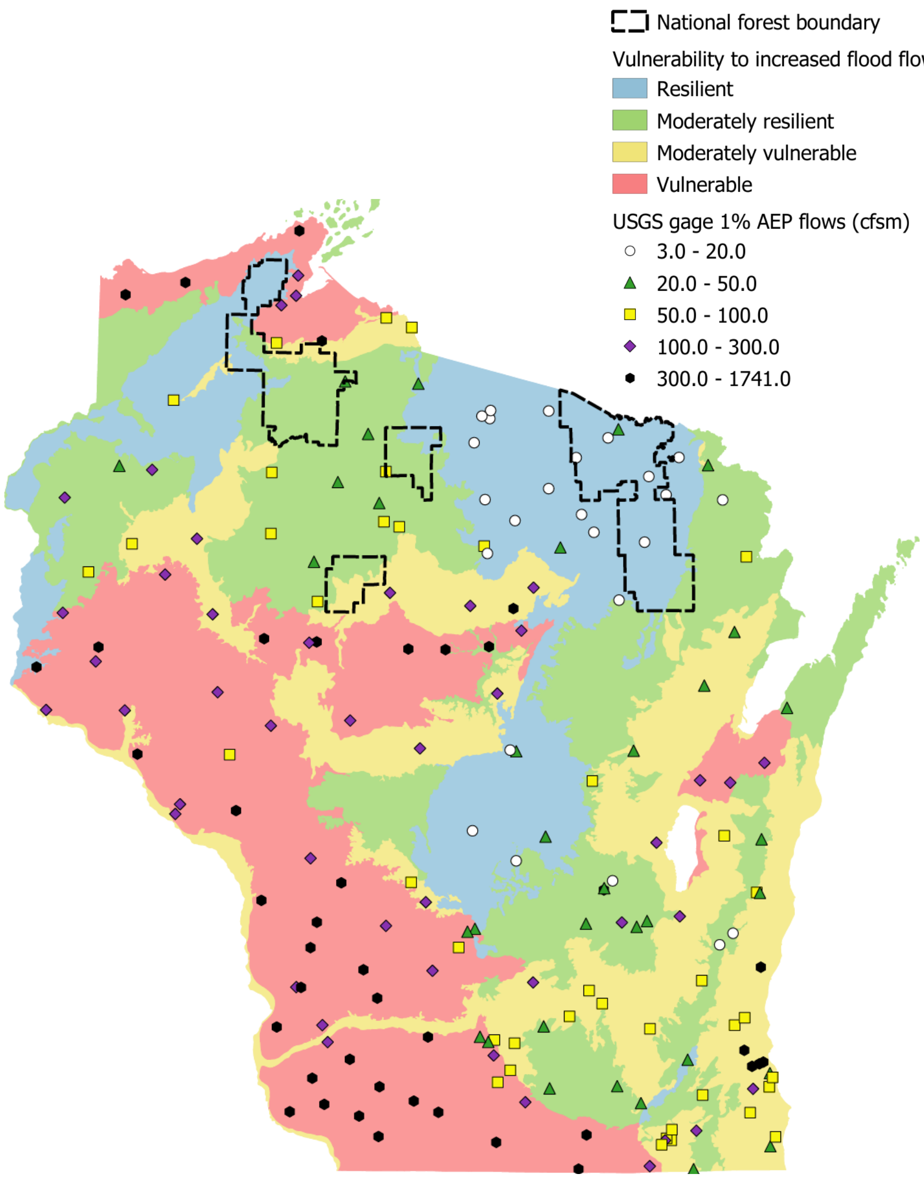

Figure 10.—Vulnerability to increases in flood flows based on classification of landtype associations; USGS = U.S. Geological Survey, AEP = annual exceedance probability, cfsm = cubic feet per second per square mile. Classification based on drainage density, storage, and 1 percent AEP flows for USGS gaging stations less than 75 square miles; scale 1:250,000. Sources: Higgins et al. 2025a, b.

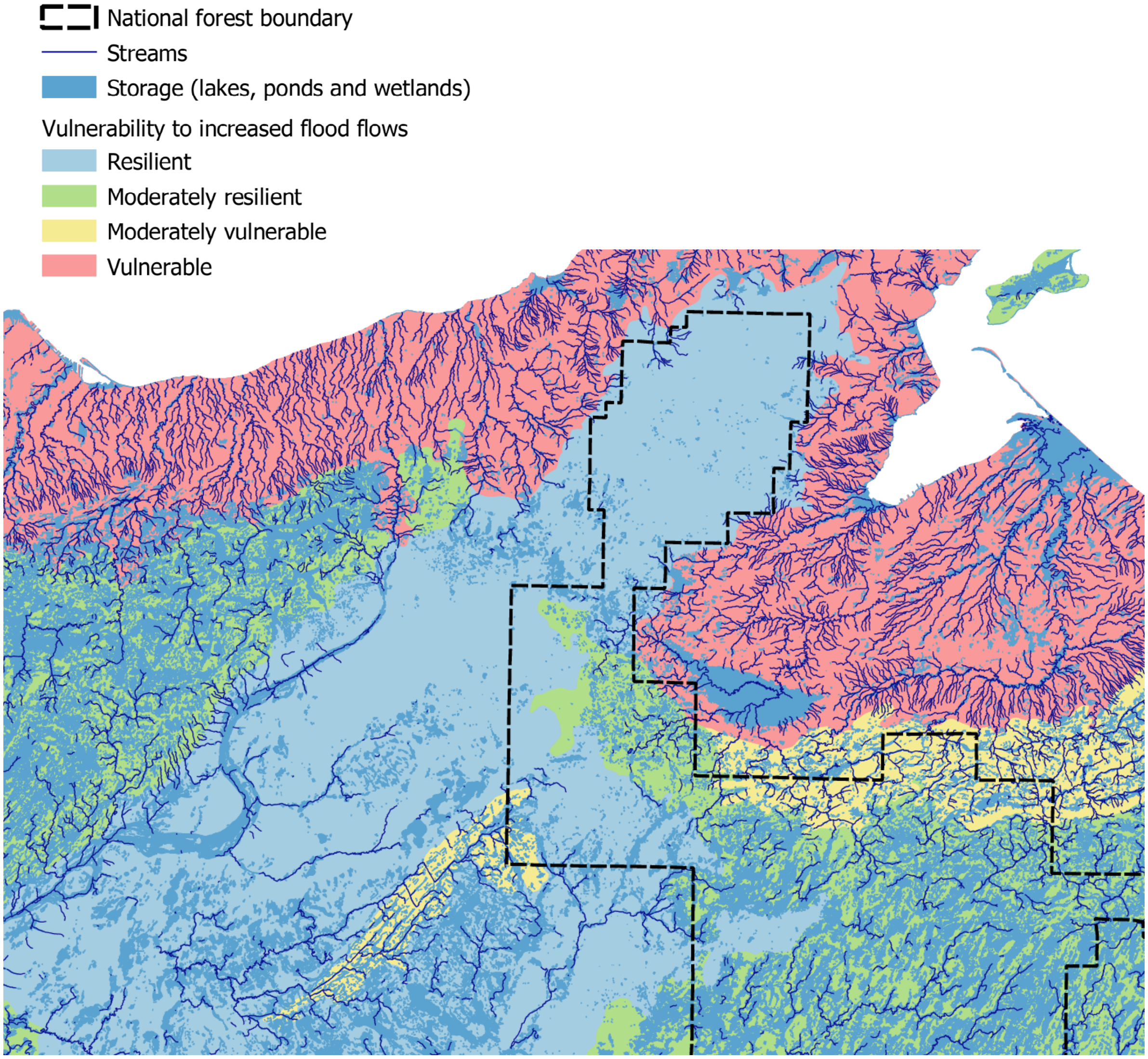

Figure 11.—Map of Wisconsin flood area: an example of differences in drainage density and storage (lakes, ponds, and wetlands) for the vulnerability to increased flood flow classes.

Analysis of the 288 LTAs in Wisconsin (excluding Lake Winnebago) resulted in the classification of 30.2 percent of the state as vulnerable, 22.5 percent as moderately vulnerable, 28.2 percent as moderately resilient, and 19.1 percent as resilient. Most vulnerable areas occur in the driftless region of southwest Wisconsin, the Lake Superior clay till plain, a portion of central Wisconsin, and near the mouth of Green Bay (Fig. 10). Vulnerable areas tend to have steeper terrain, poorly or moderately drained clay or silt soils, bedrock near the surface, or a combination thereof. Moderately vulnerable areas are common in east-southeast Wisconsin and portions of central Wisconsin adjoining vulnerable areas. These areas typically have moderate or somewhat poorly drained silt or loam soils formed in glacial till. They also occur in bedrock-controlled landforms such as the Penokee-Gogebic Iron Range, Smoky Hill Basalt Ridge, Pipestone Hills, and Blue Hills in northwestern Wisconsin. Resilient areas are concentrated in predominantly forested regions of northwest, north-northeast, and central Wisconsin. They typically occur in outwash plains and have excessively or well drained sandy or loamy soils. Moderately resilient areas occur across much of the remaining northern portion of the state and surrounding the central sands region; they are also predominantly forested and have well drained sandy or loamy soils.

The three key variables used in the classification process varied among the vulnerability classes across Wisconsin (Fig. 12). DD ranged from 0.0 to 3.9 mile/square mile for the 288 LTAs (Table 8). The means were 0.65 (SE = 0.04) for resilient, 1.19 (SE = 0.05) for moderately resilient, 1.55 (SE = 0.0.05) for moderately vulnerable, and 2.07 (SE = 0.08) for vulnerable. One-way ANOVA indicated that DD was significantly different among vulnerability classes ($F_{3, 284} = 107.620$, $p < 0.001$) and Tukey-Kramer pairwise comparison, indicated all vulnerability classes significantly differed among each other ($p < 0.01$).

STO ranged from 3 to 93 percent for the 288 LTAs (Table 8) and had two seemingly contrary results. The first was that STO increased as vulnerability decreased except for the resilient class (Fig. 12B). This pattern occurred because many areas with excessively or well drained sands have very few lakes or wetlands, yet the high infiltration capacity makes the landform resilient to increases in flood flows. For this reason, DD was given preference over storage during the classification process. The STO percentages were 21.4 (SE = 1.76), 29.5 (SE = 1.83), 21.8 (SE = 1.51), and 10.0 (SE = 1.87), for resilient, moderately resilient, moderately vulnerable, and vulnerable classes, respectively. One-way ANOVA indicated STO was significantly different among vulnerability classes ($F_{3, 284} = 20.237$, $p < 0.01$) and Tukey-Kramer pairwise comparison indicated that all vulnerability classes significantly differed among each other ($p < 0.01$) except for the resilient and moderately vulnerable classes.

The second surprising relationship was the high STO at two vulnerable LTAs. These included 86 percent for Bibon Marsh and 69 percent for Kakagon Slough. These LTAs are in the Lake Superior clay till plain and were lumped with adjoining LTAs during classification because of the likelihood that tributary streams entering those landforms would be vulnerable to flood flow increases. The next highest STO in the vulnerable class was 26 percent.

Flood flows for the 1 percent AEP for drainage areas less than 75 square miles ranged from 3 to 1,741 cfsm for the 184 gauges (Table 8). The greater the flow per unit of drainage area, the more responsive the watershed is to rapid runoff and more likely it is to experience increased flood flows in the future. While each class had a wide range of flood flows, there were distinct differences with the vulnerable class mostly greater than 150 cfsm, the moderately vulnerable class 65 to 150 cfsm, the moderately resilient class 25 to 65 cfsm, and the resilient class 5 to 25 cfsm (Table 8, Fig. 12C). One-way ANOVA indicated flood flows were significantly different among vulnerability classes ($F_{3, 180} = 110.106$, $p < 0.001$). The back-calculated log10 means for each class, all significantly different from one another as indicated by Tukey HSD pairwise comparison test (all tests $p < 0.01$), were 14.1 (SD = 2.10), 43.9 (SD = 2.11), 101.3 (SD = 2.14), and 297.9 (SD = 2.38) cfsm for resilient, moderately resilient, moderately vulnerable, and vulnerable classes, respectively.

Table 8.—Drainage density, storage, and unit flood flows by landtype association vulnerability to increasing flood flows

Parameter	Landtype association vulnerability	N	Minimum	25th percentile	Median	75th percentile	Maximum
Drainage density (mi/sq mi)	Resilient	67	0.0	0.4	0.6	1.0	1.7
	Moderately resilient	89	0.1	1.0	1.2	1.5	2.7
	Moderately vulnerable	72	0.9	1.3	1.5	1.8	2.8
	Vulnerable	60	0.9	1.7	2.0	2.5	3.9
Storage (%)	Resilient	67	3	12	19	28	93
	Moderately resilient	89	6	17	27	35	93
	Moderately vulnerable	72	2	11	19	30	50
	Vulnerable	60	0	2	6	12	86
1% annual exceedance probability flood (cfsm)[a]	Resilient	27	3	9	14	20	52
	Moderately resilient	38	10	27	42	66	637
	Moderately vulnerable	51	22	62	87	141	700
	Vulnerable	68	43	178	296	522	1,741

Note: cfsm = cubic feet per second per square mile; the horizontal dashed lines delineate different parameters.

[a] The 1% annual exceedance probability flood values represent data collected from U.S. Geological Survey gages with drainage areas <75 square miles.

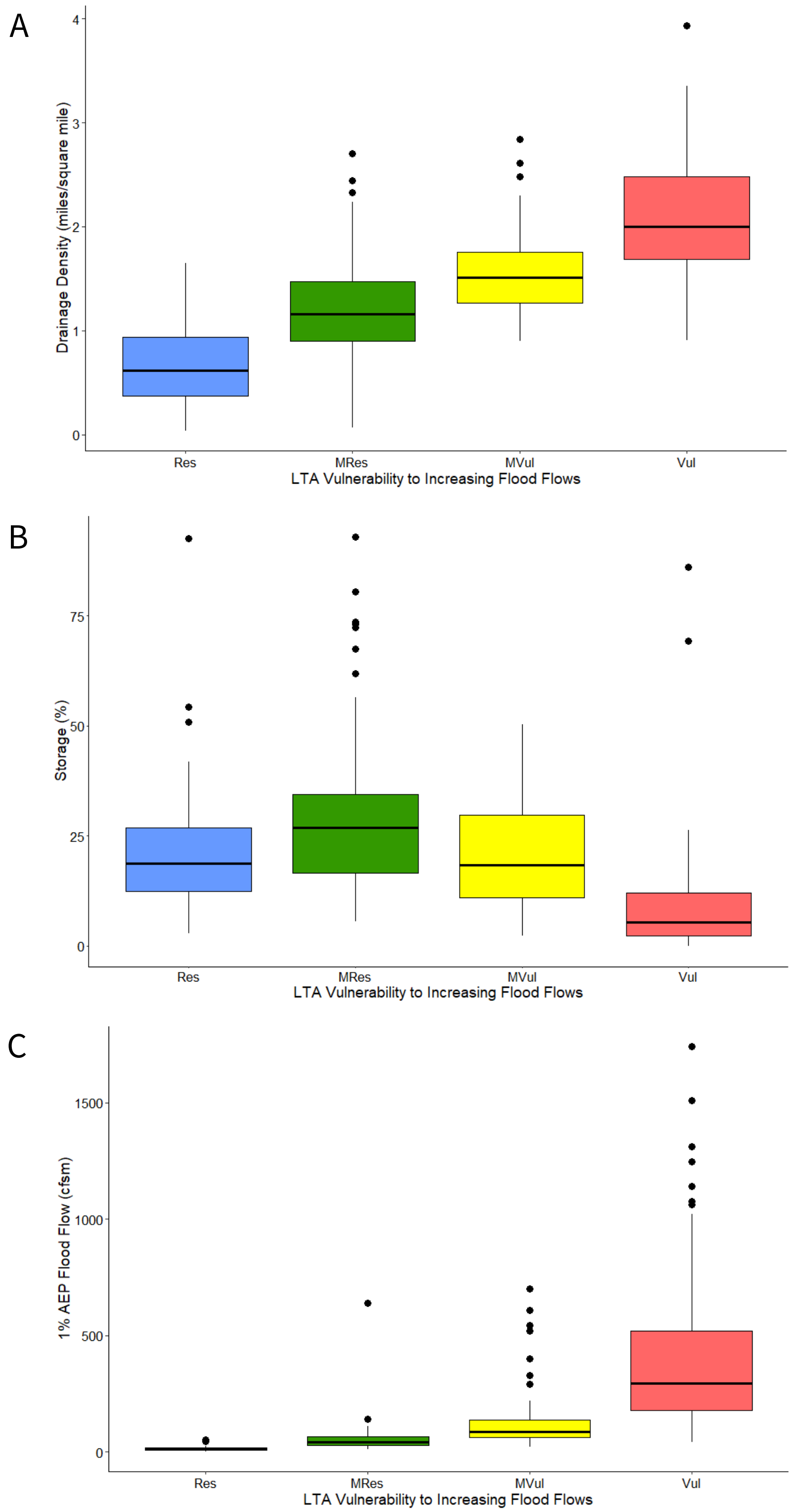

Figure 12.—Drainage density (n=288), storage (n=288), and unit flood flows (n=184) by landtype association (LTA) vulnerability to increasing flood flows across Wisconsin. AEP = annual exceedance probability, cfsm = cubic feet per second per square mile, Res = resilient, MVul = moderately vulnerable, MRes = moderately resilient, Vul = vulnerable.

DISCUSSION

Uncertainty of Peak Flow Estimates

The relative magnitude of flooding at each site, expressed as cfsm, varied greatly among the 26 watersheds. Each flood flow estimate has some level of uncertainty, and one source of error is the accuracy of the high-water mark. In most cases, there was upstream ponding with distinct marks that were considered reasonably accurate, although a few sites presented challenges for getting a consistent reading on each side of the road or floodplain. In addition, no attempt was made to measure the tailwater elevation because of time constraints, difficulty identifying consistent marks, and the need for several readings along the profile's sloping water surface. There can also be errors in the hydraulic modeling because of a limited number of cross-sections and imperfect n-values.

Road erosion is another source of error. At five sites, flow over the road eroded some of the gravel surface and in these cases, erosion was assumed to begin at the high-water mark. If it had begun sooner it would have provided more overflow area and a greater discharge than estimated by the hydraulic model. In these cases, the flood flow may be an underestimate. Flood flow at site 15 was most likely inaccurate, given it had the lowest flood flow rate (245 cfsm), the most erosion (Fig. 5B), and lowest estimated flow over the road (39 percent) of the five sites. If an average of 0.5 foot eroded from that road during the rising limb of the flood over a width of 222 feet (Table 3), and flow had an average velocity of 1.6 feet/second (Table 3), the peak flow could have been 178 cfs higher at that site, amounting to an increase of 50 percent with a peak flow of 370 cfs. Aside from these five sites, potential errors are assumed to be random.

The USGS regression equations provide another possible source of error. For flood frequency area 4, the equivalent standard error of estimate for the 1 percent AEP is 22 percent. Assuming the error for the 0.2 percent AEP is similar, then some of those estimates could be off by 20 percent or more. This would affect the 0.2 percent AEP flood ratio used to characterize the magnitude of the flood at each site. For example, the largest ratio of 10.5 for site 2 could be partially the result of an underestimate by the USGS regression equation. If the 0.2 percent AEP flood estimate were 22 percent higher, the ratio would be 8.6; if it were 35 percent higher, the ratio would be 7.8. For smaller ratios, the change would be proportionately less. Since the ratios ranged from about 1 to 10, the error associated with the USGS regression equations would not appreciably change the conclusions regarding the extreme nature of the flood event, and the substantial difference in response among watersheds. The rainfall data used to adjust the average peak flows for each LTA group also has some uncertainty. While that potential error is unknown, the gridded rainfall values that were obtained for the storm were the most accurate data available and incorporated both radar and gauge data.

Costs

The cost comparison portion of this study was conducted because SS culverts are typically much larger than culverts they replace, and these are designed using traditional H&H, causing many road managers to cite higher SS construction costs as a concern. This section discusses the limited number of economic studies regarding the differences between SS and traditional culverts.

Fiscal benefits associated with ecological design culverts (structure 1.2 times bankfull width set at appropriate depth) were conducted by O'Shaughnessy et al. (2016) using 461 culverts from a road-stream crossing inventory in the Green Bay watershed of Michigan and Wisconsin. The authors evaluated costs for replacement, catastrophic failure, culvert maintenance, and flood damage maintenance, and did not consider ecological costs or benefits. Using lifetime cost benefit analysis, they concluded that nearly half of all ecological design culverts (i.e., essentially SS design) had a net financial benefit. The higher replacement cost was offset by cost savings associated with longer life, flood resiliency, less maintenance, and less flood damage maintenance. The benefits were greatest for small streams less than 5 feet wide where 80 percent had positive net fiscal benefits compared to 30 percent for larger streams. For all sizes, these fiscal benefits do not account for the ecological benefits, which are often a primary objective for SS designed culverts. Yet for smaller culverts, these design approaches can be justified on lower lifetime costs alone. The moderate cost increases associated with construction of SS culverts less than10 feet wide in the 2016 flood area (Table 5) are consistent with the findings of O'Shaughnessy et al. (2016).

A 2008 cost comparison for SS and traditional designs on the Green Mountain National Forest (GMNF) in Vermont found SS designs were expected to increase construction costs between 24 and 61 percent, but actual costs only increased between 9 and 22 percent (Gillespie et al. 2014). The R-I-K costs for the forest's projects were between $92,000 and $107,000, indicating they are likely comparable in size to the bridge-type structures in this study. R-I-K costs for this study ranged between $17,956 and $27,538 (Table 5). Given the disparity with these R-I-K costs, the existing structures at these GMNF sites were likely much closer to bankfull width than those in this study, leading to the much lower increase for SS for GMNF sites.

Data from Alaska in 2006 suggest a 50 percent increase in structure width resulted in a 20 to 33 percent increase in total project cost (R. Gubernick, USDA Forest Service, pers. comm., 2022). The SS costs for culverts (less than 10 feet wide) in this study compare very favorably to those for Alaska, especially considering the substantial increase in structure width, where SS culvert width increased an average of 98 percent (range 36–205 percent) with an average construction cost increase of 21 percent (range 7– 52 percent). When SS bridge-type structures were required to replace existing culverts, width increased an average of 281 percent (range 45–650 percent) and resulted in an average construction cost increase of 479 percent (range 105–982 percent). While these cost increases are markedly higher than those reported for Alaska, the difference is accounted for mostly by the substantial increase in structure width because existing structures were considerably undersized.

Landscape Vulnerability Classes

Using LTAs as the basis for classifying landscape vulnerability was purposeful and had the benefit of using an existing landform classification map (Fig. 10) that correlates well with hydrologic processes. This resulted in classified areas that reasonably reflect their vulnerability to future increases in flood flows, as well as their current differences in runoff rates. The tradeoff is that these units do not necessarily follow watershed boundaries, which is not a problem because once a road-stream crossing site is identified for replacement, a site-specific watershed analysis would be conducted. Classifying LTAs vs. 5th- or 6th-level watersheds also avoids the problem of averaging vulnerability where two or more LTAs with different vulnerabilities occur in one watershed, which would reduce the accuracy of the classification. For long-term planning where many individual sites are being considered or evaluated, the sites can be reviewed quickly on a map to determine if their watersheds contain more than one vulnerability class and can be adjusted accordingly.

The vulnerability classes do not directly account for the effects of land use. However, the following five factors tend to mitigate this possible shortcoming. (1) The 1 percent AEP flood flows from gaging stations across the state that were used in the classification process also reflect existing land uses and therefore partially account for this factor. (2) The size, location, and characteristics of the 288 LTAs across the state likely have some correlation with land use. (3) The vulnerability classes are not necessarily intended for use in urban areas where runoff rates and flood flows are substantially affected by impervious surfaces and modified drainage networks. (4) Conducting a watershed analysis for an individual site is expected to account for land use effects on watershed vulnerability. (5) Prevailing scientific evidence indicates that land use effects tend to decrease with increasing flood magnitude (Grant et al. 2008).

The vulnerability classification, while guided by DD, STO, and unit flood flow rates, was based on professional judgement and not a rigid, data driven, statistical process. This could be viewed as a shortcoming. We note that there is a strong correlation between LTAs and hydrologic processes, and this relationship has been verified by differences in DD, STO, and unit flood flows. Our process resulted in vulnerability classes that provide a reasonable way to identify the relative vulnerability or resilience of areas to increases in future flood flows.

Designing Culverts for Resiliency to Future Floods

Based on our findings, we suggest the following steps to design culverts in rural areas of Wisconsin that will be more resilient to potential flood flow increases from more-intense rainfall in the future. These design considerations were developed based on the survival of most SS culverts to the 2016 flood, the characteristics of those sites, observed differences in flood runoff response among the 26 watersheds, correlation of those differences in flood flows with landscape characteristics and LTAs, extrapolation of those concepts to LTAs across Wisconsin, and general observations of the 2016 flood effects. Our design considerations also draw on more than 20 years of experience designing and constructing over 250 road and trail stream crossings on the Forest.

1. Use SS as the first step for designing flood resilient culverts. SS provides a baseline of flood resiliency through increased capacity, freeboard (i.e., pass 1 percent AEP flood at HW/D <0.8), and ability to minimize plugging by passing wood, ice, and other debris.

2. Use the LTA vulnerability classes described earlier, or develop similar indices, to identify the vulnerability of landforms to flood flow increases. Such vulnerability classes serve several useful purposes. (a) They identify areas that are more or less likely to experience increased flood flows from more intense rainfall in the future. (b) The vulnerability classes can be used to refine design standards for flood resiliency. (c) The information is useful for planning the level of survey, design, and construction work that needs to be accomplished and funded in the future, especially regarding climate change adaptation. (d) This information can help prioritize work and funding requests.

3. For a specific site, determine the general watershed vulnerability class from step 2. Next, perform an assessment including hydrologic analysis of the watershed and a topographic survey of the site. Use the hydrologic analysis to determine whether the watershed vulnerability class should be adjusted to a more resilient class if the watershed has more porous soils, less relief, more storage, or lower estimated flood flows than anticipated; or adjusted to a more vulnerable class if the opposite occurs. Then use the topographic survey to determine whether the site has characteristics which increase vulnerability to failure such as a narrow floodplain (i.e., low entrenchment), required high road fill, steep channel, or poor alignment between road and stream. Sites with a wide floodplain, opportunity for a low road fill, low channel slope, and good alignment between road and stream will be more resilient to failure, especially at overtopping.

4. Use the guidelines provided (Fig. 13) or similarly developed criteria to refine culvert design based on watershed and site vulnerability.

The road-stream crossings from this study were placed into the watershed and site vulnerability classes to provide a brief demonstration of this approach (Table 9). The watershed classifications were based on the vulnerability map (Figs. 10 and 11), which resulted in 1 resilient, 11 moderately resilient, 14 moderately vulnerable, and no vulnerable watersheds. The watershed vulnerability classes could be used as is, but the very high estimated 1 percent AEP flows from the Wisconsin regression equations and minimal storage (Table 1), would justify changing the classification to vulnerable for site 10, site 3, site 1, and site 5. The high upland saturated conductivity, low drainage density, and substantial lake storage would justify reclassifying site 26 to a resilient watershed.

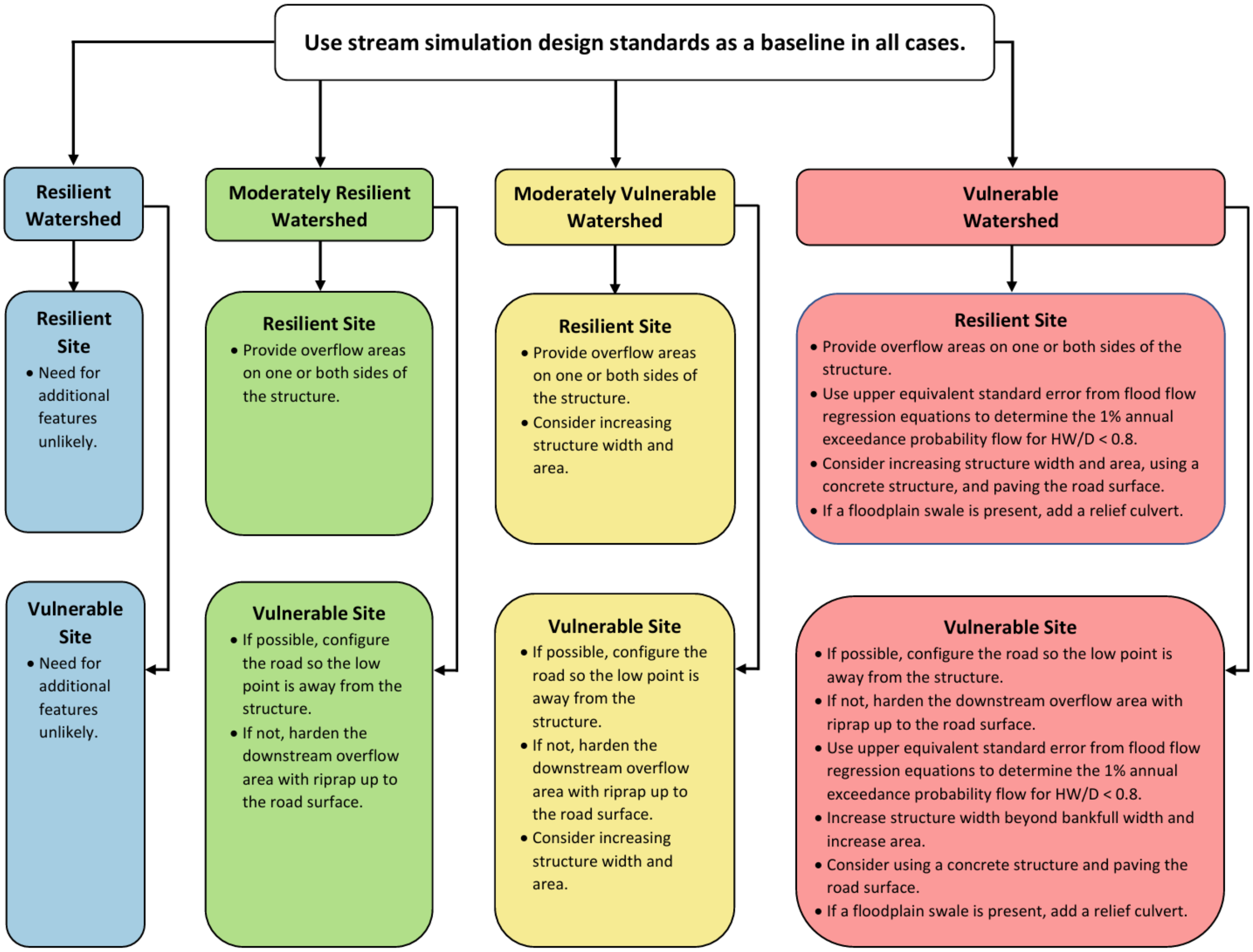

Figure 13.—Baseline and supplemental design features to enhance flood resiliency. HW/D = headwater-to-depth ratio.

Data in Table 3 were used to classify the vulnerability of the road-stream crossing sites to failure at overtopping; 10 crossings had site conditions classified as vulnerable and 16 as resilient (Table 9). Crossings with vulnerable site conditions, especially with moderately vulnerable watersheds or those reclassified to vulnerable, would logically require design steps beyond SS, to increase their survivability during an extreme flood. Two examples of crossings that have both vulnerable watershed and site conditions follow. Site 5, which failed, may have been more survivable with a larger concrete structure and large rock riprap that extended to the top of road and across the entire narrow floodplain. Site 4 was an “Other” road-stream crossing that failed but would likely have survived with an SS culvert and hardened overflow area.

Table 9.1.—Peak flows, flood impacts, and watershed and site vulnerability ratings for 26 road-stream crossings, Chequamegon-Nicolet National Forest, July 2016 flood

Site #	Stream	Design method	Drainage area (sq mi)	Flood peak flow (cfsm)	1% Annual exceedance probability (AEP) peak flow (cfsm)	Flood impact: Site overtopped?	Flood impact: result
10	Unnamed tributary to (UNT) Marengo	Stream simulation (SS)	0.21	657	405	No	Survived
1	UNT Morgan	Other	0.51	2,629	290	Yes	Failed
5	Hawkins	SS	3.37	1,019	150	Yes	Failed
14	Preemption	SS	2.67	262	118	Yes	Survived
4	UNT Whisky, East	Other	1.03	1,103	107	Yes	Failed
16	Trout	SS	5.09	221	92	Yes	Survived
17	Mineral Lake Inlet	Other	2.09	215	78	Yes	Failed
7	UNT Morgan Falls	SS	0.29	759	300	Yes	Survived
3	UNT Hawkins	SS	0.94	1,223	299	Yes	Survived
6	UNT Whisky, West	SS	0.21	762	143	Yes	Survived
11	UNT Trout, East	SS	0.61	598	141	Yes	Survived
9	20 Mi	SS	0.31	677	139	Yes	Survived
2	20 Mi	SS	0.78	1,712	119	Yes	Survived
8	UNT Whisky, Middle	SS	0.65	692	106	Yes	Survived
23	Porcupine	Other	5.83	60	49	No	Survived
20	18 Mi	Other	20.47	125	45	Yes	Failed
25	WF Chippewa	Other	11.5	25	35	No	Survived
12	UNT Marengo, South	SS	0.37	581	146	Yes	Survived
19	UNT Marengo	Other	0.69	145	101	No	Survived
15	UNT Marengo, North	SS	1.43	245	97	Yes	Survived
13	McCarthy	SS	1.35	348	86	Yes	Survived
18	UNT Brunsweiler	Other	1.28	195	59	Yes	Survived
21	UNT WF Chippewa	SS	2.16	102	48	Yes	Survived
22	Brush	SS	6.84	60	36	No	Survived
24	18 Mi	SS	10.4	27	17	No	Survived
26	Red Ike	Other	5.02	9	32	No	Survived

Notes: 1 percent AEP flood flows were derived from regression equations for Wisconsin watersheds (Walker and Krug 2003) using drainage area, storage, main channel slope, soil permeability, and snow; cfsm = cubic feet per second per square mile.

Table 9.2.—Peak flows, flood impacts, and watershed and site vulnerability ratings for 26 road-stream crossings, Chequamegon-Nicolet National Forest, July 2016 flood

Site #	Stream	Storage (%)	Main channel slope (ft/mi)	Drainage density (mi/sq mi)	Saturated hydraulic conductivity (KSAT), upland all layers (in/hr)	Watershed vulnerability rating	Site vulnerability rating
10	Unamed tributary to (UNT) Marengo	0.4	247	3.82	2.00	MVUL	VUL
1	Unt Morgan	3.5	218		2.30	MVUL	VUL
5	Hawkins	6.2	109	1.90	1.92	MVUL	VUL
14	Preemption	11.6	85	1.25	3.03	MVUL	VUL
4	Unt Whisky, East	17.2	58	1.15	2.74	MVUL	VUL
16	Trout	26.0	82	1.03	1.96	MVUL	VUL
17	Mineral Lake Inlet	18.4	34	0.68	2.04	MVUL	VUL
7	UNT Morgan Falls	22.4	36		2.57	MVUL	RES
3	UNT Hawkins	1.3	172	0.99	1.09	MVUL	RES
6	UNT Whisky, West	14.7	59		2.76	MVUL	RES
11	UNT Trout, East	10.0	57	3.08	1.47	MVUL	RES
9	20 Mi	17.1	75	1.08	1.86	MVUL	RES
2	20 Mi	12.6	56	1.11	1.99	MVUL	RES
8	UNT Whisky, Middle	23.7	59	1.27	2.77	MVUL	RES
23	Porcupine	32.1	23	0.28	3.34	MRES	VUL
20	18 Mi	24.2	25	0.71	5.55	MRES	VUL
25	WF Chippewa	36.6	13	0.53	5.01	MRES	VUL
12	UNT Marengo, South	9.2	57		7.35	MRES	RES
19	UNT Marengo	24.1	55	0.71	2.54	MRES	RES
15	UNT Marengo, North	21.8	60	0.56	1.51	MRES	RES
13	McCarthy	23.0	38	1.28	1.94	MRES	RES
18	UNT Brunsweiler	43.0	31	1.90	2.56	MRES	RES
21	UNT WF Chippewa	34.0	15	0.97	3.86	MRES	RES
22	Brush	50.0	16	0.61	2.71	MRES	RES
24	18 Mi	23.9	15	0.47	5.55	MRES	RES
26	Red Ike	35.6	8	0.36	6.60	RES	RES

Notes: Horizontal dashed lines delineate groups with a similar combination of watershed and site vulnerability. RES = resilient, MVUL = moderately vulnerable, MRES = moderately resilient, VUL = vulnerable.

CONCLUSIONS

Rainfall and Flood in 2016

Extreme flooding caused by up to 9.2 inches of rainfall in 6 hours on July 11, 2016 provided lessons regarding the design and construction of flood resilient culverts, and the identification of areas more vulnerable to increases in future floods. Flood flows estimated at 26 road-stream crossing sites from high water marks and hydraulic calculations ranged from 9 to 2,629 cfsm with 21 over 100 cfsm and 12 over 500 cfsm. On average, flood flows were three times greater than the estimated discharge for the 0.2 percent AEP flood determined using USGS regression equations.

Culvert Survivors and Failures

Of the 26 sites where 2016 flood flow estimates were obtained, 17 were designed and installed from 2001 to 2015 using SS methods. Specific design standards for the other nine sites are mostly unknown. Two key criteria for SS design are that structure width equal or exceed bankfull channel width, and HW/D <0.8 at the 1 percent AEP flood. These and other design features provide substantial flood resiliency in the form of hydraulic capacity and passage of debris.

For the SS sites, there were 16 structures that survived and 1 that failed although 13 of the structures overtopped. While none of these structures was designed to accommodate the 2016 flood flows, our study suggests that their survival is attributed to several factors listed in relative order of importance.

1. SS design provides substantially greater flood capacity than traditional culvert design. It also allows the tailwater to rise in conjunction with the headwater to near the top of the structure so that if overtopping occurs, the head difference and erosive power are less.

2. None of these sites became plugged with debris. In all but one case where a log caused a partial constriction of the culvert, the bankfull width structures were able to pass any woody debris that was in transport.

3. Broad flood-prone areas provided for substantial flow over the road in many cases.

4. On several sites the road ramped up over the structure allowing overflow to one or both sides of the structure rather than directly over it.

5. Construction and maintenance practices that provided erosion protection to the road surface and fill including pavement, riprap, and vegetative ground cover appeared to help structure survival.

Costs

Construction costs for SS structures were compared to estimated costs for R-I-K structures at 17 sites in the 2016 flood area. Costs were adjusted to the year 2020. Increased costs were compared to changes in constriction ratio and open area which served as surrogates for some benefits associated with SS structures. The 17 SS structures were installed from 2001 through 2015, and while 14 overtopped during the 2016 flood, only one failed. The 17 SS sites were divided into two groups: 11 sites where pipe-arch culverts less than 10 feet wide were installed, and 6 sites where upgrades to bridge-type structures 10 feet wide or greater were required. SS construction costs for the 11 pipe-arch culverts ranged from $17,230 to $27,405, and the average cost was 21 percent higher or 1.21 times the estimated R-I-K cost. On average, the SS culverts increased the constriction ratio from 0.55 to 1.25 and open area by 340 percent. SS construction costs for the six bridge-type structures ranged from $52,708 to $278,710, and the average cost was nearly 5 times the estimated R-I-K cost, with a range of 2 to 11 times. On average, the bridge-type structures increased the constriction ratio from 0.41 to 1.31 and open area by 540 percent. R-I-K culverts may not have been eligible for environmental permits and would not meet U.S. Forest Service environmental and infrastructure objectives.

The modestly higher costs for 11 SS pipe-arch culverts appear to be easily offset by their substantial infrastructure and ecological benefits. The substantial increase in costs for the six bridge-type structures is attributable to existing structures that were severely undersized, the need to upgrade from a culvert to a bridge-type structure, and incorporation of other design features that would accomplish environmental and road management objectives. At 14 of the 17 sites, the substantial increase in open area for SS structures was achieved at a lower cost per square foot compared to R-I-K structures. Hydraulic modeling indicated 11 to 15 of the 16 surviving SS sites likely would have failed during the 2016 flood had they been R-I-K structures, and this resulted in substantial replacement cost savings at those sites. These sites provide an example of how SS design can extend structure life span.

Differences in Watershed Response

While all but a few of the 26 sites experienced extreme flooding, there were substantial differences among the sites because of differences in watershed characteristics. During the July 2016 flood, peak flows ranged from less than 50 to over 2,600 cfsm. These differences illustrate how some areas are more vulnerable to increases in future flood flows by producing more rapid runoff from rainfall than more resilient areas. Factors contributing to more rapid runoff include steep slopes, less storage, and finer soils whereas gentle slopes, greater storage, and coarser soils lead to more attenuated runoff.

Watershed and Site Vulnerability to Future Flood Increases

Differences in flood response among the 26 watersheds were correlated with LTAs. Based on that evidence and their dominant hydrologic properties, LTAs across Wisconsin were assigned one of four vulnerability classes to demonstrate how differences in vulnerability can be identified and used for refining flood resilient culvert designs and prioritizing areas for climate change adaptation. The classifications ranged from vulnerable areas dominated by rapid runoff to resilient areas dominated by groundwater recharge. Streamflow gauges within each class helped verify differences in flood runoff among them. While there was considerable variation within each class, the median flood flows for the 1 percent AEP flood from USGS gaging stations across the state for vulnerable, moderately vulnerable, moderately resilient, and resilient areas were 296, 87, 42, and 14 cfsm, respectively (Table 8).

Site characteristics that increase the potential for overtopping, vulnerability to failure at overtopping, and the consequences of failure, include narrow floodplains (entrenchment <2.2), high road fill (>7 feet), steep stream channel slopes (>1 percent), and type of structure. Characteristics that lead to a site being less vulnerable to failure include broad floodplains (entrenchment >5) and low road fill (<4 feet) where there is opportunity for road overtopping, and gentle stream channel slopes (<0.4 percent). The values in parentheses are based on site characteristics from this study for watershed areas averaging less than 10 square miles; these insights are intended as general guidelines that can be refined with local knowledge and experience in a given area.

Designing Flood Resilient Culverts

We suggest using SS to design culverts and bridges at all road- and trail-stream crossings in rural Wisconsin. Doing so will provide a baseline resiliency for current floods and potential increases in future floods. During design, we recommend evaluating the vulnerability of the watershed to flood flow increases and vulnerability of the site to failure if such floods overtop the road. For more vulnerable watersheds and sites, it may be best to implement additional design features such as stable overflow areas, increased structure area, paved road surface, or concrete structures as appropriate. In combination, these actions can advance climate adaptation in rural Wisconsin, specifically improving resilience to more-intense rainfall and extreme floods.

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GLOSSARY

AEP (annual exceedance probability; percent)—Probability that an event such as a flood will be exceeded in any given year. Following are examples of AEPs and corresponding return intervals (RIs).

0.01 percent AEP (i.e., 1,000-year average RI)

0.2 percent AEP (i.e., 500-year average RI)

1 percent AEP (i.e., 100-year average RI)

2 percent AEP (i.e., 50-year average RI)

4 percent AEP (i.e., 25-year average RI)

66.7 percent AEP (i.e., 1.5-year average RI)

72.4 percent AEP (i.e., 1.38-year average RI)

100 percent AEP (i.e., 1-year average RI)

Aggrade—To build up the level of any surface by the deposition of sediment.

Bankfull—The elevation where water begins to flow onto the floodplain. Bankfull width is the width of the channel at bankfull elevation. The bankfull flow is associated with a frequent flood with an average AEP of 0.667 (i.e., average RI of 1.5 years).

Bedform—Channel units such as pools, riffles, or steps which typically occur in repeating sequences.

CRO (Cable Rolling Outwash) LTA—Collapsed and uncollapsed outwash with loamy sand over outwash.

CWTO (Chequamegon and Washed Till and Outwash) LTA—A complex of rolling collapsed moraine and outwash plain with predominantly fine sandy loam over loamy sand till.

DD (drainage density; miles/square mile)—The total length of stream channel divided by the total area. Higher drainage densities occur where there is more rapid runoff because of lower soil permeability and steeper slopes. Climate can also be a factor; higher volumes and intensities of rainfall or snowmelt can lead to higher drainage densities. Drainage density is normally determined from the total length of both perennial and intermittent streams. Unmapped streams can lead to an underestimate of drainage density.

Drumlin—An elongated or oval-shaped hill created by glacial activity.

Flood-prone width—The width of the floodplain determined by a water surface or flood elevation that is twice the maximum bankfull channel depth measured perpendicular to the valley.

Freeboard—The vertical distance from the design flood water surface elevation to the top of the culvert or bottom of bridge stringers, or the low point in the road. It is a measure of clearance before the structure completely fills, or the road overtops.

GD (Glidden Drumlins) LTA—Outwash plains and swamps between drumlins with predominantly fine sandy loam over loamy sand till.

H&H (hydrology and hydraulics)—The two primary steps in traditional culvert design, where hydrology refers to the determination of a design flood flow and hydraulics refers to analyses to determine whether the proposed structure will meet design criteria, typically a specific HW/D.

Headwater—The elevation of the water surface at the inlet of a hydraulic structure such as a culvert or bridge. It can also be expressed as a depth above the invert of the culvert at the inlet or the thalweg of the channel at the inlet where there is a streambed in the culvert or at a bridge.

HEC-RAS (Hydrologic Engineering Center River Analysis System)—A software program developed and maintained by the U.S. Army Corps of Engineers for modeling river systems, including the design of structures at road-stream crossings.

HW/D (headwater-to-depth ratio)—Depth of the headwater divided by the height of the culvert or other hydraulic structure. Used as a criterion in the hydraulic design of culverts. In traditional H&H culvert design, an allowable HW/D is typically specified for a given AEP flood.

Invert—The inside bottom of a culvert or pipe.

Isohyet—A line on a map connecting points having the same amount of rainfall in a given period.

KSAT (saturated hydraulic conductivity; inches/hour or micrometers/second)—The ease with which water can move through a saturated medium such as soil.

LTA (landtype association)—Ecological units based on geomorphic process, general topography, geology, surficial geology, soil families, local climate, and potential natural communities (Cleland et al. 1997). They are mapped at a scale of 1:60,000 to 1:250,000 and typically are thousands to tens of thousands of acres in size.

MCS (main channel slope; feet/mile)—As determined by USGS methods for flood frequency analysis, it is the slope of the main channel between the 10th and 85th percentile of the distance between the mouth of the stream and the basin boundary. The difference in altitude between the points is divided by the main channel stream length between the two points to compute the slope in feet/mile.

PG (Penokee-Gogebic Iron Range) LTA—A hilly bedrock-controlled moraine with predominantly sandy loam soils over sandy loam till or bedrock. While depth to bedrock can range up to 50 feet, this LTA contains some of the shallowest soils in NW WI and bedrock outcrops are common.

RI (recurrence interval; years)—The average time between the occurrence of an event such as a flood. It is a statistical measure where RI = 1/(AEP × 100). It is also referred to as a return period. While commonly used, RI can be misinterpreted because floods do not occur at average intervals. As such, AEP is more appropriate for quantifying flood frequencies.

Shaid_typ—A two-character code in Wisconsin's GIS 24k Hydrography layer that defines types of areal water features.

SS (stream simulation)—A culvert design method that designs a stream channel through the road crossing then wraps a structure around the channel. It has its foundations in efforts to pass fish through culverts and incorporates the sciences of stream geomorphology, hydrology, and engineering. It results in low maintenance, long lasting, flood resistant, and environmentally friendly road-stream crossings.

SSURGO (Soil Survey Geographic database)—Digital soils data produced and distributed by the NRCS, which has information on soil types and their distribution including map and tabular data. The information covers soil characteristics and soil properties, and addresses limits, risks, and suitability for various uses.

STO (storage percent)—Area of a watershed, expressed as a percentage of the drainage area, that can store water including lakes, ponds, and wetlands.

Tailwater—The water depth located immediately downstream from a hydraulic structure such as a culvert or bridge.

Thalweg—A line connecting the lowest points in a stream channel.

Thiessen polygon—Polygons generated from a set of sample points where each polygon defines an area of influence around its sample point, so that any location inside the polygon is closer to that point than any of the other sample points.

TWEM (Telemark Washed End Moraine) LTA—Hilly collapsed moraine with sandy loam over outwash or loamy sand till.

UNT—Unnamed tributary

VMM (Valhalla-Marenisco Moraines) LTA—Rolling collapsed moraine with predominantly fine sandy loam over sandy loam till or outwash.

APPENDIX

Results from Analysis of Variance for Three Landtype Association Groups and Pairwise Comparisons

Table 10.—Differences in rainfall, peak flow, and four watershed characteristics among three landtype association (LTA) groups

Parameter	Source	SS	df	Mean SS	F	p-value
Rainfall (reflected $\log_{10}$ inches)	LTA group	0.0220	2	0.0110	0.46	0.0232
	Error	0.5477	23	0.0238		
	Total	0.5697				
Peak flow ($\log_{10}$ cfsm)	$\log_{10}$ rainfall	0.8949	1	0.8949	11.45	0.0027
	LTA group	3.7184	2	1.8592	23.78	<0.0001
	Error	1.7199	22	0.0782		
	Total	6.3332				
MCS ($\log_{10}$ ft/mi)	LTA group	2.1303	2	1.0652	17.86	<.0001
	Error	1.3720	23	0.0597		
	Total	3.5023				
KSAT upland, all layers ($\log_{10}$ in/hr)	LTA group	0.4347	2	0.2174	9.26	0.0011
	Error	0.5397	23	0.0235		
	Total	0.9744				
DD ($\log_{10}$ mi/sq mi)	LTA group	0.9120	2	0.4560	12.08	0.0004
	Error	0.7175	19	0.0378		
	Total	1.6295				
STO	LTA group	0.1885	2	0.0943	9.89	0.0008
(%)	Error	0.2192	23	0.0095		
	Total	0.4077				

Notes: SS = sum of squares, df = degrees of freedom, cfsm = cubic feet per second per square mile, MCS = main channel slope, KSAT = saturated hydraulic conductivity, DD = drainage density, STO = storage in the form of lakes and wetlands. Comparisons are based on one-way analysis of variance, except for peak flow, which was based on analysis of covariance and includes the result of the interaction with rainfall.

Table 11.—Pairwise comparisons of landtype association (LTA) groups

Parameter	LTA group 1	LTA group 2	Difference	Standard error	Degrees of freedom	t-ratio	Adjusted p-value
Rainfall	CWTO/VMM	CWTO	0.0532	0.0976	23	0.545	0.8499
(reflected	PG/VMM	CWTO	0.2091	0.0791	23	2.644	0.0372
$\log_{10}$ inches)	PG/VMM	CWTO/VMM	0.1559	0.0791	23	1.971	0.1420
Peak flow	CWTO/VMM	CWTO	-0.72	0.176	22	-4.07	0.0014
($\log_{10}$ cfsm)	PG/VMM	CWTO	-1.07	0.159	22	-6.71	<0.0001
	PG/VMM	CWTO/VMM	-0.35	0.153	22	-2.28	0.0798
MCS	CWTO/VMM	CWTO	0.448	0.154	23	2.899	0.0213
($\log_{10}$ ft/mi)	PG/VMM	CWTO	0.738	0.125	23	5.901	<.0001
	PG/VMM	CWTO/VMM	0.291	0.125	23	2.322	0.0726
KSAT upland	CWTO/VMM	CWTO	-0.1802	0.0969	23	-1.860	0.1730
All layers	PG/VMM	CWTO	-0.3298	0.0785	23	-4.202	0.0010
($\log_{10}$ in/hr)	PG/VMM	CWTO/VMM	-0.1496	0.0785	23	-1.906	0.1597
DD	CWTO/VMM	CWTO	0.370	0.130	19	2.835	0.0273
($\log_{10}$ mi/sq mi)	PG/VMM	CWTO	0.502	0.102	19	4.914	0.0003
	PG/VMM	CWTO/VMM	0.133	0.111	19	1.196	0.4698
STO	CWTO/VMM	CWTO	-0.0282	0.0617	23	0.457	0.8919
(%)	PG/VMM	CWTO	-0.1882	0.0500	23	3.763	0.0028
	PG/VMM	CWTO/VMM	-0.1600	0.0500	23	3.200	0.0107

Notes: CWTO = Chequamegon Washed Till & Outwash, VMM = Valhalla/Marenisco Moraines, PG = Penokee/Gogebic Iron Range, cfsm = cubic feet per second per square mile, MCS = main channel slope, KSAT = saturated hydraulic conductivity, DD = drainage density, STO = storage in the form of lakes and wetlands. Comparisons based on one-way analysis of variance, except for peak flow, which was based on analysis of covariance and includes the result of the interaction with rainfall.

Table 12.—Metric Equivalents

When you know:	Multiply by:	To find:
Inches (in)	2.54	Centimeters
Feet (ft)	0.305	Meters
Square feet (ft^2)	0.0929	Square meters
Cubic feet (ft^3)	0.0283	Cubic meters
Cubic yard	0.764	Cubic meters
Acres (ac)	0.405	Hectares
Miles (mi)	1.6093	Kilometers
Square miles (mi^2)	2.59	Square kilometers
Miles per square mile (mi/mi^2)	0.6214	Kilometers per square kilometer
Feet per mile (ft/mi)	0.1895	Meters per kilometer
Inches per hour (in/hr)	2.54	Centimeters per hour
Feet per second (ft/sec)	0.305	Meters per second
Cubic feet per second (ft^3/sec or cfs)	0.0283	Cubic meters per second
Cubic feet per second per square mile ($ft^3/sec/mi^2$ or cfsm)	0.0109	Cubic meters per second per square kilometer

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