Historical Analysis of Hydraulic Bridge Collapses in the Continental United States

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Abstract: Predictions of the risk to built infrastructure posed by climate and land-use change have suggested that bridge collapses may increase due to more frequent or intense flooding. Assessments of the United States often assume that bridges may collapse when the 100-year flood (i.e., a flood with 1% annual frequency of exceedance) occurs, but this assumption has not been fully tested because of a lack of comprehensive collapse records. Thirty-five bridges for which a stream gauge on or near the bridge recorded the flow during total or partial collapse were identified and used to test this assumption. Flood frequency analyses, other statistical analyses, and structural reliability methods were used to quantify the return periods of collapse-inducing flows, identify trends linked to event and site characteristics, and evaluate the potential importance of collapse return period variability in assessing the impact of climate and land-use change on hydraulic collapse risk. The results indicate that the collapse-inducing flow return periods varied considerably (range: 1 to >1,000 years) and were frequently lower than values considered in many climate impact assessments: 23 of the 35 bridges were estimated to have collapsed during flows with return periods of lower than 100 years. Annual failure probabilities computed using the full distribution of return periods of the collapse-inducing flows, as opposed to central values (e.g., means), were more sensitive to an assumed increase or decrease in the underlying frequency of flooding. These results suggest that linking bridge collapse to only the 100-year flow does not capture significant variability associated with collapse return periods, potentially reducing sensitivity to flood frequency changes and reducing the robustness of assessments of the impact of climate, land-use, and streamflow-regulation change on hydraulic bridge collapse risk. **DOI:** 10.1061/(ASCE)IS.1943-555X.0000354.

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Introduction

Floods, scour, and other hydraulic events are thought to be the most common causes of total or partial bridge collapse in the United States (Cook et al. 2015; Arneson et al. 2012; Kattell and Eriksson 1998). Several studies have estimated an annual hydraulic collapse frequency of approximately 1/5,000 (e.g., Cook et al. 2014; Nowak and Collins 2012). Scour—erosion of the soil supporting bridge foundations—alone has been estimated to cause the collapse of 20–100 bridges per year in the United States (Briaud et al. 2007; Cook et al. 2015; Stein and Sedmera 2006) of a total population of bridges over water of approximately 504,000 (FHWA 2012). Hydraulic and other collapse causes have been linked to substantial direct and indirect costs, casualties, and user delays and increased greenhouse gas emissions resulting from detours and delays (Suarez et al. 2005; Stein and Sedmera 2006; Stein et al. 1999; Neumann et al. 2015; Wright et al. 2012; Cook et al. 2015; Briaud et al. 2007, 2014).

Global climate and land-use change have been identified as potentially altering the frequency and magnitude of flooding in the United States (Milillo et al. 2014). Climate change is expected to alter the frequency and severity of precipitation across the United States, with some regions, especially the Northeast, experiencing an increase in annual precipitation (Milillo et al. 2014). In the East and Northwest, a greater portion of precipitation is expected to occur during extreme events (Singh et al. 2013). These changing precipitation patterns, combined with land-use changes, are frequently linked to an increasing risk of extreme floods (e.g., Meyer et al. 2013). Studies of historical streamflow data have yielded mixed results in terms of current trends in annual peak flows and the existence of abrupt changes (Lins and Slack 1999; Mallakpour and Villarini 2015; Hirsch and Ryberg 2012; Villarini et al. 2009; Kundzewicz et al. 2014).

The hypothesized increasing flood risk has been predicted to increase the rate of hydraulic bridge collapses (Wright et al. 2012; Neumann et al. 2015; Meyer and Weigel 2011; Khelifa et al. 2013; Suarez et al. 2005). Studies of the Gulf Coast (Center for Climate Change and Environmental Forecasting 2013), Boston (Suarez et al. 2005), and the continental United States (Wright et al. 2012; Neumann et al. 2015; Khelifa et al. 2013) predict significant increases in user delays and high costs related to climate adaptation and repair of damaged bridges. The studies use a variety of methods to identify vulnerable bridges and to link increasing exposure to an increasing rate of collapse. Studies over large geographic regions frequently use data from the National Bridge Inventory (NBI) (FHWA 2012) to identify vulnerable bridges and make simplifying

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assumptions regarding the magnitude of floods likely to cause bridge collapse and their rate of change. Furthermore, these studies generally consider only precipitation directly, as it is the hydrologic variable that is most directly accessible in global climate model archives. Because of several factors, such as the degree of soil saturation, extreme precipitation events do not necessarily result in extreme runoff (Ivancic and Shaw 2015). For these reasons, these studies caution that predicted direct climate impact costs of \$140 billion to \$250 billion (Wright et al. 2012), or increased direct and indirect losses of 17% (Khelifa et al. 2013), can serve only as indicators of the true magnitude of costs related to the impact of climate change on bridges.

The assumed return periods of collapse events used in climate impact studies (e.g., Wright et al. 2012; Khelifa et al. 2013) are frequently derived from bridge design procedures. In the United States, bridges are designed using manuals developed by state and municipal DOTs, and other agencies, which in turn reference manuals and guides provided by professional organizations and the government (e.g., AASHTO 2014; Lagasse et al. 2009; Brown et al. 2009). Guidelines specify the use of a "100-year flood," or a flood with an annual probability of exceedance of 1%, when analyzing overtopping of modern interstate bridges receiving federal funding (FHWA 2009). For bridges not using federal funding, or those outside the 100-year floodplain, the owner has leeway in selecting the return period for design: values between 50 and 100 years for flood design are common (e.g., PennDOT 2015; ConnDOT 2000; CDOT 2004; NYSDOT 2014a). The design flood is then paired with a scour design flood of a higher return period and a scour design check flood of an even greater return period (e.g., 100, 200, and 500 years), although smaller floods may also need to be checked (Arneson et al. 2012). The methods to determine the flow magnitude of the design flood also vary, with recommended methods including TR-55 (National Resources Conservation Service 1986), Bulletin 17B (Hydrology Subcommittee 1982), and *StreamStats* (Atkins et al. 2007), among several others.

In addition to the variation of current design practices across states and bridge route classifications, the evolution of standards and other factors suggest the possibility of a lack of uniformity in the return periods of collapse-inducing floods. The 1990s saw increased focus on hydraulic design following the 1987 scour-induced collapse of New York's Schoharie Creek Bridge and the 1989 stream migration–related collapse of the US-51 bridge over the Hatchie River in Tennessee (Richardson and Lagasse 1999). Scour design provisions developed during this period suggest a design check for the scour resulting from a 500-year flow (Stein and Sedmera 2006; Meyer and Weigel 2011). However, as shown in Fig. 1, of the approximately 504,000 bridges over water in the United States, more than 70% were constructed before 1991 (FHWA 2012) and were not required to be explicitly designed for scour. Even if it were possible to identify the design standard and analysis method for all collapsed bridges, confounding factors (e.g., debris blockage of the approach spans or change in land use) are likely to produce return periods of collapse-inducing flows different from those expected during the design. On the other hand, bridge collapses are rare, and there is substantial evidence that many bridges have survived 100-year or even 500-year flows. The combination of these factors—variation in hydrological analysis methods, limited knowledge of the theoretical design reliability of older bridges, possible changes in hydraulic condition, and existence of high-performing bridges—suggests that it may not be reasonable to deterministically link change in a 100-year flow (or precipitation) to change in collapse risk.

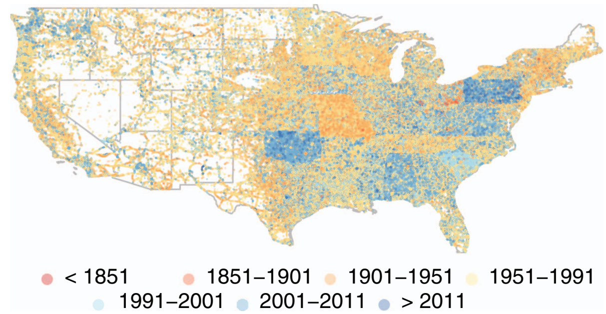

Fig. 1. (Color) Continental U.S. bridges over water by construction year; approximately 370,000 of 504,000 were built before 1991, when new scour design provisions were adopted (range: 1697–2011; median: 1973) (data from FHWA 2012)

The lack of robust data on the range and variability of the return periods of collapse-inducing flows hinders the validation of climate impact assessments and motivates the development of a geographically distributed and methodologically consistent analysis of the return periods of collapse-inducing flows. Some data are available from individual or regional case studies; for example, Cook (2014) studied collapses in New York using the New York State DOT (NYSDOT) bridge failure database (NYSDOT 2014b), which was also used in this study. However, because of the limited number of bridges studied and inconsistencies in methodology, these studies are not easily extrapolated to the U.S. bridge stock. To provide a more robust estimate of the range of flow return periods associated with hydraulic bridge collapses, 35 collapsed bridges for which a stream gauge on or near the bridge recorded the collapse flow were identified, and a number of hydrological and statistical analyses were performed for these sites. These 35 collapses represent 3% of the collapses linked to hydraulic causes in the collapse database used (NYSDOT 2014b).

A set of analysis methods was selected both to answer the fundamental research question (identifying the return periods associated with historical bridge collapses) and to provide additional insights that can inform assessments of collapse risk and of the impact of climate change on bridges. The analyses undertaken included flood frequency analysis using the Bulletin 17B methodology and partial duration analysis, comparison of the flows and return periods using tests for correlation and linear regression, evaluation of the influence of event and site characteristics (e.g., collapse cause) on collapse return periods, and identification of trends in annual peak flows. Each analysis was selected because of its potential to provide insight into hydraulic bridge performance: comparison of flood frequency analysis methods and data sources can provide guidance for analyses using climate projections; evaluation of event and site characteristics can provide an indicator for the accuracy of results; and identification of trends provides preliminary data to assess possible changes to collapse risk. To assess the importance of collapse return period variability in climate impact assessments, annual failure probabilities estimated from nominal reliabilities and from the collapsed bridges were compared and then assessed for sensitivity to assumed changes in the underlying frequency of flooding. The results of this set of analyses can be used to assess the assumptions made in climate impact assessments and to provide a more comprehensive description of the performance of U.S. bridges during extreme hydraulic events.

Methods

Several methods were used to select collapsed bridges for analysis and to answer five key research questions: (1) what were the magnitudes of collapse-inducing flows, how do they relate to other observed flows, and how does the use of daily mean or instantaneous flow values bias the results; (2) what were the return periods of collapse-inducing flows and how do they vary with analysis methods and type of flow data used; (3) how do the return periods of the collapse flows vary with characteristics of the events and sites; (4) are there trends in annual peaks series at the sites studied; and (5) what is the importance of collapse return period variability in assessing the impact of climate or land-use change on collapse risk? Analysis of event and site characteristics—including coincidence of collapse and maximum recorded flows, collapse cause, relationship to hurricanes, presence of streamflow regulation (e.g., dams), and site drainage area (area of land drained by the stream at a given location)—provides some basis for extending the results obtained for the 35 collapsed bridges to other sites and to the risk of future collapses.

Identification of Collapsed Bridges

The NYSDOT maintains the only nationwide database of bridge collapses, with 1948 entries as of 2014 (NYSDOT 2014b). Totally or partially collapsed bridges are added to the database on the basis of searches of journalism databases and quadrennial surveys of other DOTs. The version of the database referenced in this paper contained 1,127 bridge collapses linked to hydraulic causes. The types of recorded information include the identifier in the NBI (65% of hydraulic collapse entries); the location of the collapsed bridge (descriptive, 100%); the feature under the bridge (78%); the year of construction (53%); the date (12%) or year (99%) of collapse; the bridge material (78%) and structure type (74%); the type of collapse (total or partial, 39%); the number of casualties related to the collapse (19%); and other comments (6%). Many entries lacked one or more data types, and descriptions of locations and features were sometimes imprecise (e.g., 25% of the hydraulic collapse entries listed no term or a generic term such as *creek* as the feature under the collapsed bridge). Latitude and longitude information is not contained in the database, preventing direct identification of bridges with nearby stream gauges.

To identify the geographic position of the collapsed bridges, string-matching algorithms were developed and used to identify bridges in the NBI (FHWA 2012) consistent with available descriptions of the collapsed bridge. The NBI identifier, state, county, city, road, feature intersected, and other information were used in the matching (FHWA 1995). Bridges that were built, collapsed, and not replaced before 1981 may not be present in the NBI, introducing some risk of a false-positive match. Matches between the collapse and NBI databases were confirmed using additional information, including construction dates, news reports related to bridge collapses and floods, and a crowd-sourced website on historic bridges (Baughn 2015). Bridges were sought according to the following criteria:

1. Existence of a stream gauge listed in the USGS National Water Information System database (USGS 2015d) with a station name consistent with the recorded feature under the collapsed bridge.
2. Distance from bridge to gauge site of less than 10 km.
3. High likelihood that the gauge recorded the collapse flow because (a) flow on known collapse date was recorded; (b) all dates in collapse year were recorded; or (c) flow on date of annual peak was recorded (if collapse date was unknown).

The matching process yielded 35 bridges meeting these requirements. Because of the proximity of two Virginia bridges, which failed in 1994 and 2005, the same gauge was used to analyze both sites, yielding 34 gauges linked to the 35 bridges. News reports, National Oceanic and Atmospheric Administration (NOAA) Weather Prediction Center (NOAA 2015b) reports, and the HURDAT2 database (NOAA 2015a) were used to identify hurricanes that may have been related to the collapse events. The drainage area of the bridges was evaluated using the National Hydrography Data Set (USGS 2015c).

USGS Gauge Data

Depending on the collapse mode, a subset of the hydrograph (flow over time) characteristics, such as the peak flow volume, velocity, or duration, may be relevant. In particular, scour is a gradual process and may be influenced by a number of hydrograph characteristics. Hydrograph-based models of scour have been considered by Oliveto and Hager (2005) and Briaud et al. (2007), among others, with Guney and Bor Turkben (2015) finding that, between peak flow and flow duration, the peak flow rate better predicts scour depth. Because of the inconsistent availability of data on flood stage and other measures, the daily mean or instantaneous volumetric rate was considered the best representation of the flow inducing collapse, or collapse flow.

Daily mean, instantaneous, and annual peak flow measurements were obtained from USGS (2015d, e) for the 34 gauges linked to the 35 collapse sites. The three types of flow values are derived from USGS stream gauge recordings of the instantaneous volumetric rate of the streamflow, measured over a period of 5–60 min, and are reported in cubic feet per second (cu ft/s). Daily mean flows are time averaged from instantaneous measurements over a 24-h period and are generally available over the full historical record of a stream gauge. Annual peak flows are usually the maximum instantaneous value recorded over a year; in some cases, a daily mean value is reported when instantaneous data are not available. Both daily mean and annual peak flows are thoroughly processed and corrected by USGS before publication, whereas the raw instantaneous flow data are not processed and are not available at all sites and recording periods. Because they are time averaged, daily mean flows sometimes introduce a bias related to site drainage area: they may underestimate maximum flows for sites with small drainage area because these sites ramp up to flood stage very quickly (i.e., they experience flash flooding), and averaging a day of flow values damps out very rapid events. In contrast, sites with a large drainage area may take days or weeks to reach flood stage, and there is usually little difference between daily mean and instantaneous values at these sites.

A set of assumptions were used to estimate the collapse flow from the USGS time series. To reduce the likelihood of underestimating the collapse flow return period, restrictions on the collapse date were sometimes loosened. For 14 bridges with a known collapse date, daily mean values of the collapse flow were calculated using the daily mean on that date (13 bridges) or on the previous date if larger (one bridge). Instantaneous/peaks collapse flows were available at 11 sites with known collapse date. At 6 sites the annual peak coincided with the collapse date, and this value was used. At 5 sites the annual peak did not coincide with the collapse date, and the maximum recorded instantaneous flow over a period ± 1 day from the collapse date was used. This approach was conservative in that it minimized the possibility of overstating the incidence of collapses caused by sub-100-year flows, which could lead to overestimating the impact of climate change on bridge collapses. Daily mean data were used as the

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default source of collapse and maximum flow values, as it was the only data source available at all sites. Given the adjustments for known failure dates, instantaneous collapse flow data were available and obtained for 13 bridges, and annual peaks collapse flow data were obtained for an additional 18 bridges, yielding a total of 31 sites for which instantaneous or peak collapse flows could be analyzed.

Distributions and Return Periods

The methods for estimating the return period of a flood event include block-maxima approaches, in which a series of annual peak floods is used to define an extreme value distribution; peaks-over-threshold approaches, in which distributions are fit to both the frequency of floods above a threshold and their magnitudes; and partial-duration analysis, which considers the percentage of time a certain level of flow is equaled or exceeded over the entire flow record. Both block-maxima and partial-duration methods were used.

The Bulletin 17B flood frequency analysis methodology is a block-maxima approach developed by an interagency committee led by USGS and is frequently referenced in bridge design manuals. The Bulletin 17B method assumes that annual peak flows are distributed according to a log-Pearson Type III distribution and requires as input the skew of the distribution (asymmetry of the tails), which can be obtained by state or regional procedures, from a generalized plate provided in the bulletin, or from the station data itself (Hydrology Subcommittee 1982). Bulletin 17B also provides methods for treatment of low outliers and historical peaks data.

The Bulletin 17B analysis was carried out using the U.S. Army Corps of Engineers *HEC-SSP* software (v2.0) with annual peaks data downloaded by *HEC-SSP* directly from USGS. Regional skews were interpolated from the Bulletin 17B–generalized skew coefficient plate and used directly as the skew value in the analysis. Low outliers were removed by *HEC-SSP* according to the Bulletin 17B methodology; no historical peaks were used. The Bulletin 17B analysis was also conducted using annual maximum daily mean flows in place of annual peak flows. The collapse, maximum, and maximum precollapse (i.e., from the time of construction up to and including the collapse date) return periods were estimated using log-log interpolation of the *HEC-SSP*-computed curve. A return period of 1 year was assumed as a lower bound for extrapolated return periods. *HEC-SSP*-generated 5 and 95% confidence intervals were used to describe the uncertainty in the return period estimates.

Partial-duration analysis is an alternate approach for analyzing the return period of a flow, which considers all recorded flow values rather than the annual maxima considered in Bulletin 17B. Analogously to the bias between instantaneous and daily mean flow values, partial-duration analysis may produce more extreme return periods than block-maxima approaches for rare events at sites with smaller drainage area: A flash flood has a very short duration, whereas a flood on a large river may last for days or weeks. Partial-duration data in graphical form were obtained from USGS, and the tabulated exceedance values $(\min, 0.05, \ldots, 0.95, \max)$ were read using an optical character recognition software package. The exceedance probability of the maximum flow was set equal to $1/n$, where n was the number of values used to develop the partial duration curve. Log-log interpolation was used to estimate return periods using the partial-duration data. The partial-duration return periods were divided by 365.25 to obtain return periods in years.

Statistical Evaluation of Results

Various statistical tests were used to (1) analyze the population statistics of different sets of bridges; (2) assess the relationship between various estimates of flow and return periods; (3) evaluate the influence of event and site characteristics on collapse return periods; and (4) identify the presence of trends in USGS annual peak flow series. For all tests of significance level, α was 0.05 unless otherwise stated.

The comparison of the population statistics for all U.S. bridges over water and the subset of collapsed bridges was performed using the chi-square test (Upton and Cook 2014). This test assesses whether two samples are likely to come from the same underlying distribution.

Assessment of the relationship between collapse and maximum flows as well as return periods was performed using three methods: correlation as measured by Pearson's product-moment correlation coefficient, ρ, correlation as measured by Kendall's rank correlation coefficient, τ (Upton and Cook 2014), and linear regression. Pearson's correlation coefficient is frequently used to assess the linear dependence of two samples, whereas Kendall's rank correlation coefficient is a nonparametric test and less sensitive to outliers. Consistency of the return periods produced by the Bulletin 17B daily mean and instantaneous/peaks analyses was assessed for each individual site using the confidence intervals (5 and 95%) provided by *HEC-SSP*, in which the nonoverlapping of the confidence intervals was deemed a sign of an inconsistent return period estimate.

The nonparametric Mann-Whitney test was used to analyze differences in the return period distributions between subsets of the collapsed bridges, e.g., subsets based on collapse cause or link to a hurricane (Upton and Cook 2014). The chi-square test was also used to test for independence between return period means obtained from different analyses and flow data sets.

The analysis of trends in USGS annual peak flows used the nonparametric, rank-based Mann-Kendall test (Villarini et al. 2009). This test evaluates whether a monotonic negative or positive trend is present in the data, and it has been frequently used in analyses of streamflow trends because it does not require assumption of normality (Lins and Slack 1999).

Failure Probability and Risk

Collapse return periods were translated into annual and lifetime failure probabilities to extend the results of the analyses of individual bridges to current and future hydraulic collapse risk. This translation allows the investigation of the importance of variability in collapse return periods and the comparison of the results with those of other assessments.

Assessments of hydraulic collapse risk usually consider uncertainty in one of two ways. One option is to analyze a single characteristic, e.g., 100-year–flood, and attribute uncertainty to bridge vulnerability during this flood. This approach has frequently been used in collapse risk studies: additional bridge information, e.g., scour criticality as considered by Wright et al. (2012) and Khelifa et al. (2013), or requirement for underwater inspections as considered by Cook (2014), is used to compute a probability of failure in a given event. Alternately, vulnerability can be modeled over a range of possibly collapse-inducing flows, and the uncertainty is then assigned both to the vulnerability and to the occurrence of floods. Because it was not possible to consider factors such as scour criticality at the sites analyzed because majority of collapses predate the NBI, the second approach was selected to link the estimated collapse flow return periods to collapse risk.

A series of structural reliability analyses were performed using standard methods (e.g., Melchers 1999) to derive annual and lifetime failure probabilities from nominal values and from the estimated collapse flow return periods. Nominal reliability approaches implicitly consider a distribution of collapse vulnerability either over the bridge lifetime (and multiple potential collapse causes and modes) or during a specified event. Nominal event failure probabilities are conceptually similar to previous approaches (e.g., Khelifa et al. 2013; Pearson et al. 2000) in which collapse risk at a specified event return period is tied to bridge characteristics. Lifetime failure probabilities can be computed from nominal event failure probabilities by assuming a Poisson process for flood occurrence.

Two structural reliability methods were used to derive annual failure probabilities from the collapse return periods. The first method assumed that collapse occurs if a flow with a given return period is exceeded, with that return period being derived from the central values (median and mean) of the Bulletin 17B collapse return periods. The second approach included variability in the collapse return periods by fitting a kernel-smoothed distribution (Gaussian kernel: bandwidth of 25.7 years for daily data and 43.1 years for instantaneous/peaks data) to the collapse return periods. This distribution was convolved with the return period hazard curve (range: 1–12,000 years) to estimate the annual failure probability. In both cases, lifetime failure probabilities were obtained in the same approach used for nominal event reliabilities.

To approximate the possible impact of climate, land-use, or regulation change, the nominal and estimated failure probabilities were analyzed by assuming a shift in the underlying frequency of flooding events. In this analysis, any given flood was assumed to occur 10% more frequently, and 10% less frequently, representing a uniform shift in the flood hazard curve. This analysis does not reflect a specific change in the distribution attributable to climate, land-use, or regulation change but does provide an estimate of the sensitivity of the failure probability estimates to changes in flood frequency.

Collapsed Bridges and Comparison with U.S. Bridges

Data and statistics related to the bridge and gauge sites are presented in Table 1. The locations of the collapsed bridges are shown in Fig. 2, which also encodes their collapse cause, possible relation of collapse flow to hurricanes, drainage area, and availability of a confirmed collapse date. Additional data, including the original collapse database entries for location and feature, are available in a permanent repository (Flint et al. 2016), which also links to an interactive online version of Fig. 2. Plotting scripts and data are available in a Git repository (Flint 2016).

Because of the lack of comprehensiveness of the NYSDOT database (with relative over-representation of New York and few entries for certain states) and the lack of a geographically uniform placement of stream gauges, it was not possible to obtain an even distribution of bridges across the continental United States. Hence, it is reasonable to assume that the clustering of collapses in New York, Maryland, and Virginia reflects not a higher rate of hydraulic collapses in these states but rather the limitations of the data available.

The compositions of all U.S. bridges over water and the set of collapsed bridges are compared in Fig. 3. Using the chi-square test, the set of collapsed bridges was determined to be significantly different from the general population of U.S. bridges over water in terms of age, structure material, and structure type. Steel through-truss and concrete deck arch bridges are significantly overrepresented in the set of collapsed bridges, whereas culverts of all materials are underrepresented. When only bridges in New York, Maryland, and Virginia were considered (17 bridges), the collapsed bridges were *not* found to be statistically different from the general population in those states in terms of material ($p = 0.08$) or type ($p = 0.22$). This result suggests that the collapsed bridges may be representative of a portion of the U.S. bridge stock, although it is noted that analyzing bridges in only three states reduced the statistical power of the test for material to 0.65 as opposed to >0.99 for the other tests.

The degree to which the flow at the gauge site could be assumed to be fully representative of the flow at the collapsed bridge varied considerably. Five bridges had a functioning gauge on the bridge or within 100 m. An additional 20 had a gauge within 5 km, and the remaining 10 bridges had a gauge between 5 and 10 km away. Thirteen bridges had a bridge drainage area of $\pm 10\%$ from the USGS-estimated gauge drainage area (an additional six were within 20%). Four bridge drainage areas were $>50\%$ greater or less than the gauge area (ratio of bridge to gauge drainage area: 1.16 ± 0.95, all sites; 0.98 ± 0.19, excluding four outliers).

Flow and Frequency Analyses

Relationship between Collapse and Maximum Flows

Collapse flows, maximum flows, and maximum precollapse flows are compared in Table 2 and Fig. 4(a). The ordering and labeling of the bridges are consistent between Tables 1 and 2 and Fig. 4, and indicators are provided for the relative difference in the drainage areas of the bridge and gauge sites to highlight the possibility of bias. Collapse flows ranged considerably in magnitude and relationship to the maximum recorded flow. Seventeen of 35 bridges collapsed during the maximum (daily mean) flow ever recorded at the site, and an additional three collapsed on the maximum flow recorded up to and including the collapse date. This includes 9 of 14 bridges in which the confirmed collapse date coincided with the date of the maximum recorded flow. Eleven of 14 bridges with a confirmed collapse date collapsed during the maximum flow during the collapse year. The use of instantaneous and/or peaks data (provided in Table 2 but not shown) resulted in uniformly higher estimates of the collapse flow, as would be expected, with 21 of 31 bridges collapsing during the maximum flow and one additional bridge collapsing during the maximum precollapse flow.

Also shown in Fig. 4(a) are nominal design and check flood values, $Q100$ (flow with a return period of 100 years) and $Q500$ (flow with a return period of 500 years), estimated using the Bulletin 17B methodology and daily mean data. In addition to allowing the comparison of observed floods with nominal design floods, these values provide some indication of the distribution skews—in some cases, $Q100$ and $Q500$ are relatively close together (e.g., the top-four sites on the plot), whereas the values are farther apart in others (e.g., the bottom-three sites).

Fig. 5(a) shows the correlation between collapse flows obtained using daily mean and instantaneous/peaks data, emphasizing the roles of site drainage area and stream regulation. The collapse flows tended to increase with increasing drainage area, and the very highest collapse flows were recorded at unregulated sites (these were also maximum recorded flows as indicated by the full opacity of the circles). As expected, the smaller sites had a larger relative difference between instantaneous/peaks flows and daily mean flows.

Despite the variations with drainage area, according to some measures of correlation provided in Fig. 5(a) and Table 3, the instantaneous/peaks collapse flows were reasonably well correlated

Table 1. Collapsed Bridge and Gauge Site Information

Statistics	Year built–year collapsed–state[a,b]	Collapse date[a]	Cause[a]	Comment[a]	Hurricane [distance (km)][c]	Type[a]	Material[a]	Staid[d]	Area (km^2)[d]	Area ratio bridge: gauge[d,e]	Distance to gauge	Number of peaks[d]	Skew[f]	Regulation[d]	Mann–Kendall trend
	1857–1987–Maine	1987-04-01	Hydraulic flood	—	—	Covered	Wood	01031500	769	1.0	0.03	112	0.3	Low flow	Positive[i]
	1910–1982–Arkansas	1982-12-03[g]	Hydraulic flood	—	—	Truss through	Steel	07075000	781	0.9	2.7	76	−0.3	None	Positive
	1915–1982–Arkansas	1982-12-03[g]	Hydraulic flood	—	—	Truss through	Steel	07069500	3,007	0.9	7.4	79	−0.3	None	Positive
	1916–1946–Washington	1946-12-11[g]	Hydraulic flood	—	—	Arch	Concrete	12087000	213	1.0	0.1	40	0	Small diversion	Positive[i]
	1920–1981–New York	1981-10-28	Hydraulic flood	—	—	Truss	Steel	04234000	320	1.0	2.8	88	0.2	Diversion	Negative
	1926–1987–Maine	1987-04-01	Hydraulic flood	—	—	T-beam	Concrete	01049500	561	1.0	0.1	112	0.4	Cobbosseecontee Lake	Negative
	1929–1951–Kansas	1951-07-1	Hydraulic flood	—	—	Slab	Concrete	06891500	1,103	1.0	0.1	79	−0.3	Clinton Lake	Negative[i]
	1935–1996–Virginia	1996-09-06[g]	Hydraulic flood	—	Hurricane Fran (43)	Beam	Steel	01624800	187	1.2	6.7	30	0.6	Unknown	Positive
	1936–1985–West Virginia	1985-11-05[g]	Hydraulic flood	—	Hurricane Juan[h]	Beam	Steel	01611500	1,748	1.0	3.2	91	0.4	None	Negative
	1940–1996–Maryland	1996-01-19[g]	Hydraulic flood	—	—	Stringer	Steel	01596500	124	0.8	2.5	66	0.4	None	Positive
	1950–1989–New York	1989-06-23[g]	Hydraulic flood	—	—	Culvert	Steel	04216500	62	1.4	5.2	85	0	Unknown	Negative
	NA–1985–Virginia	1985-11-05[g]	Hydraulic flood	—	Hurricane Juan[h]	Truss	—	02021500	851	1.3	6.3	85	0.5	Some at times, Lake Merriweather	Positive
	NA–1985–West Virginia	1985-11-05	Hydraulic flood	—	Hurricane Juan[h]	Truss	Steel	01606500	1,684	1.3	8.5	87	0.4	None	Positive
	1925–2011–New York	2011-08-28	Hydraulic scour	—	Hurricane Irene (107)	Truss	Steel	01349711	13	6.4	4.2	16	0.4	None	Positive
	1928–2011–New York	2011-08-28	Hydraulic scour	—	Hurricane Irene (110)	Girder floorbeam	Steel	01349810	73	1.1	2.1	17	0.4	None	Positive
	1929–2011–New York	2011-08-28	Hydraulic scour	Approach wash out/scour	Hurricane Irene (64)	Arch	Concrete	01387400	231	0.9	2.9	35	0.7	Occasional, Lake Sebago	Positive
	1936–1989–Tennessee	1989-04-01	Hydraulic scour	Channel migration/scour	—	T-beam	Concrete	07030050	5,967	1.0	0.4	51	0.2	None	Negative
	1936–2011–New York	2011-08-28	Hydraulic scour	Abutment scour	Hurricane Irene (59)	Jack arch	Steel	01387450	32	1.6	2.5	54	0.6	Occasional, unknown source	Positive[i]
	1938–1964–Montana	1964-06-09	Hydraulic scour	—	—l	Truss deck	Steel	12358500	2,939	0.8	2.0	76	0.6	None	Negative
	1940–1987–New Hampshire	1987-04-2	Hydraulic scour	—	—	—	Concrete	01090800	163	1.1	6.6	51	0.4	High flow April 1987, Everett Lake	Positive[i]
	1941–1989–Mississippi	1989-01-15[g]	Hydraulic scour	—	—	Truss	Steel	02482550	3,478	1.0	0.1	58	0.05	None	Negative
	1950–2004–Missouri	2004-12-07[g]	Hydraulic scour	—	—	Girder	Steel	06927000	667	0.8	6.9	60	−0.4	None	Positive
	1955–2005–New York	2005-04-0	Hydraulic scour	—	—	Girder	Steel	01365000	99	0.7	3.9	77	0.5	None	Positive
	1959–1998–New York	1998-06-27	Hydraulic scour	—	—	Multigird	Steel	04273800	176	1.1	0.6	21	0.2	None	Negative
	1962–1972–Maryland	1972-06-2	Hydraulic scour	—	Hurricane Agnes (328)	Beam	pc	01651000	127	0.7	1.3	77	0.7	Small diversion since 1962	Positive[i]
	1968–1995–South Carolina	1996-07-27	Hydraulic scour	—	—	—	pc	02164000	125	1.8	8.7	63	0	Diversion, North Saluda Reservoir	Positive[i]
	1991–1999–Mississippi	1999-01-31[g]	Hydraulic scour	—	—	702	Timber	02487500	1,099	0.8	5.2	86	0.05	None	Positive

Table 1. (*Continued.*)

Statistics	Year built–year collapsed–state[a,b]	Collapse date[a]	Cause[a]	Comment[a]	Hurricane [distance (km)][c]	Type[a]	Material[a]	Staid[d]	Area (km^2)[d]	Area ratio bridge: gauge[d,e]	Distance to gauge	Number of peaks[d]	Skew[f]	Regulation[d]	Mann–Kendall trend
	N/A–1962–South Dakota	1962-04-01	Hydraulic scour	Pier scour during flood	—	Girder	Steel	06480000	8,138	0.7	0.8	61	−0.4	None	Positive
	N/A–1986–Nevada	1986-02-19[g]	Hydraulic scour	—	—	T-beam	Concrete	10312000	3,800	0.9	5.1	104	0.05	None	Negative
	1929–1972–Maryland	1972-06-22	Hydraulic Agnes	—	Hurricane Agnes (336)	Girder	Concrete	01648000	136	1.2	3.0	85	0.7	Needwood Lake, Bernard Frank Lake	Positive[i]
	1900–1971–Maryland	1971-09-11[g]	Hydraulic	—	Tropical storm Heidi[h]	Truss	Steel	01591000	89	1.0	1.5	70	0.6	None	Positive
	1920–1972–Maryland	1972-06-23[g]	Hydraulic	—	Hurricane Agnes (352)	Truss through	Steel	01643000	2,112	1.0	2.2	86	0.6	Occasional. low/medium flow, Linganore Reservoir.	Positive
	1930–1972–Maryland	1972-06-2	Hydraulic	—	Hurricane Agnes (354)	—	—	01643500	163	0.4	4.4	65	0.6	None	Positive[i]
	1953–1995–Virginia	1995-06-27[g]	Hydraulic debris	—		Beam	Steel	01662800	66	0.7	2.6	54	0.6	None	Positive[i]
	N/A–2004–Virginia	2004-09-28[g]	Hydraulic	—	Hurricane Jeanne (151)	Beam	Steel	01662800	66	1.1	0.8	54	0.6	None	Positive[i]
	Year built	Year collapsed													
Mean	1934	1986	—	—	—	—	—	—	1,180	1.16	3.24	67.2	—	—	—
Standard deviation	24	16	—	—	—	—	—	—	1,830	0.95	2.58	24.7	—	—	—
Minimum	1857	1946	—	—	—	—	—	—	12.7	0.38	0.03	16	—	—	—
Maximum	1991	2011	—	—	—	—	—	—	8,140	6.37	8.72	112	—	—	—

[a]NYSDOT (2014b).
[b]FHWA (2012).
[c]NOAA (2015a).
[d]USGS (2015d).
[e]Computed using USGS (2015c).
[f]Interpolated from regional skew plate of Bulletin 17B (Hydrology Subcommittee 1982).
[g]Collapse date was unknown and assumed to coincide with date of maximum daily mean flow in collapse year.
[h]Hurricane Juan and Tropical Storm Heidi were absorbed by extratropical cyclones before collapse date but contributed to heavy rain and flooding (NOAA 2015b).
[i]Statistically significant ($\alpha = 0.05$).

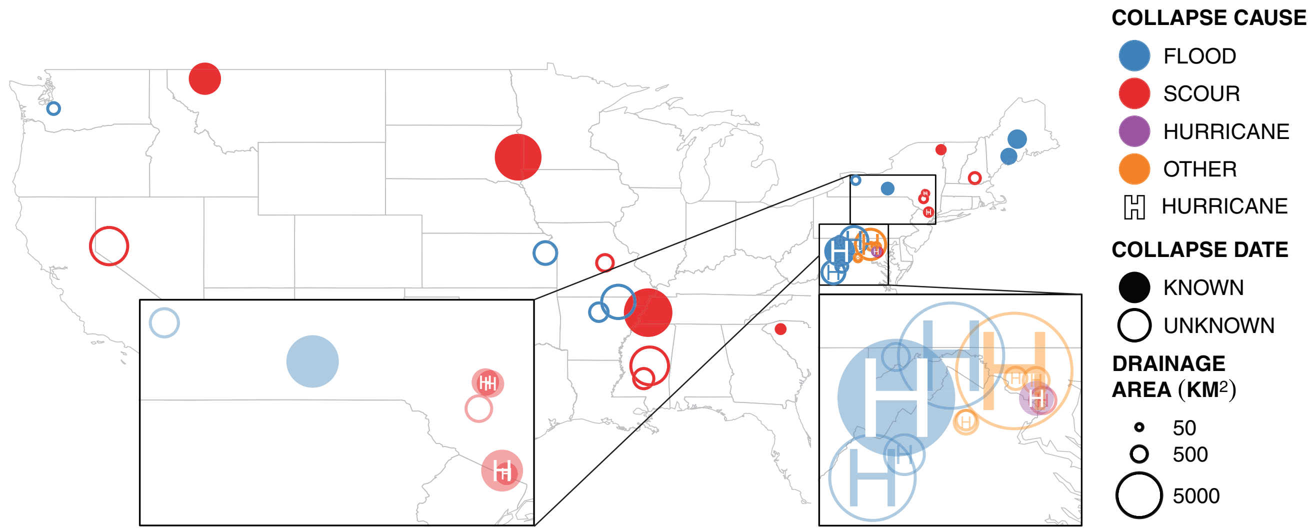

Fig. 2. (Color) Location of 35 historical bridge collapses; 13 bridges collapsed because of floods, 16 because of scour, one during Hurricane Agnes, and five from other causes (four were coded "hydraulic," and one was coded "hydraulic debris"); superimposed H denotes sites where a hurricane or tropical storm occurred in the region of the collapsed bridge and may have influenced the flow on the collapse date (14 bridges); the collapse date was known or confirmed for 13 bridges; area of circles is proportional to drainage area of the bridge site (range: 46–5,920 km^2); sites are plotted as semiopaque in inset maps for visual clarification; bridge and gauge data are provided in Table 1; additional maps are available in a permanent repository (Flint et al. 2016)

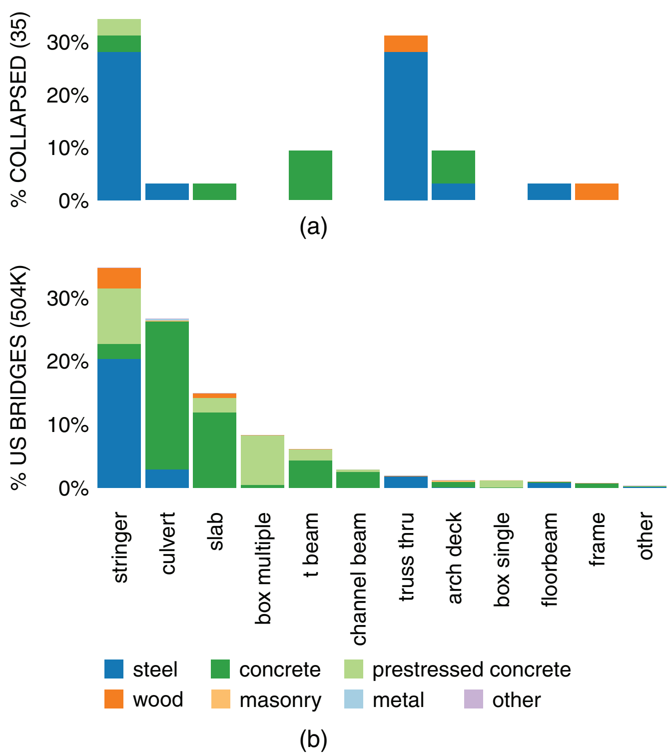

Fig. 3. (Color) Comparison of bridge type and construction material: (a) 35 collapsed bridges (data from NYDSOT 2014b); (b) approximately 504,000 U.S. bridges over water (data from FHWA 2012); for U.S. bridges, no distinction was made on the basis of span continuity (e.g., "steel" and "steel continuous" were combined); only structure types with more than 1,000 in existence (>0.2% of total) are plotted; the collapsed bridges are not representative of all U.S. bridges over water in terms of type, material, or age, although the dissimilarity is partially attributable to the relative overrepresentation of collapsed bridges in three states

with the daily mean values ($\rho = 0.925$, $\tau = 0.689$, and $R^2 = 0.855$) and were also biased high, as would be expected (linear regression slope: $m = 1.91$). It would also be expected that instantaneous/peaks and daily mean flows would be better correlated for large basins, and indeed that was the case for the eight basins with a drainage area greater than 1,000 km^2 where instantaneous/peaks collapse flows were available ($\rho = 0.952$, $\tau = 0.923$, and $R^2 = 0.907$), with a slight increase in the bias ($m = 1.93$).

Because of the temporal coincidence of collapse and maximum flows at 21 of the 31 sites plotted in Fig. 5(a), the correlation of maximum recorded daily mean and maximum recorded peak flows would be expected to be similar to the collapse flow correlation, which was the case considering all 34 gauge sites ($\rho = 0.889$, $\tau = 0.761$, $R^2 = 0.791$, and $m = 1.67$). Again, the maximum flows were better correlated for the 11 large basins ($\rho = 0.908$, $\tau = 0.891$, and $R^2 = 0.824$), although the bias was greater ($m = 1.90$).

Return Periods of Collapse and Maximum Flows

Bulletin 17B Analyses

The return periods of collapse and maximum flows obtained using different types of flow data are quantified in Table 2. Like the flow values, the collapse return periods varied considerably between the bridges as evidenced by the four-order-of-magnitude range of the estimated values (daily mean: 1–1,644 years; instantaneous/peaks: 1.2 to >10,000 years). In addition to the large range, the distributions were highly skewed (daily mean: 2.5; instantaneous/peaks: 4.0). The high variability and skew decrease the representativeness of central values and indicate that the median value may be more representative than the mean. The discrepancy in median value between that obtained using daily data (43 years; 35 bridges) and that obtained using instantaneous or peaks data (120 years; 31 bridges not including several below-maximum collapses) was large. The distance between the means of the daily mean (216 years) and instantaneous/peaks (674 years) return periods was found to be statistically significant.

Table 2. Flow values Q and Return Periods T_R of Collapse and Maximum Events Obtained using the Bulletin 17B Methodology

Year built–year collapsed–state	STAID	Drain area (km^2)	Q (m^3/s) Collapse D	Collapse I/P	Maximum D	Maximum P	T_R (year) Collapse D	Collapse I/P	Collapse Reported	Maximum D	Maximum P	Cause
1857–1987–Maine	01031500	769	898	1,060^{P}	898	1,060	1,042	314^{P}	>100^{a}	1,042	314	Flood
1910–1982–Arkansas^{b}	07075000	781	2,120	6,820^{P}	2,120	6,820	729	10,456^{P}	—	729	10,456	Flood
1915–1982–Arkansas^{b}	07069500	3,007	3,170	6,910^{P}	3,170	6,910	306	3,037^{P}	—	306	3,037	Flood
1916–1946–Washington^{b}	12087000	213	158	226^{P}	158	226	42	87^{P}	$10 < T_R < 25^{c}$	42	87	Flood
1920–1981–New York	04234000	320	200	337^{P}	234	439	180	308^{P}	—	472	1,363	Flood
1926–1987–Maine	01049500	561	112	120^{P}	122	142	23	19^{P}	—	36	43	Flood
1929–1951–Kansas^{b}	06891500	1,103	640	685^{P}	640	685	129	64^{P}	—	129	64	Flood
1935–1996–Virgina$^{H+,b}$	01624800	187	130	459^{P}	130	459	26	157^{P}	—	26	157	Flood
1936–1985–West Virgina^{H,b}	01611500	1,748	759	—	1,920	2,480	15	—	—	217	98	Flood
1940–1996–Maryland$^{-,b}$	01596500	124	57	187^{P}	74	213	22	161^{P}	—	90	282	Flood
1950–1989–New York$^{+,b}$	04216500	62	19	82^{P}	46	82	3	53^{P}	—	140	53	Flood
N/A–1985–Virgina$^{H+,b}$	02021500	851	1,180	2,480^{P}	1,180	2,480	502	565^{P}	>100^{d}	502	565	Flood
N/A–1985–West Virgina^{H+}	01606500	1,684	2,180	3,680^{P}	2,180	3,680	1,663	401^{P}	>100^{d}	1,663	401	Flood
1925–2011–New York^{H++}	01349711	13	39	122^{I}	39	122	124	26^{I}	—	124	26	Scour
1928–2011–New York^{H}	01349810	73	188	541^{I}	188	541	95	93^{I}	—	95	93	Scour
1929–2011–New York^{H}	01387400	231	232	388^{I}	232	388	42	50^{I}	—	42	50	Scour
1936–1989–Tennessee	07030050	5,967	244	—	1,580	1,580	1	—	—	30	23	Scour
1936–2011–New York^{H++}	01387450	32	44	105^{I}	44	105	78	119^{I}	—	78	119	Scour
1938–1964–Montana	12358500	2,939	2,620	3,960^{P}	2,620	3,960	1,647	3,699^{P}	—	1,647	3,699	Scour
1940–1987–New Hampshire^{b}	01090800	163	44	47^{P}	48	50	11	12^{P}	—	15	16	Scour
1941–1989–Mississippi^{b}	02482550	3,478	411	416^{I}	2,660	2,890	1	1^{I}	—	476	595	Scour
1950–2004–Missouri$^{-,b}$	06927000	667	120	196^{I}	564	1,590	1	1^{I}	—	79	1,654	Scour
1955–2005–New York$^{-,b}$	01365000	99	59	178^{P}	102	232	7	12^{P}	15^{e}	41	25	Scour
1959–1998–New York	04273800	176	137	204^{I}	137	204	155	122^{I}	—	155	122	Scour
1962–1972–Maryland$^{H-,b}$	01651000	127	99	510^{P}	155	510	24	81^{P}	—	80	81	Scour
1968–1995–South Carolina^{++}	02164000	125	2	—	117	165	1	—	—	232	93	Scour
1991–1999–Mississippi$^{-,b}$	02487500	1,099	244	250^{I}	852	909	2	2^{I}	—	134	92	Scour
N/A–1962–South Dakota^{-}	06480000	8,138	147	—	883	960	4	—	25^{f}	277	255	Scour
N/A–1986–Nevada^{b}	10312000	3,800	379	470^{P}	566	631	76	119^{P}	>100^{g}	265	308	Scour
1929–1972–Maryland^{H+}	01648000	136	142	354^{P}	142	354	126	170^{P}	—	126	170	Hurricane
1900–1971–Maryland^{H,b}	01591000	89	73	617^{P}	73	617	36	138^{P}	—	37	138	Other
1920–1972–Maryland^{H,b}	01643000	2,112	2,100	2,310^{P}	2,100	2,310	317	187^{P}	—	317	187	Other
1930–1972–Maryland$^{H--,b}$	01643500	163	156	912^{P}	156	912	61	384^{P}	>100^{h}	61	384	Other
1953–1995–Virginia$^{-,b}$	01662800	66	50	258^{P}	57	258	36	35^{P}	500 (storm)^{i}	48	35	Other
N/A–2004–Virginia^{H,b}	01662800	66	27	184^{P}	57	258	8	18^{P}	—	48	35	Other
Median (mean)	—	—	—	—	—	—	43 (216)	120 (674)	—	126 (280)	123 (718)	—

Note: Collapse flow return periods are also provided from other reports; the superscript H indicates link to hurricane at time of failure; plus sign indicates a bridge drainage area that is >1.2 times greater than the gauge drainage area; two plus signs indicate a bridge drainage area that is >1.4 times greater than the gauge drainage area; minus sign indicates a bridge drainage area that is <0.8 times smaller than the gauge drainage area; two minus signs indicates a bridge drainage area that is <0.6 times smaller than the gauge drainage area. D = daily mean; I = instantaneous; P = peak.

[a]Fontaine and Nielsen (1994).

[b]Collapse date was assumed.

[c]Sumioka et al. (1998).

[d]Lescinsky (1986).

[e]Suro and Firda (2007).

[f]Rostvedt (1968).

[g]Paulson et al. (1988).

[h]Bailey et al. (1975).

[i]Pontrelli et al. (1999).

As shown in Fig. 4(b), there were both consistencies and discrepancies when comparing daily mean and instantaneous/peaks data at individual sites. Using daily mean data, 19 sites were found to have experienced a maximum flow with a return period greater than 100 years. Of these sites, 11 had a collapse flow that coincided with the maximum flow and thus also were found to have a collapse flow return period greater than 100 years. Seven of these bridges had a confirmed collapse date. Using peaks data, 19 sites were estimated to have experienced a 100-year flow (only 13 of these sites were also found to have a 100-year flow using daily mean data). Of the 31 bridges for which instantaneous or peaks collapse flow data were available, 16 bridges were found to have collapsed during a flow with a return period greater than 100 years, 13 of which coincided with the maximum flow. Seven of these bridges had a confirmed collapse date (one of which was not found to have collapse flow return period greater than 100 years using daily mean data). Conversely, five bridges that had experienced a flow with a return period greater than 100 years later collapsed during a smaller flow.

Fig. 4(b) also compares the estimated collapse flow return periods with those reported from other studies (Bailey et al. 1975; Fontaine and Nielsen 1994; Lescinsky 1986; Paulson et al. 1988; Pontrelli et al. 1999; Rostvedt 1968; Sumioka et al. 1998; Suro and Firda 2007). The references for the individual collapses are

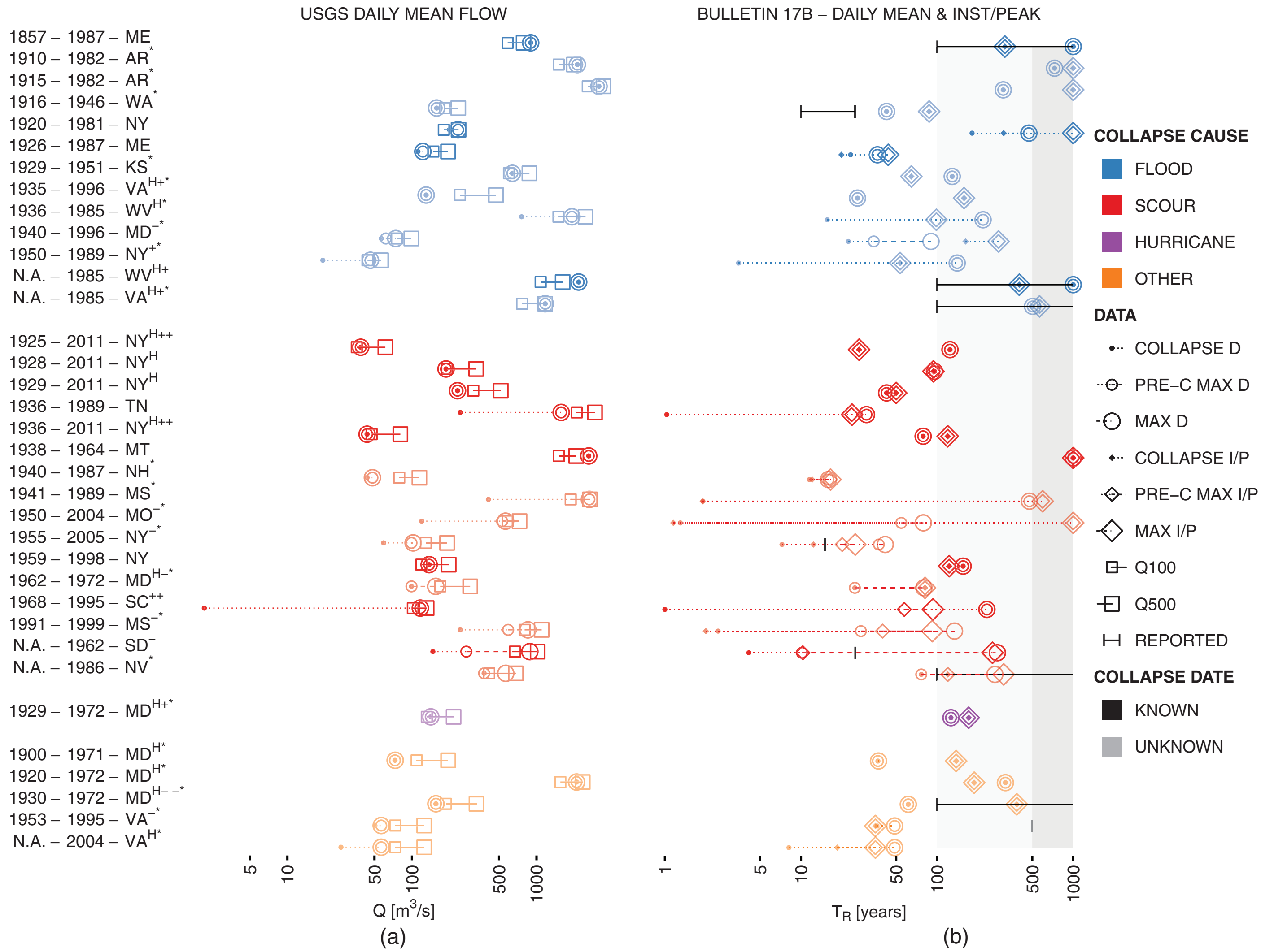

Fig. 4. (Color) Collapse, maximum recorded, and maximum recorded precollapse: (a) USGS daily mean flows, Q; (b) return periods estimated using the Bulletin 17B methodology, T_R; the superscript H indicates a link to hurricane at time of collapse; + and ++ indicate a bridge drainage area that is >1.2 and >1.4 times larger than the gauge drainage area, respectively; — and — — indicate a bridge drainage area that is <0.8 and <0.6 times smaller than the gauge drainage area, respectively; * indicates an unknown collapse date; return periods greater than 1,000 years have been plotted at 1,000 years; values are provided in Table 2; gray shaded regions in (b) denote design and check flood return periods assumed in modern bridge design for flood (lighter gray) and scour (darker gray); black and gray bars and lines demarcate collapse flow and storm return periods, respectively, obtained from other reports

provided in Table 2. Of the eight bridges with flow-related reports available, three collapse return periods estimated using daily mean data agreed with the report; in four cases, the Bulletin 17B return periods estimated using daily mean flows were lower than provided in the report. When using instantaneous and peak flows, five of seven collapse return periods were consistent, and only one site had an estimated value lower than reported. Variations in flood frequency analysis methods, or Bulletin 17B assumptions—including the use of regional skew, inclusion of historical peaks, or treatment of low outliers—may have contributed to discrepancies in the estimated return periods. Different assumptions with regard to skew, e.g., the use of station as opposed to regional skew, would be expected to have a particularly strong effect.

Plots highlighting the correlation between different estimates of collapse flow return periods are provided in Figs. 5(b–d), with correlation statistics provided in Table 3. Unlike the flow values, the collapse flow return periods estimated using daily mean or instantaneous/peaks were not well correlated ($\rho = 0.420$; $\tau = 0.656$; $R^2 = 0.721$), although the bias ($m = 1.89$) was similar to that found for collapse flows. Compared with collapse flows [Fig. 5(a)], collapse return periods [Fig. 5(b)] do not show as large a distinction on the basis of drainage area between daily mean and instantaneous/peaks values.

The maximum flow return periods obtained using daily mean and instantaneous/peaks data were poorly correlated ($\rho = 0.397$, $\tau = 0.519$, and $R^2 = 0.158$). The correlation was worse than found for maximum flows and collapse return periods. The bias ($m = 1.83$) was similar to that found for maximum flows.

In summary, the Bulletin 17B methodology produced sometimes substantially different collapse and maximum return periods, depending on the type of flow data used, and on the whole tended to underestimate the return periods if the instantaneous/peaks values were taken as the most reliable (which would be in line with assumptions made in practice).The difference between daily mean and instantaneous flows was expected to be larger at sites with a small drainage area, as explained previously. This effect of drainage

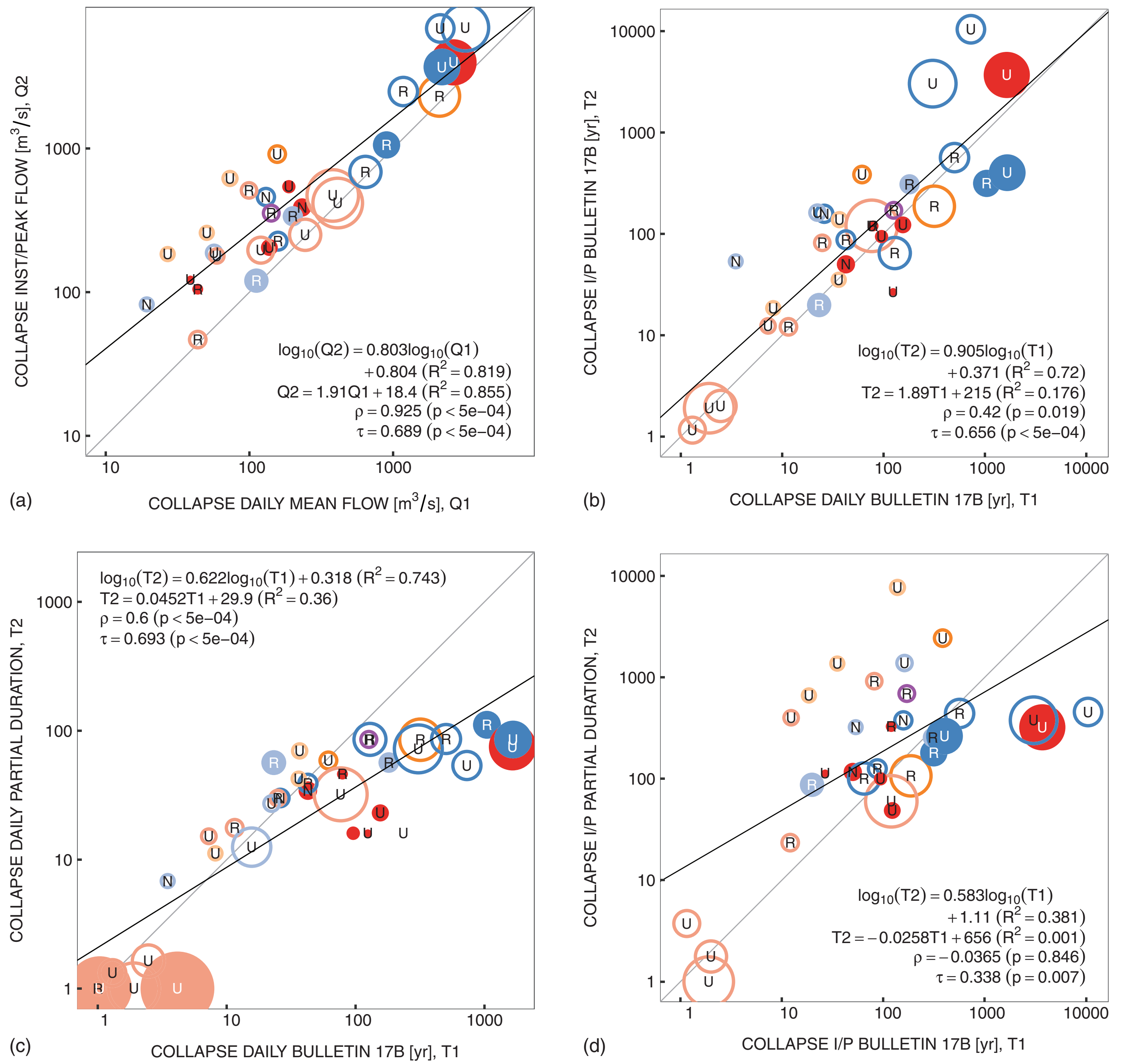

Fig. 5. (Color) Correlation of collapse flows and return periods: (a) daily mean versus instantaneous/peak flows; (b) daily mean versus instantaneous/peak Bulletin 17B return periods; (c) daily mean Bulletin 17B versus partial-duration return periods; (d) instantaneous/peak Bulletin 17B versus partial-duration return periods; filled circle = collapse date known; open circle = collapse date unknown; superimposed U = unregulated; R = regulated; N = no information on degree of regulation available; full opacity = collapse and maximum flow date coincided; area of circles scales with gauge drainage area and is consistent across (a) to (d)

Table 3. Slope of Linear Regression (m) and Measures of Correlation (Pearson ρ and Kendall τ) of Collapse and Maximum Recorded Flows, Q, and Return Period Estimates, T_R

	USGS		Bulletin 17B		Partial duration	
	I/P versus D		I/P versus D		D versus D	I/P versus I/P
Statistics	Q collapse	Q maximum	T_R collapse	T_R maximum	T_R collapse	T_R collapse
m (R^2)	1.91 (0.855)	1.67 (0.791)	1.89 (0.176)	1.83 (0.158)	0.045 (0.36)	−0.026 (0.001)
ρ (p)	0.925 ($<10^{-3}$)[a]	0.889 ($<10^{-3}$)[a]	0.420 (0.019)	0.397 (0.018)	0.600 ($<10^{-3}$)[a]	−0.036 (0.846)
τ (p)	0.689 ($<10^{-3}$)[a]	0.761 ($<10^{-3}$)[a]	0.656 ($<10^{-3}$)[a]	0.519 ($<10^{-3}$)[a]	0.693 ($<10^{-3}$)[a]	0.338 (0.007)

Note: Partial-duration statistics are relative to Bulletin 17B values. D = daily mean; I = instantaneous; P = peak.
[a]p-Value is approximate.

area, combined with the use of regional skew values derived for use with annual peaks data and the relatively lower variability of daily mean as opposed to annual peaks data, may account for the smaller number of very large (>1,000 year) return periods in the analysis using daily mean data. Despite this difference in the tails of the return period distributions, there were no nonoverlapping 5–95% confidence intervals when collapse flow return periods were compared between daily mean and instantaneous/peaks data in the Bulletin 17B analyses. Despite this consistency, these findings give cause for concern in implementing Bulletin 17B using climate projection data, which are generally only available at large spatial resolution—larger than some of the small watersheds—and at daily or greater temporal resolution.

Partial-Duration Analysis

Partial-duration analysis compares a given flow value to the entire flow record and can be thought of as an empirical, rather than parametric, probability distribution. Fig. 5(c) compares annualized partial-duration collapse flow return periods with those obtained from Bulletin 17B using daily mean data. Correlation statistics are also provided in Table 3. The return period estimates were not well correlated ($\rho = 0.600$, $\tau = 0.693$, and $R^2 = 0.360$) and were not improved when only the 31 sites considered in all other correlations presented were included (i.e., those that had both instantaneous and peaks data). The slope of the linear regression indicated a near lack of a relationship between the return period estimates ($m = 0.045$). Additionally, 21 of 35 partial-duration return periods were outside the 5–95% confidence intervals of the Bulletin 17B return periods.

The correlation was substantially worse when instantaneous/peaks data were used in the partial-duration and Bulletin 17B analyses, as shown in Fig. 5(d). Correlation statistics were poor to very poor ($\rho = -0.037$, $\tau = 0.338$, and $R^2 = 0.001$), and the linear regression again indicated a near lack of relationship ($m = -0.03$). A larger percentage of the instantaneous/peaks partial-duration return periods (20 of 31) were inconsistent with the Bulletin 17B confidence intervals.

As shown in Fig. 5(d), when instantaneous/peaks data were used, the partial-duration return periods of sites with a smaller drainage area (<100 km^2) were strongly biased ($m = 31.1$ and $R^2 = 0.340$), whereas sites with a medium-to-large drainage area (≥100 km^2) had a very weak relationship ($m = 0.011$ and $R^2 = 0.002$): The failure events for small sites were estimated to be more extreme using partial-duration analysis, which is in line with fact that instantaneous flows capture short-duration peaks. A distinction between smaller and larger drainage area sites was not found using the daily mean data, even when three sites with a large drainage area (>1,000 km^2) and very low collapse return periods—and where instantaneous or peak collapse data were not available—were excluded from the analysis. These results suggest that it may not be advisable to use partial-duration analysis using only daily mean data for sites with a small drainage area.

Relationship between Event and Site Characteristics and Collapse Return Periods

Although the sample size was insufficient to analyze trends in collapse flow return period related to bridge age, type, or material, certain commonalities between the bridges appeared to be robust.

- Coincidence of collapse and maximum flows: When collapse and maximum recorded flows coincide, the collapse, by definition, has occurred during an unprecedented flow at the site, and the flow would not have been available in the record at the time of bridge design. As shown in Fig. 4(a) and Table 2, more than half of the bridges collapsed during the maximum recorded flow up to and including the collapse flow. This finding may be partly explained by recall bias in that these extreme events may be more likely to be remembered or reported on and therefore be recorded in the NYSDOT database. Regardless of whether the true frequency of collapse and maximum flow coincidence is represented by the 35 bridges studied, this finding suggests that many bridge collapses can be attributed to unprecedented events that could not have informed the bridge design.
- Collapse cause: On the basis of the historical evolution of bridge design standards, it would be expected that collapse flow return periods would vary according to collapse cause. As shown in Figs. 4(b) and 5, some trends were indeed identified and evaluated using the Mann-Whitney test. Flood collapse return periods were found to be significantly higher than nonflood return periods in all Bulletin 17B analyses and the daily mean partial-duration analysis. Conversely, scour collapse return periods were significantly lower than the nonscour return periods in all analyses. Bridges that collapsed from scour were the most likely to collapse during a nonmaximum flow, which would be consistent with the lack of requirement for scour design for those bridges as indicated by their pre-1991 construction years (although it is certainly possible that some of these bridges were designed for scour or had scour mitigation in place). This finding was statistically significant by the chi-square test at a lower significance level ($\alpha = 0.10$). Although the lack of requirements for scour design could plausibly explain the lower scour return periods, it is also possible that the flow metric used (volumetric rate) does not accurately describe the conditions leading to scour, given that scour may accumulate over a number of floods. Explicit modeling of the scour process or consideration of a series of floods could be used to better characterize scour collapse risk in the context of other hydraulic causes.
- Hurricanes: Although tropical cyclones occur on a seasonal basis, a particular region may go decades between events. The lengthy time between occurrences decreases the likelihood that a hurricane-related flow will be included in the record at the time of design and motivates the use of mixture analysis of the flood records, i.e., considering as separate the processes generating peak flows from storms or snowmelt as opposed to tropical cyclones. As shown in Figs. 2 and 4 and Table 1, there were hurricanes near 14 of the 35 collapsed bridges during or a few days before the assumed collapse date (despite only one bridge being labeled a hurricane collapse). The collapse date was known at six sites with linked cyclones (causes: one "Agnes"; four "scour" during Hurricane Irene; one "flood" during Hurricane Juan). For the other eight sites, a hurricane was linked to the maximum flow in the year of collapse but could not be definitively linked to the collapse itself (causes: three "flood"; one "scour"; four "hydraulic"). The prevalence of hurricane-related collapse may therefore be overstated. Assuming that all 14 cases were truly hurricane-related, these collapses were more often linked to unprecedented events: They coincided with the maximum recorded flow at 10 of 14 sites versus 9 of 21 for the nonhurricane collapses. An additional two hurricane-related collapses occurred during the maximum flow recorded precollapse. The hurricane-related collapse flow return periods were found to be significantly higher than the non-hurricane-related return periods using the Mann-Whitney test in the daily mean Bulletin 17B and partial-duration analyses ($\alpha = 0.10$) and the instantaneous/peaks partial-duration analysis ($\alpha = 0.05$). However, 11 of the 14 hurricane-related collapses were linked to one of three events (Agnes, Irene, or Juan), suggesting that these collapse flow return periods are not independent and that the

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statistical analysis of hurricane-related return periods may not be robust. Furthermore, the collapses linked to Hurricane Juan and Tropical Storm Heidi (Table 1) did not occur during the storms but rather a few days after, although NOAA reports that increased moisture availability contributed to significant flooding in the aftermath of these events. Further caution in interpreting these findings is suggested because mixture analysis (which was not performed) is known to affect the distribution upper tails, and the return periods estimated could be improved using such approaches (Villarini and Smith 2010). Despite these reasons for caution, analyses of future collapse risk would likely benefit from considering the changing exposure to tropical cyclones.

- Regulation: The presence of flood control structures (e.g., dams) regulating high flows would be expected to reduce the occurrence of very large flows. As shown in Fig. 5, when gauges were classified as regulated or unregulated (regardless of the degree or type of regulation), it was found that unregulated streams were associated with higher maximum and collapse return periods only in the partial-duration analysis when daily mean data were used. When only sites with possible regulation at high flows were considered, there was no significant difference in the daily mean partial-duration (or any other) analysis. These findings suggest that, contrary to expectation, regulation was not particularly influential for bridges that actually collapsed, although presumably regulation may have prevented other collapses. Although the importance of regulation cannot be assessed by considering only collapsed bridges, this finding does imply that it may be possible to directly use hydrological projections to assess changes to collapse risk without performing additional analyses to correct for regulation.
- Drainage area: Sites with a large drainage area would be expected to produce similar estimates of collapse flow return periods using daily mean and instantaneous/peaks data. As best shown in Fig. 5, drainage area had mixed effects depending on analysis and data type. Considering collapse, Bulletin 17B return periods obtained using instantaneous/peaks data and associated with sites with a drainage area that is less than 100 km^2 were significantly lower than return periods associated with sites with a larger drainage area [Fig. 5(b)]. Conversely, in the partial-duration analysis with instantaneous/peaks data, sites with a smaller drainage area (<100 km^2) had significantly higher return periods, whereas sites with a larger drainage area (>1,000 km^2) had significantly lower collapse return periods [Fig. 5(d)]. These mixed results are somewhat encouraging with regard to the potential validity of using climate projections to characterize change in collapse risk, as drainage area seems to be most critical when instantaneous/peaks data are used, and instantaneous/peaks data are rarely available in climate projections.

Trends in Annual Peak Flows

A critical question for managing bridge collapse risk in the future is whether the probability of high flow is changing. Through the Mann-Kendall test, 10 of 34 gauge sites were found to have statistically significant trends in annual peak flows over the recorded period (Table 1 and Fig. 6). Only one site with a significant trend had an annual peak flow series of less than 50 years, named as a criterion for robustness (Hirsch and Ryberg 2012; Kundzewicz and Robson 2004). Of the sites with statistically significant trends, nine had increasing trends in annual peak flow. Eight of the sites with statistically significant increasing trends had a below-median

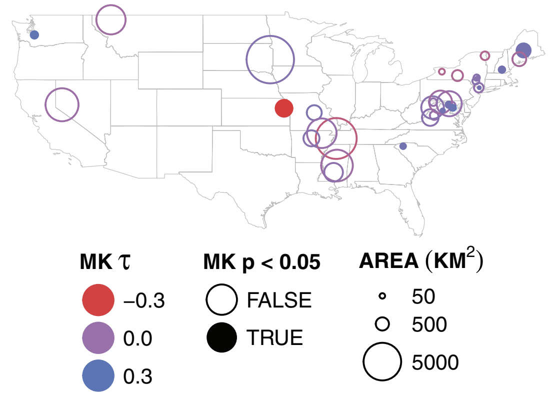

Fig. 6. (Color) Trends in annual peak flows at 34 stream gauge sites assessed using the Mann-Kendall (MK) test for monotonic trends; area of circles scales with gauge drainage area (range: 12–8,140 km^2); nine sites had a statistically significant increasing trend; one site had a statistically significant decreasing trend; trend directions, statistical significance, and number of annual peaks used in the analysis are provided in Table 1

drainage area (<276 km^2). The site with a negative trend had the largest drainage area of the significant sites (1,104 km^2) and was regulated by a dam upstream. Only two of the sites with significant trends were unregulated according to USGS, although the regulated sites were frequently regulated only at low flow or had only small diversions.

The results of the trend analysis were consistent with other studies (Hirsch and Ryberg 2012; Kundzewicz and Robson 2004; Groisman et al. 2001; Mallakpour and Villarini 2015; Villarini et al. 2009; Milillo et al. 2014) in that statistically significant trends in annual peak flows were not found at the majority of sites. Trends could be affected by changes in land use, streamflow regulation, or climate. The presence of statistically significant increasing trends suggests that the collapse-inducing flood could be expected to occur more frequently in the future at nine sites. More detailed analyses of trends at these sites, e.g., using peaks-over-threshold approaches, and analysis of local changes in land use would support a more robust estimate of the change in bridge collapse rates resulting from increasing annual peak flows. In addition, further analysis of the interacting effects of land-use, streamflow-regulation, and climate change could help characterize changes in future risk across the United States.

Collapse Risk and Potential Impact of Climate, Land-Use, or Regulation Change

Failure probabilities obtained from reasonable nominal reliabilities are compared with those obtained using the estimated return periods of the collapse flows in Table 4. The nominal lifetime and nominal event reliability approaches produced annual failure probabilities in the range of 0.023–0.08% annually (1.7–5.8% over a 75-year lifetime). These probabilities are of the same order of magnitude as previous annual collapse rate estimates (1 in 5,000 or 0.02%) (Cook et al. 2015; Arneson et al. 2012; Kattell and Eriksson 1998), which are inclusive of all possible collapse causes and thus more comparable to the nominal lifetime reliability value.

The predicted annual failure probabilities derived using central values (median and mean) of the Bulletin 17B collapse return periods ranged from 0.15% (mean value of instantaneous/peaks values) to 2.4% (median of daily mean values), resulting in lifetime failure

Table 4. Analysis of Annual Failure Probability, $p_{f,a}$, and 75-Year Lifetime Failure Probability, $p_{f,l}$, Using Nominal Lifetime and Flood-Event Reliability; Central Values of Collapse Flow Return Periods, T_R, Estimated using the Bulletin 17B Methodology; and Kernel-Smoothed Distributions of Collapse Flow Return Periods Estimated using the Bulletin 17B Methodology

Risk basis	Analysis	Data and assumptions	T_R	$p_f \mid T_R$	$p_{f,a}$	$p_{f,l}$	$\%\Delta p_{f,a} \mid \Delta_{\text{flood frequency}}$	
							+10%	−10%
Nominal	Lifetime	$\beta = 3.5$	—	—	0.00023	0.017	—	—
	Event	$\beta = 1.75$	50	0.04	0.00080	0.058	11	−9
		$\beta = 1.75$	100	0.04	0.00040	0.030	13	−10
Collapses	Median	D	43	1	0.024	0.84	8	−13
		I/P	120	1	0.0084	0.47	11	−10
	Mean	D	216	1	0.0046	0.29	13	−9
		I/P	674	1	0.0015	0.11	7	−13
	Kernel	D	All	Varies	0.011	0.56	18	−19
		I/P	All	Varies	0.0049	0.31	25	−16

Note: $\Delta p_{f,a}$ is the relative change in annual failure probability produced by a shift in the flood frequency curve that increases or decreases the frequency of any given flood event by 10%. D = daily mean; I = instantaneous; P = peak.

probabilities of 11–84%. The much lower failure probabilities produced by instantaneous/peaks data are partially attributable to the lack of several scour failures with very low return periods in this data set. The collapse data–derived failure probabilities were one to two orders of magnitude larger than the nominal values, which was expected given that they provide a pessimistic view of bridge reliability (as they include only bridges that did collapse).

Using the kernel-smoothed, full-distribution, approach, the annual failure probabilities fell in between the median- and mean-derived values, at 0.49–1.1% annually (31–56% lifetime). Depending on what central value was taken as the baseline, using the kernel-smoothed distribution produced a relative change in the estimated failure probability of −53 to +232%. The large relative changes produced by considering a range of potentially collapse-inducing floods indicate that the variability of collapse return periods was significant in assessing collapse risk.

The potential impact of a change in the underlying frequency of flooding events is provided in Table 4 ($\Delta p_{f,a} \mid \Delta_{\text{flood frequency}}$). When any given flood was assumed to occur 10% more frequently, nominal event annual failure probabilities increased by slightly more than 10%, and the central-value collapse-estimated failure probabilities increased in the range of 7–13%. The kernel-smoothed results produced substantially larger increases in annual failure probability: 18% using daily mean flow data and 25% using instantaneous/peaks data. Similar trends were produced when the frequencies of floods were assumed to decrease by 10%: the kernel-smoothed results were more sensitive to changes in the hazard curve. This increased sensitivity resulted from the kernel density convolution approach's use of the entire hazard curve, as opposed to considering only one flood return period in the nominal event and central-value approaches. Although it may not be realistic to assume a uniform shift in the flood hazard curve, these results indicate that changes to the full distribution of floods should be taken into account to accurately capture the impact of climate and land-use change on collapse rates.

Summary and Conclusions

Major findings include the following:

1. Bridge collapses frequently coincided with the maximum flow recorded at the gauge site (daily mean flow: 19 of 35; peaks: 21 of 31) and also frequently coincided with tropical cyclones (14 of 35), suggesting that, in many cases, collapses occur during unprecedented or rare events. Collapse flows obtained using daily mean and instantaneous/peaks data were reasonably well correlated ($\rho = 0.925$, $\tau = 0.689$, and $R^2 = 0.855$) although biased ($m = 1.91$); correlation was improved when only sites with a large drainage area (>1,000 km^2) were considered ($\rho = 0.952$, $\tau = 0.923$, $R^2 = 0.907$, and $m = 1.93$).
2. The return periods of the collapse flows varied considerably, with ranges of 1–1,644 (1.2 to >10,000) years and 1–111 (1–7,710) years for Bulletin 17B and partial-duration analyses using daily mean (instantaneous/peaks) data, respectively. The estimates of collapse return periods using the Bulletin 17B methodology and different flow data (daily mean and instantaneous/peaks) were poorly correlated ($\rho = 0.420$, $\tau = 0.656$, $R^2 = 0.721$, and $m = 1.89$) but consistent, given the uncertainty in the estimates. Partial-duration return periods were very poorly correlated with Bulletin 17B return periods, especially using instantaneous/peaks data ($\rho = -0.037$, $\tau = 0.338$, $R^2 = 0.001$, and $m = -0.03$); 21 (20) of 35 (31) were outside the Bulletin 17B confidence intervals using daily mean (instantaneous/peaks) data.
3. Scour-induced collapses were associated with lower return periods, whereas flood-induced and hurricane-related collapses were associated with higher return periods. No association was found between presence of regulation or drainage area.
4. There was a statistically significant trend of increasing annual peak flows in the historical record at 9 of the 34 gauge sites; these sites tended to have relatively small drainage areas. One site had a significant decreasing trend; this site had a relatively larger drainage area.
5. The use of the full distribution of collapse return periods produced more robust estimates of failure probabilities than central-value-based approaches and was more sensitive to hypothetical uniform ±10% shifts in the flood hazard curve (daily mean: $+18/-19\%$; instantaneous/peaks: $+25/-16\%$) than the nominal event ($\beta = 1.75$ for 100-year flow: $+13/-10\%$) and central-value-based approaches (mean from daily mean flows: $+13/-9\%$).

Using the Bulletin 17B methodology and daily mean (instantaneous/peaks) flow values, 23 (15) of 35 (31) bridges were estimated to have collapsed during a flow with a return period of less than 100 years, which is a return period frequently considered in bridge climate impact assessments and also used in modern bridge design guidelines. Overall, the finding that a significant portion of the bridges collapsed during sub-100-year flows was unsurprising given that (1) the majority of the bridges studied were built before modern design standards; (2) modern design standards require design for a 100-year flow only for federally funded interstate bridges, only one of which (collapsed in 1962) was studied; and (3) conditions at the time of collapse may have varied significantly from those assumed during design because of channel mobility, presence of debris, change in land use, or other factors.

Several aspects of the collapse analysis as a whole limit its broad interpretation. There were possible inconsistencies between the flow values at the bridge and gauge sites: at only 18 sites was the drainage area at the bridge site within 10% of the drainage area of the related gauge. The collapse date was not provided or not confirmed at 21 sites, although the assumption that collapse occurred during the maximum flow in the collapse year was true for 11 of 14 bridges with a known collapse date. Given these limitations—and the differences in bridge age, material, type, and geographic distribution—it cannot be said with confidence that the prevalence of sub-100-year collapse flows, and effect of collapse cause or other factors reported in this paper, is representative of all U.S. hydraulic bridge collapses, although the results may be more representative for New York, Maryland, and Virginia, which were overrepresented in the set of collapsed bridges. Discrepancies in return periods estimated using daily mean and instantaneous/peaks flow data, and using the Bulletin 17B methodology for flood frequency analysis or partial-duration analysis, were considerable in some cases, especially for sites with small drainage areas. Given that daily mean data at large spatial scales is most easily accessible in climate projections, more research is required to validate the use of such data to analyze possible changes in collapse risk.

Under the assumption that the findings in this paper are representative of the U.S. bridge stock—which there is substantial reason to doubt—a number of compounding factors suggest that estimates of the current and future risk of hydraulic bridge collapse require revision. The majority of U.S. bridges cannot be assumed to meet modern design standards because of their construction year; there is increasing probability of high flows at some sites in the historical period; and there is high likelihood of additional global warming and further land-use change by midcentury. The finding of high variability in collapse return periods suggests that analyses of the effect of climate and land-use change on bridge collapse risk should consider a range of floods, rather than a single (100-year) event, to ensure robustness. Furthermore, the reliability analysis demonstrated that failure probabilities were more sensitive to changes in flood hazard when variability in collapse return periods was considered. In combination, these two findings indicate that although previous assessments of the impact of climate change on hydraulic bridge performance have included uncertainty associated with bridge vulnerability, they likely underestimate the impact of change in flow frequency, whether positive or negative, because of their focus on the 100-year event. The analysis of historical collapses offers a method to constrain estimates of collapse rates, potentially increasing the accuracy of projections of future risk.

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