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Cedar Rapids Flood of 2008: Manmade or Statistical Anomaly?

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CEDAR RAPIDS FLOOD OF 2008:
MANMADE OR STATISTICAL ANOMALY?

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Introduction

During the month of June 2008, Eastern Iowa saw record river stages and flooding that devastated numerous communities. Cedar Rapids was one the most heavily impacted cities. On June 13th, according to the Iowa Department of Agriculture and Land Stewardship, the Cedar River crested at 31.12 feet, 19.12 feet above flood stage and 11.12 feet higher than the previous record set in 1929 ("Iowa Monthly Weather Summary", 2008, p. 1). When flooding like this occurs and homes and businesses are destroyed, the biggest question people want answered is how did this happen? More specifically, people want to know what caused the flood, how the flood affects them personally in the form of insurance, and how likely is it that a flood of this magnitude will happen again.

One of the main research questions for this thesis is what factors caused the flooding last summer and why was it so extreme? This is an important question to answer because it is vital to understand what causes a flood of this magnitude so communities can take steps to avoid or minimize damage that may result from floods in the future. The second question is what impact does this flood have on flood insurance? Flood insurance is one tool used to protect property owners from substantial loss due to flooding and plays a vital role in recovery after a flood, determining who is able to repair and continue occupying flood damaged properties. The third question is what is the probability of a flood like this recurring? This will show that even though the situation in Cedar Rapids was rare, floods are occurring more frequently and severely. The final question is how many years will pass until the next flood hits, being of same or greater level, and by how much will the next flood exceed the previous extreme record? The people of Cedar Rapids never imagined that a flood of this scale could happen, but it did and citizens must be prepared and aware that it is possible for a flood of greater proportions to hit.

This thesis will give information on the various factors that contributed to the flood so that more research can be done as to what changes will help avoid or minimize future floods. Also, statistical estimates from data analysis of recurrence intervals and flow duration will be found. These estimates will show that floods are occurring more frequently and are becoming more severe. It is necessary to understand more about flooding because as Cedar Rapids has shown, floods have the ability to devastate a community.

Causes of Flooding

A. Natural Causes

A flood can be defined as "the inundation of normally dry land by a stream overtopping its banks" (Burnett, 1993, p. 479). Floods begin naturally by having an excessive amount of rain or snowmelt when the soil is saturated and cannot absorb any more water, so water levels rise. This was the case in Eastern Iowa, where there were large amounts of snowfall and precipitation in the winter and spring leading up to June 2008. According to the National Climatic Data Center, "much of the Upper Mississippi and Ohio River Basins had experienced wet conditions during the 2007-2008 winter and into the spring," causing eastern Iowa to have a "Standardized Precipitation Index (SPI) [value] greater than +2," which normally occurs less than 2.5% of the time (NCDC, 2008, Flooding Summary section). This heavy precipitation in earlier months saturated Iowa's soil to the point where it couldn't absorb the additional precipitation that fell in June 2008.

Not only did Iowa receive higher levels of precipitation in the months leading up to the floods, but there was also an excessive amount of rainfall during the month of June that compounded the flooding problems. The rare weather conditions in the Midwest were perfect for creating extremely wet conditions. The heavy amounts of rain that Iowa received during the first half of June was due to a:

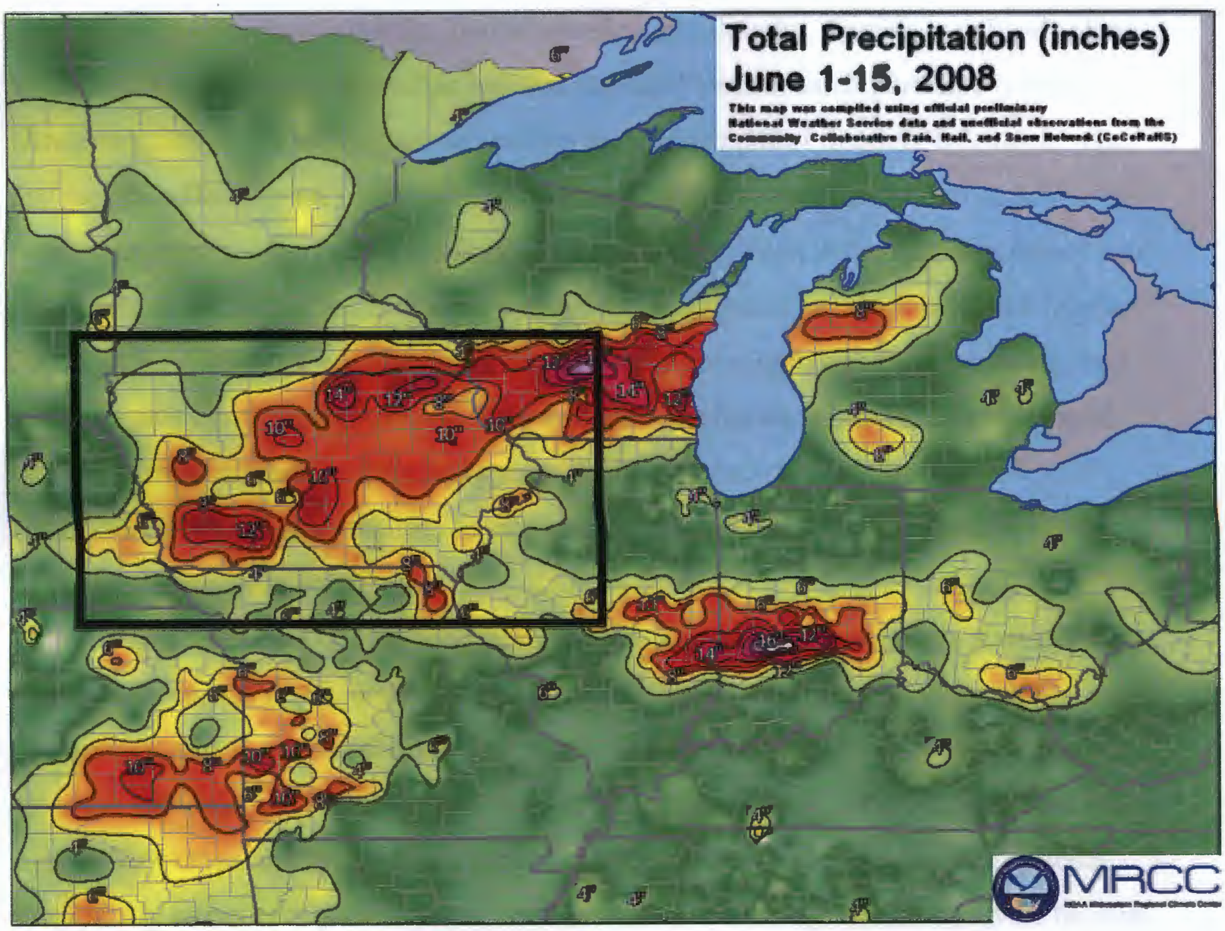
> large-scale weather pattern during the first two weeks of June primarily [consisting] of a high pressure system over the southern Plains and Ohio Valley and abnormally low pressure situated over the northern Plains. The boundary between these two pressure systems was the focal point for the development of the heavy rainfall and severe storms. (NCDC, 2008, Flooding Summary section)

A map showing the precipitation totals for the first half of June 2008 in Figure 1 shows that Iowa (located in the black rectangle), received some of the heaviest rainfall totals in the Midwest, ranging from 4 to 14 inches of precipitation. The map also shows that

Eastern Iowa received the heaviest totals for the state, which is where the most extreme flooding occurred.

The normal statewide average for rainfall during the period from May 29 through June 12 is 2.45 inches, but in 2008 the average was 9.03 inches ("Iowa Monthly Weather Summary – June 2008", 2008, p. 1). That is an additional 6.58 inches of rain. This significant amount of precipitation inundated Iowa's already saturated soil. The excess water had nowhere else to go but into the rivers and streams that were already swelling from precipitation received earlier in the year.

Figure 1
Source: *NCDC: Climate of 2008: Midwestern U.S. Flood Summary*



B. Manmade Causes

In general, as the human population has grown, people have moved closer to floodplains and rivers because they provide convenient sources of "irrigation, water transport, cooling-water supply, and waste-water discharge" (Burnett, 1993, p. 480). The use of land for agriculture and urban development has changed the landscape of floodplains, therefore increasing the risk of floods. Attempts to prevent flooding may have made conditions worse. Modifying drainage systems in order to prevent flooding can create flooding somewhere else. Ultimately, "any modification of the landscape has the potential to cause changes in the drainage system, and such changes can have severe consequences" (Nelson, 2007, Human Intervention section).

Before settlers came to Iowa, the land was a "vast tall grass prairie ecosystem, interspersed with upland savanna, prairie marshes and sloughs, riparian woodlands along small streams and rivers, as well as isolated stands of trees in small park-like groves" (Andersen, 2001). These natural wetlands were able to control flooding by storing water and reducing flood levels. However, the wetlands were drained for optimal agricultural conditions provided by Iowa's flat surface and rich soil. Now, water can only runoff into rivers, which are incapable of dealing with excess amounts of rain and snowmelt (Zedler, 2003). It has been approximated that "99 percent of the original wetlands, marshes, and small streams of north-central Iowa were drained and plowed, while the larger streams and rivers were dredged and straightened to facilitate removal of surface water" (Andersen, 2001). These changes have significantly impacted the efficiency of Iowa soil to manage large amounts of water.

Pipes, tiles, and ditches have been used to drain natural wetlands in states like Iowa where corn is now a dominant crop (Zedler, 2003). An example of this can be seen in Figures 2 and 3, where Johnson County has gone under a remarkable transformation from the 1850's to the 1990's. Land that used to be prairie and forest has now been taken over by row crops and urban development. These row crops require drainage systems that quickly move excess water from farmland into surrounding creeks and rivers because "most farmers object to standing water on their productive fields" (Hey, 2002, p. 94). The problem with these systems is that they move water into rivers too quickly and the rivers cannot handle that much runoff in such a short period of time.

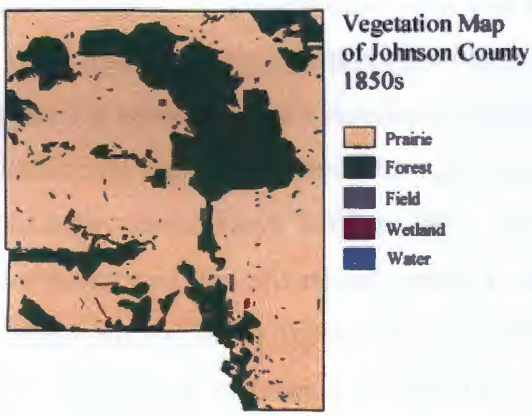
Levees are another manmade modification that can contribute to flooding. Levees were initially designed by engineers to protect land by preventing seasonal flooding of flood zones. The problem with levees is that, like agricultural drainage systems, they move water into rivers too fast. They also "focus the energy of [a] flood on a narrow section of the river system," causing a flood wave to grow, which becomes more destructive as it moves downstream (Hey et al., 2004, p. 4). Eventually, the flood wave overflows the channel and moves into the surrounding floodplain (Hey et al., 2004). Even though levees were created to solve the problem of flooding, they may only make the problem worse by not letting water flow naturally.

Urbanization has also increased the risk of flooding by changing land surface from soil to concrete. As cities continue to grow, soil is becoming compacted and land is being covered by roads, parking lots, buildings, and other structures. When land is cleared and paved for development, vegetation and soil that could naturally absorb water are removed and this creates greater runoff into rivers. Urbanization causes more frequent flooding and makes flooding worse because water moves too quickly into rivers and has nowhere else to go (Booth, 1991).

Once flooding does occur, structures that are built around rivers and in floodplains can add to the problem. Railways or roads that cut through floodplains can act as dams during flooding that accumulate water and floating debris. If the railway or road gives away, it creates a destructive flash flood wave that moves downstream to create even more problems (Langhammer, 2008, p. 62). This could have been the case in Cedar Rapids where numerous boathouses located along the Cedar River by Ellis Park floated downstream and smashed into the Quaker Oats railroad bridge that crosses over the river. Also, the Cedar Rapids and Iowa City Railway (CRANDIC) bridge collapsed and sent 20 rail hopper cars into the river "that were loaded with rock on the span in hopes of weighing the bridge deck down and keeping it in place" (Smith, 2008). It may be possible that these structures acted as dams that prevented water from flowing downstream.

By deciding to live in and modify natural floodplains, people risk being affected by floods. Human intervention can be reversed by restoring wetlands, adopting new agricultural practices, and dismantling levees. But it is very unlikely for urban residents to move away from rivers or accept "water being stored on their back lawns, driveways, or basements" (Hey, 2002, p.94). In Figure 4 (Radeloff et al., n.d.), it can be seen that most of the growth in housing density in the state of Iowa has occurred around major cities such as Des Moines, Iowa City, Cedar Rapids, and Waterloo, all of which located along the banks of major Iowa rivers and will continue to do so in the future. Therefore, individuals living near rivers must accept that flooding will continue to occur and more research and modeling must be done in order to assess the frequency and severity of floods in an attempt to minimize the damage that they may cause.

Figures 2 & 3
Source: *Iowa's* ***Statewide Land Cover Inventory***



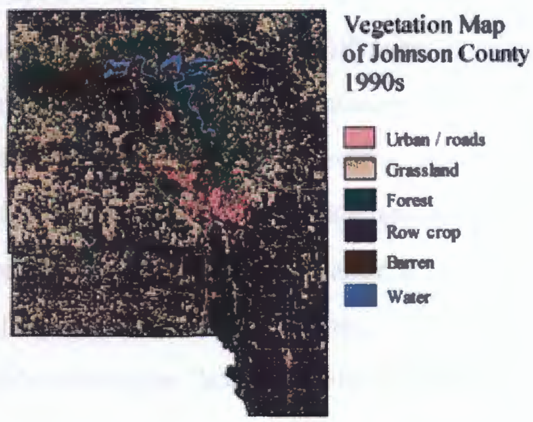
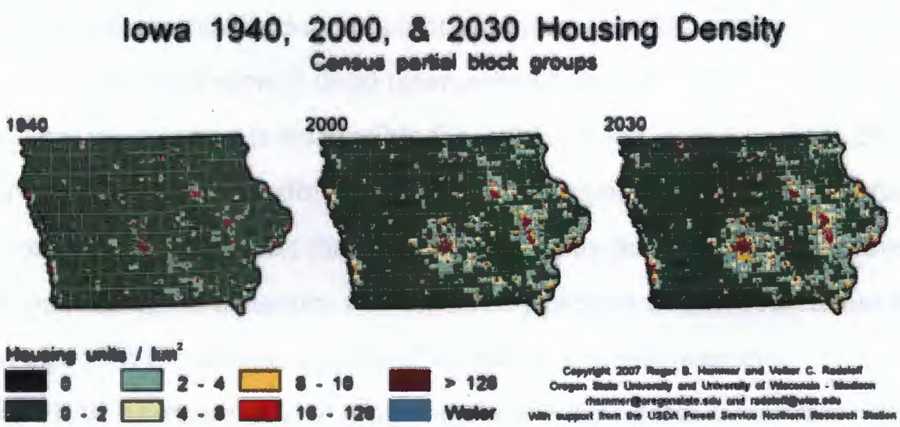


Figure 4
Source: *Housing Density Data*



Flood Insurance

Since flooding is accepted as a natural hazard that must be dealt with while living in flood zones, flood insurance has been developed to help cope with damage that may be endured. According to Burby (2001), there are more than six million buildings located within 100-year floodplains and "88% of U.S. counties [have] experienced at least one flood disaster during the second half of the twentieth century" (p. 111). According to Blanchard-Boehm, Berry, and Showalter (2001), more than 12% of the American population lives in areas that experience periodic flooding; that is more than 30 million people. With so many people and their property located in floodplains, there needs to be some form of financial protection from loss.

In 1968, the government formed its own flood insurance program to help victims of flood loss across the country. The National Flood Insurance Program (NFIP) was "created to provide flood insurance at subsidized rates to homeowners and business and to reduce the exposure to flood through land-use limits and other control measures" (as cited in Browne & Hoyt, 2000, p. 292). The NFIP brings together federal, state, and local government and also private insurance in order to provide helpful flood insurance to citizens. Burby (2001) explains the various roles of each contributor by saying:

> At the federal level, the Federal Insurance Administration (housed in the Federal Emergency Management Agency since 1979) sets flood insurance premium rates, identifies flood-hazard areas and the degree of flood risk, and establishes criteria for construction in floodplains. State governments authorize and assist local government regulation of building in floodplains. Flood insurance is not available until local governments adopt regulations that meet NFIP standards. Private insurers market and service flood insurance policies. (p. 112)

While the federal government is responsible for making flood insurance available, state and local government are responsible for making sure requirements are met so that flood insurance is used in an appropriate manner. Hopefully, by including different levels of government, national flood insurance will effectively prevent as much flood loss as possible and also provide insurance to flood victims in a timely manner.

The NFIP is a comprehensive program, but one aspect that sometimes makes it ineffective is that not every homeowner is required to own flood insurance and many go without it. The only property owners who are required to buy flood insurance are those who obtain mortgages from federally regulated institutions (Burby, 2000). Therefore, not everyone who lives within a floodplain is required to by flood insurance and they may live there without any protection against the natural threat. Since extreme flooding is not a common occurrence, many live with a false sense of safety, believing that they do not need flood insurance. Individuals have various reasons for not buying flood insurance, including that some homeowners believe that it will never happen to them, if it does happen the government will help them out, they can not afford flood insurance, or it will be a poor investment (Blanchard-Boehm et al., 2001). Whatever the reason, many property owners are living with a very serious risk.

In Cedar Rapids alone, "less than 20 percent of the 35,000 homeowners evacuated due to [last summer's] record floods are believed to have insurance, even though all but a handful lived in a federally mandated flood zone" ("Most Flood Victims Don't Have Flood Insurance", 2008). Also, according to "Flood Statistics" on the Corridor Recovery website, which was created to respond to the floods and jumpstart recovery, "666 of 1,834 flood-affected homes in the 100-year floodplain have flood insurance for a total of $107,148,100 in coverage." This accounts for only 36% of the homes that were damaged in the 100-year floodplain. Many people in Cedar Rapids are waiting for federal help from FEMA, but federal disaster relief will only cover the costs of clean-up and temporary housing; not the replacement costs for lost property ("Most Flood Victims Don't Have Flood Insurance", 2008).

After an event like the flooding in Cedar Rapids, individuals are more likely to buy flood insurance now that they have experienced such a traumatic event. In the case of Grand Forks, North Dakota, more than ninety percent of the population was evacuated in April 1997 when the Red River crested at more than 20 feet above flood stage (Pynn & Ljung, 1999). Surveys were conducted after the flood to research why citizens did not buy flood insurance after numerous warnings from the Federal Emergency Management Agency (FEMA). Why would residents continue to live in a high-risk area without insurance? It was found that:

> Even with periodic, severe flooding of the Red River, the flood of 1997 caused property damage which people had never previously experienced. As a result, the

> community was clearly unprepared for property damage on such a wide scale… Those who purchased flood insurance did so based on what they saw happening, record snowfalls, a rising river, and an increased flood potential because of their proximity to the river. (Pynn & Ljung, 1999, p. 179)

Homeowners seem to not understand the threat they face by living in floodplains until they actually experience a record-breaking flood. Once they become aware of the risk, then they seem willing to buy flood insurance and protect themselves from future loss. Browne and Hoyt (2000) found in a recent study "that the number of flood insurance policies sold during the current period is positively correlated with flood losses during the prior period" (p. 303-4), meaning that as the amount of flood loss in the prior period increases, so does the number of policies sold in the current period. Hopefully this is true and the citizens of Grand Forks and Cedar Rapids will finally take steps toward obtaining flood insurance. It would be even more favorable if those outside Grand Forks and Cedar Rapids learn by example, instead of experience, and buy flood insurance so that they are protected against loss before it is too late.

Flood Frequency Analysis

If flood insurance is to be used as a reliable tool to protect against losses from flooding, flood frequency must be calculated for areas that are being developed so that property owners know the risk they are taking on and can obtain the proper flood insurance. Burnett and Watson's (1993) Hydrology: An Environmental Approach and Bedient and Huber's (2001) Hydrology and Floodplain Analysis give excellent reviews of the basic concepts of flood analysis. They emphasize prediction of flood frequency, flow duration, recurrence intervals, and fundamental distributions used to fit flood data. These statistical methods will later be applied to historical data collected by U.S. Geological Survey gauges located on various rivers throughout Iowa.

Recurrence intervals use fundamental probability concepts to predict the time until the next occurrence. Bendient and Huber (2001) define recurrence interval as the "time interval in which an event will occur once on the average" (p. 746). This information is the basis for predicting 100-year and 500-year floods, which are used to create topographical maps that show the boundaries of how far historic floods could reach. These maps are used to establish flood-insurance premiums and play a key role in urban planning and risk management (Burnett, 1993, p. 481).

The name 100-year flood can be misleading, causing many to think that floods of this magnitude will only occur once every one hundred years, which is false. A 100-year flood is actually the "highest river discharge that may be expected to occur once every one hundred years" (Burnett, 1993, p. 481), but it doesn't mean that this type of flood cannot occur more than once within one hundred years. Instead, each year has the same probability of a 100-year flood occurring. The recurrence intervals based on historical data for various levels of river discharge can then be plotted on a graph and used to predict recurrence intervals that are too large for historical data to account for by choosing a line or curve that best fits the data.

Prediction of flow duration is also a very important concept and is used to create flow-duration curves. These plots show "what percentage of the time the flow of a stream is likely to be greater than some value of interest" (Burnett, 1993, p. 485). This will be helpful in gaining a better perspective on the flood in Cedar Rapids by being able to compare the flow and its corresponding probability the Cedar River normally has to the flow that occurred during the flood last summer. If flow duration from before the flood is less than flow duration after the flood, then this will show that floods of higher magnitudes are occurring more frequently.

Researchers have attempted to find mathematical models that accurately fit the frequency distribution of the annual peak flows of rivers. These models are important because they can help predict when certain flood events will happen again or when extreme events will be exceeded. The most commonly used distributions are Poisson, Gamma, Gumbel, Exponential, Binomial, Bernoulli, Pareto (Önöz & Bayazit, 2001), Normal, Log-Normal, Log-Pearson Type III ("OSU Streamflow"). While some of these distributions can easily be applied to a historical river dataset for a specific site, others involve very complex math that take into account far more than the common variables of peak stage value and peak flow discharge. It is not the intention of this thesis to expand on any pre-existing models or create new models, but instead to apply more basic models to data that is provided for Iowa rivers.

Methods and Results

To begin the process of flood analysis, data was collected from the USGS Iowa Water Science Center website, which provides historic real-time data for streamflow from 156 gauging sites across Iowa. Table 1 shows the cities and towns that were selected to be analyzed and the rivers that are gauged there. Since the flooding last summer was statewide and affected most of Eastern Iowa's main rivers, sites were chosen so that a variety of rivers in different locations were included. Even though some towns, such as Palo and Davenport, experienced extreme flooding, they were not included because they did not have a gauge in that location or the data did not go back a sufficient number of years to provide for reliable and accurate analysis.

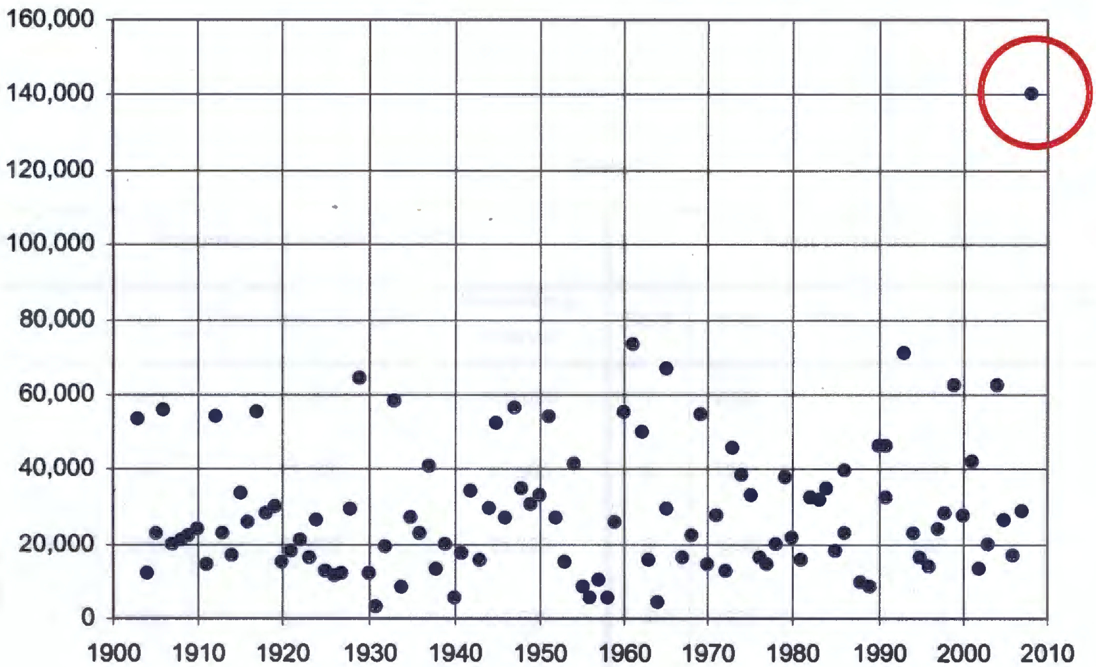
Peak streamflow data was collected for each site that provides the date, gauge height, and other variables for the annual peak discharge, in cubic feet per second (cfs), for each year. The annual peak discharge is the largest amount of water moving through the river at a given gauging site for that year and it is the most popular variable used for mathematical analysis of flood data. Two sets of data were gathered for each site, one set including data from 2008 and the other only containing data up to 2007. This was done in order to compare results from before and after June 2008, to see what kind of impact the flooding had. In Graph 1, the scatter plot shows the annual peak discharge values for Cedar Rapids, going back to the early 1900's. The most extreme data point is circled in red, which represents the annual peak discharge for 2008. This shows how extreme last summer's flooding was compared to floods that have occurred in the past. To see the raw data for Cedar Rapids and the other seven locations used for analysis, please refer to the appendix.

Table 1

Cedar Rapids	Cedar River
Clinton	Mississippi River
Decorah	Upper Iowa River
Des Moines	Des Moines River
Independence	Wapsipinicon River
Iowa City	Iowa River
Oskaloosa	South Skunk River
Waterloo	Cedar River

Graph 1

Peak Annual Discharges for Cedar Rapids (1903-2008)



A. Recurrence Interval

The data for each year, including date, gauge height and annual peak discharge, was imported into an excel spreadsheet where it was organized by rank of the annual peak discharge from largest to smallest values. To find the recurrence interval for a given event, the formula $\frac{n+1}{m}$ is used where n is the number of records or years in the dataset and m is the rank of that event. Table 2 shows the recurrence intervals for the first five records from the 2007 and 2008 datasets for Cedar Rapids.

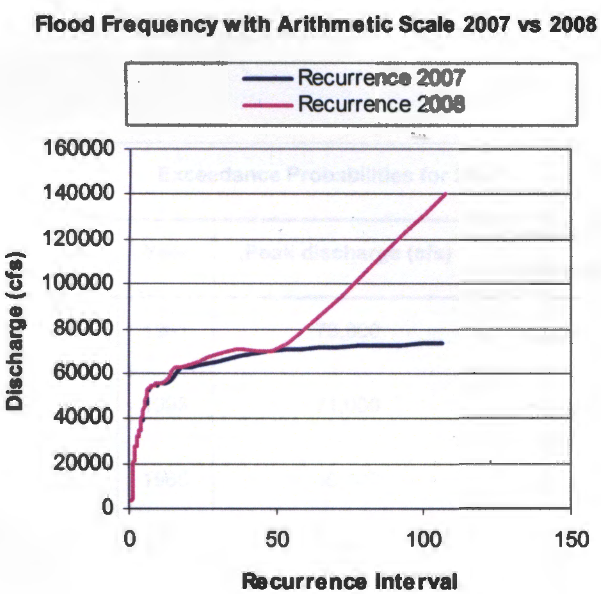
Notice that the recurrence intervals for the peak discharge values all decreased from 2007 to 2008. For example, the recurrence interval for a peak discharge of 73,000 cfs decreased from 106 to 53.5 years. This means that a flood of this magnitude has a higher probability of being equaled or exceeded in any single year, as compared to 2007. Graphs 2 and 3 are plots of all the recurrence intervals for Cedar Rapids. Graph 2 has an arithmetic scale where each interval on the discharge axis are of the same width where as Graph 3 has a logarithmic scale that is not even, but shows a large range of numbers.

Both graphs show that there is a visible rise in discharge for a given recurrence interval, especially for the larger recurrence intervals.

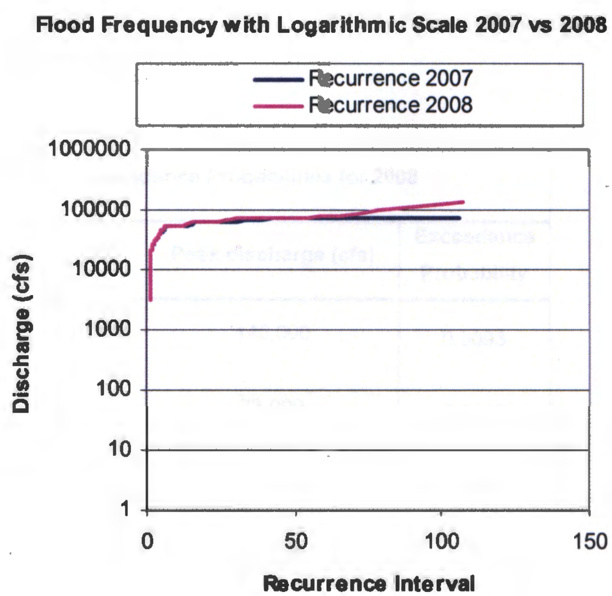
Table 2

Recurrence Intervals for 2007				Recurrence Intervals for 2008			
Rank	Year	Peak discharge (cfs)	Recurrence Interval	Rank	Year	Peak discharge (cfs)	Recurrence Interval
1	1961	73,000	106.000	1	2008	140,000	107.000
2	1993	71,000	53.000	2	1961	73,000	53.500
3	1965	66,800	35.333	3	1993	71,000	35.667
4	1929	64,000	26.500	4	1965	66,800	26.750
5	2004	62,500	21.200	5	1929	64,000	21.400

Graph 2



Graph 3



B. Flow Duration

The next step was to find flow duration for each year in order to produce flow duration curves that show a "plot of magnitude versus percent of time the magnitude is equaled or exceeded" (Bedient & Huber, 2002, p. 740). The equation for flow duration, or exceedance probability, is $\dfrac{1}{\text{Recurrence Interval}}$ or $\dfrac{m}{n+1}$, where n is the number of records and m is the rank of the event. Table 3 shows the exceedance probabilities for the first five records from the 2007 and 2008 datasets for Cedar Rapids.

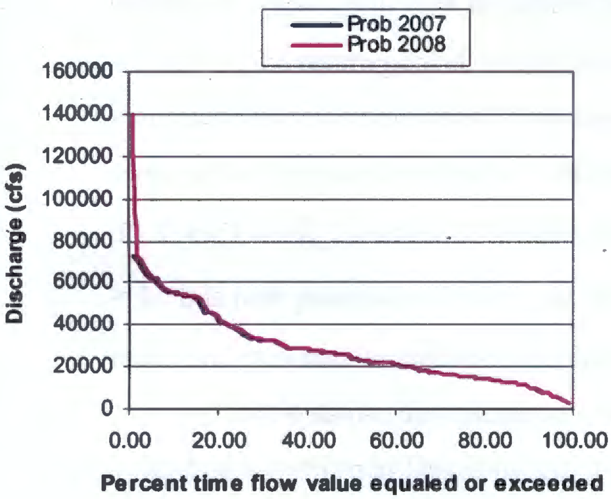
Notice that the exceedance probabilities for the peak discharge values all increased from 2007 to 2008. For example the exceedance probability for a peak discharge of 73,000 cfs increased .0094 to .0187. This means that a flood of this magnitude has a higher probability of being equaled or exceed in any single year, as compared to 2007. Graphs 4 and 5 show flow duration curves, or plots of all the exceedance probabilities, for Cedar Rapids. Both graphs show lines for 2007 and 2008 that are very similar for the majority of the graph. But between 0 to 5% there is a large jump in discharge value that you can clearly see in Graph 4. This is due to the flood last summer that had a peak discharge value of 140,000 cfs, which is almost double what the previous peak discharge value on record was before 2008, which was 73,000 cfs back in 1961.

Table 3

Exceedance Probabilities for 2007				Exceedance Probabilities for 2008			
Rank	Year	Peak discharge (cfs)	Exceedance Probability	Rank	Year	Peak discharge (cfs)	Exceedance Probability
1	1961	73,000	0.0094	1	2008	140,000	0.0093
2	1993	71,000	0.0189	2	1961	73,000	0.0187
3	1965	66,800	0.0283	3	1993	71,000	0.0280
4	1929	64,000	0.0377	4	1965	66,800	0.0374
5	2004	62,500	0.0472	5	1929	64,000	0.0467

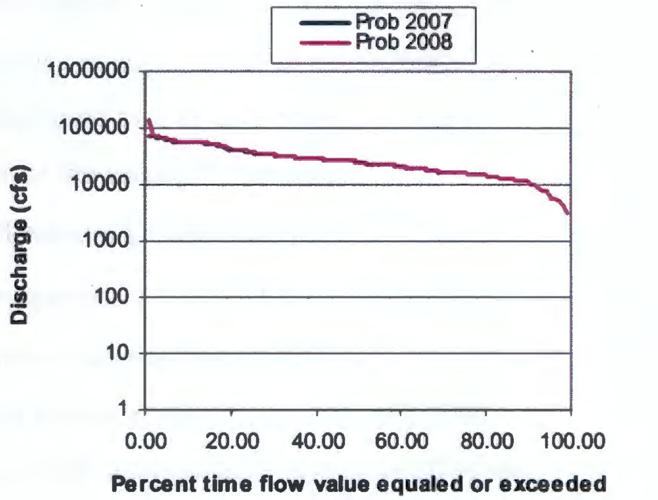
Graph 4

Flow Duration Curve with Arithmetic Scale 2007 vs 2008



Graph 5

Flow Duration Curve with Logarithmic Scale 2007 vs 2008



C. Normal Distribution

The final phase of analysis is trying to fit the data to different probability models. There are many models that can be used to analyze flood data, but only 4 were chosen for this thesis: Normal, Lognormal, Gamma-3, and Log Pearson 3. These models were chosen because they are basic, well-known, user friendly models, and they are commonly used and tested for flood analysis. To see more information about these distributions, please refer to the appendix.

The Normal Distribution is a key mathematical concept that is "fundamental to the entire realm of probability and statistics" (Bedient & Huber, 2001, p. 198). The Normal Distribution has a symmetric bell-shaped curve with parameters mean μ and variance σ^2. Bedient and Huber (2001) explain that "the symmetric nature of the normal distribution usually makes it unsuitable for flood flows" (p.199), but it is still a good starting point for data with a large number of variables.

For each of the eight selected sites, the average and standard deviation of the peak discharge values were found for the datasets up to 2007 and 2008. For all of the models used, the discharge value corresponding to a set of standard return periods (2, 5, 10, 25, 50, 100, 200) needs to be calculated so that 100-year floods and so on can be found according to the models and river data. For Normal Distributions, The standard normal variate formula, also known as the Z-score, is used to standardized datasets with $\mu \neq 0$

and $\sigma^2 \neq 1$ so that they can fit a Normal Distribution. In order to find the corresponding discharges, which will now be labeled as Q_T, the equation $F(Q_T) = 1 - 1/T$ is used, where T is the return period, which is the probability of not equaling or exceeding a magnitude in any single year. This number is then used to look up a corresponding Z-score in the Normal Distribution table, which gives the value of Z for which $P(X < Z) = F(Q_T)$ where X follows a Standard Normal Distribution with $\mu = 0$ and $\sigma^2 = 1$. It is now possible to find Q_T by solving the equation $Q_T = \mu + (Z \times \sigma)$, using the mean and standard deviation of the discharge values that was found earlier.

Table 4 shows the discharge values found for Cedar Rapids for 2007 and 2008 according to a Normal Distribution. It can be seen that the discharge values for all of the given return periods have increased from 2007 to 2008. This agrees with the findings earlier that showed the recurrence intervals for the peak discharge values decreased from 2007 to 2008, because larger discharges are expected on average to recur in shorter spans of time.

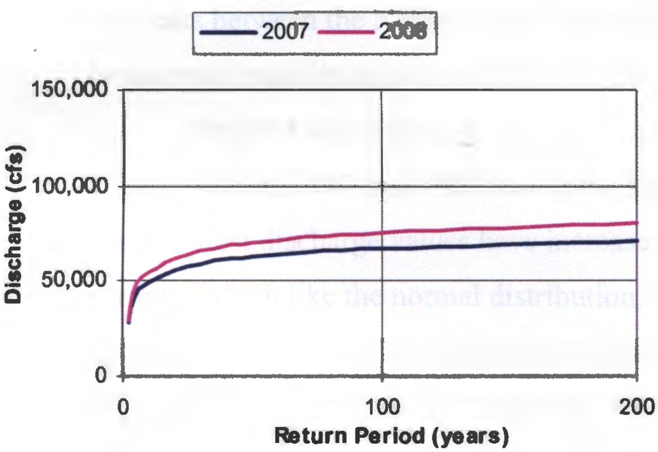
Graphs 6 and 7 show plots of the return periods and their corresponding discharge values, for both 2007 and 2008, using the Normal Distribution from Table 4. In both graphs, it is clear that the discharge values have increased from 2007 to 2008. Graph 7 uses logarithmic scales on both axes and therefore shows a greater ranger of return periods and discharge values. One way to judge the fit of the data to a Normal Distribution is to see if the data plots a straight line when using logarithmic scales. Both lines for 2007 and 2008 are fairly straight, except for the left-tail which tends to curve. This is common because peak discharge values that are expected to return on average every 1 to 10 years are river stages that are not extreme. The model is satisfactory as long as the line is straight for the higher return periods since it is extreme flooding with high discharge values that cause more concern.

Table 4

Normal Distribution for Cedar Rapids 1903-2007				Normal Distribution for Cedar Rapids 1903-2008			
Return Period (T)	$F(Q_T)$	Z-score (Z)	Discharge (Q_T)	Return Period (T)	$F(Q_T)$	Z-score (Z)	Discharge (Q_T)
2	0.5000	0.0000	28,263	2	0.5000	0.0000	29,317
5	0.8000	0.8420	42,280	5	0.8000	0.8420	45,993
10	0.9000	1.2820	49,604	10	0.9000	1.2820	54,708
25	0.9600	1.7510	57,411	25	0.9600	1.7510	63,996
50	0.9800	2.0540	62,455	50	0.9800	2.0540	69,997
100	0.9900	2.3260	66,983	100	0.9900	2.3260	75,384
200	0.9950	2.5760	71,145	200	0.9950	2.5760	80,336

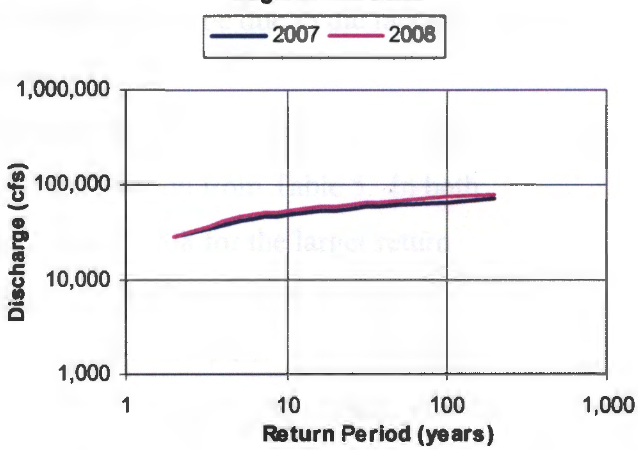
Graph 6

Comparison of 2007 vs 2008 Flood Frequency Analysis for Cedar River at Cedar Rapids using Normal Analysis with Arithmetic Scale



Graph 7

Comparison of 2007 vs 2008 Flood Frequency Analysis for Cedar River at Cedar Rapids using Normal Analysis with Logarithmic Scale



D. Lognormal Distribution

Normal Distribution turned out to be a reasonable fit for the Cedar Rapids data, but it is not logical to assume that river data will be perfectly symmetrical without any skewness. Therefore, the Lognormal Distribution can be used to easily fit river data since it so closely related to the Normal Distribution and also has a positive skewness. Positive skewness implies that a graph of the distribution will have a long right-hand tail, meaning that it is skewed to the right. It is said that "a random variable has a Lognormal Distribution if the log of the random variable is distributed normally" (Bedient & Huber, 2001, p. 201).

For each of the eight sites, the average and standard deviation of the log of peak discharge values were found for the datasets up to 2007 and 2008. For Lognormal Distributions, the Z-score may also be used to fit a Normal Distribution. In order to find the corresponding discharges, solve $Q_T = 10\wedge(\mu + (Z \times \sigma))$, using the mean and standard deviation of the log of discharge values that was found earlier.

Table 5 shows the discharge values found for Cedar Rapids for 2007 and 2008 according to a Lognormal Distribution. It can be seen that, like the Normal Distribution, the discharge values for the larger return periods have increased. The discharge values that decreased are still very close to their previous value from 2007. The slight difference in results between the Normal and Lognormal Distributions may be due to the fact that Lognormal includes skewness while Normal does not.

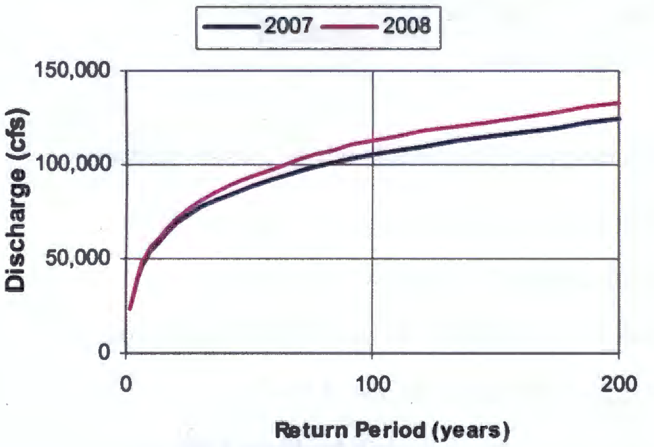
Graphs 8 and 9 show plots of the return periods and their corresponding discharge values, for both 2007 and 2008, using the lognormal distribution from Table 5. In both graphs, most discharge values have increased from 2007 to 2008 for the larger return periods. Much like the normal distribution, both lines for 2007 and 2008 are fairly straight, except for the left-tail which tends to curve a little in Graph 9. Also, there is a large jump in discharge values from the normal distribution to its corresponding value for the lognormal distribution. In Table 6, it shows that the discharge value for the 200-year return period jumped from 124,289 cfs to 133,036. The lognormal estimate is higher because "the lognormal distribution more accurately reflects the skewness of the actual flow data" (Bedient & Huber, 2001, p. 202).

Table 5

Lognormal Distribution for Cedar Rapids 1903-2007				Lognormal Distribution for Cedar Rapids 1903-2008			
Return Period (T)	$F(Q_T)$	Z-score (Z)	Discharge (Q_T)	Return Period (T)	$F(Q_T)$	Z-score (Z)	Discharge (Q_T)
2	0.5000	0.0000	23,472	2	0.5000	0.0000	23,871
5	0.8000	0.8420	40,473	5	0.8000	0.8420	41,855
10	0.9000	1.2820	53,803	10	0.9000	1.2820	56,128
25	0.9600	1.7510	72,879	25	0.9600	1.7510	76,740
50	0.9800	2.0540	88,664	50	0.9800	2.0540	93,925
100	0.9900	2.3260	105,726	100	0.9900	2.3260	112,606
200	0.9950	2.5760	124,289	200	0.9950	2.5760	133,036

Graph 8

Comparison of 2007 vs 2008 Flood Frequency Analysis for Cedar River at Cedar Rapids using Lognormal Analysis with Arithmetic Scales



Graph 9

Comparison of 2007 vs 2008 Flood Frequency Analysis for Cedar River at Cedar Rapids using Lognormal Analysis with Logarithmic Scales

Table 6

Lognormal Distribution for Cedar Rapids 1903-2007				Lognormal Distribution for Cedar Rapids 1903-2008			
Return Period (T)	$F(Q_T)$	Z-score (Z)	Discharge (Q_T)	Return Period (T)	$F(Q_T)$	Z-score (Z)	Discharge (Q_T)
200	0.9950	2.5760	124,289	200	0.9950	2.5760	133,036

E. Gamma-3 Distribution

The Gamma-3 Distribution is a Gamma Distribution with three parameters, $\gamma, \alpha,$ and β. Like the Lognormal Distribution, Gamma-3 can have a positive skewness and also negative skewness, which makes it useful for hydrological purposes (Bedient & Huber, 2001, p. 203). Unlike Normal and Lognormal, the Gamma-3 distribution does not use a Z-score. Instead it uses frequency factor K. The Frequency Factors K Table (Bedient & Huber, 2001, p. 204) lists K values for corresponding skew coefficients and recurrence intervals. The skew coefficient of the peak discharge values were found for the datasets up to 2007 and 2008. Since the table lists skew coefficients to the first decimal, the true K value needs to be interpolated by using the follow equation:

$$K(skew_{Q_T}) = \left(\frac{K(low) - K(high)}{low - high} \right) * \left(skew_{Q_T} - low \right) + K(low)$$

where $skew_{Q_T}$ is the skew coefficient for discharge values, high and low are the skew coefficient rounded up and down to the nearest one-decimal number, and finally K(high) and K(low) are the frequency factors corresponding to the low and high skew coefficients and the desired recurrence interval. To find the discharge values, use the equation $Q_T = \mu + (K \times \sigma)$, which is the same equation as for a normal distribution that uses the mean and standard deviation of the discharge values, but with K instead of Z.

The discharge values found for Cedar Rapids for 2007 and 2008 according to a Gamma-3 Distribution can be found in Table 7. Like the Lognormal Distribution, the discharge values for the larger return periods have increased. But notice that there is a larger increase in the Gamma-3 results between 2007 and 2008, but the estimates for discharge values are greater overall for the Lognormal results. The discharge values that decreased are also still very close to their previous value from 2007.

Graphs 10 and 11 show plots of the return periods and their corresponding discharge values, for both 2007 and 2008, using the Gamma-3 Distribution from Table 7. In Graph 10, the discharge values increase significantly from 2007 to 2008 for the larger return periods. Graph 11 also shows an increase from 2007 to 2008, but it isn't as dramatic as Graph 10 since it uses a logarithmic scale. Much like the Normal

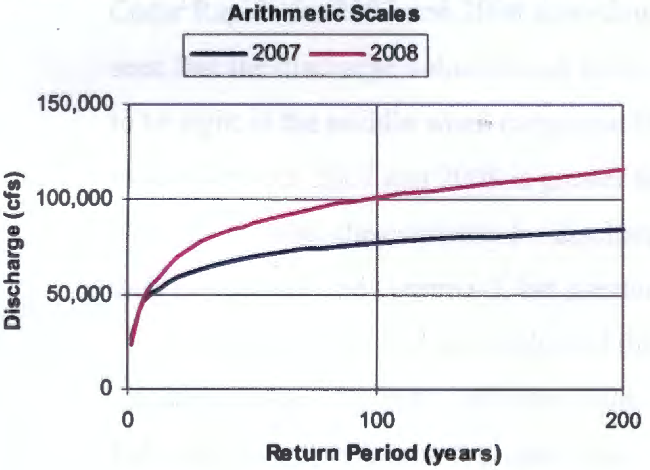
Distribution, both lines for 2007 and 2008 are fairly straight, except for the left-tail which tends to curve a little in Graph 10.

Table 7

Gamma-3 Distribution for Cedar Rapids 1903-2007				Gamma-3 Distribution for Cedar Rapids 1903-2008			
Return Period (T)	$F(Q_T)$	K(0.8566)	Discharge (Q_T)	Return Period (T)	$F(Q_T)$	K(2.0839)	Discharge (Q_T)
2	0.5000	-0.1411	25,915	2	0.5000	-0.3171	23,038
5	0.8000	0.7738	41,144	5	0.8000	0.5947	41,096
10	0.9000	1.3377	50,531	10	0.9000	1.2953	54,971
25	0.9600	2.0072	61,675	25	0.9600	2.2282	73,448
50	0.9800	2.4785	69,521	50	0.9800	2.9372	87,489
100	0.9900	2.9284	77,010	100	0.9900	3.6478	101,563
200	0.9950	3.3624	84,235	200	0.9950	4.3601	115,670

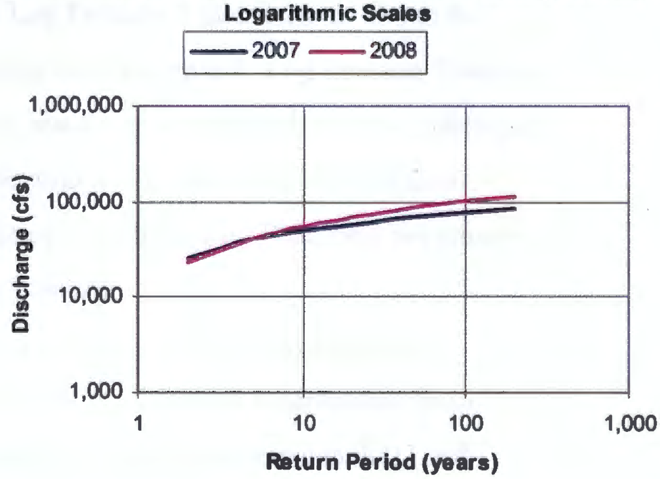
Graph 10

Comparison of 2007 vs 2008 Flood Frequency Analysis for Cedar River at Cedar Rapids using Gamma-3 Analysis with Arithmetic Scales



Graph 11

Comparison of 2007 vs 2008 Flood Frequency Analysis for Cedar River at Cedar Rapids using Gamma-3 Analysis with Logarithmic Scales



F. Log Pearson 3 Distribution

Log Pearson 3 is the final distribution applied. This is the recommended distribution used by the U.S. Interagency Advisory Committee on Water Data (as cited in Bedient & Huber, 2002, p. 206). Like the relationship between Normal and Lognormal Distributions, and random variable is said to have a Log Pearson 3 Distribution if the log of the random variable is follows a Gamma-3 Distribution. In other words, random variable X follows a Log Pearson 3 Distribution if $\log X \sim Gamma(\gamma, \alpha, \beta)$.

Like Lognormal, the mean and standard deviation of the logs of discharge values must be found in order to use this distribution. Frequency factor K will also be used in the same manner as it was in Gamma-3, but uses a weighted skew coefficient instead. The weight skew coefficient, C_w, is found using the equation $C_w = WC_s + (1-W)C_m$ where C_s is the skewness of the sample data, Cm is regional skewness that is determined from a map, and W is a weighting factor. By taking into account data skewness and regional skewness, results from this distribution are likely to be more reliable than the previous distributions.

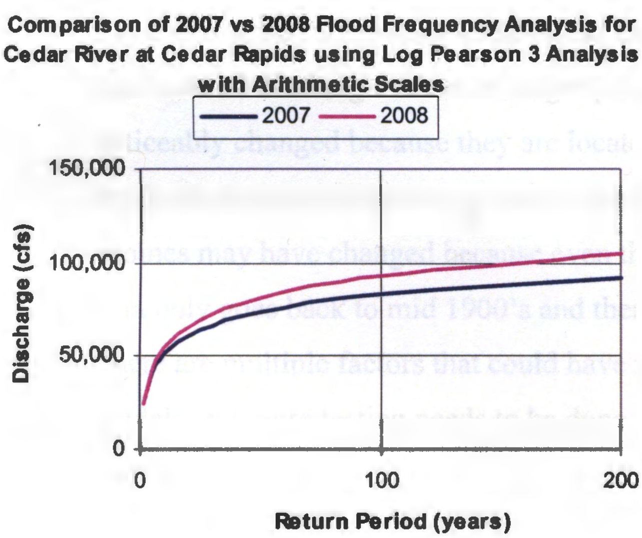
Once the K value is found, the corresponding discharges are found by solving the equation $Q_T = 10^\wedge(\mu + (K \times \sigma))$, using the mean and standard deviation of the log of discharge values that was found earlier. Table 8 shows the discharge values found for Cedar Rapids for 2007 and 2008 according to a Log Pearson 3 distribution. It can be seen that the discharge values for all return periods have increased. Log Pearson 3 seems to be right in the middle when compared to other results, since the difference in discharge values between 2007 and 2008 is greater than Normal and Lognormal, but less than Gamma-3. Also, the estimates for discharge values overall for Log Pearson 3 are smaller than Lognormal and Gamma-3, but greater than Normal.

Graphs 12 and 13 show plots of the return periods and their corresponding discharge values, for both 2007 and 2008, using the Log Pearson 3 Distribution from Table 8. Like the Gamma-3 graphs, there is noticeable difference between 2007 and 2008 in the arithmetic graph while there is a minor difference in the logarithmic graph. This makes sense since the arithmetic graph only looks at return periods up to 200 years and discharges up to 150,000 cfs because those are reasonable spans for the flooding experiences that have been recorded in Cedar Rapids so far. There is not enough data to know how high the discharge value for the Cedar River at Cedar Rapids can truly get.

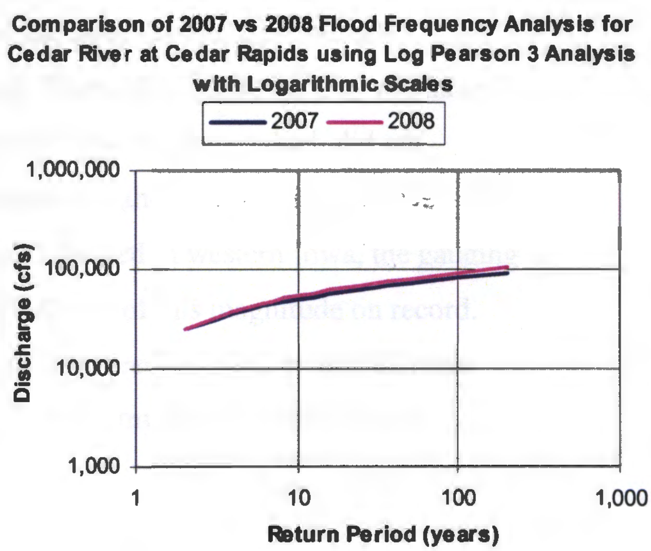
Table 8

Log Pearson 3 Distribution for Cedar Rapids 1903-2007				Log Pearson 3 Distribution for Cedar Rapids 1903-2008			
Return Period (T)	$F(Q_T)$	K(-0.5028)	Discharge (Q_T)	Return Period (T)	$F(Q_T)$	K(-0.3753)	Discharge (Q_T)
2	0.5000	0.0835	24,774	2	0.5000	0.0621	24,879
5	0.8000	0.8560	40,842	5	0.8000	0.8545	42,205
10	0.9000	1.2155	51,538	10	0.9000	1.2345	54,377
25	0.9600	1.5659	64,652	25	0.9600	1.6151	70,092
50	0.9800	1.7754	74,037	50	0.9800	1.8478	81,859
100	0.9900	1.9529	83,047	100	0.9900	2.0475	93,519
200	0.9950	2.1054	91,661	200	0.9950	2.2240	105,197

Graph 12



Graph 13



G. Cedar Rapids Compared to the Other Seven Sites

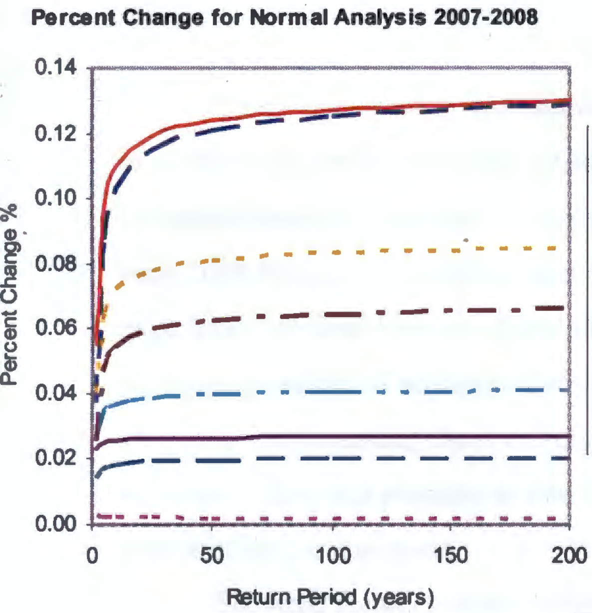
Cedar Rapids was the main focus of research, but it was not the only Iowa town affected by the flood. The seven other sites were analyzed the same as previously demonstrated for Cedar Rapids. But as you can seen from Graphs 14 through 17, out of all eight site, Cedar Rapids had some of the highest percents of change for all four models. Other towns that had high percent changes are Clinton and Decorah. Percent change was calculated by using the equation:

$$\text{Percent Change} = \frac{Q_T(2008) - Q_T(2007)}{Q_T(2007))}.$$

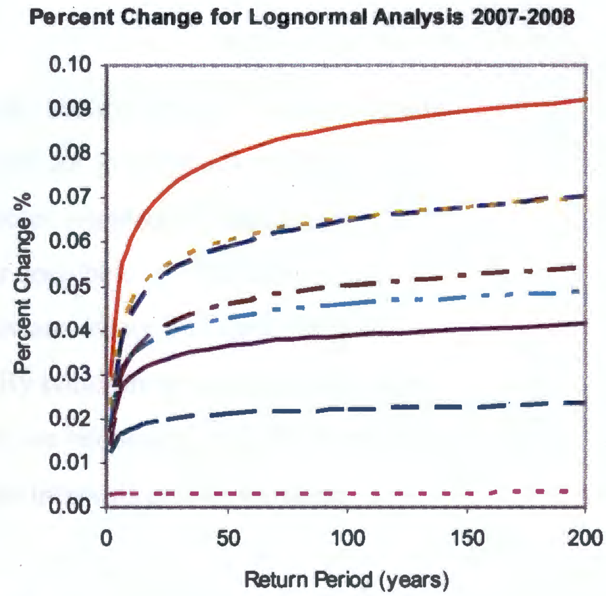
This basically takes the old discharge value for a given return period, subtracts it from the new discharge value, and then divides by the old discharge value so that the change from 2007 to 2008 is given in terms of data observed in 2007.

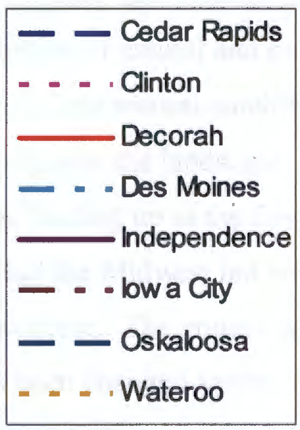
The percent change from 2007 to 2008 may be affected by where the gauging site is located in relation to where the heavy flooding occurred and how far back the data goes for that gauging site. Sites such as Clinton may not have changed much from 2007 to 2008 because the gauging data for that site goes back to the late 1800's. The distributions would not be as greatly affected by 2008 because older data may contain flood events that are similar to 2008 and therefore the 2007 models have already encountered discharge values of larger proportions. Sites like Oskaloosa may not have noticeably changed because they are located more in Western Iowa, which did not received as much rain and was not as heavily impacted by the floods. Sites like Des Moines may have changed because even though it is located in western Iowa, the gauging data only goes back to mid 1900's and there are no floods of this magnitude on record. There are multiple factors that could have influenced the results found by the different models and more testing needs to be done in order to minimize the effects of those factors.

Graph 14

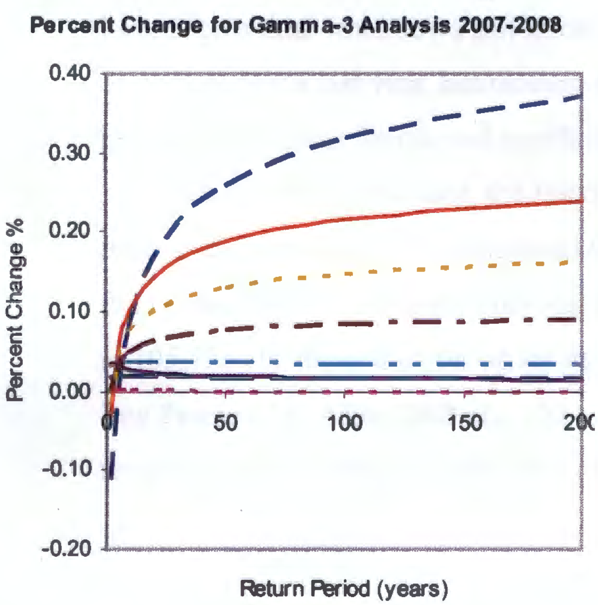


Graph 15

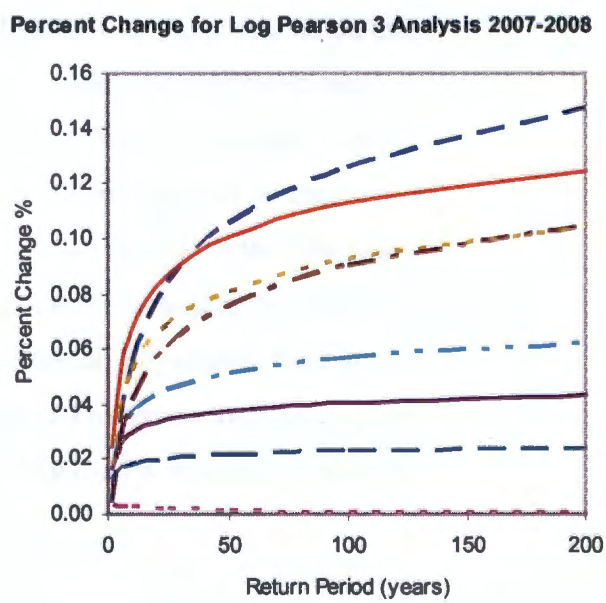




Graph 16



Graph 17



Conclusion

Overall, most sites were similar to Cedar Rapids: recurrence intervals decreased, exceedance probabilities increased, and discharge values for given return periods increased from 2007 to 2008. All of this is to be expected because the magnitude of the June 2008 flood was something that Iowans had never seen before. The state of Iowa experience a record-breaking flood that now makes Iowans aware that they are not completely certain of how high the rivers can reach. By continuing to collect data year after year and recording these extreme floods, datasets are becoming larger and more accurate so that it is possible to find the true recurrence intervals and exceedance probabilities for our rivers.

So, what factors caused the flooding last summer and why was it so extreme? Like most flooding, it was a combination of natural and man made causes that led to last summer's extreme statewide flooding. The natural conditions were conducive to flooding and man invited it with changes to the landscape. Iowa received record precipitation in the winter and spring leading up to the flood and also during the month of June when the flooding occurred. Had the Midwest not received so much precipitation, the flooding may not have been as extreme. The ground was not able to absorb the excess water because Iowa land has been changed so that water is quickly removed from fields and pavement and channeled directly into our local streams and rivers. There may have been a time when Iowa soil in its natural state could deal with the precipitation that Iowa experience last year, but modern developments have made it very difficult and perhaps impossible for our soil to effectively manage large amounts of rain and snow.

According to the data, the flood that Cedar Rapids experienced last summer has a recurrence interval of 107 years, even though the flood reached outside of the 500-year floodplain. Before 2008, the 100-year flood had a discharge value ranging from 66,983 to 105,726 cfs, depending on which model you look at (Normal, Lognormal, Gamma-3, Log Pearson 3). After 2008, the 100-year flood has increased to have a discharge value ranging from 75,384 to 112,606 cfs. More extreme flooding can be expected to happen again. The people of Cedar Rapids never imagined that a flood of this scale could happen, but they better be prepared now.

The state of Iowa has seen the devastation that flooding can cause. Iowans saw it in 1993 and many believed that everyone would be safe for awhile since it was a 500-year flood. But 500-year floods do not just happen once in every 500 years, they can happen during any given year. That is why it is important for property owners to have flood insurance if they live in a floodplain. Extreme flooding can happen at any time; it is always a risk, but that seems to be easily forgotten. The lesson to prepare for and protect against flooding was not learned after 1993, but hopefully it will be now after the 2008 floods. If Iowa soil continues to be modified and citizens choose to live in floodplains, they must accept the risk and take every action to avoid major losses due to flooding.

Appendix

Annual Peak Discharge (in cfs), Cedar Rapids
Source: *USGS Real-Time Data for Iowa Streamflow*

Year	Annual Peak Discharge	Year	Annual Peak Discharge	Year	Annual Peak Discharge	Year	Annual Peak Discharge
1903	53,600.00	1930	12,200.00	1957	9,900.00	1984	35,000.00
1904	11,800.00	1931	3,270.00	1958	5,240.00	1985	18,000.00
1905	23,000.00	1932	19,100.00	1959	25,800.00	1986	39,600.00
1906	55,700.00	1933	58,400.00	1960	55,100.00	1986	22,700.00
1907	19,800.00	1934	8,620.00	1961	73,000.00	1988	9,760.00
1908	21,000.00	1935	26,900.00	1962	50,000.00	1989	8,130.00
1909	22,100.00	1936	22,700.00	1963	15,600.00	1990	46,300.00
1910	24,100.00	1937	40,700.00	1964	4,270.00	1991	46,100.00
1911	14,300.00	1938	12,900.00	1965	66,800.00	1991	32,100.00
1912	54,000.00	1939	19,700.00	1965	29,200.00	1993	71,000.00
1913	22,500.00	1940	5,540.00	1967	16,000.00	1994	22,500.00
1914	17,000.00	1941	17,100.00	1968	22,200.00	1995	16,400.00
1915	33,600.00	1942	33,900.00	1969	54,500.00	1996	13,700.00
1916	25,700.00	1943	15,800.00	1970	14,300.00	1997	24,100.00
1917	54,900.00	1944	29,100.00	1971	27,400.00	1998	28,400.00
1918	28,200.00	1945	52,300.00	1972	12,400.00	1999	62,300.00
1919	29,700.00	1946	27,100.00	1973	45,500.00	2000	27,700.00
1920	14,800.00	1947	56,200.00	1974	38,200.00	2001	42,000.00
1921	17,900.00	1948	34,500.00	1975	32,800.00	2002	13,000.00
1922	21,000.00	1949	30,800.00	1976	16,100.00	2003	19,700.00
1923	16,000.00	1950	33,000.00	1977	14,600.00	2004	62,500.00
1924	26,300.00	1951	54,100.00	1978	19,700.00	2005	26,200.00
1925	12,800.00	1952	27,000.00	1979	37,600.00	2006	17,000.00
1926	11,500.00	1953	15,200.00	1980	21,800.00	2007	28,800.00
1927	11,800.00	1954	41,400.00	1981	15,400.00	2008	140,000.00

1928	29,200.00	1955	8,100.00	1982	32,200.00		
1929	64,000.00	1956	5,400.00	1983	31,700.00		

Annual Peak Discharge (in cfs), Clinton
Source: *USGS Real-Time Data for Iowa Streamflow*

Year	Annual Peak Discharge	Year	Annual Peak Discharge	Year	Annual Peak Discharge	Year	Annual Peak Discharge
1874	98,400	1908	134,000	1942	169,600	1976	154,000
1875	167,000	1909	123,000	1943	158,700	1977	50,800
1876	165,000	1910	73,100	1944	168,500	1978	128,000
1877	104,000	1911	84,000	1945	164,400	1979	153,000
1878	80,800	1912	120,000	1946	144,800	1980	108,000
1879	96,800	1913	123,000	1947	125,500	1981	113,000
1880	250,000	1914	111,000	1948	108,300	1982	163,000
1881	174,000	1915	92,000	1949	85,300	1983	179,000
1882	237,000	1916	195,000	1950	129,900	1984	134,000
1883	174,000	1917	142,000	1951	221,500	1985	139,000
1884	144,000	1918	123,000	1952	225,400	1986	195,000
1885	146,000	1919	166,000	1953	104,100	1987	201,000
1886	157,000	1920	222,000	1954	175,900	1988	65,800
1887	149,000	1921	85,300	1955	96,900	1989	104,000
1888	248,000	1922	212,000	1956	127,000	1990	123,000
1889	64,300	1923	106,000	1957	103,000	1991	137,000
1890	142,000	1924	106,000	1958	64,500	1992	147,000
1891	130,000	1925	93,900	1959	112,000	1993	239,000
1892	238,000	1926	83,600	1960	151,000	1994	148,000
1893	174,000	1927	133,000	1961	143,000	1995	121,000
1894	157,000	1928	116,000	1962	138,000	1996	187,000
1895	70,000	1929	146,000	1963	90,900	1997	237,000
1896	148,000	1930	83,600	1964	84,000	1998	178,000
1897	198,000	1931	40,700	1965	307,000	1999	146,000
1898	88,800	1932	97,500	1966	143,000	2000	126,000
1899	149,000	1933	92,100	1967	207,000	2001	270,000
1900	100,000	1934	81,400	1968	131,000	2002	168,000
1901	142,000	1935	123,000	1969	231,000	2003	162,000
1902	123,000	1936	133,000	1970	93,000	2004	175,000
1903	176,000	1937	95,800	1971	168,000	2005	123,000
1904	174,000	1938	167,400	1972	153,000	2006	129,000
1905	172,000	1939	144,900	1973	207,000	2007	139,000
1906	169,000	1940	74,100	1974	158,000	2008	198,000
1907	171,000	1941	128,200	1975	214,000		

Annual Peak Discharge (in cfs), Decorah
Source: *USGS Real-Time Data for Iowa Streamflow*

Year	Annual Peak Discharge	Year	Annual Peak Discharge	Year	Annual Peak Discharge	Year	Annual Peak Discharge
1952	8,850	1967	11,800	1982	3,290	1997	4,900
1953	11,100	1968	3,950	1983	5,730	1998	11,000
1954	11,600	1969	14,900	1984	5,020	1999	6,700
1955	1,700	1970	2,460	1985	5,210	2000	15,300
1956	5,500	1971	8,440	1986	6,430	2001	9,730
1957	8,000	1972	8,340	1987	375	2002	1,930
1958	1,200	1973	6,290	1988	430	2003	1,920
1959	5,400	1974	7,120	1989	2,240	2004	8,180
1960	5,880	1975	6,090	1990	7,200	2005	7,090

1961	20,200	1976	9,500	1991	3,700	2006	3,500
1962	14,800	1977	2,010	1992	3,200	2007	8,610
1963	3,600	1978	7,060	1993	20,500	2008	27,900
1964	3,870	1979	5,260	1994	3,500		
1965	10,100	1980	6,120	1995	2,340		
1966	9,330	1981	3,190	1996	5,180		

Annual Peak Discharge (in cfs), Des Moines
Source: *USGS Real-Time Data for Iowa Streamflow*

Year	Annual Peak Discharge	Year	Annual Peak Discharge	Year	Annual Peak Discharge	Year	Annual Peak Discharge
1902	32,900	1928	5,700	1944	34,000	1960	36,200
1903	42,000	1929	23,300	1945	18,700	1961	26,700
1906	13,700	1930	6,140	1946	16,900	1997	13,700
1915	29,000	1931	2,290	1947	39,500	1998	16,000
1916	17,200	1932	28,500	1948	20,300	1999	15,700
1917	26,800	1933	30,600	1949	15,300	2000	10,000
1918	27,300	1934	5,370	1950	10,600	2001	18,500
1919	16,900	1935	17,600	1951	32,400	2002	8,940
1920	18,700	1936	13,600	1952	17,800	2003	16,000
1921	10,700	1937	15,800	1953	11,400	2004	17,400
1922	12,000	1938	24,800	1954	60,200	2005	16,800
1923	8,000	1939	13,400	1955	6,840	2006	14,500
1924	7,570	1940	6,810	1956	1,420	2007	17,700
1925	4,920	1941	8,100	1957	9,680	2008	46,700
1926	12,000	1942	9,330	1958	8,480		
1927	12,800	1943	9,500	1959	16,100		

Annual Peak Discharge (in cfs), Independence
Source: *USGS Real-Time Data for Iowa Streamflow*

Year	Annual Peak Discharge	Year	Annual Peak Discharge	Year	Annual Peak Discharge	Year	Annual Peak Discharge
1934	1,010	1953	6,350	1972	8,200	1991	9,110
1935	7,220	1954	2,780	1973	9,760	1992	4,920
1936	6,900	1955	1,560	1974	6,020	1993	13,400
1937	7,900	1956	2,100	1975	7,580	1994	3,440
1938	3,990	1957	3,270	1976	7,600	1995	2,970
1939	2,790	1958	1,210	1977	1,100	1996	3,610
1940	2,540	1959	11,800	1978	5,630	1997	4,100
1941	5,250	1960	15,200	1979	16,300	1998	8,990
1942	9,060	1961	15,700	1980	7,380	1999	31,100
1943	4,090	1962	14,400	1981	8,020	2000	8,450
1944	13,800	1963	3,450	1982	8,260	2001	6,610
1945	10,900	1964	1,830	1983	7,650	2002	7,150
1946	9,910	1965	8,880	1984	5,490	2003	5,060
1947	21,500	1966	6,250	1985	3,090	2004	22,600
1948	11,100	1967	3,340	1986	11,300	2005	3,530
1949	7,420	1968	26,800	1987	3,300	2006	4,010
1950	10,300	1969	14,900	1988	1,820	2007	10,600
1951	20,800	1970	4,910	1989	2,390	2008	22,700
1952	6,460	1971	6,580	1990	24,400		

Annual Peak Discharge (in cfs), Iowa City
Source: *USGS Real-Time Data for Iowa Streamflow*

Year	Annual Peak Discharge	Year	Annual Peak Discharge	Year	Annual Peak Discharge	Year	Annual Peak Discharge
1903	22,500	1930	13,600	1957	4,700	1984	9,430
1904	9,880	1931	2,790	1958	5,000	1985	10,500
1905	9,740	1932	7,880	1959	9,380	1986	10,100
1906	11,900	1933	8,700	1960	10,700	1987	6,100
1907	13,000	1934	2,140	1961	9,940	1988	4,450
1908	6,000	1935	8,710	1962	12,300	1989	3,240
1909	12,500	1936	12,900	1963	8,220	1990	13,500
1910	12,600	1937	17,100	1964	3,000	1991	13,700
1911	17,200	1938	5,000	1965	11,100	1992	8,080
1912	20,000	1939	9,870	1966	10,100	1993	28,200
1913	7,030	1940	3,020	1967	4,420	1994	9,440
1914	9,620	1941	7,490	1968	3,310	1995	9,530
1915	20,000	1942	7,800	1969	15,000	1996	6,900
1916	16,100	1943	10,600	1970	8,540	1997	10,500
1917	17,500	1944	31,100	1971	11,100	1998	9,960
1918	42,500	1945	9,560	1972	10,600	1999	7,760
1919	13,800	1946	14,800	1973	11,300	2000	6,890
1920	8,300	1947	33,800	1974	11,900	2001	11,100
1921	15,600	1948	18,000	1975	10,500	2002	4,650
1922	5,880	1949	14,100	1976	7,350	2003	6,310
1923	8,590	1950	13,700	1977	5,240	2004	7,550
1924	19,900	1951	15,700	1978	9,510	2005	6,310
1925	1,590	1952	10,400	1979	10,700	2006	5,030
1926	17,800	1953	11,900	1980	5,650	2007	10,100
1927	10,900	1954	7,690	1981	6,230	2008	40,900
1928	10,000	1955	4,600	1982	10,400		
1929	21,900	1956	3,100	1983	9,490		

Annual Peak Discharge (in cfs), Oskaloosa
Source: *USGS Real-Time Data for Iowa Streamflow*

Year	Annual Peak Discharge	Year	Annual Peak Discharge	Year	Annual Peak Discharge	Year	Annual Peak Discharge
1946	9,600	1962	8,460	1978	8,760	1994	5,640
1947	20,000	1963	5,720	1979	11,000	1995	8,410
1948	9,050	1964	6,000	1980	4,070	1996	8,270
1949	10,800	1965	11,200	1981	4,120	1997	4,800
1950	9,320	1966	12,400	1982	12,000	1998	11,900
1951	7,810	1967	7,600	1983	10,900	1999	7,220
1952	6,700	1968	4,120	1984	12,300	2000	6,190
1953	9,980	1969	10,300	1985	12,600	2001	10,500
1954	5,420	1970	8,160	1986	9,460	2002	5,890
1955	3,700	1971	6,700	1987	14,000	2003	7,890
1956	782	1972	7,180	1988	2,220	2004	11,500
1957	4,860	1973	15,000	1989	3,660	2005	9,810
1958	5,680	1974	12,700	1990	15,200	2006	5,180
1959	8,500	1975	9,740	1991	10,500	2007	11,000

1960	14,800	1976	12,300	1992	7,380	2008	16,900
1961	6,280	1977	5,830	1993	20,700		

Annual Peak Discharge (in cfs), Waterloo
Source: *USGS Real-Time Data for Iowa Streamflow*

Year	Annual Peak Discharge	Year	Annual Peak Discharge	Year	Annual Peak Discharge	Year	Annual Peak Discharge
1941	11,700	1958	4,320	1975	24,100	1992	20,900
1942	23,000	1959	24,700	1976	17,200	1993	68,100
1943	17,700	1960	48,100	1977	3,120	1994	17,200
1944	26,400	1961	76,700	1978	18,000	1995	11,600
1945	53,300	1962	51,200	1979	32,500	1996	9,100
1946	25,200	1963	16,200	1980	25,900	1997	23,300
1947	55,600	1964	4,210	1981	16,300	1998	23,300
1948	38,200	1965	69,500	1982	27,500	1999	69,300
1949	23,100	1966	33,400	1983	27,200	2000	24,900
1950	38,400	1967	18,400	1984	26,300	2001	45,800
1951	56,400	1968	22,800	1985	10,400	2002	7,820
1952	28,000	1969	58,600	1986	33,400	2003	16,200
1953	19,800	1970	9,340	1987	19,700	2004	58,500
1954	49,400	1971	26,700	1988	5,150	2005	23,700
1955	6,590	1972	9,030	1989	6,730	2006	18,100
1956	5,920	1973	37,300	1990	47,100	2007	24,200
1957	7,680	1974	35,500	1991	48,200	2008	104,000

Normal Distribution

X is said to have a Normal Distribution with parameters μ and σ^2 ($\sigma^2 \geq 0$) if X has pdf :

$$f_X(x) = \left\{ \frac{1}{\sigma\sqrt{2\pi}} e^{-\left(\frac{1}{2}\right)\left[\frac{x-\mu}{\sigma}\right]^2} \right. , \quad -\infty \leq x \leq \infty$$

The mean, or expected value, of a Normal Distribution can be expressed as $E(X) = \mu$

The variance of a Normal Distribution can be expressed as $Var(X) = \sigma^2$

$$z = \text{standard normal variate} \equiv \frac{x-\mu}{\sigma} \sim N(0,1)$$

Lognormal Distribution

X is said to have a Lognormal Distribution if log X ~Normal(μ, σ^2). In other words,

if X ~ LogN(μ, σ^2) then ln(X) ~ LogN(μ, σ^2). The pdf of a Lognormal Distribution is :

$$f_X(x) = \left\{ \frac{1}{x\sigma\sqrt{2\pi}} e^{-\left(\frac{1}{2}\right)\left[\frac{\log(x)-\mu}{\sigma}\right]^2} \right. , \quad 0 \leq x \leq \infty$$

The mean and variance of a Lognormal Distribution can be expressed as :

$$E(X) = e^{\frac{\mu+\sigma^2}{2}}$$

$$Var(X) = \left(e^{\sigma^2} - 1\right)e^{2\mu+\sigma^2}$$

Gamma Distribution

X is said to have a Gamma Distribution with parameters α and β $(\alpha > 0, \beta > 0)$ if X has pdf :

$$f_X(x) = \begin{cases} 0 & , \ x < 0 \\ \dfrac{x^{\alpha-1}}{\beta^{\alpha}\Gamma(\alpha)} e^{-\frac{x}{\beta}} & , \ x \geq 0 \end{cases}$$

The mean and variance of a Gamma Distribution can be expressed as :

$$E(X) = \alpha\beta$$

$$Var(X) = \alpha\beta^2$$

Gamma considers skewness, which tells if a distibution is left - tailed (negative) or right - tailed (positive) :

$$\gamma_1 = \text{coefficient of skewness} \equiv E\left(\frac{X - E(X)}{\sqrt{Var(X)}}\right)^3 = \frac{2}{\sqrt{\alpha}}$$

Gamma-3 Distribution

X is said to have a Gamma - 3 Distribution with parameters $\alpha, \beta,$ and γ $(\alpha > 0, \beta > 0, \gamma \in (-\infty, \infty))$

if X - γ ~ Gamma(α, β). Gamma - 3 Distributions have the pdf :

$$f_X(x) = \begin{cases} 0 & , \ x < \gamma \\ \dfrac{(y-\gamma)^{\alpha-1}}{\beta^{\alpha}\Gamma(\alpha)} e^{-\frac{y-\gamma}{\beta}} & , \ x \geq \gamma \end{cases}$$

In other words, if X has a Gamma(α, β, γ) Distribution then X has the same distribtion as W + γ

where W ~ Gamma(α < β)

Pearson Type III Distribution

X is said to have a Pearson Type III Distribution with parameters C_0, C_1, and m ($C_1 \neq 0$), if X has the pdf:

$$f_X(x) = k(C_0 + C_1, x)^m e^{-\frac{x}{C_1}} \quad , \quad x > -\frac{C_o}{C_1}$$

Here, K is the constant such that

$$\int_{-\infty}^{\infty} f_X(x)dx = 1$$

$$X \sim P_3(C_0, C_1, m) \Leftrightarrow X \sim \text{Gamma}(\gamma = -\frac{C_0}{C_1}, \alpha = m+1, \beta = C_1)$$

If $X \sim P_3(C_0, C_1, m)$ then the coefficient of skewness $= \dfrac{2}{\sqrt{m+1}}$

Log Pearson Distribution

X is said to have a Log Pearson Distribution ($X \sim \log P_3$) if $\log X \sim \text{Gamma}(\alpha, \beta, \gamma)$

$$\log X \sim \text{Gamma}(\gamma, \alpha, \beta) \Leftrightarrow X \overset{d}{=} e^Y \quad \text{where} \quad Y \sim \text{Gamma}(\gamma, \alpha, \beta)$$

If $X \sim \log P_3$ then :

$E(\log X) = \alpha\beta + \gamma$

$Var(\log X) = \alpha\beta^2$

$$\gamma_1^* = \begin{cases} \text{coefficient of skewness of the distribution of } \log X \\ E\left(\dfrac{\log X - E(\log X)}{\sqrt{Var(\log X)}}\right)^3 \\ \dfrac{2}{\sqrt{\alpha}} \end{cases}$$

$$\log \xi_p = E(\log X) + K(p, \gamma_1^*)\sqrt{Var(\log X)}$$

$\xi_p =$ the $100p-th$ percentile of the distribution of X so that $P(X \leq \xi_p) = p$

$K(p, \gamma_1^*) =$ the $100p-th$ percentile of the distribution of $\dfrac{\log X - E(\log X)}{\sqrt{Var(\log X)}}$

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