# Selma, Alabama

Flood Risk Management Study
Integrated Feasibility Report and Environmental Assessment

# APPENDIX A

US Army Corps
of Engineers
Mobile District

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# APPENDIX-A: Engineering

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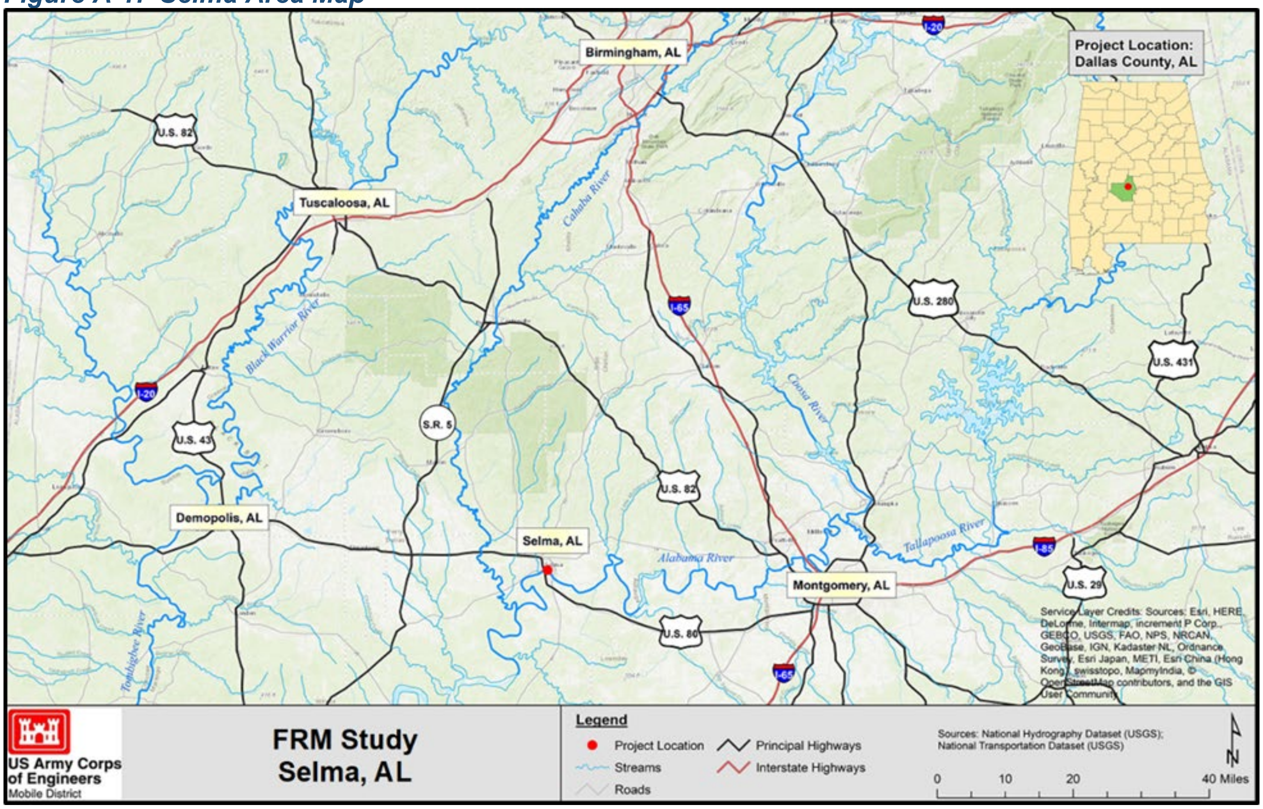
## A.1. Study Area

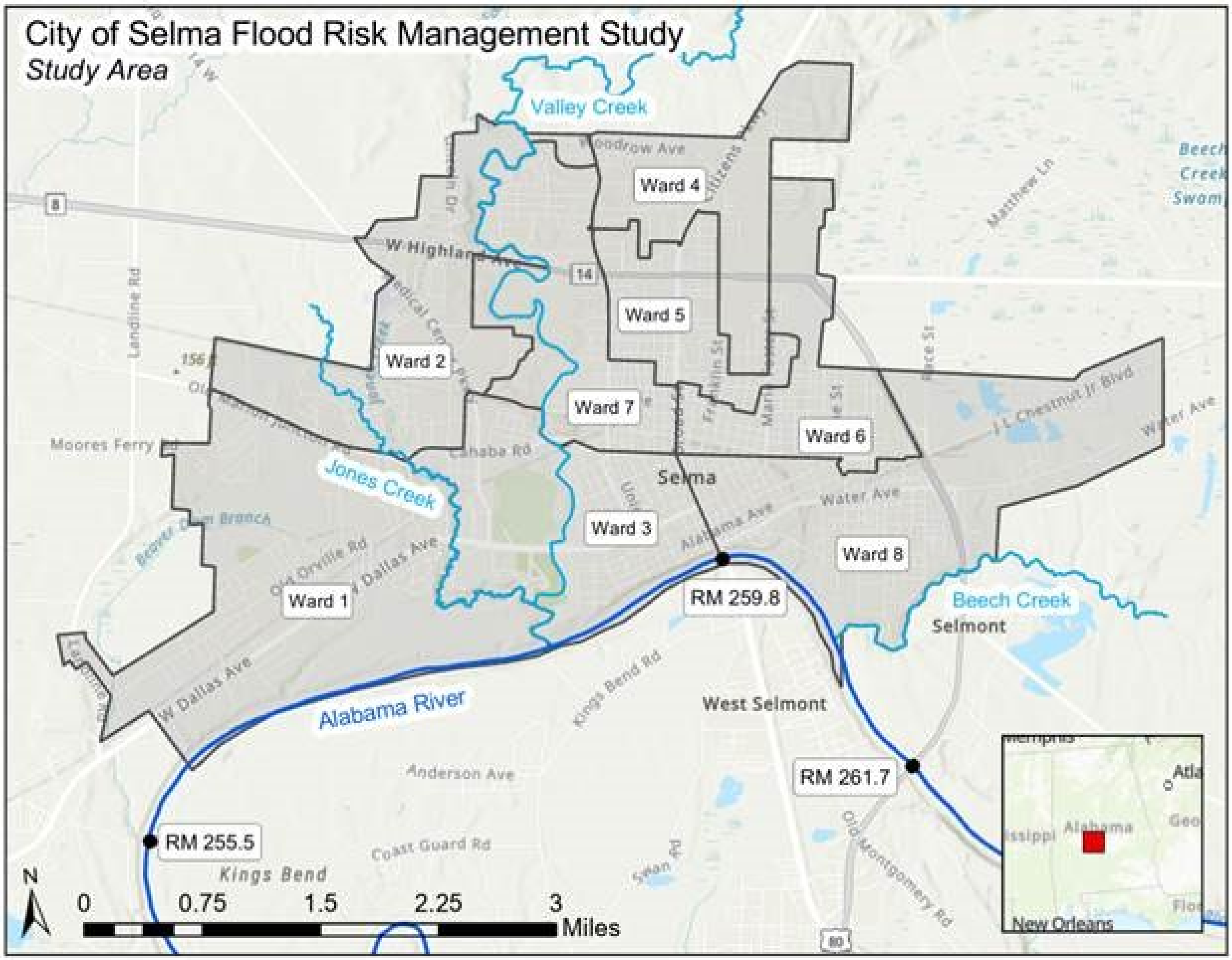
The City of Selma is located on the right bank of the Alabama River in Dallas County, in south central Alabama. The city is located on United States Highway 80, halfway between the cities of Montgomery and Demopolis, AL. Both cities are approximately 51 miles away, with Montgomery to the east and Demopolis to the west. **Figure A-1** shows the location of the City of Selma with respect to the cities of Birmingham, Tuscaloosa, Montgomery, and Demopolis, Alabama.

***Figure A-1: Selma Area Map***




Selma consists of 8 jurisdictions known as wards and are shown on **Figure A-2**. Wards 1, 3, 6, and 8 are the primary areas within the City of Selma where historical flooding has occurred. The study area includes several historically significant buildings, some of which are located directly on the riverbank near Selma's downtown historic district and near the Edmund Pettus Bridge.

***Figure A-2: Locations of Wards in Selma, Alabama***




### *A.1.1. Watershed Characteristics*

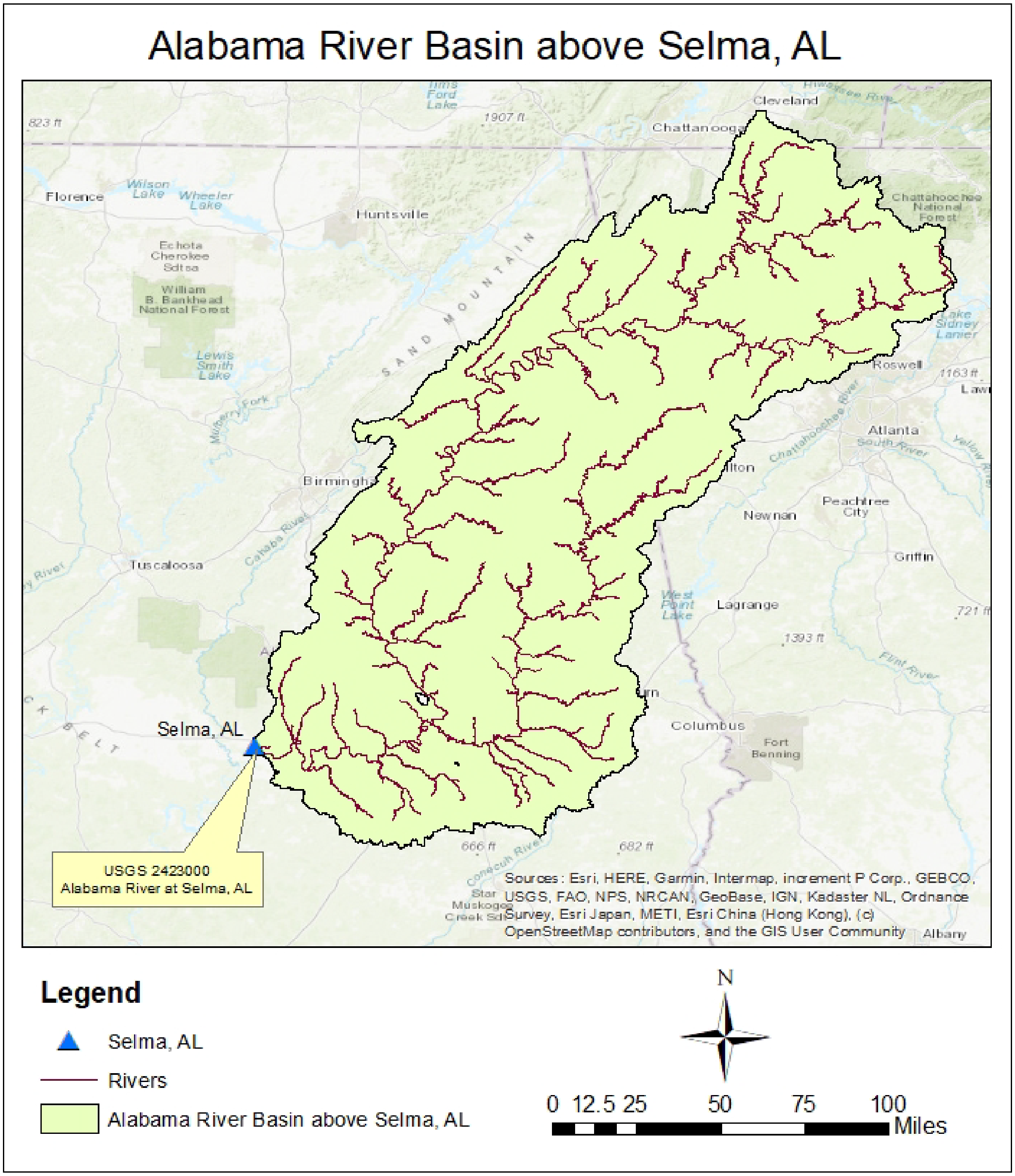
### *A.1.1.1. Drainage Area Description*

The Alabama-Coosa-Tallapoosa (ACT) River System drains a small portion of Tennessee, northwestern Georgia, and northeastern and east-central Alabama. The Alabama River Basin has its source in the Blue Ridge Mountains of northwest Georgia. The main headwater tributaries are the Oostanaula and Etowah Rivers, which join near Rome, Georgia, to form the Coosa River. The Coosa River in turn joins the Tallapoosa River near Wetumpka, Alabama, approximately 14 miles above Montgomery, Alabama, to form the Alabama River.

The upper and middle ACT basin have several federal and private dams located on the main stem rivers. **Figure A-8** shows where all the USACE and privately owned dams are located compared to Selma, AL. There are six flood risk management projects located on these systems. They are, Allatoona Dam, Carters Dam, owned and operated by USACE, and Weiss Dam, Logan Martin Dam, H.N. Henry Dam and Harris Dam, owned and operated by the Alabama Power Company. While these provide a great deal of flood protection for moderate flood events directly downstream of each structure, they provide very little peak stage and flow reduction on the Alabama River near Selma and are not intended to do so. There are also several run-of-river and navigation dams located throughout the basin. These have no impact on the Alabama River near Selma.

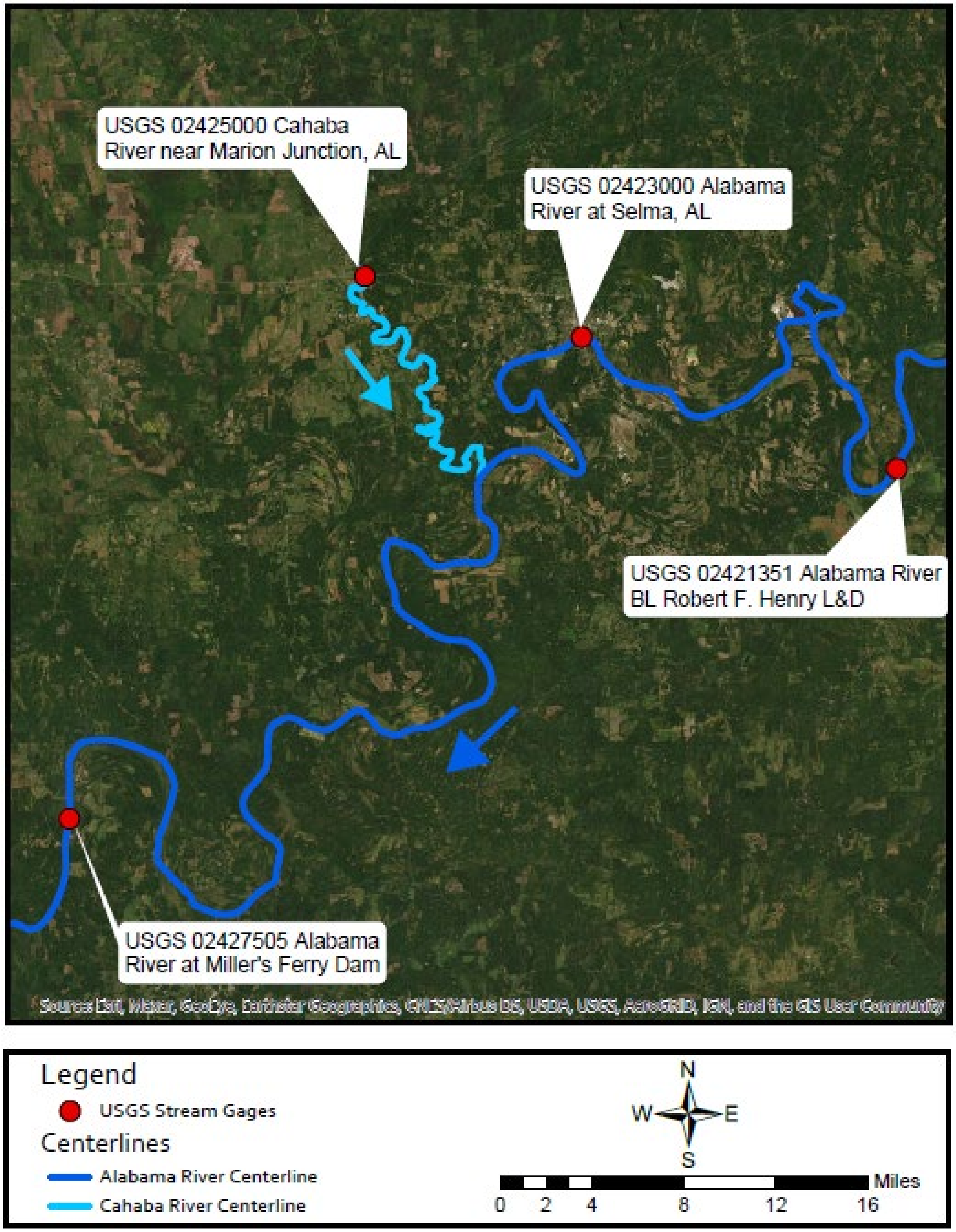
The City of Selma is located on the Alabama River at river mile (RM) 259.77 (above the confluence of the Tombigbee and Alabama Rivers, which form the Mobile River in southwestern Alabama). Above Selma, the Alabama River Basin has a total drainage area of 17,095 square miles (shown on **Figure A-3**).

*Figure A-3: Alabama River Basin map showing drainage basin upstream of Selma, AL*



The study area sits in the pool of Millers Ferry Lock and Dam, located about 30 river miles downstream of the city (RM 187.35), and downstream of Robert F. Henry Lock and Dam, located about 72 river miles upstream of the city (RM 290.4) (shown on **Figure A-4**).

*Figure A-4: Stream gages used hydrologic and hydraulic analysis.*




The impoundment of Millers Ferry Lock and Dam (L&D) raised the river level near Selma several feet, however the operation of these projects have no further impact on the study area as they are both run-of-river navigation dams. Within the study area, there are three tributaries including Valley Creek, Jones Creek, and Beech Creek. The main cause of flooding in Selma is from backwater from the Alabama River flowing into these tributaries.

### A.1.1.2. Available Data

Four (4) United States Geological Survey (USGS) stream gages were utilized for the hydrologic and hydraulic analysis of this study. The gage locations are shown on **Figure A-4** and include USGS 02421351 Alabama River BL Robert F. Henry L&D, USGS 02423000 Alabama River at Selma, AL, USGS 02425000 Cahaba River near Marion Junction, AL, and USGS 02427505 Alabama River at Miller's Ferry Dam NR Camden, AL. The USGS 02423000 gage located at Selma, AL has the longest record of the four gages, with continuous data starting in 1891. Additionally, one historic peak (1886) is attributed to the continuous record. The other three gages have mostly continuous data starting in the early/mid-1970s. In addition, the Marion Junction gage has flow data from 1939-1954. All of the USGS gages used for this study are recorded in NGVD 29 and were converted to NAVD88. **Table A-1** shows the conversion for each location.

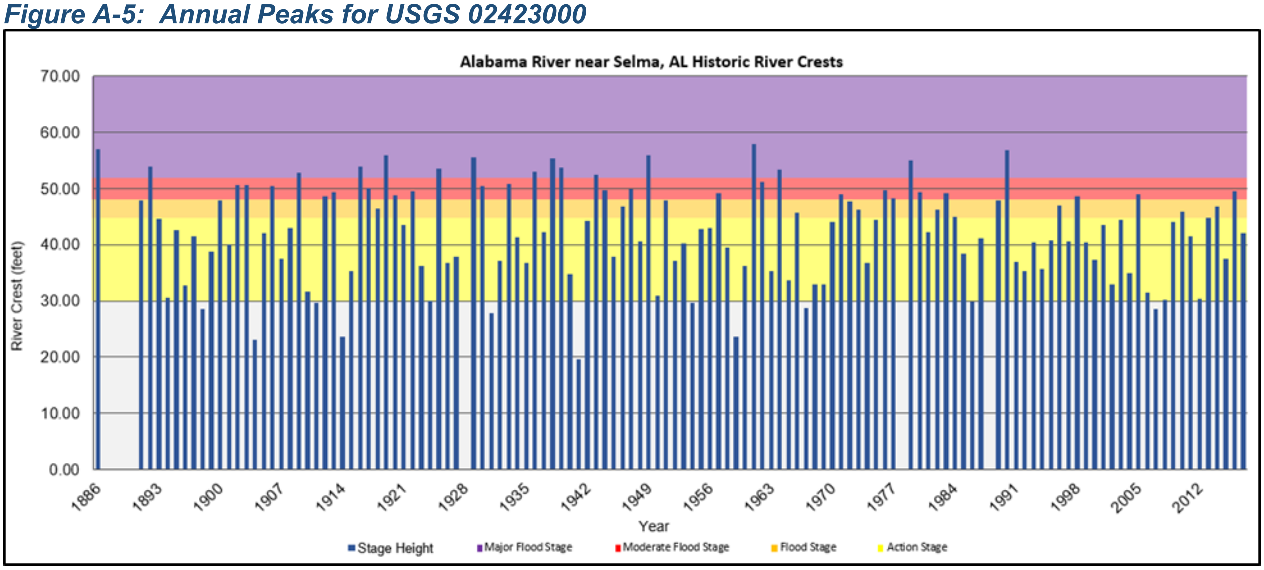
*Table A-1: Datum conversion from NAVD88 to NGVD29 for each gage.*

| Location | NAVD88-NGVD29 (FT) |
|---|---|
| **Miller's Ferry Lock and Dam** | 0.16 |
| **Robert F. Henry Lock and Dam** | 0.09 |
| **Marion Junction, AL** | 0.11 |
| **Selma, AL** | 0.10 |

### A.1.1.3. Flooding History

The City of Selma has a long record of flooding based on available historical data with an observed historical event in 1886. **Figure A-5** shows the annual peaks for the USGS gage 02423000 Alabama River at Selma, AL. This gage location is representative of flood conditions within the project area. There have been 16 major floods, defined by the National Weather Service as the gage height of 52 feet (113.9 feet NAVD88) or above.

*Figure A-5: Annual Peaks for USGS 02423000*



One of the largest floods events on record occurred in 1990. A major storm system in the spring of 1990 produced record floods on the Alabama River. On 16 March 1990, with the river still high from previous rains, the entire basin received very heavy rainfall for two days. For the two-day total, R. F. Henry reported nine inches, Millers Ferry reported 6.75 inches and Claiborne had 9.5 inches. The upper basin received an average of six to seven inches during this period. R. F. Henry passed a record breaking flow of 220,000 cfs on 20 March 1990, producing a record tailwater of 135.5 feet NAVD88. This resulted in the second largest flow on record (280,000 cfs) at the USGS gage located at Selma, AL. The largest known flood for the entire period of record is the historical flood of February-March 1961 with a peak discharge of 284,200 cfs. Another significant flood occurred on 11-16 March 1929, when 10 inches of rainfall over a period of three days was recorded in the vicinity of Auburn, Alabama. The recorded flow was 220,000 cfs at Selma. **Figure A-6** shows an aerial view of the flooding in the Selma and Selmont, AL areas in this 1929 event. For the historical flood in April 1886, the peak discharge of 248,000 cfs was recorded at the Selma gage. This was the greatest flood on record for the Millers Ferry Project which is downstream of Selma.

***Figure A-6: Aerial Image of Selma, AL during 1929 Flood (Source: NWS Floods in Alabama)***

### *A.1.1.4. Hydrology/Runoff Characteristics*

#### *A.1.1.4.1. Temperature*

The average daily low and high temperatures in the study area range from the mid to upper-30s to upper-50s/low-60s (in ˚F) for the winter months and the high-60s to the upper-80s/low-90s in the summer months. (US Climate Data, 2020)

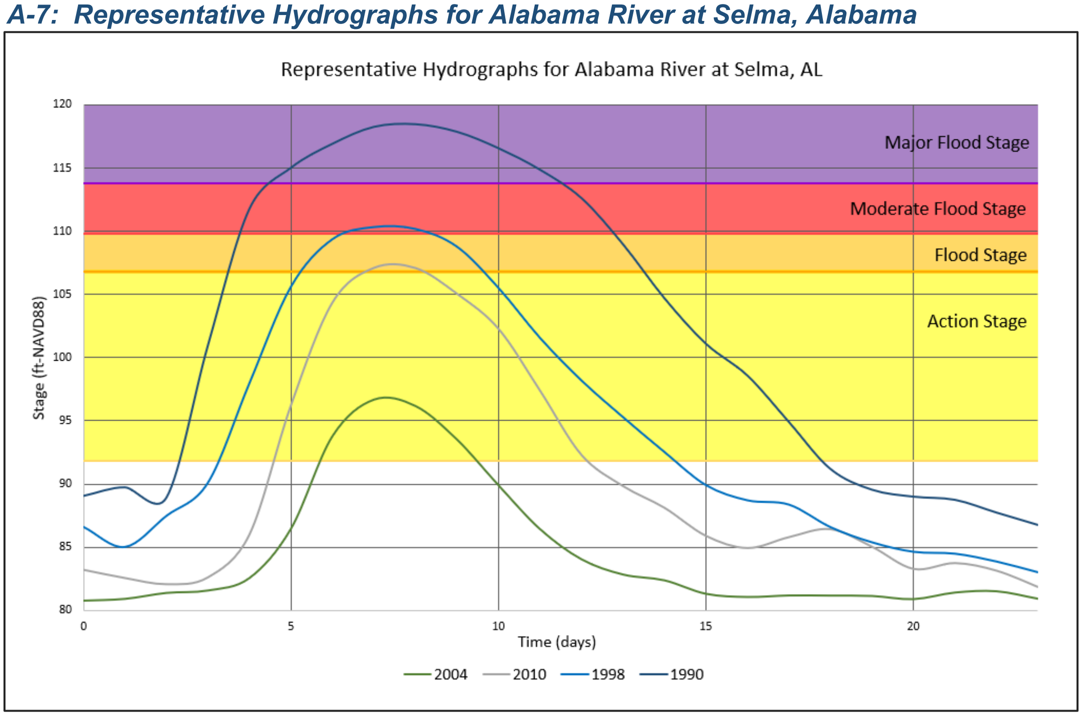
### *A.1.1.4.2. Rainfall*

The average annual precipitation is approximately 55 inches, with monthly averages ranging from a low of 3.54 inches in April to a high of 6.46 inches in July (this data comes from the same source as that listed above). Synthetic rainfall data for the study area, per National Oceanic Administration (NOAA) Atlas 14, show that rainfall depths range from 0.437 inches for the 1-year, 5-minute storm to 12.4 inches for the 500-year, 24-hour storm.

### *A.1.1.4.3. Hydrograph Characteristics*

The streams which constitute the Alabama River above the City of Selma exhibit wide variations in runoff characteristics, ranging from very flashy in the mountainous regions of the Coosa Basin above Rome, Georgia, to very slow rising and falling in the lower reaches, which includes the stretch of river near Selma. A typical hydrograph at Selma increases slowly over several days before reaching a peak flow, then recedes at a slower pace. Large events usually occur over several weeks, sometimes lasting over a month. **Figure A-7** shows representative hydrographs of major (i.e., extensive inundation of structures and roads), moderate (i.e., some inundation of structures and roads near streams), minor (i.e., minimal or no property damage, but possibly some public threat), and action (i.e., some type of mitigation action in preparation for possible significant hydrologic activity) stage events for the Alabama River at Selma, Alabama. Major, moderate, minor, and action stage descriptions are per the National Weather Service definitions.

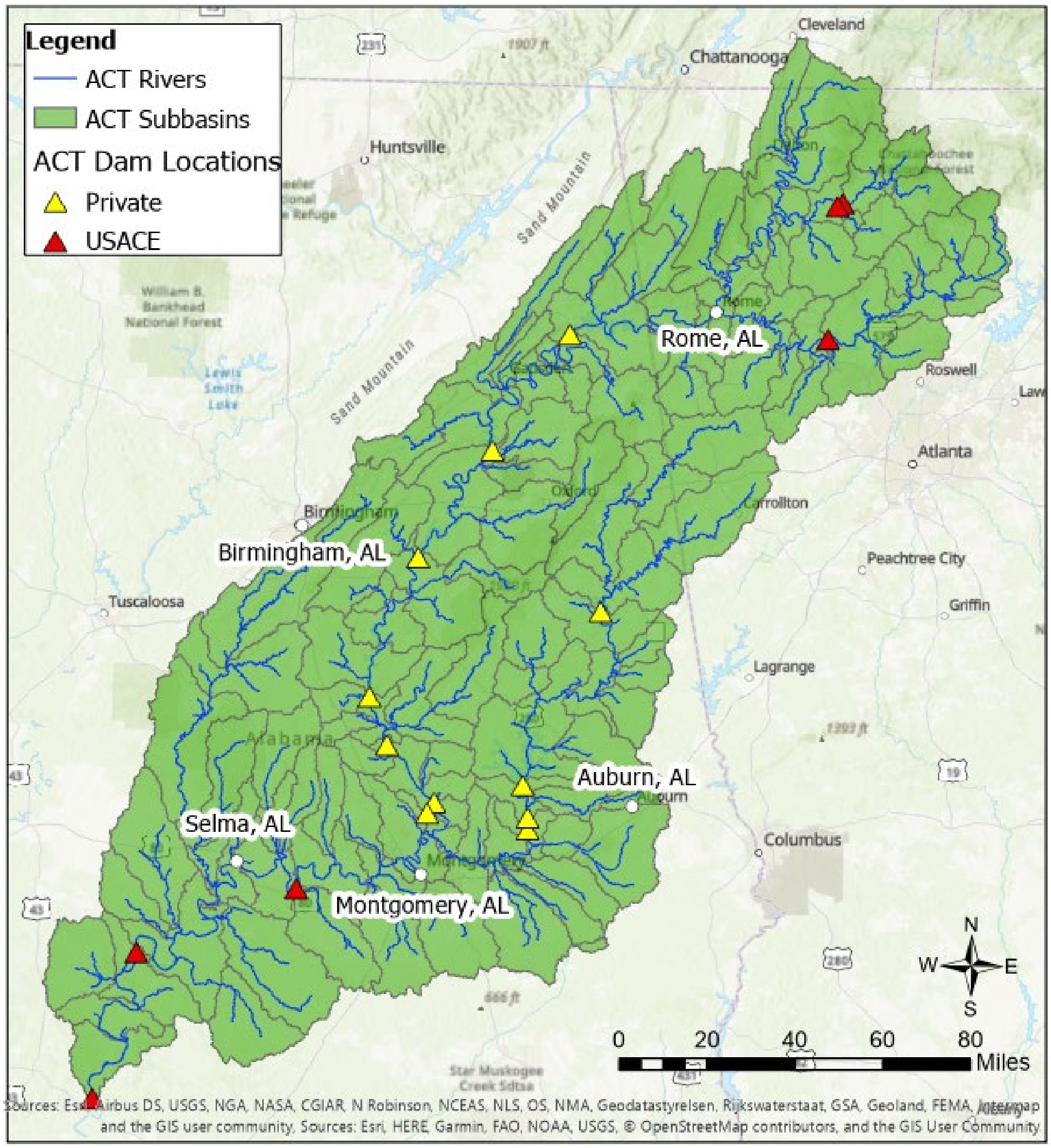
*Figure A-7: Representative Hydrographs for Alabama River at Selma, Alabama*



### *A.1.1.5. Hydrologic and Hydraulic Characteristics*

The Alabama River Basin is a large, diverse basin consisting primarily of broad wooded areas in the upper basin as well as several large urban areas near and upstream of Selma, AL. Overland flow from rain events and stream conveyance in forested and wooded areas found within the upper basin will result in a slow moving flow whereas water will typically convey much faster in the urban areas due to increased land coverage of impervious areas such as asphalt parking lots and roadways. Urbanization within the Alabama River Basin is primarily occurring in areas such as Rome, GA and Montgomery, AL. **Figure A-8** shows some of the locations where urbanization is occurring with respect to Selma, AL.

*Figure A-8: Alabama River Basin and contributing rivers and tributaries.*

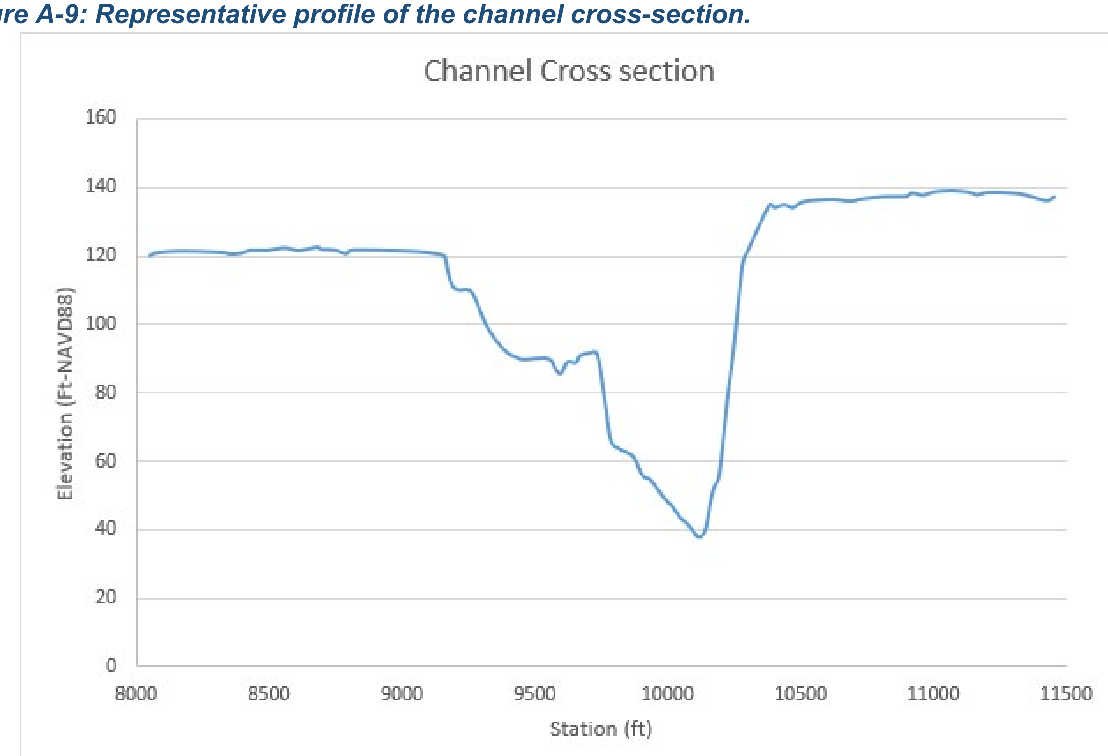



The basin is located over two distinct topographies. The middle and norther portion of the basin is steep and mountainous with narrow floodplains, causing streamflow to be flashier with short, acute high flow events. The southern portion of the basin below Montgomery, Alabama becomes extremely flat with many sections of wide floodplain. Hydrographs in this area of the basin, including the study area, are very slow moving.

The Alabama River channel is approximately 35 feet deep in the vicinity of Selma Alabama with an approximate width of 700 feet at bank-full capacity. The river is fairly clear of debris with some vegetation on the slopes of the river. The floodplain upstream and downstream of the river ranges from cleared farmland to densely vegetated forests. Roughness coefficients (Manning's n-values) used in modeling ranged from 0.032 -0.037 for the channel section. Roughness on the overbanks and floodplain ranged from 0.05 – 0.12

Side slopes of the underwater portion of the river channel along the Alabama River in the entire vicinity of Selma are fairly consistent as this channel has historically been dredged for navigation. This dredging is limited to areas well under the normal water level of the river. Historically the area of the river has been dredged up to annually, depending on need. But all dredging stopped in the late 2000s as this waterway was classified as low use. **Figure A-9** shows a profile of the channel as well as the overbank near downtown Selma looking downstream based on a 2019 bathymetric survey of the river and 2016 LiDAR flow for the USGS 3DEP program. Upstream and downstream of the city of Selma, the land is very flat on both sides of the river, with a floodplain width of up to 4 miles. The downtown area of Selma sits on a high bluff on the right bank of the river with a very steep, almost vertical bluff. This bluff is very susceptible to erosion as the river fluctuates during flood events. The Alabama River fluctuates over 35 feet from typical average flow levels to the 0.01 AEP flow.

***Figure A-9: Representative profile of the channel cross-section.***



### *A.1.1.6. Land Use*

In the Alabama River Basin above Selma, AL, there is a large variety of land use including impervious areas within metropolitan areas and forests throughout the basin. **Table A-2** shows the breakdown of percentages for each land use type. **Figure A-10** shows the land use in the basin above Selma Alabama. The study area itself is primarily impervious areas surrounded by pastures and woody wetlands as seen on **Figure A-11**. There are areas of forests and crop land located sporadically outside of the city with very little inside of Selma city limits.

*Table A-2: Percentage of Alabama River Basin Land Use Types above Selma, AL*

| Land Use Type | Percentage of Area Above Selma, AL |
|---|---|
| **Open Water** | 2.1% |
| **Developed, Open Space** | 6.4% |
| **Developed, Low Intensity** | 2.3% |
| **Developed, Medium Intensity** | 0.8% |
| **Developed, High Intensity** | 0.3% |
| **Barren Land Rock/Sand/Clay** | 0.3% |
| **Deciduous Forest** | 33.3% |
| **Evergreen Forest** | 17.8% |
| **Mixed Forest** | 6.1% |
| **Shrub/Scrub** | 7.5% |
| **Grassland/Herbaceous** | 4.3% |
| **Pasture/Hay** | 12.6% |
| **Cultivated Crops** | 3.0% |
| **Woody Wetlands** | 3.1% |
| **Emergent Herbaceous Wetlands** | 0.2% |

***Figure A-10: Land Use in the Alabama River Basin and surround areas upstream of Selma, AL***

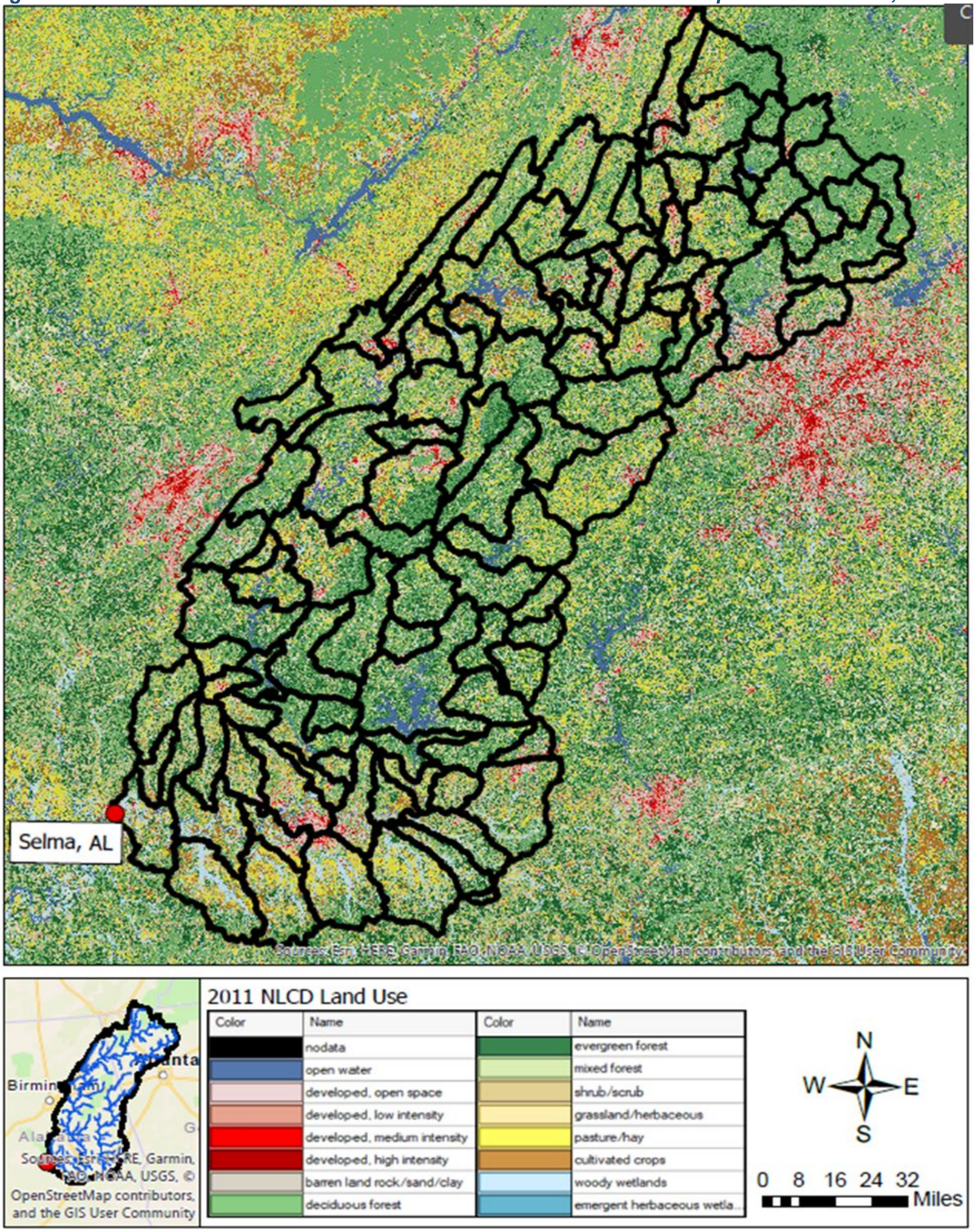
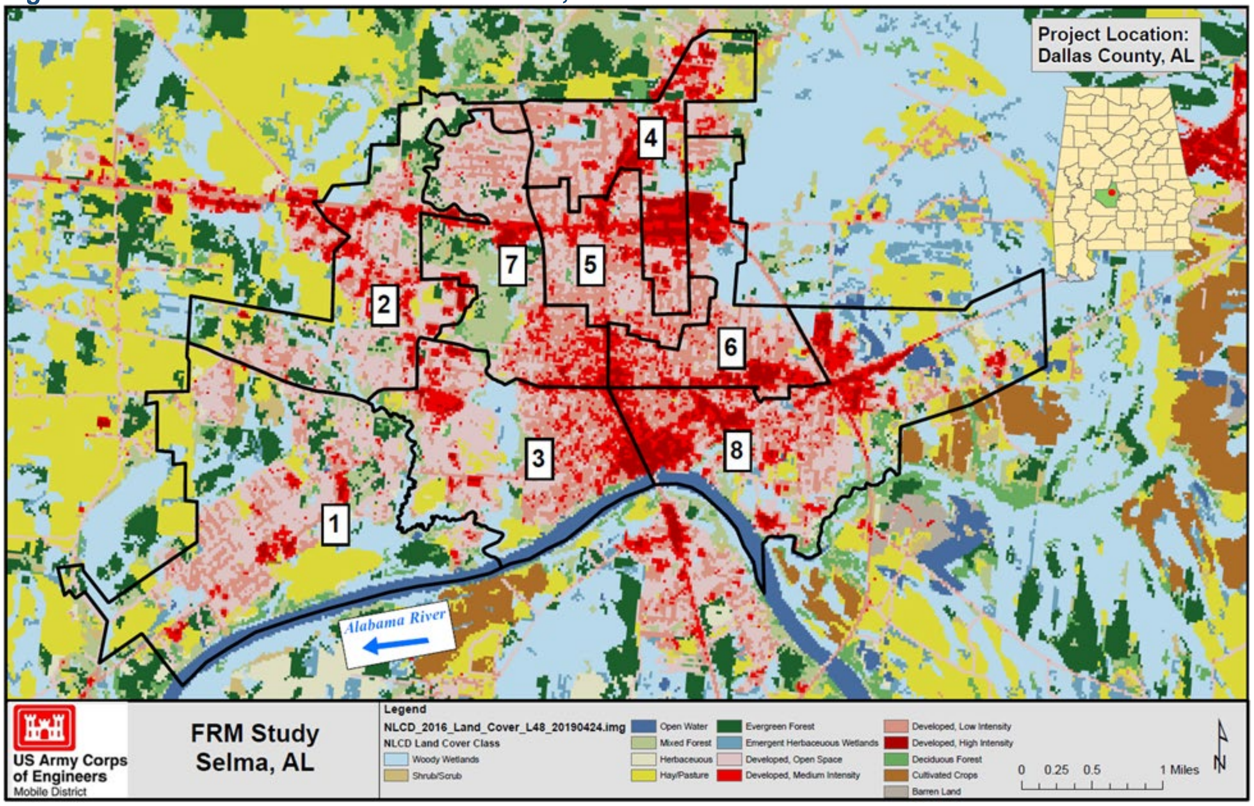


***Figure A-11: Land Use in and around Selma, AL***




## *A.1.1.7. Alluvium and Soils*

The geology in and around the City of Selma consists of alluvial deposits, or sands, silts and clays left behind as a result of flowing water. These deposits are underlain by various formations within the Selma Group, the most prevalent of these being the Mooreville Chalk. Alluvium deposits consist of a mixture of varicolored, fine to coarse sand with clay lenses and gravel. The Mooreville Chalk is generally characterized as a yellowish-gray to olive-gray clayey chalk or chalky marl. Visual survey in the vicinity of the study area indicates that the banks are steep (1v:1.5h and steeper) and comprised of sands, silts, and clays that sit atop a layer of “chalk”. Historical borings from past geotechnical explorations confirm this assessment, noting that the “chalk” layer is dense and strong. Recent soil borings have determined that the “chalk” layer is composed of hard, fat clay soils and is documented to be segment of the Mooreville formation. The north river bank along the downtown Selma area ranges in height between 30 to 50 feet above the water’s surface (water surface elevation at the Edmund Pettus Bridge is about 84 ft NAVD88 in normal flow conditions). The interface of the overburden and the chalk is easily spotted from the river, and this interface appears anywhere from 5 to 20 feet above the water’s surface. It is likely that the presence of the hard, relatively impervious clay strata has directed the Alabama River to turn westward at Selma.

The alluvial soils typically have a thickness of approximately 30 feet at the street level and approximately 15 feet along the bluff. The upper portion of the alluvial layer is clays, clayey sands and silty sands. Below this layer is fairly clean sands with an approximate thickness ranging between 10 and 20 feet. This layer sits atop the Mooreville clay statum and contains a suspended groundwater table. The significance of this arrangement of soil layers is to focus the flow of groundwater (or drainage of river water due to sudden drawdown during flood events) and can readily erode the sands to the river. Overtime the progression of this erosional process will undermine the overlying alluvium soils supporting the buildings.

#### *A.1.1.8. Geology and Soils*

The ACT River Basin covers an unusually wide range of geologic conditions. The location of the river basin is in portions of five physiographic provinces: the Blue Ridge Province; the Valley and Ridge Province; the Piedmont Plateau; the Cumberland Plateau; and, the Coastal Plain. Each of these physiographic sub-divisions influences drainage patterns. Rugged crystalline rocks characterize the northeastern portion of the basin in the Blue Ridge Province. Folded limestone, shale, and sandstone compose the Valley and Ridge Province. The axes of the folds that trend northeast-southwest influence the course of the streams in that they tend to flow southwestward along the alignment of the geologic structure. Like the Valley and Ridge Province -- folded, faulted, and thrusted rocks form the Cumberland Plateau -- with the deformation being less than the Valley and Ridge rocks. The east-central portions of the basin are in the Piedmont Province, characterized by sequences of metamorphic and igneous rocks. Prominent topographic features generally reflect the erosional and weathering resistance of quartzite, amphibolite, and plutonic rocks. The residual soils are predominately red sandy clays and gray silty sand derived from the weathering of the underlying crystalline rocks. The more recent sedimentary formations of the Coastal Plain underlie the entire southern portion of both river basins. The contact between the Coastal Plain on the south and the previously described physiographic provinces to the north is along a line that crosses the Cahaba River near Centreville, Alabama; the Coosa River near Wetumpka, Alabama; and the Tallapoosa River near Tallassee, Alabama. As the rivers leave the hard rocks above this line and enter the softer formations of the Coastal Plain, the erosion properties change, resulting in the formation of rapids. This line is a geological divide commonly known as the "fall line". The rocks of the Coastal Plain are typically poorly consolidated marine sediments.

The Selma area is situated near the center of the Black Prairie subdivision of the Gulf Coastal Plain physiographic province in the southern portion of the ACT Basin. The Black Prairie subdivision is a belt of low relief which crosses the state in and east-west direction. In the Selma area, it is about 20 miles wide and consists of flat to gently undulating prairie land. The major drainage of the area is by the entrenched and meandering Alabama River which crosses the prairie belt in a southwesterly direction. The Black Prairies correspond in length and width to the weathered outcrop of the Selma Group of late Cretaceous age which is a chalky to argillaceous limestone formation with a maximum known thickness of about 900 ft. The general dip of the strata in the Selma area is about 30 ft per mile to the south.

## A.2. Climate Change

### *A.2.1. Introduction*

In 2016, USACE issued Engineering and Construction Bulletin No. 2016-25 (hereafter, ECB 2016-25) which mandated climate change be considered for all federally funded projects in planning stages (USACE, 2016). This guidance was updated with ECB 2018-14 (USACE, 2018), which mandates a qualitative analysis of historical climate trends and assessment of future projects. Even if climate change does not appear to be an impact for a particular region of interest, the formal analyses outlined in the guidance, result in better-informed planning and engineering decisions.

### *A.2.2. Literature Review*

A literature review was performed to summarize climate change literature and highlight both observed and projected assessments of climate change variables relevant to the study area. Since this is a flood risk management project, the primary variable that is relevant is streamflow. However, this variable is also affected by precipitation and air temperature. Therefore, this review focuses on observed and projected changes in precipitation, air temperature, and hydrology.

#### *A.2.2.1. Temperature*

##### *A.2.2.1.1. Observed Temperature*

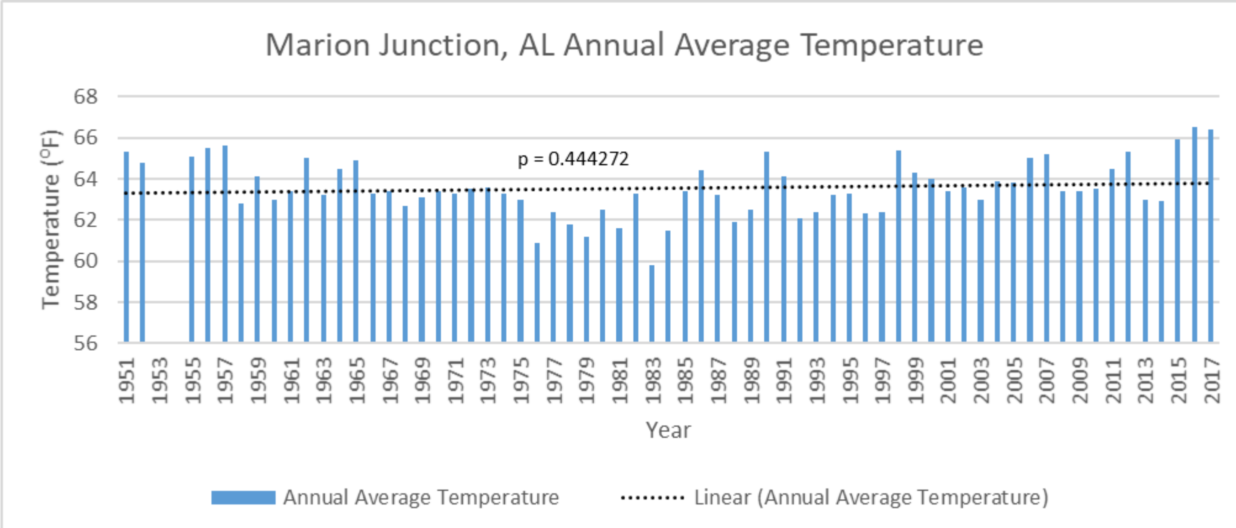
The Fourth National Climate Assessment (USGCRP, 2017) states that observed temperatures in the United States have increased up to 1.9 degrees Fahrenheit since 1895, with an acceleration in increasing temperatures since the 1970s. Warming is projected for all parts of the United States (USGCRP, 2017).

The USACE Institute for Water Resources (IWR) conducted a review in 2015 which summarized the available literature on climate change for the South Atlantic-Gulf Region, including the study area (USACE, 2015). In general, studies have shown that over the last century, a period of warming in the region has been observed since a transition point in the 1970s. This transition period was precluded by an observed cooling period (see Patterson et al., 2012; Laseter et al., 2012; and Dai et al., 2011). The overall warming trend is fairly inconsistent for the region over the last century. The IWR report indicates only mild increases in annual temperature for the region with significant variability. However, there is a clear consensus in general warming since the early 1970s (USACE, 2015).

For the project area, there are a few NOAA gages in proximity of Selma with records longer than thirty years. The NOAA gage located in Selma, AL (beginning in 1895) was going to be analyzed, however, the dataset has large gaps for the more recent years. The trend from this data shows a decreasing trend, which is inconsistent with the national and regional reports. Therefore, the NOAA gage located in Marion Junction, AL with a record from 1951 - 2017 (continuous record 1955 – 2017) was used to analyze temperature trends in the area.

A statistical analysis was performed on the entire dataset from Marion Junction, AL. Data from the USGS gage was tabulated in an excel spreadsheet and, using the t-Test tool in the Analysis ToolPak the probability value, or p-value for the dataset was determined. The results of this analysis are shown in **Figure A-12** with the associated p-value.

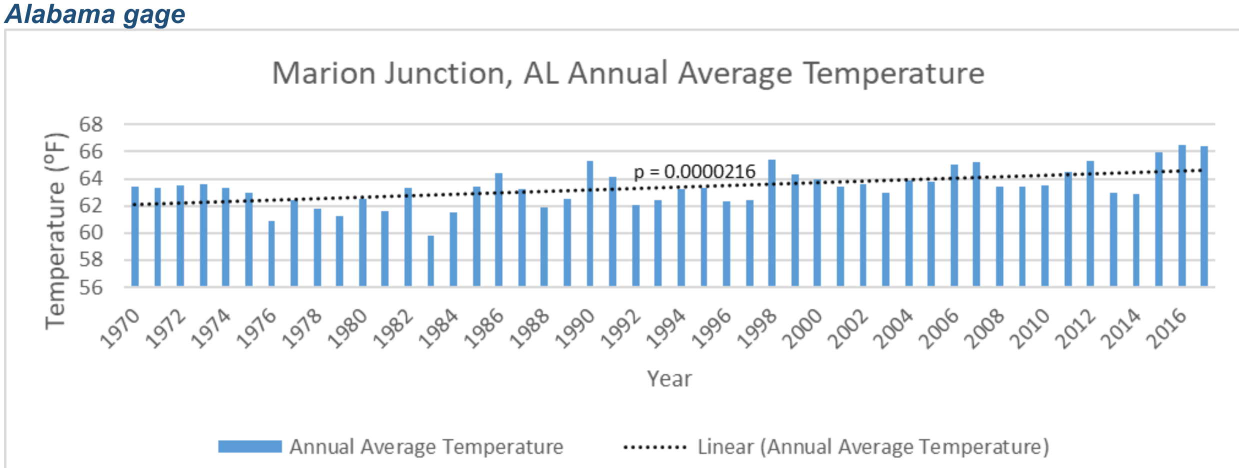
***Figure A-12: Annual average temperature and p-value from 1951 - 2017 for Marion Junction, Alabama gage***




The alternative hypothesis of an apparent trend is accepted to be true at the 0.05 significance level, meaning that p-values less than 0.05 are indicative of statistical significance. This is a threshold commonly adopted within statistical references, but consideration should also be given to trends whose p-values are close to this reference threshold. In this case, the period of record data produces a high p-value of 0.444272; therefore, it is not considered to have a significant increasing or decreasing trend.

However, performing the same test of average annual temperatures from 1970 - 2017 (shown on **Figure A-13**) produces a p-value of 0.0000216. This would be considered very indicative of a statistically significant upward trend in temperatures.

***Figure A-13: Annual average temperature and p-value from 1970 - 2017 for Marion Junction, Alabama gage***




The temperature gage located in Rome, GA was also analyzed (shown in **Figure A-14**). The p-value for the entire period of record is 0.000482, which indicates the downward trend is statistically significant. However, there is a cooling period that occurred in the 1970s that may be skewing the data. **Figure A-15** shows the Rome, GA gage temperature data from 1970 -2018.

Both gages show a statistically significant upward trend from 1970 – 2018. Visually, there appears to be an oscillating pattern with the annual average temperature. The temperatures prior to the cooling period (1970s) look similar to temperatures in the early and mid-1900s. Without longer periods of record to compare with, it is difficult to come up with a conclusion.

***Figure A-14: Annual average temperature and p-value from 1902 - 2018 for Rome, GA gage.***

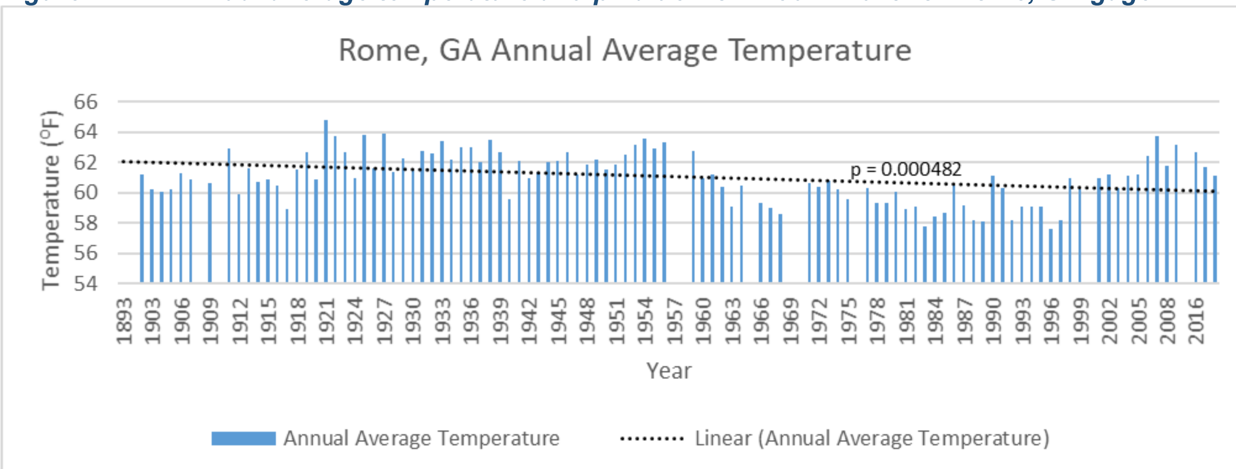
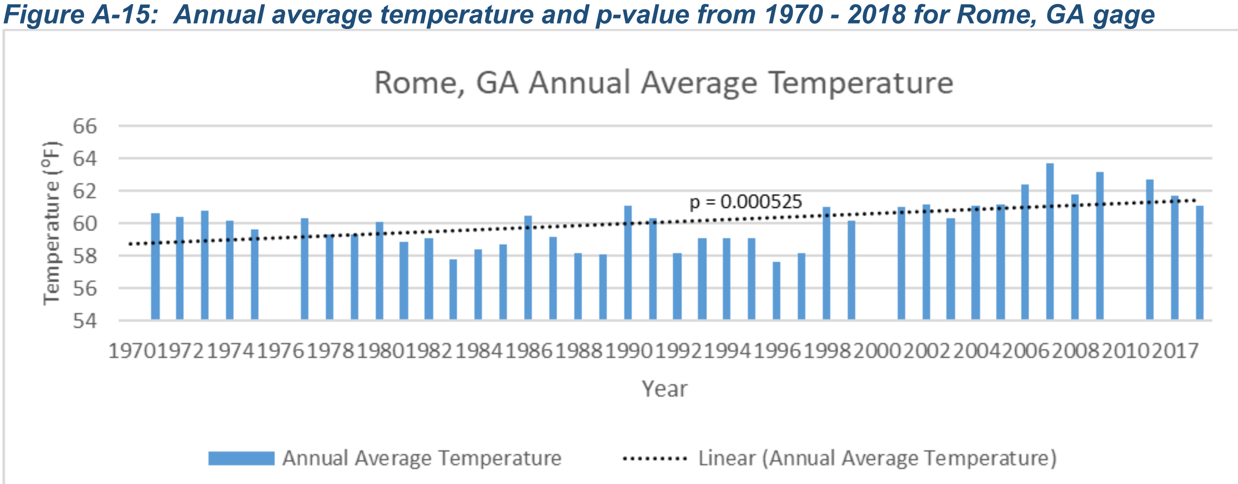


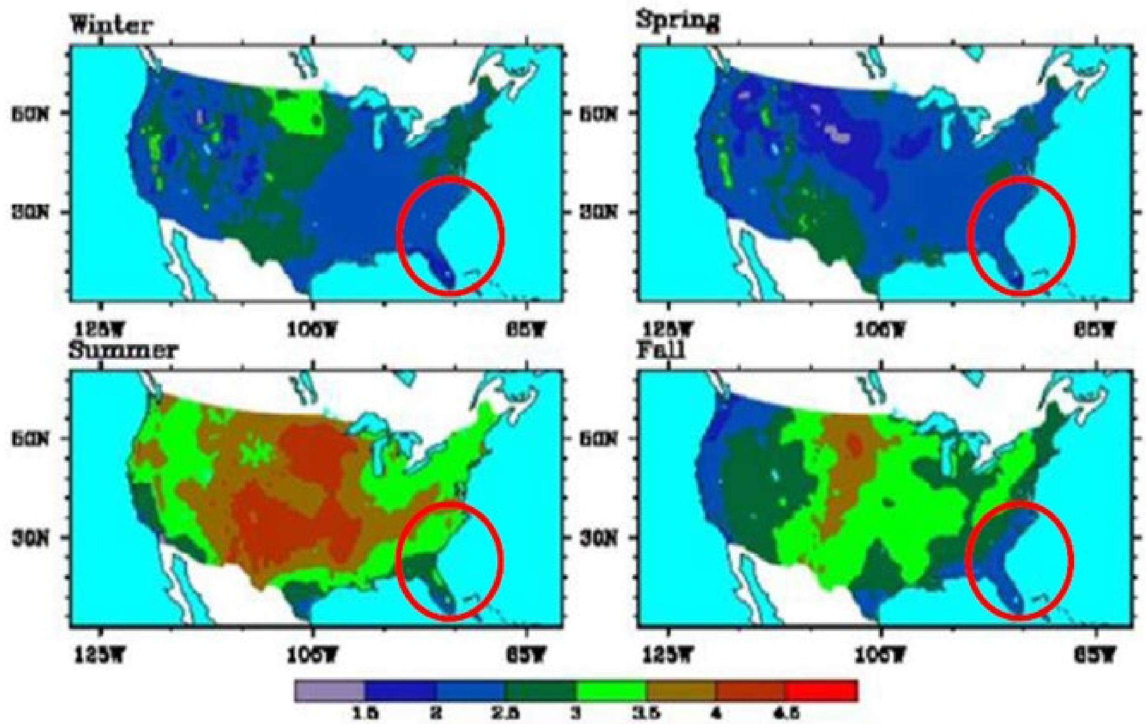

***Figure A-15: Annual average temperature and p-value from 1970 - 2018 for Rome, GA gage***




### *A.2.2.1.2. Projected Temperature*

Global Circulation/Climate Models (GCMs) have been used to project future climate conditions in the U.S. including the southeast regions. Results show a significant warming trend at a national and regional scale. **Figure A-16** shows the projected changes in seasonal maximum air temperatures from Liu et al. (2013), which is based on a "worst case" greenhouse gas emissions scenario. This shows that, overall, there is a projected warming trend of 2 to almost 4 degrees by 2070.

*Figure A-16: Projected changes in seasonal maximum air temperature, ºC, 2041 – 2070 vs. 1971 – 2000. The South Atlantic-Gulf Region is within the red oval (Liu et al., 2013; reprinted from USACE, 2015)*




### A.2.2.2. Precipitation

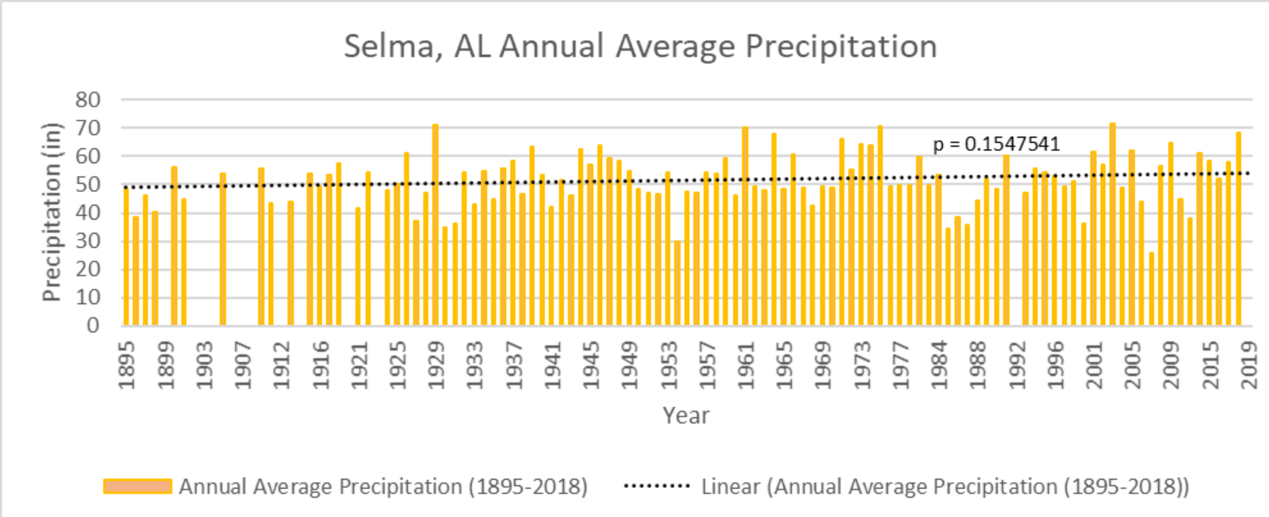
#### A.2.2.2.1. Observed Precipitation

The IWR report (USACE, 2015) shows there is a general increase in precipitation for the southeast region; however, it is highly variable for the region. Analysis of gridded data spanning years 1950-2000 showed that winter precipitation has consistently increased over the last century (Wang et al., 2009). Other seasons have shown high variability including increases, decreases, and little change in precipitation across the region.

A study by Patterson et al. (2012) did not identify any patterns of precipitation change using monthly and annual trend analysis for a number of climate and streamflow stations within the South Atlantic-Gulf Region (data included 1934 - 2005). However, the study found that more sites exhibited mild increases in precipitation than those that exhibited decreases.

Similar results were seen at the NOAA gage in Selma. Similar to the temperature analysis, aata from the USGS gage was tabulated in an excel spreadsheet and, using the t-Test tool in the Analysis ToolPak the p-value for the dataset was determined. The gage has a large record for precipitation spanning from 1895 – 2018, however, the p-value is 0.1547541 which means there is no statistical significance (see **Figure A-17**). Visually, the dataset seems to be consistent with high and low values being similar throughout the entire record. It appears that there are more low values for precipitation in recent years, even though the trend appears to increase overall.

*Figure A-17: Annual total precipitation and p-value from 1895 - 2018 for Selma, Alabama gage*

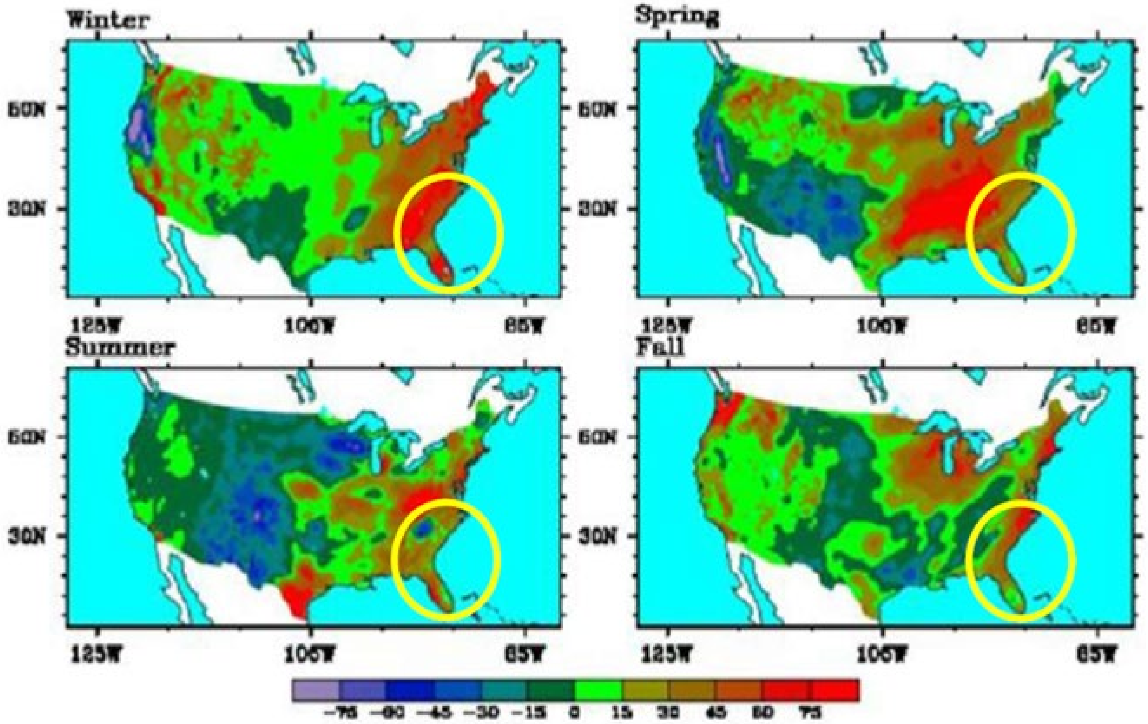



Most studies analyzed by the IWR (USACE, 2015) suggests significance in increasing precipitation severity and frequency trends in observed storms are not definitive. Some of the analyzed literature shows mild increasing trends in these parameters. For instance, Li et al. (2011) investigated anomalous precipitation (based on deviation from the mean) in summer months in the southeastern U.S. and found a greater number of climate stations within the region did not exhibit increasing trends in the frequency of occurrence of heavy rainfall. Increases were also shown by Wang and Killick (2013), who found that 20% sites analyzed, within 56 southeastern watersheds, exhibited increasing trends for the 90th quantile precipitation months. Though there is not a strong consensus regarding trends in extreme precipitation events, it is important to remain mindful of the identified increasing trends in intensity and frequency of rainfall within the region.

### *A.2.2.2.2. Projected Precipitation*

Projected of future changes in precipitation for the southeast region are variable and lack consensus. Liu et al. (2013) quantified significant increases in winter and spring precipitation associated with a 2055 future condition for the South Atlantic Region. However, other seasons showed almost no increase or a slight decrease in precipitation. **Figure A-18** illustrates the projected change in seasonal precipitation. The authors also project increases in the severity of future droughts for the region, leading to projected temperature and evapotranspiration impacts that outweigh the increases in precipitation.

*Figure A-18: Projected changes in seasonal precipitation, 2055 vs. 1985, mm. The South Atlantic-Gulf Region is within the yellow oval (Liu et al., 2013; reprinted from USACE, 2015)*




### A.2.2.3. Hydrology

#### A.2.2.3.1. Observed Streamflow

Generalized observations of streamflow trends in the southeast lack a clear consensus, with some models showing positive trends in some areas and others showing negative. Generally, most studies in the southeast showed no trend in streamflow or a negative trend. Most notably, studies have shown that the negative trend in streamflow being more consistent for the region since the 1970s (Kalra et al., 2008; and Patterson et al., 2012).

For the study area, there is a noticeable decreasing trend for streamflow in the Alabama River based on the excel analysis on streamflow. At the gage upstream of the study area (USGS 02420000 near Montgomery, AL), the p-value is 0.004737 which indicates the trend is statistically significant (**Figure A-19**). At USGS 02428400, Alabama River at Claiborne L&D near Monroeville, there is a decreasing trend as well; however, it is not considered statistically significant (p-value of 0.236750; **Figure A-20**). The gages indicate that there is decreasing trends in stream flow for the Alabama Basin based on the observed data. This could be the result from flood control projects in the upper portions of the basin. Some of the larger projects were built prior to 1976, therefore the notably decreasing trend in streamflow may not be as apparent compared to the Montgomery, AL stream gage.

***Figure A-19: Annual Peak Streamflow USGS 02420000 Alabama River near Montgomery, AL***

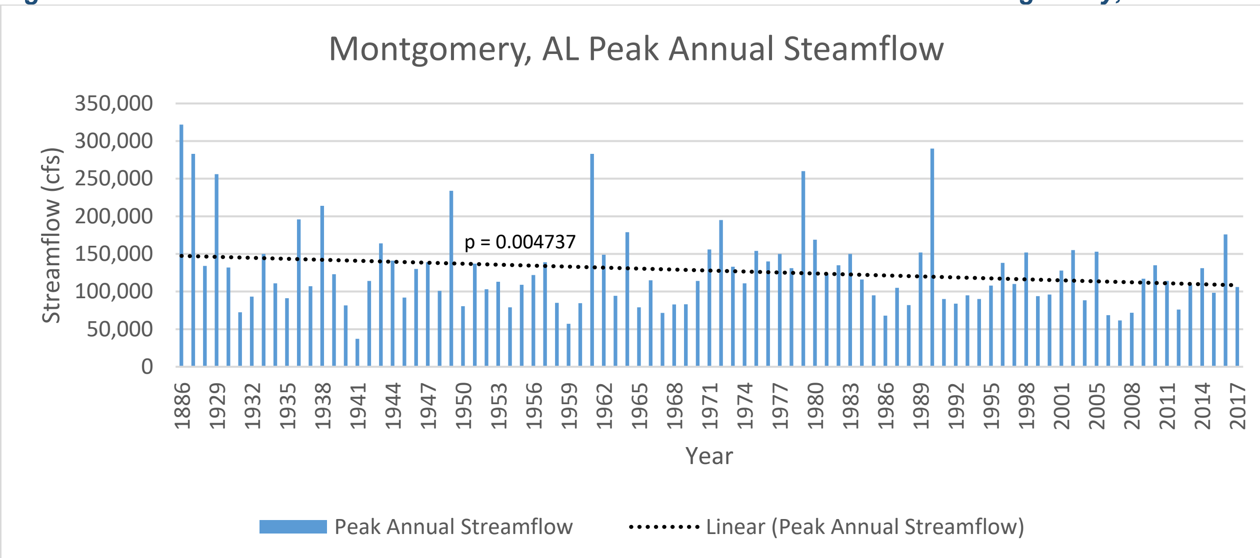
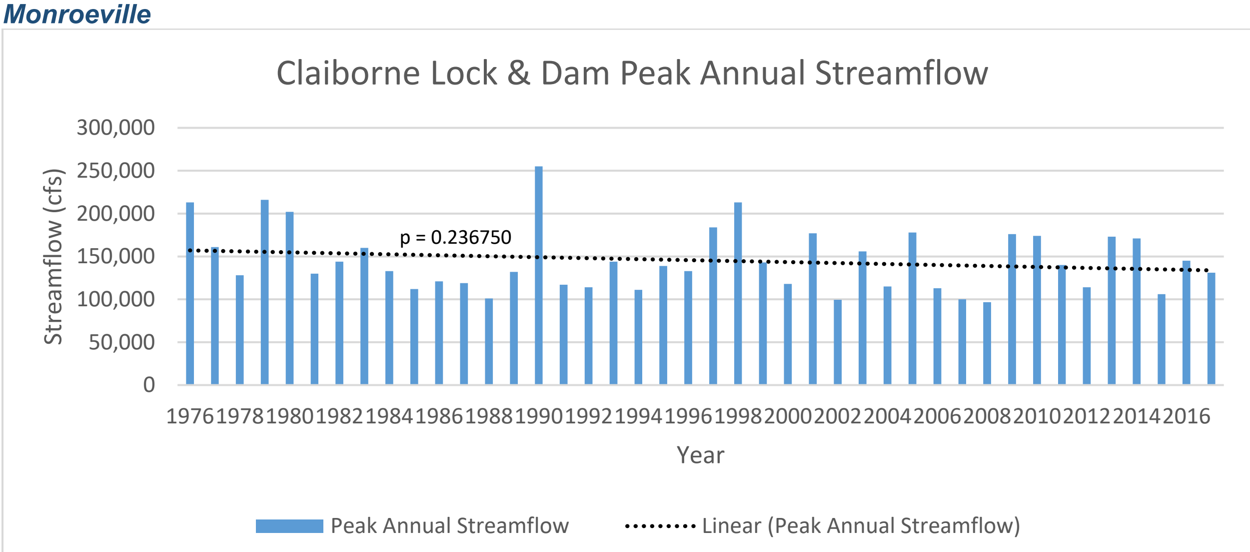



***Figure A-20: Annual Peak Streamflow at USGS 02428400 Alabama River at Claiborne L&D near Monroeville***



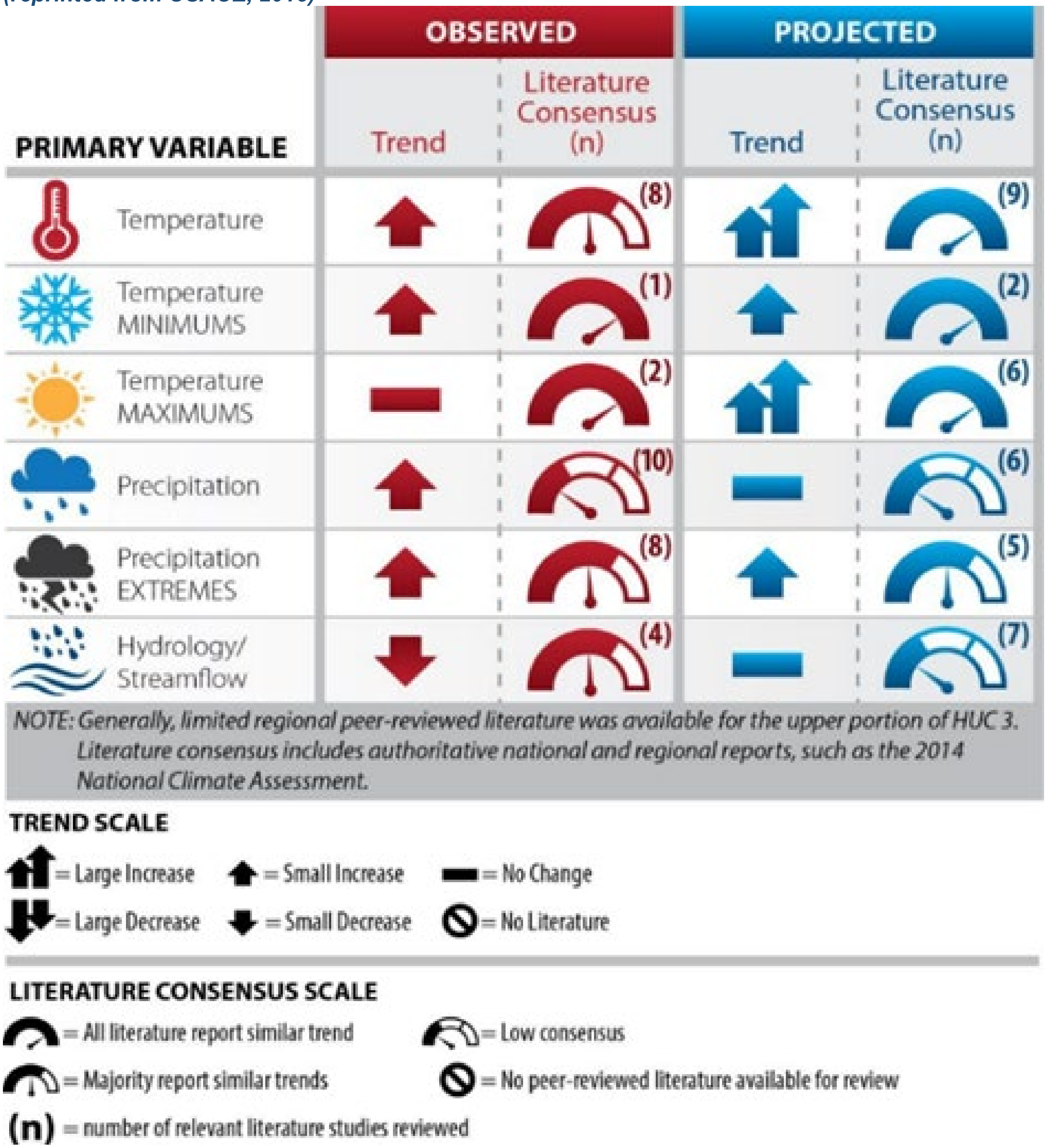

### *A.2.2.3.2. Projected Streamflow*

Review of projected hydrology for the southeast region show that there is very low consensus in projected changes. This is due to the additional uncertainties that are added when coupling climate models to hydrologic models, both of which carry their own uncertainties. Overall, there are little indications of an increasing or decreasing trend in hydrology based on the reviewed literature presented in IWR report (USACE, 2015).

### *A.2.2.4. Summary*

**Figure A-21** shows the discussed variables and their overall consensus in trends for both observed and projected scenarios based on the findings of the 2015 USACE IWR literature synthesis. There is evidence that supports an increasing temperature trend from the observed data and less supporting evidence for trends in precipitation or streamflow for a majority of the region. However, there is some evidence that precipitation is increasing, while streamflow appears to be decreasing in some areas within the region.

***Figure A-21: Summary matrix of observed and projected climate trends and literary consensus (reprinted from USACE, 2015)***



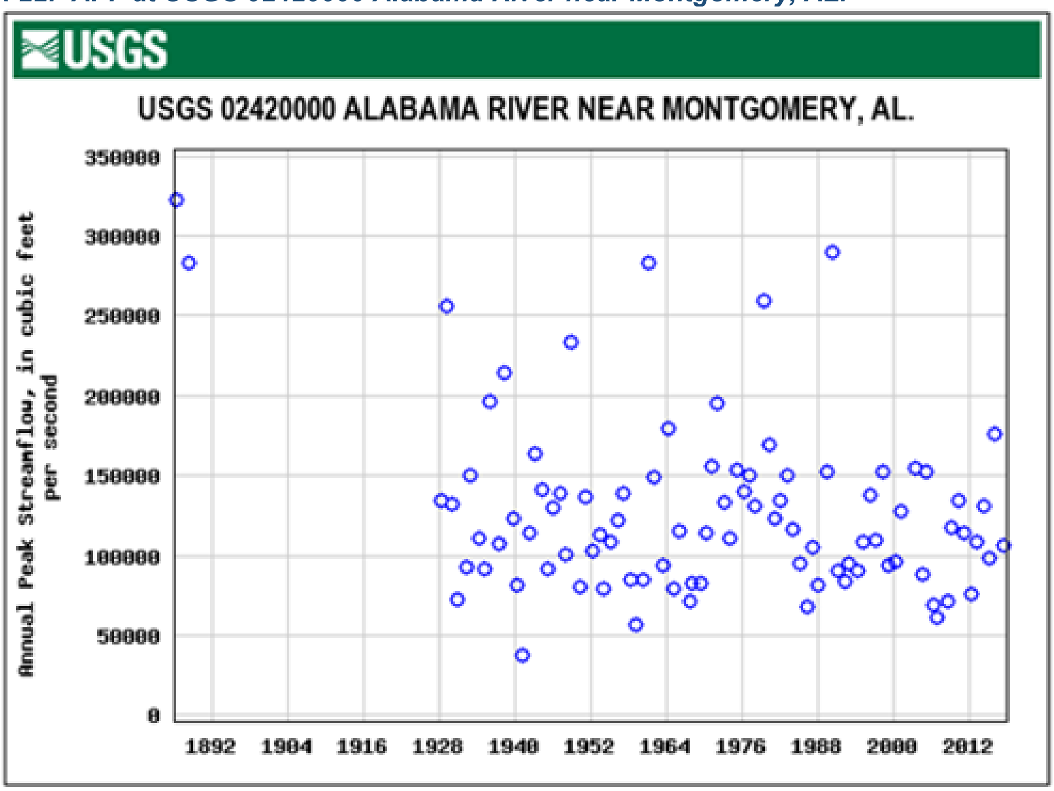

Projections indicate a strong consensus of an increase in projected temperature of approximately 2 to 4 degrees Celsius by the late 21st century. There is some consensus that precipitation extremes may increase in the future, both in terms of intensity and frequency. However, in general, projections of precipitation have been shown to be highly variable across the region. There is not a consensus regarding the directionality of trends in observed streamflow. Very few conclusions can be drawn regarding future hydrology in the region largely due to the substantial amount of uncertainly in these projections when coupling climate models with hydrology models.

### *A.2.3. Non-Stationarity Assessment*

In accordance with ECB 2018-14, a stationarity analysis was performed to determine if there are long-term changes in peak streamflow statistics within the study area and its vicinity. Assessing trends in peak streamflow is considered appropriate as opposed to a focus on precipitation and temperature as one of the primary purposes of this feasibility study is to assess and reduce flooding in the study area. However, trends in these should also be considered as they are both drivers in hydrology.

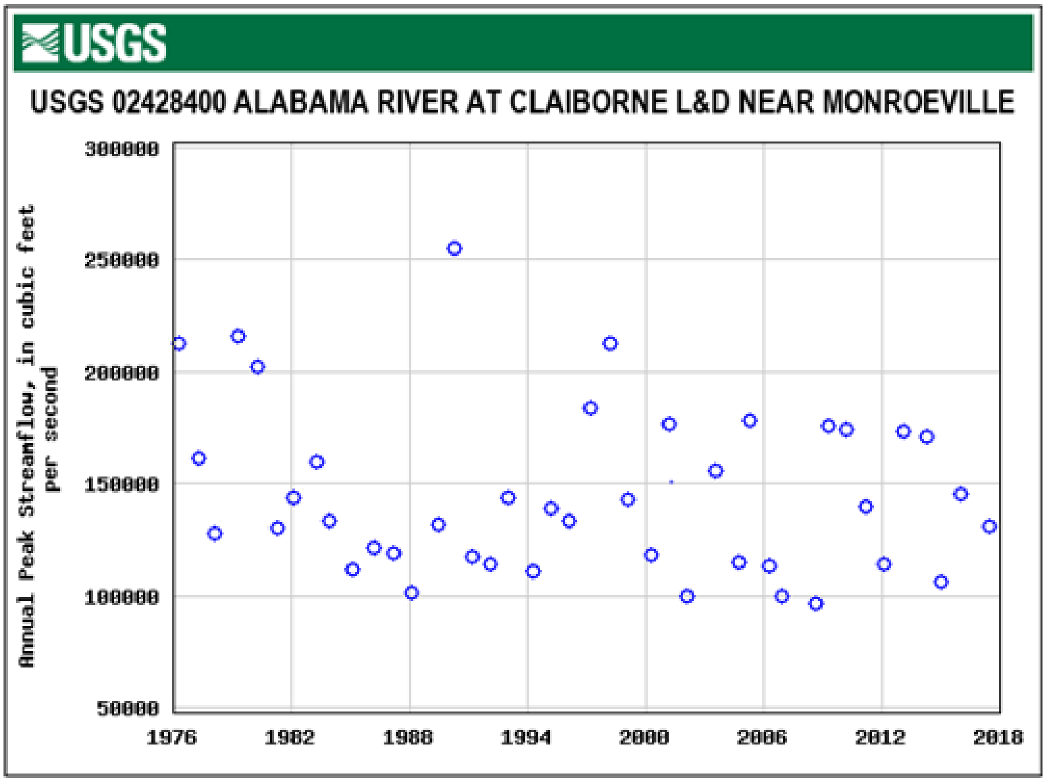
The USACE Non-Stationarity Tool was used to assess possible trends and change points in peak streamflow in the region. USGS 02420000 and USGS 0228400 were used for this analysis. The first gage used in this analysis, USGS 02420000, is located 83 miles upstream of Selma on the Alabama River near Montgomery, AL. The gage has a long and nearly continuous record from 1928-2018, includes two historical events, but is missing one year (2003). **Figure A-22** shows the time series of Annual Peak Streamflow (APF) for the gage located near Montgomery, AL.

*Figure A-22: APF at USGS 02420000 Alabama River near Montgomery, AL.*




The second gage used in this analysis was located at Claiborne Lock and Dam, which is located approximately 79 miles downstream from Selma. This gage has a continuous record from 1976 to present. **Figure A-23** shows the time series of APF for the gage located at Claiborne Lock and Dam. To run the non-stationarity tool, it is recommended to have at least 30 continuous years of record. Both of these gages meet that requirement.

*Figure A-23: APF at USGS 02428400 Alabama River at Claiborne L&D near Monroeville*

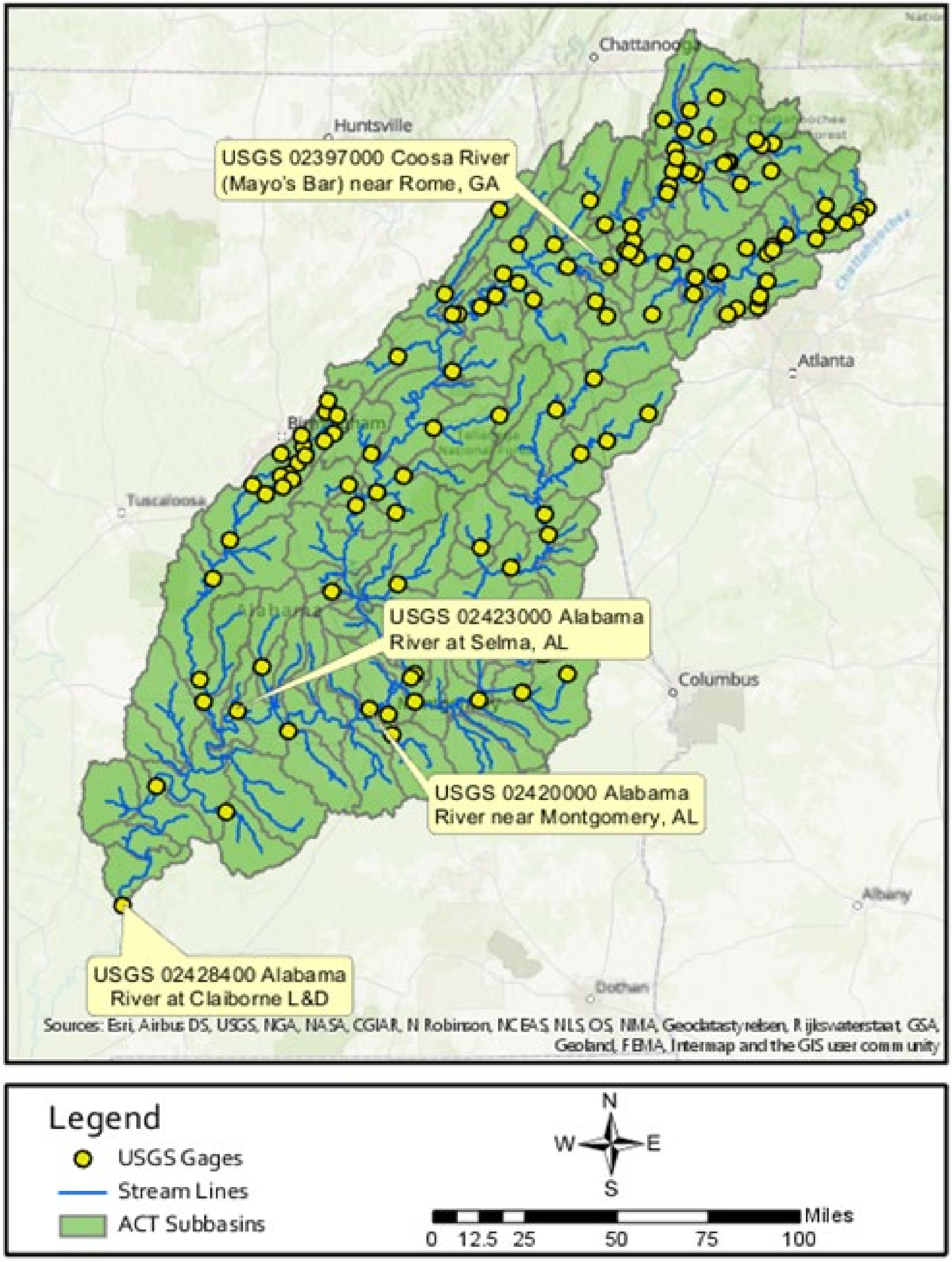



In **Figure A-24** the green area encompasses the entire drainage area delineated from Claiborne Lock and Dam and shows the location of the Selma, Alabama gage relative to the two gages used for this analysis.

The following 16 statistical tests were conducted on the APF time series shown on **Figure A-22** and **Figure A-23** using the Non-Stationarity Tool:

1. Cramer-von-Mises distribution
2. Kolmogorov-Smirnov distribution
3. LePage distribution
4. Energy Divisive distribution
5. Lombard (Wilcoxon) abrupt mean
6. Pettitt mean
7. Mann-Whitney mean
8. Bayesian mean
9. Lombard (Mood) abrupt variance
10. Mood variance
11. Lombard (Wilcoxon) smooth mean
12. Lombard (Mood) smooth variance
13. Mann-Kendall trend
14. Spearman rank trend
15. Parametric trend
16. Sen's slope trend

***Figure A-24: Study area and locations of the Montgomery, AL gage, Claiborne Lock and Dam gage, Selma, AL gage, and Rome, GA gage used in this analysis***




Tests 1-12 are used to detect change points in the distribution, mean, and/or variance of the time series. These non-stationarity tests can be useful in detecting changes in annual instantaneous streamflow peaks driven by natural and human driven changes in the climate, addition/removal of water control structures, changes in land cover, and any other drivers of non-stationarity. Meanwhile, tests 13-16 are used to analyze monotonic trends. The variety of tests is essential for increasing confidence in the overall stationarity analysis. Significant findings in one or two tests are generally not enough to declare non-stationarity.

For this analysis, the continuous period of water years 1976-2014 for the gage located at Claiborne Lock and Dam and water years 1928-2002 for the gage located near Montgomery, AL were used. All sensitivity parameters were left in their default positions. For both gages, there were no non-stationarities detected, as seen on **Figure A-25** and **Figure A-26**. The Alabama River is a regulated system with multiple run-of-river projects and flood control projects. This may be the reason why non-stationarities were not detected. The monotonic trend test indicates that there are no trends for the entire record (not including historical peaks) for both gages, **Figure A-27** and **Figure A-28**.

USGS water year summaries were checked and do not reveal any information that would indicate gage errors or issue with flow recording. For the gage located near Montgomery, AL, the two extremes recorded prior to the period of record were estimated based on high water marks and an extended rating curve. These two extremes were excluded from the non-stationarity analysis.

*Figure A-25: Non-Stationarity Tool result for USGS 2420000 located near Montgomery, Alabama*

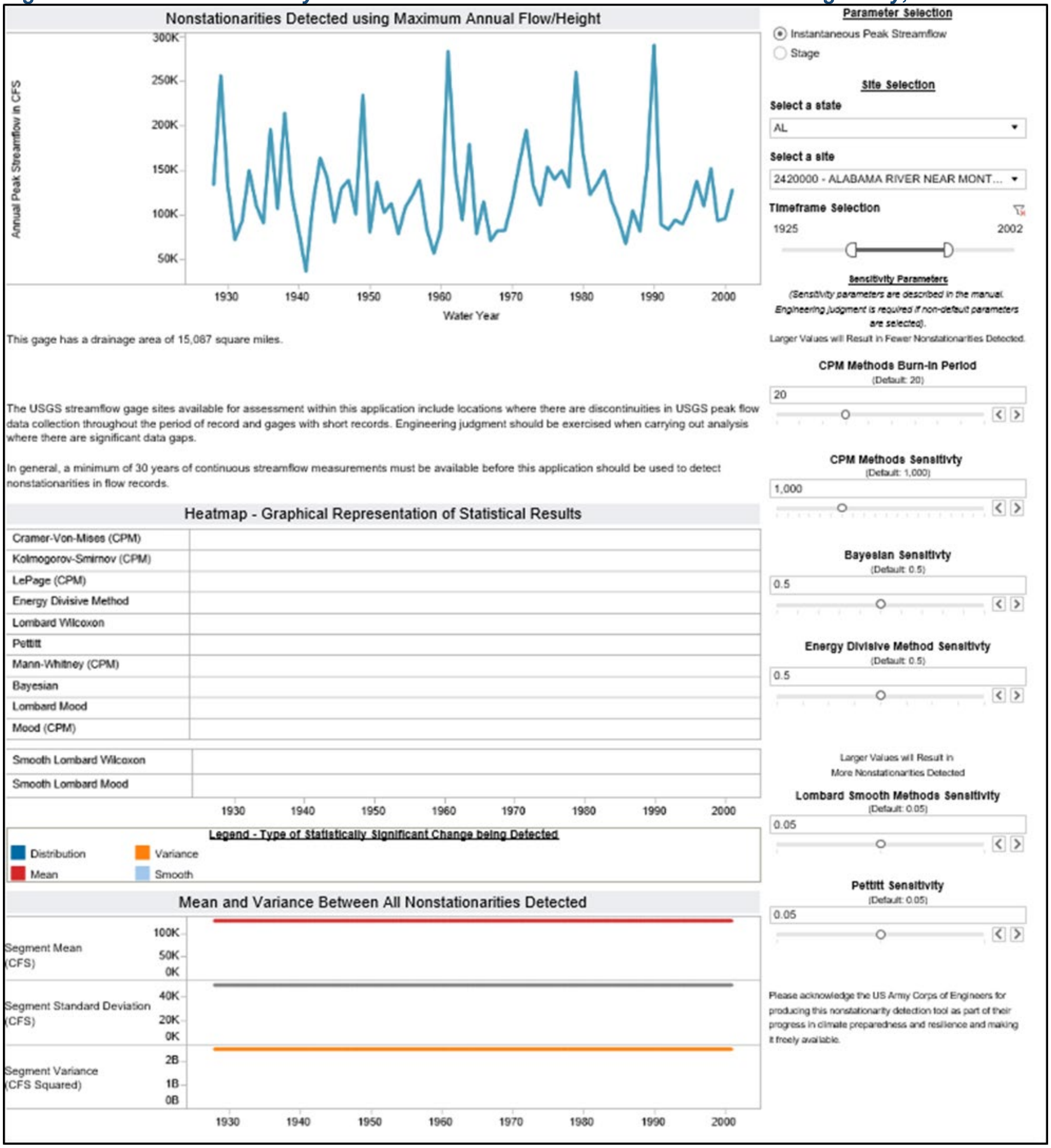


*Figure A-26: Non-Stationarity Tool result for USGS 2428400 located at Claiborne Lock and Dam*

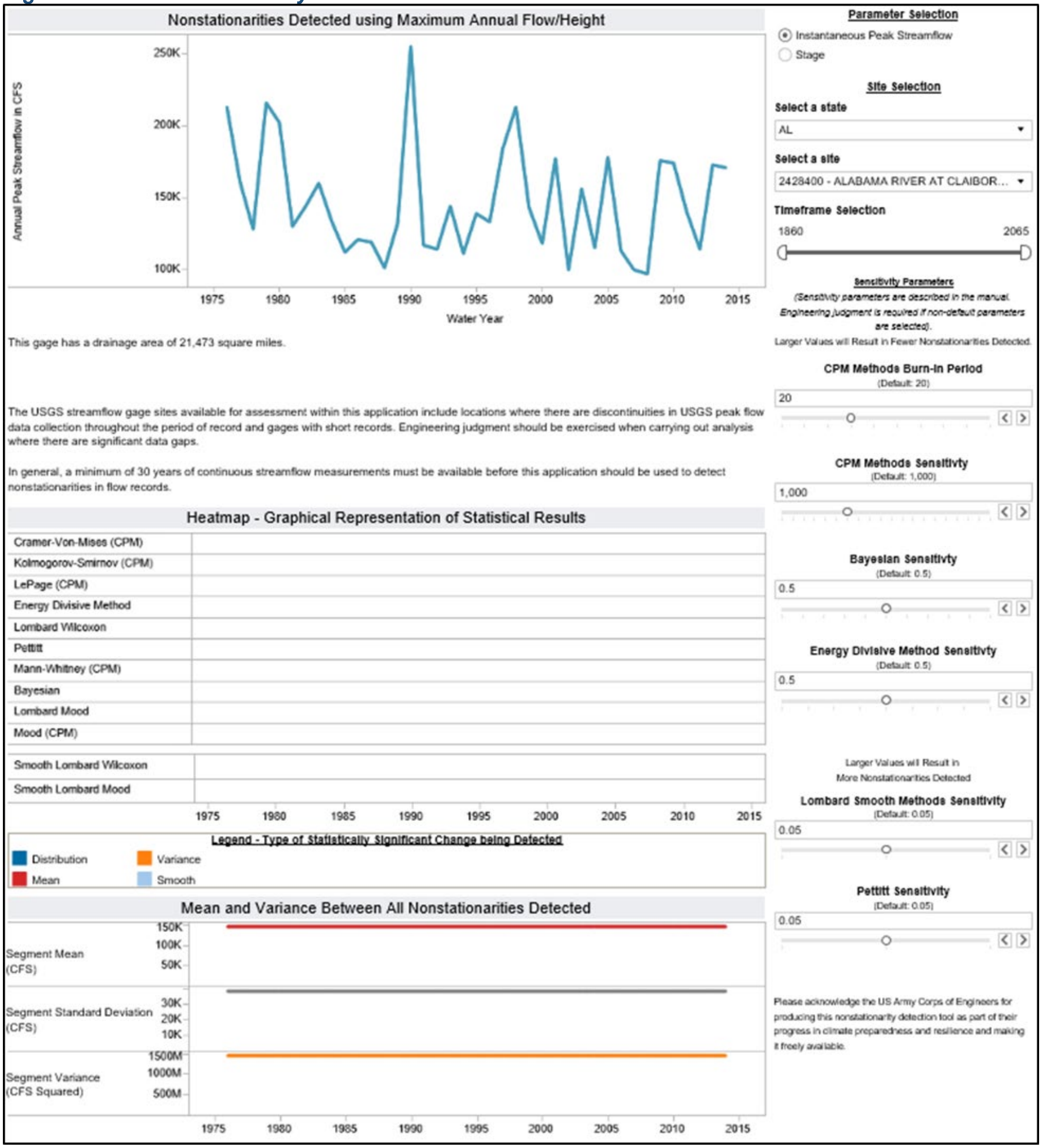
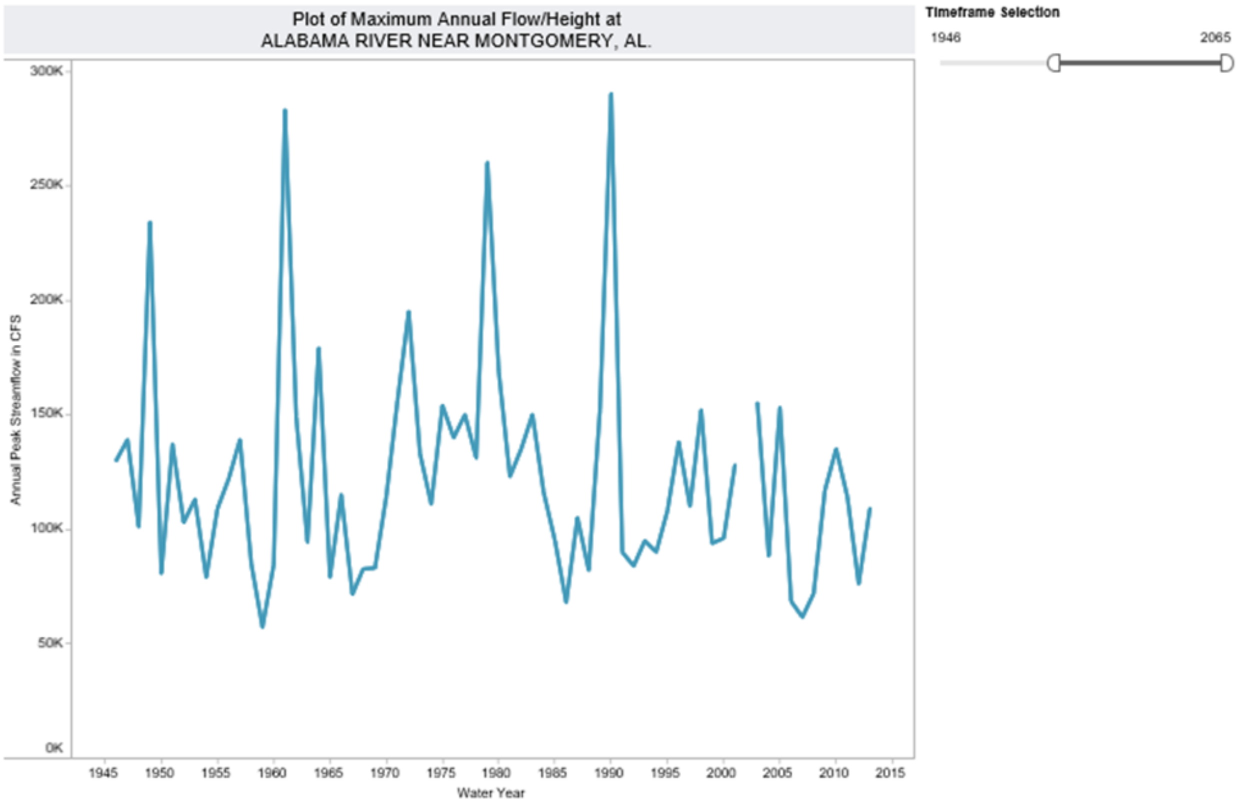


***Figure A-27: Monotonic trend analysis for USGS 2420000 located near Montgomery, Alabama***




*WARNING: The period of record selected has missing data points. There are potential issues with the trends detected.*

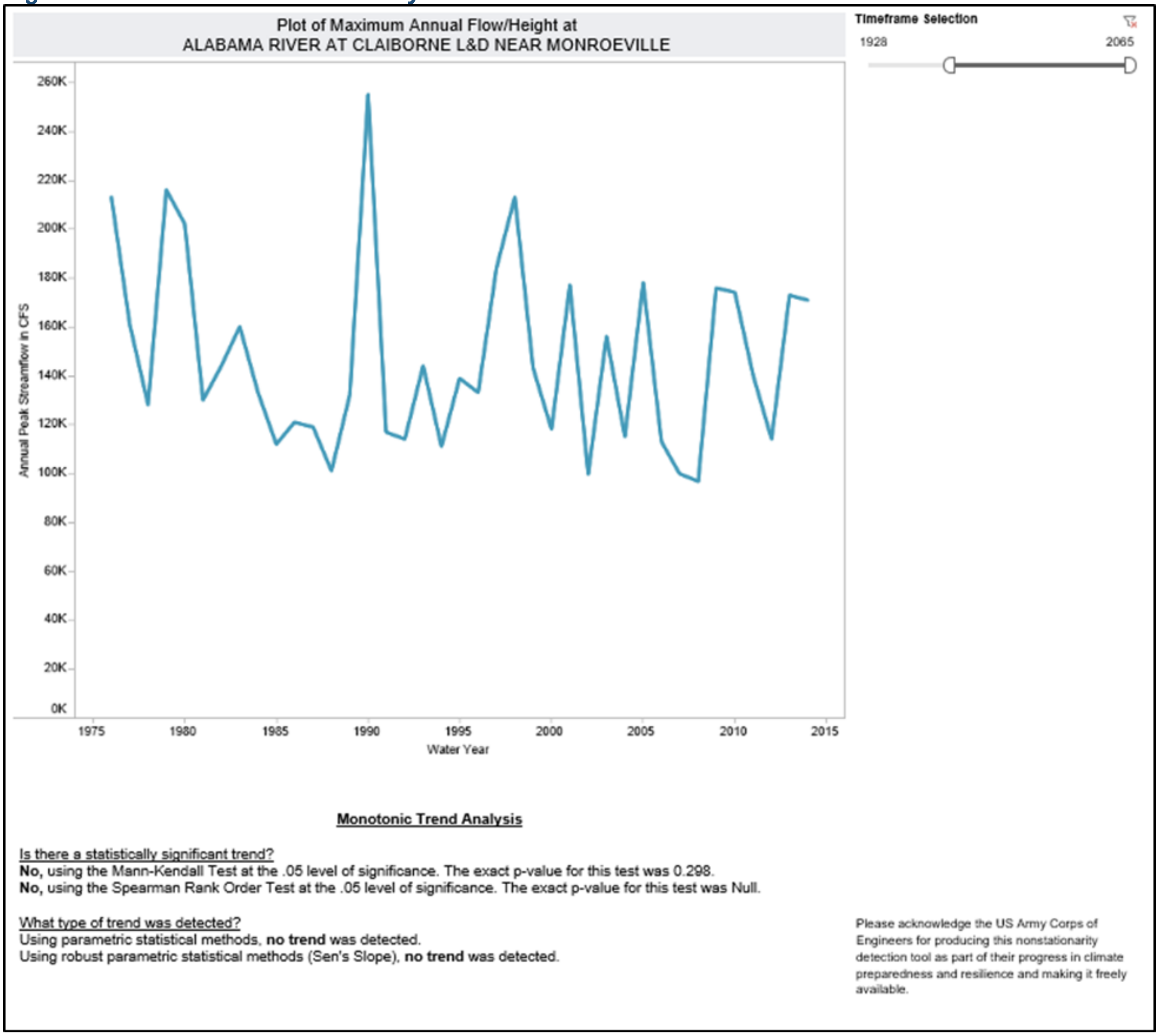
**Monotonic Trend Analysis**

Is there a statistically significant trend?
**No,** using the Mann-Kendall Test at the .05 level of significance. The exact p-value for this test was 0.458.
**No,** using the Spearman Rank Order Test at the .05 level of significance. The exact p-value for this test was 0.432.

What type of trend was detected?
Using parametric statistical methods, **no trend** was detected.
Using robust parametric statistical methods (Sen's Slope), **no trend** was detected.

Please acknowledge the US Army Corps of Engineers for producing this nonstationarity detection tool as part of their progress in climate preparedness and resilience and making it freely available.

*Figure A-28: Monotonic trend analysis for USGS 2428400 located at Claiborne Lock and Dam*




### *A.2.4. Climate Hydrology Assessment Tool*

In addition to the stationarity assessment, the USACE Climate Hydrology Assessment Tool (CHAT) was used to assist in the determination of future streamflow conditions. For this assessment, three gages were analyzed within the Alabama Basin. **Figure A-29** shows the CHAT output for USGS 02428400 located at Claiborne Lock and Dam and **Figure A-30** shows the CHAT output for USGS 02420000 located near Montgomery, AL. The p-values for these gages are 0.380259 and 0.275589, respectively. Neither of them are considered statistically significant. For USGS 02397000 Coosa River near Roma, GA, the p-value is 0.0006056 (**Figure A-31**). This indicates that this downward trend is statistically significant. However, this gage is farther upstream from the study area compared to the other two gages, which are within 100 miles upstream and downstream of the Selma area. The decrease in streamflow at this gage most likely is due to the flood control projects built upstream of the gage, which was discussed in the streamflow section above

*Figure A-29: CHAT output for USGS 02428400 Alabama River at Claiborne Lock and Dam*

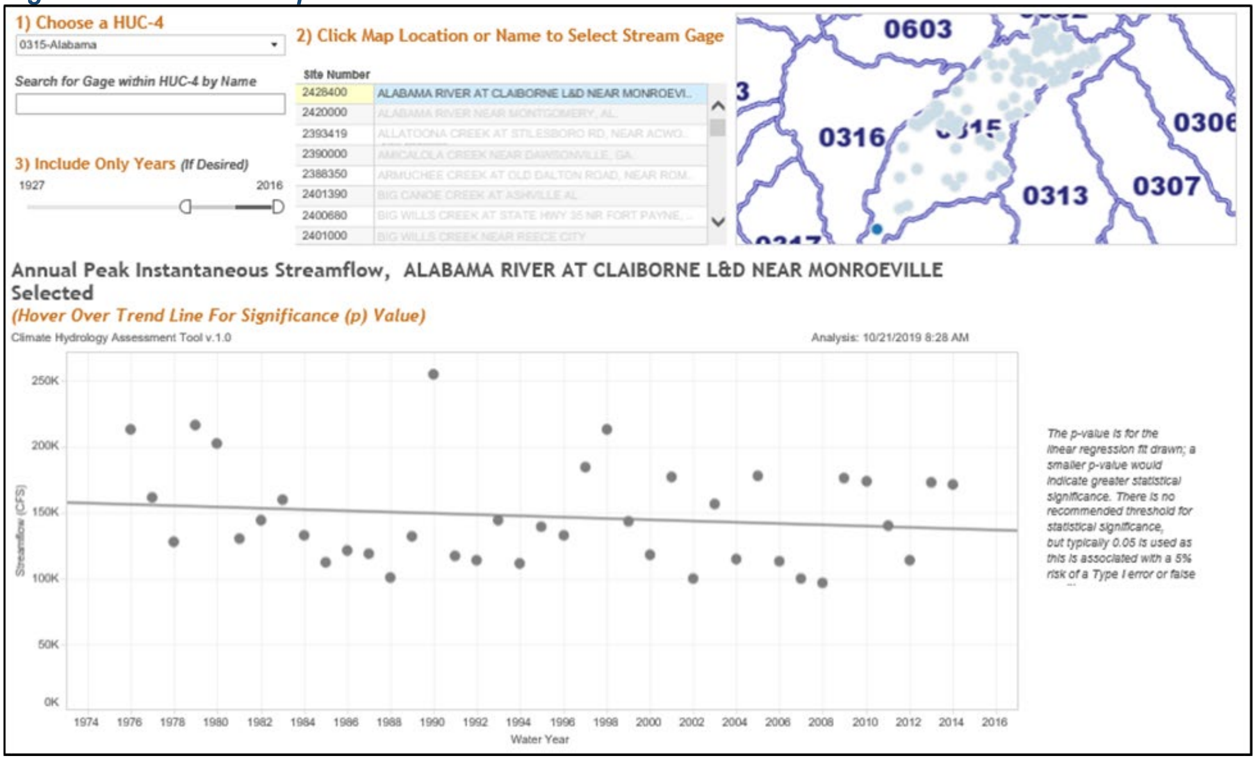



*Figure A-30: CHAT output for USGS 02420000 Alabama River near Montgomery, Alabama*

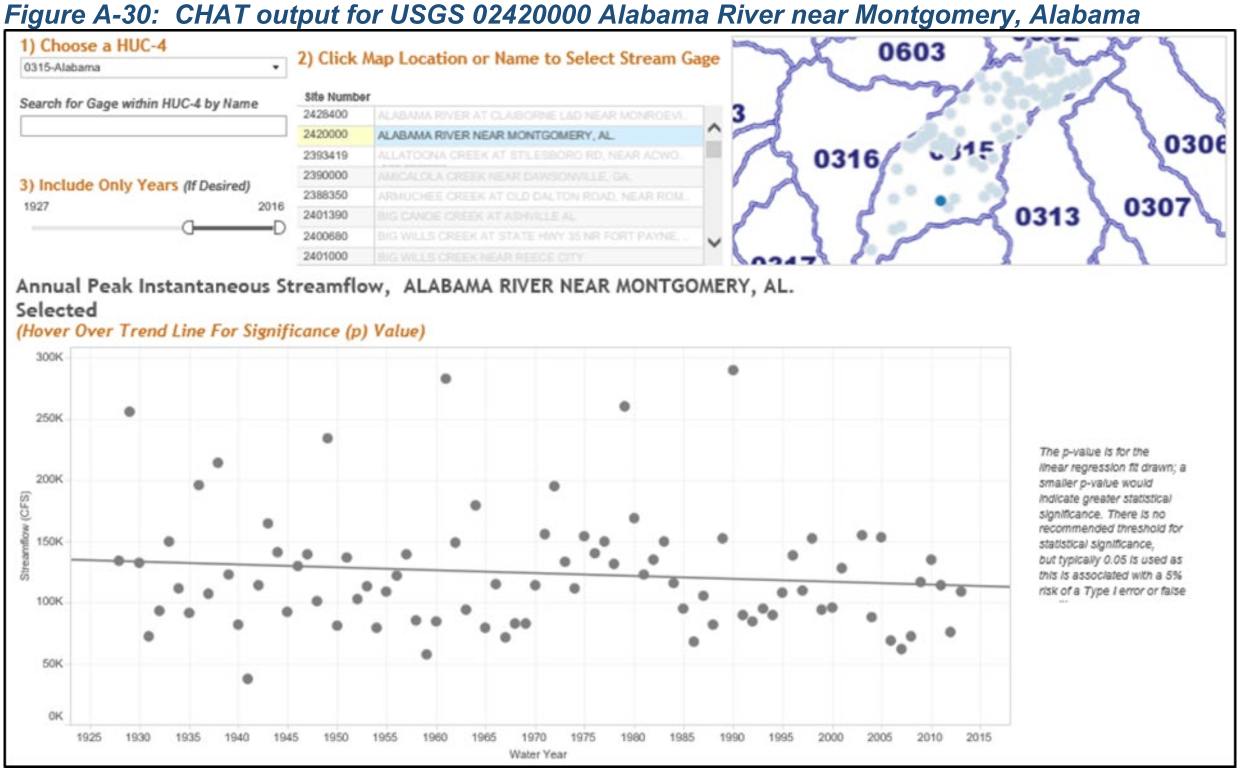
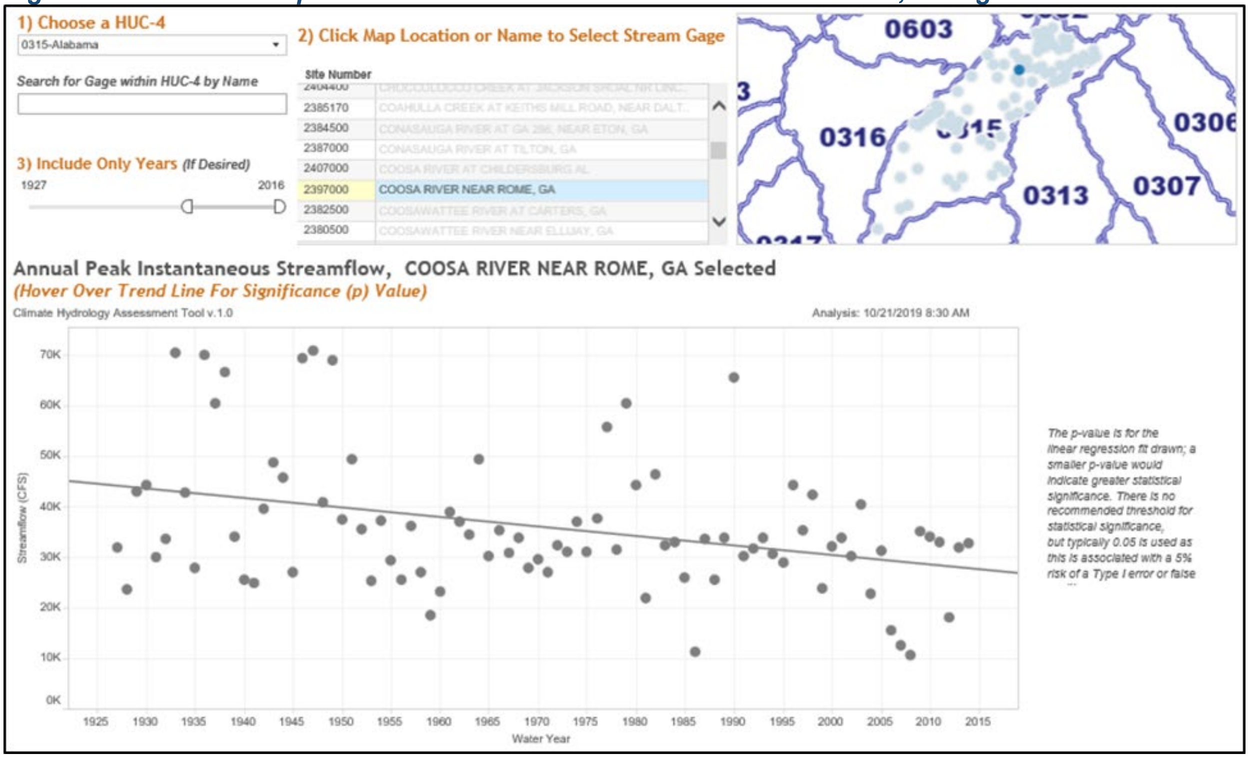


*Figure A-31: CHAT output for USGS 02397000 Coosa River near Roma, Georgia*




A Hydrologic Unit Code 4 (HUC-4) level analysis of mean projected annual maximum monthly streamflow was also performed. The trends in mean projected annual maximum monthly streamflow presented in this analysis represent outputs from the Global Climate Models (GCMs) using different representative concentration pathways (RCPs) of greenhouse gasses that are then translated into a hydrologic response using the United States Bureau of Reclamation (USBR) Variable Infiltration Capacity (VIC) model. The VIC model, forced with GCM meteorological outputs is used to produce a streamflow response for both the hindcast period (1950-1999) and the future period (2000-2099). This dataset is unregulated and does not account for the many flood control structures located on the mainstem rivers within this HUC-4 basin.

The analysis indicates an upward trend in mean projected annual maximum monthly streamflow for the Alabama Basin, as shown in **Figure A-32**. The forecast visually indicates an upward trend in projected streamflow from years 2000 to 2099 within the basin and is considered statistically significant with a p-value of 0.01442. The hindcast data shows no statistically significant trend from 1950 to 1999 (p-value: 0.795219).

*Figure A-32: Mean projected annual maximum monthly streamflow for the Alabama HUC-4*

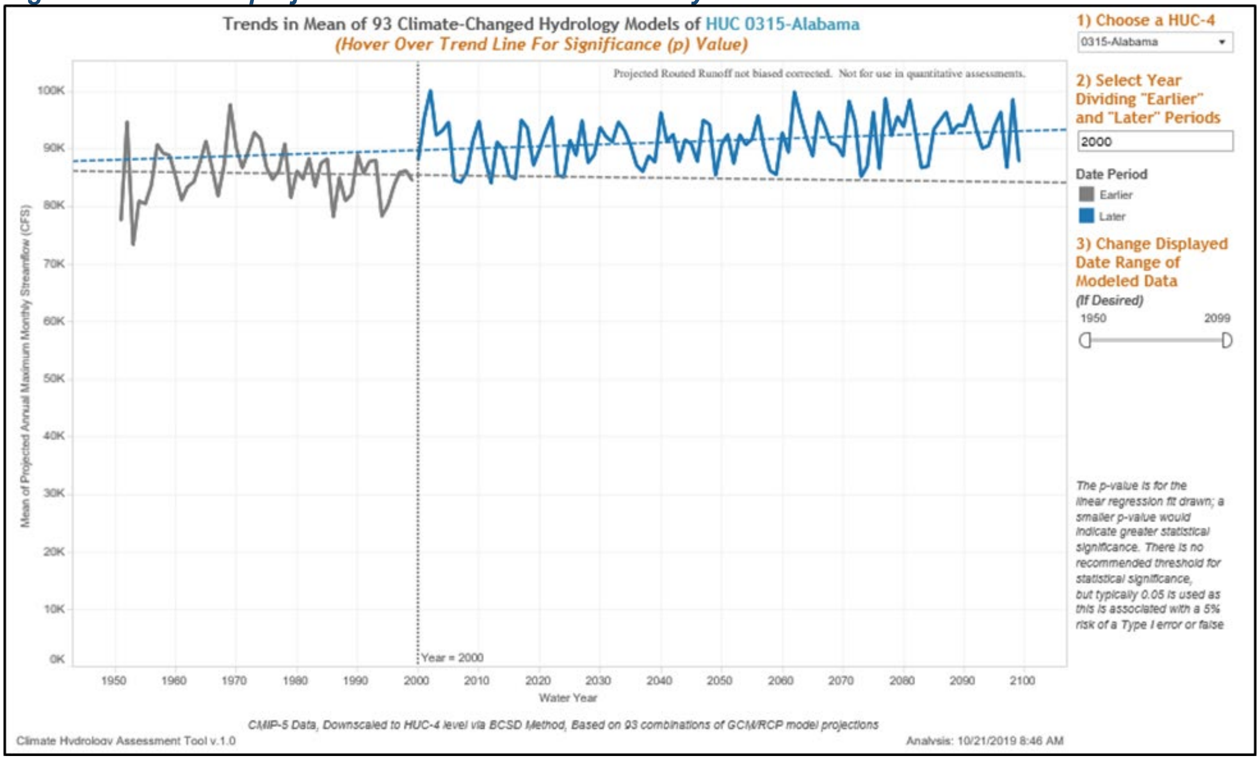
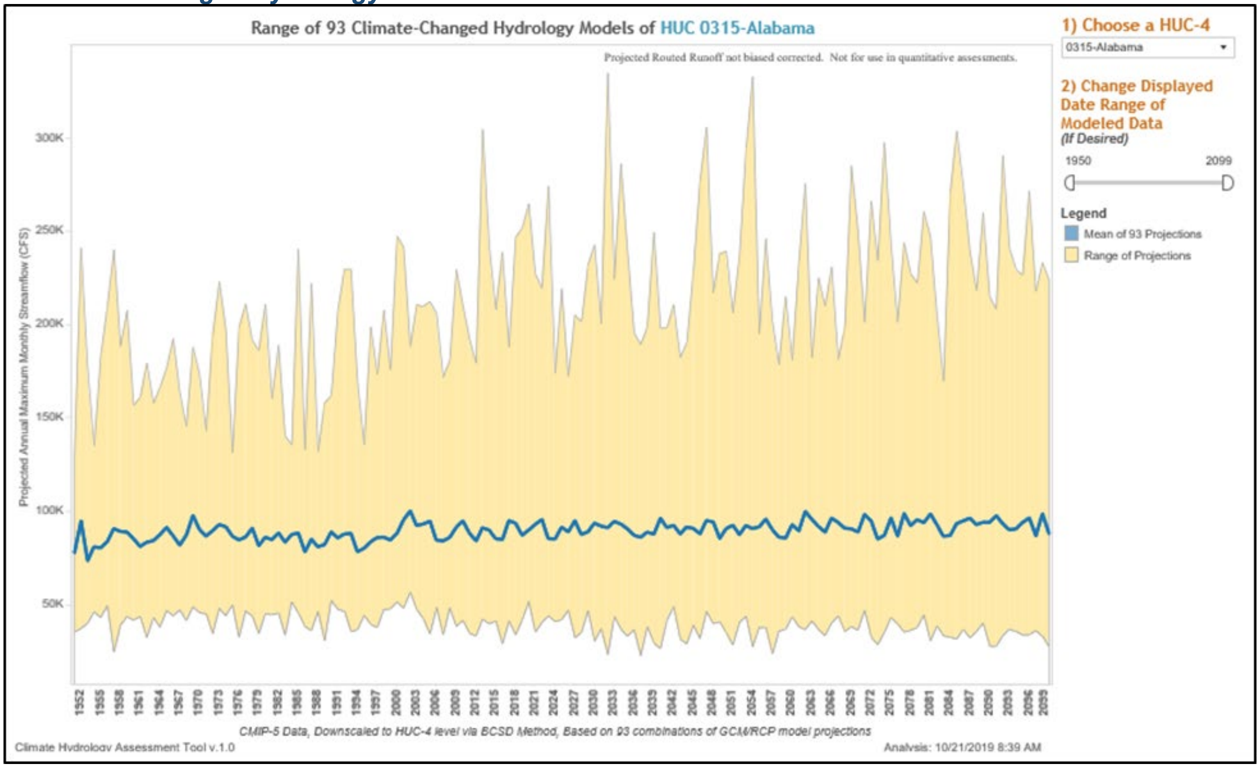



**Figure A-33** provides the mean value of the 93 projections of future, streamflow projections considered through water year 2099, as well as the range of projected streamflow values produced for the watershed. The variability of the spread is fairly consistent for the projected portion of the record: 2000 to 2099.

It can be seen on **Figure A-33** that there is significant uncertainty in projections of future streamflow. The yellow shaded area is indicative of the spread in the data produced. It is important to understand that this uncertainty comes from each of the model sources that are used to develop the projected streamflow datasets. GCMs have uncertainty in the bounds of their atmospheric input such as the RCPs. Downscaling the output of these models to a smaller region may not account for some regional effects.

Changes in future conditions that drive the hydrologic model are also a major source of uncertainty. An example of this uncertainty is land use changes, such as increased impervious areas, which can have a major effect on peak streamflow. There are many different land use projections for this region from many sources. Other uncertainties such as changes in temperature extremes and the seasonality of the extreme precipitation can also have a significant effect on the rainfall/runoff transformation. For these reasons, this quantitative analysis should be used with caution, with an understanding that this data should only be considered within the large uncertainly bounds of the analysis.

***Figure A-33: Projected hydrology for the Alabama HUC-4 based on the output from 93 projections of climate-changed hydrology***




### *A.2.5. Vulnerability Assessment*

To understand potential climate change effects and to increase resilience/decrease vulnerability of flood risk management alternatives to climate change, the relative vulnerability of the basin to such factors was analyzed. In accordance with ECB 2018-14, the USACE Watershed Climate Vulnerability Assessment tool was used to identify vulnerabilities to climate change on a HUC-4 watershed scale relative to other HUC-4 basins across the nation. As this study is an assessment of flood risk management alternatives, vulnerability with respect to the Flood Risk Reduction business line is presented in this analysis.

To address vulnerabilities due to climate change, the Vulnerability Assessment tool utilizes two 30-year epochs centered on 2050 (2035-2064) and 2085 (2070-2099) as well as a base epoch. These epochs, while arbitrary, line up well with other national climate change assessments. For each epoch, the tool utilizes the results of 100 combinations of Global Circulation/Climate Models (GCM) run using different Representative Concentration Pathways of greenhouse gas emission to produce 100 traces per epoch for a given watershed. The results of the GCMs are translated into flow and are then sorted by cumulative runoff projections. Traces of the highest 50% of cumulative runoff are categorized as wet and traces with the lowest 50% of cumulative runoff are categorized as dry. This provides two scenarios (wet and dry) for each of the two epochs, excluding the base epoch. Consideration of both wet and dry scenarios reveals some of the uncertainties associated with the results produced using the climate-changed hydrology and meteorology used as inputs to the vulnerability tool.

The tool uses specific indicators of vulnerability relative to the business line being considered. There is a total of 27 indicators in the tool, 5 of which are used to derive the vulnerability score in the Alabama HUC 4 with respect to the Flood Damage Reduction business line. **Table A-3** lists the indicators and their descriptions.

*Table A-3: Indicator Variables used to derive the flood risk management Vulnerability score for the Alabama Basin as determined by the Vulnerability Assessment tool*

| Indicator Short Name | Indicator Full Name | Description |
|---|---|---|
| **175C_Annual_COV** | Annual CV of unregulated runoff (cumulative) | Long term variability in hydrology: ratio of the standard deviation of annual runoff to the annual runoff mean. Includes upstream freshwater inputs (cumulative). |
| **277_RUNOFF_PRECIP** | % change in runoff divided by % change in precipitation | Median of: deviation of runoff from monthly mean times average monthly runoff divided by deviation of precipitation from monthly mean times average monthly precipitation. |
| **568L_FLOOD_MAGNIFICATION** | Flood magnification factor (local) | Change in flood runoff: Ratio of indicator 571L (monthly runoff exceeded 10% of the time, excluding upstream freshwater inputs) to 571L in base period. |
| **568C_FLOOD_MAGNIFICATION** | Flood magnification factor (cumulative) | Change in flood runoff: ratio of indicator 571C (monthly runoff exceeded 1-% of the time, including upstream freshwater inputs) to 571C in base period. |
| **590_URBAN)500YRFLOODPLAIN** | Acres of urban area within 500-year floodplain | Acres of urban area within the 500-year floodplain. |

**Figure A-34** and **Figure A-35** shows a comparison of WOWA scores for the flood risk reduction business line for HUC-4 watersheds nationally, and for the South Atlantic

Division only, for the wet and dry scenarios as well as the 2050 and 2085 epochs, respectively. This shows that the WOWA score for the Alabama HUC-4 Basin (highlighted in yellow) is not relatively vulnerable to climate change impacts for the flood risk management business line. Within the wet subset of traces for the South Atlantic Division, there are only two HUC04 watersheds for both epochs. For the dry subset of traces, there are only three HUC04 watersheds that are considered relatively vulnerable to climate change for the Flood Risk Reduction business line. All three watersheds in question are in Florida. This further reinforces that the Alabama basin is does not have significant vulnerabilities to the Flood Risk Reduction business line with respect to other watersheds in the United States, or the region.

***Figure A-34: Comparison of national vulnerability scores for CONUS HUC-4s***

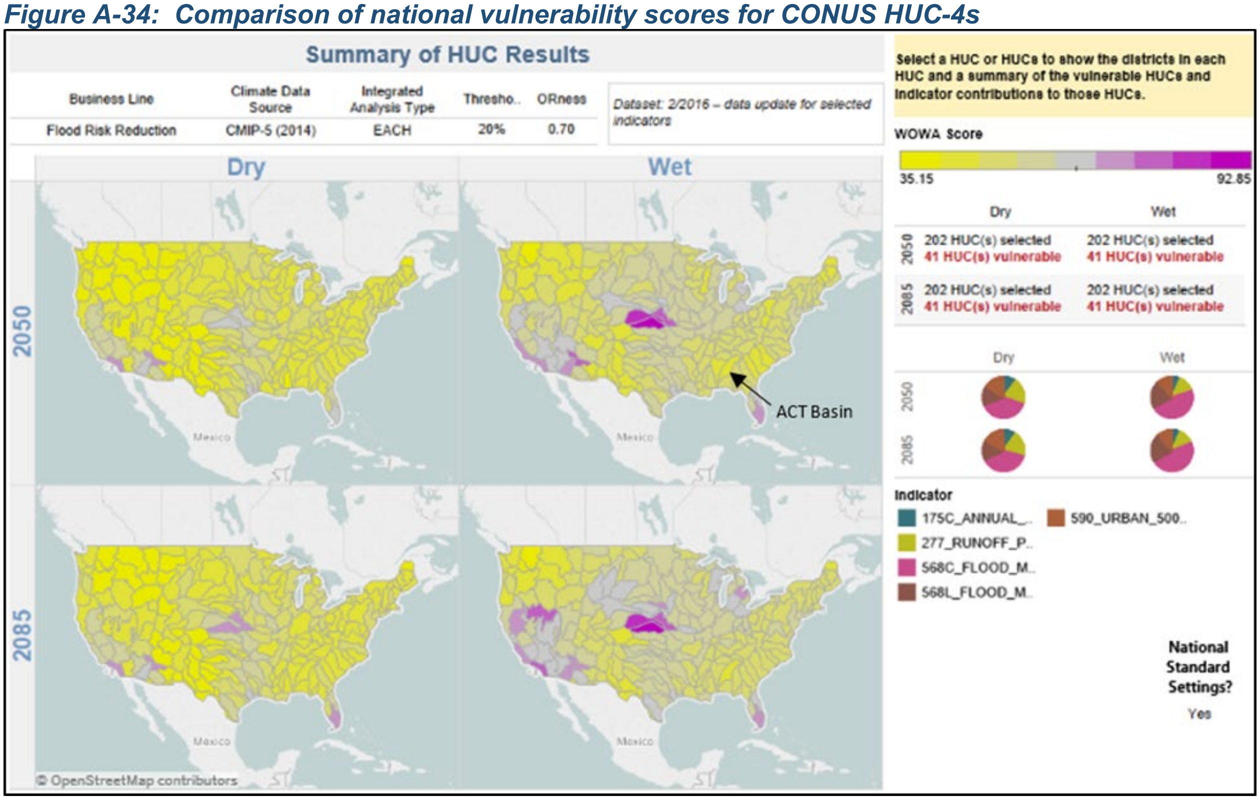
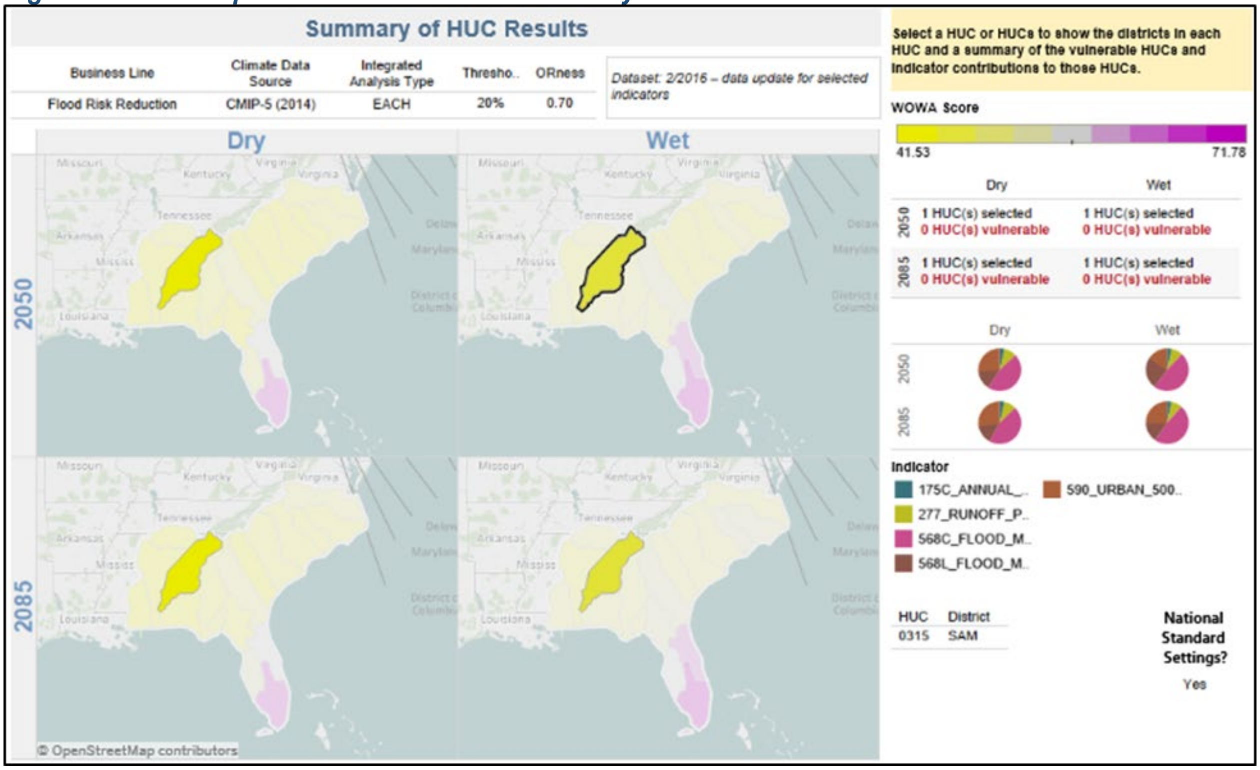


*Figure A-35: Comparison of national vulnerability scores for South Atlantic Division HUC-4s*




It is important to note that the vulnerability assessment only indicates vulnerability relative to the rest of the nation. It does not state that the basin itself is invulnerable to impacts of climate change on the Flood Risk Reduction business line. The assessment only concludes that it is not in the top 20% of vulnerable basins based on WOWA scores. There are locally significant impacts relative to climate change driven by many different factors. Therefore, it is beneficial to understand the composition of the relevant HUC 04's (Alabama Basin) vulnerability score, in terms of how much each flood risk reduction indicator variable contributes to the vulnerability score for each subset of traces, and for both epochs of time. **Figure A-36** and **Figure A-37** below show the dominant indicators relative to flood risk reduction and that cumulative flood magnification is the prevailing indicator variable driving the flood damage reduction vulnerability score, followed by local flood magnification for both the dry and wet scenarios, respectively. This aligns with the literature review that indicates the potential for more frequent and more severe storms in the southeast.

***Figure A-36: Dominate indicators for the Flood Risk Reduction Business Line for the Dry Scenario***

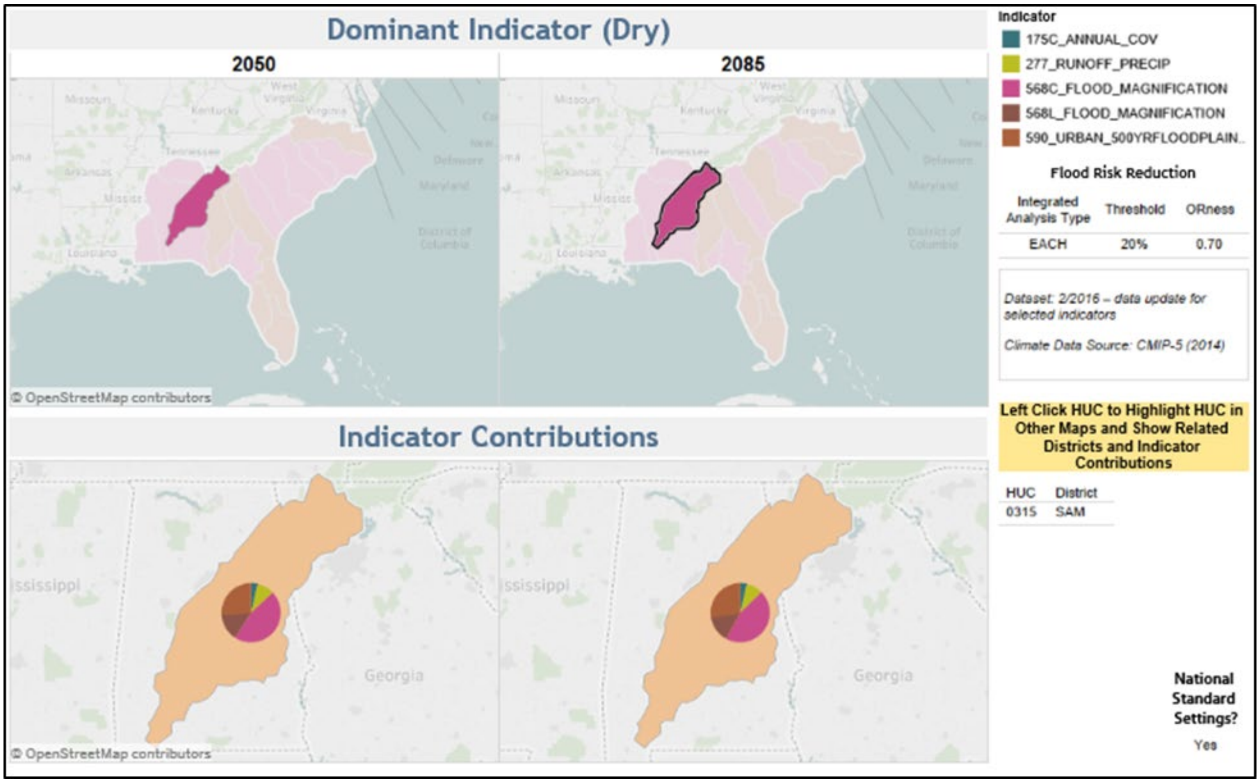
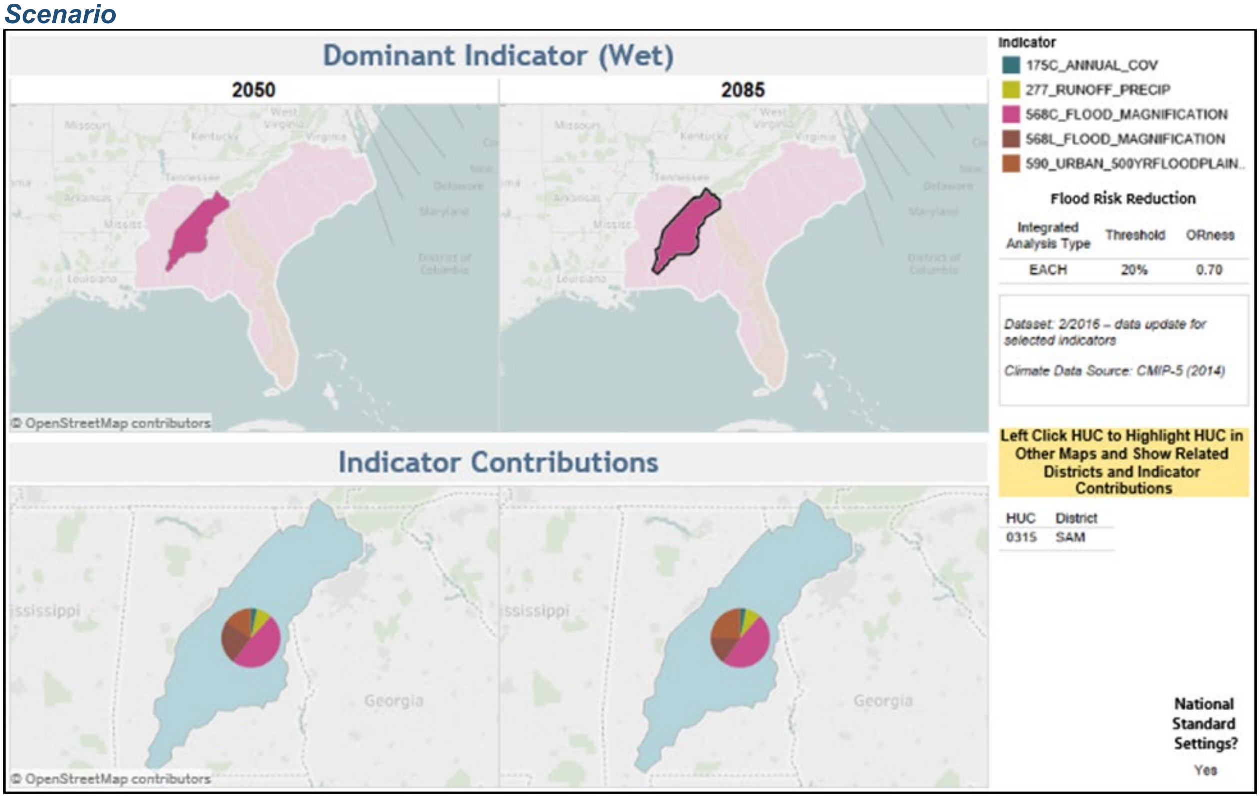



***Figure A-37: Dominate indicators for the Flood Risk Reduction Business Line for the Wet Scenario***



### *A.2.6. Climate Change and Impacts on Recommended Plan*

The Recommended Plan for this study includes a solider pile wall to protect and stabilize the streambank in downtown Selma, and a Flood Response Plan for the city.

*Table A-4: Risk assessment results of each measure in the Recommended Plan*

| Feature or Measure | Trigger | Hazard | Harm | Qualitative Likelihood |
|---|---|---|---|---|
| **Bank Stabilization-Solider Pile Wall** | Increase in frequency and magnitude of extreme storms | Peak elevations during floods could increase | Damage to soldier pile wall and the foundations of structures behind the wall | Highly Unlikely |
| **Flood Response Plan** | Increase in frequency and magnitude of | Peak elevations during floods could increase | Areas previously unaffected by flooding become inundated, affecting the plan | Highly Unlikely |

An increase in the magnitude of extreme storms could cause the peak elevations of floods to increase for the same frequency storm. This hazard however is very unlikely to lead to damage of the solider pile wall, or any negative effect. The wall is being designed for overtopping and submergence. An increase in flood depth would have no effect on the performance or integrity of the wall. Therefore, it can be said that it is highly unlikely that there would be a negative effect on this measure.

When considering this same trigger and hazard applied to the Flood Response Plan there is the possibility that areas previously unaffected by flooding become inundated. This however will not lead to the plan not accounting for any flooding based on an increase in flow. This is because the plan will be tied to certain elevations near the city of Selma based on forecast gage locations, and not a flow-frequency event. If flows are to increase on the Alabama River, stages will increase as well; however, the inundation for a stage or elevation will not change. Therefore, the plan will still be applicable as hydrology changes.

### *A.2.7. Conclusions*

Based on the literature review of relevant climate data, there is some consensus that there will be mild increases in the severity and frequency of storms in the region. However, there is no consensus on future changes in hydrology. Observed data from gages near the study area show temperatures have been gradually rising since the 1970s, after a cooling period in the middle part of the century. From these data, it is difficult to come to a conclusion on whether temperature is increasing, or if this is a reoccurring pattern. Annual precipitation seems to be variable for the region. It appears there may be more extremes occurring in recent years, such as extreme low annual precipitation values. However, the overall trends appear to be constant or increasing slightly. There is some consensus on peak streamflow for the region decreasing since the middle of the century, however, the literature lacks a clear consensus. For the Alabama Basin, this decreasing streamflow could be related to the increase in flood control projects within the region since the late 1940s.

The non-stationarity assessment on the Alabama River Basin was performed using two gages (USGS 02420000 Alabama River near Montgomery, AL and USGS 02428400 Alabama River at Claiborne Lock and Dam). Neither gage displayed non-stationarities, nor were monotonic trends detected. However, the USGS gage located near Rome, GA (Coosa River) displayed four non-stationarities, which occurred in the years 1951, 1952, 1983, and 2005. Non-stationarities in the years 1951 and 1952 can be attributed to projects, such as dams, built upstream of the gage. One of the largest projects built upstream was the Allatoona Dam, which was completed and began filling in December 1949. There appears to be a large drop in streamflow from the early 1980s to mid-1980s. This could have triggered a non-stationarity. Similarly, for the change point in 2005, there was a large decrease in streamflow. This may be the result of the 2005 drought that occurred in the northern part of the Alabama Basin.

The USACE CHAT tool indicates that there are no statistically significant trends in the two streamflow datasets for USGS 02420000 Alabama River near Montgomery, AL and USGS 02428400 Alabama River at Claiborne Lock and Dam. However, the CHAT tool was used to detect any changes in streamflow further upstream in the Alabama Basin at USGS 02397000 Coosa River near Roma, GA. The tool indicates that there is a statistically significant decrease in streamflow. This gage had several flood risk management dams built upstream since the 1940s, which most likely a key contributor to the decrease in flow. The further downstream, it appears that this significant trend is not as noticeable since this basin is large.

Furthermore, the HUC-4 analysis on streamflow on the Alabama basin only shows an increasing trend in projected streamflow based on GCM model output translated into a hydrologic response. These analyses provide some indication that there will be significant increases in peak annual streamflow in the future as a result of climate change. However, the projections seem to oppose the trend in observed flow. Caution should be used in making any definitive statements on potential future hydrology as there is substantial uncertainty in both the climate and hydrologic models that drive these analyses. The vulnerability assessment helps to further reinforce a lack of evidence in increasing flood risk. Findings of the vulnerability assessment show that the Alabama HUC-4 basin is not considered vulnerable to increased flood risk as a result of climate change, with respect to other HUC-4s in the nation.

## A.3. Existing Conditions - Hydrologic and Hydraulic Modeling

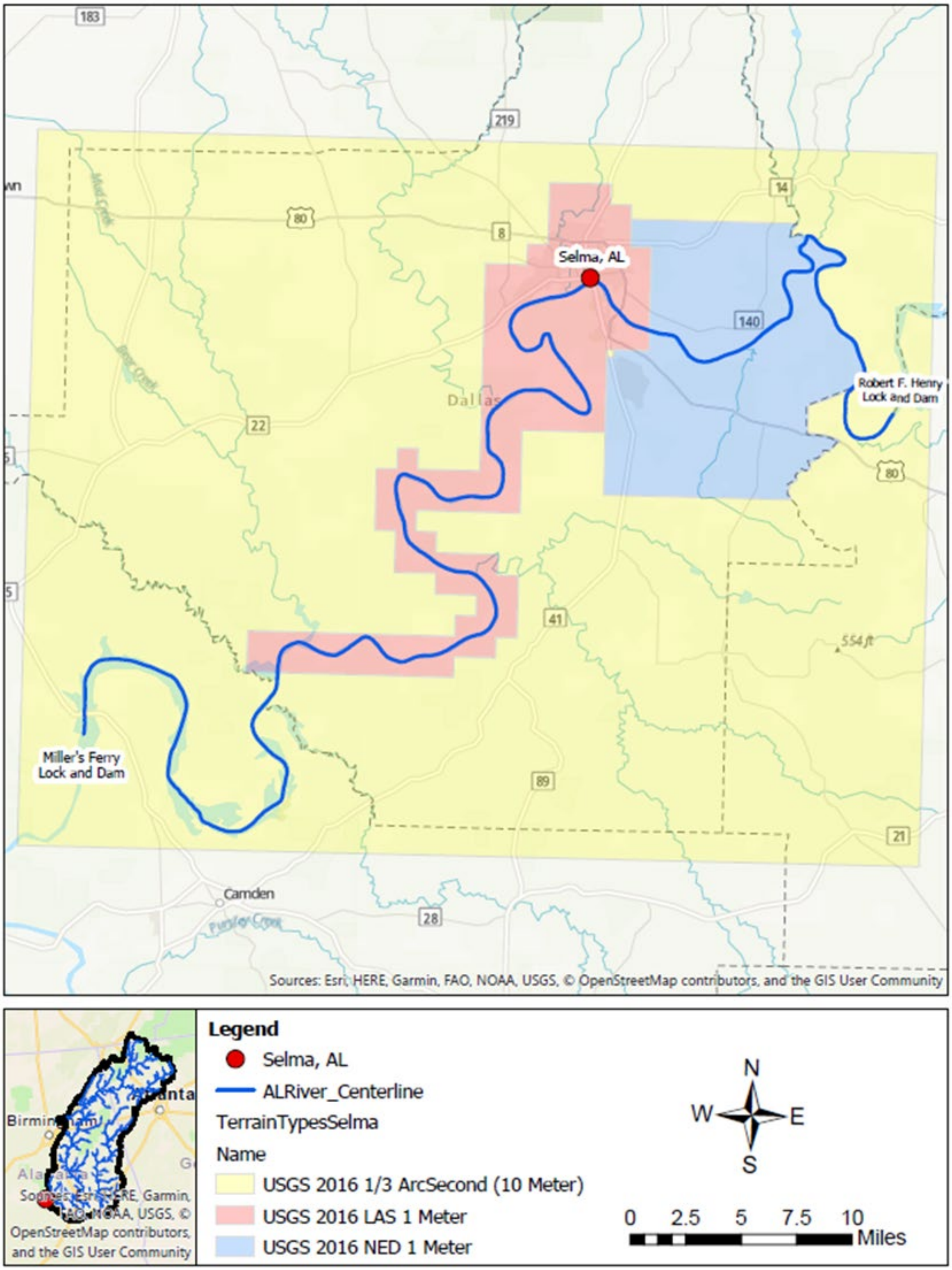
Hydrologic analysis and Hydraulic modeling were performed on the Alabama River near Selma to support the intermediate evaluation of the initial and focused array of alternatives as well as detailed modeling to support the determination of economic damages and damages reduced for the final array of alternatives. The goal of modeling the existing conditions of the study area was to establish a baseline for developing future without project conditions by which all flood risk management alternatives were evaluated.

### A.3.1. Terrain and Geometric Data

#### A.3.1.1. Digital Terrain Development

The terrain used for modeling the area in HEC-RAS was updated to use more recent LiDAR of the area. The terrain was developed using the USGS National Elevation Dataset (NED) and USGS LiDAR Point Cloud datasets from the USGS 3DEP site (**Figure A-38**). The quality information for these datasets are not listed within the metadata obtained. The 10-meter dataset used in the model was updated in 2016, however the data ranges from 1955 to 2016 in order to provide a continuous covered area. The data within the 1 meter LiDAR was collected December 2016 to March 2017 and published on the 3DEP website in 2018. For the majority of the Alabama River and overbanks stretching from Robert F. Henry Lock and Dam to Miller's Ferry Lock and Dam, the terrain is a 1-meter resolution. The entire areas of Selma and Selmont, AL are also a 1-meter resolution. The remaining portions of the terrain have a horizontal resolution of 10 meters. The horizontal projection for the terrain file was NAD 1983 2011 UTM zone 16N. Within the study area where 2D mesh was planned to be placed, HEC-RAS was utilized to burn out locations where the terrain had not been hydraulically corrected to remove obstructions that water could realistically pass underneath, such as small bridges and overpasses. Bathymetry of the river was provided by the Operations Division site office in Tuscaloosa, Alabama. These bathymetry data were acquired in early 2019. **Figure A-38** shows the various data sources and their extents in the study area.

*Figure A-38: Data source locations and corresponding extents utilized for the Selma FRM project*




### *A.3.1.2. Field Reconnaissance and Survey Data*

To date, only a bathymetric survey of the Alabama River between Millers Ferry and Robert F. Henry has been completed. Bridge data used within the model was obtained directly from the FEMA Flood Insurance Study HEC-RAS modeling effort. The FIS Report states that bridge geometry was determined from field surveys, as-built plans and field verification. Pier spacing and deck/roadway elevations were surveyed for each bridge, except for the railroad bridge which was determined using as-built drawings. (FEMA, 2014)

### *A.3.2. Hydrologic Model*

The hydrology of the Alabama River and upstream drainage area is extremely complex. The drainage area consists of over 17,000 square miles above Selma, 5 flood risk management projects and several other navigation dams on upstream rivers. It was initially planned to include an HEC-HMS hydrologic model to support flow input to the HEC-RAS model. This would have consisted of a heavily modified version of the Corps Water Management System (CWMS) HEC-HMS model for the ACT basin as well as modeling complex Reservoir Operations in HEC-ResSim. This was determined to be an unnecessary level of detail for the hydrologic needs of the study as well as a high risk to budget and schedule expectations.

The development of synthetic or balanced hydrographs was also considered as the input hydrology. This would consist of scaling observed flow hydrographs at locations with gaging along the Alabama River to match peak flow and volume of frequency events determined by a flow-frequency and volume-frequency analysis. One of the major drawbacks to this is the inaccuracy of recorded data at the upstream location of Robert F. Henry. The only available flow data at this location is computed using gate opening tables in the water control manual for this project. These tables have been historically inaccurate in determining the dam’s releases.

The engineering team decided it would be acceptable to use peak flows from a statistical analysis of gages as input into the hydraulic model. This was deemed acceptable for several reasons. First as a steady flow approach would be acceptable to capture the flow-stage relationship on the Alabama River as the duration of flood events is very long with peak stages maintained for several days. Also, as will be discussed later, levees were the only structural alternative carried forward to modeling, making storage and timing effects far less important to alternative screening. In the event that detailed modeling of floodwave timing would be needed to support an assessment of life risk behind the levee, the model could be modified to include flow hydrographs.

#### *A.3.2.1. Flow Frequency Analysis*

The Alabama River Basin has several gages throughout, however, only two flow gages were utilized for the flow frequency analysis to determine the frequency flows that were used as input into the hydraulic model. The gage located upstream of Selma is USGS 02421530 Alabama River at Robert F. Henry Lock and Dam and has a record starting in 1970 until present. Flow shown at this gage is computed based on releases using a gate operating schedule from the project’s Water Control Manual (reference, e.g. USACE, 19.). The second gage used in the analysis is the USGS gage 2423000 located at Selma, AL with a record of 99 events. The record begins in 1886, ends at 1990, and has missing years of 1887-1890, 1978, and 1988. Most of the peak flows at this location are the result of field measurements and therefore are considered highly accurate. Flows for the frequency analysis where not deregulated as would typically be required for a bulletin

17C analysis for several reasons. First regulation patterns in the dataset where determined to be consistent over time. This was determined using a Mann-Kendall test of daily mean flows performed by the USGS on a pre and post regulation dataset (Anderson, 2015). The single mass curve was used to assess the pattern in regulation. A change in the slope of this mass curve can determine if patters of regulation have remained relatively consistent over time. As the data analyzed for the Selma gage did not show a change in slope from regulated to deregulated, its full dataset was considered homogeneous and acceptable for use using bulletin 17C. While this trend analysis was not completed by the USGS for the Robert F. Henry gage, it was completed for several nearby USGS gages including USGS gage 02420000 at Montgomery, located just upstream of Robert F Henry and, USGS gage 02427500 near Millers Ferry located downstream of Selma. There was no significant trend any gages entire period of record indicating flow patterns where consistent over time (Hedgecock, 2003).

Also, unregulated peak flows would be higher than the actual peak flows that would occur as a result of a flood event. This would lend itself to potential overdesign of structures such as levees and storage facilities as they would be designed for higher flood volumes and peaks than would be needed in a real world, regulated condition for a specific AEP event.

The U.S. Army Corps of Engineers (USACE) Statistical Software Package (HEC-SSP) was used to calculate the frequency flows for both of these gages. **Table A-5** shows the 100-year peak discharges derived from a Bulletin 17C (see England et al., 2017) flow frequency analysis in HEC-SSP. Regional Skew was not available for the study area, therefore the station skew was utilized in the analysis. **Table A-6** shows a full range of frequency flows calculated for both gages. These flows where utilized for development of the design storm events in the hydraulic model.

**Figure A-39** shows a comparison of frequency events computed for the study area, the observed data, and the frequency flows from the 2014 FEMA FIS Study

Regression Equations were not utilized for this study due to the size of the basin. The drainage area above Selma, AL is approximately 17,000 sq. miles for the Alabama River. The regression equations have a limitation of the drainage area between 0.44 to 1,344 sq. miles. Depending on which region the area is in, these values vary in-between this range. (USGS, 2003)

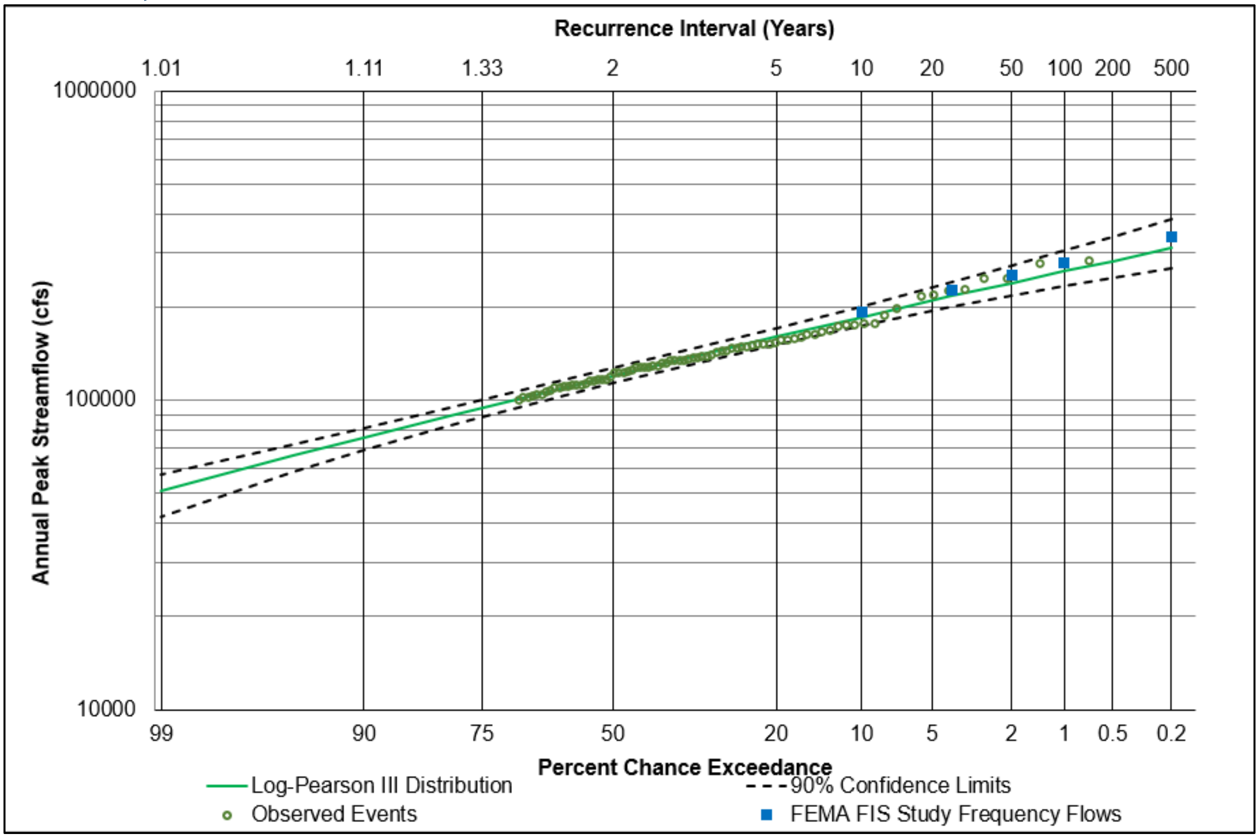
***Table A-5: 100-Year Frequency Flows using Bulletin 17C***

| Location | Program | | Skew | MSE Error | Period | Historic Period | # of Events | Historical Events | 1% Flows (cfs) |
|---|---|---|---|---|---|---|---|---|---|
| **Robert F. Henry Lock and Dam** | HEC-SSP | Bulletin 17C | -0.107 | 0.049 | 1886, 1891-2009 | 124 | 118 | 1 | 259,000 |
| **Selma, AL** | HEC-SSP | Bulletin 17C | -0.045 | 0.055 | 1886, 1891-1990 | 105 | 99 | 1 | 272,000 |

*Table A-6: Gage Estimate Flows at USGS Gages 02423000 and 02421351 in frequency (cfs)*

| Site Location | 0.5 AEP | 0.2 AEP | 0.1 AEP | 0.04 AEP | 0.02 AEP | 0.01 AEP | 0.005 AEP | 0.002 AEP |
|---|---|---|---|---|---|---|---|---|
| **Robert F. Henry Lock and Dam** | 122,000 | 161,000 | 186,000 | 216,000 | 238,000 | 259,000 | 279,000 | 306,000 |
| **Selma, AL** | 123,000 | 165,000 | 191,000 | 217,000 | 249,000 | 272,000 | 296,000 | 328,000 |

*Figure A-39: Annual exceedance probabilities and corresponding flow rates for Alabama River near Selma, Alabama*




### *A.3.3. Hydraulic Modeling Approach*

A FEMA developed HEC-RAS model utilized in the 2014 Flood Insurance Study was used to create an updated model. The HEC-RAS version 5.0.5 steady state model was converted to a version 5.0.7 unsteady 1D/2D Model and heavily modified. The model covered 102 river miles along the Alabama River in-between the projects Miller's Ferry Lock and Dam and Robert F. Henry Lock and Dam. The Cahaba River was included to better model the inflows from the Cahaba into the Alabama River along with any backwater effects on either system. The stretch of the Cahaba River included 22 miles from the confluence up to Marion Junction, AL. It was determined that the 2D mesh was needed in several locations within the floodplain of the study area. Reasons supporting 2D modeling included the following:

- The terrain in the area is extremely flat, meaning water flows in multiple directions as it enters the floodplain.
- Sharp meanders in the river cause the direction of flow to change sharply as flow escapes the river.

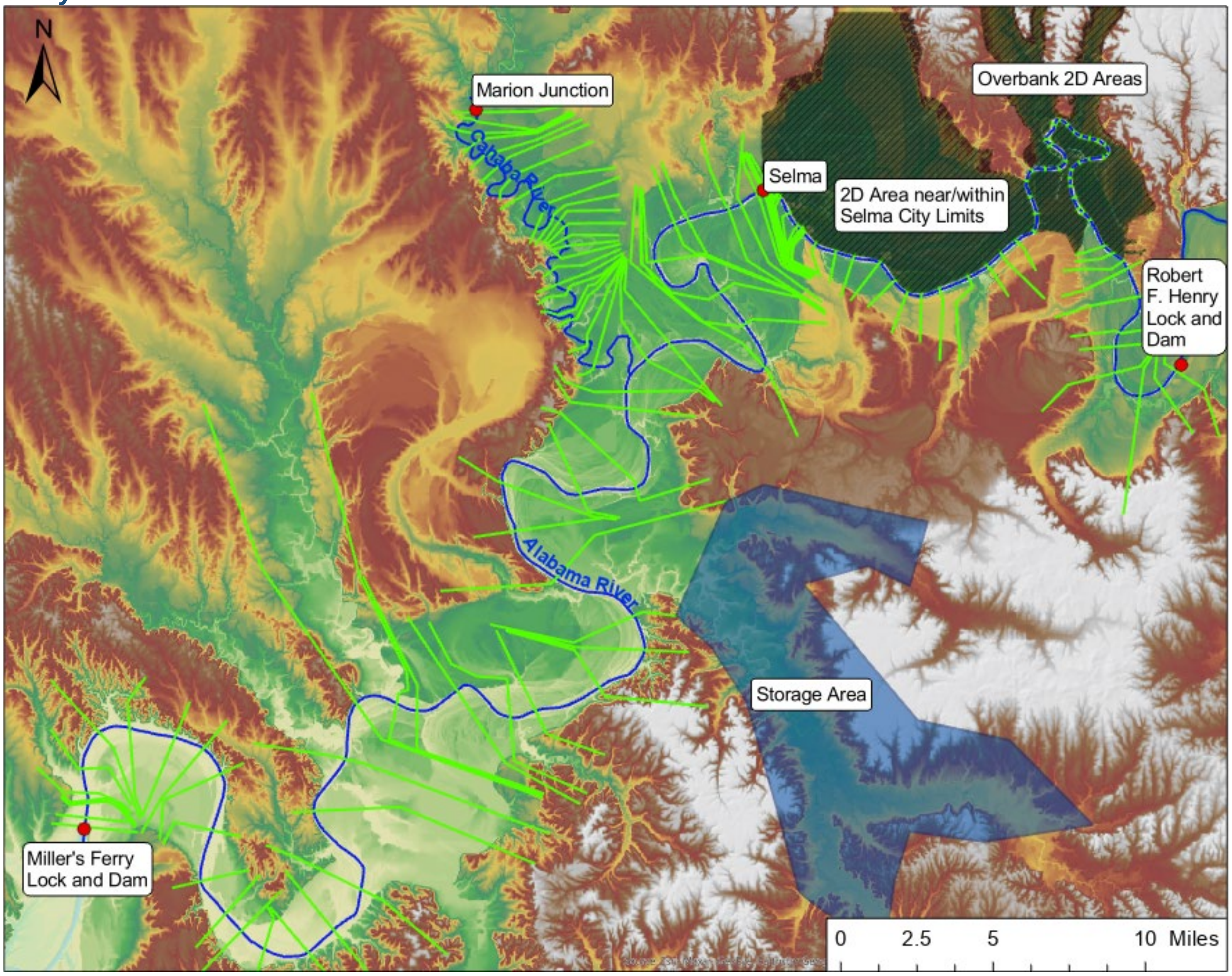
- The modeling of ring levees, as was anticipated for this effort, would be difficult and less accurate in 1D. It is more straight forward to input oddly shaped hydraulic structures within a 2D mesh.

The previously described terrain model was utilized in supporting all hydraulic modeling efforts. The frequency flows used for this analysis were based off the HEC-SSP Bulletin 17C analysis described above for USGS Gages 02423000 Alabama River at Selma, AL and 02421350 at Robert F. Henry Lock and Dam.

All flows input into the unsteady HEC-RAS model where constant peak flows. At flow change locations additive flow was input as a lateral inflow to reach the statistically derived peak flow at a location. Careful consideration was given to using constant peak flows in the unsteady model as opposed to the development of balanced hydrographs but, it was ultimately determined to be acceptable for the required level of analysis. There is often concern using only peak flowrates in an unsteady flow analysis or using a steady flow analysis can overestimate inundation as this assumes a constant flowrate over an extended or infinite amount of time. This is because, in reality, flowrate in rivers is dynamic with peak flows only accounting for a portion of the flow hydrograph. The time at which the river remains at or near its peak flowrate affects the amount of inundation an area may receive. The Alabama River is a slow-moving river with peak flows during floods remaining high for days. Therefore, the use of peak flows in the unsteady model would not overestimate inundation. In the event flood flow timing becomes a critical component, balanced hydrographs could be developed. For instance, if it becomes necessary to investigate levee breaches and response times, an understanding of floodwave timing a breach formation would be critical to assessing life safety.

For the Cahaba River reach, two 2D areas and one storage area were added to the original model to account for backwater effects up the Cahaba River and other small tributaries. The first 2D area was added halfway between Robert F. Henry Lock and Dam and Selma, AL. The high degree of sinuosity in the upstream extent of the model presents challenges to a 1D approach, especially in modeling of out-of-bank stages. The second 2D area was added just upstream Selma, AL where several woody wetlands are located. This area is very flat and appears to be where the Alabama River may have once flowed, leaving oxbow landmarks in the earth (visible via aerial photography and in digital elevation models). Flow in this area is primarily two-dimensional for large, low frequency events.

***Figure A-40: Schematic of the hydraulic modeling extents for the Alabama River and surrounding study area***



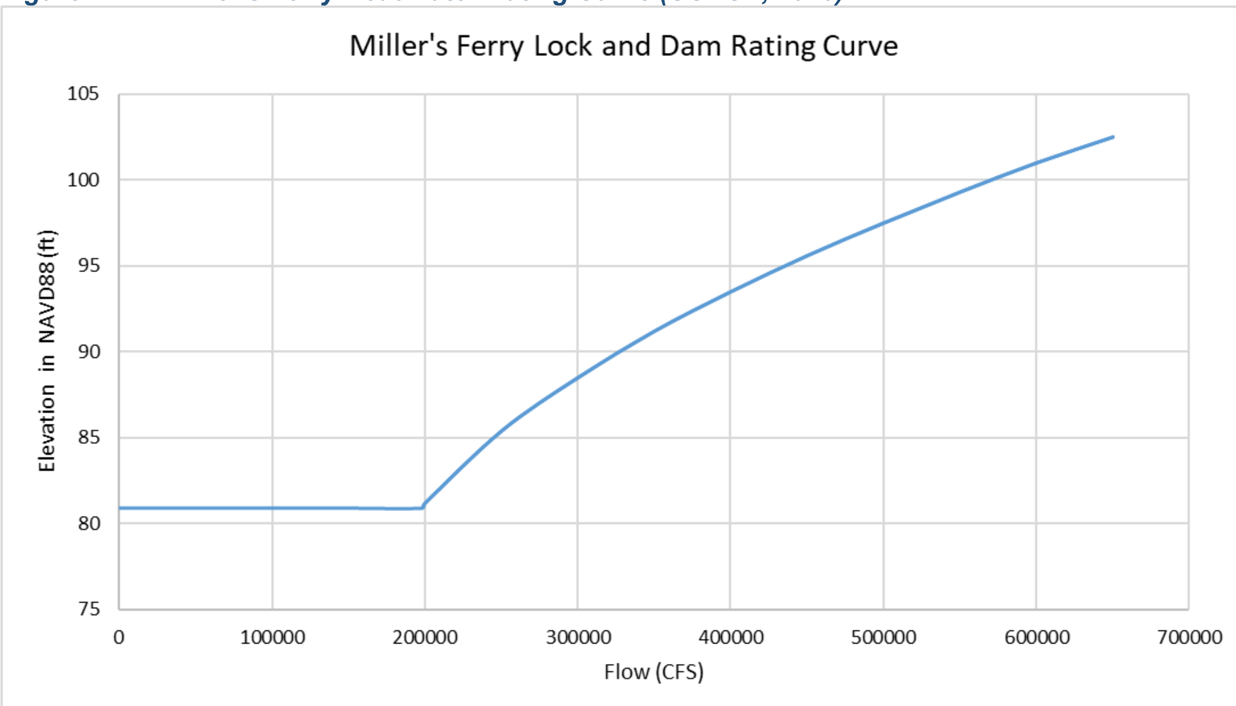

The hydrologic flow change locations in the model are located downstream of Robert F. Henry Lock and Dam, upstream of Selma, AL, and midway between Selma and Miller's Ferry Lock and Dam (Downstream of Cahaba River (D.A. 1824), Cedar Creek (D.A. 461), Bogue Chitto Creek (D.A. 364)) to account for flow from the Cahaba River, a major tributary to the Alabama River downstream of the study area.

### *A.3.3.1. Boundary Conditions and Tie-ins*

The downstream boundary condition for the model was the headwater rating curve for Miller's Ferry Lock and Dam in Wilcox County, Alabama. Miller's Ferry Lock and Dam is the next available gage location downstream of Selma, AL and gives a more accurate downstream boundary condition for modeling the backwater effect the pool has on the Alabama River system. The curve was obtained from the current water control manual (USACE, 2015) for the project, and is shown on **Figure A-41**. The Cahaba River ties into the Alabama River downstream of Selma, Alabama and has an upstream boundary condition at USGS Gage 02425000 near Marion Junction, AL. The purpose of this tie-in is to account for any effect the Cahaba River may have on the Alabama River system. The gage located at Marion Junction, AL is the next upstream gage located on the Cahaba River and would account for any backwater effect the Alabama River has on the

Cahaba. The upstream boundary condition for the Alabama River is below Robert F. Henry Lock and Dam.

*Figure A-41: Miller's Ferry Headwater Rating Curve (USACE, 2015)*




### A.3.3.2. Structures

There are four bridges in the model extents that cross the Alabama River including the US Highway 80 Bridge, Edmund Pettus Bridge, US Highway 28 Bridge, and Railroad Bridge directly upstream of the Edmund Pettus Bridge. Three bridges are located at Selma, AL and one is located near Miller's Ferry Lock and Dam. Upstream and downstream river cross sections are shown for each bridge on **Figure A-42**, **Figure A-43**, **Figure A-44**, and **Figure A-45**, respectively.

The bridges were modeled using 1D instead of 2D due to the current capabilities for modeling hydraulic structures within HEC-RAS. It was determined that 1D would better represent the bridge hydraulics.

***Figure A-42: Upstream and downstream cross-sections of the Alabama River at US Highway 80 Bridge***

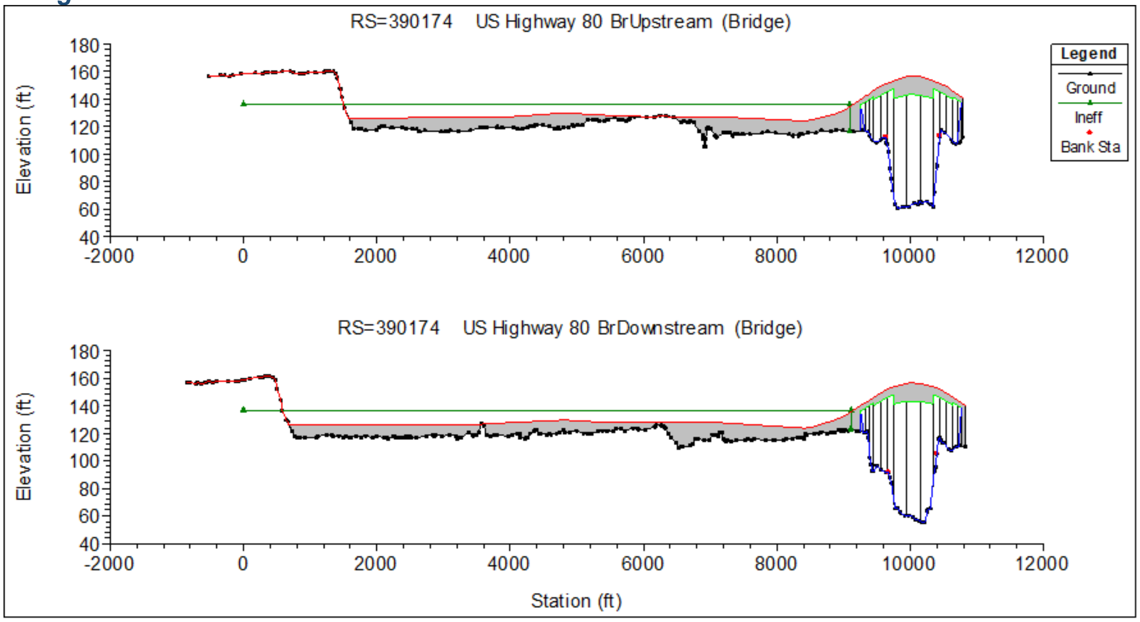



***Figure A-43: Upstream and downstream cross-sections of the Alabama River at Edmund Pettus Bridge***

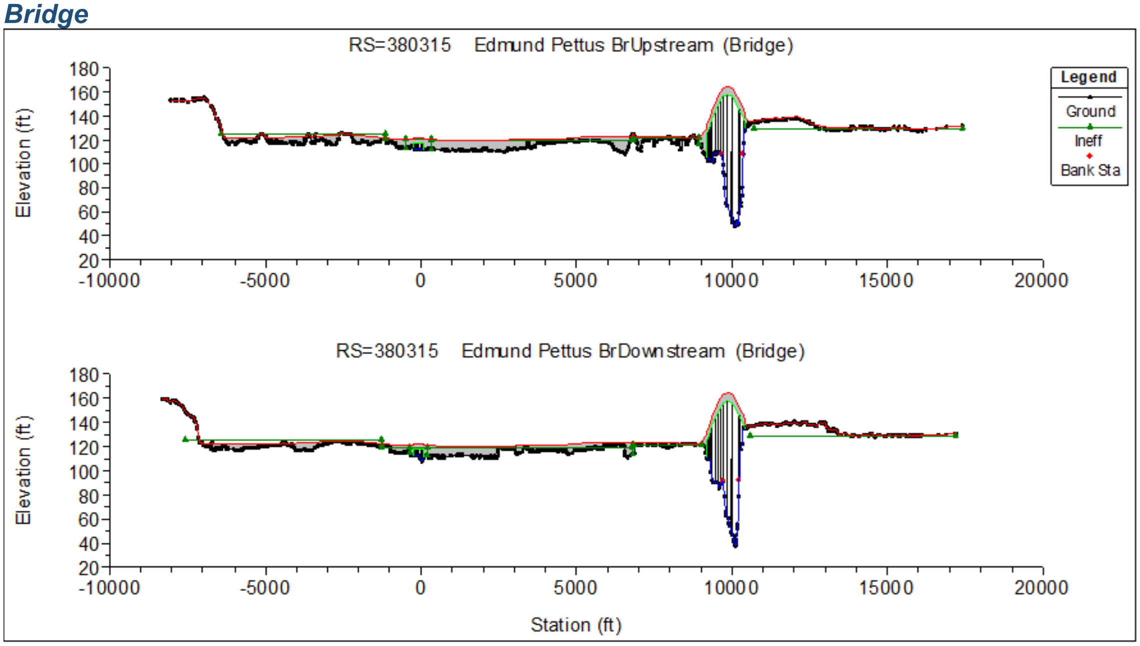


*Figure A-44: Upstream and downstream cross-sections of the Alabama River at US Highway 28*

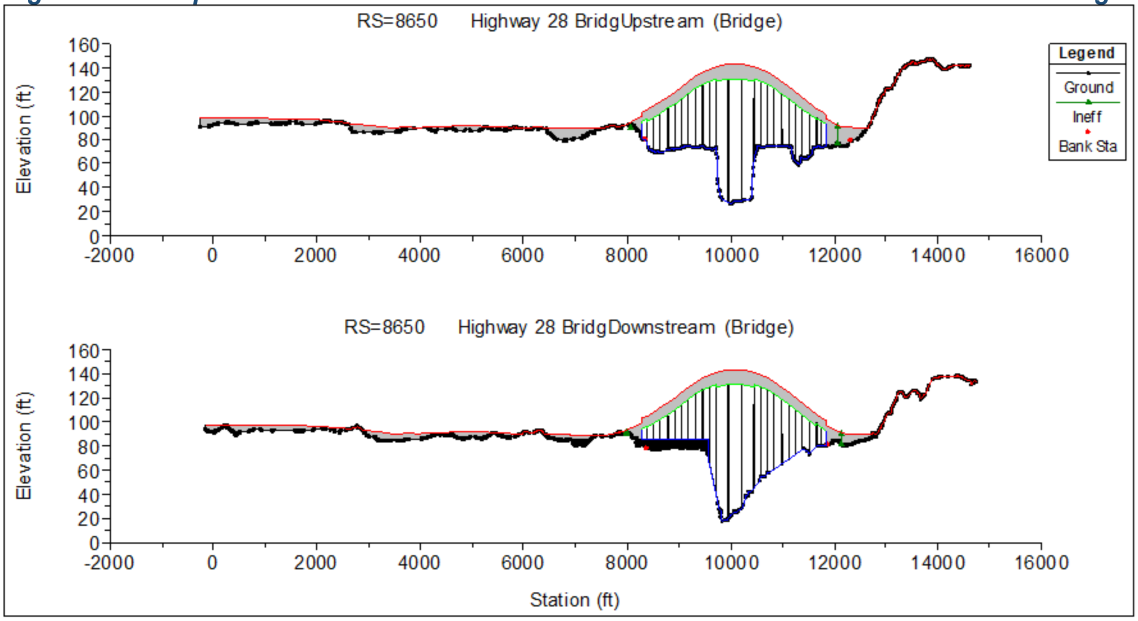
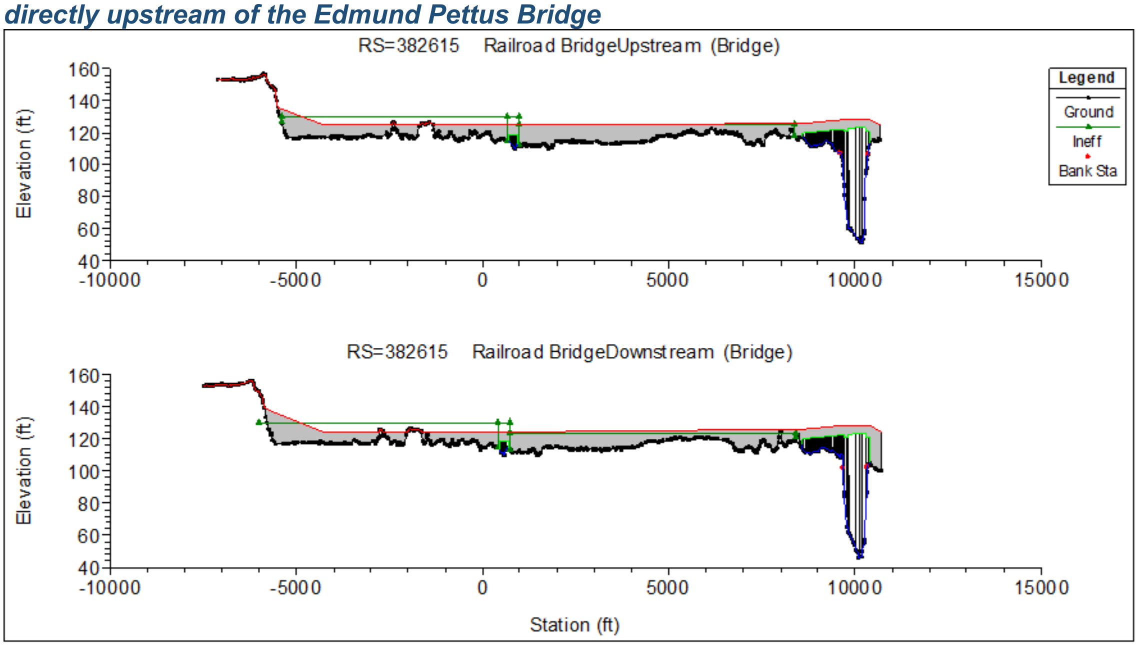



*Figure A-45: Upstream and downstream cross-sections of the Alabama River at Railroad Bridge directly upstream of the Edmund Pettus Bridge*




### *A.3.3.3. Ineffective Flow Areas*

The reduced conveyance due to expansion and contraction at structures is reflected in the HEC-RAS model by defining ineffective flow areas for the cross sections immediately upstream and downstream of the structures. The station and elevation of the ineffective flow areas were located based on the HEC-RAS Hydraulic Reference Manual (USACE, 2016).

In addition to the application of the ineffective flow areas upstream and downstream of the structures, the ineffective flow areas were also applied to the cross sections in the areas where the topography indicates that the flows may not be fully effective. These are generally the areas where the floodplain expands and contracts suddenly or where there is divided flow. Stationing of the ineffective flow areas was defined using the same flow contraction and expansion rule applied to the structures.

### A.3.3.4. Channel Roughness Values

Manning's roughness coefficients (Manning's "n-values") were established using guidance from the HEC-RAS *Hydraulic Reference Manual* (Reference 22). Manning's n-values used in the hydraulic computations were chose based on engineering judgment from field observations of the streams and floodplain areas and utilizing the 2011 NLCD Land Use Dataset. Roughness values used for the study streams varied from 0.030 to 0.040 for the channel and 0.04 to 0.12 for the overbank areas. The lowest value for the overbank areas was for open fields that were mostly flat, downstream of Selma, AL. The higher values for the overbank areas represented the heavily wooded and forested areas. **Figure A-46** below contains the Manning's n-values associated with the NLCD Dataset imported into HEC-RAS.

*Figure A-46: Manning's n Value assigned for 2011 NLCD Land Use Dataset*

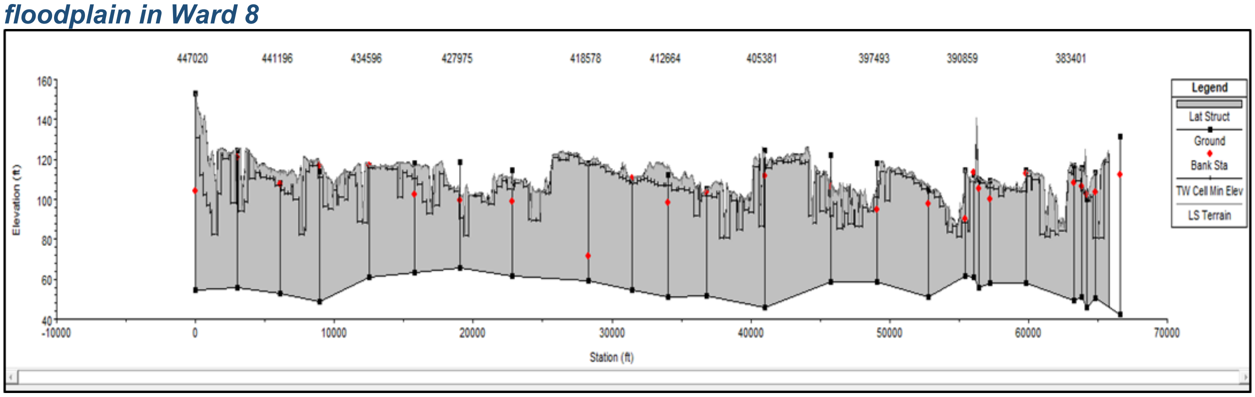
| Color | Value | Name | Default Manning's n |
|---|---|---|---|
| | 0 | nodata | |
| | 11 | open water | 0.01 |
| | 21 | developed, open space | 0.025 |
| | 22 | developed, low intensity | 0.05 |
| | 23 | developed, medium intensity | 0.06 |
| | 24 | developed, high intensity | 0.06 |
| | 31 | barren land rock/sand/clay | 0.06 |
| | 41 | deciduous forest | 0.1 |
| | 42 | evergreen forest | 0.09 |
| | 43 | mixed forest | 0.08 |
| | 52 | shrub/scrub | 0.08 |
| | 71 | grassland/herbaceous | 0.07 |
| | 81 | pasture/hay | 0.07 |
| | 82 | cultivated crops | 0.07 |
| | 90 | woody wetlands | 0.08 |
| | 95 | emergent herbaceous wetla... | 0.07 |

The values from this figure were used for the 2D area and were cross referenced with the existing Manning's n-values for 1D cross-sections. Override regions were utilized for tying in the values within the existing 1D model to the 2D area based on the land use type defined in the 2011 NLCD Land Use Dataset.

### A.3.3.5. Lateral Structures

As discussed above, the overbank areas upstream of Selma, AL were modeled using a 2D area to better represent flow within the overbanks. Lateral structures were used to connect the 1D cross sections to the 2D overbank area. The lateral structure represents the ground elevation at the interface between the river channel and the overbanks. Modeling the lateral structure as a weir provided the most stability in the model. The hydraulic structures in the model were set as zero height weirs with a weir coefficient of 0.2. The weir coefficients were chosen based on Lateral Weir Coefficients within the HEC-RAS 2D Modeling User's Manual (USACE, 2016). **Figure A-47** shows the HEC-RAS modeled lateral structure used to connect the 1-D river channel to the floodplain in Ward 8.

*Figure A-47: HEC-RAS model lateral structure used to connect the 1-D river channel to the floodplain in Ward 8*




### A.3.3.6. HEC-RAS Results and Calibration

To ensure that the model is a good representation of the Alabama River near Selma, calibration to three large observed flood events was performed. Typically, a series of flow hydrographs would be run through the hydraulic model, however, as discussed in the report above, there was significant difficulty in developing these. Therefore steady, continuous peak flows where run through the model to match peak stages observed at the Selma USGS gage.

Three events were utilized to support the Existing Conditions hydraulic model calibration. The events occurred in 1979, 1987, and 1990 with discharges of 250,000, 110,000, and 280,000 cubic feet per second, respectively (**Table A-7**). All of these events were chosen due to construction of Miller's Ferry Lock and Dam and Robert F. Henry Lock and Dam in the late 1960s and were run through the HEC-RAS model as continuous flow. Flow change locations where modeled as lateral inflows at the described locations.

In addition to the calibration simulations, two additional runs where made to ensure the composite parameters used reasonably represented peak flood stages. It is worth noting that in these two validation events of March of 2001 and April of 2005, the hydraulic model underestimates the stage by about 1 and 2 feet, respectively. However, this is well within the uncertainty in the peak flow measured for these events and deemed adequate.

*Table A-7: Flood events from 1979, 1987, and 1990 used for model calibration*

| Flood Event | Calibration/Validation | Peak Discharge in Selma (cfs) at Gage 02423000 | Estimated Peak Flood Recurrence Interval & Magnitude | Model Peak Stage | Actual Peak Stage |
|---|---|---|---|---|---|
| **April 18, 1979** | Calibration | 250,000 | >2%; 251,000 cfs | 117.85 | 116.82 |
| **March 2, 1987** | Calibration | 110,000 | >50%; 124,000 cfs | 102.59 | 102.91 |
| **March 21, 1990** | Calibration | 279,000 | <1%; 276,000 cfs | 118.87 | 118.60 |
| **March 23, 2001** | Validation | 127,000 | <50%; 124,000 cfs | 106.25 | 105.26 |
| **April 3, 2005** | Validation | 186,000 | <20%; 166,000 cfs | 112.86 | 110.77 |

Using the validated hydraulic model and flows from the flow-frequency analysis as inputs to the model, the frequency simulations were run. The 0.50, 0.20, 0.10, 0.04, 0.02, 0.01, 0.005, and 0.002 annual exceedance probability (AEP) event simulations produced profiles representative of the flooding potential for existing conditions. **Table A-8** shows the existing conditions inflows by frequency for all inflow locations in the hydraulic model.

*Table A-8: Flow rates (cfs) at stream gages within the study area for various flood events and annual exceedance probabilities with existing conditions*

| Event | AEP | Cross Section: 539014 (Alabama River Below RE Henry Lock and Dam) | Cross Section: 400732 (Alabama River upstream of Selma, AL) | Cross Section: 143555 (Between Bogue Chitto Creek and Chilachee Creek) | Cross Section: 10275 (Upstream of Highway 28 Bridge and Miller's Ferry Lock and Dam) | Cross Section: 113041 (Cahaba River near Marion Junction) |
|---|---|---|---|---|---|---|
| **0.50 AEP** | 0.5 | 122000 | 1000 | 7000 | 4000 | 14600 |
| **0.20 AEP** | 0.2 | 161000 | 4000 | 9500 | 5800 | 19600 |
| **0.10 AEP** | 0.1 | 186000 | 5000 | 11000 | 6700 | 22700 |
| **0.04 AEP** | 0.04 | 216000 | 1000 | 12500 | 7500 | 25800 |
| **0.02 AEP** | 0.02 | 238000 | 11000 | 14300 | 8600 | 29600 |
| **0.01 AEP** | 0.01 | 259000 | 13000 | 15600 | 9500 | 32400 |
| **0.005 AEP** | 0.005 | 279000 | 17000 | 17000 | 10300 | 35200 |

| Event | AEP | Cross Section: 539014 (Alabama River Below RE Henry Lock and Dam) | Cross Section: 400732 (Alabama River upstream of Selma, AL) | Cross Section: 143555 (Between Bogue Chitto Creek and Chilachee Creek) | Cross Section: 10275 (Upstream of Highway 28 Bridge and Miller's Ferry Lock and Dam) | Cross Section: 113041 (Cahaba River near Marion Junction) |
|---|---|---|---|---|---|---|
| **0.002 AEP** | 0.002 | 306000 | 22000 | 18800 | 11400 | 39000 |

### *A.3.4. Future Without-Project Conditions*

As conditions in the basin above Selma are expected to change over the 50-year planning period, a future without project conditions scenario was developed based on the existing conditions model. The two primary drivers to changes in hydrology for this area were determined to be climate change and changes to land use. The climate change assessment presented in **Section A.2** of this appendix states that there is not enough evidence to support an adjustment to the hydrology as a result of climate change. Changes in land use however can be estimated for the 17,000 square mile basin above the project.

The future conditions were determined by utilizing the existing Corps Water Management System (CWMS) hydrology model for the subbasins upstream of Selma, AL and land use changes from the 2070 Integrated Climate and Land Use Scenario (ICLUS) dataset. This dataset utilizes population projections through the end of the century, reflecting different assumptions about fertility, mortality, and immigration to determine the demand for new homes, and estimates the amount of impervious surface that can be expected. The majority of projected development was observed far upstream of Selma, AL, in areas such as near Montgomery, AL and Birmingham, AL.

Impervious projections were utilized for each subbasin in the CWMS hydrology model where adjusted from existing conditions. The model was then run with a series of precipitation inputs ranging from small 0.1 AEP floods to large 0.002 AEP floods. In every case considered, peak flows where increased by approximately 2 percent with a variability of about 0.15 percent per event. It was decided that an increase in peak flows of 2 percent would be a reasonable adjustment to hydrology to account for future development.

It is important to note that changes to upstream regulation were not considered when running these scenarios to determine the future without project condition. Upstream flood operations are however currently being considered for modification and in some cases, reduction of flood pools. At the time of this analysis report, there is no confidence that these changes will actually occur or what amount of storage will be reallocated, if any, as part of a recommended plan. Therefore, the future without does not incorporate upstream regulation changes. If changes are to occur at these projects, consideration should be given to impacts on the Recommended Plan in the Preconstruction Engineering and Design phase.

Climate change was also a consideration for the future without project condition. The climate change analysis presented in this report does indicate some consensus that there will be an increase in extreme precipitation events in the southeast but, there is not a strong consensus that this will result in an increase in peak river flows. One of the main reasons for this is there has been, and will continue to be, an increase in temperatures and an increase in the severity and frequency of droughts in the southeast. Since the 1970s, temperatures in the southeast have been gradually increasing. This has caused an increase in soil moisture deficits, increasing groundwater infiltration and evapotranspiration. This is one contributor that is likely to offset the increase in runoff. This is reinforced by the lack of extreme flow events the Alabama River has experienced since the 1990s despite no sharp drop in peak annual precipitation. In the climate change assessment, observed gage data shows there has been a sharp and consistent drop in annual peak flows near Selma. Given these considerations, there were no changes to hydrology based on climate change.

It was the decision of the PDTs engineering team that a 2 percent increase in flows provided a practical and reasonably conservative change in peak flows based on land use changes for the future without project conditions. **Table A-9** shows the updated flows used as inputs to the hydraulic model for the future without project condition. **Table A-10** and **Table A-11** show the comparison of stages at the Selma, Alabama USGS gage resulting from the hydraulic model runs for the Existing Condition to the Future Without Project Condition. Additional model results of water surface profiles for existing conditions and future without project conditions can be found in **Section A.9 Subpart 1** and **Subpart 2**, respectively.

***Table A-9: Flow rates (cfs) at model flow-change locations corresponding with stream gages and flow change locations from the FEMA FIS model within the study area for various flood events and annual exceedance probabilities (AEP) with respect to future without project conditions***

| Event | AEP | Cross Section: 539014 (Alabama River Below RE Henry Lock and Dam) | Cross Section: 400732 (Alabama River upstream of Selma, AL) | Cross Section: 143555 (Between Bogue Chitto Creek and Chilachee Creek) | Cross Section: 10275 (Upstream of Highway 28 Bridge and Miller's Ferry Lock and Dam) | Cross Section: 113041 (Cahaba River near Marion Junction) |
|---|---|---|---|---|---|---|
| **2 year** | 0.5 | 12440 | 1020 | 7242 | 4386 | 14892 |
| **5 year** | 0.2 | 164220 | 4080 | 9690 | 5916 | 19992 |
| **10 year** | 0.1 | 189720 | 5100 | 11220 | 6834 | 23154 |
| **25 year** | 0.04 | 220320 | 1020 | 12750 | 7752 | 26316 |
| **50 year** | 0.02 | 242760 | 11220 | 14586 | 8874 | 30600 |
| **100 year** | 0.01 | 264080 | 13260 | 15912 | 9690 | 33048 |
| **200 year** | 0.005 | 284580 | 17340 | 17340 | 10506 | 35904 |
| **500 year** | 0.002 | 312120 | 22440 | 19176 | 11628 | 39780 |

*Table A-10: Water surface elevations and river stages associated with various flood events and annual exceedance probabilities (AEP) at Selma, Alabama showing Existing Conditions*

| Year | AEP | Elevation (ft) (NAVD88) | Stage (ft) |
|---|---|---|---|
| **2** | 0.5 | 104.94 | 43.14 |
| **5** | 0.2 | 110.24 | 48.44 |
| **10** | 0.1 | 113.36 | 51.56 |
| **25** | 0.04 | 115.53 | 53.73 |
| **50** | 0.02 | 117.69 | 55.89 |
| **100** | 0.01 | 119.05 | 57.25 |
| **200** | 0.005 | 120.44 | 58.64 |
| **500** | 0.002 | 122.52 | 60.72 |

*Table A-11: Water surface elevations and river stages associated with various flood events and annual exceedance probabilities (AEP) at Selma, Alabama showing future without project (FWOP) conditions with uniform 2% increases*

| Year | AEP | Elevation (ft) (NAVD88) | Stage (ft) |
|---|---|---|---|
| **2** | 0.5 | 105.21 | 43.41 |
| **5** | 0.2 | 110.83 | 49.03 |
| **10** | 0.1 | 113.63 | 51.83 |
| **25** | 0.04 | 115.91 | 54.11 |
| **50** | 0.02 | 118.01 | 56.21 |
| **100** | 0.01 | 119.33 | 57.53 |
| **200** | 0.005 | 120.89 | 59.09 |
| **500** | 0.002 | 122.85 | 61.05 |

## A.4. Formulation of Alternatives

Plan formulation is the process of building alternative plans that meet planning objectives and avoid planning constraints. Alternative plans are a set of one or more management measures functioning together to address one or more planning objectives. With the problems and objectives in mind, the project delivery team first identified measures which were used to develop an array of alternatives. These measures along with the initial array of alternatives were presented at the Alternative Milestone Meeting held January 16, 2019.

This study includes consideration of atypical flood risk management measures such as streambank stabilization to prevent structural foundation failures for buildings located along the riverbank. Hydraulic modeling was completed for the streambank stabilization measures considered and indicated that there would be no impact to the water surface elevations. As a result, no damages were derived through an HEC-FDA model. Additional information regarding the consideration of these specific measures are included in the plan formulation section of the main report as well as in the Appendix E – Economics.

### *A.4.1. Problems and Opportunities*

#### *A.4.1.1. Problem Identification*

There are several problems related to flooding in the basin. While there are some small local flooding issues, the large scale issues are the result of flooding from the Alabama River. The Alabama River basin above Selma, AL is a nearly 17,000 square mile drainage area. This basin has multiple flood control projects however, these are not targeted or capable of providing meaningful flood reduction from the Alabama River near Selma. The specific problems identified for the Selma area are as follows.

- structural damages caused by flooding predominantly in Ward 8;
- shear bank failure along the Alabama River throughout the City of Selma caused by the rise and fall of the river;
- stormwater drainage during flooding events;
- flow resiliency of the City of Selma;
- flood risks to nationally registered historic and cultural resources;
- high social vulnerability; and
- threats to community cohesion.

#### *A.4.1.2. Opportunities*

There are several opportunities to address these issues. They are as follows.

- reduce effects of riverine flooding in the Selma;
- reduce structural inundation damages;
- reduce threats to Nationally Registered historic and cultural resources;
- improve resiliency;
- improve social vulnerability; and
- improve community cohesion.

### *A.4.2. Study Goals, Objectives, and Constraints*

The objectives are what the alternative plans should achieve. To support accomplishment of the study goals and the Federal objectives, the PDT developed the following planning objectives to apply to this area over the next 50 years.

- Increase community resiliency and maintain community cohesion by reducing risk to vulnerable populations (human health and safety);
- to reduce threat to nationally registered historic resources between river miles 256-261 introduced by high water events; and
- increase resiliency by reducing damages to property and infrastructure.

### *A.4.3. Constraints*

The planning constraints limit plan formulation. There are generally two types of planning constraints, universal and study specific constraints. The universal constraints are typically considered in every planning study and include the following for this study:

- do not increase impacts to floodplain management;
- avoid impacts to existing Federal projects in the Study Area. If impacts are unavoidable, engineer solutions and incorporate revisions as part of the study;
- avoid or minimize adverse impacts to T&E Species and wildlife habitat ;
- avoid or minimize adverse impacts to historic properties and cultural resources; and

- no use of public funds on private property without an overriding public benefit.

Legal constraints may include those associated with impacting existing Federally constructed projects. Policy constraints may include expanding the Study Area beyond the scope of the approved authority, including functional programs (i.e. capability to address bank-line erosion) not previously approved by SAD or HQ.

### *A.4.4. Design Criteria*

Criteria used for the design of flood risk management measures was developed by the PDT based on the specific study objectives and constraints. A listing of the criteria organized by restoration objective is shown below.

- Objective: Reduce average annual flood damages to residential and commercial structures.
    - Criteria:
        - Structural
            - reduce/maintain level of life safety risk;
            - reduce peak flood elevations; and
            - reduce max footprint of floods.
        - Non-Structural
            - reduce/maintain level of life safety risk;
            - reduce risk to structures in the floodplain; and
            - remove risk from the floodplain.
- Objective: Improve community resilience by reducing flood risk of vulnerable populations.
    - Criteria:
        - implement measures that reduce flood risk in vulnerable communities.

### *A.4.5. General Types of Flood Risk Management Measures Considered*

A suite of structural and non-structural measures were considered in this study to help satisfy the objectives and design criteria. These measures were utilized during the development of alternative designs and applied throughout the study area based on location-specific problems, constructability, and the flood risk management objectives. The PDT determined that a Flood Response Plan could be combined with any of the measures considered to address life safety and therefore was not incorporated into each alternative description. These measures are discussed below.

#### *A.4.5.1. Structural Measures*

Six different structural measures were identified and analyzed for effectiveness in reducing flood risk in the vicinity of Selma. These six measures were: in-line detention/retention, off-line detention/retention, bridge/culvert modification, levees/floodwalls, channel diversions and stabilization of the streambank. These measures were screened in order to provide a solid basis for the formulation of alternatives. Measure screening was based on each measure’s ability to meet the study objectives and avoid constraints. Measures were identified by the PDT by analyzing aerial imagery, digital terrains, project photos, model results, holding discussions with the non-federal sponsor and considering the practicality and feasibility of implementing each measure in this basin.

#### *A.4.5.1.1. In-Line Detention/Retention*

In-line detention/retention consists of a damming surface being constructed across the stream/floodplain to create flood storage. The damming surface could be designed to create a permanent pool or to only attenuate flow during large events. Topography plays a critical role in determining the viability of in-line detention/retention. These structures are often placed in narrow floodplains directly downstream of wide, low floodplains in an effort to maximize the ratio of storage capacity to structure size, which generally translates to a higher performing, lower cost project.

In-line detention/retention must be carefully evaluated to ensure that the impoundment of water does not pose an increased risk to life safety or economic damage in the event of a structural failure. Effects of upstream water surface elevations must be investigated to ensure that the structure does not create incremental risk. Also, unless specifically designed for fish passage, inline structures often act as a hydraulic disconnect, which can have detrimental impacts to the upstream aquatic ecosystem. In-line reservoirs must conform to appropriate dam safety standards.

The topography in the Selma area and the size of the Alabama River Basin above the city preclude the use of inline detention. Additionally, Selma resides between two navigation dams, Robert F. Henry and Millers Ferry. These projects do not provide flood reduction, as the drainage basin above these projects is too large relative to the storage available. As it is known that storage of floodwaters in this area for the Alabama River is impractical, this measure was screened out.

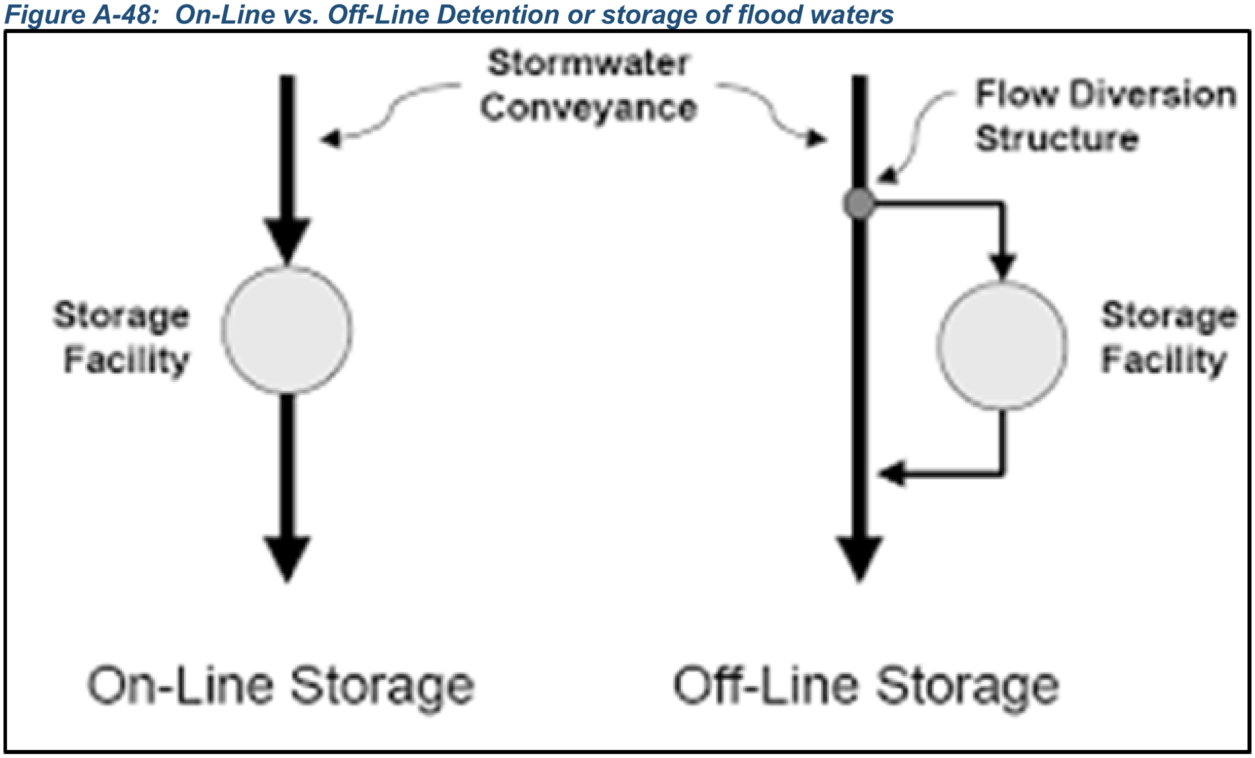
#### *A.4.5.1.2. Off-Line Detention/Retention*

Off-line detention/retention consists of detention/retention reservoirs being constructed adjacent to the stream (**Figure A-48**). These features are typically achieve storage through excavation and berm construction. Flood waters overflow into the reservoir, often through a weir, and are held until the falling limb of the hydrograph is observed, when they are slowly released back into the river. Typically, a berm is built around a low portion of the floodplain, with an overflow section being constructed near the upstream end of the berm, and adjacent to the stream. The height and length of the overflow section are optimized through model assessment of different weir geometries. The frequency and timing of filling can be tailored to optimize flood risk reduction in the study area. The height of the non-overflow berm is usually set optimize storage capacity and flood risk reduction. Constraints to reservoir capacity include available space, topography, however ideally, the features are constructed with sufficient bottom grade slope to permit gravity drainage. Discharge to the stream course is typically accomplished through constructed outlet works, such as culverts or gates.

Because these reservoirs generally have smaller storage capacity and less upstream to downstream water surface head differential than in-line reservoirs, they generally pose a lower risk to life safety, however, these impacts must still be considered. Changes to upstream and adjacent water surface elevations must also be investigated to ensure that the floodplain impingement does not cause additional flooding. Off-line reservoirs must conform to appropriate dam safety standards.

Storage of floodwater is impractical in this part of the basin due to the size of the upstream basin and required storage volume. Off-channel storage is typically only an option on small creeks and tributaries where the drainage basin is reasonably small. For instance, storing 10% of the 10 year flood volume along the Alabama River would require a 20 foot deep pond that would extend 10 miles by 10 miles on the surface. This would neither be practical to construct nor cost effective for several reasons. There is not enough available space within the adjacent locations along the Alabama River to construct a structure of the magnitude necessary to make a meaningful difference in flooding. Cost of acquisition and excavation of such land would be in the order of billions of dollars which would clearly far exceed the benefits that could be derived.  Therefore, this was screened out.

*Figure A-48:  On-Line vs. Off-Line Detention or storage of flood waters*



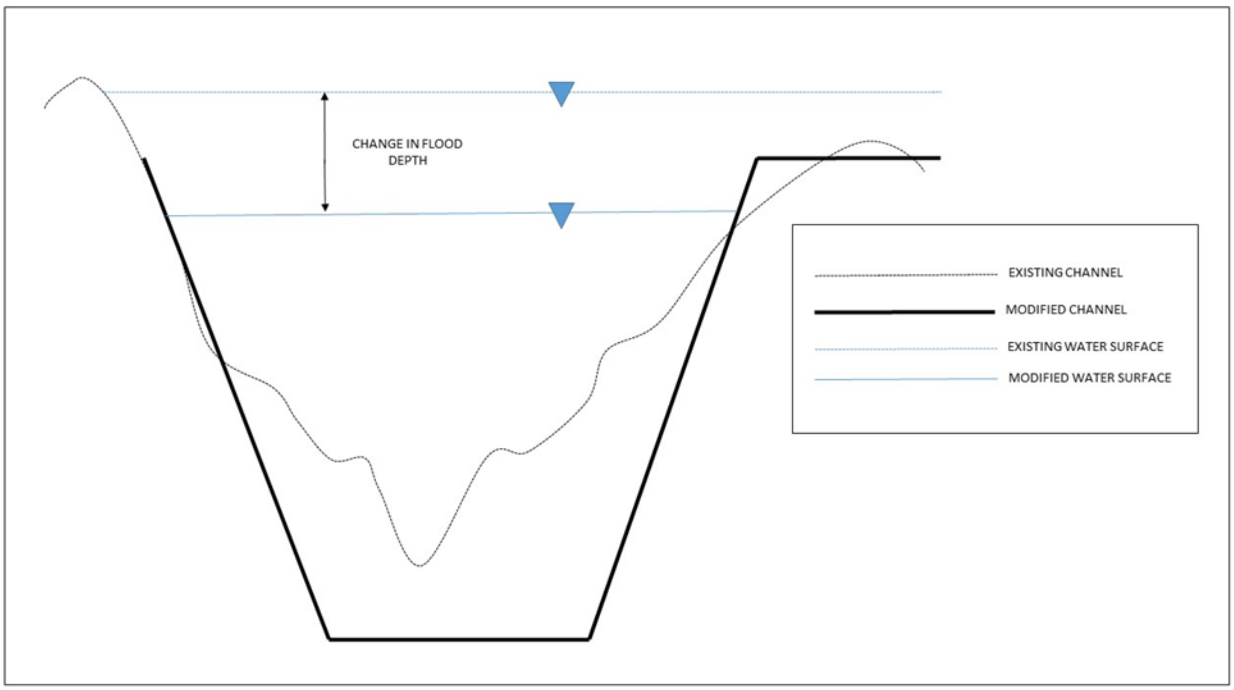

### *A.4.5.1.3.  Bridge/Culvert Modification*

Bridge/culvert modification is the modification to or replacement of a bridge or culvert to allow for increased flow capacity. This measure-type aims to lower upstream water surface elevations. Changes to downstream water surface elevations must be investigated to ensure that the increased flow capacity does not create additional downstream flooding. Costs associated with this measure are generally high. This measure was not pursued further as there appears to be no locations where bridge constrictions result in flooding of structures along the Alabama River. Possible small constrictions at bridges along several tributaries where identified however, there were very few or no structures in the upstream floodplain in these locations. Therefore, it was determined that bridge modification would not be a cost-effective measure and was screened out.

### *A.4.5.1.4. Channel Modification*

Channel modification consists of the enlargement of the stream channel to increase capacity and lower adjacent and upstream water surface elevations (**Figure A-49**). Changes to downstream water surface elevations must be investigated to ensure that the increased flow capacity does not create additional downstream flooding. Since flooding in the Selma area are typically from the mainstem Alabama River, it was determined a channel modification would be impractical. There were no constriction points identified which could be modified to effectively increase conveyance and reduce flooding. Furthermore, preliminary hydraulic modeling showed that increased storage in the Alabama River would not produce a meaningful reduction in peak river stages for flood events that affected structures in the study area.

***Figure A-49: Example schematic of channel modification to increase stream capacity and reduce flood depths***

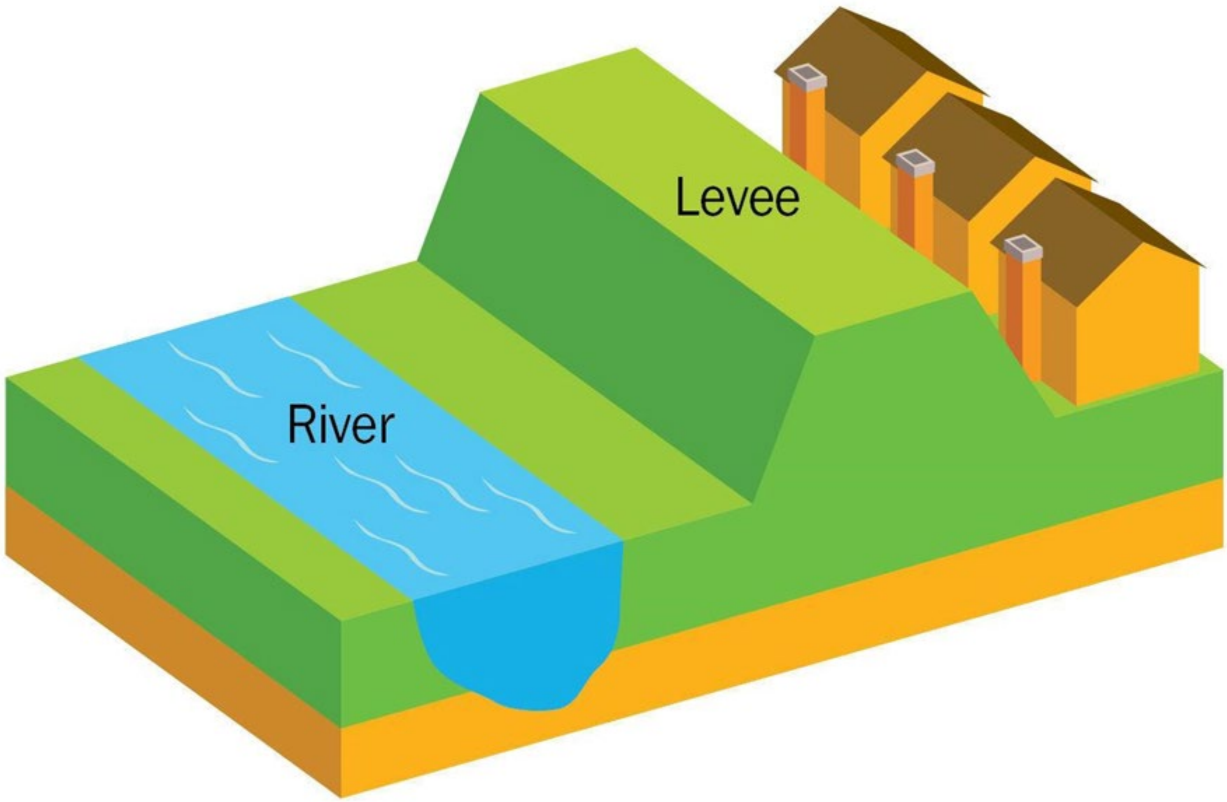



### *A.4.5.1.5. Levee(s)/Floodwall(s)*

Levees/floodwalls are usually constructed adjacent to the stream to protect low-lying areas from being inundated by floodwaters (**Figure A-50**). Levees/floodwalls are usually designed to withstand a low frequency event such as the .01 AEP event. Although levees can prevent flood water from impacting structures up to the design event, it is important to note that they do not eliminate the flood risk. Two reasons contribute to this: first, there is always a risk of a levee/floodwall failure, and secondly, flood elevations can exceed a levee's design crest elevation. Levees/floodwalls are usually expensive to construct and maintain.

Any use of levees/floodwalls must be carefully evaluated to ensure that a failure of the proposed levee/floodwall would not pose an increased risk to life safety. Changes to upstream and adjacent water surface elevations must also be investigated to ensure that the floodplain impingement does not cause additional flooding. Several levee options were considered for protection of Wards 1, 6 and 8 in the city of Selma. Site specific levee configurations are discussed below in the site-specific structural measures section.

*Figure A-50: Example of a constructed levee to protect inland structures adjacent and near a river*




### *A.4.5.1.6. Channel Diversion*

Channel diversions are used to convey floodwaters around segments of the natural channel that are vulnerable to flooding. This is usually done by creating a shorter, straighter overflow channel that moves water to a lower portion of the stream, or in some cases, takes water out of the basin and directs it to a different system (oftentimes this requires pump stations to move water across basin divides). Changes to downstream water surface elevations must be investigated to ensure that the modifications to the timing of the flow hydrograph do not create additional downstream flooding.

A diversion channel could possibly be feasible near the study area for small flood events below the 10-year flood. Floods in excess of 10-year floods would fully inundate the area available for channel diversions. Furthermore, the channel would have to be very large. For instance to divert just 10% of the 10 year flood peak, roughly 20,000 cfs, one would need to excavate a trapezoidal channel that was about 4.5 miles long, 100 feet wide at the bottom, 200 feet wide at the top and 12 feet deep. Also, preliminary model results showed there would be issue with backwater flow of the channel due to the very flat terrain that would further reduce its capacity to convey flow during even minor floods. Channel diversion was considered for this study, but this measure was determined to be impracticable based on expected costs, and because there are very few structures at risk for frequent floods, which suggests associated benefits would be very low. A potential channel diversion location and configuration for the Alabama River at Selma, AL is shown on **Figure A-51**.

***Figure A-51: Potential channel diversion for the Alabama River near Selma***

#### *A.4.5.1.7. Sluice Gate(s)*
Sluice gates are hydraulic structures that can be opened and closed to control the flow of water through an opening. These structures could be used to prevent flood waters from backing up tributaries that feed main stem rivers. During high flow events, as the river rises water may begin to flow up local tributaries causing flooding from the backwater effect of the main river.

### *A.4.5.2. Non-Structural Measures*

#### *A.4.5.2.1. Flood Response Plan*
A Flood Response Plan can provide residents a comprehensive plan to direct evacuations of areas forecast to experience flooding. A properly utilized plan can also provide adequate time to prepare and move out of flood prone areas with the assistance of employing flood forecasting. This plan would assist an area in directing the evacuation of

residents based on certain forecasted flood elevations and would include recommended locations to be evacuated, safe evacuation routes and identification of locations that would be inaccessible.

#### *A.4.5.2.2. Land Use Regulations*

Land use regulations can be implemented to prevent future construction in the floodplain; however, because the flooding comes from the basin above Selma, land use regulation changes in the study area would have little effect on the flooding cause by the Alabama River.

#### *A.4.5.2.3. Acquisition/Buy Outs & Relocation Assistance*

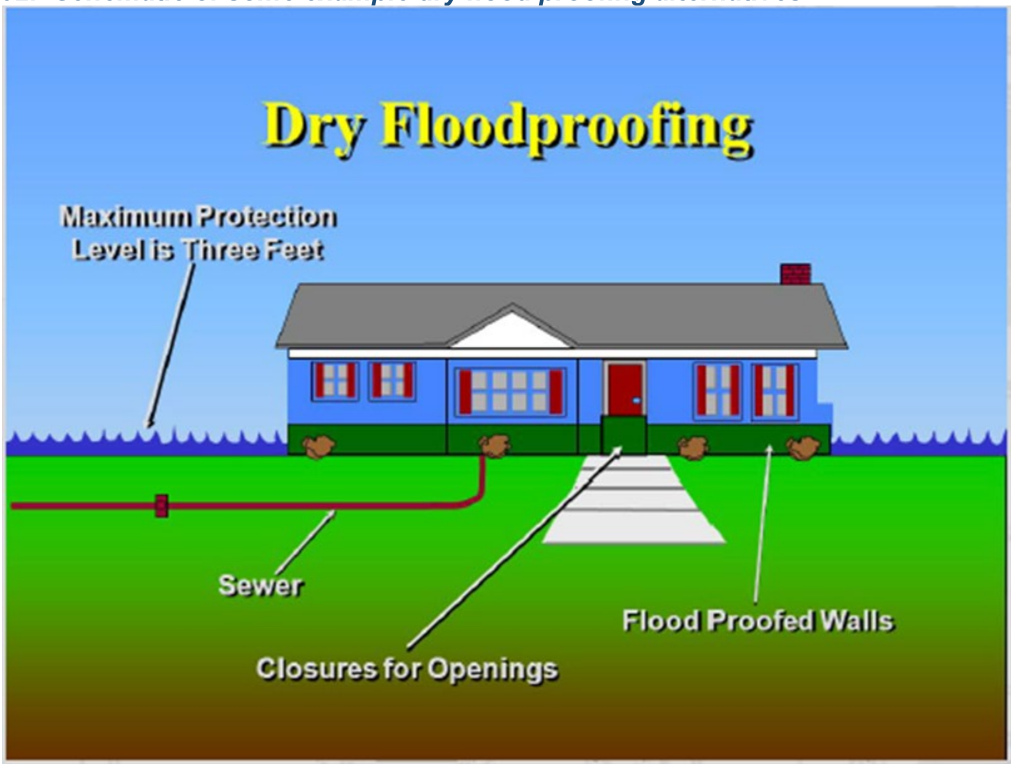
Structures within a specific frequency floodplain can be acquired and occupants can be relocated to areas outside of flood zones. This is the only measure that completely removes risk from the floodplain, as the structures are demolished after the acquisition is complete, and the property cannot be redeveloped.

#### *A.4.5.2.4. Flood Proofing of Structures*

Floodproofing typically involves constructing building walls and openings to create a water tight barrier. Some options for dry floodproofing structures include measures such as installing closures for openings (i.e., doors and windows) and flood proofed walls that would be impervious to flood waters (**Figure A-52**). For dry floodproofing, it is important only to use these alternatives if the building can withstand the hydrostatic pressure of flood waters without failing.

There are also a variety of measures which can reduce building damage, while allowing the structure to flood (i.e. wet floodproofing). The building must be properly anchored and ballasted to combat buoyant forces and should include flood drains to allow water to flow in and out of the building without causing damage to the foundation. Additionally, all electrical outlets and utilities should be elevated above the anticipated flood elevation or appropriately protected.

*Figure A-52: Schematic of some example dry flood proofing alternatives*




### *A.4.6. Site Specific Measures Considered for the Array of Alternatives*

### *A.4.6.1. Site Specific Structural*

#### *A.4.6.1.1. Levees/Floodwalls*

Additional site specific levee alignments were initially considered during previous study phases and some were screened out early on due to circumstances such as the alignment being outside the study area (e.g., L1 option). For the purposes of this study and report, four levee alignment alternatives were evaluated to reduce flood damages within the Study Area including alignments or options such as 1967 Selma Levee with Selmont Levee (USACE, 1967) alignment with floodgates/pumps where needed; a shortened/optimized levee version of this alignment; and a U.S. Highway 80 tie-in and floodgates/pump stations where needed (**Figure A-53**). Three of the four alignments considered specifically focused on flood damage reduction within Ward 8. In general, the levees evaluated would largely consist of an earthen structure with 3:1 side slopes, a top elevation around 121 feet, and a height that typically ranges from 5 to 12 feet.

#### *A.4.6.1.1.1. L2 Option*

This alignment focused on the Selma portion of the 1967 levee (USACE, 1967). Preliminary professional judgment determined that this alignment would not provide additional benefits as compared to L3 and would cost a substantial amount more. Therefore, this alignment was not selected as the optimized footprint.

#### *A.4.6.1.1.2. L3 Option*

Alignment L3 footprint ran across the southern portion of Ward 8 with a tie-in feature to U.S. Highway 80. Review of modeled profiles showed that U.S. Highway 80 could withstand flooding up to the 0.1 AEP flood event with added features such as clay revetment and floodgates. This design was the least costly levee alignment while protecting the same amount of structures as the L2 and L5 options; therefore, this footprint was chosen as the optimized levee alignment.

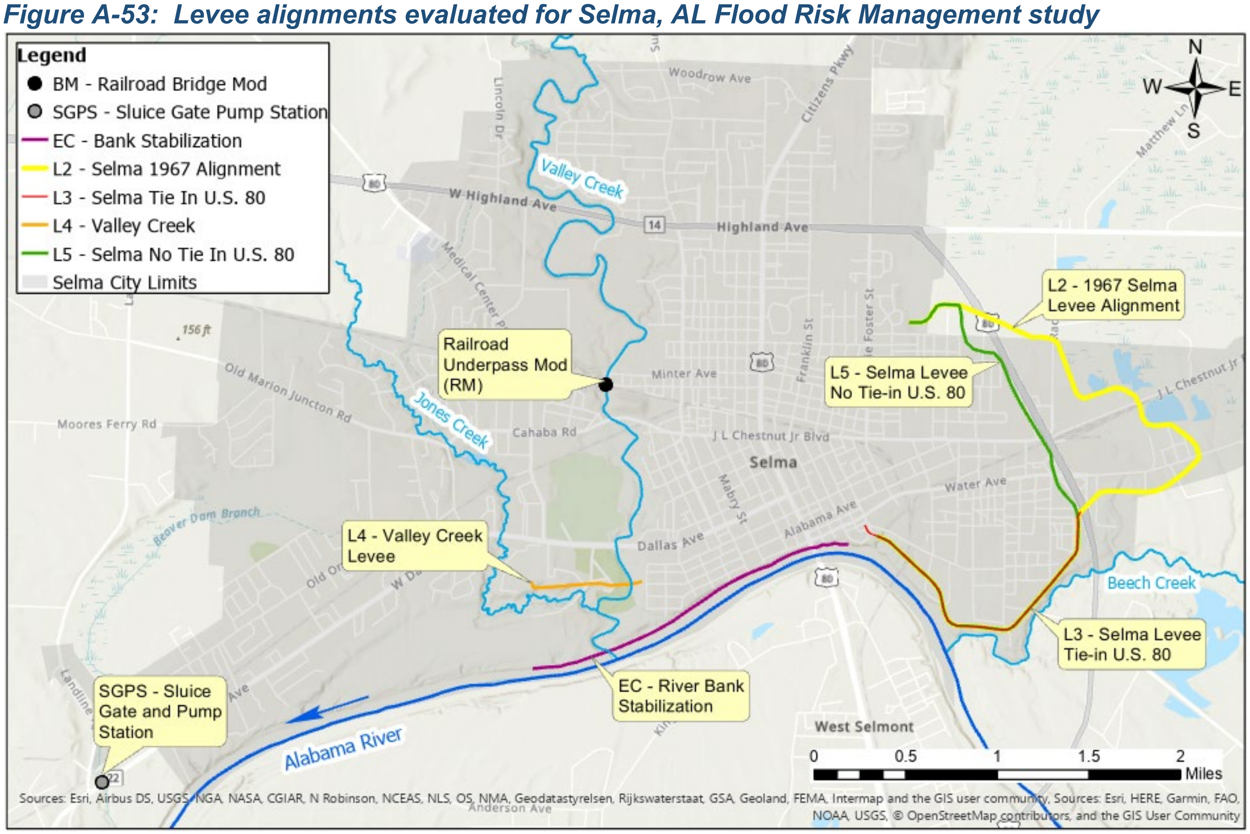
### *A.4.6.1.1.3. L4 Valley Creek*

This measure includes a levee across Valley Creek that would have prevented backwater flow from the Alabama River up Valley Creek. The levee would have been located just above the confluence with Jones Creek and include a sluice gate to restrict flows during a large flood event on the Alabama River. Flood mapping showed that, of the structures within the Valley Creek floodplain, very few were affected by the 0.01 AEP or less severe flood event. This measure was ultimately screened out in the evaluation of the initial array of alternatives due to the lack structures in the Valley Creek floodplain.

### *A.4.6.1.1.4. L5 Option*

The footprint of L5 was planned similar to L3; however, the levee ran parallel with U.S. Highway 80 rather than utilizing a tie-in feature. Like L2, preliminary professional judgment determined that this alignment would not provide additional benefits as compared to L3 and would cost a substantial amount more. Therefore, this alignment was not selected as the optimized footprint.

*Figure A-53: Levee alignments evaluated for Selma, AL Flood Risk Management study*



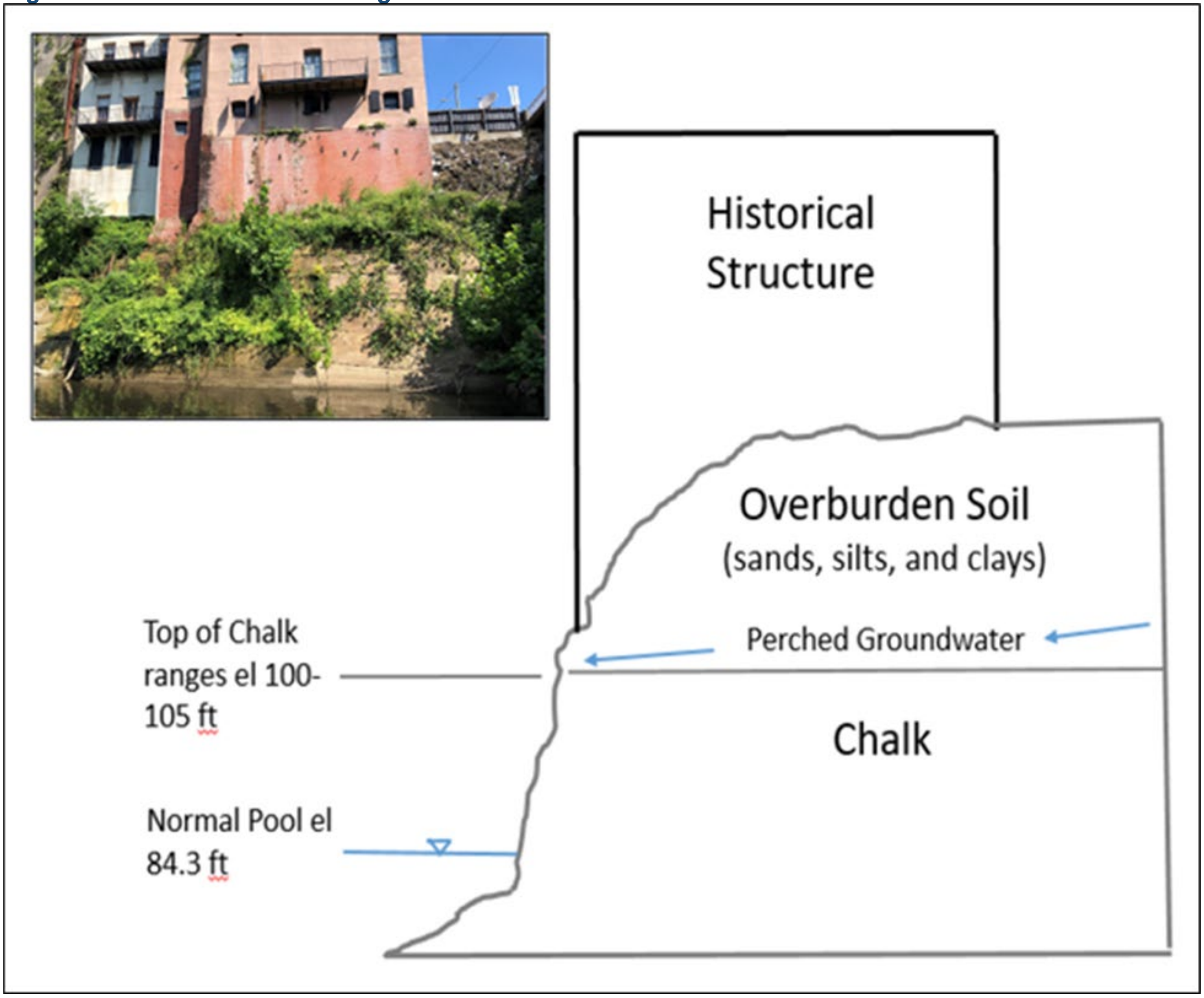

### *A.4.6.1.1.5. Sluice Gate and Pump Station (Beaver Dam Branch)*

This measure was considered to prevent backflow of water from the Alabama River up the Beaver Dam Branch tributary into communities in Ward 1. This consisted of a sluice gate located under Dallas Avenue that could be closed during high flows along the Alabama River. A pump station would prevent the backup of water along the tributary. This was carried forward into the initial array of alternatives, but quickly screened based on the limited number of structures affected in Ward 1.

##### *A.4.6.1.2. Bank Stabilization*

As previously discussed in **Section A.4**, this study included non-traditional flood risk management measures such as bank stabilization which were included in the alternative analysis. bank stabilization was considered due to the benefits of protecting existing buildings along the riverbank from structural failure that could result in the buildings collapsing into the Alabama River. Many historical buildings are situated along the riverbank between Franklin and Church Streets. Their foundations appear to be set in the overburden alluvial deposits, with little to no soil coverage on the riverside of the foundation. The chalk is somewhat impervious, causing concentrated groundwater to exit the bank slopes within the overburden material as this layer becomes saturated. This continual process could potentially result in material loss beneath the building foundations which, over time, would destabilize the buildings. **Figure A-54** shows a generalized cross section of the geology of the riverbank.

*Figure A-54: Illustrated existing condition cross section of the downtown Selma bluffs*




The interface of the overburden soils and underlying chalk fluctuates from approximate elevation 100 to 105 ft in the Study Area. When comparing this to river elevation, it puts the boundary of the two layers approximately 15 to 20 ft above the normal pool level of 84.3 ft. According to historical hydrologic data, this layer would see loading due to the river cresting at around the 0.5 AEP (2-year) flood event. This a fairly frequent loading and shows that minor flooding of the River could contribute to the building instability.

In addition to flooding, there were other possible contributors of building instability that are not linked to flooding. Historical and current photos show that there is a history of allowing vegetation to grow in the slopes where the building foundations are set. At times, this vegetation appears to have been removed, allowing for root systems to rot, and thus, allowing voids within the foundation soils to form.

### *A.4.6.1.3. Bank Stabilization Options*

This option consists of measures used to stabilize the riverbank and protect structures such as buildings located along the river bank from experiencing failure due to erosion. Construction methods, presented as “options”, included a range of river shoreline stabilization techniques that were based on similar USACE projects. Bank stabilization methods considered are further described in the below sections.

##### *A.4.6.1.3.1. Bank Stabilization Option 1, Sheet Pile Wall*

This option consists of driving sheet piles into the ground to form a continuous wall. The sheet pile would be driven to the necessary embedment as determined by design. Additionally, dependent upon the final configuration, the sheet pile wall would likely require tie backs at a set spacing along the wall, anchored into the existing earth on the inland or dry side of the wall.

Vibrations from the placement (driving) of the sheet-pile wall could affect existing structures and foundations which could lead to failure of the structures. Contractors may be reluctant to assume the liability for this construction method. Because this variant of the alternative could negatively impact the stability of the historic structures along the bank, this option was screened from further evaluation and comparison.

##### *A.4.6.1.3.2. Bank Stabilization Option 2a/b, Riprap and/or Extension*

This option consists of reinforcing the bank by providing a large amount of riprap/large stone to the existing bank, creating a more gradual slope that extends out into the river.

This construction method presents both constructability and aesthetic concerns. This method would require a severe setback and the toe would extend far into the Alabama River, which would cause navigation impediments. As such, this configuration was screened out from further analysis.

##### *A.4.6.1.3.3. Bank Stabilization Option 3, Cast in Place*

This option consists of dewatering, excavating, prepping the foundation, constructing formwork, and pouring a continuous cast-in-place concrete wall along the length of bank to be stabilized.

Although it would be aesthetically pleasing, the requirement for coffer dams and dewatering would add a significant amount to the cost of construction. Environmental impacts resulting from the dewatering would also be substantial. Therefore, this configuration was screened out from further analysis

##### *A.4.6.1.3.4. Bank Stabilization Option 4, Solider-Pile Wall and Riprap*

Bank stabilization utilizing a soldier pile wall consists of installing intermittently spaced piles (i.e., soldier piles) into the ground surface, which form part of the main structural resisting system. As opposed to the driving method of embedding the sheet piles, soldier piles can be installed into pre-drilled holes and grouted in-place. Horizontally spanning members, commonly referred to as lagging, span between the soldier piles and collect most of the retained earth pressures which are then transferred to the soldier piles. Riprap can be used at either end of the wall structure to help protect from erosion and scouring. Additional riprap may be used as the wall continues under the Edmund Pettus Bridge to protect bridge abutments from scouring.

Since driving the piles can be avoided, construction of a soldier pile wall and placement of riprap is not likely to affect existing structures and foundations. It also presents the least environmentally damaging impacts to natural resources, cultural artifacts, and

Unexploded Ordnances (UXO(s)). Therefore, this configuration was selected as the Bank Stabilization structural design for Alternative 4.

### *A.4.6.2. Site Specific Non-Structural*

#### *A.4.6.2.1. Buyout 1 option of 300 parcels:*

Buyout 1 consists of the buyout and removal of 300 structures in Ward 8. **Figure A-55** below shows the areas buyouts were considered.

For owner-occupants, Housing of Last Resort will ensure availability of DSS housing, notwithstanding cost implications. For tenant-occupants, preliminary market research has indicated a shortage of DSS rental accommodations that would be in the financial capability of those displaced, and within the general project area. This negatively impacts the ability to implement this variant of the alternative. In the opinion of USACE - RE, the City of Selma does not have sufficient manpower to manage and/or execute this level of relocation assistance/buyout IAW P.L. 91-646. This option was screened out as a possible component of the Recommended Plan.

#### *A.4.6.2.2. Buyout 2 option of 157 Parcels:*

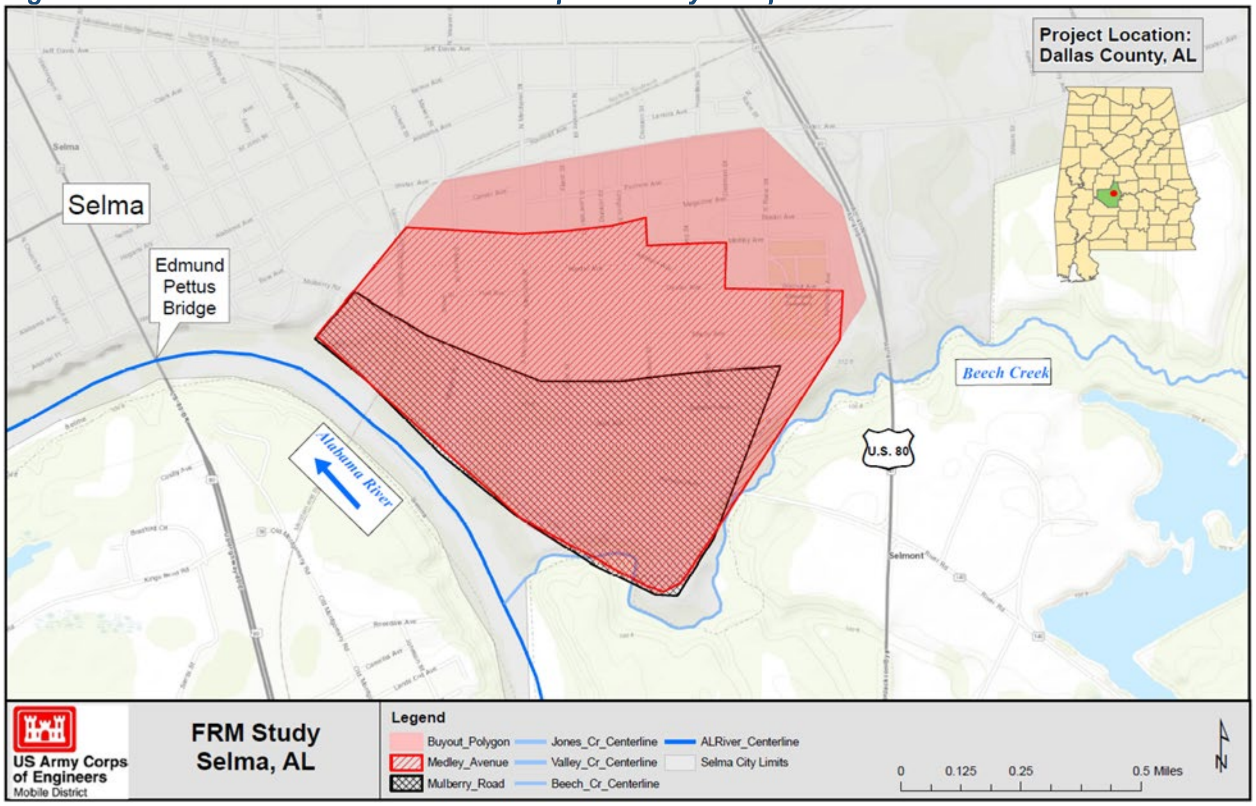
For owner-occupants, Housing of Last Resort will ensure availability of DSS housing, notwithstanding cost implications. For tenant-occupants, preliminary market research has indicated a shortage of DSS rental accommodations that would be in the financial capability of displaced and within general project area. This negatively impacts the ability to implement of this variant of the alternative. In the opinion of USACE - RE, City of Selma does not have sufficient manpower to manage and/or execute this level of relocation assistance/buyout IAW P.L. 91-646. This option was screened out as a possible component of the Recommended Plan.

#### *A.4.6.2.3. Buyout 3 option of 25 Parcels:*

Buyout 3 consists of buyout of 25 parcels in Ward 8. Since 29 owners would be involved, and several of these would involve non-residential displacements, hypothetically a P.L. 91-646 involuntary relocation may be plausible. Nevertheless, shortage of DSS tenant-based housing would be a prevailing issue impacting the project's schedule, as well as questions regarding the capability of the City of Selma to execute the plan in accordance with P.L. 91-646. Discussions are currently underway with City Attorney/Planning Office regarding level of City's Capability. In the opinion of USACE-RE, the City would require additional specialized manpower to implement a P.L. 91-646 buyout/relocation alternative. Contractor resources could be used, but the City would be responsible for execution of all P.L. 91-646 provisions. This option was incorporated as a possible component to the Recommended Plan.

Pursuant to an analysis of the prevailing rental markets in the City of Selma conducted in December 2019, the market survey indicated an inadequacy of available Decent, Safe, and Sanitary (DSS) tenant dwellings, effectively rendering a larger-scale buyout effort nonviable to effectively execute within the City in accordance with the requirements of the Uniform Relocation Act, P.L. 91-646, as amended.

*Figure A-55: Areas considered for each respective buyout option*




### *A.4.7. Initial Array of Alternatives*

The "future with project condition" is the most likely condition expected to exist in the future if a specific project is undertaken. A total of ten alternatives based on the site specific measures discussed above were considered for the Selma Flood Risk Management Study. Of these, three were structural, one was nonstructural, and the remaining seven were combinations of structural plans with the nonstructural plan. In addition, a Flood Response Plan could be tailored to any of these alternatives to further address responsible floodplain management and life safety risk. The nonstructural plan did not include a recreation plan in the initial array. A description of the alternatives is listed in **Table A-12** below.

*Table A-12: Initial Array of Alternatives*

| Array of Alternatives | Plan Description |
|---|---|
| **No Action Alternative (NAA)** | No Federal undertaking would occur and the results would be consistent with FWOP conditions. |
| **Alt. 1: Non-Structural (A-Buyouts, B-Raise Structural Elevation, Structural move)** | There are two (2) non-structural options considered for the same group of structures. Alternative 1.A includes buyouts which entails the acquisition of parcels, relocation of inhabitants, and demolition of structures. Alternative 1.B includes elevating structures and relocations within Ward 8. |

| Array of Alternatives | Plan Description |
|---|---|
| **Alt. 2: 1967 Selma Levee** | 1967 Selma Levee with Selmont Levee alignment with floodgates/pumps where needed, and buyouts as necessary |
| **Alt. 3: Optimized (Short) Selma Levee** | Shortened/optimized levee alignment, U.S. Highway 80 tie in, floodgates/pump station where needed, and buyouts as necessary |
| **Alt. 4: Bank Stabilization** | Provide bank stabilization along all or part of RM 256-261 |
| **Alt. 5: Bank Stabilization + Buyouts** | Combines Alternatives 4 & 1.A-Buyouts. |
| **Alt. 6: Optimized Selma Levee + Buyouts + Bank Stabilization** | Combines Alternatives 3 & 4 & Partial Non-Structural Alt.1 in areas not within the Optimized Levee alignment |
| **Alt. 7: Optimized Selma Levee + Valley Creek Levee + Pump Station & Sluice Gate + Bank Stabilization** | Combines Alternatives 3 & 4 & a smaller levee at Valley Creek & a pump station with a sluice gate at Beaver Dam Branch (maximum structural protection) |
| **Alt. 8: Optimized Selma Levee + Valley Creek Levee + Buyouts + Bank Stabilization** | Combines Alternative 6 plus Valley Creek levee (only purchase, relocation or raising elevation in the Ward 8 considered) |
| **Alt. 9: Optimized Selma Levee + Valley Creek Levee + Buyouts** | Combines Alternative 3, levee at Valley Creek (purchase, relocation or raising elevation in the Ward 8 considered) |
| **Alt. 10: Optimized Selma Levee + Valley Creek Levee + Pump Station with Sluice Gate** | Alternative 7 with No bank stabilization (maximum structural protection without bank stabilization) |

### *A.4.7.1. Initial Screening*

The initial round of screening was presented at an IPR held June 26, 2019 and captured in a Memorandum to the Chief of Planning and Policy Division at South Atlantic Division dated August 1, 2019. As a result of this meeting, Alternative 1.B Elevating or Relocating of Structures out of Ward 8 was screened due to the age and condition of the structures. Levee alternatives 2 and 3 were also screened from further analysis, as preliminary professional judgment determined that these alignments would be cost prohibitive (both initial construction cost and maintenance), would not provide additional benefits, has the potential to have cultural and environmental impacts, and would likely induce flooding in the adjacent town of Selmont, Alabama. A number of recommendations for buyout options were made and it was requested that the PDT include recreation benefits as part of the array of alternatives. The PDT determined no additional benefits would derive from recreation in the proposed buyout area as Ward 8 is too far removed from the economic/tourism hub of downtown Selma. Further analysis of the economic/tourism benefits of downtown Selma is detailed in the Economic Appendix.

The team then further refined the remaining alternatives and identified sub-options for the buyout and bank-line stabilization alternatives that were presented at the IPR held October 9, 2019. Alternative 1.A was expanded to include sub-options for the removal of

25, 157, or 300 parcels; and a range of construction methods were presented for Alternative 4, each based on techniques employed at similar USACE projects. A more detailed discussion on the feasibility of each of the options considered are provided in **Appendix C – Economics**.

### *A.4.8. Focused Array of Alternatives*

After further refinement and screening of the initial array, the Focused Array of Alternatives was selected and is summarized in **Table A-13**.

*Table A-13: Focused array of alternatives evaluated for Selma Flood Risk Management study*

| Array of Alternatives | Plan Description |
|---|---|
| **No Action Alternative** | No Action Alternative |
| **Alt. 1: Non-Structural (NS-1-Buyouts)** | The non-structural measures would be optimized through cost evaluations and viewpoints. |
| **Alt. 2: Optimized Selma (1967) Levee (L3)** | Levee tying into existing road (L3) |
| **Alt. 3: Optimized Selma Levee + Non-Structural Measure** | Combines Alternatives 2 & Partial Non-Structural Alt.1 in areas not protected by the Optimized Levee |
| **Alt. 4: Bank Stabilization** | 1500 feet of bank stabilization along Water Avenue |
| **Alt. 5: Bank Stabilization + Buyouts** | Combines Alternatives 1 & 4 |
| **Alt. 6: Optimized Selma Levee + Buyouts + Bank Stabilization** | Combines Alternatives 2 & 4 & Partial Non-Structural Alt.1 in areas not protected by the Optimized Levee |

#### *A.4.8.1. Screening of Focused Array*

The focused array of alternatives (Alternatives 1.A, and 2-6) were screened based on their ability to meet objectives, avoid/minimize constraints, adherence to the planning criteria, as well as their resiliency and sustainability. All alternatives received an equal preliminary comparison using Rough Order of Magnitude (ROM) costs, National Economic Development (NED), Regional Economic Development (RED), Environmental Quality (EQ), and Other Social Effects (OSE) analysis. Of the entire focused array, only Alternative 2 was screened from further analysis. Alternative 2 met the study objectives but did not avoid the study constraints, particularly, the City of Selma's ability to maintain a large levee system. More details are available in the Plan Formulation section of the main report. Furthermore, this alternative was screened because it was determined to be more costly and have the potential to induce greater environmental and cultural impacts when compared with Alternative 3.

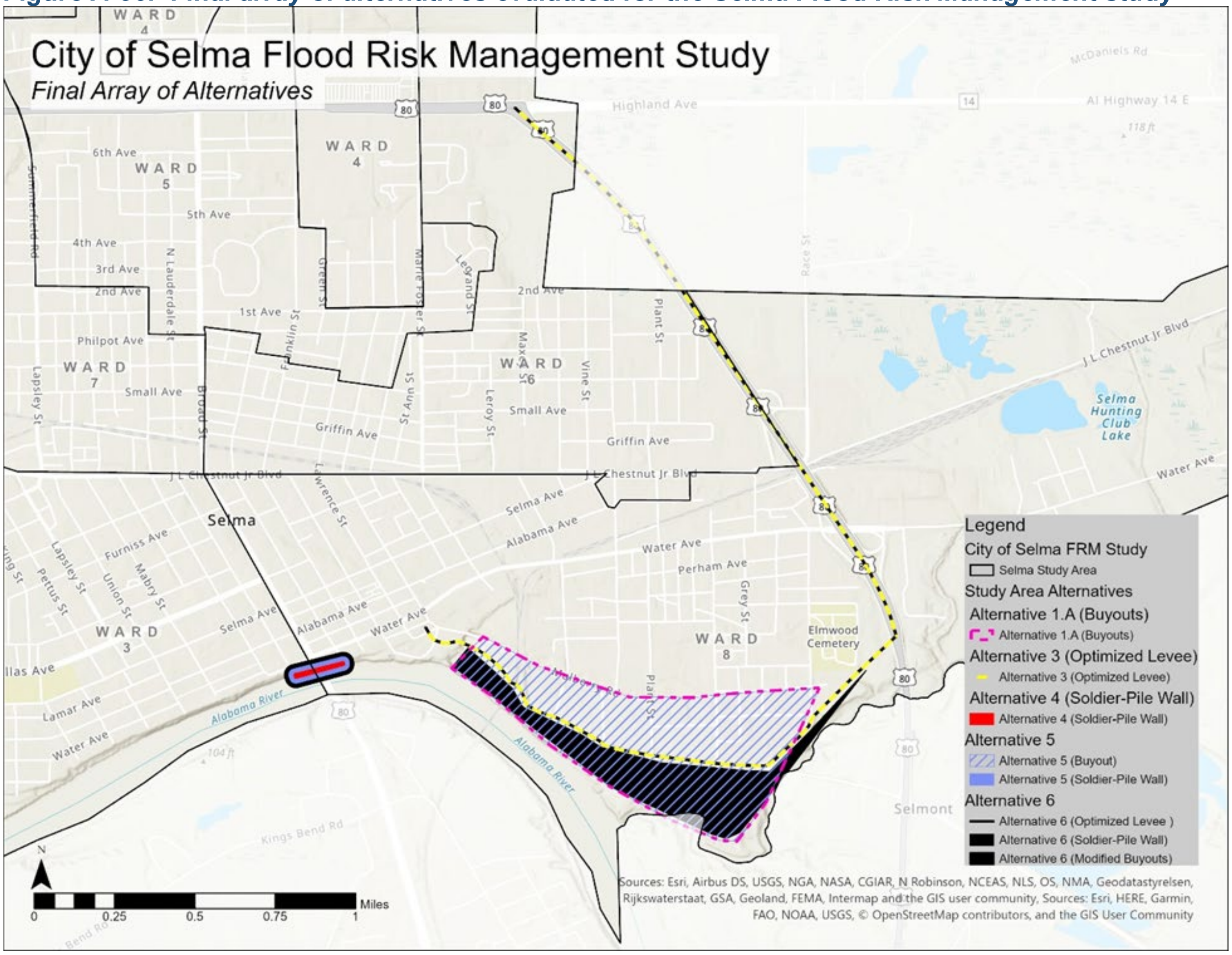
### *A.4.9. Final Array of Alternatives*

The final array of alternatives and locations are shown on **Figure A-56** and included the following:

- Alternative 1.A (Buyout);
- Alternative 3 (Optimized Levee);
- Alternative 4 (Bank Stabilization);

- Alternative 5 (Bank Stabilization and Buyout); and
- Alternative 6 (Combination of Alternative 1.A and 5, but with a modified buyout footprint to capture parcels within Ward 8 and outside the levee alignment).

*Figure A-56: Final array of alternatives evaluated for the Selma Flood Risk Management study*

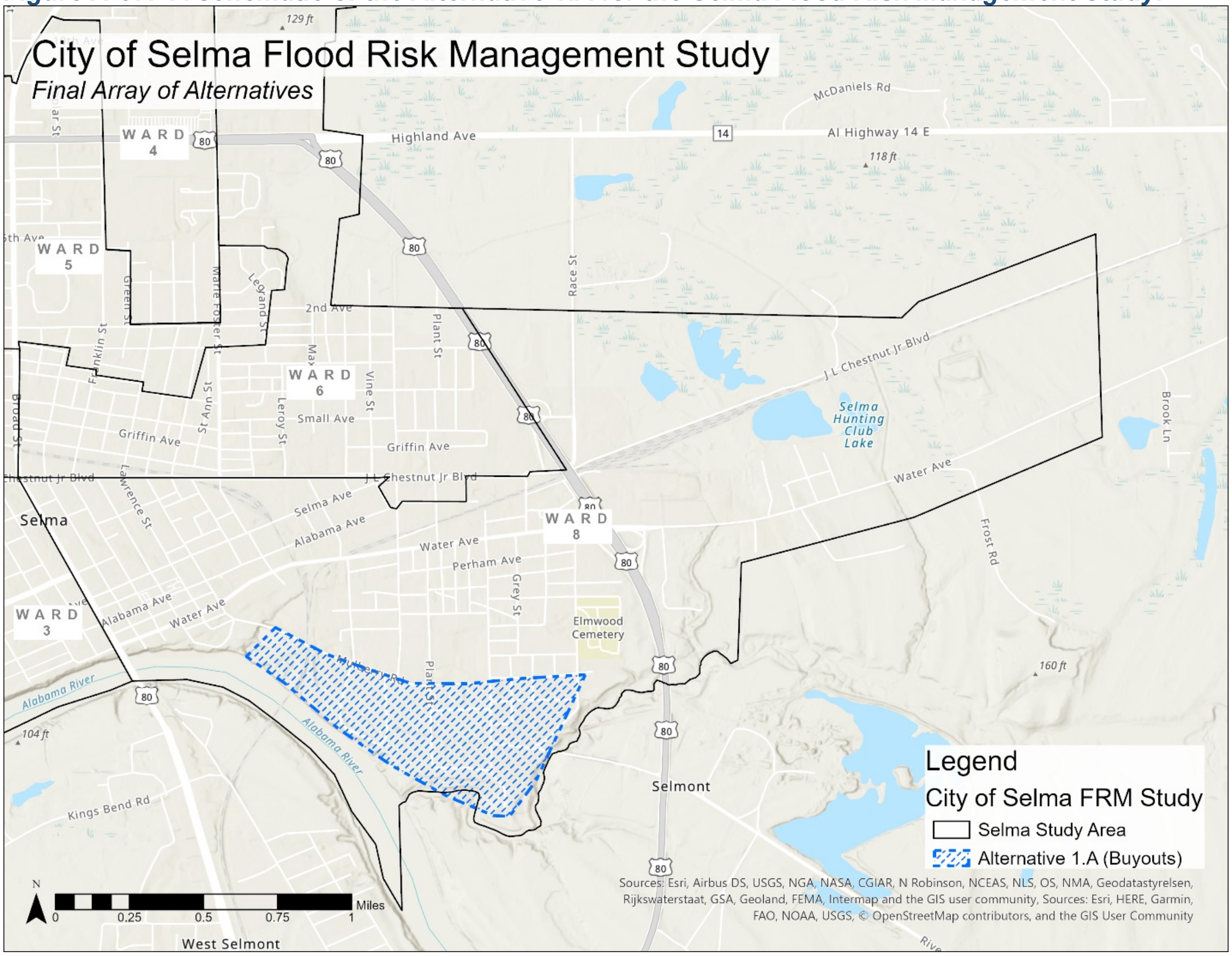



### *A.4.9.1. Description of Final Array of Alternatives*

#### *A.4.9.1.1. Alt. 1A: Buyouts*

Alternative 1.A schematic is shown on **Figure A-57** below. Approximately 25 parcels were identified within the buyout footprint encompassing approximately 170 acres. Implementation of this alternative would require acquisition of structures and relocation of inhabitants. Structures would then be demolished. Staging areas for demolition would be located within each parcel. This alternative would take approximately 2.7 months to complete.

*Figure A-57: A schematic of the Alternative 1.A for the Selma Flood Risk Management Study.*




### *A.4.9.1.2. Alt. 3: Optimized Levee Alignment*

Alternative 3 is an optimized levee with two components: new levee construction and Highway 80 revetment and reinforcement (shown on **Figure A-58**). The full alignment would include approximately 1.6 miles of new levee construction across the southern portion of Ward 8 and approximately 2.0 miles of Highway 80 revetment and reinforcement for a total of 3.6 miles. The base of the new levee within Ward 8 would span approximately 94 feet, which would require a construction footprint of approximately 18 acres. Two flood gates would be placed at intersections along Highway 80. **Table A-14** itemizes the quantities of fill material for each section of the alternative. Disposal areas would be required to place excavated material. Staging areas would also be required to contain all construction material necessary to build the levee and reinforce Highway 80; however, potential locations for this alternative have not been identified. This alternative would take approximately 21.5 months to complete.

*Figure A-58: A schematic of the Alternative 3 for the Selma Flood Risk Management Study.*

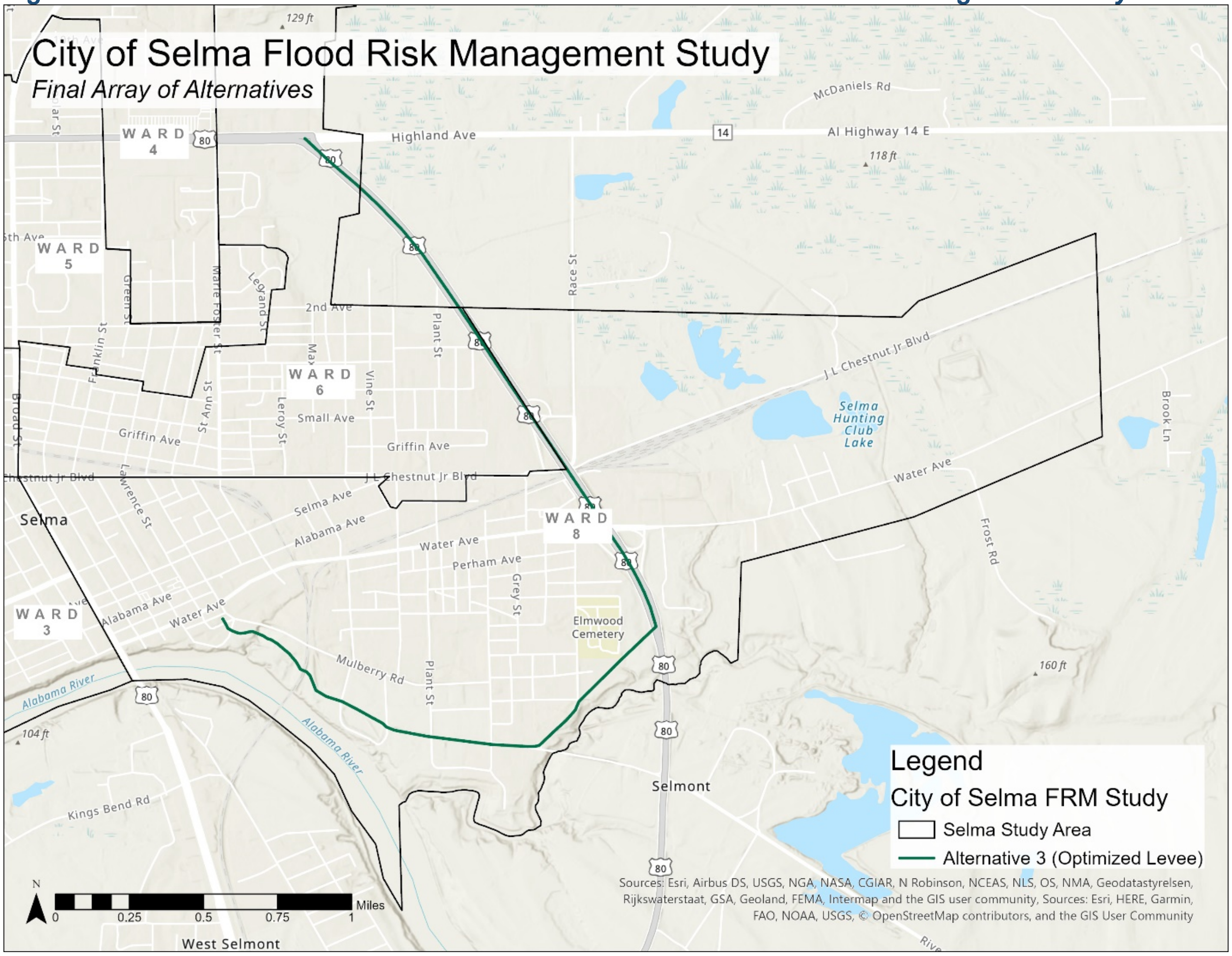



*Table A-14: Levee Alignment Fill Materials and Quantities in Cubic Yards (cy)*

| Material | Levee (1.6 miles) | Highway 80 (2.0 miles) |
|---|---|---|
| **Clay Core** | 80,592 cy | 40,000 cy |
| **Select Fill** | 241,777 cy | 60,000 cy |
| **Total Fill** | 322,369 cy | 100,000 cy |

### *A.4.9.1.3. Alt. 4: Soldier-Pile Wall*

Alternative 4 includes the construction of a new solider pile wall for the purpose of bank stabilization along the Alabama River bank near downtown Selma (**Figure A-59**). Staging and construction of the solider pile wall would occur from the Alabama River and a conceptual schematic of a soldier pile wall is shown on **Figure A-60**. **Table A-15** is a preliminary/conceptual estimation of materials and quantities necessary to construct the soldier pile wall. Preliminary structural calculations are enclosed in **Section A.8.2**. Approximately 96 H–Piles would be placed at approximately 8 feet on center throughout the proposed project area protecting approximately 1000 feet of bank. The H-Piles would be installed into pre-drilled holes and grouted/concreted in-place. One or multiple tiebacks would be required for each H-Pile, as determined by structural design. Reinforced precast concrete lagging wall panels will be placed between each H-Pile and riprap would cap each end to protect against scouring. Integral to the bank stabilization plan would be a drainage system constructed to address both seepage waters and flood waters behind the lagging wall. This drainage system would employ a very porous gravel backfill material (e.g., #57 gravel stone) behind the wall to adequately drain during river drawdown events and the use of filter/geotechnical fabric to prevent seepage waters from eroding upper horizon soils. The drainage system would include a sleeved header pipe extending parallel to the slope of the bank with laterals which outfall to the face of the lagging wall. The drainage system may be constructed at multiple levels as necessary. At this phase of the study it has not been determined if clearing and grubbing of the riverbank would be required; however, the maximum potential vegetation removal would encompass eight (8) acres. In total, this alternative would take approximately 18 months to complete.

***Figure A-59: A schematic of the Alternative 4 for the Selma Flood Risk Management Study.***

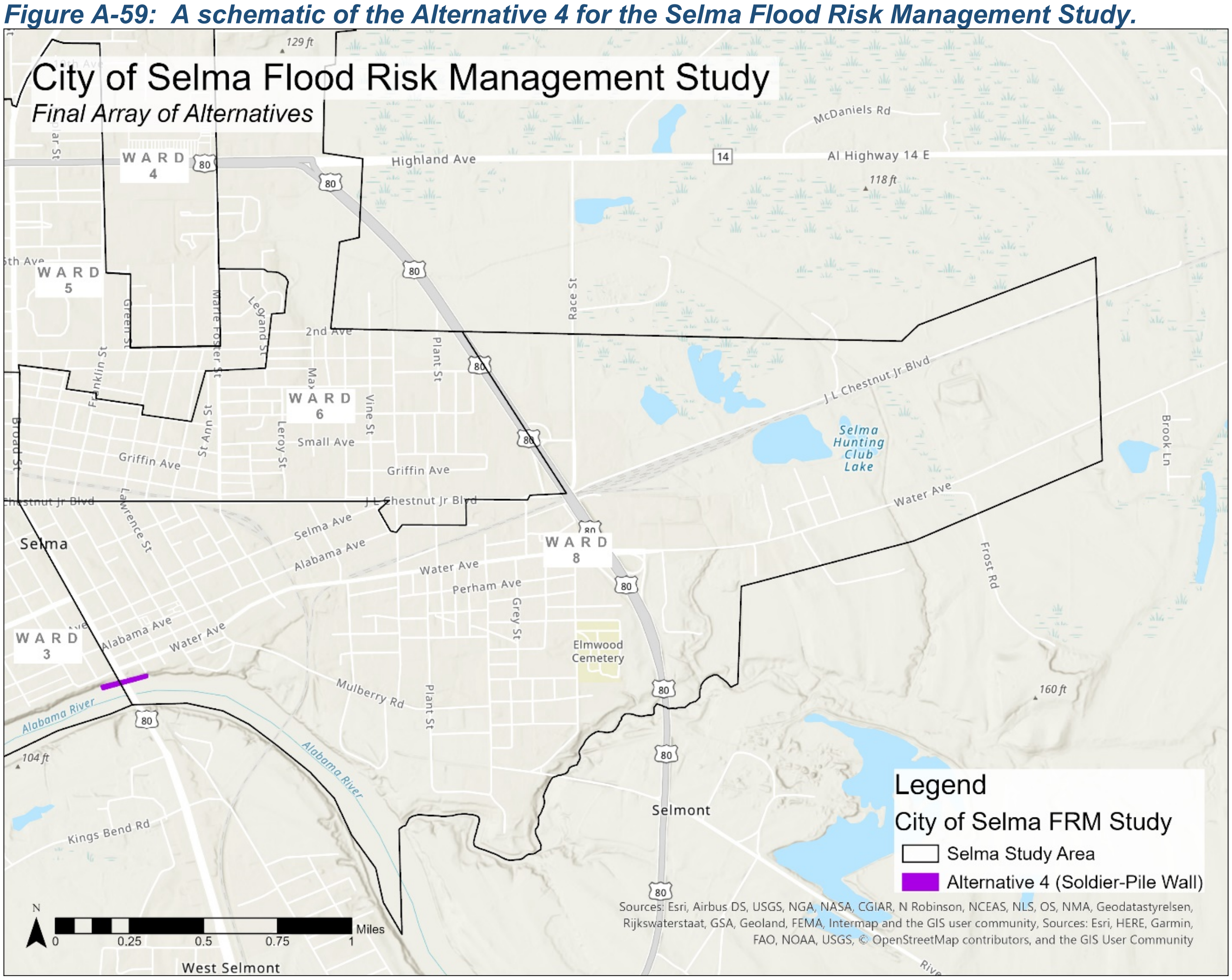
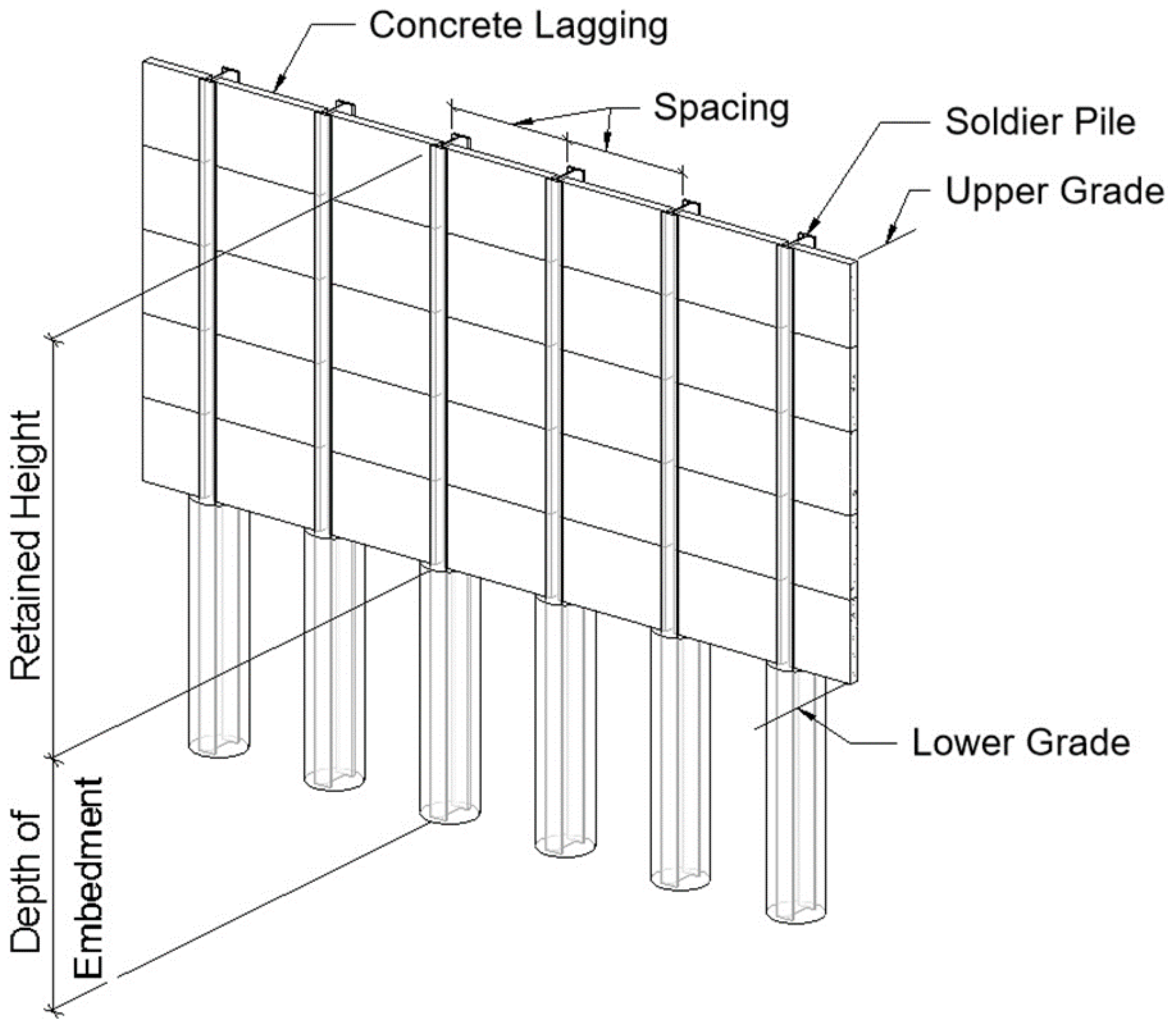



***Table A-15: Soldier-Pile Wall Materials and Estimated Quantities***

| Material | Soldier-Pile Wall (1000 Linear Feet) |
|---|---|
| **H-Piles (lengths vary from 10-ft to 50-ft)** | 96 (approximate) |
| **Steel Anchor Tiebacks** | 192 (approximate) |
| **Concrete Lagging** | 465 cubic yards (cy) |
| **Geotextile Fabric** | 10,000 square yards (sy) |
| **Granular Fill** | 12,500 cubic yards (cy) |

| Material | Soldier-Pile Wall (1000 Linear Feet) |
|---|---|
| **Riprap** | 3,333 cy |
| **Total Fill** | **15,833 cy** |

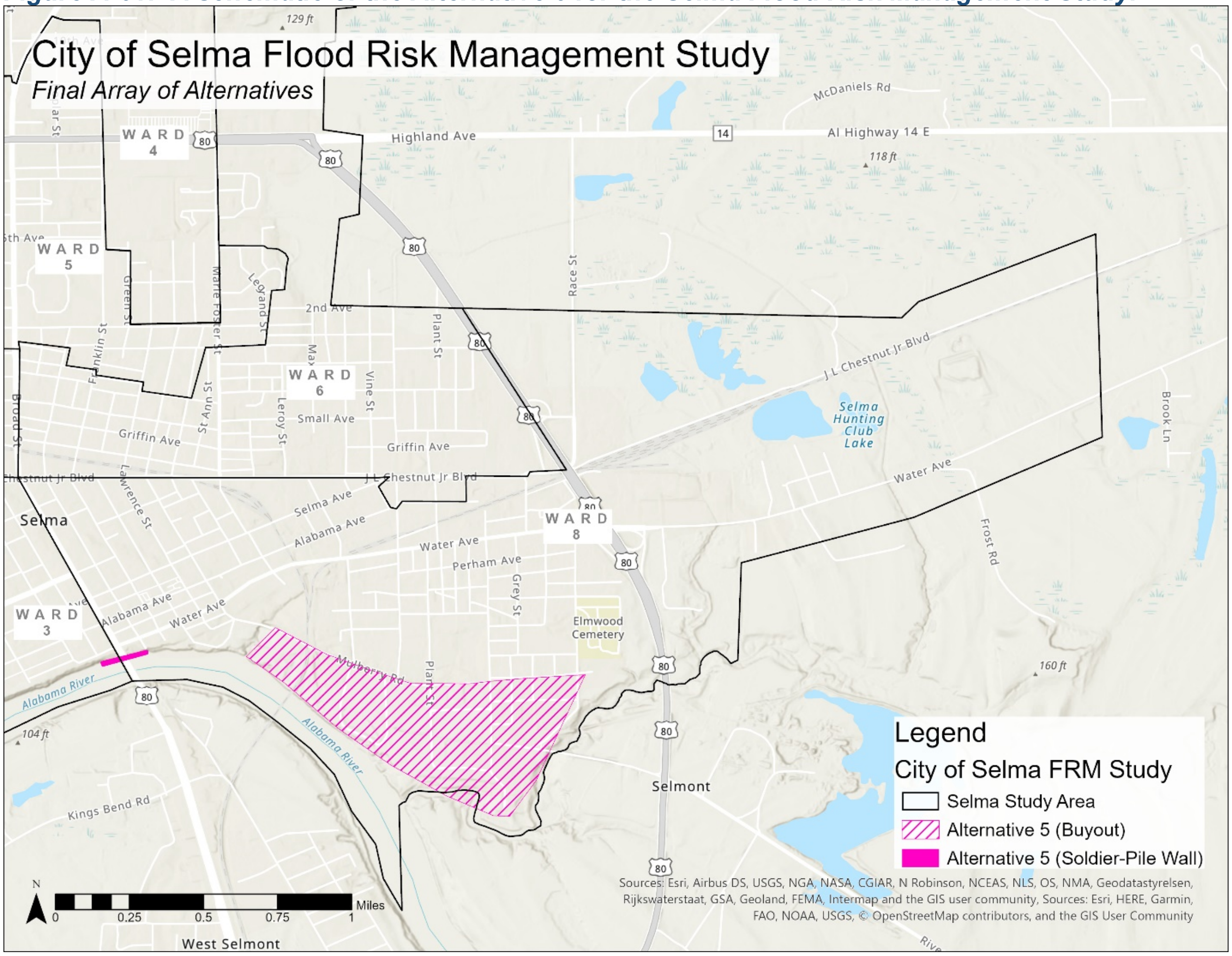
*Figure A-60: Example conceptual schematic of a constructed soldier-pile wall.*




### *A.4.9.1.4. Alt. 5: Bank Stabilization and Buyout*

Alternative 5 is a combination of Alternatives 1.A and 4. A schematic of the bank stabilization and area buyouts were considered is shown on **Figure A-61**. This alternative would take approximately 18.3 months to complete.

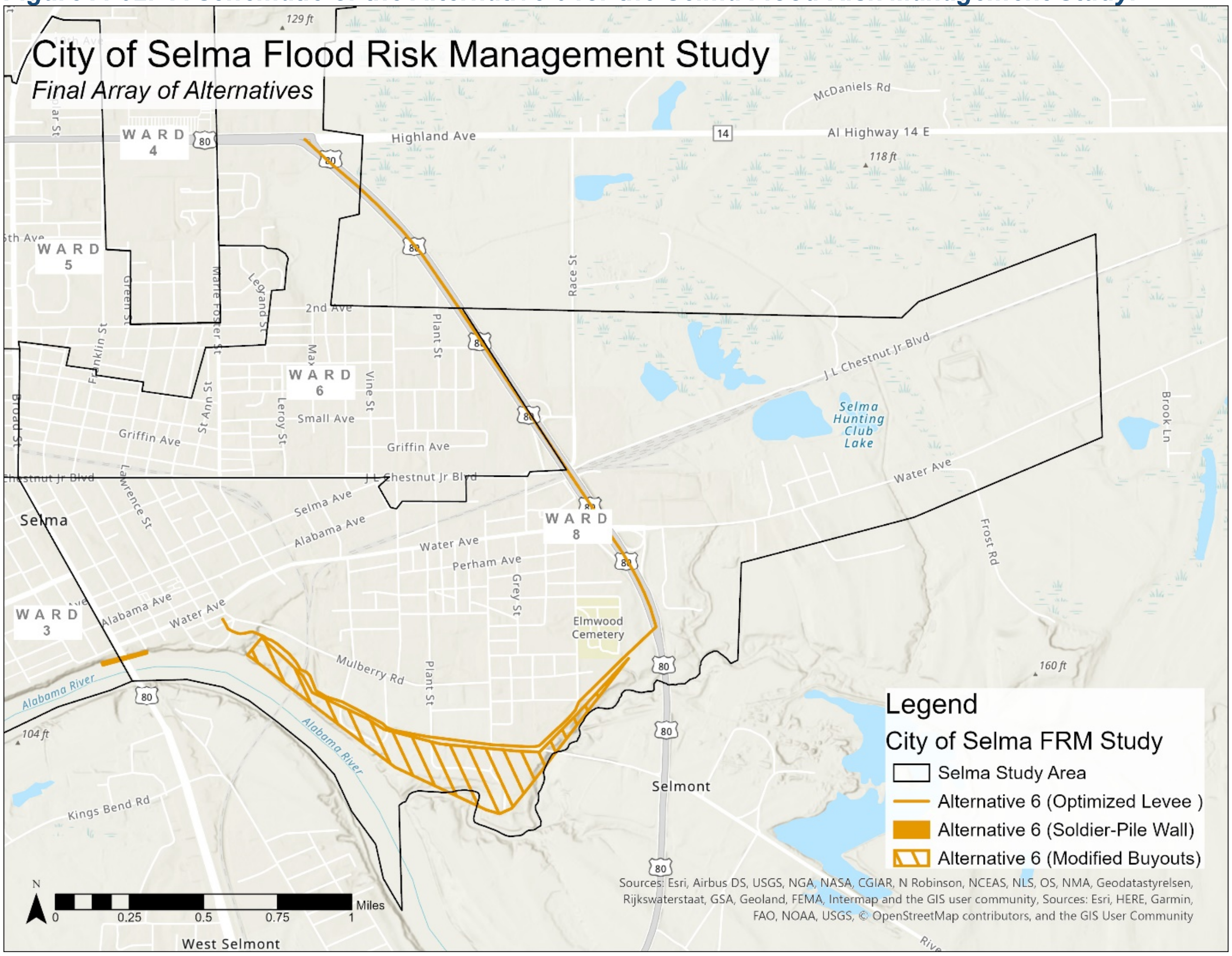
*Figure A-61: A schematic of the Alternative 5 for the Selma Flood Risk Management Study.*




### *A.4.9.1.5. Alt. 6: Optimized Levee Alignment, bank stabilization, and Buyout*

Alternative 6 is a combination of Alternatives 3 and 5 with the exception of differences in the buyout footprint to account for buyouts of structures outside the levee area along the Alabama River. A total of nine (9) parcels in Ward 8 identified within the proposed 68-acre buyout footprint for this alternative would be located outside the levee alignment. This alternative would take approximately 26.9 months to complete.

*Figure A-62: A schematic of the Alternative 6 for the Selma Flood Risk Management Study.*




### *A.4.9.2. Hydraulic Modeling of Final Array of Alternatives*

Hydraulic modeling of the final array of alternatives was performed to support the economics evaluation of any alternative tied to flood inundation of structures. There were five alternatives carried forward to the final array. Bank stabilization measures did not involve any additional modeling and were not evaluated based on reduced damages.

Of the five alternatives included in the Final Array of Alternatives, four (4) were directly tied to flood inundation including Alterative 1 (Buyouts), Alternative 3 (Optimized Levee), and Alterative 6, which is a combination of 2 measures. Of those alternatives, there were only two unique measures considered, including the Optimized Levee and Buyout Option 3. Hydraulic modeling of alternatives are discussed below.

#### *A.4.9.2.1. Alternative 1.A: Buyout Option 3 Modeling*

As discussed above, buyout option 3 consists of buying 25 parcels located near the bank of the Alabama River in Ward 8. When determining inundation for buyout alternatives, no additional hydraulic modeling was required. The future without project conditions modeling and inundation is utilized in HEC-FDA to determine damages with and without structures to determine economic benefit.

### *A.4.9.2.2. Alternative 3: Optimized Levee Modeling*

Modeling of the optimized levee alternative involves developing and performing a hydraulic model with modified terrain data to include the addition of the new optimized levee alignment and Highway 80 revetment. The full alignment would span approximately 1.6 miles of new levee construction across the southern portion of Ward 8 and approximately 2.0 miles of Highway 80 revetment and reinforcement for a total of 3.6 miles. This levee alternative was designed to provide complete protection for Wards 8 and 6 up to the 0.01 AEP event. This elevation corresponds to a minimum top elevation of 120.6 feet-NAVD88 with a top width of 10 feet and an average bottom width of 94 feet.

To model this alternative, the levee dimensions where burned directly into the terrain. Then a 2D area connection was modeled, connecting an internal 2D area to an external 2D area. The 2D area connection was modeled as a weir with a coefficient of 2.6. Sensitivity testing was performed on the weir coefficient using the values of 1.5 and 2.6 for comparison. The model output showed minimal increase in water surface elevation around 0.01 feet for overtopping events. **Figure A-63** shows a schematic of the levee burned into the terrain. **Figure A-64** shows the weir used to model the levee crest.

*Figure A-63: Image depicting the burned in levee alignment for the L3 Optimized Levee*

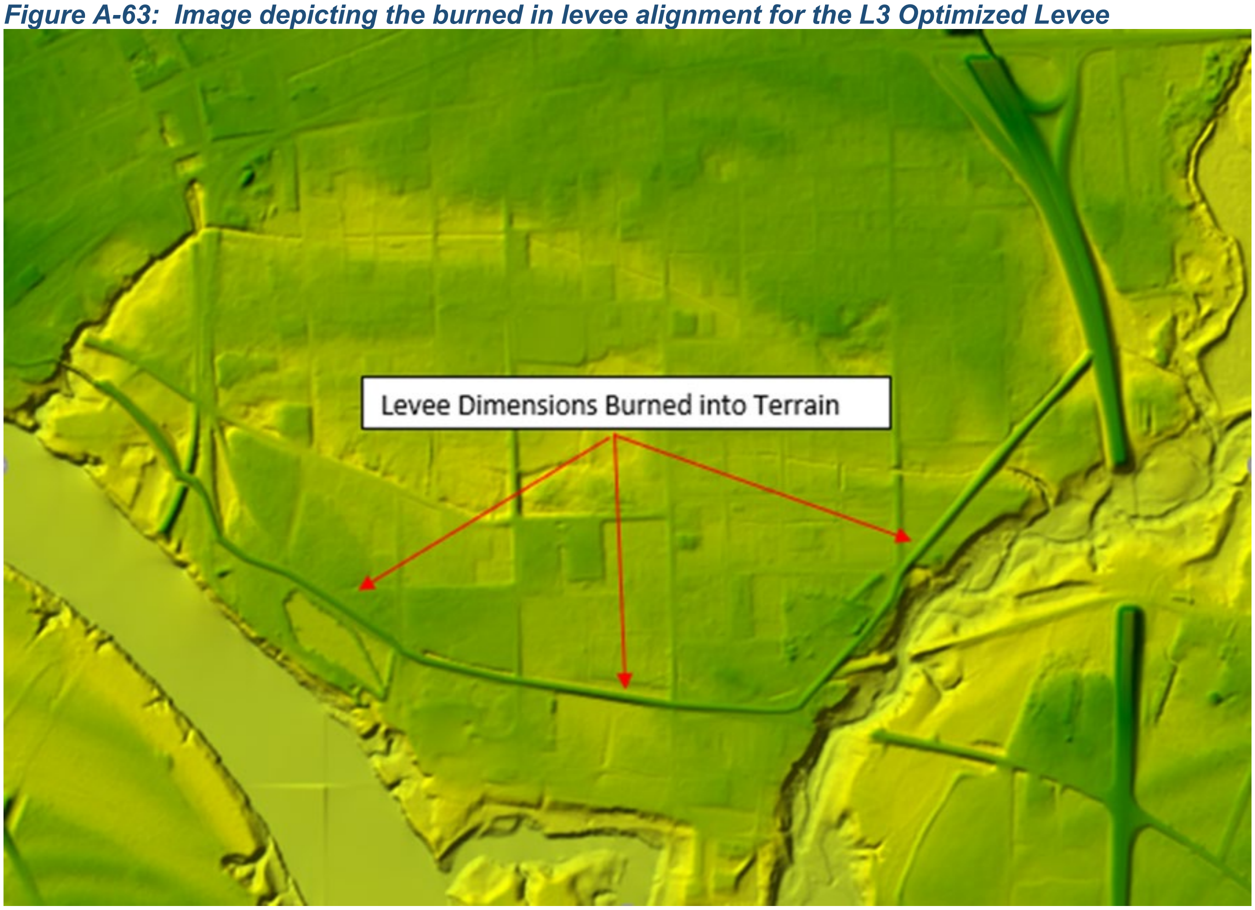


*Figure A-64: The 2D area connection used to model the L3 Optimized Levee*

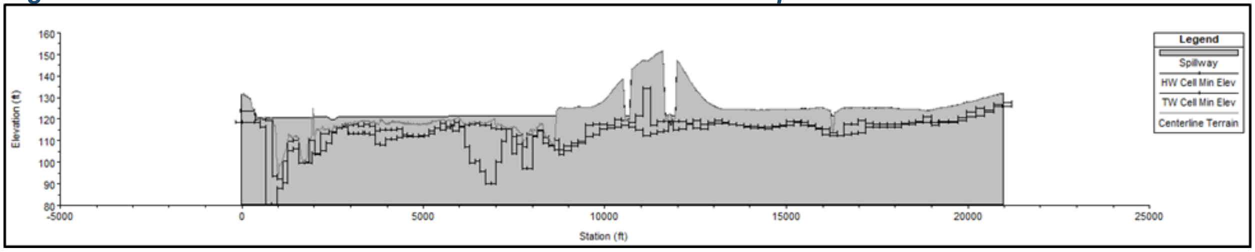
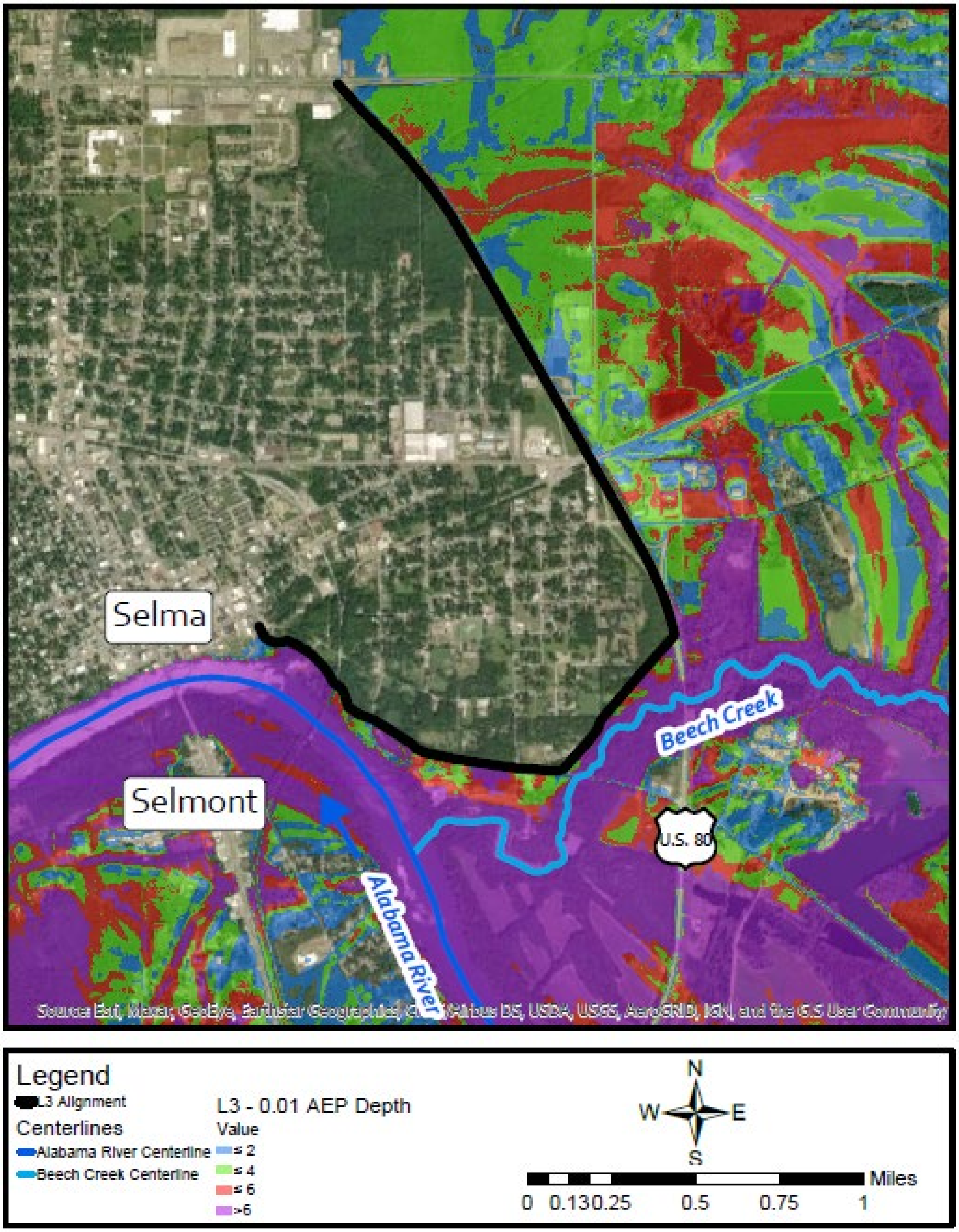
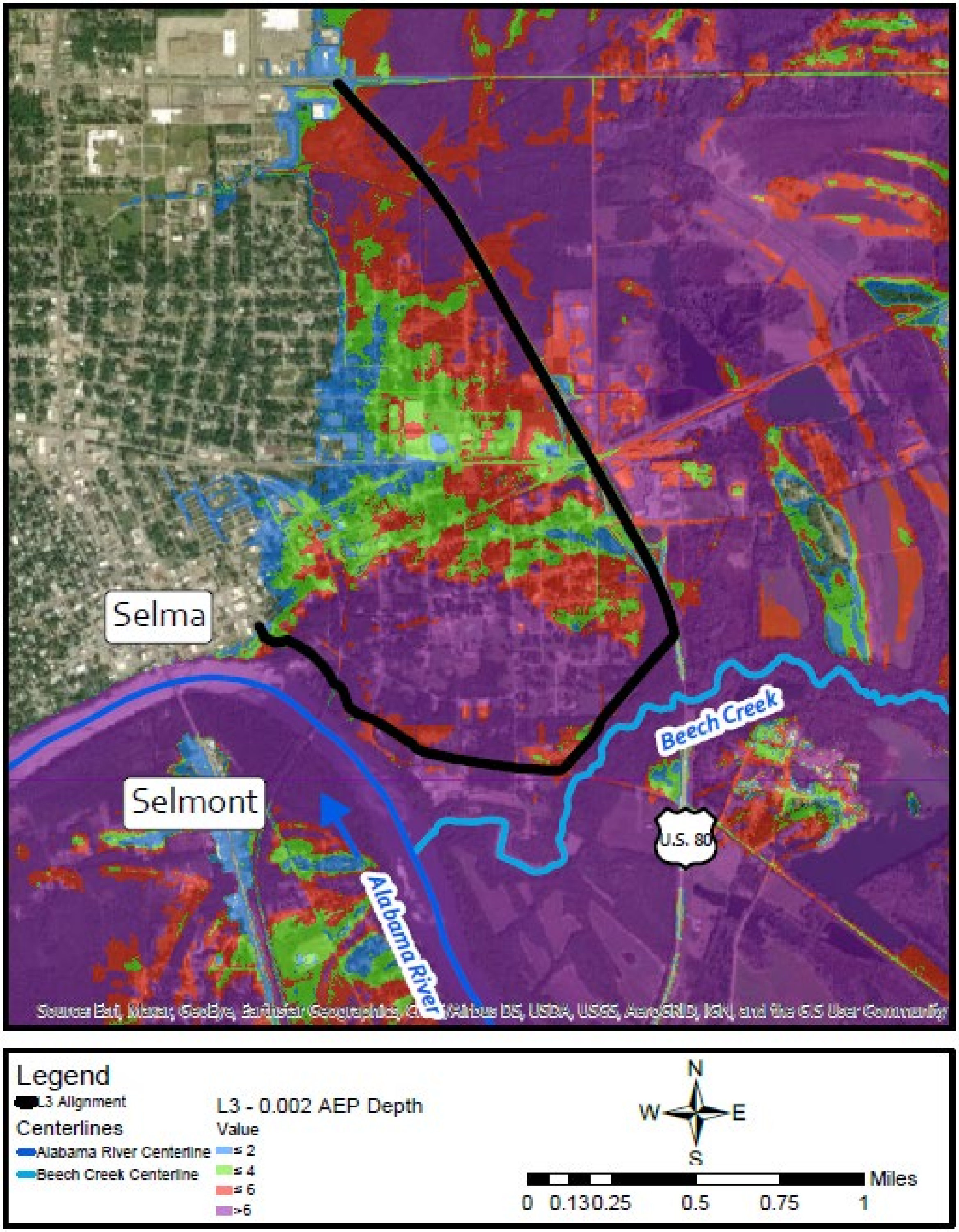



*Figure A-65: 0.01 AEP flood inundation for the L3 Optimized Levee*



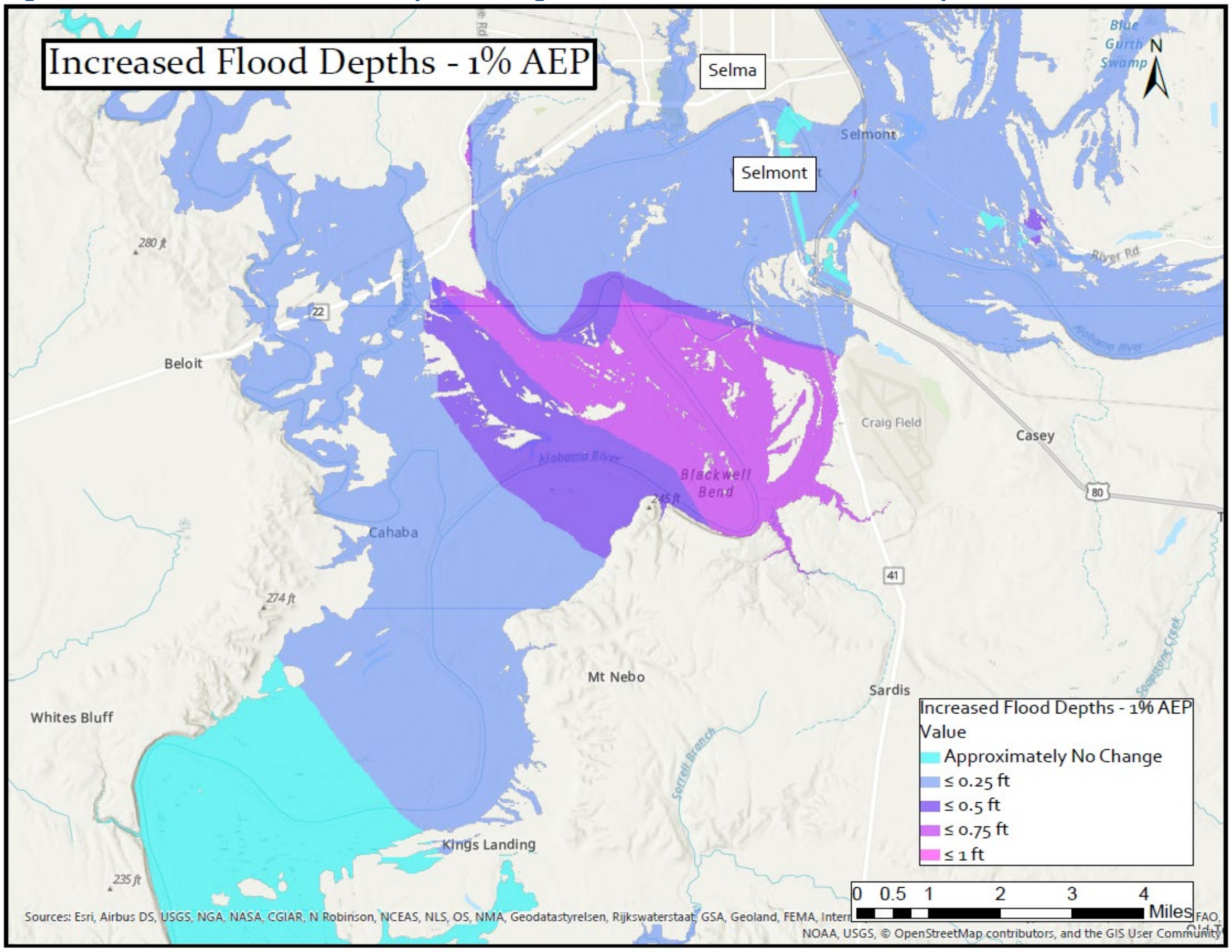
Results of the modeling show that the levee provides protection for wards 6 and 8 up to the 0.01 AEP event. For less frequent events, the levee is overtopped producing flooding in the interior areas. **Figure A-65** shows the levee modeled with the 0.01 AEP event. **Figure A-66** shows the levee modeled with the 0.002 AEP event.

*Figure A-66: 0.002 AEP flood inundation for the L3 Optimized Levee*



Hydraulic modeling results show that flood risk reduction is provided by the selected levee alignment; however, there are several factors that would require further evaluation. First, as this levee does overtop for extremely infrequent events, a quantitative assessment of residual life risk would need to be performed. Second, consideration would need to be given to mitigation of induced flooding to any areas outside the levee system caused by constricting the flood plain. Results of the model show that there is increase depth outside the levee system of populated areas across the river as well as downstream of Selma. **Figure A-67** shows this increase in flood depth for the 0.01 AEP.

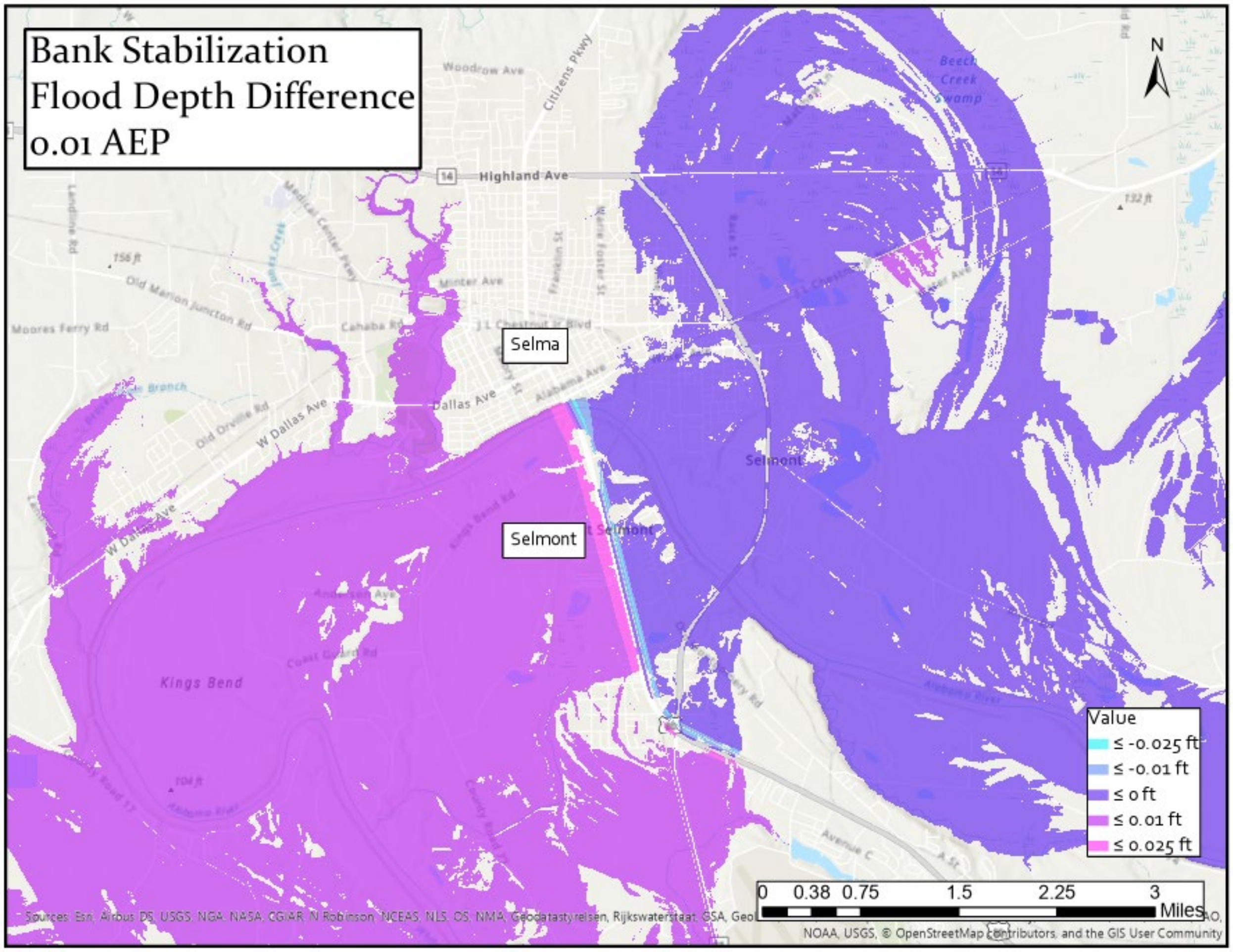
***Figure A-67: Increase in flood depths along the Alabama River for the L3 Optimized Levee***




### *A.4.9.2.3. Alternative 4: Bank Stabilization*

As previously discussed in **Section A.4**, hydraulic modeling was performed and indicated no impact to water surface elevations in the area. **Figure A-68** shows the flood depth differences as modeled in HEC-RAS. The maximum was +/- 0.025 ft, which has no effect on the damages that were derived within the HEC-FDA model. This study utilized the approach to formulating a project as applied under Section 14 of the Flood Control Act of 1946. As in Section 14 projects, the formulation and evaluation focus on the least cost alternative solution and that alternative plan is considered to be justified if the total cost of the alternative is less than the costs to relocate the threatened facility.

*Figure A-68: Flood Depth Difference for the 0.01 AEP*




### A.4.9.2.4. Alternative 5: Bank Stabilization and Buyout Option 3

Similar to Alternative 4, hydraulic modeling was performed and indicated no impact to water surface elevations in the area. **Figure A-68** shows the flood depth differences as modeled in HEC-RAS. The maximum difference was ± 0.025 ft, which has no effect on the damages that were derived within the HEC-FDA model. Buyout option 3 was modeled as a standalone measure for Alternative 1A and, was not incrementally justified. Furthermore, no damages were derived through HEC-FDA for the soldier pile wall. This study utilizes the approach to formulating a project as applied under Section 14 of the Flood Control Act of 1946. As in Section 14 projects, the formulation and evaluation focus on the least cost alternative solution and that alternative plan is considered to be justified if the total costs of the alternative are less than the costs to relocate the threatened facility.

### A.4.9.2.5. Alternative 6: Bank Stabilization, Optimized Levee and Buyout Option 3 Modified

No additional modeling was done to support this alternative, because Buyout Option 3 and the Optimized Levee standalone alternatives could not be incrementally justified. Therefore, no further consideration was given to this alternative.

### A.4.10. Recommended Plan

Alternative 4, (Bank Stabilization), was selected as the Recommended Plan with the addition of a Flood Response Plan to address flood and life safety risk. The Recommended Plan is not based on NED benefits but on several other factors; some unrelated to engineering. These include community cohesion and the national and historical significance of the structures the solder pile wall would protect. The solider pile wall addresses the most pressing need of the city, which is protection of historic structures along the bank of the downtown area. More information on the determination of this as the Recommended Plan can be found in **Section 4.0** of the Integrated Feasibility Report and Environmental Assessment.

There is still the outstanding issue of life safety, especially in Wards 6 and 8. As another structural or non-structural measure was justified in addressing this issue, it was decided to include Flood Response Plan discussed in **Section A.4.5** to address this. All other structural and non-structural measures were screened from the study. Buyouts were determined to negatively affect community cohesion and also were determined to have negative net annual benefits. All other structural alternatives were deemed impractical or had severely negative net annual benefits. It was therefore determined that the driving factor for addressing flooding in the city was life safety risk as opposed to a reduction in economic damages. A Flood Response Plan adequately address these risks with little cost to the federal government and the sponsor.

### *A.4.11. Recommended Plan Design Summary*

The Recommended Plan includes bank stabilization with a soldier pile wall along approximately 1000-foot of the riverbank and bluff at the proposed project site. The soldier pile wall will be constructed to a top elevation of 110-ft which is above the Mooreville chalk and overburden soil layer interface where erosion is occurring. Soldier piles will be placed into pre-drilled holes and grouted in place and reinforced precast concrete lagging panels will be installed between each solider pile. Tie-back anchors will be installed at multiple levels between soldier piles and the bank. The quantity of required tie-back anchors at each respective soldier pile will be determined during PED. Installation of piles directly under the bridge would not be practical considering the limited vertical clearance and obstruction to crane support. Where required to pass under the bridge, a shorter, cantilevered reinforced concrete wall or T-wall section of bank stabilization is being considered. Based on the proposed wall alignment, the toe elevation of the cantilevered concrete or T-wall section under the bridge will be in the range of approximately 90-ft to 100-ft (i.e., above the Alabama River normal pool elevation, 84.3 ft NAVD88 and would not require cofferdams for construction).

Integral to the bank stabilization plan would be a drainage system constructed to address both seepage waters and flood waters behind the lagging wall. This drainage system would employ a very porous gravel backfill material (e.g., #57 stone) behind the wall to adequately drain during river drawdown events and the use of filter/geotechnical fabric completely wrapping the gravel backfill material to prevent seepage waters from eroding upper horizon soils. The drainage system would include a perforated header pipe extending parallel to the slope of the bank with laterals which outfall to the face of the lagging wall. The drainage system may be constructed at multiple levels as determined necessary during PED. Grouted riprap will be placed behind the wall at the “heal” (i.e., bottom of wall) to retain backfill material from escaping beneath any potential voids at the interface of the bottom of the soldier pile wall and the riverbank. Graded topsoil and seeding would be placed above the top of the wall sloping back to the buildings along the riverbank at a slope no greater than 5:1 (H:V) to allow for mowing and maintenance. Field investigations and surveys completed during the study found that the elevations where the building foundations intersect the bluff vary along the bluff in the project area from approximately 108-ft to approximately 128-ft. A secondary cast-in-place retaining wall structure is proposed to be constructed in areas along the proposed project site where determined necessary to retain soils above the top of wall elevation of 110-ft. A conceptual site plan of the bank stabilization is shown on **Figure A-69**. Conceptual section views of the bank stabilization with and without the proposed secondary retaining wall are shown on **Figure A-70** and **Figure A-71**, respectively. Modification to the conceptual bank stabilization design such as riprap end caps, additional scour protection at the toe of the wall, and other erosion control methods may be determined necessary during PED. Additional details of design analysis and supporting documentation can be found in Section A.8.

***Figure A-69: Conceptual site plan for the Recommended Plan***

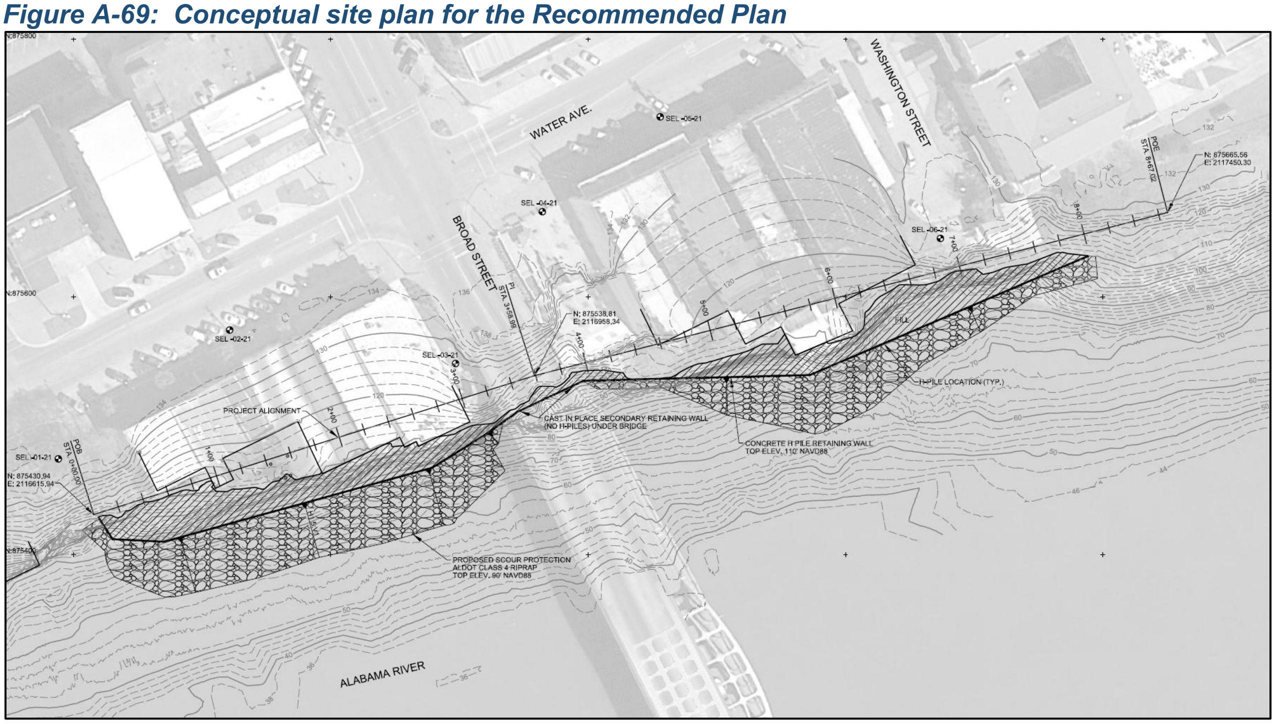


***Figure A-70: Conceptual section view for bank stabilization in areas where secondary retaining wall is required***

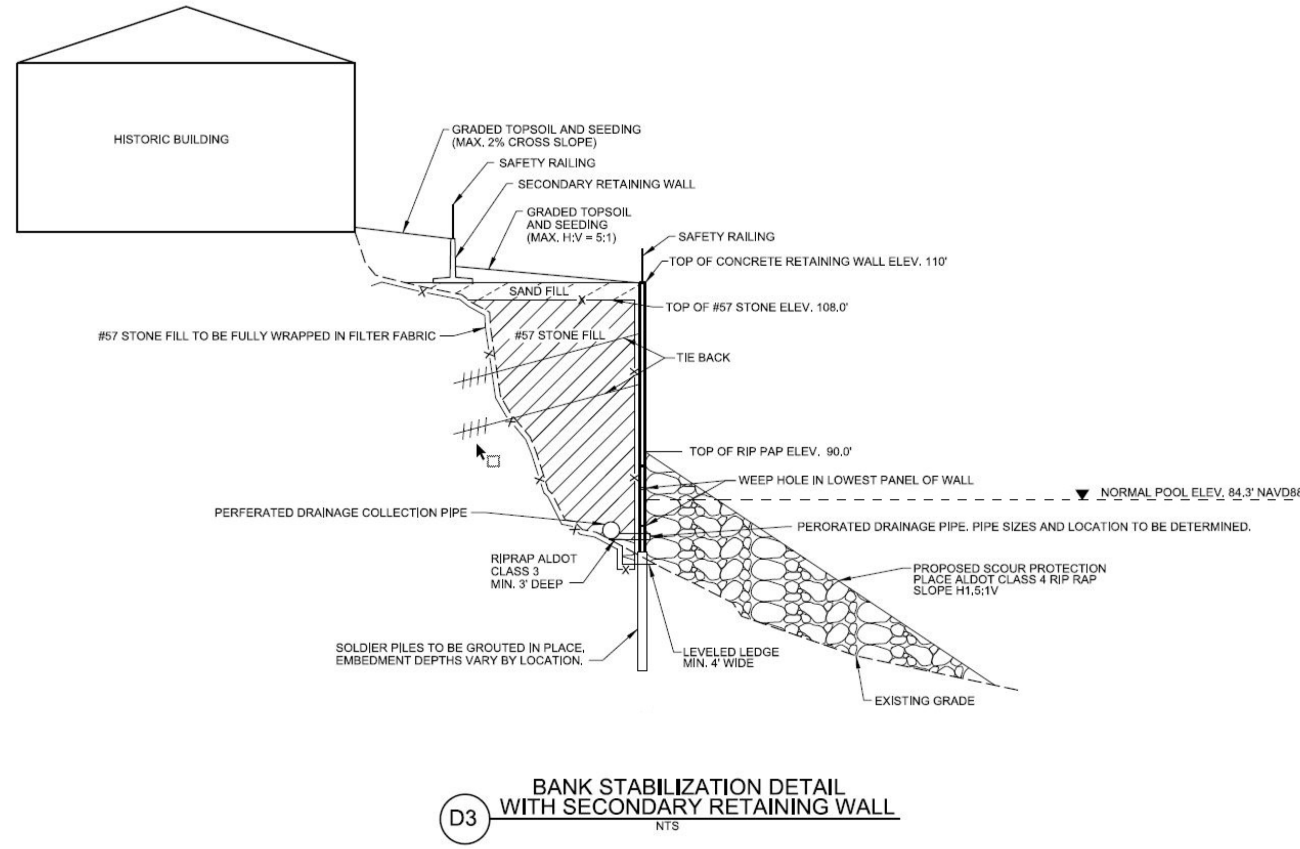
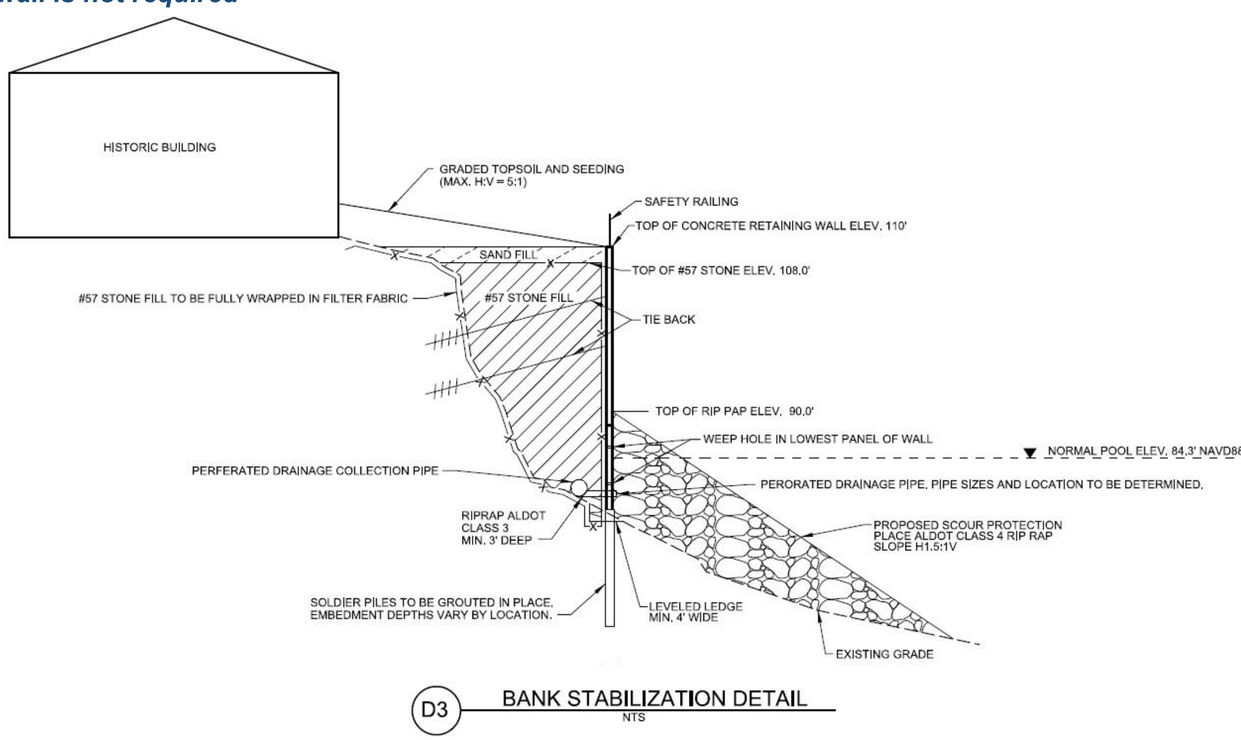



***Figure A-71: Conceptual section view for bank stabilization in areas where secondary retaining wall is not required***



## A.5. Field Investigations Supporting Recommended Plan

Field investigations were conducted in support of the tentatively selected plan and to further refine the conceptual feasibility-level design of the soldier pile wall. All activities performed share the objective of improving the confidence in cost estimates supporting the selected plan and the understanding of the constructability of the selected plan.

Geotechnical investigations including soil borings along the bluff of the Alabama River throughout the extents of the proposed project site for the soldier pile wall and subsequent lab analysis were performed to better understand subsurface conditions in the study area. Results from these investigations and analysis support Engineering Division's efforts to complete a feasibility-level design, and to refine assumed construction costs and reduce contingencies of the soldier pile wall cost estimate. A better understanding of the subsurface conditions and an increased confidence in the strength of the subsurface materials will help determine the length and embedment depth of the soldier piles supporting the embankment stabilization. Additional information regarding soil borings and geotechnical lab analysis is provided in **Section A.8.1.3**. Detailed results from geotechnical investigation, corresponding lab analysis, and historical boring data is provided in Exhibit A-1.

Light Detection and Ranging (LiDAR) and topographic surveys of the proposed project site and surrounding area were performed to support a feasibility-level design. Survey data supported an increased confidence in engineering design assumptions such as soldier pile wall alignment and proposed layout, estimated quantities for backfill material behind the soldier pile wall, soldier pile wall toe elevations, required soldier pile lengths and a clear understanding of the existing conditions along proposed project site. Knowledge of the existing conditions and elevations of building foundations, roadways, existing utilities, and private property boundaries were essential in developing a feasibility-level design and understanding the requirements needed to complete a future preconstruction, engineering and design (PED) phase, and constructability of the Recommended Plan.

The USACE Mobile District worked with the USACE Omaha District to contract an unexploded ordinance (UXO) survey of the proposed project area to determine if any unexploded ordinances present were present. Civil War munitions were produced in Selma, AL during the late 1800s, and the PDT worked to reduce potential risks by completing the survey. Initial findings from the field survey and an analysis of available historical documentation suggest the potential presence of UXO in the direct project area is unlikely but not impossible. However, at the time of this report a detailed summary of findings UXO survey was not available, and the PDT should continue to develop a better understanding of the risk of UXO in the area during future phases of work.

## A.6. Cost Estimates

A Total Project Cost Summary (TPCS) was prepared for each alternative. The TPCS combines the real estate (RE) costs, construction costs, contingency, preconstruction engineering and design (PED), and construction management (CM), and applies escalation factors to calculate a first cost and total project cost for each alternative. The first cost is used for the economics analysis in conjunction with the damage reduction estimates to determine net benefits for each alternative. **Table A-16** shows the first costs, estimated operations and maintenance (O&M) costs, and estimated construction durations for each of the final array of alternatives. More information is available on the development of costs in **Appendix F**.

*Table A-16: First costs, estimated O&M costs, and duration of construction for final array of alternatives*

| Alternative | First Cost | Annual O&M | Construction Duration |
|---|---|---|---|
| **Alt 1.A Acquisition and Buy-Out** | $4,950,000 | $0 | 18.0 Months |
| **Alt 3. Optimized Levee Alignment** | $74,040,000 | $27,000 | 36.0 Months |
| **Alt 4. Bank Stabilization and Flood Management Plan** | $21,323,000 | $4,000 | 18.0 Months |
| **Alt 5. Bank Stabilization and Buy-Out** | $32,400,000 | $4,000 | 30.0 Months |
| **Alt 6. Combination Alternative** | $104,860,000 | $29,500 | 42.0 Months |

## A.7. Risk and Uncertainty

The Selma FRM Recommended Plan is unique in that it does not include any traditional FRM structural or non-structural measures. Then bank line stabilization measure addresses risk to historic structures, and to some small extent, life safety to anyone occupying a building nearing failure along the bank line. Risk drivers with respect to this measure are mostly independent of hydrologic conditions on the Alabama River. That is, hydraulic model variability and uncertainty have no effect on this aspect of the Recommended Plan.

Another risk associated with the implementation of this measure is the stability of the foundations as tie backs are placed in between foundations of the structures along the bank. As the purpose is to protect these structures, any further damage could lead to a failure and condemnation of a building and therefore a failure of the measure's intent. Planned surveys, structural analysis in PED and additional geotechnical investigations will ensure that this risk is minimized.

The Flood Response Plan has risk associated with the hydrology of the Alabama River. This plan will be largely driven by the inundation results of the hydraulic analysis presented in this report. Flood events can be examined as the results of a meteorological risk-driver, basin development, stormwater management practices, and hydraulic characteristics. In the area of study, the meteorological risk-driver is considered heavy rainfall produced from frontal or dissipating tropical events falling in the middle northern portion of the ACT basin. The frequency and severity of the risk-driver and its response (flooding in this case) have associated uncertainties. Fuguitt and Wilcox (1999) distinguish between the two types of uncertainty: future unknowns and data inaccuracy / measurement error. Future unknowns, in the case of this study, may be encountered in forecasting future watershed development, storm water management throughout the large basin, or the effect of climate change on hydrology. Measurement uncertainty may be encountered in model calibrations to observed data, whereby error may be associated with reported values (i.e. stage and discharge). These uncertainties create future unknowns when attempting to tie a response (evacuation route) to a flow-frequency event. To mitigate this issue, this plan will tie specific actions to a given stage, as opposed to a frequency flow. In other words, there will be direction with respect to forecasted water surface elevations on the Alabama River as opposed to a flow-frequency event. There are, however, still uncertainties associated with the accuracy of inundation mapping that will drive the plan. Incorrectly mapped topography could, and often does, result in inaccurate representation of a flooded area. The only reasonable way to but this down is to obtain high quality topography and ensure proper quality checks are done on the resulting surface developed for modeling. Communication to the sponsor on this uncertainty in also extremely important for them to understand risk associated with the recommended plan.

The overall purpose of the Flood Response Plan is to address life safety. This plan would address life safety in two ways. First, it would provide the City of Selma with a comprehensive plan to direct evacuations of areas forecast to flood. The Alabama River is a slow-moving river with floods often taking days to reach the City of Selma. This is adequate time for the City to prepare and move residence out of flood prone areas. AS discussed, though the use of stream gages near Selma, Robert F Henry Lock and Dam and Montgomery, Alabama with flood forecasting already being provided by the Southeast River Forecast Center, an evacuation plan would assist the city in directing the evacuation of residents based on certain forecasted flood elevations. This would include recommended locations to be evacuated, safe evacuation routes and identification those locations that would be inaccessible, all based on a forecasted flood elevation. Second, the Floodplain Management Plan would address future use of the floodplain within the city limits. As structures are condemned in the future and residents move out of heavily flood prone areas, responsible redevelopment of the floodplain can reduce or eliminate life safety risk in the future.

In theory, this plan would eliminate flood risk with respect to life safety from the areas it covers. If followed, residents would have adequate time to fully evacuate. In practice, this will greatly reduce life safety risk but not eliminate it. Even mandatory evacuations are often ignored by residents who decide to accept the risk of remaining in a flood prone location during a flood. Historically, it has been impractical to fully enforce a complete evacuation of an area. Furthermore, future floodplain management of the area will ultimately be at the discretion of the city to enforce. It will likely involve locale legislation to enforce the recommendations laid out in the Floodplain Management portion of this to prevent residential redevelopment of the floodplain. In this case residual life risk is directly correlated to degree at which this document is utilized and enforced by the City of Selma.

## A.8. Documentation Supporting Feasibility-level Design

The following sections layout a summary of applicable engineering data needed to support a feasibility-level engineering design as outlined in ER-1110-2-1150 Engineering and Design for Civil Works Projects.

### *A.8.1. Geotechnical*

#### *A.8.1.1. Site Geology*

The Selma area is situated near the center of the Black Prairie subdivision of the Gulf Coastal Plain physiographic province. The Black Prairie subdivision is a belt of low relief which crosses the state in and east-west direction. In the Selma area, it is about 20 miles wide and consists of flat to gently undulating prairie land. The major drainage of the area is by the entrenched and meandering Alabama River which crosses the prairie belt in a southwesterly direction. The Black Prairies correspond in length and width to the weathered outcrop of the Selma Group of late Cretaceous age which is a chalky to argillaceous limestone formation with a maximum known thickness of about 900 ft. The general dip of the strata in the Selma area is about 30 ft per mile to the south.

The geology in and around the City of Selma consists of alluvial deposits, underlain by various formations within the Selma Group, the most prevalent of these being the Mooreville Chalk. Alluvium deposits consist of a mixture of varicolored, fine to coarse sand with clay lenses and gravel. The Mooreville Chalk is generally characterized as a yellowish-gray to olive-gray clayey chalk or chalky marl. A visual survey of the banks indicate that the banks are steep (1V:1.5H and steeper), and they are comprised of sands, silts, and clays that sit atop a layer of chalk. Historical borings from past geotechnical explorations confirm this assessment, noting that the chalk layer is dense and strong. Banks in the downtown area range in height between 30 to 50 ft above the water's surface (average water surface elevation at the Edmond Pettis Bridge is +84.30 ft. msl). The interface of the overburden and the chalk is easily spotted from the river, and this interface appears anywhere from 5 to 20 ft above the water's surface (i.e., elevations range from approximate +90 to +105 feet NAVD88).

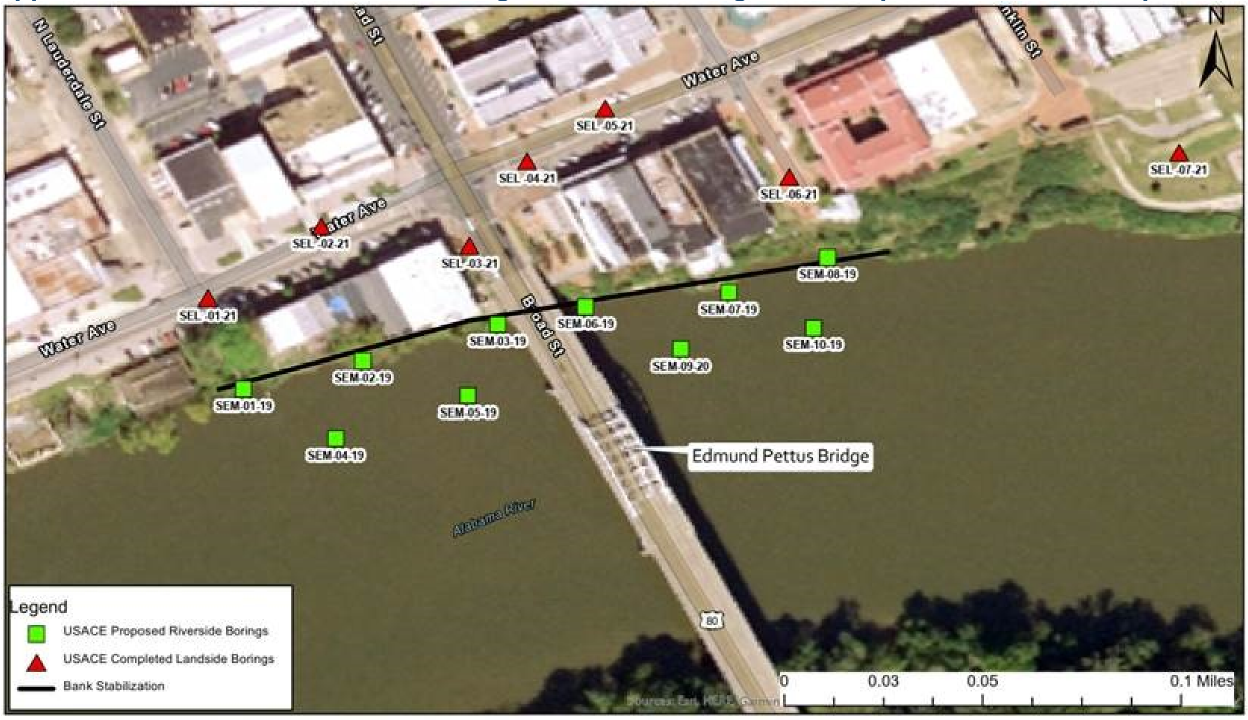
### *A.8.1.2. Subsurface Investigations*

The USACE Mobile District team performed a landside subsurface investigation along the bluff and within the projected area during February and March of 2021. The investigation consisted of seven (7) Standard Penetration Test (SPT) borings advanced to depths ranging between 20 and 100 feet below existing grades (BEG). The approximate test locations are shown on **Figure A-72** and were determined in the field by using a handheld GPS unit, developed site plan, Google Earth aerial imagery, and existing field reference points on and adjacent to the site. Ground elevations were developed based on the correlation of the GPS coordinates with the topographic (lidar) survey and then verified in the field using handheld GPS unit. After the borings were completed, each location was surveyed using a RTK unit to verify coordinates and determine spot elevations.

Soil samples were obtained at selected intervals of depth based on the soil stratigraphy and material. The SPT sample was obtained by driving a standard split-spoon sampler having a length of 18 inches and an outside diameter of 2 inches into the borehole using a 140-pound hammer dropping a length of 30 inches. The number of blows to drive the sampler for 3 consecutive, 6-inch penetration increments was recorded. The SPT blowcount is the sum of the latter two increments and reported on the soil boring log.

Within the Mooreville Chalk relatively undisturbed samples were obtained using a Triple Barrel Core sampler. This sampler has an inside diameter of four inches and a barrel length of 5 feet. Representative soil samples obtained during the investigation were collected and transported to the soil laboratory for further testing. All soil samples were classified in the field by a licensed Geologist in accordance with the USCS classification system. Detailed results from the soil boring and subsurface investigation are provided in the following sections of this report.

*Figure A-72: Current geotechnical investigation boring locations performed during this study and approximate location of future riverine geotechnical boring locations planned for the PED phase*




The coordinates of the test boring locations are summarized in **Table A-17** below.

*Table A-17: Boring locations, ground elevations at each boring location, and groundwater elevations observed at each boring locations.*

| Boring # | Lat | Long | Boring Elev. (ft-MSL) | Groundwater Elev. (ft-MSL) | Borehole Depth (ft) |
|---|---|---|---|---|---|
| **SEL-01-21** | 32.4060094° | -87.0201647° | +135.5 | +111.5 | 100 |
| **SEL-02-21** | 32.4062828° | -87.0197319° | +135.0 | +112.1 | 80 |
| **SEL-03-21** | 32.4062091° | -87.0191636° | +128.9 | +110.8 | 100 |
| **SEL-04-21** | 32.4065323° | -87.0189433° | +134.8 | +112.3 | 80 |
| **SEL-05-21** | 32.4067337° | -87.0186455° | +134.6 | +111.4 | 75.8 |
| **SEL-06-21** | 32.4064715° | -87.0179412° | +126.9 | +110.4 | 19.5 |
| **SEL-07-21** | 32.4065651° | -87.0164498° | +115.9 | +107.9 | 70.4 |

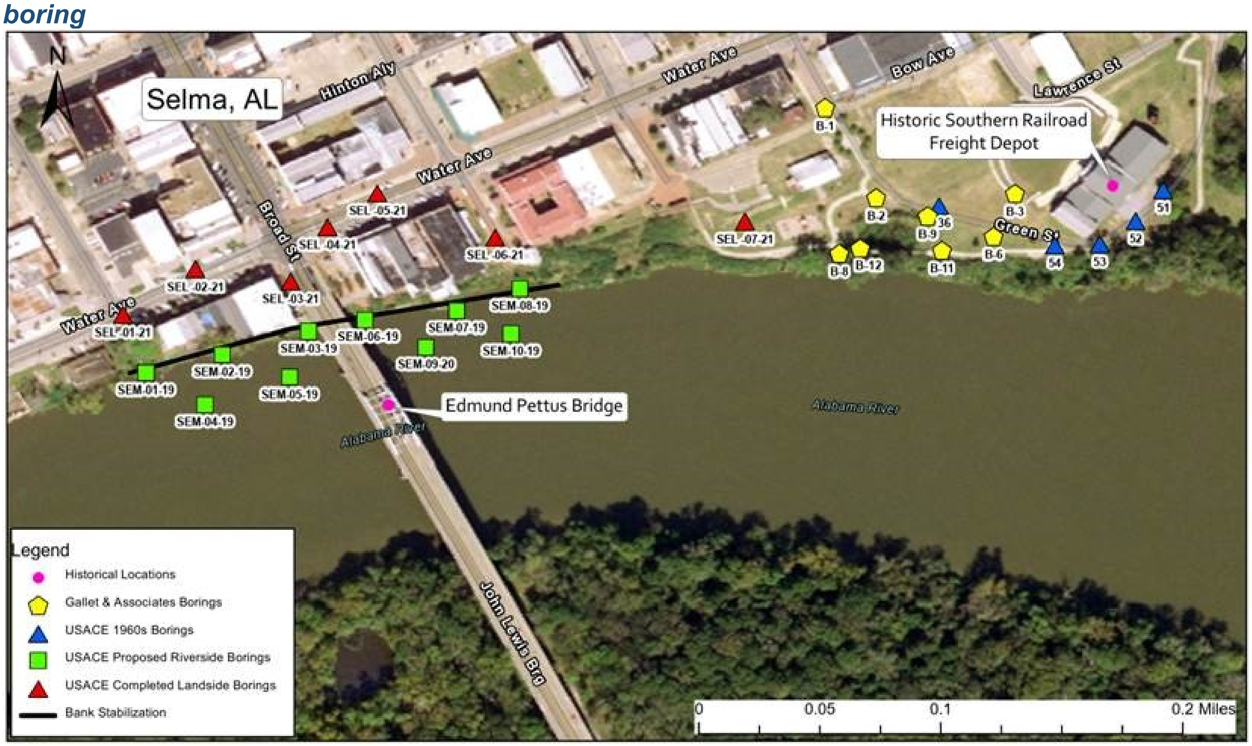
At soil boring location SEL-06-21 a petroleum odor was noted from cuttings within the soil boring and further drilling at this location was terminated. Review with the engineering office concluded that further advancement of this boring will require research for the possible source of the contamination. An alternate soil boring location was selected further east of SEL-06-21 and its location has been labeled SEL-07-21.

A planned concurrent riverine exploration soil boring program was intended to immediately follow the landside exploration program, but, was postponed when concerns were raised over the possible presence of UXO within the exploration area. Historical research of the Selma area uncovered information regarding UXO disposal along the shoreline from the Civil War period. Records show that recovered UXO from this site has been advertised and sold in the past. Although submerged for 100 years, there may be the potential for a black-powder ordnance to still be dry and a possible danger to the drill crew once brought to the surface. A UXO survey is currently underway for small arms (cannon balls, hand grenades, bullets, etc.) within the planned exploration area. Although the study will delay the riverine program, this portion of the study is deemed necessary to complete the geotechnical/geologic study and is planned to be completed during PED phase.

### A.8.1.3. Subsurface Conditions

Subsurface conditions were analyzed based on observations from a geotechnical exploration program conducted as part of this study as well as past geotechnical information available to the PDT. The locations of geotechnical information used for this study are found on **Figure A-73**. Additional information and boring logs are found in Exhibit A-1.

***Figure A-73: Locations of current geotechnical investigation borings performed during this study, future proposed riverine geotechnical borings planned for the PED phase, and past geotechnical boring***




#### A.8.1.3.1. Past Geotechnical Explorations

A geotechnical investigation was performed by the Mobile District in the 1960s, as detailed in the *Interim Report on Alabama-Coosa River System at and In the Vicinity of Selma, AL (1967).* A total of 54 auger and split spoon borings were made along and adjacent to the center line of the proposed Selma levee and floodwall and Selmont levee locations, considered in the 1967 report. Borings 36 and 51 through 54 were sampled upstream of the project location along the landside northern bluff, approximately 800 feet from the Recommended Plan project footprint. The exact location is unknown as no coordinates were provided, but their approximate locations based on available information can be seen on **Figure A-73**. Boring 36 was advanced to 10 BEG using the

Standard Penetration Test (SPT). There was no elevation data associated with this boring. Poorly graded sands (SP) were encountered from 0 to 1.5 feet BEG. This layer was underlain by lean and fat clays (CL and CH) from 1.5 to 7.5 feet BEG. From 7.5 to 10.0 feet BEG, the soil transitions from a clayey sand (SC) to and clean sand. The water table was encountered at 9.0 feet BEG. Borings 51 through 54 borings had a top elevation of 118.0 feet (vertical datum unknown) and were advanced down to elevation 107.5 feet by SPT. Generally, the borings showed a black to dark brown conglomerate fill from 118.0 feet to 116.5. From 116.5 down to 107.5, the soil types trend from clays and clayey sands (CL, CH, and SC) to clayey sands, silty sands, and clean sands (SC, SM, and SP). The water table was not encountered at any of these boring locations.

Gallet & Associates conducted a geotechnical investigation in the area in 2009 to support the design of a walking path and bridge abutments for the City. The borings were located upstream of the project study limits. Seven SPT borings (B-1, B-2, B-3, B-5, B-6, B-8, and B-9) were advanced to 5.0 feet BEG in support of the walking path. Two SPT borings (B-11 and B-12) were advanced to 30.0 feet BEG near the proposed pedestrian footbridge abutments. These borings were not referenced to a vertical datum. A 3-foot layer of fill was consistent in all of the Gallet borings from ground surface down to 3.0 feet BGS. The fill was characterized as sand and clay (SC and CL), and some of it was mixed with slag and coal. The fill material was underlain by a layer of sandy lean clay (CL). Borings B-1, B-2, B-3, B-5, B-8, and B-9 were all terminated in this layer at 5.0 feet BGS. Borings B-11 and B-12 showed that this layer extended down another 3 feet to approximately 8.0 feet BGS. A medium dense to dense clean sand (SP and SW) underlies the clay layer. This layer is approximately 10 feet thick in boring B-11 and 5 feet thick in B-12, terminating at depths of 18.5 feet BGS and 13.5 feet BGS respectively. Boring B-12 showed another 5-foot thick layer of silt underlying the sand that terminates at the top-of-chalk at 18.5 feet BGS. The water table was measured in both borings at 13.0 feet BGS. A layer of gray chalk underlies the clean sands and silts, measured from 18.5 feet BGS to 30.0 feet BGS. The chalk is very hard, and refusal was encountered in all SPT drives. All borings mentioned in this section are approximately located as shown on **Figure A-73**.

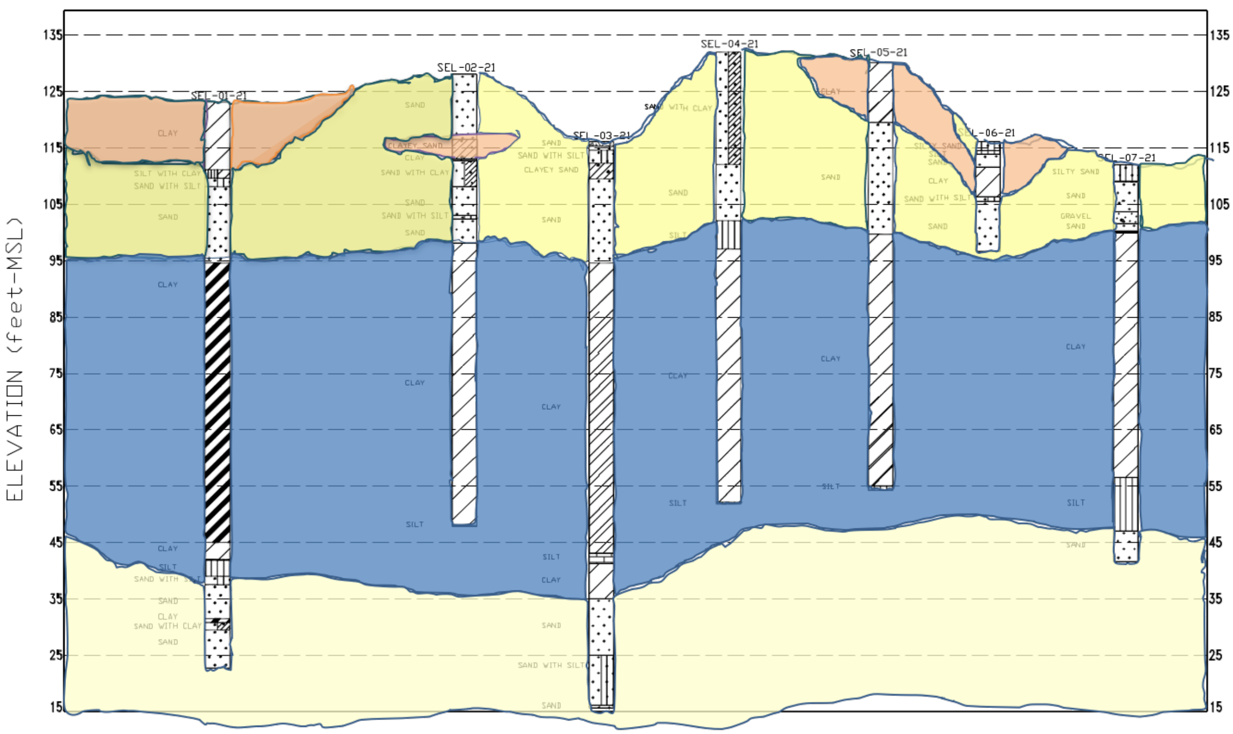
#### *A.8.1.3.2. Current Geotechnical Exploration Program*

Soil borings were performed to develop a profile of the soil stratigraphy composing the shoreline of the Alabama River at Selma, AL. Boring locations were limited due the presence of existing buildings, bridges, patios, fences and existing topography. In very few locations was the "bluff" accessible for drilling without extravagant means and/or requirements. It should be noted that the marine exploration soil boring program has not been started and further investigation and subsequent lab analysis of samples taken from locations throughout the river in the project area is recommended during the PED phase.

From existing grades and extending to approximate elevations ranging between +108 and +120 feet-msl, clayey sands (SC), low plasticity clays (CL), silty sands (SM) and fine sands (SP) were sampled. This layer appears to "cap" the underlying erodible sand layer. Standard Penetration Test (SPT) blow counts ranged from 4 to 17 blows per foot. Beneath the surficial clays and sands and extending to approximate elevations ranging between +95 and +100, fairly clean sands were encountered. SPT values ranged between 5 and 23 blows per foot. Within this layer groundwater was encountered at elevations ranging from 97.8 feet to 109.5 feet-msl.

The bottom of the Mooreville Chalk ranged between elevation +38 feet to +50 feet and had SPT values ranging from 48 to 50 blows for 0.2 ft. Beneath the chalk and extending to the minimum explored elevation of +15 ft-msl, primarily sands (SP) were encountered with some lenses of fat clays (CH), low plasticity silts (ML), and low plasticity clays (CL). **Figure A-74** shows a fence diagram from landside soil boring investigations conducted during the current geotechnical exploration program.

***Figure A-74: Fence diagram from landside soil boring investigations completed during the current geotechnical exploration program***




### *A.8.1.3.3. Future Geotechnical Exploration Program*

A riverine exploration soil boring program is planned during PED phase to support the landside exploration. Qualification of the UXO risk/potential is currently underway with a compilation of a UXO report. A side scan survey showed many targets along the shoreline which varied in height from less than 2 feet to approximately 3 feet. Some of these were fairly numerous in proximity to each other and prevented individual targets. A magnetometer survey could not be performed during the side scan survey due to strong river currents. The decision to re-mobilize to perform the magnetometer survey will await the conclusions of the initial survey report.

### *A.8.1.4. Groundwater Conditions*

As mentioned above, the groundwater was encountered within the fairly clean sands overlying the Mooreville Chalk. The elevation of the groundwater ranged from +108 to +112 ft-msl during our field exploration program. It is assumed that the groundwater drains to the river and is perched on the Mooreville Chalk formation. It is anticipated that groundwater levels will fluctuate seasonally, with rainfall events, and may be influenced by flood levels of the adjacent river.

### *A.8.1.5. Lab Testing Program*

Currently, the soil laboratory testing program is underway with only minimal reporting of data at the time of this report. Testing of soil characteristics, strength, gradation and corrosiveness values will entail the laboratory program. Triaxial strength testing has provided initial strength data of the Mooreville Chalk "undisturbed" samples. These data show a very competent material with very high strength values. Of note, the characterization testing has indicated that the "chalk" is akin to a highly over consolidated fat clay. More data will be available as further testing is completed.

### *A.8.1.6. Major Subsurface Strata and Initial Material Properties*

The encountered subsurface stratigraphy can be described as three major strata: upper sands, fat clay (chalk) and lower sands. The upper sands can be described as an upper level with more clays, clayey sands and silty sands. Underlying these sands is a continuous layer of fairly clean sands. The groundwater level was encountered within these clean sands. Consistency of the sands ranged between very loose to firm. Beneath the upper sands the Mooreville Chalk formation was encountered. The top of this layer is very consistent at an approximate elevation of +108 ft.-msl. The material at Selma is a fat clay and has a thickness ranging between approximately 47 and 60 feet. Below the Mooreville formation sands were sampled to a minimum elevation of +25 ft.-msl. (penetration of 100 feet).

Selection of the strength properties of the foundation layer for the planned lagging wall focused on the Mooreville Chalk formation since the bottom of the wall will reside in this layer. Although only initial laboratory analyses have been submitted, the initial strength data shows very high values for phi/cohesion. Triaxial strength test results indicate phi and cohesion of 40 degrees and 4,000 psf and phi and cohesion of 2.6 and 24,000 psf, respectively. These values vary considerably and will require further review in addition to further sample testing. Initial analyses have used a phi angle of 39 degrees and zero cohesion. The upper sands are described as SM, SC, and SP sands with a phi angle of 30 degrees. The underlying lower sands are in a very dense state and will be described with a phi angle of approximately 35 degrees.

### *A.8.1.7. Geotechnical Engineering Analysis*

Analysis of the planned retaining feature considered sloping backfill into the river, sheetpile wall, rigid concrete wall, reinforced earth (MSEW) and lagging wall designs. The purpose of the wall is to stabilize the upper layers of sands which support the historical shoreline buildings. Using the competent clay layer as a stable base of our wall base is believed to be the most competent design method. A lagging wall was selected due to the ability to install anchors to tieback the wall loadings with requiring a minimal base width. Above the lagging wall it is envisioned that a standard concrete retaining wall will be constructed in appropriately selected sections to stabilize the soils immediately riverside of the building's shallow foundations and above the top of the soldier pile wall. In addition, this will create an access and work platform for the regular maintenance of the slope and establishment of vegetative cover. Concrete retaining wall section placement will be selected based on the varying elevations along the bluff where deemed necessary during PED to stabilize overburden soil layer from erosion.

The lateral loading analysis has relied on hand computations for the initial wall section design as the project is being developed. Future analyses will utilize a computer program to allow development and review of multiple sections of the wall to accommodate the changes in slope profile. Anchor adhesion values of around 1200 psf have been incorporated and based on correlated values from the laboratory strength testing. The anchors will be installed in the competent clay layer and extend to a length adequate for development of the required capacity. It is desired to provide all the lateral support from the tiebacks to reduce the load at the base of the wall. This is will reduce the amount of "cover" required to develop lateral and bending capacity at the embedded steel pile sections.

The wall will be constructed by establishing a level base notched out of the clay so that a drilled "pilot" hole can be installed for the vertical column (H-pile or W-section). Drilling the base of the steel pile will reduce the potential for vibration within the soils supporting the buildings. The steel section will be grouted in place to a depth adequate to develop the required pile reaction (estimated at 15-20 feet). Lagging panels will be installed to the first layer of tiebacks. A layer of grout will be placed at the base of the columns to protect the top of the exposed clay layer. Once the tiebacks are installed backfilling of the wall will use open graded gravel. A layer of geotextile will be placed along the face of the clay and/or soil as the backfill is installed. This is intended to prevent migration of the retained, natural soils. Additional layers of wall panels, tiebacks and backfill will be installed in sequence as the wall construction continues. The final backfill will be a sandy soil separated from the gravel by a geotextile. The top of the wall will support a concrete retaining wall and an access slab. Details are being considered for a handrail, access points along the shoreline, individual structures extending within the project areas and underdrain collection system within the backfill profile. Material sources have been identified through a regional supplier and local borrow pits. Since most of the backfill will be "self-compacting" materials, minimal disturbance to the existing structures is anticipated. Monitoring of vibrations will be required at the building structures to direct the compaction of the backfilling if needed.

#### *A.8.1.8. Future Geotechnical Stability Analysis*

Slope stability analyses are planned using a computer aided analysis program to further evaluate the stability of the river bank. Additional data is required from the ongoing subsurface investigation's laboratory test program and the marine exploration soil boring program to define the conditions within the river bottom to complete this analysis. Historical soil boring information from the bridge exploration provide some indication of soil conditions beneath the river bottom, however, variances in soil classifications and descriptions as well as elevations do not allow a high level of confidence with strata comparisons (i.e. landside to marine). Further marine exploration soil borings and laboratory testing should provide the required information for completion of the stability review.

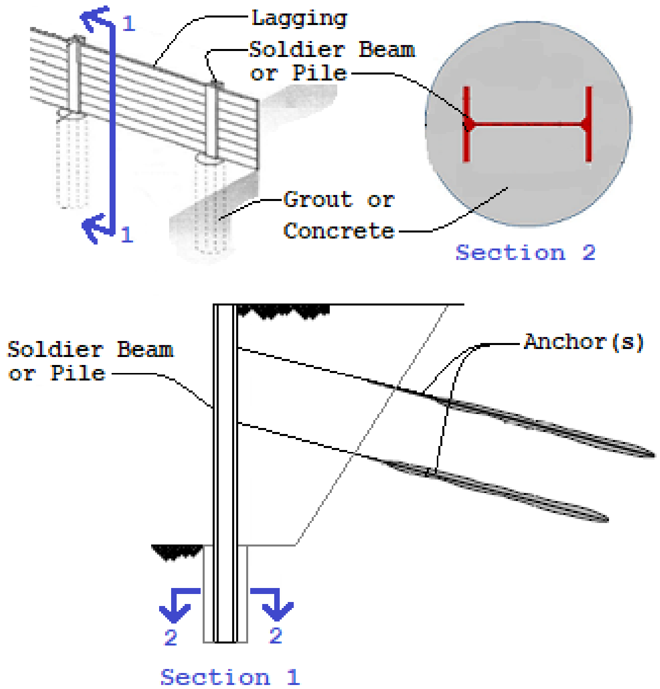
### *A.8.2. Riprap Sizing and Analysis*

The proposed design includes riprap to be placed at each end of the soldier pile wall to a top elevation of 110 feet and at the toe along the riverine face of solider pile wall at a maximum slope of 1.5:1 (H:V) to a top elevation of 90 feet. A riprap sizing analysis was performed and indicated an ALDOT Class 3 riprap will be used for endcap and scour protection. Additionally, riprap may be used along the bankline as the wall continues under the Edmund Pettus Bridge to help protect bridge abutments from scour. A detailed scour analysis to determine the exact location of necessary scour protection along the project site is planned to be completed during PED phase.

### *A.8.3. Structural Design Criteria*

The structural elements of the proposed wall design primarily include the soldier piles (i.e., beams) embedded in the ground surface and grouted in place, lagging or wall panels placed between the soldier piles forming the retaining wall, and tie back anchors providing lateral support to the soldier piles, as illustrated in **Figure A-75**. Detailed structural design calculations can be found in **Section A.10** Structural Calculations for Soldier Pile Wall.

*Figure A-75: Soldier pile wall conceptual layout and anchoring schematic*




USACE engineering manuals contain limited guidance for the design of anchored soldier pile walls. As this type of wall is typically used for highway applications, the American Association of State Highway and Transportation Officials (AASHTO) Bridge Design Specifications (hereafter referred to as Ref 1 in this section of the report) provides more complete information. Therefore, for the anchored soldier pile wall of the Selma Flood Risk Management Study, this document was found to be more relevant. For the wall type, AASHTO specifically refers to Publication No. FHWA-IF-99-015, Geotechnical Engineering Circular No. 4 (hereafter referred to as Ref 2 in this section). It should be noted that the referenced design guidance is more specific to anchored walls constructed from the top down, meaning the soldier beams are installed and then one side is excavated down to the required elevation, installing anchors at specific elevations as required. In contrast, the proposed wall of this project would not be a top down design, as excavation is not required to obtain the elevation differential. This condition affects the design and construction, as explained in subsequent sections.

As is typical for most earth retention projects of similar size and complexity, final engineering design will also utilize a retaining wall design software to supplement current design calculations and considerations. RetainPro by ENERCALC, Inc. has been identified as an appropriate retaining wall design software and is currently being procured for use during PED phase. More specific design requirements and criteria, and additional references, are provided in the below sections.

#### *A.8.3.1. Structural Design Loads*

The design loads consist of lateral earth pressures and hydrostatic pressures that develop behind the wall. The earth pressures include the weight of the retained soil, any surcharge loads, and any loads developed from potential earthquake ground motions. For the current feasibility level design, a combination of retained soil, surcharge, hydrostatic load, and earthquake load was considered. Loads were not factored. An allowable stress or safety factor design approach was utilized per the referenced design guidance.

Three different lateral earth pressure conditions were considered, including: active earth pressure, passive earth pressure, and at-rest earth pressure. Per Section C3.11.1 of Ref 1, walls which can move away from the soil mass should be designed for pressures between active and at-rest conditions, depending on the magnitude of the tolerable movements. Per this recommendation, an average of the active and at-rest lateral earth pressure coefficient was used to develop the lateral earth pressures, as depicted in the calculations.

Whether the wall construction is to be “top-down” or “bottom-up” has a direct effect on the design loading. Anchored soldier pile walls for highway applications are most often constructed from the top of the wall to the base of the excavation (i.e., top-down construction). However, the design concept for the Recommended Plan is a fill situation, in which the wall is to be constructed from the base to the top (i.e., bottom-up construction). Ref 2, Section 5.11.5 states, "Design loadings for fill anchored walls are based on earth pressures acting on the wall when the wall is completely backfilled and all surcharge loadings are applied." With the type of incremental backfilling and staged load testing for a fill anchored wall, "the ground anchors will typically be designed to carry actual earth pressure loads as compared to loads from apparent earth pressure envelopes as may be used for anchored systems constructed from the top-down. This distinction complicates parts of the design. For one, the somewhat simplified approach of using the apparent earth pressure diagram is not applicable, thereby making the design guidance of Ref 2 much less helpful. Moreover, the application of theoretical / conventional earth pressures tends to provide a less even distribution of anchor forces and tends to increase the load experienced at the base of the pile. This difference was observed while revising the analysis to utilize conventional earth pressures as opposed to apparent earth pressure, which were inappropriately assumed in the initial analysis.

#### *A.8.3.2. Soldier Pile Section Design*

Earth pressures were applied to determine the maximum bending moment on the soldier pile. Per Ref 2, Section 5.4.1, a recommended allowable stress of $0.55F_y$ was used to select an adequate steel section. As an additional check, design capacity computed per the American Institute of Steel Construction (AISC) 360-10, Chapter F was checked against the applied bending moment.

### *A.8.3.3. Soldier Pile Embedment Capacity Design*

The passive resistance of the embedded portion of the pile must exceed the reaction force by the subgrade, R, and the force from the active pressure acting over the embedded length of the pile, with a minimum safety factor of 1.5. Ref 2, Section 5.5.2 guidance was used to determine the ultimate passive resistance. More specifically, the Wang-Reese equations of Ref 2 - Appendix B were utilized. The current design assumes an embedment depth of 15 ft, which provides a factor of safety of 4.24, well above the required minimum of 1.5 in accordance with the referenced design guidance. This embedment depth will provide an additional safety net to the lateral embedment resistance where the potential for erosion/deterioration of the chalk may exist.

### *A.8.3.4. Lagging Design*

Precast reinforced concrete panels, with a concrete compressive strength of 5,000 psi, are proposed. The panels are designed for the applied bending moment derived from the lateral earth pressure. For the current feasibility design of the lagging, load and resistance factor design (LRFD) was utilized, with a load factor of 1.5. The bending design capacity was calculated per equations of the American Concrete Institute (ACI) 318-14, with a resistance factor of 0.9. The clear cover for the reinforcement was conservatively assumed as 4.25 inches.

### *A.8.3.5. Anchor Design – Bond Length*

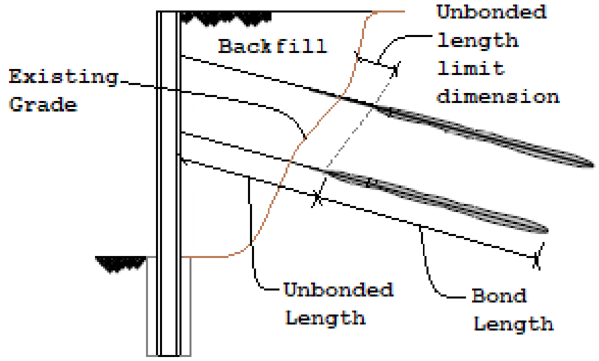
The anchor design consists of determining the unbonded length and the bond length, as indicated in **Figure A-76**. The existing slope surface was assumed as the critical failure surface. The unbonded length limit dimension, or the start of the unbonded length, was assumed to start at 7 feet past the critical failure surface per guidance found in Ref 2. The required bond length is dependent on the material within the bond zone, which has been considered per the geotechnical recommendations discussed in **Section A.8.3.8** below. Additionally, per Ref 2 guidance, a bond length that provided a minimum factor of safety of 2 was used.

### *A.8.3.6. Anchor Design – Steel*

Although site soil classification for purposes of determining corrosion protection has yet to be confirmed for structural design consideration, it is conservatively assumed that Class I (double protection) encapsulated tendons will be provided.

The anchor design load was determined from the applied earth pressures. For design assuming prestressing bar anchors, a bar grade and diameter was selected from Table 9 of Ref 2, using an allowable tensile capacity of 60 percent of the specified minimum tensile strength. If using strand anchors, the required number of strands would be selected from Table 10 of Ref 2.

*Figure A-76: Tie Back Anchor design schematic*




### A.8.3.7. Survey Data supporting Structural Design

A topographic / vessel mounted LiDAR survey of the project area was performed by Seaside Engineering and Surveying, LLC per survey report dated November 6, 2020. One-foot contours of the bank, above the water line elevation, were provided in AutoCAD Civil 3D format. From the provided alignment of the wall, a section cut was taken where the elevations indicated a maximum retained height. This data was used as the basis for structural design scenario. The section used is shown in **Figure A-77**.

### A.8.3.8. Hydrologic and Geotechnical Data supporting Structural Design

Recommended geotechnical assumptions have been provided based on initial findings from geotechnical analysis. The profile shown in **Figure A-77** was considered to be representative of the soil stratigraphy and soil properties along the alignment of the wall. This includes the existing material, with a saturated sand layer over an impermeable clay or chalk layer, and the proposed backfill material. Though the provided data indicates water within the sand layer, a drainage system and weep holes are proposed to ensure that differential hydrostatic pressure does not develop within the backfill; however, a hydrostatic pressure consideration was still applied for conservatism.

It was recommended during internal reviews to perform an analysis with lateral earth pressure from the existing material and a separate analysis with lateral earth pressure from the backfill. In place of this, the condition shown in **Figure A-78** was conservatively assumed for the current feasibility level design.

Per the Geotechnical report sections preceding this section, an anchor adhesion value of 1200 psf has been recommended for design of the anchor bond length. This adhesion value has been correlated to a bond zone transfer rate of 3.77 kip/ft for dense sand, following guidance provided in Ref 2 as discussed above.

*Figure A-77: Design Data*

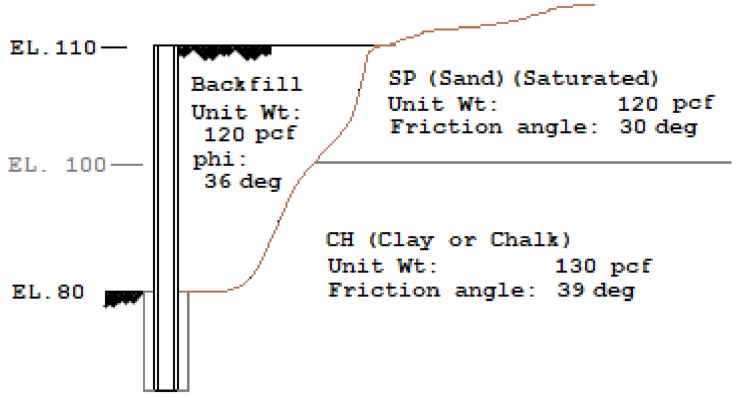
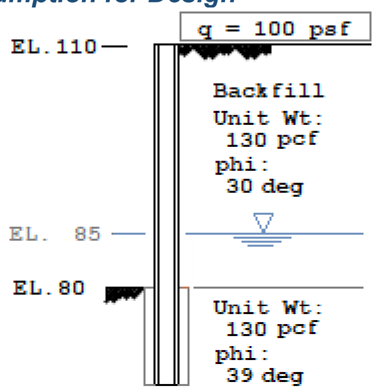



*Figure A-78: Conservative Assumption for Design*




### *A.8.3.9. Basis for Site Selection*

The site was selected for bank stabilization to benefit the existing buildings along the bank and the basis for site selection is discussed in the main body of the feasibility report.

### *A.8.3.10. Technical Basis for Type and Configuration of Appurtenant Structures*

Alternate methods that were considered for bank stabilization are discussed in **Section A.4.6** and included a continuous sheet pile wall, a riprap extension, and a cast-in place concrete gravity wall. The continuous sheet pile wall presented concerns for vibration affects to the near-by historic structures during construction, as well as concerns over the ability to successfully drive the piles into the chalk. The riprap was considered to require excessive quantities that would extend out into the river and negatively impact navigation. The coffer dams and dewatering required for the cast-in place concrete gravity wall was determined to excessively increase the cost of construction and increase environmental impact. As opposed to driving, it was determined that the soldier piles could be installed into predrilled holes and grouted/concreted into place, without the need for coffer dams and dewatering. Thus, the soldier pile wall construction method was concluded to be the least environmentally damaging and to be the method least affected by any Unexploded Ordnances (UXO's).

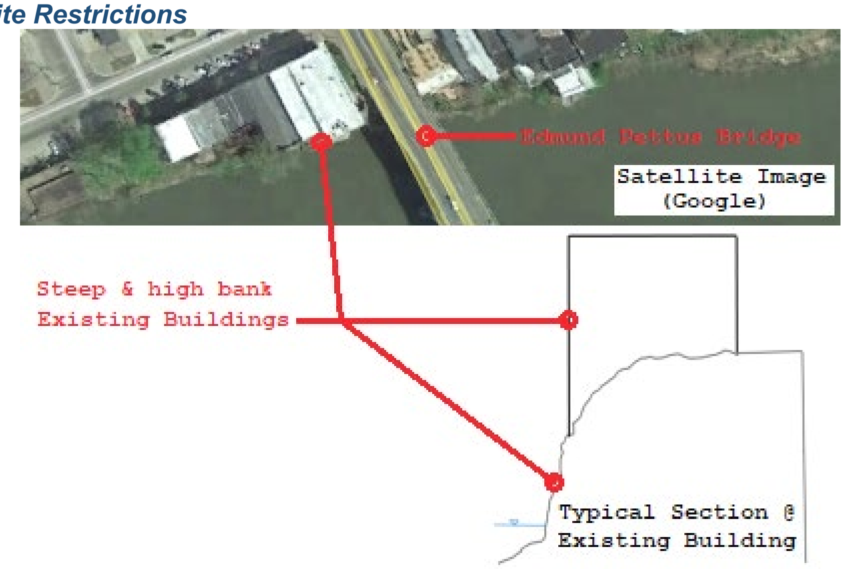
### *A.8.3.11. Evaluation and Selection of Substructure Alternatives*

Specific substructure alternatives are not applicable to the project.

### *A.8.3.12. Site Restrictions and Construction Considerations*

Site restrictions are illustrated on **Figure A-79**. The steep slope of the existing bank and the proximity of many of the existing buildings present challenges with accessing the work area. These conditions were considered in evaluating the wall alternatives, as previously described. Work under and/or near the Edmund Pettus Bridge is expected to have additional access issues and will require coordination with the State Department of Transportation. Installation of piles directly under the bridge would not be practical considering the limited vertical clearance and obstruction to crane support. Where required to pass under the bridge, a shorter, cantilevered reinforced concrete wall or T-wall is being considered.

*Figure A-79: Site Restrictions*




Anchored soldier pile walls are most often constructed from the top of the wall to the base of the excavation (i.e., top-down construction). However, the design concept herein is a fill situation, in which the wall is to be constructed from the base to the top (i.e., bottom-up construction). Per the Federal Highway Administration (FHWA) Publication No. FHWA-IF-99-015, significant differences exist with respect to the design, construction, and anchor load testing for an anchored wall built from the bottom-up as compared to a wall built from the top-down.

The recommended construction sequence per FHWA-IF-99-015 for a fill anchored wall, assuming two anchor levels, is as follows:

- install soldier beams;
- backfill behind the wall and place lagging as required concurrently up to approximately the mid-height between the bottom level anchors and the top level anchors;
- install the bottom level anchors and stress the bottom level anchors to a load that will not result in significant inward wall movement, which will likely be less than the design lock-off load;
- backfill behind the wall and place lagging as required concurrently up to a minimum of 3 ft above the level of the top anchors;

- restress the bottom level anchors to the design lock-off load, then install and temporarily stress the top level anchors;
- backfill and place lagging up to finished grade; and
- restress the top-level anchors to the design lock-off load.

In general, anchor stressing is carried out in an iterative manner as backfilling progresses. Initial anchor stressing is only meant to develop a small nominal load to remove slack from the anchors, after which the anchor is temporarily locked-off. As backfilling continues, the anchor load should increase, and restressing is required to prevent the wall from excessively deflecting outward. Only after backfilling is complete, can the anchors be load tested to 133 percent of the design load.

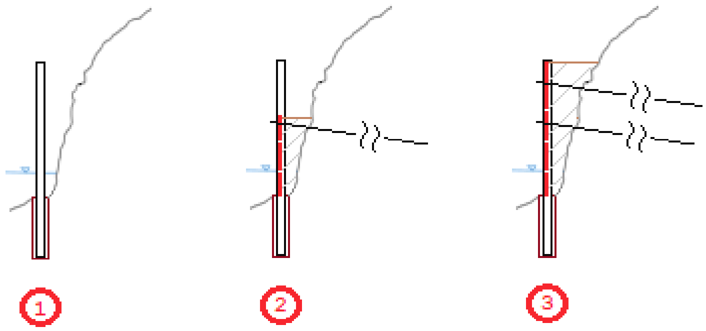
Installation and load testing of anchors for a top-down wall can be performed from the ground surface. However, with the bottom-up construction this ground surface does not exist. Therefore, platforms or lifts will be required for anchor installation.

The incremental backfilling and anchor stressing for the bottom-up wall requires the anchors to be designed to carry actual earth pressure loads as opposed to loads from simplified apparent earth pressure envelopes. This distinction complicates the establishment of design forces. Additionally, the application of theoretical / conventional earth pressures tends to provide a less even distribution of anchor forces and tends to increase the load experienced at the base of the pile.

Small compaction equipment should be used to avoid damaging the tendons. Therefore, a backfill material that permits compaction at low energy should be specified. Coupled with these constraints, adequate compaction must be achieved to ensure that significant settlement of backfill does not occur. Excessive settlement can cause bending forces to develop at the anchor/soldier beam connection. This is an additional concern that should be carefully monitored during construction since anchors are not designed to carry significant bending forces.

One sequence of construction for fill anchored walls, assuming two levels of anchors, is described in Section 5.11.5 of Ref 2. The proposed and most probable construction sequence would be as shown in **Figure A-80**, in which (1) soldier piles are installed; (2) backfill and lagging placed concurrently up to an elevation between the bottom and top level of anchors, followed by bottom level anchor installation; and (3) additional lagging and backfill is placed to just above the elevation of the next anchor (or to top of wall), followed by anchor installation. Final design should consider the construction sequence and associated loads. It is important to note that this sequence of construction is only a recommendation and other site conditions, or concerns may require a different approach. All design disciplines and an experienced contractor should provide input for determining the most suitable construction sequence. Backfilling after anchor installation and the potential need for tie back anchor protection should also be considered, as described in the preceding geotechnical sections of this report. Additionally, the contractor will be required to perform proof testing of tie back anchor system during construction.

*Figure A-80: Proposed Construction Sequence*




### *A.8.3.13. Stability Analysis and Criteria*

Structural stability analyses, such as the type performed for concrete monoliths, were not applicable to this project. However, per guidance provided in Ref 2, external stability of the anchored wall should be evaluated during PED. Refer to the geotechnical section of this report. Results may affect the location of the critical failure surface and minimum required unbonded length of anchor.

### *A.8.3.14. Results of Stress Analysis and Strength Criteria*

This information is provided in the results summary of the feasibility level structural calculations. Refer to **Section A.10** for further detail.

### *A.8.3.15. Initial Seismic Analysis and Criteria*

Per Ref 2, Section 5.10.1, few observations of the seismic performance of anchored walls have been made. Those observations that are available indicate overall good performance of anchored wall systems subject to strong ground motions in earthquakes. Two modes of earthquake-induced failure for anchored walls are considered for design: internal failure and external failure. Internal failure is characterized by failure of an element of the wall system such as the tendons, ground anchors, or wall itself. External failure is characterized by a global failure of the wall similar to that which occurs in many slope stability problems, with the failure surface passing beyond the end of the anchors and below the toe of the wall.

The seismic loading on anchored walls is typically evaluated using pseudo-static analysis. A common method for seismic design of retaining structures is the pseudo-static method developed by Okabe (1926) and Mononobe (1929), known as the Mononobe-Okabe method.

Engineering Manual (EM) 1110-2-2504 (Design of Sheet Pile Walls) discusses earthquake forces in Section 4-6.e, where it indicates that earthquake forces should be considered "in zones of seismic activity." This section indicates that earth pressures should be determined in accordance with procedures outlined in EM 1110-2-2502 (Retaining and Flood Walls). EM 1110-2-2502 presents the Mononobe-Okabe method as well.

The Mononobe-Okabe equations use a horizontal seismic coefficient ($k_h$) and a vertical seismic coefficient ($k_v$). As taken from Ref 2, Section 5.10.2.1, the vertical acceleration is usually ignored in practice in the design of anchored structures since the vertical motions are not considered capable of applying significant loads to the anchors.

Per Ref 2, Section 5.10.2.2, design of brittle elements of the wall system should be governed by the peak force. Therefore, the peak ground acceleration (PGA) should be used with the Mononobe-Okabe equation. The section further states that the design of ductile elements should be governed by cumulative permanent seismic deformation, indicating that a horizontal seismic coefficient, $k_h$, equal to half the PGA is appropriate. For design of brittle and ductile elements per this guidance, a factor of safety of 1.1 is recommended.

From the structural load data tool referenced in Unified Facility Criteria (UFC) 3-301-01, which is available on the Whole Building Design Guide, the PGA was determined to be 0.085. Following the recommendations described in the above paragraph, the horizontal seismic coefficient, $k_h$, was set to 0.085 (design brittle elements) and 0.043 (design of ductile elements). Conservatively considering the case for design of brittle elements, the resulting dynamic active coefficient $K_{AE}$ was determined to be 0.41. Since the static active lateral earth pressure coefficient for the current feasibility level design was conservatively assumed as 0.4, negligible difference exists between the dynamic and static earth pressures. Therefore, at this design level, seismic is not considered a controlling force. Refer to the calculations in Section A.10 for further detail.

### *A.8.4. Proposed Schedule for Design and Construction*

Below is a preliminary Design and Construction schedule for the Selma, AL Flood Risk Management Study's Pre-Construction, Engineering, and Design (PED) phase and Construction Award assuming funds are received, and a design agreement is executed in FY 22. Schedule provided below is also contingent that Federal funding is included in the FY 23 President's Budget.

*Table A-18: Proposed Selma, Alabama FRM Preconstruction, Engineering, and Design (PED) Phase to Construction Award Schedule*

| ID | Task | Duration (calendar days) | Start Date | Scheduled End Date | Predecessors | Fiscal Year |
|---|---|---|---|---|---|---|
| 1 | Revise Final Report per comments and submit to Division | 35 | 12-Aug-21 | 16-Sep-21 | | **22** |
| 2 | **Signed Chief's Report** | **0** | **7-Oct-21** | **7-Oct-21** | | **22** |
| 3 | Receive Funds/Execute Design Agreement* | 180 | 7-Oct-21 | 5-Apr-22 | 2 | **22** |
| 4 | Survey | 120 | 5-Apr-22 | 3-Aug-22 | 3 | **22** |
| 5 | Geotechnical Investigations/lab results | 120 | 5-Apr-22 | 3-Aug-22 | 3 | **22** |

| | | | | | | |
|---|---|---|---|---|---|---|
| 6 | ***100% Unreviewed Design Submittal (CW310)*** | 180 | 5-Apr-22 | 2-Oct-22 | 3 | **23** |
| 7 | DQC Review and Incorporate Comments | 15 | 2-Oct-22 | 17-Oct-22 | 6 | **23** |
| 8 | Develop Final Design Package | 15 | 17-Oct-22 | 1-Nov-22 | 7 | **23** |
| 9 | ATR/VE/IEPR Concurrent Review and Incorporate Comments | 45 | 1-Nov-22 | 16-Dec-22 | 8 | **23** |
| 10 | ***RTA - Approved Plan Set (CW330)*** | 45 | 16-Dec-22 | 30-Jan-23 | 9 | **23** |
| 11 | ***Signed PPA (CW130) completed**** | 90 | 30-Jan-23 | 30-Apr-23 | 10 | **23** |
| 12 | **Signed BCOES (CW320)** | 30 | 30-Apr-23 | 30-May-23 | 11 | **23** |
| 13 | Issue Advanced Notice | 1 | 30-May-23 | 31-May-23 | 12 | **23** |
| 14 | ***Advertise (CW401)*** | 30 | 31-May-23 | 30-Jun-23 | 13 | **23** |
| 15 | ***Award Construction Contract (CC800)*** | 90 | 30-Jun-23 | 28-Sep-23 | 14 | **23** |
| 16 | **Construction Start** | 0 | 28-Sep-23 | 28-Sep-23 | | **23** |
| 17 | Issue NTP | 21 | 28-Sep-23 | 19-Oct-23 | 15 | **24** |
| 18 | Obtain Real Estate Easements | 240 | 28-Sep-23 | 25-May-24 | 17 | **24** |
| 19 | Obtain Construction Permits | 30 | 25-May-24 | 24-Jun-24 | 18 | **24** |
| 20 | Construction Complete (18 Month Duration) | 540 | 24-Jun-24 | 16-Dec-25 | 19 | **25** |

***** - Assumes FY22 Funds are provided to initiate PED Phase***
******- Assumes Construction Dollars are provided in FY23***

### *A.8.5. Hazardous and Toxic Materials*

During field investigations, the potential presence of contaminates that may be classified as hazardous and toxic material was identified at soil boring location SEL-06-21 in the area of Water Avenue and Washington Street near the Alabama River in downtown Selma (i.e., 2 blocks north and inland of the Alabama River bank / proposed project site). A sheen with a strong petroleum like odor was observed in the groundwater during soil boring at this location. The presence of the observed sheen was not encountered at any other soil boring location during field investigations. Upon further investigation it was learned that a business located in this area was previously occupied by gas / service station which is no longer open. The PDT alerted the non-Federal sponsor of our findings and the Alabama Department of Environmental Management (ADEM) was contacted by the City of Selma. No additional information was available at the time of this report.

## A.9. Water Surface Profiles

***Subpart 1: Existing Conditions***

***Figure A-81: Existing Conditions Water Surface Profiles***

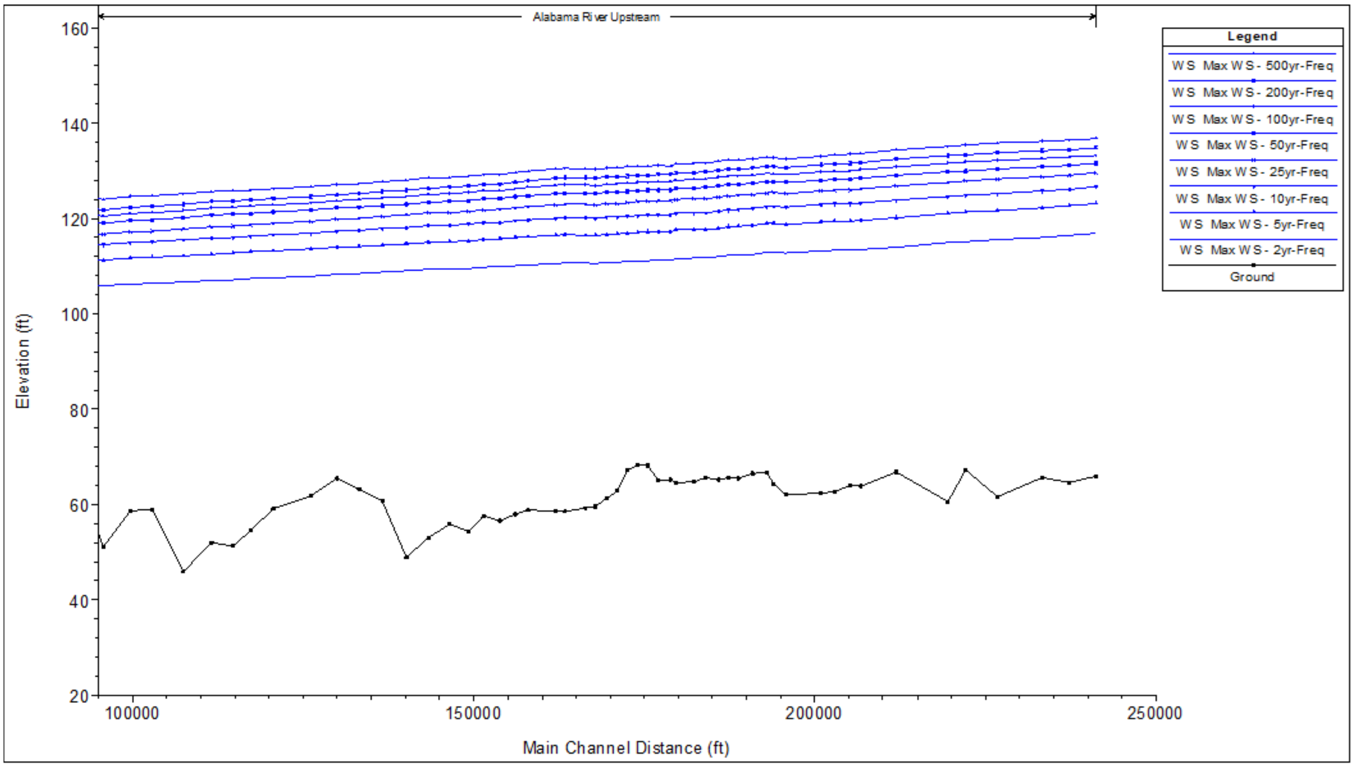


***Figure A-82: Existing Conditions Water Surface Profiles 2***

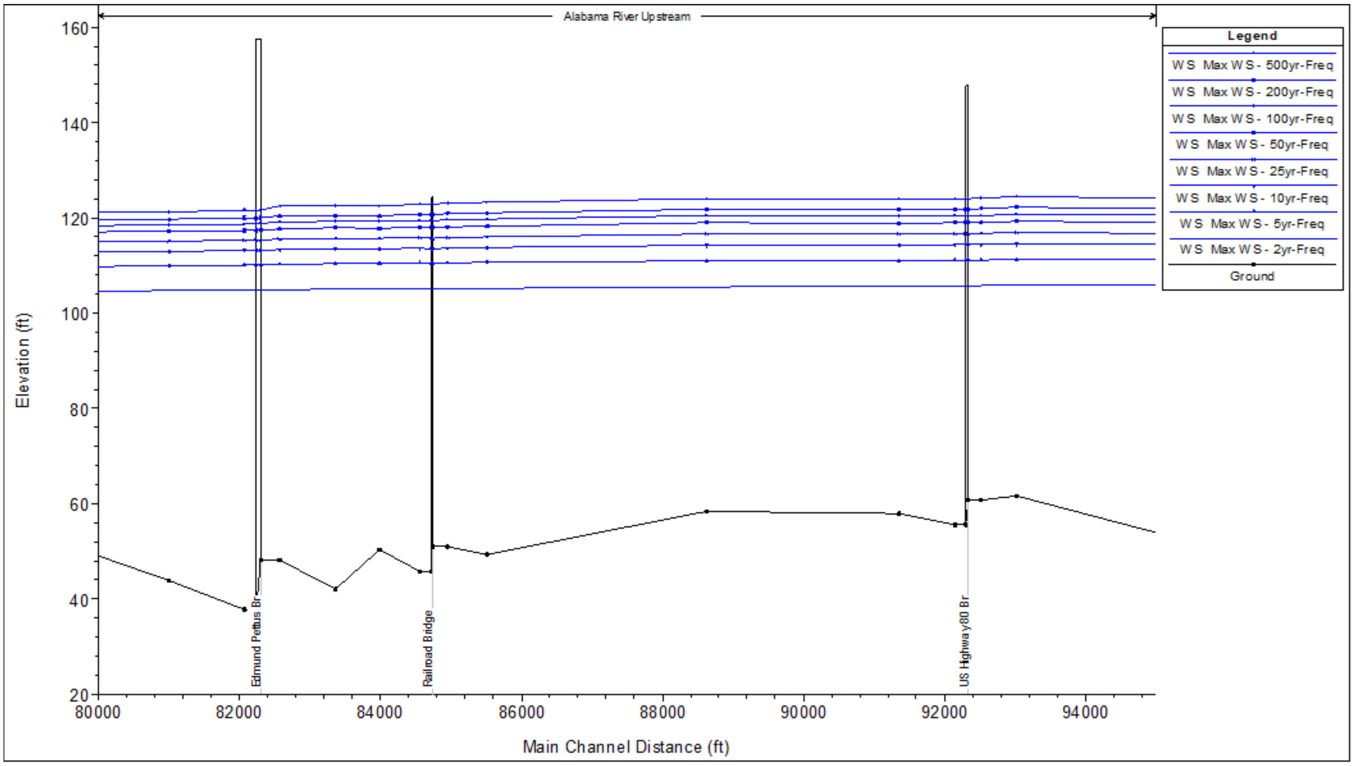



***Figure A-83: Existing Conditions Water Surface Profiles 3***

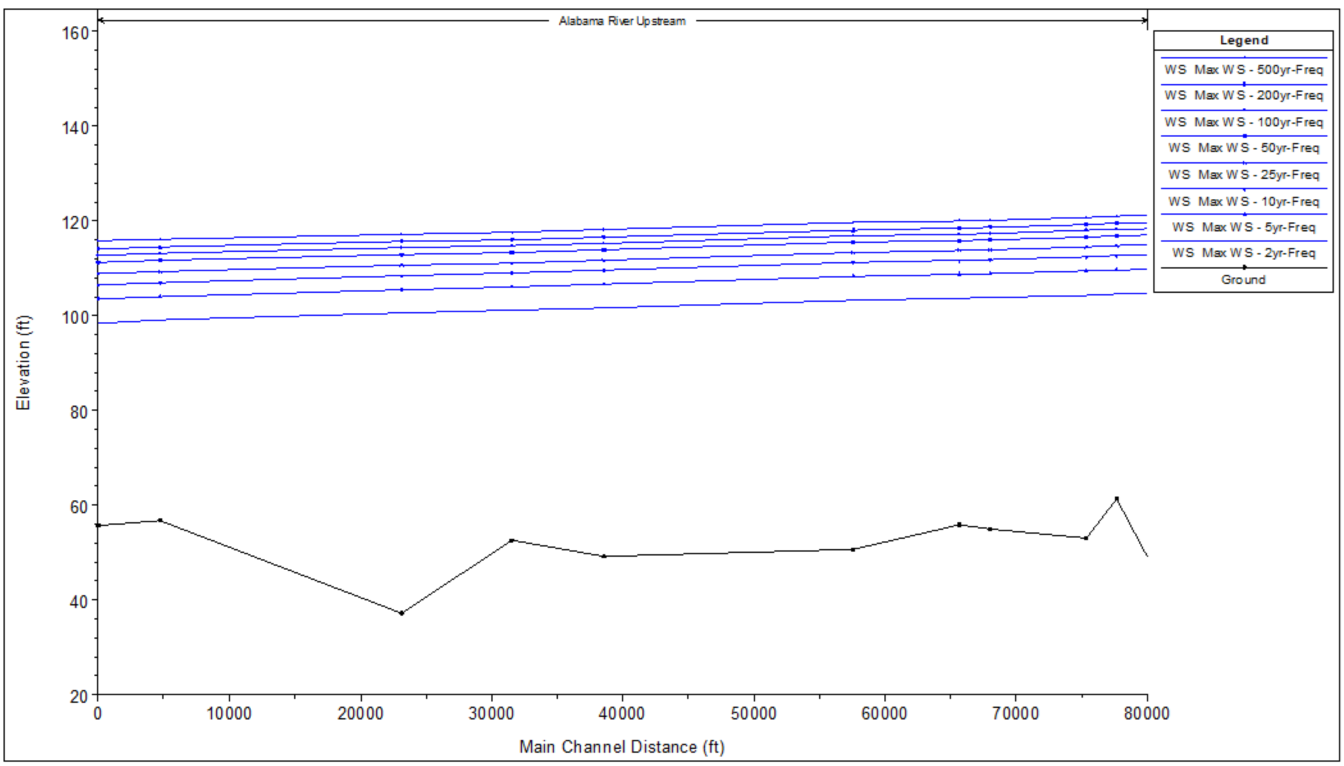
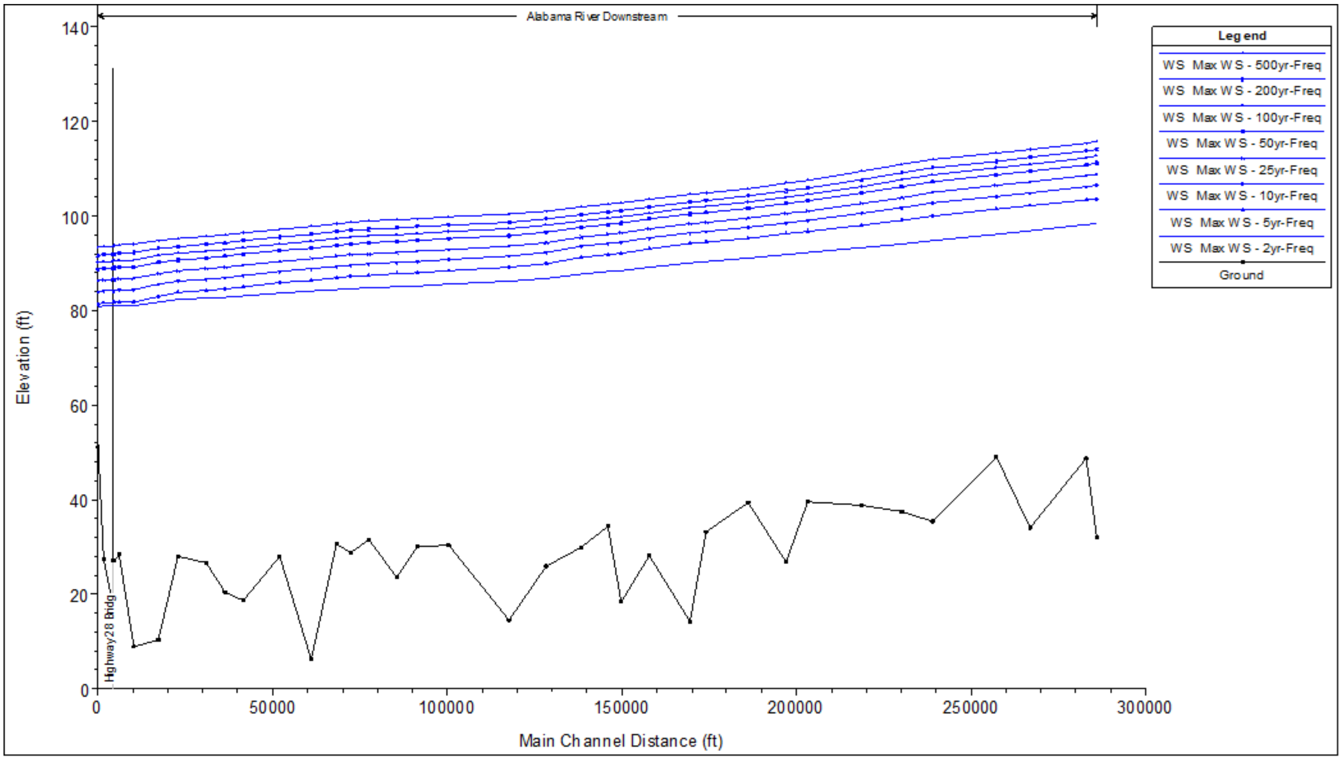


***Figure A-84: Existing Conditions Water Surface Profiles 4***




***Subpart 2: Future Without Project***

***Figure A-85: Future Without Project Water Surface Profiles***

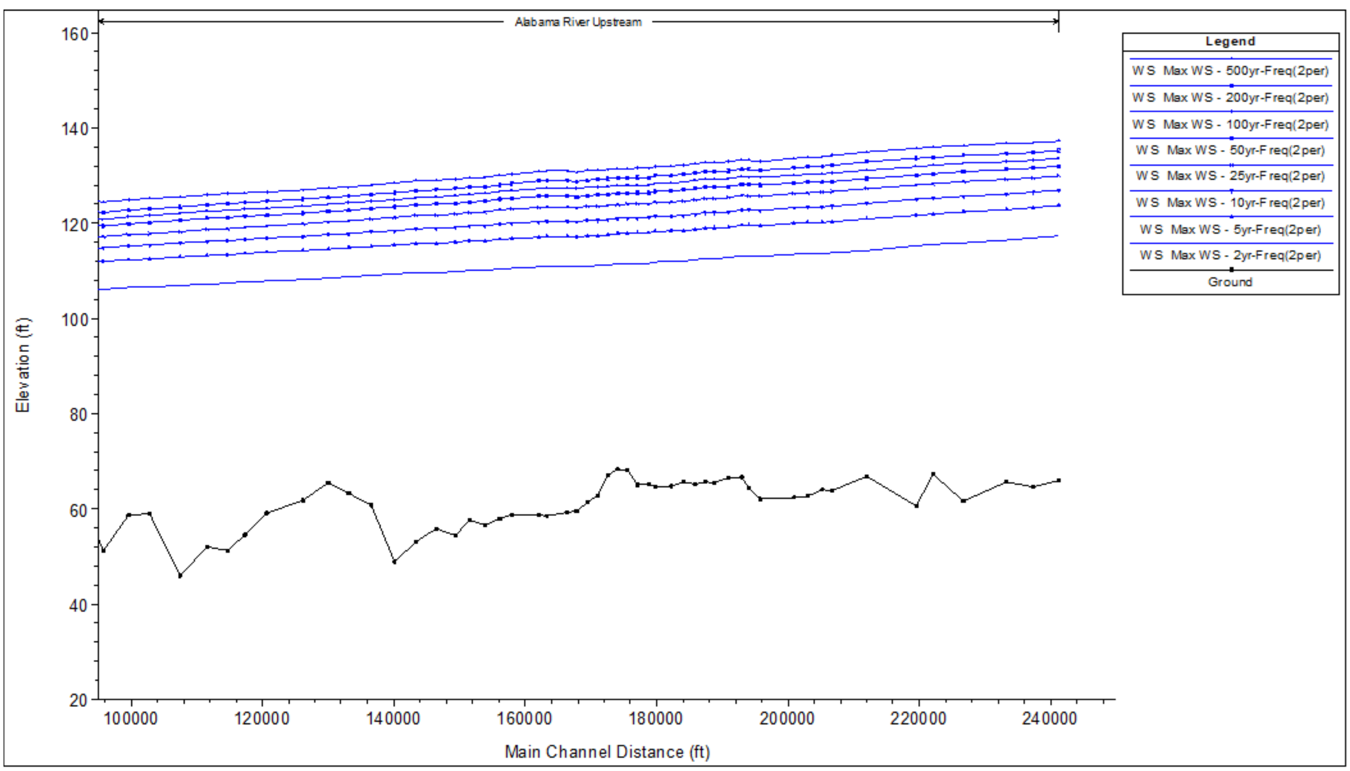


*Figure A-86: Future Without Project Water Surface Profiles 2*

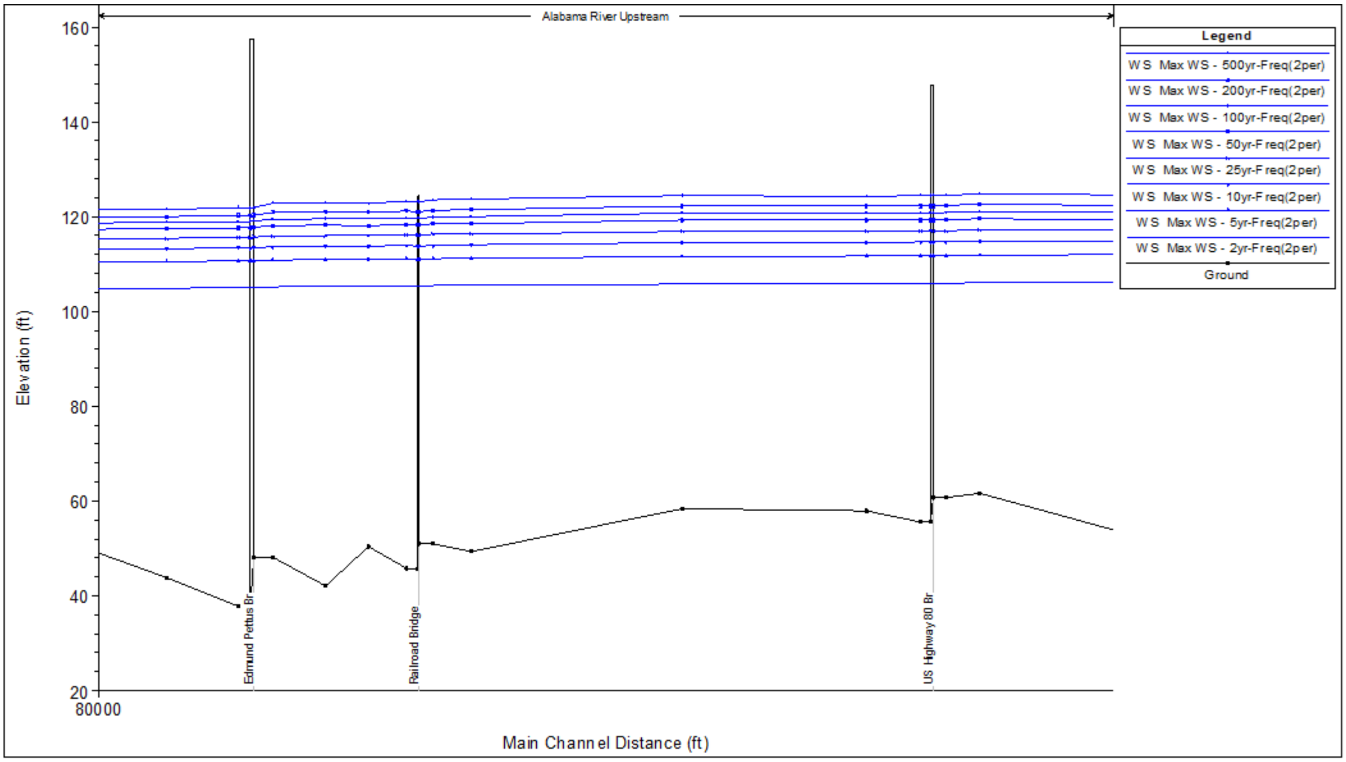



*Figure A-87: Future Without Project Water Surface Profiles 3*

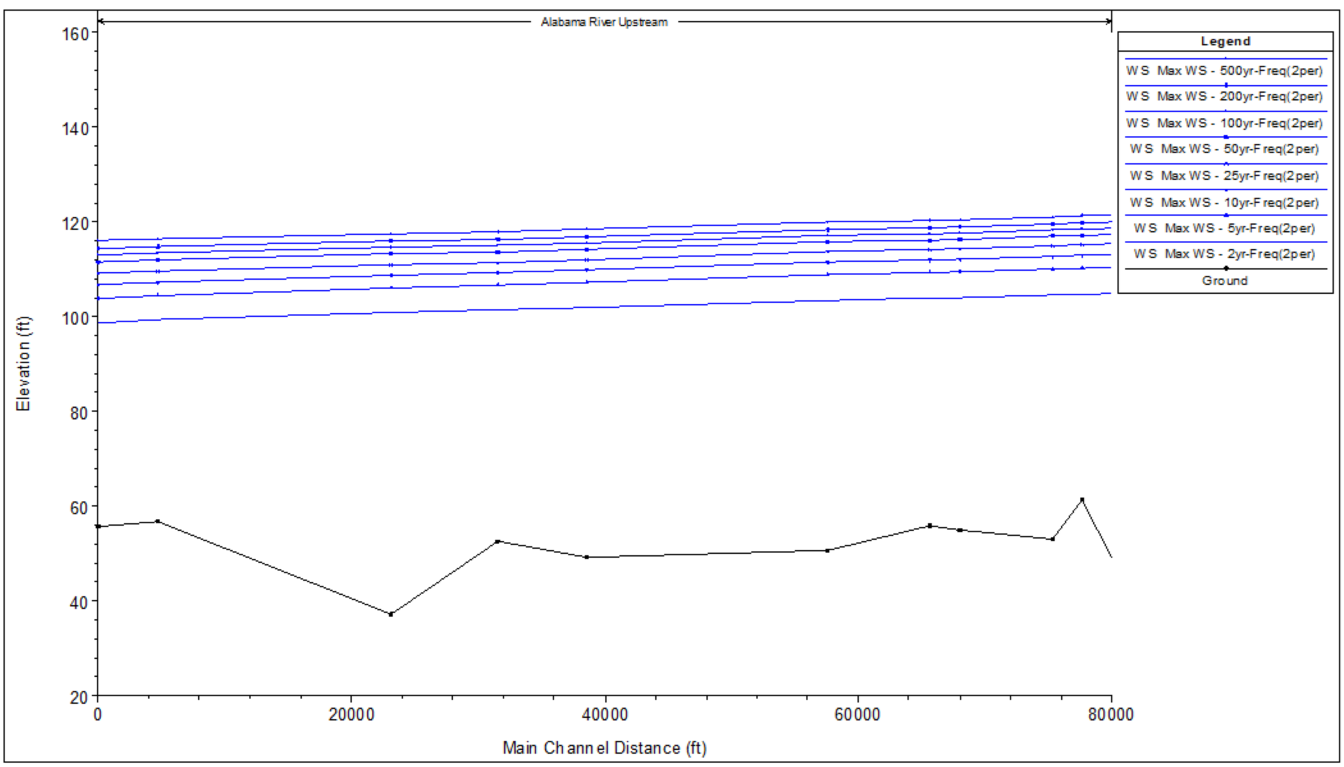
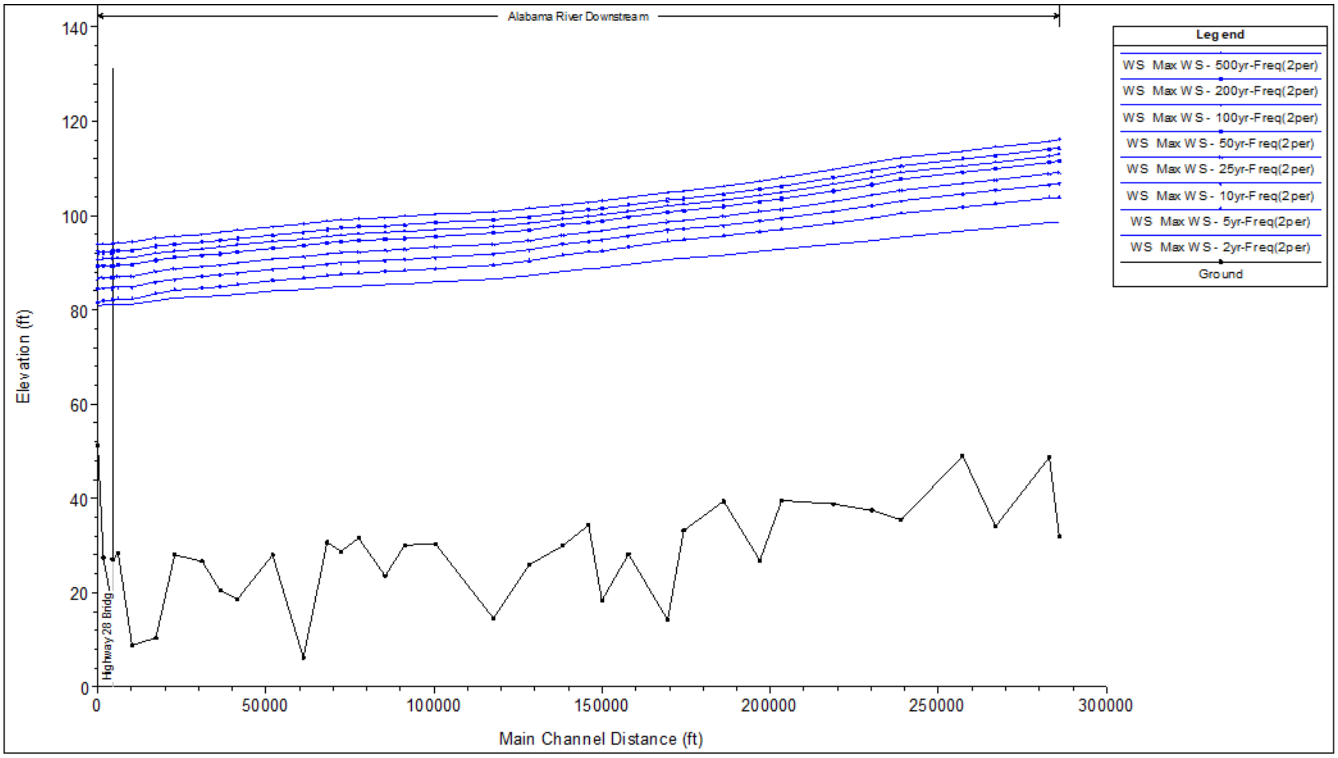
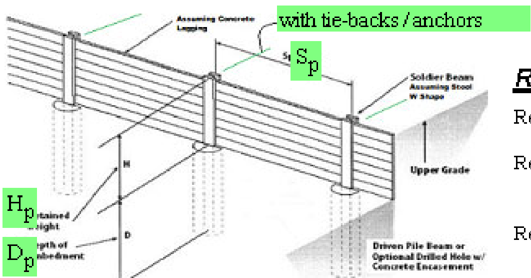


*Figure A-88: Future Without Project Water Surface Profiles 4*




## A.10.Preliminary Structural Calculations for Soldier Pile Wall

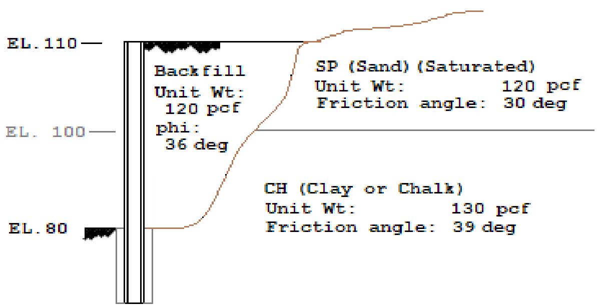
## SOLDIER BEAM RETAINING WALL DESIGN



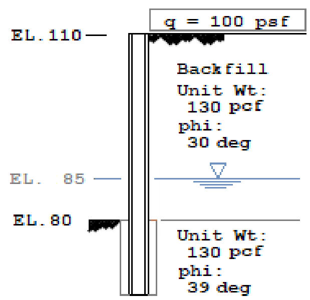

3D Conept

### *References:*

| | |
|---|---|
| Reference 1: | 2007 AASHTO LRFD Bridge Design Specifications |
| Reference 2: | Publication No. FHWA-IF-99-015, Geotechnical Engineering Circular No. 4 (June 1999) |
| Reference 3: | Modeling Soil Behavior with Simple Springs, Part 1 (Bohnhoff, April 2014) |




Design Cross Section




Simplified Design Cross Section

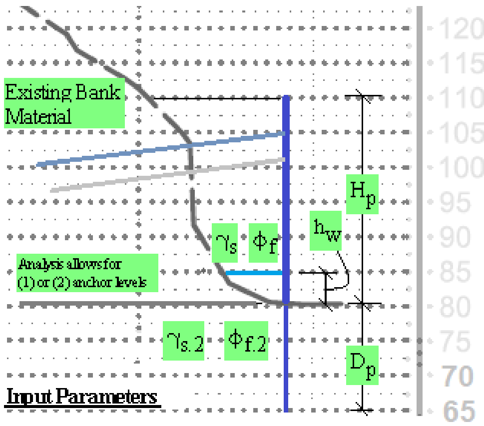
(Anchors not shown for clarity)

US Army Corps of Engineers
Mobile District

**SELMA BANKLINE STABILIZATION**
**Feasibility Level Design**

Computed By: ALJ
Date: 4/9/2021
Checked By: JCO
Page: 2 of 37

## Initial Input Parameters




| | |
|---|---|
| Retained Height: | $H_p := 30ft$ |
| Embedment Depth: | $D_p := 15ft$ |
| Spacing of Soldier Piles / Beams: | $S_p := 8ft$ |
| effective unit weight of soil: (layer 1 /backfill) | $\gamma_s := 130pcf$ |
| effective angle of friction: (layer 1 /backfill) | $\phi_f := 30deg$ |
| effective unit weight of soil: (layer 2, @ embedment) | $\gamma_{s.2} := 130pcf$ |
| effective angle of friction: (layer 2, @ embedment) | $\phi_{f.2} := 39deg$ |
| Height of water assumed: | $h_w := 5ft$ |

## Loading

Unfactored loads / service loads, designing to an allowable stress or factor of safety is used.

### Lateral Earth Pressure

Per Reference 2, Section 5.11.5, anchored walls for highway applications are most often constructed from the top of the wall to the base of the excavation (i.e., top-down construction). However, the design concept for the subject project is a fill situation, in which the wall is planned to be constructed from the base to the top (i.e., bottom-up construction).

Reference 2, Section 5.11.5 states, "Design loadings for fill anchored walls are based on earth pressures acting on the wall when the wall is completely backfilled and all surcharge loadings are applied."

With the type of incrimental backfilling and staged load testing for a fill anchored wall, "the ground anchors will typically be designed to carry actual earth pressure loads as compared to loads from apparent earth pressure envelopes as may be used for anchored systems constructed from the top-down. The pattern of wall movement for a fill anchored wall is consistent with theoretical earth pressure envelopes".

*Lateral Earth Pressure Coefficients*

Reference 2 - FHWA-IF-99-015, Section 5.2.4

1st layter / Backfill

Coefficient of at-rest earth pressure
(for normally consolidated soil).

$$K_0 := 1 - \sin(\phi_f) = 0.5$$

Rankine active pressure coefficient

$$K_a := \tan\left(45\deg - \frac{\phi_f}{2}\right)^2 = 0.33$$

Rankine passive earth pressure coefficient

$$K_p := \tan\left(45\deg + \frac{\phi_f}{2}\right)^2 = 3$$

User input lateral pressure coefficient:

$$K_{input} := 0.4$$

Second layer of soil (at embedment)

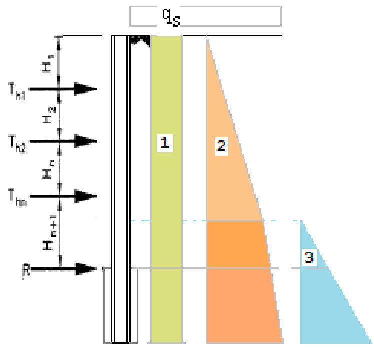
$$K_{o.2} := 1 - \sin(\phi_{f.2}) = 0.37$$

$$K_{a.2} := \tan\left(45\deg - \frac{\phi_{f.2}}{2}\right)^2 = 0.23$$

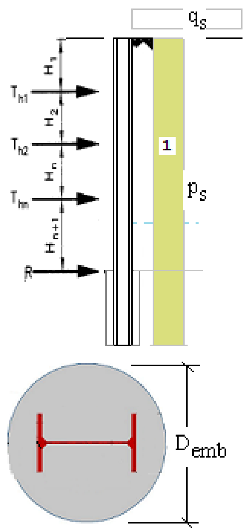
$$K_{p.2} := \tan\left(45\deg + \frac{\phi_{f.2}}{2}\right)^2 = 4.4$$

*Lateral Pressure Diagram*

Analysis considers two or three anchor levels.



## 1. Lateral Pressure from Surcharge




Vertical ground pressure: $q_s := 100psf$

Lateral surcharge pressure: $p_s := K_a \cdot q_s = 33.33 \cdot psf$

(Assumed to act uniformly over the entire wall height)

Assumed diameter of embedded section:

$D_{emb} := 2ft$

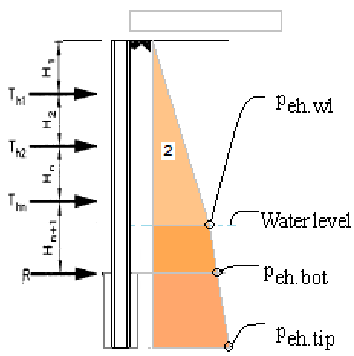
Distributed surcharge load to pile, above embedment:

$\boxed{P_s := p_s \cdot S_p = 0.27 \cdot klf}$

Distributed surcharge load to pile, along embedment:

$\boxed{P_{s.emb} := p_s \cdot D_{emb} = 0.07 \cdot klf}$

## 2. Soil Pressure




Active, at-rest, or modified coefficient input:

$K_a = 0.33$

$K_{input} = 0.4$

$K_{use} := K_{input}$

$p_{eh.wl} := K_{use} \cdot \gamma_s \cdot (H_p - h_w) = 1300 \cdot psf$

$p_{eh.bot} := p_{eh.wl} + K_{use} \cdot (\gamma_s - \gamma_w) \cdot h_w = 1435.2 \cdot psf$

$p_{eh.tip} := p_{eh.bot} + K_{use} \cdot (\gamma_s - \gamma_w) \cdot D_p = 1840.8 \cdot psf$

Distributed triangular loads to pile:

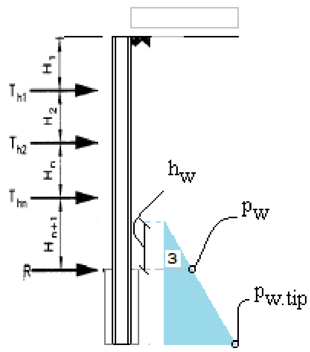
$\boxed{P_{eh.wl} := p_{eh.wl} \cdot S_p = 10.4 \cdot klf}$

$\boxed{P_{eh.bot} := p_{eh.bot} \cdot S_p = 11.48 \cdot klf}$

$\boxed{P_{eh.bot.emb} := p_{eh.bot} \cdot D_{emb} = 2.87 \cdot klf}$

$\boxed{P_{eh.tip} := p_{eh.tip} \cdot D_{emb} = 3.68 \cdot klf}$

3. Water Pressure




Height of water: $h_w = 5\,ft$

Unit weight of water: $\gamma_w = 62.4 \cdot pcf$

$p_w := \gamma_w \cdot h_w = 312 \cdot psf$

$p_{w.tip} := \gamma_w \cdot \left(h_w + D_p\right) = 1248 \cdot psf$

Distributed triangular loads to pile:

$\boxed{P_w := p_w \cdot S_p = 2.5 \cdot klf}$

$\boxed{P_{w.tip} := p_{w.tip} \cdot D_{emb} = 2.5 \cdot klf}$

## Earthquake Forces

Per guidance from Ref 2, Section 5.10.1, seismic loading will be evaluated using a pseudo-static analysis (Mononobe-Okabe equations). Vertical seismic forces are neglected. This reference recommends values for the horizontal seismic coefficient ($k_h$) depending on the element being designed.

- Design of brittle elements: kh = full peak ground acceleration (PGA)
- Design of ductile elements: kh = 0.5*PGA

The seismic data for Selma, AL was found from the stuctural load data tool for Unified Facility Criteria (UFC) 3-301-01, available through the Whole Building Design Guide website.

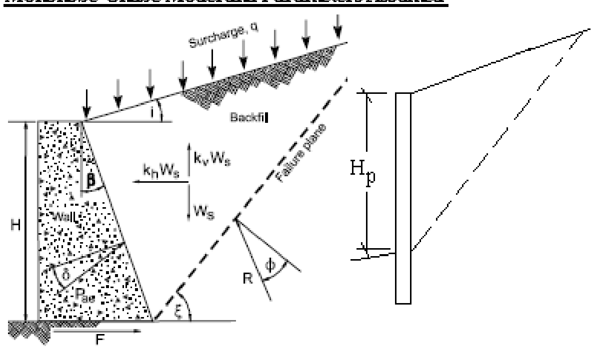
Peak Ground Acceleration: $PGA := 0.085$

Horizontal seismic coefficient (brittle element design): $k_{hb} := PGA = 0.09$

Horizontal seismic coefficient (ductile element design): $k_{hd} := 0.5 \cdot PGA = 0.04$

Vertical seismic coefficient: $k_v := 0$

**Mononobe-Okabe Model and Parameters Assumed**




$H_{eq} := H_p = 30\,ft$

$\gamma_{eq} := \gamma_s = 130 \cdot pcf$

$\phi_{eq} := \phi_f = 30 \cdot deg$

$\delta_{eq} := 0deg$

$\beta_{eq} := 0deg$

$i_{eq} := 5deg$

## Mononobe-Okabe Equations

### Brittle Element Design

$$\theta_b := \operatorname{atan}\left(\frac{k_{hb}}{1-k_v}\right) = 4.86\cdot°$$

$$D_{eq.b} := \left(1 + \sqrt{\frac{\sin(\phi_{eq}+\delta_{eq})\cdot\sin(\phi_{eq}-\theta_b-i_{eq})}{\cos(\delta_{eq}+\beta_{eq}+\theta_b)\cdot\cos(i_{eq}-\beta_{eq})}}\right)^2 \qquad D_{eq.b} = 2.01$$

$$K_{AE.b} := \frac{\left(\cos(\phi_{eq}-\theta_b-\beta_{eq})\right)^2}{\cos(\theta_b)\cdot\left(\cos(\beta_{eq})\right)^2\cdot\cos(\beta_{eq}+\delta_{eq}+\theta_b)\cdot D_{eq.b}} \qquad K_{AE.b} = 0.41$$

$$P_{AE.b} := \frac{1}{2}\cdot K_{AE.b}\cdot\gamma_{eq}\cdot H_{eq}^{\ 2}\cdot(1-k_v) \qquad \boxed{P_{AE.b} = 24.07\cdot klf}$$

### Ductile Element Design

$$\theta_d := \operatorname{atan}\left(\frac{k_{hd}}{1-k_v}\right) = 2.43\cdot°$$

$$D_{eq.d} := \left(1 + \sqrt{\frac{\sin(\phi_{eq}+\delta_{eq})\cdot\sin(\phi_{eq}-\theta_d-i_{eq})}{\cos(\delta_{eq}+\beta_{eq}+\theta_d)\cdot\cos(i_{eq}-\beta_{eq})}}\right)^2 \qquad D_{eq.d} = 2.07$$

$$K_{AE.d} := \frac{\left(\cos(\phi_{eq}-\theta_d-\beta_{eq})\right)^2}{\cos(\theta_d)\cdot\left(\cos(\beta_{eq})\right)^2\cdot\cos(\beta_{eq}+\delta_{eq}+\theta_d)\cdot D_{eq.d}} \qquad K_{AE.d} = 0.38$$

$$P_{AE.d} := \frac{1}{2}\cdot K_{AE.d}\cdot\gamma_{eq}\cdot H_{eq}^{\ 2}\cdot(1-k_v) \qquad \boxed{P_{AE.d} = 22.24\cdot klf}$$

### *Analysis Performed in RAM Elements*

#### *Soil Springs*

*Modeling Soil Behavior with Simple Springs, Part 1 (Bohnhoff April 2014)*

From Bohnhoff article:

**Table 1. Presumptive Properties for Silt and Clay (Cohesive) Soils**

| Soil Type | Unified Soil Classification | Consistency | Moist unit weight, $\gamma$ | Undrained soil shear strength[a], $S_U$ | Young's modulus for soil, $E_S$[b][c] |
|---|---|---|---|---|---|
| | | | lbf/ft³ | lbf/in² | lbf/in² |
| Homogeneous inorganic clay, sandy or silty clay | CL | Soft | 125 | 3.5 | 3920 |
| | | Medium to Stiff | 130 | 7 | 6160 |
| | | Very Stiff to Hard | 135 | 14 | 8400 |
| Homogeneous inorganic clay of high plasticity | CH | Soft | 110 | 3.5 | 1680 |
| | | Medium to Stiff | 115 | 7 | 2800 |
| | | Very Stiff to Hard | 120 | 14 | 4480 |

(a) Loading assumed slow enough that sandy soils behave in a drained manner.
(b) Estimate of stiffness at rotation of 1° for use in approximating structural load distribution. For evaluation of serviceability limit state, use values that are 1/3 of tabulated value.
(c) Constant values of stiffness used for calculation of clay response. Stiffness increasing with depth from a value of zero used for calculation of sand response.

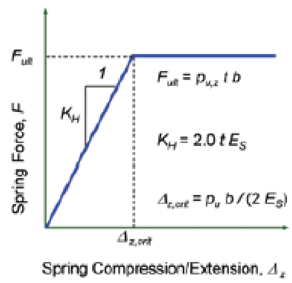



**Figure 4.** Load-displacement relationship for a soil spring.

b - width of element in contact with soil

$b := D_{emb} = 2\,ft$

t - thickness of soil layer representing the spring

$t := 12in$

Where assuming cohesive soil, Table 1 values to use:

Undrained soil shear strength: $S_u := 7psi$

Young's modulus: $E_{s.c} := 934psi$

Moist unit weight of soil: $\gamma_{sp.c} := 115pcf$

**Table 2. Presumptive Properties for Sand and Gravel (Cohesionless) Soils**

| Soil Type | Unified Soil Classification | Consistency | Moist unit weight, $\gamma$ | Drained soil friction angle, $\phi$[a] | Increase in Young's modulus per unit depth below grade[b][c][d], $A_E$ | |
|---|---|---|---|---|---|---|
| | | | lbf/ft³ | Deg | $\frac{lbf/in^2}{/ft}$ | $\frac{lbf/in^2}{/in}$ |
| Silty or clayey fine to coarse sand | SM, SC, SP-SM, SP-SC, SW-SM SW-SC | Loose | 105 | 30 | 440 | 37 |
| | | Medium to Dense | 110 | 35 | 660 | 55 |
| | | Very Dense | 115 | 40 | 880 | 73 |

(a) Rapid undrained loading will typically be the critical design scenario in these soils. Laboratory testing is recommended to assess clay friction angle for drained loading analysis.
(b) Estimate of stiffness at rotation of 1° for use in approximating structural load distribution. For evaluation of serviceability limit state, use values that are 1/3 of tabulated value.
(c) Constant values of stiffness used for calculation of clay response. Stiffness increasing with depth from a value of zero used for calculation of sand response.
(d) Assumes soil is located below the water table. Double the tabulated $A_E$ value for soils located above the water table.

Where assuming cohesionless soil, Table 2 values to use:

Drained soil friction angle: $\phi_{sp} := 35deg$

Young's modulus (per inch depth): $E_{s.sp} := 19\frac{psi}{in}$ 1/3 of tabulated value per note (b)

Moist unit weight of soil: $\gamma_{sp} := 110pcf$

**Sample calculations for 1st spring location:**

1st Spring Location: $z_1 := 6in$

**For case assuming cohesive soil**

Horizontal spring stiffness for 1st spring: $\boxed{K_{h1.c} := 2 \cdot t \cdot E_{s.c} = 22416 \cdot \frac{lbf}{in}}$

Ultimate lateral soil resistance @ 1st spring:

$3 \cdot S_u \cdot \left[1 + \frac{z}{(2b)}\right]$ for_0 < z < 4b

$9 \cdot S_u$ for_z > 4b

$p_{u.sp1.c} := 3 \cdot S_u \cdot \left[1 + \left(\frac{z_1}{2 \cdot b}\right)\right] = 23.63 \cdot psi$

Maximum force allowed in 1st spring: $\boxed{F_{ult.sp1.c} := p_{u.sp1.c} \cdot t \cdot b = 6804 \cdot lbf}$

**For case assuming cohesionless soil**

Young's modulus @ 1st spring: $E_{s.sp1} := E_{s.sp} \cdot z_1 = 114 \cdot psi$

Horizontal spring stiffness for 1st spring: $\boxed{K_{h1} := 2 \cdot t \cdot E_{s.sp1} = 2736 \cdot \frac{lbf}{in}}$

Passive coefficient of lateral pressure: $K_{p.sp} := \frac{1 + \sin(\phi_{sp})}{1 - \sin(\phi_{sp})} = 3.69$

Pore water pressure @ 1st spring: $u_{w1} := \gamma_w \cdot z_1 = 31.2 \cdot psf$

Eff vertical soil stress @ 1st spring: $\sigma_{v1} := \gamma_{sp} \cdot z_1 - u_{w1} = 0.17 \cdot psi$

Ultimate lateral soil resistance @ 1st spring: $p_{u.sp1} := 3 \cdot \sigma_{v1} \cdot K_{p.sp} = 1.83 \cdot psi$

Maximum force allowed in 1st spring: $\boxed{F_{ult.sp1} := p_{u.sp1} \cdot t \cdot b = 526.96 \cdot lbf}$

For each of these conditions, a model in RAM Elements structural analysis software was analyized, with springs located (both on the river side and land side) along the penetration depth for z = 6", 18", 30"....174". The remaining spring parameters were calculated with Excel and are attached.

For the embedded portion, rigid link members are provided at each spring location. At the end of these links, compression springs (with calculated stiffnesses) were specified.

## Anchor Springs

For each of the conditions of cohesive and cohesionless surrounding soils, cases with 2 and 3 anchors will be analyzed.

### Assumptions (can be adjusted based on final anchor selections, as felt necessary to refine design)

Anchor yield stress (Fu = 150 ksi): $F_{y.a.sp} := 120\text{ksi}$

Modulus of Elasticity: $E_{a.sp} := 29000\text{ksi}$

Anchor area (diameter = 1.25 in): $A_{a.sp} := 1.23\text{in}^2$

Assumes horizontal anchor, adequate at this level of design.

Top/1st anchor unbonded length: $L_{a1.u} := 18\text{ft}$

2nd anchor unbonded length: $L_{a2.u} := 15\text{ft}$

3rd anchor unbonded length: $L_{a3.u} := 12\text{ft}$

Limiting / yielding tension force: $F_{t.a} := A_{a.sp} \cdot F_{y.a.sp} = 147.6\,\text{kip}$

Top/1st anchor spring parameters

Spring stiffness, k

$$k_{a1} := \frac{E_{a.sp} \cdot A_{a.sp}}{L_{a1.u}} = 165.14\,\frac{\text{kip}}{\text{in}}$$

Limiting displacement

$$\Delta_{t.a1} := \frac{F_{t.a} \cdot L_{a1.u}}{E_{a.sp} \cdot A_{a.sp}} = 0.89\,\text{in}$$

2nd anchor spring parameters

Spring stiffness, k

$$k_{a2} := \frac{E_{a.sp} \cdot A_{a.sp}}{L_{a2.u}} = 198.17\,\frac{\text{kip}}{\text{in}}$$

Limiting displacement

$$\Delta_{t.a2} := \frac{F_{t.a} \cdot L_{a2.u}}{E_{a.sp} \cdot A_{a.sp}} = 0.74\,\text{in}$$

3rd anchor spring parameters

Spring stiffness, k

$$k_{a3} := \frac{E_{a.sp} \cdot A_{a.sp}}{L_{a3.u}} = 247.71\,\frac{\text{kip}}{\text{in}}$$

Limiting displacement

$$\Delta_{t.a3} := \frac{F_{t.a} \cdot L_{a3.u}}{E_{a.sp} \cdot A_{a.sp}} = 0.6\,\text{in}$$

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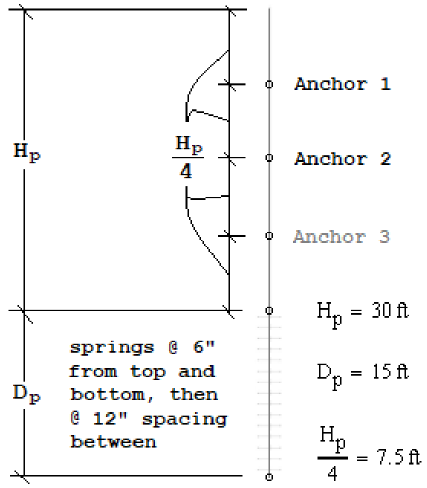
**Feasibility Level Design**



## Model Configuration Considered for All Cases

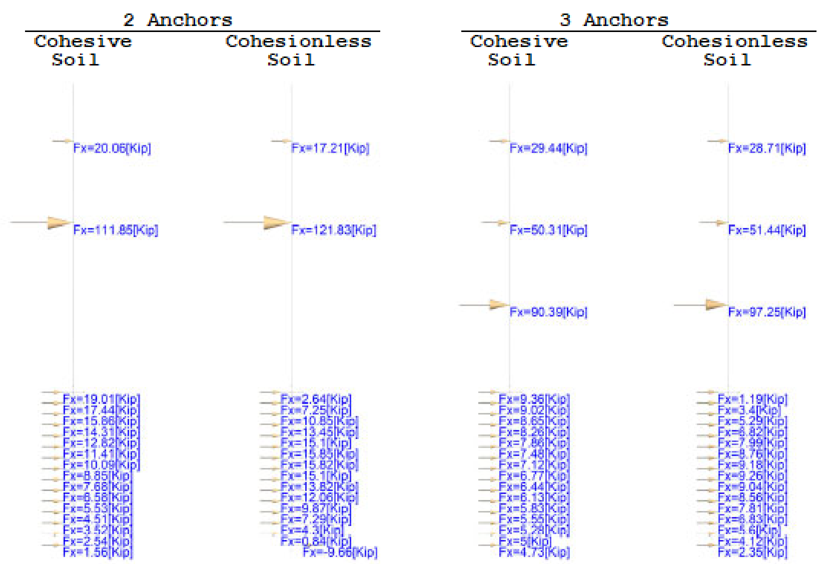



Section in Model is HP14x117

Loading is combined lateral pressure of surcharge, lateral earth pressure, hydrostatic, and seismic / dynamic.

## Model Output

### Reactions



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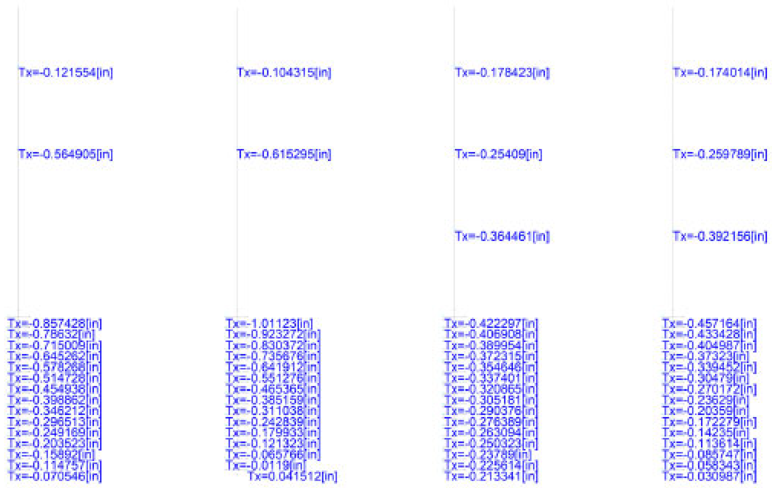
|---|---|---|

## Node / Spring Displacements

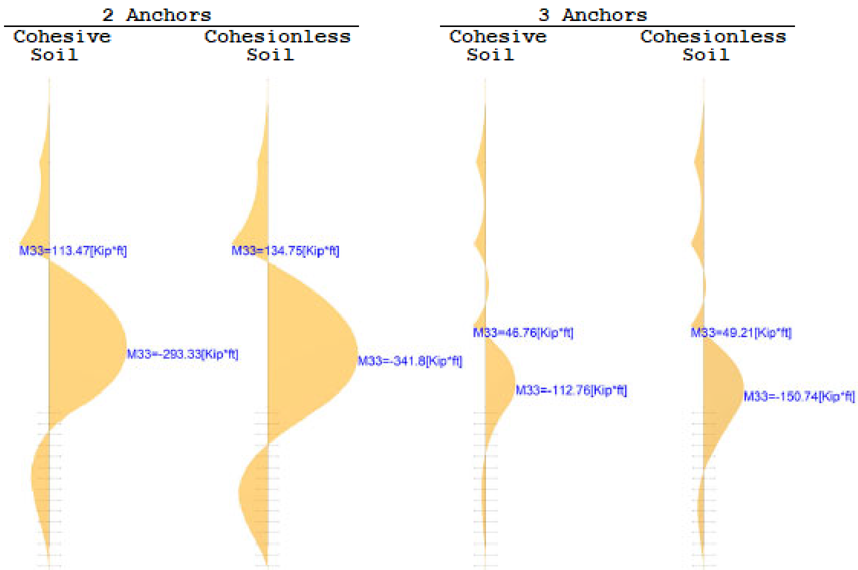
| 2 Anchors | | 3 Anchors | |
|---|---|---|---|
| Cohesive Soil | Cohesionless Soil | Cohesive Soil | Cohesionless Soil |




## Pile Deformed Shape and Max Deflection

| 2 Anchors | | 3 Anchors | |
|---|---|---|---|
| Cohesive Soil | Cohesionless Soil | Cohesive Soil | Cohesionless Soil |

## Bending Moments




## Noted Source of Error

Linear vs. Non-linear Analysis

The software does not perform a non-linear analysis to account for the springs reaching their ultimate displacement and ultimate force, as computed above. Even for the case where 3 anchors are used, the first spring (z = 6") reaction is reported as 9.36 kip (vs. $F_{ult}$ = 6.8 kip) for the cohesive condition and 1.19 kip (vs. $F_{ult}$ = 0.53 kip) for the cohesionless condition. This source of error is noted but ignored at this level of the design.

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### *Forces for Design*

#### Summary of Analysis Output

| Forces | 2 Anchors: Cohesive Soil | 2 Anchors: Cohesionless Soil | 3 Anchors: Cohesive Soil | 3 Anchors: Cohesionless Soil |
|---|---|---|---|---|
| Anchor Reactions | | | | |
| Anchor 1 | $T_{2a.1.c} := 20kip$ | $T_{2a.1} := 17.5kip$ | $T_{3a.1.c} := 29.5kip$ | $T_{3a.1} := 29kip$ |
| Anchor 2 | $T_{2a.2.c} := 112kip$ | $T_{2a.2} := 122kip$ | $T_{3a.2.c} := 51kip$ | $T_{3a.2} := 52kip$ |
| Anchor 3 | | | $T_{3a.3.c} := 91kip$ | $T_{3a.3} := 98kip$ |
| Total Embedment Reaction | $R_{2a.c} := 142kip$ | $R_{2a} := 135kip$ | $R_{3a.c} := 104kip$ | $R_{3a} := 97kip$ |
| Bending | $M_{2a.c} := 294kip\cdot ft$ | $M_{2a} := 342kip\cdot ft$ | $M_{3a.c} := 113kip\cdot ft$ | $M_{3a} := 151kip\cdot ft$ |
| Member Deflection | $\Delta_{2a.max} := 1.372in$ | | $\Delta_{3a.max} := 0.419in$ | |

#### Forces per Number of Anchor Levels Considered

Number of anchor levels: $N_{AL} := 3$

**Bending**

$$M_{max} := \begin{cases} \max(M_{3a.c}, M_{3a}) & \text{if } N_{AL} = 3 \\ \max(M_{2a.c}, M_{2a}) & \text{otherwise} \end{cases}$$

$$\boxed{M_{max} = 151\cdot kip\cdot ft}$$

**Max Total Reaction @ Embedment**

$$R_{max} := \begin{cases} \max(R_{3a.c}, R_{3a}) & \text{if } N_{AL} = 3 \\ \max(R_{2a.c}, R_{2a}) & \text{otherwise} \end{cases}$$

$$\boxed{R_{max} = 104\cdot kip}$$

**Anchor Max Tension Force**

$$T_{anc.max} := \begin{cases} \max(T_{3a.1.c}, T_{3a.2.c}, T_{3a.3.c}, T_{3a.1}, T_{3a.2}, T_{3a.3}) & \text{if } N_{AL} = 3 \\ \max(T_{2a.1.c}, T_{2a.2.c}, T_{2a.1}, T_{2a.2}) & \text{otherwise} \end{cases}$$

$$\boxed{T_{anc.max} = 98\cdot kip}$$

### *Design*

### *Soldier Beam Section*

Maintain bending design per Geotechnical Engineering Circular No. 4. Section 5.4.1 indicates allowable stress design, with a recommended allowable stress of 0.55 Fy.

Modulus of Elasticity: $E := 29000\text{ksi}$

Steel Yield Stress/Strength: $F_y := 50\text{ksi}$

Allowable Bending Stress: $F_b := 0.55 \cdot F_y$ $F_b = 27.5 \cdot \text{ksi}$

Required Section Modulus: $S_{req} := \dfrac{M_{max}}{F_b}$ $\boxed{S_{req} = 65.89 \cdot \text{in}^3}$

Assuming a W or HP Section for the soldier beam, select from the AISC sections below.

W Section Table

SaveData( data , Get_Save_Clear , index) := W14X132HSS24X12X3/4

( data  Get_Save_Clear  index )

BM :=

W14X311
W14X283
W14X257
W14X233
W14X211
W14X193
W14X176
W14X159
W14X145
W14X132
W14X120
W14X109
W14X99

SaveData( "" , "Get" , 0)

**Alternately, use an HP section.**

**HP14x117**

**$S_x = 172\ in^3$**

SaveData(BM , "Save" , 0) = "W14X132"

BM = "W14X132"

V Lookup Functions

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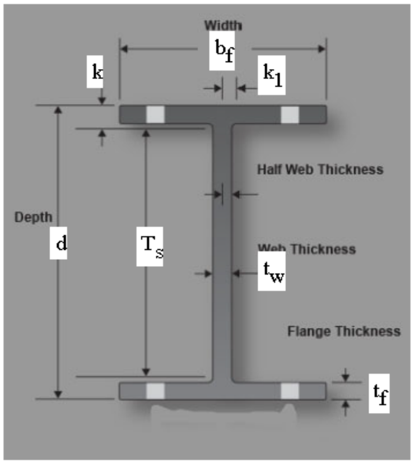


## *Section Properties*

$d = 14.7 \cdot in$

$b_f = 14.7 \cdot in$

$t_w = 0.65 \cdot in$

$t_f = 1.03 \cdot in$

$k = 1.63 \cdot in$

$k_1 = 1.56 \cdot in$

$T_s = 11.44 \cdot in$

$\frac{b_f}{2 \cdot t_f}$ $r_{b.f\_2t.f} = 7.15$

$\frac{h}{t_w}$ $r_{h\_t.w} = 17.7$

Distance between flange Centroids: $h_0 = 13.7 \cdot in$

| | |
|---|---|
| Beam Weight: | $W_{BM} = 132 \cdot plf$ |
| Beam Area: | $A_s = 38.8 \cdot in^2$ |
| Section Moment of Inertia about the X Axis: | $I_x = 1530 \cdot in^4$ |
| Section Modulus about the X Axis: | $S_x = 209 \cdot in^3$ |
| Radius of Gyration about X: | $r_x = 6.28 \cdot in$ |
| Plastic Section Moduls about X: | $Z_x = 234 \cdot in^3$ |
| Section Moment of Inertia about the Y Axis: | $I_y = 548 \cdot in^4$ |
| Section Modulus about the Y Axis: | $S_y = 74.5 \cdot in^3$ |
| Radius of Gyration about Y: | $r_y = 3.76 \cdot in$ |
| Plastic Section Moduls about Y: | $Z_y = 113 \cdot in^3$ |

| | |
|---|---|
| Torsional Constant: | $J_s = 12.3 \cdot in^4$ |
| Warping Constant: | $C_w = 25500 \cdot in^6$ |
| Effective Radius of Gyration: | $r_{ts} = 4.23 \cdot in$ |

### *ADDITIONAL PROPERTIES*

$T_s = 11.44 \cdot in$

$\phi_b := 0.90$ (AISC, Sec. F1)

$\phi_v := 0.90$ (AISC, Sec. G1)

Check using required section modulus:

$S_x = 209 \cdot in^3$ $\quad S_{req} = 65.89 \cdot in^3$

$\text{if}\left(S_x > S_{req}, \text{"OK"}, \text{"NO GOOD"}\right) = \text{"OK"}$

Check using AISC (360-10) Chapter F - Design of Members for Flexure:

$M_{n.all} = 583.83 \cdot kip \cdot ft$ $\quad M_{max} = 151 \cdot kip \cdot ft$

$\text{if}\left(M_{n.all} > M_{max}, \text{"OK"}, \text{"NO GOOD"}\right) = \text{"OK"}$

## Soldier Beam Deflection

### Deflection Output

$$\Delta_{max} := \begin{cases} \max\left(\Delta_{3a.max}\right) & \text{if } N_{AL} = 3 \\ \max\left(\Delta_{2a.max}\right) & \text{otherwise} \end{cases}$$

$\Delta_{max} = 0.42 \cdot in$

### Span of Deflection

$$H_{def} := \begin{cases} \left(0.25 \cdot H_p\right) & \text{if } N_{AL} = 3 \\ \left(0.5 \cdot H_p\right) & \text{otherwise} \end{cases}$$

$H_{def} = 7.5\, ft$

### Span to Deflection Ratio

$$\Delta_{ratio} := \frac{H_{def}}{\Delta_{max}} = 215$$

Assume max deflection criteria is L/120.

$\text{if}\left(\Delta_{ratio} \ge 120, \text{"OK"}, \text{"NO GOOD"}\right) = \text{"OK"}$

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## Lateral Capacity of Embedded Portion of Soldier Beam

Geotechnical Engineering Circular No. 4

5.5.2 Evaluation of Ultimate Passive Resistance

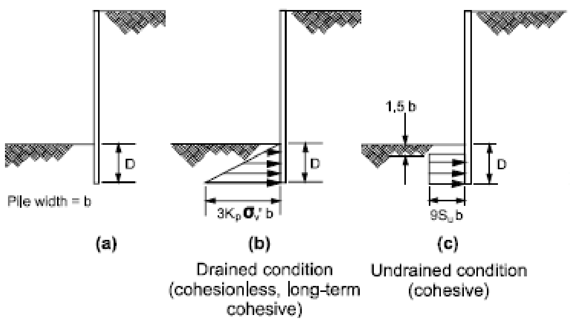
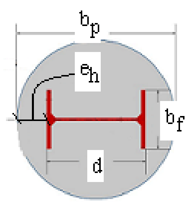



Figure 41. Broms method for evaluating ultimate passive resistance.

The embedment must be sufficient to develop passive resistance to carry the lateral load of the reaction force by the subgrade, R. A factor of safety of 1.5 is required.




$b_f = 14.7 \cdot in$

$d = 14.7 \cdot in$

Embedment hole centerline clearance:

$e_h := 5in$

Embedment hole width/diameter:

$b_p := d + 2 \cdot e_h = 24.7 \cdot in$

Total lateral load to be resisted by passive force of embedment.

Total lateral load: $R_{Load} := R_{max} = 104 \cdot kip$

The ultimate passive resistance is assumed to be the minimum ultimate passive resistance calculated from the Wang-Reese equations (equations B-2, B-4, B-5, and B-6 in Appendix B of the Geotechnical Engineering Circular No. 4). The factor of safety is calculated as the ratio of the ultimate passive resistance force, $F_p$ to $R_{Load}$.

| | |
|---|---|
| Unit weight of soil, γ: | $\gamma_s = 130 \cdot pcf$ |
| Height of soldier beam above penetration level: | $H_p = 30\,ft$ |
| Drilled shaft diameter: | $b_p = 24.7 \cdot in$ |
| Soldier center to center spacing: | $S_p = 8\,ft$ |
| Clear spacing between drilled shafts: | $s_c := S_p - b_p = 5.94\,ft$ |
| Soil friction angle: | $\phi_{f.2} = 39 \cdot deg$ |
| $\beta_f := 45deg + \left(\dfrac{\phi_{f.2}}{2}\right)$ | $\beta_f = 64.5 \cdot deg$ |
| α=$\phi_f$ for dense sands, $\phi_f$/3 to $\phi_f$/2 for loose sands. Assume: | $\alpha_f := \phi_{f.2} = 39 \cdot deg$ |

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*Wedge Resistance (single pile - Equation B-2)*

$$P_{pu.B2} := \gamma_s \cdot D_p \cdot \left[ \frac{K_{0.2} \cdot D_p \cdot \tan(\phi_{f.2}) \cdot \sin(\beta_f)}{\tan(\beta_f - \phi_{f.2}) \cdot \cos(\alpha_f)} + \frac{\tan(\beta_f)}{\tan(\beta_f - \phi_{f.2})} \cdot (b_p + D_p \cdot \tan(\beta_f) \cdot \tan(\alpha_f)) \ldots \\ + K_{0.2} \cdot D_p \cdot \tan(\beta_f) \cdot (\tan(\phi_{f.2}) \cdot \sin(\beta_f) - \tan(\alpha_f)) \right]$$

$$\boxed{P_{pu.B2} = 3728.81 \cdot \frac{kN}{m}}$$

$$\boxed{P_{pu.B2} = 255.5 \cdot \frac{kip}{ft}}$$

*Depth of Intersecting Wedges (Equation B-3)*

$$D_p = 4.57 \cdot m$$

$$d_i := D_p - \frac{s_c}{2 \cdot \tan(\alpha_f) \cdot \tan(\beta_f)} = 13.25\ ft$$

$$d_i = 4.04 \cdot m$$

*(Equation B-4)*

$$P_{pu.B4} := \gamma_s \cdot d_i \cdot \left[ \frac{K_{0.2} \cdot d_i \cdot \tan(\phi_{f.2}) \cdot \sin(\beta_f)}{\tan(\beta_f - \phi_{f.2})} \cdot \left( \frac{1}{\cos(\alpha_f)} - 1 \right) + \frac{d_i \cdot \tan(\beta_f) \cdot \tan(\alpha_f)}{\tan(\beta_f - \phi_{f.2})} - K_{0.2} \cdot d_i \cdot \frac{\sin(\beta_f)^2}{\cos(\beta_f)} \cdot \tan(\phi_{f.2}) \cdot (t \right.$$

$$P_{pu.B4} = 897.43 \cdot \frac{kN}{m} \qquad \text{where:} \quad d \le d_i$$

$$P_{pu.B4} = 61.49 \cdot \frac{kip}{ft}$$

(Equation B-2, d = d.i)

$$P_{pu.B2.di} := \gamma_s \cdot d_i \cdot \left[ \frac{K_{o.2} \cdot d_i \cdot \tan(\phi_{f.2}) \cdot \sin(\beta_f)}{\tan(\beta_f - \phi_{f.2}) \cdot \cos(\alpha_f)} + \frac{\tan(\beta_f)}{\tan(\beta_f - \phi_{f.2})} \cdot (b_p + d_i \cdot \tan(\beta_f) \cdot \tan(\alpha_f)) \ldots \right. \\ \left. + K_{o.2} \cdot d_i \cdot \tan(\beta_f) \cdot (\tan(\phi_{f.2}) \cdot \sin(\beta_f) - \tan(\alpha_f)) \right]$$

$$P_{pu.B2.di} = 2936.1 \cdot \frac{kN}{m}$$

$$P_{pu.B2.di} = 201.19 \cdot \frac{kip}{ft}$$

(Equation B-2, d = d.d.i & α = 0)

$$P_{pu.B2.ddp} := \gamma_s \cdot d_i \cdot \left[ \frac{K_{o.2} \cdot d_i \cdot \tan(\phi_{f.2}) \cdot \sin(\beta_f)}{\tan(\beta_f - \phi_{f.2}) \cdot \cos(0deg)} + \frac{\tan(\beta_f)}{\tan(\beta_f - \phi_{f.2})} \cdot (b_p + d_i \cdot \tan(\beta_f) \cdot \tan(0deg)) \ldots \right. \\ \left. + K_{o.2} \cdot d_i \cdot \tan(\beta_f) \cdot (\tan(\phi_{f.2}) \cdot \sin(\beta_f) - \tan(0deg)) \right]$$

$$P_{pu.B2.ddp} = 605.83 \cdot \frac{kN}{m}$$

$$P_{pu.B2.ddp} = 41.51 \cdot \frac{kip}{ft}$$

Wedge Resistance (Intersecting Wedges)

$$P_{pu.01} := P_{pu.B2} - P_{pu.B2.di} + P_{pu.B2.ddp} = 1398.54 \cdot \frac{kN}{m}$$

$$P_{pu.01} = 1398.54 \cdot \frac{kN}{m}$$

$$P_{pu.02} := P_{pu.B2} = 3728.81 \cdot \frac{kN}{m}$$

$$P_{pu.iw} := \begin{cases} P_{pu.02} & \text{if } d_i \le 0m \\ P_{pu.01} & \text{otherwise} \end{cases}$$

$$\boxed{P_{pu.iw} = 1399 \cdot \frac{kN}{m}} \quad \boxed{P_{pu.iw} = 95.83 \cdot \frac{kip}{ft}}$$

*Flow Resistance (Equation B-5)*

$$P_{pu.B5} := K_{a.2} \cdot b_p \cdot \gamma_s \cdot D_p \cdot \tan(\beta_f)^8 + K_{o.2} \cdot b_p \cdot \gamma_s \cdot D_p \cdot \tan(\phi_{f.2}) \cdot \tan(\beta_f)^4$$

$$\boxed{P_{pu.B5} = 5314 \cdot \frac{kN}{m}}$$

$$\boxed{P_{pu.B5} = 364.14 \cdot \frac{kip}{ft}}$$

*Rankine Continuous (Equation B-6)*

$$P_{pu.B6} := K_{p.2} \cdot \gamma_s \cdot D_p \cdot (s_c + b_p)$$

$$\boxed{P_{pu.B6} = 1001 \cdot \frac{kN}{m}}$$

$$\boxed{P_{pu.B6} = 68.57 \cdot \frac{kip}{ft}}$$

*Minimum Wang-Reese Passive Resistance*

$$P_{pu.min} := \min(P_{pu.iw}, P_{pu.B5}, P_{pu.B6})$$

$$\boxed{P_{pu.min} = 1001 \cdot \frac{kN}{m}}$$

$$\boxed{P_{pu.min} = 68.57 \cdot \frac{kip}{ft}}$$

*Passive Force (Equation B-1)*

$$F_{p.B1} := \gamma_s \cdot D_p^2 \cdot \left[ \frac{K_{o.2} \cdot D_p \cdot \tan(\phi_{f.2}) \cdot \sin(\beta_f)}{3 \tan(\beta_f - \phi_{f.2}) \cdot \cos(\alpha_f)} + \frac{\tan(\beta_f)}{\tan(\beta_f - \phi_{f.2})} \cdot \left( \frac{b_p}{2} + \frac{D_p}{3} \cdot \tan(\beta_f) \cdot \tan(\alpha_f) \right) \ldots \\ + \frac{K_{o.2} \cdot D_p \cdot \tan(\beta_f)}{3} \cdot (\tan(\phi_{f.2}) \cdot \sin(\beta_f) - \tan(\alpha_f)) \right]$$

$$F_{p.B1} = 5878.9 \cdot kN$$

$$F_{p.B1} = 1321.63 \cdot kip$$

*(Equation B-1, d = d.i, for reduction)*

$$F_{p.B1.red} := \gamma_s \cdot d_i^2 \cdot \left[ \frac{K_{o.2} \cdot d_i \cdot \tan(\phi_{f.2}) \cdot \sin(\beta_f)}{3 \tan(\beta_f - \phi_{f.2}) \cdot \cos(\alpha_f)} + \frac{\tan(\beta_f)}{\tan(\beta_f - \phi_{f.2})} \cdot \left( \frac{b_p}{2} + \frac{d_i}{3} \cdot \tan(\beta_f) \cdot \tan(\alpha_f) \right) \ldots \right. \\ \left. + \frac{K_{o.2} \cdot d_i \cdot \tan(\beta_f)}{3} \cdot \left( \tan(\phi_{f.2}) \cdot \sin(\beta_f) - \tan(\alpha_f) \right) \right]$$

$$F_{p.B1.red} = 923 \cdot kip$$

*(Equation B-1, d = d.i, and α = 0)*

$$F_{p.B1.add} := \gamma_s \cdot d_i^2 \cdot \left[ \frac{K_{o.2} \cdot d_i \cdot \tan(\phi_{f.2}) \cdot \sin(\beta_f)}{3 \tan(\beta_f - \phi_{f.2}) \cdot \cos(0deg)} + \frac{\tan(\beta_f)}{\tan(\beta_f - \phi_{f.2})} \cdot \left( \frac{b_p}{2} + \frac{d_i}{3} \cdot \tan(\beta_f) \cdot \tan(0deg) \right) \ldots \right. \\ \left. + \frac{K_{o.2} \cdot d_i \cdot \tan(\beta_f)}{3} \cdot \left( \tan(\phi_{f.2}) \cdot \sin(\beta_f) - \tan(0deg) \right) \right]$$

$$F_{p.B1.add} = 217.76 \cdot kip$$

*Determine Total Passive Force*    Link to depth input

$$F_{p.1} := F_{p.B1} - F_{p.B1.red} + F_{p.B1.add} = 2741.86 \cdot kN$$

$$F_{p.2} := F_{p.B1} = 1321.63 \cdot kip$$

$$F_{p.o} := \begin{cases} F_{p.2} & \text{if } d_i \le 0m \\ F_{p.1} & \text{otherwise} \end{cases}$$

$$\boxed{F_{p.o} = 616.39 \cdot kip}$$

W-R wedge resistance, accounting for intersecting wedges per Eq. B-2: $P_{pu.iw} = 95.83 \cdot \frac{kip}{ft}$

Minimum Wang-Reese (W-R) passive resistance: $P_{pu.min} = 68.57 \cdot \frac{kip}{ft}$
(Min of Eq. B-2 w/ intersecting wedges, B-5, and B-6)

Passive Force, accounting for intersecting wedges per Eq. B-1: $F_{p.o} = 616.39 \cdot kip$

A modification factor using the ratio of the minimum W-R passive resistance to the W-R wedge resistance will be applied to Eq. B-1 to obtain final total passive force results.

Modification factor:

$$x_F := \frac{P_{pu.min}}{P_{pu.iw}} = 0.72$$

Final Total Passive Force:

$$F_{p.total} := x_F \cdot F_{p.o}$$

$$\boxed{F_{p.total} = 441.05 \cdot kip}$$

Total driving/active force: $R_{Load} = 104 \cdot kip$

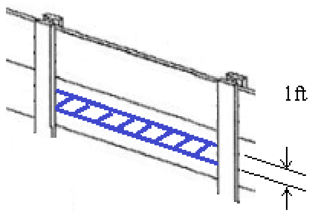
Factor of Safety for Embedment Depth:

$$FS_D := \frac{F_{p.total}}{R_{Load}} \qquad \boxed{FS_D = 4.24}$$

$$\boxed{\text{if}\left(FS_D \geq 1.5, \text{"OK"}, \text{"NO GOOD"}\right) = \text{"OK"}}$$

### *Thickness of Lagging (Assuming Concrete Lagging)*

Analyzing a 1ft width of lagging.




Simply supported span of lagging: $L_1 := S_p = 8\,ft$

Lateral pressure:

Surcharge: $p_s = 0.03 \cdot ksf$

At water level: $p_{eh.wl} = 1.3 \cdot ksf$

At embeddment / bottom lagging: $p_{eh.bot} = 1.44 \cdot ksf$

$$p_{a.lag} := p_s + p_{eh.bot} = 1.47 \cdot ksf$$

Line load to lagging strip:

$$w_l := p_{a.lag} \cdot 1\,ft = 1.47 \cdot klf$$

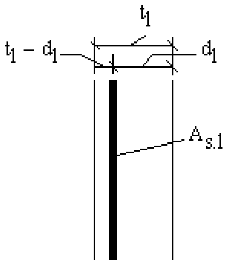
Bending moment to design strip:

$$M_l := \frac{w_l \cdot L_1^{\,2}}{8} = 11.75 \cdot kip \cdot ft$$

Consider Strength Design. Use Load Factor of 1.5 (conservative).

$$M_{u.l} := 1.5 \cdot M_l = 17.62 \cdot kip \cdot ft$$

## Lagging Properties




Assumed Section

Compressive strength assuming precast concrete lagging. $f_c := 5000psi$

Steel yield strength: $f_y := 60ksi$

Strength reduction factor: $\phi_{b.1} := 0.9$

Area of tension steel: #6 @ 4" spacing $A_{s.1} := 1.32in^2$

$$\phi M_n = \phi \cdot A_s \cdot f_y \cdot \left(d - \frac{a}{2}\right) \geq M_{u.1}$$

$$\left(d - \frac{a}{2}\right) \geq \frac{M_{u.1}}{\phi \cdot A_s \cdot f_y} \qquad d \geq \frac{M_{u.1}}{\phi \cdot A_s \cdot f_y} + \frac{a}{2}$$

$$0.85 f_c \cdot b \cdot a = A_s \cdot f_y$$

$$a_1 := \frac{A_{s.1} \cdot f_y}{0.85 \cdot f_c \cdot 12in} = 1.55 \cdot in$$

$$d_1 := \frac{M_{u.1}}{\phi_{b.1} \cdot A_{s.1} \cdot f_y} + \frac{a_1}{2} = 3.74 \cdot in$$

Assume that total thickness equals d + 4.25 inches.

$t_1 := d_1 + 4.25in = 7.99 \cdot in$

**Thickness and Reinforcement of Concrete Lagging:**

Thickness: $t_1 = 7.99 \cdot in$

Steel Reinf: $A_{s.1} = 1.32 \cdot in^2$

## Anchor Design

Number of anchor levels: $N_{AL} = 3$

Anchor inclination w/ the horiz: $\Phi_{tb} := 15deg$

Max Design Reaction/Horizontal Force:

$T_{anc.max} = 98 kip$

**Axial Design Load 1st Anchor:**

$$T_{1.max} := \begin{cases} \max(T_{3a.1.c}, T_{3a.1}) & \text{if } N_{AL} = 3 \\ \max(T_{2a.1.c}, T_{2a.1}) & \text{otherwise} \end{cases}$$

$$T_{1.max} = 29.5 \cdot kip$$

$$DL_1 := \frac{T_{1.max}}{\cos(\Phi_{tb})}$$

$$DL_1 = 30.54 \cdot kip$$

**Axial Design Load 2nd Anchor:**

$$T_{2.max} := \begin{cases} \max(T_{3a.2.c}, T_{3a.2}) & \text{if } N_{AL} = 3 \\ \max(T_{2a.2.c}, T_{2a.2}) & \text{otherwise} \end{cases}$$

$$T_{2.max} = 52 \cdot kip$$

$$DL_2 := \frac{T_{2.max}}{\cos(\Phi_{tb})}$$

$$DL_2 = 53.83 \cdot kip$$

**Axial Design Load 3rd Anchor:**

$$T_{3.max} := \begin{cases} \max(T_{3a.3.c}, T_{3a.3}) & \text{if } N_{AL} = 3 \\ 0kip & \text{otherwise} \end{cases}$$

$$T_{3.max} = 98 \cdot kip$$

$$DL_3 := \frac{T_{3.max}}{\cos(\Phi_{tb})}$$

$$DL_3 = 101.46 \cdot kip$$

Max Axial Tension Force

$$DL_{tb.o} := \max(DL_1, DL_2, DL_3)$$

$$\boxed{DL_{tb.o} = 101.46 \cdot kip}$$

## Bond Zone Design

Use Geotech Recommended Adhesion value for bond length design.***

Adhesion (as recommended): $A_{adh.o} := 1200psf$ ***********

Note: Using adhesion of only 1200 psf results in only 37 kip resistance for a 40-ft bond zone.

Adhesion (increased): $A_{adh} := 2400psf$

Assume anchor hole diameter: $D_a := 6in$

Anchor hole perimeter: $P_a := \pi \cdot D_a = 1.57\ ft$

Load transfer rate of anchor bond zone: $A_{tl} := A_{adh} \cdot P_a = 3.77\ klf$

Max anchor tension force to resist: $DL_{tb.o} = 101.46\ kip$

From Reference 2, "The design load with a factor of safety of 2.0 should be able to be achieved with a typical soil anchor bond length of 12 m (39 ft), assuming a small diameter low pressure grouted anchor."

Factor of safety, bond length: $SF_{bl} := 2.0$

Allowable bond strength for ***12 m (39-ft)***:

$$T_{allow.12m} := \frac{A_{tl}}{SF_{bl}} \cdot 12m = 74.21 \cdot kip$$

$\text{if}\left(T_{allow.12m} > DL_{tb.o}, \text{"OK"}, \text{"NO GOOD"}\right) = \text{"NO GOOD"}$

$\boxed{T_{allow.12m} = 74.21 \cdot kip}$

Bond length required / min:

$$BL_{req.o} := \frac{DL_{tb.o} \cdot SF_{bl}}{A_{tl}} = 53.82 \cdot ft$$

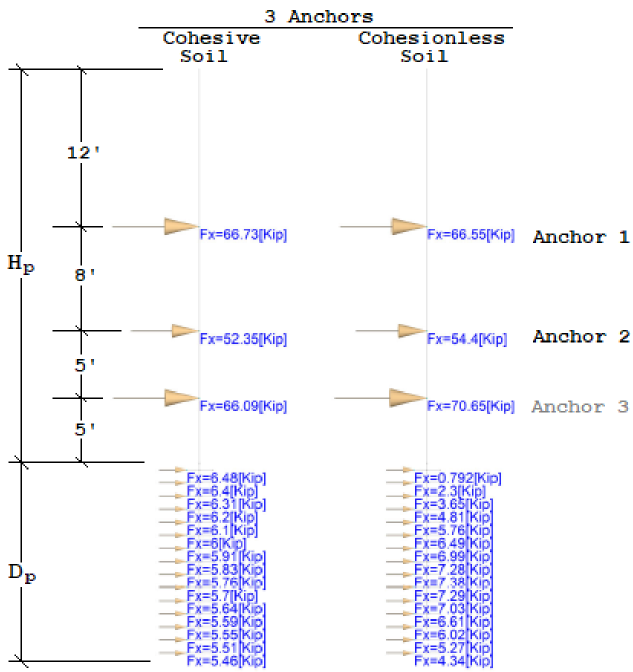
$$\boxed{T_{allow.BL.o} := \frac{A_{tl} \cdot BL_{req.o}}{SF_{bl}} = 101.46 \cdot kip}$$

$\boxed{BL_{req.o} = 53.82\ ft}$

**This bond zone is in excess of what is considered typical (approx. 40 ft).**

**Adjust elevation of anchors to get a more even distribution of anchor forces.**

Adjusted Anchor Configuration and Reactions




$T_{adjusted} := 71\,kip$

$$DL_{tb} := \frac{T_{adjusted}}{\cos(\Phi_{tb})}$$

$DL_{tb} = 73.5\,kip$

Bond Zone with Adjusted Anchors Forces

$\text{if}\left(T_{allow.12m} > DL_{tb}, \text{"OK"}, \text{"NO GOOD"}\right) = \text{"OK"}$

Bond length required / min:

$$BL_{req} := \frac{DL_{tb} \cdot SF_{bl}}{A_{tl}} = 39 \cdot ft$$

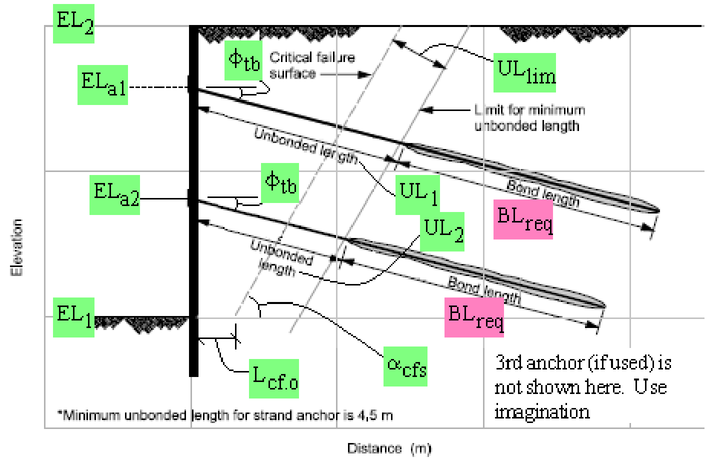
$\boxed{BL_{req} = 39\,ft}$

Allowable Force for this bond length:

$$\boxed{T_{allow.BL} := \frac{A_{tl} \cdot BL_{req}}{SF_{bl}} = 73.5 \cdot kip}$$

## Design of the Unbonded Length

Use the below figure for the input and output.




Location of unbonded and bond lengths for ground anchors.

Lower Ground Elevation: $EL_1 := 80ft$

Top of Wall Elevation: $EL_2 := 110ft$

Elevation of top anchor: $EL_{a1} := 98ft$

Elevation of 2nd anchor: $EL_{a2} := 90ft$

Elevation of 3nd anchor: $EL_{a3} := 85ft$

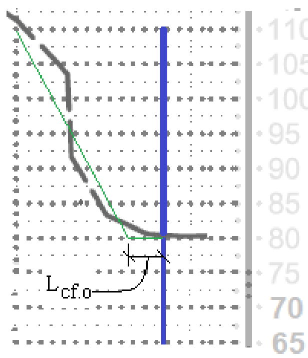
Dimension to origin of critical failure surface: $L_{cf.o} := 5ft$

Critical failure surface to unbonded length limit dimension: $UL_{lim} := 7ft$

Anchor inclination w/ the horiz: $\Phi_{tb} = 15\,deg$

Critical failure surface inclination w/ the horiz: $\alpha_{cfs} := 61.5deg$

$$45deg + \left(\frac{\Phi_f}{2}\right) = 60 \cdot deg$$




Use Lcf.o to approximate actual condition; to better estimate required length of anchors.

Design that includes bar anchors: min unbonded length selected as max of 3 m (10 ft) or the distance from the wall to a location 2 m (7 ft) beyond the critical failure surface.

Design that includes strand anchors: min unbonded length selected as max of 4.5 m (15 ft) or the distance from the wall to a location 2 m (7 ft) beyond the critical failure surface.

Find unbonded length for the upper anchor.

$$A_{u1} := L_{cf.o} + \left(EL_{a1} - EL_1\right)\cdot \tan\left(90deg - \alpha_{cfs}\right) = 14.77\cdot ft$$

$$B_{u1} := A_{u1}\cdot \sin\left(\phi_{tb}\right) = 3.82\cdot ft$$

$$a_{u1} := B_{u1}\cdot \sin\left(90deg - \alpha_{cfs}\right) = 1.82\cdot ft$$

$$b_{u1} := A_{u1} - a_{u1} = 12.95\cdot ft$$

$$UL_{1.p1} := \frac{b_{u1}}{\cos\left(\phi_{tb}\right)} = 13.41\cdot ft$$

$$UL_1 := UL_{1.p1} + UL_{lim}$$

$$\boxed{UL_1 = 20.41\ ft}$$

Find unbonded length for the 2nd lower anchor.

$$A_{u2} := L_{cf.o} + \left(EL_{a2} - EL_1\right)\cdot \tan\left(90deg - \alpha_{cfs}\right) = 10.43\cdot ft$$

$$B_{u2} := A_{u2}\cdot \sin\left(\phi_{tb}\right) = 2.7\cdot ft$$

$$a_{u2} := B_{u2}\cdot \sin\left(90deg - \alpha_{cfs}\right) = 1.29\cdot ft$$

$$b_{u2} := A_{u2} - a_{u2} = 9.14\cdot ft$$

$$UL_{2.p1} := \frac{b_{u2}}{\cos\left(\phi_{tb}\right)} = 9.46\cdot ft$$

$$UL_2 := UL_{2.p1} + UL_{lim}$$

$$\boxed{UL_2 = 16.46\ ft}$$

Find unbonded length for the 3rd anchor

$A_{u3} := L_{cf.o} + \left(EL_{a3} - EL_1\right)\cdot \tan\left(90deg - \alpha_{cfs}\right) = 7.71\cdot ft$

$B_{u3} := A_{u3}\cdot \sin\left(\phi_{tb}\right) = 2\cdot ft$

$a_{u3} := B_{u3}\cdot \sin\left(90deg - \alpha_{cfs}\right) = 0.95\cdot ft$

$b_{u3} := A_{u3} - a_{u3} = 6.76\cdot ft$

$$UL_{3.p1} := \frac{b_{u3}}{\cos\left(\phi_{tb}\right)} = 7\cdot ft$$

$UL_3 := UL_{3.p1} + UL_{1im}$

$\boxed{UL_3 = 14\,ft}$

*Depth / location of anchor from top of wall.*

*1st / Upper anchor:*

$d_{u.a} := EL_2 - EL_{a1} = 12\,ft$

*2nd anchor:*

$d_{l.a} := EL_2 - EL_{a2} = 20\,ft$

*3rd anchor:*

$d_{b.a} := EL_2 - EL_{a3} = 25\,ft$

Required total length of upper anchor:
(unbonded length + bonded length)

$L_{tot.a1} := UL_1 + BL_{req}$

$\boxed{L_{tot.a1} = 59.4\,ft}$

Required total length of 2nd lower anchor:
(unbonded length + bonded length)

$L_{tot.a2} := UL_2 + BL_{req}$

$\boxed{L_{tot.a2} = 55.46\,ft}$

Required total length of 3rd anchor:
(unbonded length + bonded length)

$L_{tot.a3} := UL_3 + BL_{req}$

$\boxed{L_{tot.a3} = 53\,ft}$

***External Stability***

Recommend external stability analysis per Geotech.

## Selection of Tendon / Tie-Backs

Site soil classification for corrosion protection (aggresive, nonaggresive, etc) is currently under investigation.

Failure from corrosion would be significant. Corrosive conditions assumed.

*Class I (double protection) encapsulated tendon is selected. Dimensions are calculated for both strand and bar tendons assuming a maximum test load of 1.33 DL*

A prestressing bar may be selected from Table 9. Recommend designing for an allowable tensile capacity of 60 percent of the specified minimum tensile strength (SMTS).

Steel grade (select from Table 9): $f_{pu} := 150ksi$

Bar diameter (select from Table 9): $bar_{dia} := 1.25in$

Nominal cross section area (select from Table 9): $bar_{area} := 1.25in^2$

Ultimate strength: $F_{pu} := f_{pu} \cdot bar_{area} = 187.5 \cdot kip$

ASD design factor: $bar_{asd} := 0.6$

Prestressing force / allowable tensile capacity: $F_{pu.allow} := bar_{asd} \cdot F_{pu}$

$DL_{tb} = 73.5 \cdot kip$ $\boxed{F_{pu.allow} = 113 \cdot kip}$

$\text{if}\left(F_{pu.allow} > DL_{tb}, \text{"OK"}, \text{"NO GOOD"}\right) = \text{"OK"}$ $\boxed{F_{pu.allow} = 112.5 \cdot kip}$

Table 9. Properties of prestressing steel bars (ASTM A722).

| Steel grade | Nominal diameter | Ultimate stress $f_{pu}$ | Nominal cross section area $A_{ps}$ | Ultimate strength $f_{pu}$ $A_{ps}$ | Prestressing force | | |
|---|---|---|---|---|---|---|---|
| | | | | | 0.8 $f_{pu}$ $A_{ps}$ | 0.7 $f_{pu}$ $A_{ps}$ | 0.6 $f_{pu}$ $A_{ps}$ |
| (ksi) | (in.) | (ksi) | (in.$^2$) | (kips) | (kips) | (kips) | (kips) |
| 150 | 1 | 150 | 0.85 | 127.5 | 102.0 | 89.3 | 76.5 |
| | 1-1/4 | 150 | 1.25 | 187.5 | 150.0 | 131.3 | 112.5 |
| | 1-3/8 | 150 | 1.58 | 237.0 | 189.6 | 165.9 | 142.2 |
| | 1-3/4 | 150 | 2.66 | 400.0 | 320.0 | 280.0 | 240.0 |
| | 2-1/2 | 150 | 5.19 | 778.0 | 622.4 | 435.7 | 466.8 |
| 160 | 1 | 160 | 0.85 | 136.0 | 108.8 | 95.2 | 81.6 |
| | 1-1/4 | 160 | 1.25 | 200.0 | 160.0 | 140.0 | 120.0 |
| | 1-3/8 | 160 | 1.58 | 252.8 | 202.3 | 177.0 | 151.7 |
| (ksi) | (mm) | (N/mm$^2$) | (mm$^2$) | (kN) | (kN) | (kN) | (kN) |
| 150 | 26 | 1035 | 548 | 568 | 454 | 398 | 341 |
| | 32 | 1035 | 806 | 835 | 668 | 585 | 501 |
| | 36 | 1035 | 1019 | 1055 | 844 | 739 | 633 |
| | 45 | 1035 | 1716 | 1779 | 1423 | 1246 | 1068 |
| | 64 | 1035 | 3348 | 3461 | 2769 | 2423 | 2077 |
| 160 | 26 | 1104 | 548 | 605 | 484 | 424 | 363 |
| | 32 | 1104 | 806 | 890 | 712 | 623 | 534 |
| | 36 | 1104 | 1019 | 1125 | 900 | 788 | 675 |

Based on the corrosion protection classification, select the minimum suggested trumpet opening size from Table 11.

$trump_{open.min} := 95mm$

Table 11. Guidance relationship between tendon size and trumpet opening size.

| Tendon type | Minimum suggested trumpet opening size (mm) | |
|---|---|---|
| | Class II corrosion protection | Class I corrosion protection |
| Number of 15-mm diameter strands | | |
| 4 | 102 | 150 |
| 7 | 115 | 165 |
| 9 | 127 | 178 |
| 11 | 140 | 191 |
| 13 | 153 | 203 |
| 17 | 165 | 216 |
| Bar diameter (mm) | | |
| 26 | 64 | 89 |
| 32 | 70 | 95 |
| 36 | 76 | 102 |

-----------For strand anchors (Grade 270), select from Table 10.--------------------------------------

Design load to anchor: $DL_{tb} = 73.5 \cdot kip$

A # strand, Grade 270, strand anchor could be used. $F_{pu.allow.strand} := 105.5kip$

$strand := 3$

The minimum suggested trumpet opening size from Table 11: $trump_{open.min.strand} := 150mm$

Table 10. Properties of 15-mm diameter prestressing steel strands (ASTM A416, Grade 270 (metric 1860)).

| Number of 15-mm diameter strands | Cross section area | | Ultimate strength | | Prestressing force | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | $0.8\ f_{pu}A_{ps}$ | | $0.7\ f_{pu}A_{ps}$ | | $0.6\ f_{pu}A_{ps}$ | |
| | (in.$^2$) | (mm$^2$) | (kips) | (kN) | (kips) | (kN) | (kips) | (kN) | (kips) | (kN) |
| 1 | 0.217 | 140 | 58.6 | 260.7 | 46.9 | 209 | 41.0 | 182 | 35.2 | 156 |
| 3 | 0.651 | 420 | 175.8 | 782.1 | 140.6 | 626 | 123.1 | 547 | 105.5 | 469 |
| 4 | 0.868 | 560 | 234.4 | 1043 | 187.5 | 834 | 164.1 | 730 | 140.6 | 626 |
| 5 | 1.085 | 700 | 293.0 | 1304 | 234.4 | 1043 | 205.1 | 912 | 175.8 | 782 |
| 7 | 1.519 | 980 | 410.2 | 1825 | 328.2 | 1460 | 287.1 | 1277 | 246.1 | 1095 |
| 9 | 1.953 | 1260 | 527.4 | 2346 | 421.9 | 1877 | 369.2 | 1642 | 316.4 | 1408 |
| 12 | 2.604 | 1680 | 703.2 | 3128 | 562.6 | 2503 | 492.2 | 2190 | 421.9 | 1877 |
| 15 | 3.255 | 2100 | 879.0 | 3911 | 703.2 | 3128 | 615.3 | 2737 | 527.4 | 2346 |
| 19 | 4.123 | 2660 | 1113.4 | 4953 | 890.7 | 3963 | 779.4 | 3467 | 668.0 | 2972 |

*Select trumpet diameter for use:*

$trump_o := 150mm$

----------------------------------------------------------------------------------------------------------------

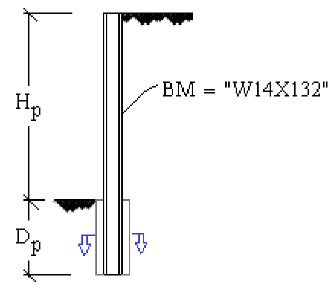
### *Summary of Design*

#### ***Soldier Beams***

**Or HP Shape - HP14x117**

Spacing: $S_p = 8\,ft$

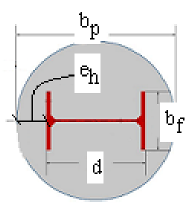
Retained height: $H_p = 30\,ft$




Embedment shaft diameter: $b_p = 2.06\,ft$

Embedment depth / penetration: $D_p = 15\,ft$

Embedment Factor of Safety (1.5 min allowed): $FS_D = 4.24$




$b_f = 14.7 \cdot in$

$d = 14.7 \cdot in$

$e_h = 5 \cdot in$

Section size / type: BM = "W14X132"

Any section type of equivalent section modulus could work as well.

| Design Analysis Information | Required Properties | Initial Design Results |
|---|---|---|
| Section Modulus: | $S_{req} = 65.89 \cdot in^3$ | $S_x = 209 \cdot in^3$ |

#### **Concrete Lagging**

Thickness: $t_l = 7.99 \cdot in$

Longitudinal reinforcement area: $A_{s.l} = 1.32 \cdot in^2$

## Anchors

Levels / rows: $N_{AL} = 3$

Size (if Grade 150 bar anchor): $bar_{dia} = 1.25 \cdot in$

Size / # of strands: (if 15-mm diameter Grade 270 strands) $strand = 3$

Note: Either bar anchor or strand anchor to be used.

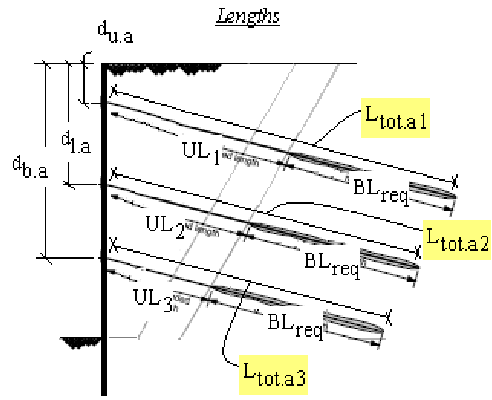
Depth / location from top of wall:

1st / Upper anchor: $d_{u.a} = 12\ ft$

2nd anchor: $d_{1.a} = 20\ ft$

3rd / Bot anchor: $d_{b.a} = 25\ ft$

Anchor inclination: (from horizontal - allrows) $\phi_{tb} = 15 \cdot deg$




UL = Unbonded Length    $L_{tot}$ = Total Length

$UL_1 = 20.41\ ft$    $L_{tot.a1} = 59.4\ ft$

$UL_2 = 16.46\ ft$    $L_{tot.a2} = 55.46\ ft$

$UL_3 = 14\ ft$    $L_{tot.a3} = 53\ ft$

Class I (double protection) encapsulated tendon.

Min bonded length: $BL_{req} = 39\ ft$

| Design Analysis Information | Required Properties | Initial Design Results / Provided |
|---|---|---|
| Allowable bond capacity: | $DL_{tb} = 73.5 \cdot kip$ | $T_{allow.BL} = 73.5 \cdot kip$ |
| Bar: $bar_{dia} = 1.25 \cdot in$ | | |
| Allowable capacity: | $DL_{tb} = 73.5 \cdot kip$ | $F_{pu.allow} = 112.5 \cdot kip$ |
| Trumpet diameter: | $trump_{open.min} = 3.74 \cdot in$ | $trump_o = 5.91 \cdot in$ |
| 15-mm Strand: $strand = 3$ | | |
| Allowable capacity: | $DL_{tb} = 73.5 \cdot kip$ | $F_{pu.allow.strand} = 105.5 \cdot kip$ |
| Trumpet diameter: | $trump_{open.min.strand} = 5.91 \cdot in$ | $trump_o = 5.91 \cdot in$ |

**Soil Spring Properties - Left Side / River Side - Using Table 1. Presumptive Properties for Silt and Clay (Cohesive) Soils**

| Node | Unified Soil Classification | Consistency | Face Width of Embedded Portion | Thickness of Soil Layer Represented by spring, t | Distance from Surface, z | Increase in Young's Modulus w/ depth, $A_z$ | Young's Modulus, $E_s$ | Horizontal Spring Stiffness, $K_H$ | Moist unit weight, | Undrained soil shear strength, $S_U$ | Coefficient of passive earth pressure, $K_p$ | Effective vertical soil stress at spring location | Ultimate lateral soil resistance at spring location, $p_u$ | Maximum force allowed in spring, $F_{ult}$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | inches | inches | inches | psi/in | psi | lb/in | pci | psi | | psi | psi | lbf |
| 1 | CH | Med. to Stiff | 24 | 12 | 6 | 0 | 934 | 22416 | 0.0667 | 7 | | | 23.6 | 6804 |
| 2 | CH | Med. to Stiff | 24 | 12 | 18 | 0 | 934 | 22416 | 0.0667 | 7 | | | 28.9 | 8316 |
| 3 | CH | Med. to Stiff | 24 | 12 | 30 | 0 | 934 | 22416 | 0.0667 | 7 | | | 34.1 | 9828 |
| 4 | CH | Med. to Stiff | 24 | 12 | 42 | 0 | 934 | 22416 | 0.0667 | 7 | | | 39.4 | 11340 |
| 5 | CH | Med. to Stiff | 24 | 12 | 54 | 0 | 934 | 22416 | 0.0667 | 7 | | | 44.6 | 12852 |
| 6 | CH | Med. to Stiff | 24 | 12 | 66 | 0 | 934 | 22416 | 0.0667 | 7 | | | 49.9 | 14364 |
| 7 | CH | Med. to Stiff | 24 | 12 | 78 | 0 | 934 | 22416 | 0.0667 | 7 | | | 55.1 | 15876 |
| 8 | CH | Med. to Stiff | 24 | 12 | 90 | 0 | 934 | 22416 | 0.0667 | 7 | | | 60.4 | 17388 |
| 9 | CH | Med. to Stiff | 24 | 12 | 102 | 0 | 934 | 22416 | 0.0667 | 7 | | | 63.0 | 18144 |
| 10 | CH | Med. to Stiff | 24 | 12 | 114 | 0 | 934 | 22416 | 0.0667 | 7 | | | 63.0 | 18144 |
| 11 | CH | Med. to Stiff | 24 | 12 | 126 | 0 | 934 | 22416 | 0.0667 | 7 | | | 63.0 | 18144 |
| 12 | CH | Med. to Stiff | 24 | 12 | 138 | 0 | 934 | 22416 | 0.0667 | 7 | | | 63.0 | 18144 |
| 13 | CH | Med. to Stiff | 24 | 12 | 150 | 0 | 934 | 22416 | 0.0667 | 7 | | | 63.0 | 18144 |
| 14 | CH | Med. to Stiff | 24 | 12 | 162 | 0 | 934 | 22416 | 0.0667 | 7 | | | 63.0 | 18144 |
| 15 | CH | Med. to Stiff | 24 | 12 | 174 | 0 | 934 | 22416 | 0.0667 | 7 | | | 63.0 | 18144 |

**Soil Spring Properties - Right Side / Land Side - Using Table 1. Presumptive Properties for Silt and Clay (Cohesive) Soils**

| Node | Unified Soil Classification | Consistency | Face Width of Embedded Portion | Thickness of Soil Layer Represented by spring, t | Distance from Surface, z | Increase in Young's Modulus w/ depth, $A_z$ | Young's Modulus, $E_s$ | Horizontal Spring Stiffness, $K_H$ | Moist unit weight, | Undrained soil shear strength, $S_U$ | Coefficient of passive earth pressure, $K_p$ | Effective vertical soil stress at spring location | Ultimate lateral soil resistance at spring location, $p_u$ | Maximum force allowed in spring, $F_{ult}$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | inches | inches | inches | psi/in | psi | lb/in | pci | psi | | psi | psi | lbf |
| 1 | CH | Med. to Stiff | 24 | 12 | 366 | 0 | 934 | 22416 | 0.0667 | 7 | | | 63.0 | 18144 |
| 2 | CH | Med. to Stiff | 24 | 12 | 378 | 0 | 934 | 22416 | 0.0667 | 7 | | | 63.0 | 18144 |
| 3 | CH | Med. to Stiff | 24 | 12 | 390 | 0 | 934 | 22416 | 0.0667 | 7 | | | 63.0 | 18144 |
| 4 | CH | Med. to Stiff | 24 | 12 | 402 | 0 | 934 | 22416 | 0.0667 | 7 | | | 63.0 | 18144 |
| 5 | CH | Med. to Stiff | 24 | 12 | 414 | 0 | 934 | 22416 | 0.0667 | 7 | | | 63.0 | 18144 |
| 6 | CH | Med. to Stiff | 24 | 12 | 426 | 0 | 934 | 22416 | 0.0667 | 7 | | | 63.0 | 18144 |
| 7 | CH | Med. to Stiff | 24 | 12 | 438 | 0 | 934 | 22416 | 0.0667 | 7 | | | 63.0 | 18144 |
| 8 | CH | Med. to Stiff | 24 | 12 | 450 | 0 | 934 | 22416 | 0.0667 | 7 | | | 63.0 | 18144 |
| 9 | CH | Med. to Stiff | 24 | 12 | 462 | 0 | 934 | 22416 | 0.0667 | 7 | | | 63.0 | 18144 |
| 10 | CH | Med. to Stiff | 24 | 12 | 474 | 0 | 934 | 22416 | 0.0667 | 7 | | | 63.0 | 18144 |
| 11 | CH | Med. to Stiff | 24 | 12 | 486 | 0 | 934 | 22416 | 0.0667 | 7 | | | 63.0 | 18144 |
| 12 | CH | Med. to Stiff | 24 | 12 | 498 | 0 | 934 | 22416 | 0.0667 | 7 | | | 63.0 | 18144 |
| 13 | CH | Med. to Stiff | 24 | 12 | 510 | 0 | 934 | 22416 | 0.0667 | 7 | | | 63.0 | 18144 |
| 14 | CH | Med. to Stiff | 24 | 12 | 522 | 0 | 934 | 22416 | 0.0667 | 7 | | | 63.0 | 18144 |
| 15 | CH | Med. to Stiff | 24 | 12 | 534 | 0 | 934 | 22416 | 0.0667 | 7 | | | 63.0 | 18144 |

| Soil Spring Properties - Left Side / River Side - Using Table 2. Presumptive Properties for Sand and Gravel (Cohesionless) Soils | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Node | Unified Soil Classification | Consistency | Face Width of Embedded Portion | Thickness of Soil Layer Represented by spring, t | Distance from Surface, z | Increase in Young's Modulus w/ depth, $A_z$ | Young's Modulus, $E_s$ | Horizontal Spring Stiffness, $K_H$ | Moist unit weight, | Undrained soil shear strength, $S_U$ | Coefficient of passive earth pressure, $K_p$ | Effective vertical soil stress at spring location | Ultimate lateral soil resistance at spring location, $p_u$ | Maximum force allowed in spring, $F_{ult}$ |
| | | | inches | inches | inches | psi/in | psi | lb/in | pci | psi | | psi | psi | lbf |
| 1 | SP-SM | Med. to Dense | 24 | 12 | 6 | 19 | 114 | 2700 | 0.0637 | | 3.69 | 0.17 | 1.8 | 528 |
| 2 | SP-SM | Med. to Dense | 24 | 12 | 18 | 19 | 342 | 8200 | 0.0637 | | 3.69 | 0.50 | 5.5 | 1584 |
| 3 | SP-SM | Med. to Dense | 24 | 12 | 30 | 19 | 570 | 13700 | 0.0637 | | 3.69 | 0.83 | 9.2 | 2640 |
| 4 | SP-SM | Med. to Dense | 24 | 12 | 42 | 19 | 798 | 19200 | 0.0637 | | 3.69 | 1.16 | 12.8 | 3696 |
| 5 | SP-SM | Med. to Dense | 24 | 12 | 54 | 19 | 1026 | 24600 | 0.0637 | | 3.69 | 1.49 | 16.5 | 4752 |
| 6 | SP-SM | Med. to Dense | 24 | 12 | 66 | 19 | 1254 | 30100 | 0.0637 | | 3.69 | 1.82 | 20.2 | 5808 |
| 7 | SP-SM | Med. to Dense | 24 | 12 | 78 | 19 | 1482 | 35600 | 0.0637 | | 3.69 | 2.15 | 23.8 | 6864 |
| 8 | SP-SM | Med. to Dense | 24 | 12 | 90 | 19 | 1710 | 41000 | 0.0637 | | 3.69 | 2.48 | 27.5 | 7920 |
| 9 | SP-SM | Med. to Dense | 24 | 12 | 102 | 19 | 1938 | 46500 | 0.0637 | | 3.69 | 2.82 | 31.2 | 8976 |
| 10 | SP-SM | Med. to Dense | 24 | 12 | 114 | 19 | 2166 | 52000 | 0.0637 | | 3.69 | 3.15 | 34.8 | 10032 |
| 11 | SP-SM | Med. to Dense | 24 | 12 | 126 | 19 | 2394 | 57500 | 0.0637 | | 3.69 | 3.48 | 38.5 | 11088 |
| 12 | SP-SM | Med. to Dense | 24 | 12 | 138 | 19 | 2622 | 62900 | 0.0637 | | 3.69 | 3.81 | 42.2 | 12144 |
| 13 | SP-SM | Med. to Dense | 24 | 12 | 150 | 19 | 2850 | 68400 | 0.0637 | | 3.69 | 4.14 | 45.8 | 13200 |
| 14 | SP-SM | Med. to Dense | 24 | 12 | 162 | 19 | 3078 | 73900 | 0.0637 | | 3.69 | 4.47 | 49.5 | 14256 |
| 15 | SP-SM | Med. to Dense | 24 | 12 | 174 | 19 | 3306 | 79300 | 0.0637 | | 3.69 | 4.80 | 53.2 | 15312 |

| Soil Spring Properties - Right Side / Land Side - Using Table 2. Presumptive Properties for Sand and Gravel (Cohesionless) Soils | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Node | Unified Soil Classification | Consistency | Face Width of Embedded Portion | Thickness of Soil Layer Represented by spring, t | Distance from Surface, z | Increase in Young's Modulus w/ depth, $A_z$ | Young's Modulus, $E_s$ | Horizontal Spring Stiffness, $K_H$ | Moist unit weight, | Undrained soil shear strength, $S_U$ | Coefficient of passive earth pressure, $K_p$ | Effective vertical soil stress at spring location | Ultimate lateral soil resistance at spring location, $p_u$ | Maximum force allowed in spring, $F_{ult}$ |
| | | | inches | inches | inches | psi/in | psi | lb/in | pci | psi | | psi | psi | lbf |
| 1 | SP-SM | Med. to Dense | 24 | 12 | 366 | 19 | 6954 | 166900 | 0.0637 | | 3.69 | 10.10 | 231.7 | 66736 |
| 2 | SP-SM | Med. to Dense | 24 | 12 | 378 | 19 | 7182 | 172400 | 0.0637 | | 3.69 | 10.43 | 235.4 | 67792 |
| 3 | SP-SM | Med. to Dense | 24 | 12 | 390 | 19 | 7410 | 177800 | 0.0637 | | 3.69 | 10.76 | 239.1 | 68848 |
| 4 | SP-SM | Med. to Dense | 24 | 12 | 402 | 19 | 7638 | 183300 | 0.0637 | | 3.69 | 11.10 | 242.7 | 69904 |
| 5 | SP-SM | Med. to Dense | 24 | 12 | 414 | 19 | 7866 | 188800 | 0.0637 | | 3.69 | 11.43 | 246.4 | 70960 |
| 6 | SP-SM | Med. to Dense | 24 | 12 | 426 | 19 | 8094 | 194300 | 0.0637 | | 3.69 | 11.76 | 250.1 | 72016 |
| 7 | SP-SM | Med. to Dense | 24 | 12 | 438 | 19 | 8322 | 199700 | 0.0637 | | 3.69 | 12.09 | 253.7 | 73072 |
| 8 | SP-SM | Med. to Dense | 24 | 12 | 450 | 19 | 8550 | 205200 | 0.0637 | | 3.69 | 12.42 | 257.4 | 74128 |
| 9 | SP-SM | Med. to Dense | 24 | 12 | 462 | 19 | 8778 | 210700 | 0.0637 | | 3.69 | 12.75 | 261.1 | 75184 |
| 10 | SP-SM | Med. to Dense | 24 | 12 | 474 | 19 | 9006 | 216100 | 0.0637 | | 3.69 | 13.08 | 264.7 | 76240 |
| 11 | SP-SM | Med. to Dense | 24 | 12 | 486 | 19 | 9234 | 221600 | 0.0637 | | 3.69 | 13.41 | 268.4 | 77296 |
| 12 | SP-SM | Med. to Dense | 24 | 12 | 498 | 19 | 9462 | 227100 | 0.0637 | | 3.69 | 13.74 | 272.1 | 78352 |
| 13 | SP-SM | Med. to Dense | 24 | 12 | 510 | 19 | 9690 | 232600 | 0.0637 | | 3.69 | 14.08 | 275.7 | 79408 |
| 14 | SP-SM | Med. to Dense | 24 | 12 | 522 | 19 | 9918 | 238000 | 0.0637 | | 3.69 | 14.41 | 279.4 | 80464 |
| 15 | SP-SM | Med. to Dense | 24 | 12 | 534 | 19 | 10146 | 243500 | 0.0637 | | 3.69 | 14.74 | 283.1 | 81520 |

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# Exhibit A-1

# Geotechnical Boring Logs and Lab Data