

APPENDIX L: Engineering Appendix

**Volume III
APPENDIX L:
Engineering**

**Convey Atchafalaya River to Northern Terrebonne Marshes
and Multipurpose Operation of Houma Navigation Lock**

**Final Feasibility Report
Appendix L – Engineering Investigations and Cost Estimates
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L1 GENERAL

135 The LCA-ARTM Study Area (Figure L1) comprises approximately 1000 square miles (~660,000
acres) in Southern Louisiana in the vicinity of the City of Houma and Terrebonne Parish. The
LCA-ARTM study area fits into the Louisiana Coastal Area Ecosystem Restoration Study (LCA
Study) Area, which has been identified as the Louisiana coastal area from Mississippi to Texas.
140 The proposed LCA-ARTM project is located in the Deltaic Plain within Subprovince 3, one of
the four Subprovinces identified in the LCA Study Area.

The overall study area is bound to the west by the Lower Atchafalaya River. The study area is
bound to the east by the Bayou Lafourche ridge. The study area is bound to the north by the
145 Bayou Black ridge, from the Lower Atchafalaya River to the City of Houma, and by the Gulf
Intracoastal Waterway (GIWW), from the City of Houma to the Bayou Lafourche ridge. The
southern boundary of the project was based on a delineation conducted in 2007 of coastal
Louisiana vegetation types. The boundary identifies the transition between saline and brackish
marsh types identified by Sasser et al. (2008).

150 Eight alternatives, including the No Action alternative were formulated to address the goals and
objectives of the study. These alternatives included 62 features dispersed throughout the project
area. These features include various water control structures, dredged channels, culverts, weirs,
plugs, terracing, marsh berms, spoil gaps, bank line protection, and removal of existing
structures.

155

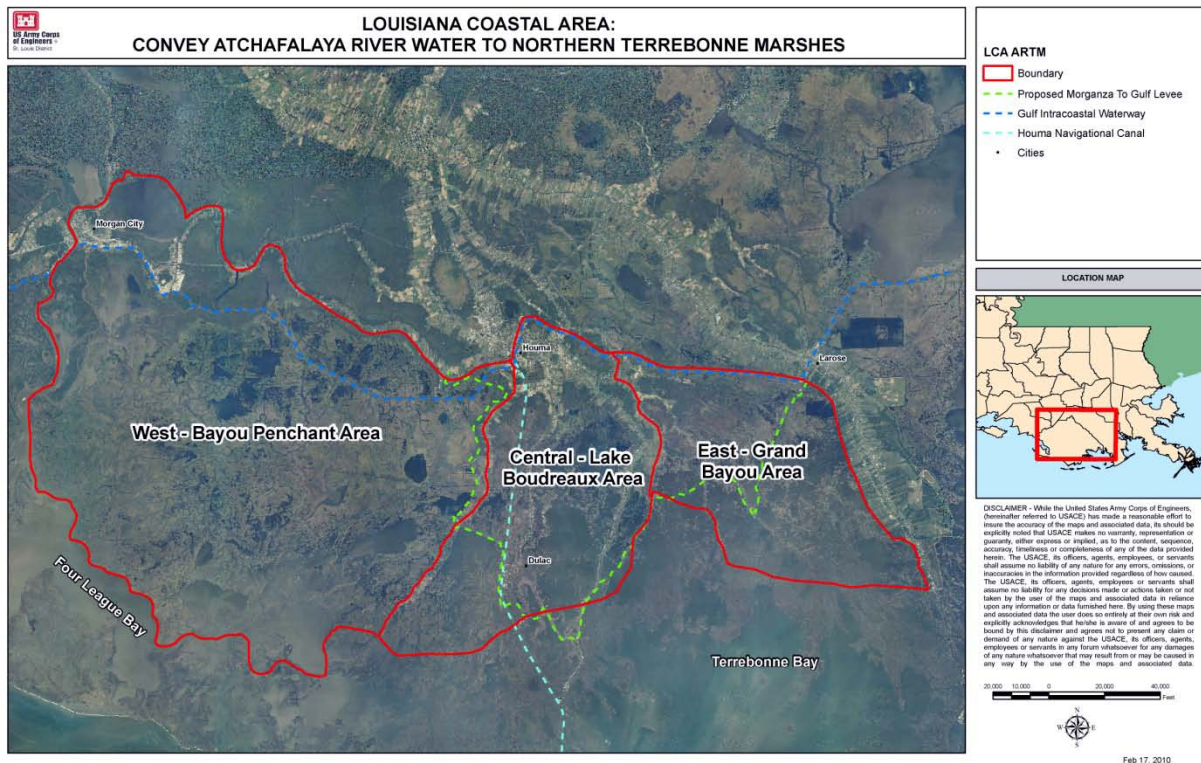


Figure L1 – Map of Study Area.

L2 HYDRAULICS AND HYDROLOGY

160

L2.1 Climatology

L2.1.1 Climate

165

The climate of the area is humid subtropical and is subject to significant polar influences during the winter as cold air masses periodically move southward over the area displacing warm moist air. Prevailing southerly winds create a strong maritime character. This movement from the Gulf of Mexico helps to decrease the range between hot and cold temperatures and provides a source of abundant moisture and rainfall.

170

L2.1.2 Temperature

175

Records of temperature are available from "Climatography of the United States No. 81" for Louisiana, published by the National Climatic Data Center. The study area can be described by using the normal temperature data observed at three stations located within the study area. These stations are shown in Table L1 with the monthly and annual mean normals based on the period 1971-2000. The average annual mean normal temperature is 68.5 degrees Fahrenheit (°F), with monthly mean temperature normal varying from 82.5°F in July to 51.8°F in January.

Table L1 - Mean Monthly and Annual Temperatures (°F)

30 Year Normals (1971 - 2000)													
Station	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	ANN
Galliano	53.0	55.7	62.3	67.9	75.1	80.1	81.9	81.8	78.5	69.9	62.2	55.3	68.6
Houma	53.1	56.2	62.7	68.4	75.8	80.7	82.5	82.3	78.9	69.9	62.1	55.4	69.0
Morgan City	51.8	54.8	61.2	67.4	74.5	79.6	81.5	81.2	78.2	70.0	61.3	54.6	68.0

180

A maximum extreme temperature of 102°F was recorded at Morgan City during July 1980 and a minimum extreme of 4°F was recorded during December 1971 at Morgan City. Figure L2 shows the location of the climate gages.

L2.1.3 Precipitation

185

Records of precipitation are also available in publications by the National Climatic Center. Three stations have been used to show the rainfall data for the study area (these stations are shown on Figure L2). Table L2 gives a list of the stations with their period of record and available extremes. Three of these stations have 30-year monthly and annual normals. The average annual normal rainfall of these stations is 64.14 in. based over the period 1961-1990. Table L3 lists the monthly and annual normals. The wettest month is July with an average monthly normal of 7.71 in. October is the driest month averaging 3.47 in.

190

Table L2 - Precipitation Extremes

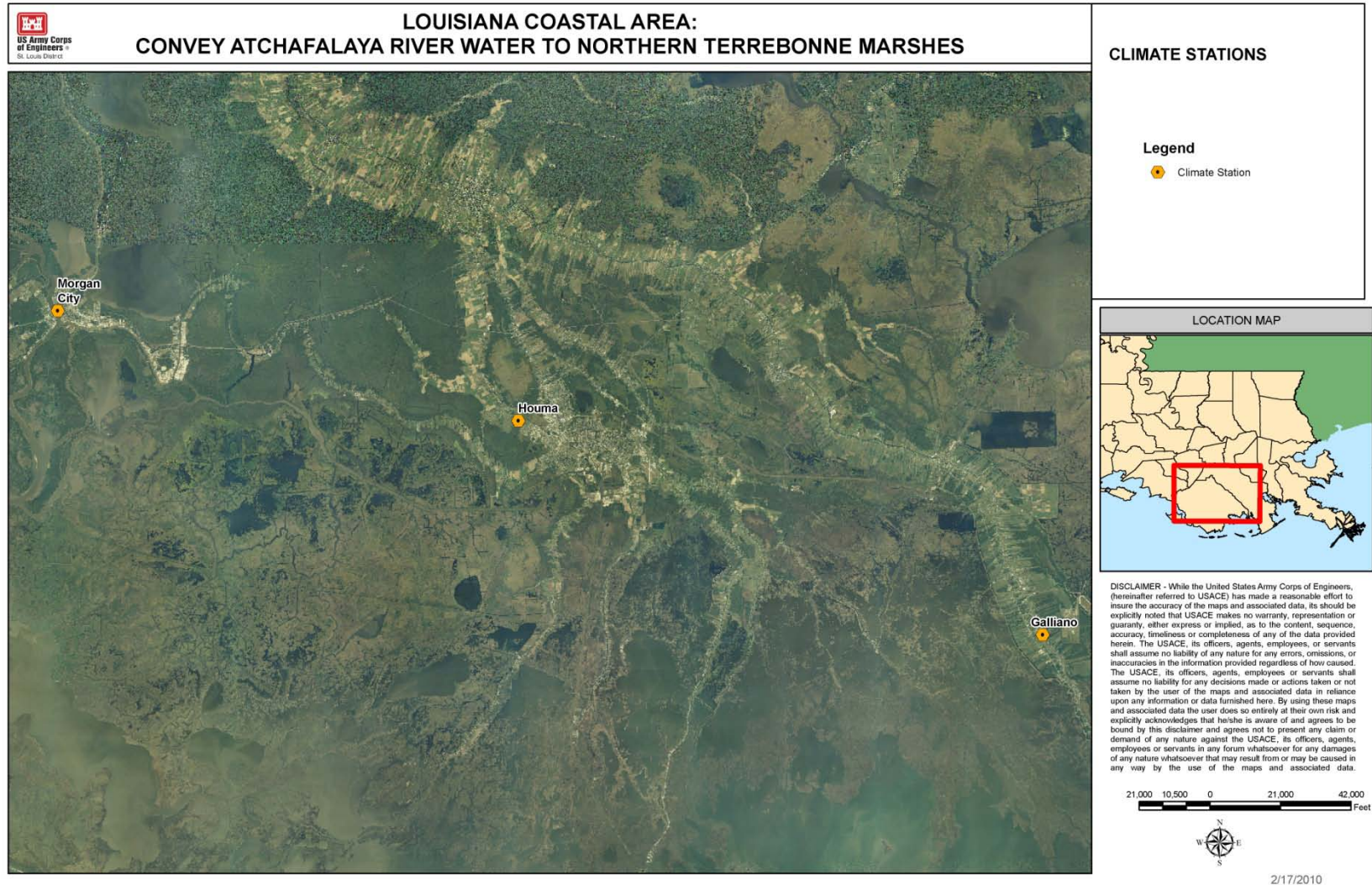
Station	Period of Record (to 2001)	Maximum		Minimum		Greatest	
		Monthly (in)	Date	Monthly (in)	Date	1-Day (in)	Date
Galliano	1968 - Date	21.35	09/98	0.12	10/78	9.90	5/30/1975
Houma	1930 - Date	20.84	05/91	0.00	10/78	11.35	5/31/1959
Morgan City	1930 - Date	18.82	05/91	0.01	10/78	10.02	5/9/1995

195 **Table L3 - Monthly and Annual Precipitation (inches)**

30 Year Normals (1971 - 2000)													
Station	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	ANN
Galliano	5.85	4.59	5.53	4.43	5.75	5.82	7.69	7.13	6.34	3.65	4.67	4.03	65.48
Houma	5.43	4.59	4.96	4.46	5.35	5.96	7.85	6.73	6.28	3.11	4.55	4.40	63.67
Morgan City	5.81	4.39	4.70	4.22	5.38	5.81	7.60	7.40	6.49	3.66	5.07	4.95	65.48

L2.1.4 Wind

200 The average wind speed in the study area is 7.9 miles per hour (mph), based on the period 1973-1998 at New Orleans Moisant Airport. Southeast winds predominate in the spring and fall while the fall and winter's prevailing wind direction is from the northeast. Winter storms in the area have produced wind speeds of up to 47 mph. The summer is often disturbed by tropical storms and hurricanes that produce the highest winds in the area. The maximum wind speed observed (highest one-minute speed) since 1963 was 69 mph, and was caused by Hurricane Betsy in September 1965.



205

Figure L2 - Climate Stations

L2.1.5 Stream Gaging Data

210 Stream gaging data are available from eight major stations in the study area. Some stations are maintained through a cooperative agreement between the U.S. Army Corps of Engineers and the U.S. Geological Survey. The stations with their maximum and minimum stages and available discharges are shown in Table L4. The station locations are shown on Figure L3.

Table L4 - Stream Gaging Data

Station	Period of Record	Maximum Stage		Minimum Stage	
		(ft) NGVD	Date	(ft) NGVD	Date
Bayou Lafourche					
at Thibodaux	1966 - Present i	9.8	6/7/2001	-0.4	12/2/1966
Bayou Grand Caillou					
at Dulac	1984 - Present i	8.89	10/28/1985	-	-
Houma Navigation Canal					
at Dulac	1992 - Present	7.17	9/12/2008	-1.42	1/8/1996
GIWW					
at Houma	1997 - Present	4.87	9/13/2008	-0.34	12/15/1997
Bayou Boeuf					
at Amelia	1955 to Present i	4.9	4/30/1975	DRY	1/20/1940
Bayou Boeuf (Lock)					
West	1954 - Present	10.56	5/27/1973	-1.98	11/8/1959
East	1954 - Present	4.68	4/17/1973	-1.14	12/24/1989
Lower Atchafalaya River					
at Morgan City	1905 - Present	10.53a	8/25/1992	-5.44c	8/25/2026
Station	Period of Record	Maximum Discharge		Minimum Discharge	
		cfs	Date	cfs	Date
Bayou Lafourche					
at Thibodaux	1984 - Present	1,450	5/9/1995	-	-
Lower Atchafalaya River					
at Morgan City	1976 - Present i	741,000	6/8/2027	-151,000	10/3/2003
i Intermittently					
a From incomplete records					

215

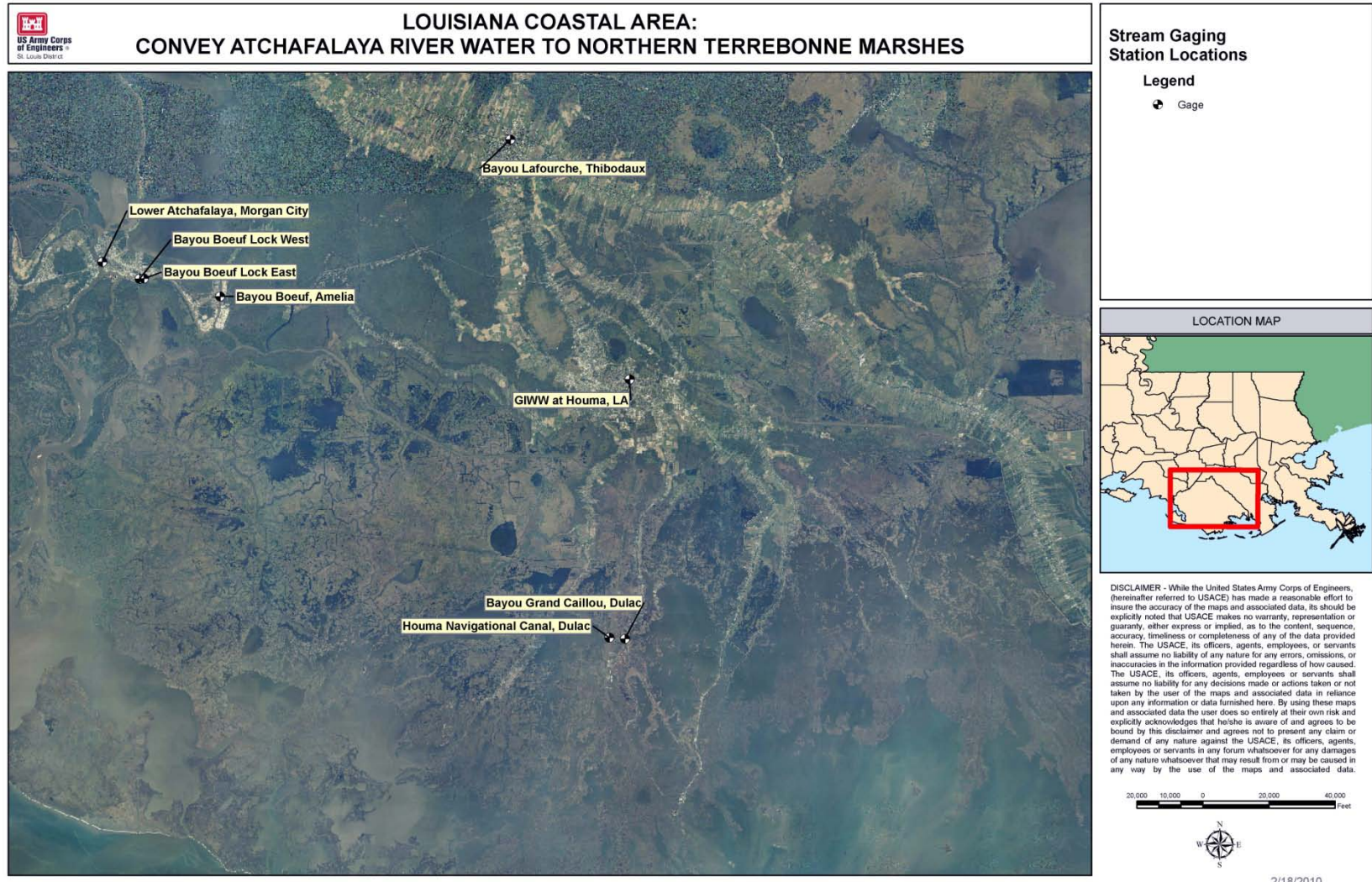


Figure L3 - Stream Gaging Stations

220 **L2.1.6 Floods of Record**

The study area floods from tidal surges associated with hurricanes and tropical storms. Lower Atchafalaya River waters enter the study area from the Avoca Island Cutoff Channel and the GIWW. Heavy rainfall also affects highly developed areas.

225 Some of the major historical floods caused by heavy rainfall or tides occurred in 1973, 1980, 1983, and 1991. A description of these significant floods follows.

230 **1973 Flood:** Flooding occurred throughout the eastern portion of the study area during the spring of 1973. Below Highway 90, tidal flooding inundated nearly all areas except the alluvial ridges of the Mississippi River, Bayou Lafourche, and the many smaller streams that drain into the Gulf of Mexico. Peak stages recorded on May 27 include 11.16 (ft), NGVD at Wax Lake Outlet at Calumet and 6.27 ft, NGVD at Lower Atchafalaya River below Sweet Bay Lake. On May 28, flooding caused a high stage of 10.53 ft, NGVD on the Lower Atchafalaya River at Morgan City.

235 **1980 Flood:** Heavy rains at the end of March and early April setup flooding which occurred over the study area during mid April. A maximum extreme was set at Bayou Black at Greenwood (4.82 ft, NGVD). Some of the one-day rainfall totals on April 13 were 9.1 in. at Morgan City and 11.8 in. at Thibodaux.

240 **1983 Flood:** Heavy rains north of the study area produced this flood. In the Atchafalaya Basin Floodway System, peak stages from this event include 8.11 ft, NGVD at Wax Lake Outlet at Calumet and 7.32 ft, NGVD at Lower Atchafalaya River at Morgan City, both on June 6.

245 **1991 Flood:** Flooding occurred throughout the study area due to above normal rainfall during most of the year. One example of this flooding occurred in the Houma-Thibodaux area during May 8-10. Three-day totals at these two sites were 12.94 and 14.33 in., respectively. The rainfall event set a maximum extreme stage of 8.76 ft, NGVD on Bayou Lafourche at Thibodaux gage on May 9. In addition to heavy rainfall, high tides in the Gulf of Mexico affected runoff.

250 Some of the major historical hurricanes that affected the study area were in 1909, 1915, 1956 (Flossy), 1957 (Audrey), 1961 (Carla), 1964 (Hilda), 1974 (Carmen), 1985 (Juan), and 1992 (Andrew). A description of these significant storms follows.

255 **1909 Flood:** Wind speeds of 80 mph were reported for Thibodaux and for the mouth of Bayou Terrebonne, 40 miles south of Thibodaux, as a tropical cyclone passed through the study area from September 19-20. The highest tides were experienced at the mouth of Bayou Terrebonne in Lafourche Parish, where an elevation of 15 ft above sea level was attained at Sea Breeze.

260 **1915 Flood:** Heavy rainfall, high winds, and extremely low barometric pressures from this hurricane from 29 September- 2 October caused headwater flooding along Bayou Lafourche where stages of 9 and 5 ft above sea level, respectively, were reported at Leeville and Golden Meadow. The U.S. Weather Bureau 5-minute sustained and extreme wind velocities for the 29

September were 66 and 75 miles per hour at New Orleans. In Leeville, approximately 13 miles west of Grand Isle, only 1 of the 100 houses remained standing as a result of this storm.

265

1956 Flood: Hurricane Flossy, during the period September 21-30, was the cause of this flood. Tides reached 5 to 8 ft above normal along most of the southeastern coast. Rainfall during the storm was quite heavy. The heaviest occurred at Golden Meadow where 16.7 in. of rain was recorded.

270

1957 Flood: Heavy rainfall and high winds associated with Hurricane Audrey, June 25-28, caused headwater flooding along the Louisiana coast. The storm set peak stages of 8.05 ft NGVD at Lower Atchafalaya River Below Sweet Bay Lake, 6.81 ft NGVD at Atchafalaya Bay at Eugene Island, 8.52 ft and 7.35 ft, NGVD respectively at IWW at Wax Lake East and West on June 27. Maximum stages were also set along the coastline on this date and included 6.00 ft NGVD at Schooner Bayou and 8.12 ft NGVD at Leland Bowman Lock.

275

1961 Flood: Hurricane Carla raised tides 3 to 4 ft above normal along the entire coastline of Louisiana during the period September 4-14. Rainfall associated with the hurricane amounted to 6.2 in. at Morgan City and 3.4 in. at Houma.

280

1964 Flood: Hurricane Hilda, during the period October 3-5, caused extensive tidal and headwater flooding in the study area. Heavy rainfall north of the study area associated with this hurricane ranged from 10.1 in. at New Roads to 8.9 in. at Baton Rouge.

285

1974 Flood: Hurricane Carmen was responsible for this flood during September 5-9. The highest known storm tide, 11.64 ft, NGVD occurred at Cocodrie in Terrebonne Parish. This stage was reportedly more than 10 ft above normal.

290

1985 Flood: Hurricane Juan, during the period October 27-31, caused massive flooding throughout the study area due to its prolonged 5-day stay along the Louisiana coast. Tides were generally 3 to 6 ft above normal, and storm surges of 5 to 8 ft were reported in several coastal parishes. Rainfall amounts in the study area ranged from 5 to nearly 17 in. for this period.

295

1992 Flood: Hurricane Andrew, during the period August 24-27, set new maximum extreme of 7.65 ft, NGVD at Round Bayou at Deer Island and recorded 6.8 ft, NGVD for the Lower Atchafalaya River at Morgan City. The Intracoastal Waterway at the Wax Lake East Control Structure recorded a stage of 6.15 ft, NGVD.

300

2002 Flood: Hurricane Lili, a major hurricane over the Gulf of Mexico, during the daylight hours of October 2, 2002 moved steadily northwest around 15 mph toward the Louisiana coast during the evening hour. The hurricane weakened rapidly to a Category 1 Hurricane by the time it made landfall during the morning of October 3rd along the south central Louisiana coast. Storm surge tides were 4 to 7 feet above normal across south Lafourche and Terrebonne Parishes. Heavy rainfall was not widespread, in part due to the rapid movement of the hurricane away from the area.

305

310 **2005 Flood:** Hurricane Rita impacted the project area from on September 23 – 24. Across
southeast Louisiana, the main affect from Hurricane Rita was the substantial storm surge
flooding that occurred in low lying communities across coastal areas of southern Terrebonne,
southern Lafourche, and southern Jefferson Parishes where numerous homes and businesses
were flooded. Some of the most substantial damage occurred in southern Terrebonne Parish
where storm surge of 5 to 7 feet above normal overtopped or breached local drainage levees
inundating many small communities.

315

L2.1.7 Tides

Tides are diurnal and range from 1.5 to 2.0 ft Inland, the extent of tidal range and area of
influence are determined by the rainfall flow exiting the drainage areas into the Gulf of Mexico
and by flows in the GIWW originating in the Atchafalaya River. Mean tide ranges are 1.3 ft at
320 Cocodrie and 0.9 ft at Leeville; inland at Houma the mean tide range is only 0.2 ft. During a
spring tidal cycle these ranges will be larger; during a neap tidal cycle these ranges will be less.

L2.2 Hydraulic Modeling

RMA-2 and RMA-11 modeling was performed on the entire project area utilizing the Resource
325 Modeling Associates versions of the models. Model extents were the Atchafalaya River to the
west, the Mississippi River to the east, the Gulf of Mexico to the south, and the Bayou Black
ridge and other hydrologic barriers to the north. The model was calibrated to the period of
October to November 2004.

330 Alternatives analysis included high and low Atchafalaya River conditions runs. Results of these
runs were used to develop yearly hydrographs using monthly averaged values for select locations
throughout the project area. The same procedure was used for salinity and stage values. Model
results were provided to the environmental team for use in benefits analysis.

335 A detailed description of the modeling effort and its results can be found in Annex 2 to this
Appendix.

L3 SURVEYING, MAPPING, AND GEOSPATIAL DATA REQUIREMENTS

340

L3.1 Geospatial Data

The geometry representing proposed features in the maps and the Engineering plan views were
created using ArcGIS 9.3. This project consisted of new features developed by the Project
Development Team and of other features developed by previous projects. Due to features being
345 incorporated from other projects into this project, different horizontal coordinate systems were
used to create the data. The two coordinate systems used for data creation were; NAD 1983
UTM Zone 15N and NAD 1983 StatePlane Louisiana South FIPS 1702 ft. The United States
Army Corps of Engineers St. Louis & New Orleans District and the Louisiana Department of
Wildlife and Fisheries created the above mentioned new geometry in 2009 and 2010. The data
350 were created using the 2008 Digital Orthophoto Quarter Quadrangles as a reference, for further
information on the photography see C. 3.2 Aerial Photography.

355 The United States Army Corps of Engineers St. Louis District created salinity point data, flow rate data, water surface elevation data, and the hydraulic matrix for this project. The data were created in the hydraulic modeling process of the project. For any further information on the data see the Hydraulic Engineering Annex to this Appendix.

360 The United States Army Corps of Engineers New Orleans District created similarity zones and historic districts data for this project. The data was created in the Cultural and Natural Resources Section of the Environmental Branch. For any further information on the data see the 4.2.13 Aesthetics section of this report.

365 ArcGIS software provided the capabilities of transforming the data and aerial photography into one uniform coordinate system for analysis of features and map production. The uniform coordinate system used for these tasks was NAD 1983 StatePlane Louisiana South FIPS 1702 ft.

Reference Data:

- Boring Logs. Louisiana: United States Army Corps of Engineers – New Orleans District, 2009.
- 370 Cities. United States Army Corps of Engineers – New Orleans District, 2009.
- Communities. Louisiana: State of Louisiana, 2009.
- District Boundary. Louisiana: United States Army Corps of Engineers – New Orleans District, 2009.
- Eco Regions. Louisiana: State of Louisiana, 2009.
- 375 Existing Land Use. Louisiana: State of Louisiana, 2009.
- Federal and Local Levees. Louisiana: Terrebonne Parish Consolidated Government, 2009.
- Federal and Local Levees. Louisiana: United States Army Corps of Engineers – New Orleans District, 2009.
- General Base Data. Tele Atlas North America, 2007.
- 380 Hurricane Tracks. National Oceanic and Atmospheric Administration Coastal Services Center, 2009.
- Hydrography. Louisiana: State of Louisiana, 2009.
- Hydrography. Louisiana: United States Army Corps of Engineers – New Orleans District, 2009.
- Landowners. Louisiana: Terrebonne Parish Consolidated Government, 2009.
- 385 Land Gain/Loss Rates. United States Geological Survey, 2009.
- National Forests. Louisiana: State of Louisiana, 2009.
- National Parks. Louisiana: State of Louisiana, 2009.
- National Wildlife Refugees. Louisiana: State of Louisiana, 2009.
- National Wildlife Refugees. Louisiana: United States Army Corps of Engineers – New Orleans District, 2009.
- 390 Oil and Gas Infrastructure. Louisiana: Louisiana Department of Natural Resources Strategic Online Natural Resources Information System, 2009.
- Oyster Seed Grounds and Leases. Louisiana: Louisiana Department of Wildlife and Fisheries, 2009.
- 395 Parish Boundaries. Louisiana: State of Louisiana, 2009.
- Parish Boundaries. Louisiana: United States Army Corps of Engineers – New Orleans District, 2009.
- Soils. Louisiana: United States Army Corps of Engineers – New Orleans District, 2009.
- State Parks. Louisiana: State of Louisiana, 2009.
- 400 State Parks. Louisiana: United States Army Corps of Engineers – New Orleans District, 2009.
- Thoroughfares. Louisiana: State of Louisiana, 2009.

Urban Centers. Louisiana: State of Louisiana, 2009.

USGS Habitat Classifications. United States Geological Survey & National Wetlands Research Center, 2009.

405 Utilities. Louisiana: Terrebonne Parish Consolidated Government, 2010.

Wildlife Management Areas. Louisiana: State of Louisiana, 2009.

Wildlife Management Areas. Louisiana: United States Army Corps of Engineers – New Orleans District, 2009.

410 **L3.2 Aerial Photography**

L3.2.1 2008 Digital Orthophoto Quarter Quadrangles

The 2008 Digital Orthophoto Quarter Quadrangles (DOQQs) were provided by the United States Army Corps of Engineers New Orleans District. The following information is provided in the metadata of the DOQQ data set. This data set was produced in accordance with USGS Standards for Digital Orthophotos, 1996. Review was provided by the USGS National Geospatial Technical Operations Center (NGTOC). The data set was created by Photo Science, Inc. in 2009 for the USGS National Wetlands Research Center and CWPPRA Task Force.

The horizontal coordinate system is projected coordinate system NAD 1983 UTM Zone 15N. The DOQQ horizontal positional accuracy and the assurance of that accuracy depend, in part, on the accuracy of the data inputs to the rectification process. These inputs consist of the digital elevation model (DEM), aerotriangulation control and methods, sensor calibration, and aerial imagery that meet National Aerial Photography Program (NAPP) standards. The vertical accuracy of the verified USGS format DEM is equivalent to or better than a USGS level 1 or 2 DEM, with a root mean square error (RMSE) of no greater than 7.0 meters. Field control is acquired by third-order class 1 or better survey methods sufficiently spaced to meet National Map Accuracy Standards (NMAS) for 1:12,000-scale products. Photo-identifiable ground test points are identified in the orthorectified image and measured. The image coordinates are compared to the known positions of these points and the radial differences for each point computed. A radial RMSE value is then calculated for the DOQQ. Note: Adjacent DOQQ's, when displayed together in a common planimetric coordinate system, may exhibit positional discrepancies across common DOQQ boundaries. Linear features, such as streets, may be offset between images. However, these edge mismatches still conform to NMAS positional horizontal accuracy requirements. The estimated accuracy is 3.34 meters which was determined by the Federal Geographic Data Committee, 1998, Geospatial Positioning Accuracy Standard, Part 3, National Standard for Spatial Data Accuracy, FGDC-STD-007.3-1998.

L3.2.2 2002 LDEQ Landsat Enhanced Thematic Mapper Pan-Sharpener Mosaic of Louisiana UTM15 NAD83, (2002) MrSID

440 The 2002 Landsat Imagery was provided by the United States Army Corps of Engineers New Orleans District. The following information is provided in the metadata of the Landsat data set.

The horizontal coordinate system is projected coordinate system NAD 1983 UTM Zone 15N. This data set is a satellite image of the lands and waters of the State of Louisiana. It was created by combining fourteen scenes of 30-meter resolution Landsat Thematic Mapper (TM) imagery with 15-meter resolution panchromatic imagery. The TM and panchromatic imagery for each scene are coincident. The original image data were geo-rectified and resampled using cubic convolution to 25-meter (TM) and 12.5-meter (pan) cells by the Earth Resources Observation Systems (EROS) Data Center. These data were purchased from EROS by the Louisiana Department of Environmental Quality (northern half of state) and the USGS's National Wetlands Research Center Lafayette (southern half of state.) The processing to produce a seamless enhanced image was performed at LDEQ by a LDEQ contractor. The work was funded by a grant from the US EPA to the LDEQ Non-Point Source Water Pollution Section. The image was constructed from a red, green, blue (RGB) composite of bands 7,5 & 3 fused with the

455 panchromatic image to produce the enhanced TM pan sharpened mosaic. The merged satellite image was produced to support on-going research for the LDEQ Non-point Source Program by providing a more current view of land cover/land use within Louisiana and to support NWRC's work in evaluating Louisiana's coastal wetlands.

460 **L3.2.3 2002 Louisiana Federal Emergency Management Agency (FEMA) LIDAR**

The 2002 LIDAR was provided by the United States Army Corps of Engineers New Orleans District. The following information is provided in the metadata of the LIDAR data set.

465 The horizontal coordinate system is projected coordinate system NAD 1983 UTM Zone 15N. These data were produced for the Louisiana Federal Emergency Management Agency (FEMA) Project under the U.S. Army Corps of Engineers, Saint Louis District contract number DACW43-00D-0511 0014.

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485 **L3.3 Ground Topographic Surveys**

No surveying was conducted during the feasibility stage of this project.

L4 GEOLOGY

490 The geologic development of coastal Louisiana is closely related to shifting Mississippi River courses since the slowing of Holocene post-glacial sea level rise (Fisk, 1955; Frazier, 1967; and Coleman and Gagliano, 1964). The Mississippi River has changed its course several times during the last 7,000 years, leading to the development of the Mississippi River deltaic and chenier plains. The deltaic plain is composed of several major delta complexes, two of which
495 (the Plaquemines/Modern and Atchafalaya) are still active. Dominant physiographic features of the deltaic plain include abandoned courses and distributaries and their associated natural levees, swamps, marsh, hundreds of lakes and bays, and barrier islands.

500 Recognition that the deltaic plain is formed by an orderly progression of events related to
shifting Mississippi River courses led to the identification and characterization of the “delta
cycle” (Scruton, 1960; Frazier, 1967). The “delta cycle” is a dynamic and cyclic process that
alternates between periods of progradation and a subsequent transgression of deltaic headlands as
deltas are abandoned and reworked by marine waters (Penland et al., 1988; Roberts, 1997).
505 Throughout most of the last 7000 years the “delta cycle” has created more land by building
deltas (regressive phase) than was destroyed by relative sea level rise and erosional processes
(transgressive phase). Since the early 1900’s man has had a major influence on many of the key
elements controlling the “delta cycle”. The Old River Control Structure has eliminated the delta
switching process by maintaining the Mississippi River in its present course. Flood protection
levees confine the flow of the Mississippi River eliminating overbank flooding and the nutrients
510 and sediments that accompany these floods. Also, the suspended sediment load of the
Mississippi River has declined by approximately 50 percent between the 1930 to 1952 period
and the 1963 to 1982 period (Kesel, 1988). This decline has been attributed to bank stabilization
by revetments, dams constructed on the Missouri River and other large tributaries, and other
erosion control measures.

515 As the natural delta-building process was restrained, relative sea level rise and erosion
(transgressive processes) began to dominate the coastal landscape. Within this environment of
diminished delta building, man began a period of extensive development in the coastal zone
beginning in the early 1900’s. Man-made alterations to the natural landscape such as dredging of
520 navigation and access canals, construction of roads and levees within the wetlands, and drainage
projects altered the natural hydrology compounding the negative effects of relative sea level rise
and wetland erosion

525 Coastal Louisiana is characterized by depositional environments and shoreline configurations
representing various phases of the “delta cycle”. Presently, most of the Louisiana coastal zone is
in the marine-dominated, transgressive phase of the “delta cycle”. Only the Modern and
Atchafalaya Deltas are in the fluvially-dominated, regressive phase.

L4.1 Geologic Setting of Study Area

530 The study area is part of the Teche and Lafourche Delta complexes which began depositing
deltaic sediments in the study area approximately 4,500 years ago (Frazier, 1967). Bayou Black
was the main course of the Teche-Mississippi River which entered the study area from the west
approximately 4,500 years before present. Major distributaries of the Teche Delta which
535 contributed sediment to the study area were Bayous Penchant, Cocodrie, Piquant, Little Horn,
and Carencro. These distributaries all trend southeast, indicative of the direction of deltaic
growth. Bayou Lafourche was the main course of the Lafourche-Mississippi River which
entered the study area from the north approximately 2,000 years before present. Major
distributaries of the Lafourche Delta which contributed sediment to the study area were Bayous
540 Mauvais Bois, du Large, Grand Caillou, Terrebonne, Little Caillou, and Pointe au Chien.
Dominant physiographic features include the natural levees associated with Bayous Black and
Lafourche, Lakes such as Palourde, Mechant, de Cade, Boudreaux, and Felicity, the Isles
Dernieres and Timbalier barrier island chains, marshes, and bays.

- 545 The entire study area is in the marine dominated/transgressive phase of the “delta cycle”. During this phase, delta abandonment takes place and the processes of subsidence, erosion, and marine transgression dominate the landscape. The result is land loss, submergence, habitat change (driven mainly by elevation decrease and salinity increase), and an increase in water area.
- 550 The surface and shallow subsurface is generally characterized by natural levee, swamp, and marsh deposits separated by abandoned distributaries. Natural levee deposits are generally characterized by medium to stiff clays and silt. Swamp and marsh deposits consist mainly of very soft clays and organic clays with peat. Natural levee, swamp, and marsh deposits are generally less than 20 ft thick. Beach deposits composed mainly of fine sand and shell are found
- 555 along the Isles Dernieres and Timbalier Island chains. Interdistributary deposits are located beneath natural levee, swamp and marsh deposits. Interdistributary deposits are commonly over 100 ft thick and consist of very soft to medium clays with minor amounts of silt, shell fragments, and organics. Prodelta deposits, characterized by medium clays, may underlie interdistributary deposits. Prodelta and interdistributary deposits vary widely in thickness throughout the study
- 560 area, but generally thicken from north to south. Massive substratum sands are located beneath interdistributary and prodelta deposits.

L4.3 Groundwater

- 565 Groundwater is at or near the surface throughout most of the study area. Point bar deposits associated with Bayou Black may be hydraulically connected to the adjacent waterway.

L5 GEOTECHNICAL INVESTIGATIONS AND DESIGN

L5.1 General

- 570 The project area is located in Terrebonne Parish, Louisiana. The following Table L5 presents the features and the analyses performed.

Table L5 – Geotechnical analysis performed

Feature	Pile Capacity	Slope Stability
WS4	X	X
EC3	X	X
ES2		X
EC5	X	X
CC3, CC4, CC13, CS1	X	
WW2		X
WD3		X
CD2 & 7		X

575

The subsurface information available is for a general design and cost estimate purpose. The subsequent geotechnical design on the detail features will be presented in a Design Report (DR) in an appropriate time prior to the preparation of the Plans and Specifications.

580 **L5.2 Field Investigation**

Borings were obtained from existing exploration done in the area by the New Orleans District (MVN). The borings used for this design were, CN99-3U, HNCL-32U, BLK-8U, 12-AIUT, CNO7-2U. Boring logs will be presented in the appendix as Figure 1-5. They were chosen based on the location relative to the different features and the fact that they were undisturbed samples.

585 These undisturbed samples were obtained by using a Shelby tube sampler.

While the borings are representative of subsurface conditions at their locations and specifically for the reach along the length of the boring, variations in characteristics of the subsurface materials are anticipated. Furthermore, many of the borings used, because of availability, are dated and located some distance from the actual design feature. True design borings must be done prior to any final design report at the actual location of the features in order to obtain a more accurate representation of subsurface conditions.

590

Only two of the borings indicate a groundwater level of 5.2 ft GW conditions vary with rainfall and drainage. Throughout the design a GW level of EL. 0 (NGVD) was assumed unless the available data indicated differently.

595

L5.3 Laboratory Testing

The lab testing performed on the undisturbed samples includes Atterberg limits, unit weight, unconfined compression tests and un-drained tri-axial shear tests. These results are presented in the boring logs. Most of the borings are characterized by thicker layers of fat clays (CH) with some silt and sandy silts. Logs are available in Figure L4 through Figure L8. The shear strength parameters obtained from the borings and utilized in the analyses are summarized in the following Table L6.

600

605

Table L6 - Summary of Mechanical Properties for borings used

Boring	Elevations (top to bottom)	Material	Unit Weight (pcf)	Cohesion (psf)	Phi (deg)
HNCL-32U	-5.4 to -35	CH	100	360	0
Figure L4	-35 to -45	CL	110	240	0
	-45 to -57	CH	105	400	0
CN99-3U	7 to -10	CH	100	400	0
Figure L5	-10 to -25	SM	122	0	30
BLK-8U	2 to -20	CH	100	500	0
Figure L6	-20 to -35	CH	90	450	0
	-35 to -55	CH	90	400	0
	-55 to -73	CL	120	850	0
CN07-2U	8.2 to 0	CH	110	500	0
Figure L7	0 to -15	CH	100	300	0
	-15 to -20	ML	115	700	15
	-20 to -60	CH	100	500	0
	-60 to -90	CH	100	800	0
12-AIUT	1.86 to -15	CH	105	400	0
Figure L8	-15 to -31	ML	110	300	15
	-31 to -105	CH	100	700	0

L5.4 Foundation Design

610 L5.4.1 Slope Stability Analyses (SS)

The results of the soil borings and laboratory test data were evaluated and the strength parameters were selected for design. Some design parameters were also taken from typical values presented in the USACE HSDRRS. Both, Q-case short term un-drained case and S-case long term consolidated drained case analyses were performed.

615

Table L7 - Typical Values for Silts, Sands and Rip Rap

Soil Type	Unit Weight (pcf)	Cohesion (psf)	Friction Angle (deg)
Silt	117	200	15
Silty Sand	122	0	30
Poorly graded sand	122	0	33
Riprap	132	0	40

S-case parameters

Silt: Cohesion = 0 psf, phi = 28

620 Clay: Cohesion = 0 psf, phi = 23

Stability of channel dredging and other earth cuts were analyzed using the GEO-Studio 2007 (Version 7.14) software, specifically the SLOPE/W platform. The Spencer Method is used in this analysis with a block specified slip surface to determine the critical slip surface and the factor of safety (FS).

625

Factor of Safety Requirements for HSDRRS

Low Water (hurricane condition) 1.4

630 Low Water (non-hurricane condition) S-case 1.4

L5.4.2 Individual Features SS

-WD3: A cross section of existing channel conditions and expected dredging was provided.

635 Boring BLK-8U was used for the design. The channel is about 40 ft deep with slopes of 1 on 4 (1V: 4H). For the Q-case analyses FS of 1.7 and 1.7 were obtained for the left bank (Figure L9) and the right bank, respectively. For the S-case FS of 2.1 and 1.9 were obtained for the left and right bank respectively. These FS comply with the 1.4.

640 -WW2: The rock weir in this feature was analyzed using a simple Infinite Slope Analyses since the granular material would yield infinite failures. The formula for the FS is:

$$F = A \frac{\tan \phi'}{\tan \beta} + B \frac{c'}{\gamma H}$$

$$F = \frac{\tan \phi'}{\tan \beta}$$

645 The second term can be eliminated because cohesion for rock is zero (0) and the parameter A of the first term is associated with pore-pressure which can be assumed as 1 because the weir is completely submerged. The angle phi corresponds to the friction angle and the angle beta

corresponds to the angle of the slope. This calculation resulted in the determination that a 1 on 2 slope was necessary to achieve a 1.7 FS.

650

- -EC5: Boring CN99-3U was used for the analyses. The channel is 22 ft deep and 470 ft wide. Analyses show that for Q-case and for S-case the slopes should be 1 on 4 in order to achieve a FS higher than 1.4 (Figure L10). Shoreline erosion is expected in these designs and mitigation methods shall be evaluated in a more detailed Design Report.

655

- CD2 & 7 (Figure L11): Boring HNCL-32U was used for the analysis. The features consist of a channel 45 ft wide and about 10 ft deep. On both sides fill will be used to create embankments. The one on the left bank will be a minimum of 6 ft tall and 10 ft wide at the crown. The embankment on the right will be a minimum of 8 ft tall and 10 ft wide crown. The analyses showed that for the smaller spoil bank the FS was on the order of 2.0 with the toe of the embankment a minimum of 30 ft away from the channel bank. The side with the larger embankment resulted in a FS of 1.6 with the toe of the embankment a minimum of 40 ft away from the channel bank. The S-case analyses results in a 1.6 FS for the smaller embankment and a 1.4 FS for the larger bank. These results indicate that the embankments should have slopes of 1 on 3 and the channel banks slopes of 1 on 2. If either of the embankments is to be larger than the minimum they would have to be moved further away from the channel bank in order to maintain the appropriate FS.

660

665

670

- ES2: Boring CNO7-2U was used for this analysis. The use of 1 on 4 slope is recommended to meet the Q-case and S-case FS. With a 1 on 4 slope the critical FS for the S-case is calculated as 1.4 (Figure L12).

675

- EC3: For EC3 a global stability analysis was conducted with the water elevation at 6 ft. At this level the water is almost to the crown of the levee. For this analysis the FS was 5.0 (Figure L13).

L5.4.3 Bedding for Culverts

680

The culverts that do not require deep pile foundations will require bedding material. This bedding should be 2 ft deep and should consist of 2a coarser gravel on the bottom with a finer gravel on the top layer of the bedding.

L5.4.4 Pile Foundation

685

Most of the heavier structures will require deep pile foundation because of the quality of the soil and the size of the feature. The pile founded features include: a pump station with obermeyer gates (ES2), The ten 5' x 5' box culverts (EC3), the 15' x 15' sluice gate through the levee section (WS4), the obermeyer gates across the channel with the highway bridge on top (EC5) and the 10' x 10' culverts (CC3, CC4, CC13, CS1).

690

695 Pile capacity analyses were conducted using the MVN Pile Capacity Program based on an Excel macro. Borings used include: 12-AIUT and CNO7-2U. These results are presented in the appendix. The pile capacity graph for WS4, EC3 and EC5 is noted as Figure L14 and Figure L15. The pile capacity graph for ES2, CC3, CC4, CC13 and CS1 is noted as Figure L16 and Figure L17.

Recommended FS for compression and tension design loads are:

700	<u>Design Case</u>	<u>With Pile Load Test</u>	<u>Without Pile Load Test</u>
	Q-Case	2.0	3.0
705	S-Case	1.5	1.5

Note: Q-Case is characterized as a short term un-drained case relative to the soil.

710 S-Case is characterized as a long term consolidated drained case relative to the soil.

L5.4.5 Construction Excavation

715 The excavations for construction will require a cofferdam system due to high GW levels as well as the presence of waterways. The cellular cofferdam structures will be constructed from flat sheet piles typical for such use to stabilize the adjacent ground and minimize groundwater infiltration. The cofferdam system should be designed and analyzed according to all applicable USACE design criteria.

- 720
- a. Excavation will vary from elevation -7 NGVD for the obermeyer gates to -21 NGVD for the pump station.
 - b. The design ground surface elevation varies from, depending on the feature, from 1.8 ft to -5.4 ft.
- 725

L5.4.6 Hydrostatic Uplift

730 Hydrostatic uplift during construction should be controlled by using a passive dewatering system with perforated pipe imbedded into trenches and then backfilled with gravel. Piezometric levels in the pervious and semi-pervious foundation strata should be reduced to no higher than the excavation surface. Adequate temporary piezometers shall be required to monitor the performance of the dewatering system. Because dewatering and pressure relief operations will lower the ground water level in the vicinity of the excavations and thus result in settlement of the adjacent ground surface, measures such as cutoff walls, recharge wells, and/or some other method may be necessary. A passive system was chosen based on the stratigraphy of the project

735

area, composed largely of impervious clays. A well point active system would be highly inefficient and costly because these systems are intended for more coarse grained pervious material.

740

The dewatering system shall be implemented in all features that require the construction of pile caps and in features built through levee sections. The trenches should be 3 ft deep 2 ft wide at the bottom and 8 ft wide at the top. The trenches should be built around the outer edges of the excavation and some trenches should also be built in the center of the excavation If they are large enough.

745

FIGURE 1

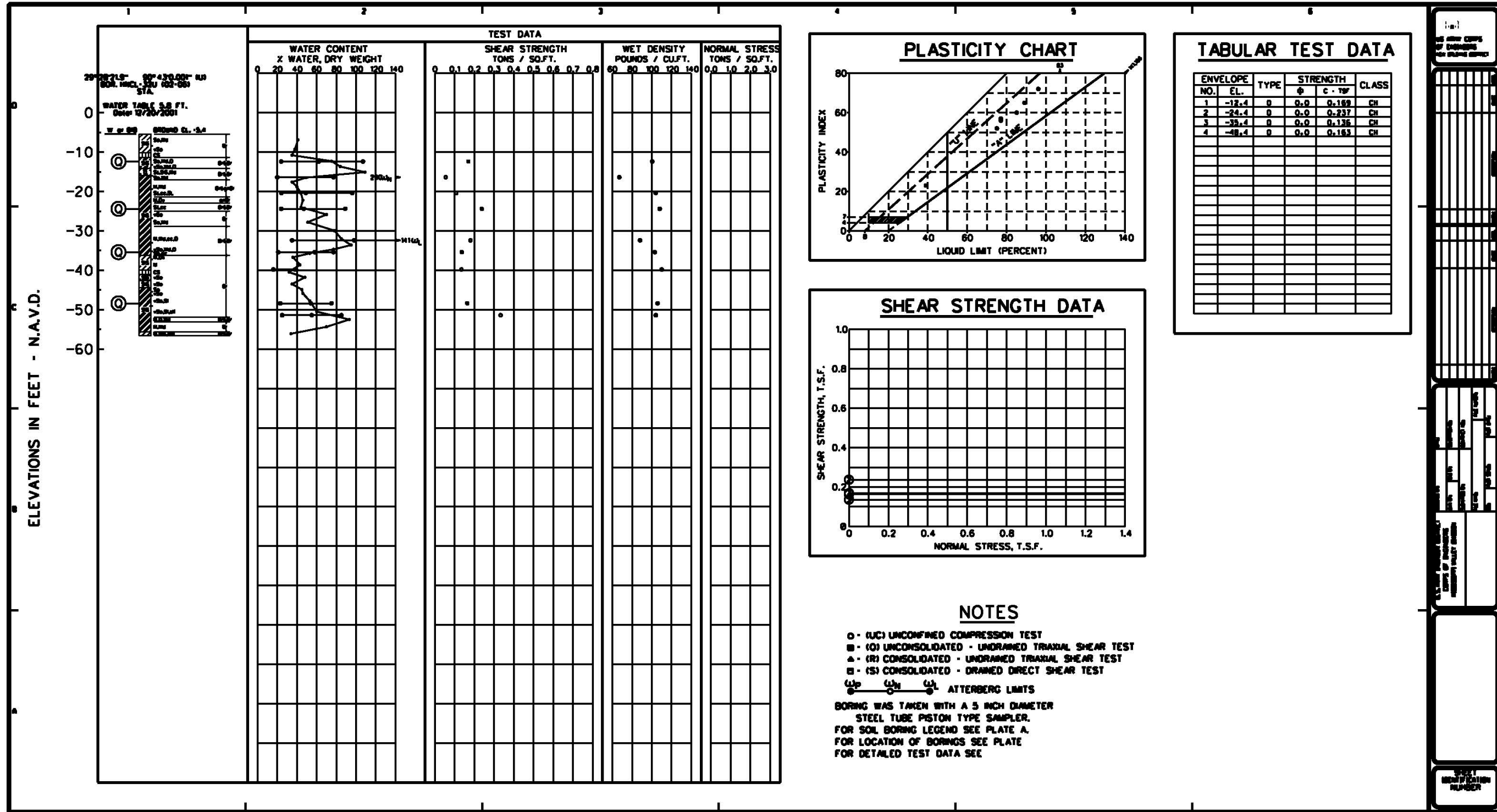


Figure L4 - Geotechnical Boring Data for Boring HNCL-32U (02-06)

FIGURE 2

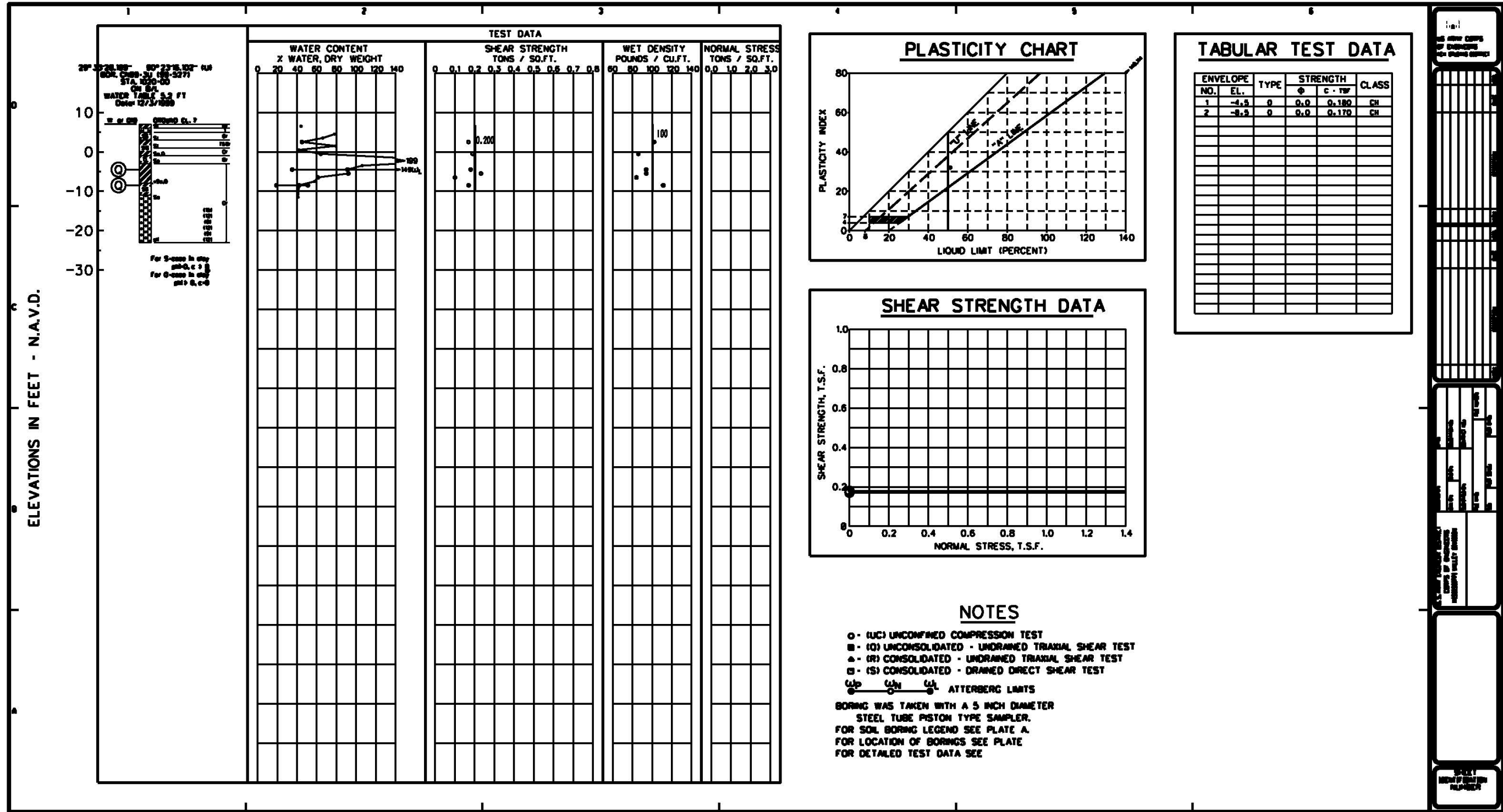


Figure L5 - Geotechnical Boring Data for Boring CN99-3U (99-527)

750

FIGURE 3

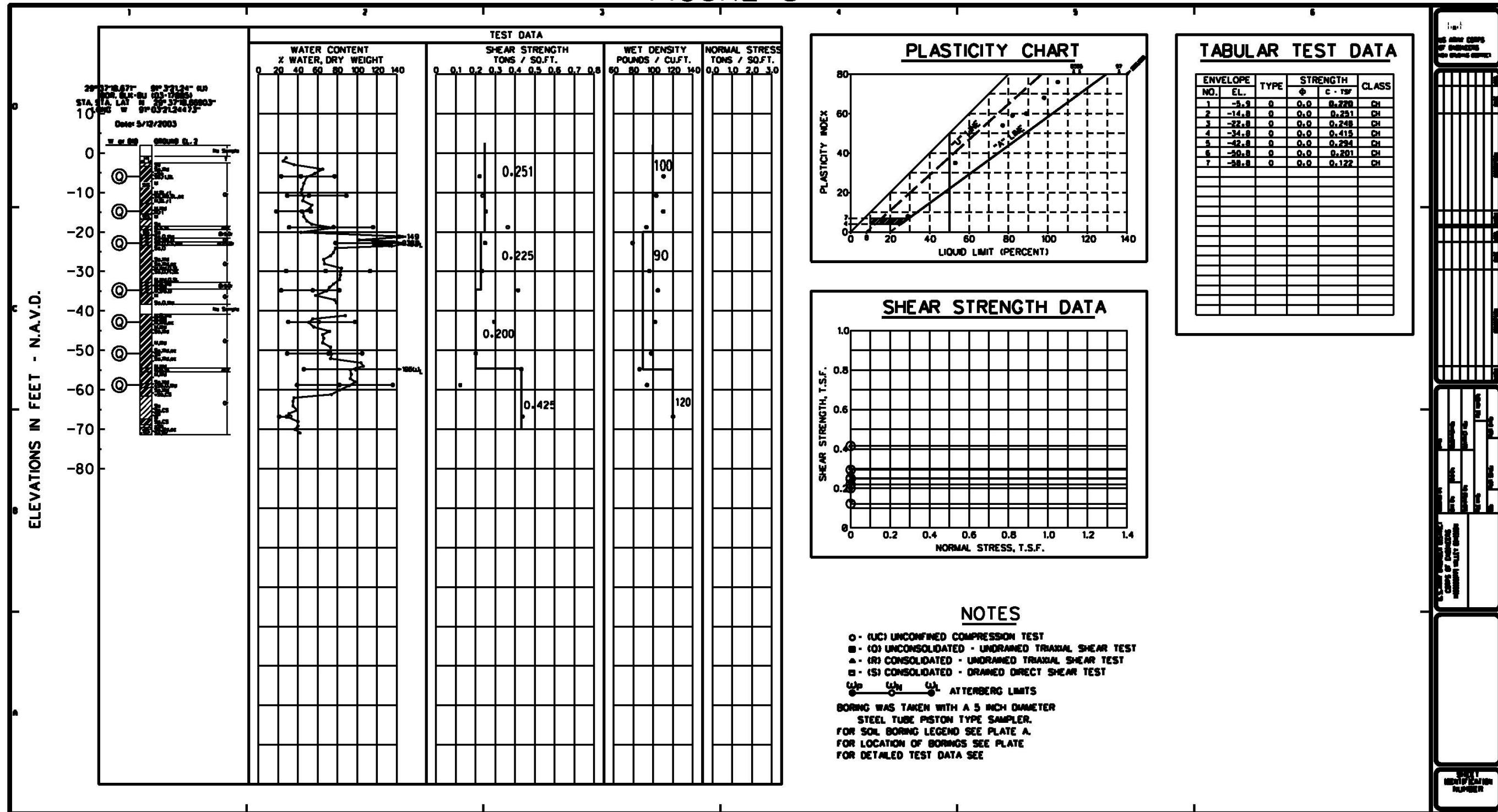


Figure L6 - Geotechnical Boring Data for Boring BLK-8U (03-17885)

FIGURE 4

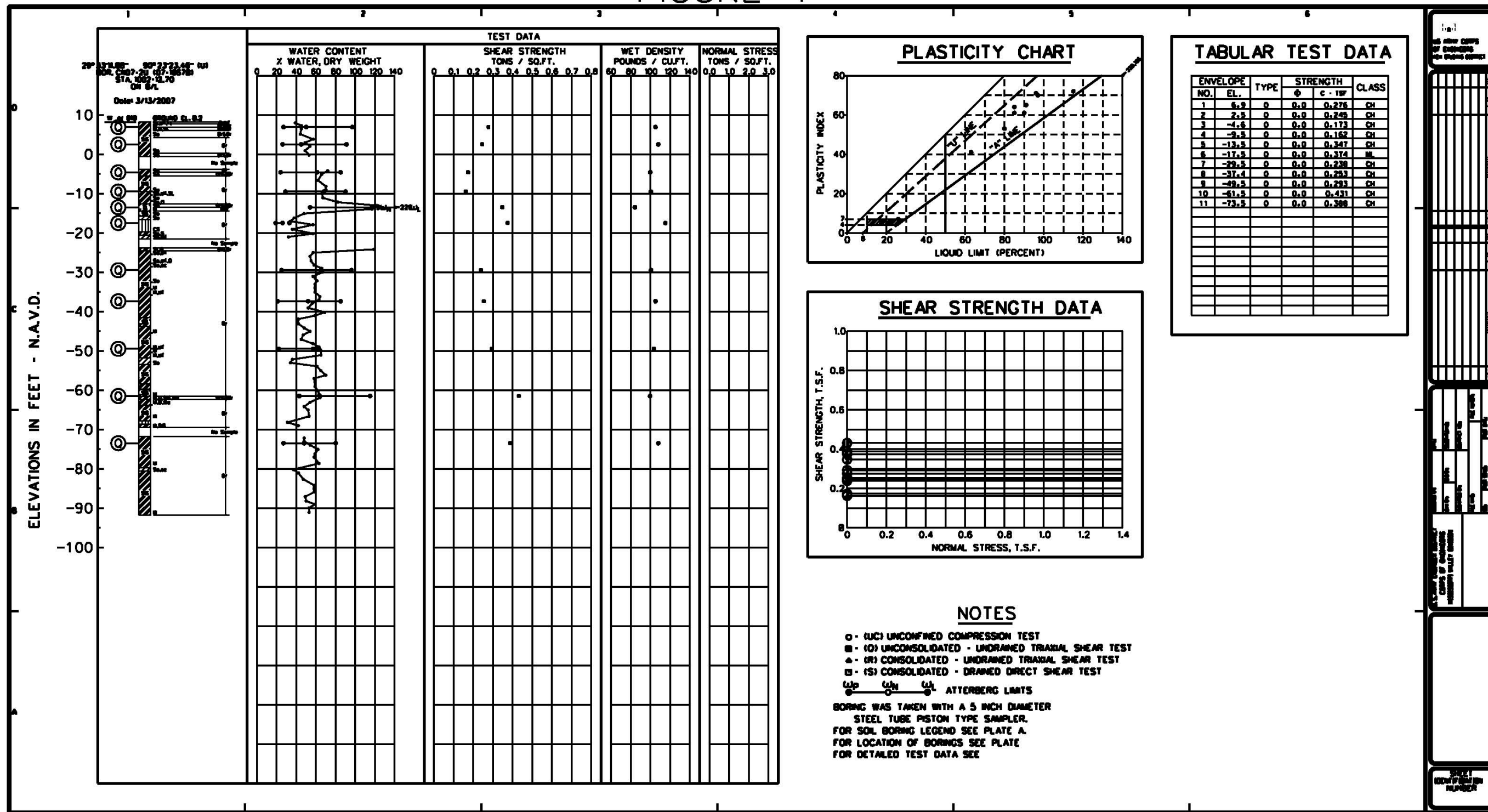
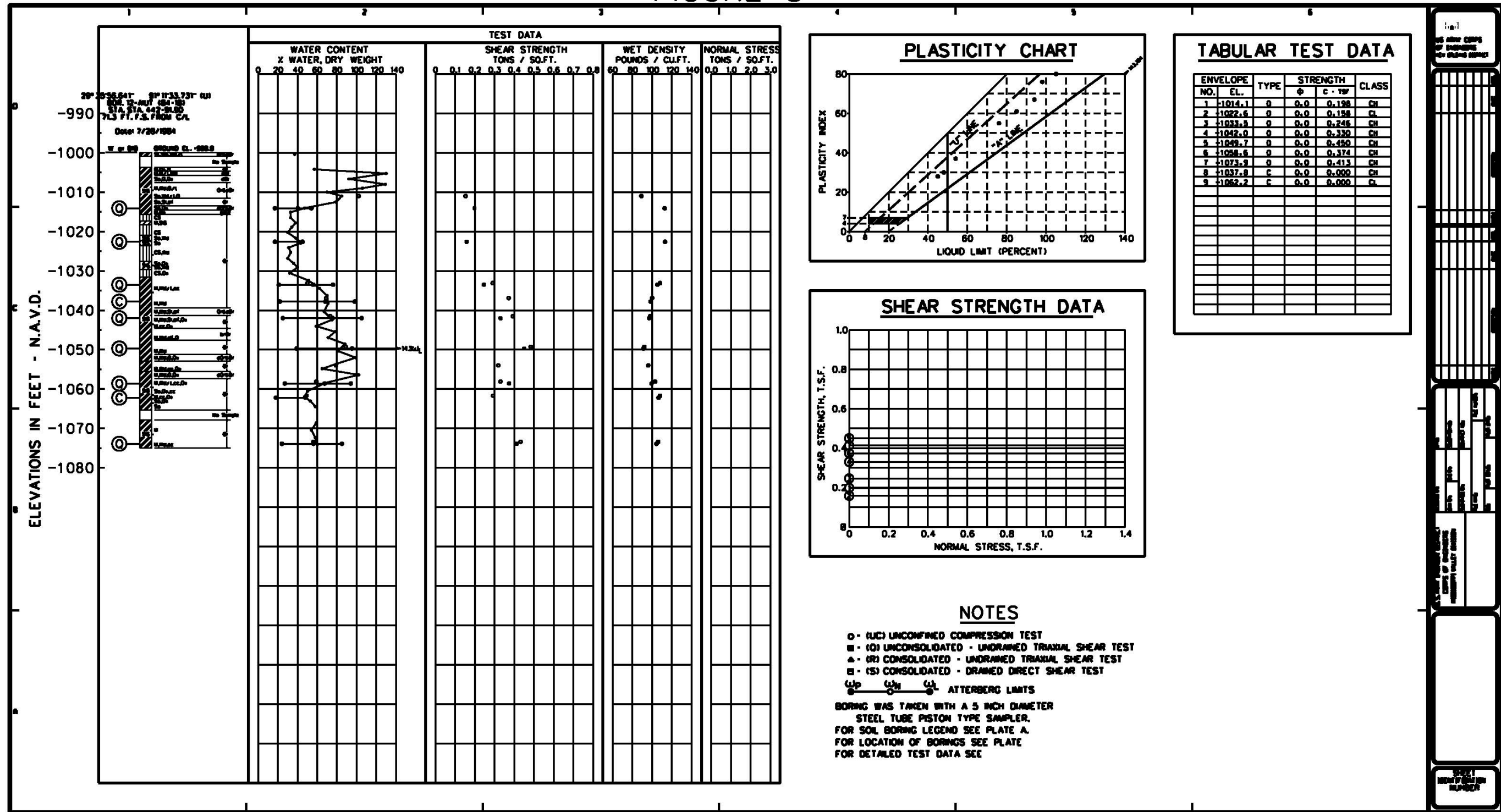


Figure L7 - Geotechnical Boring Data for Boring CN07-2U (07-19678)

FIGURE 5



760 Figure L8 - Geotechnical Boring Data for 12-AIUT (84-18)

Figure 6
 Feature WD3
 Boring # BLK-8U
 Q-case

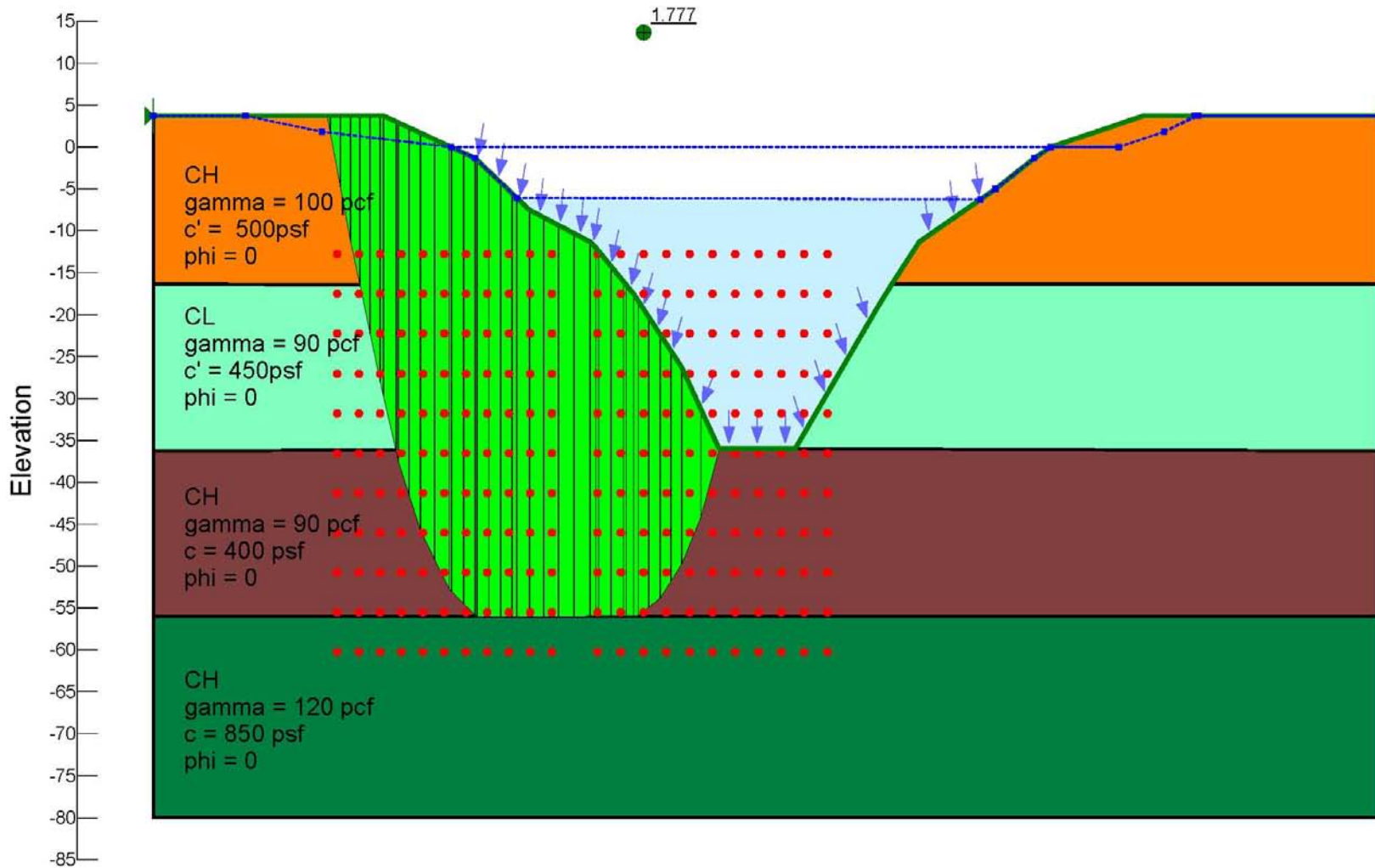
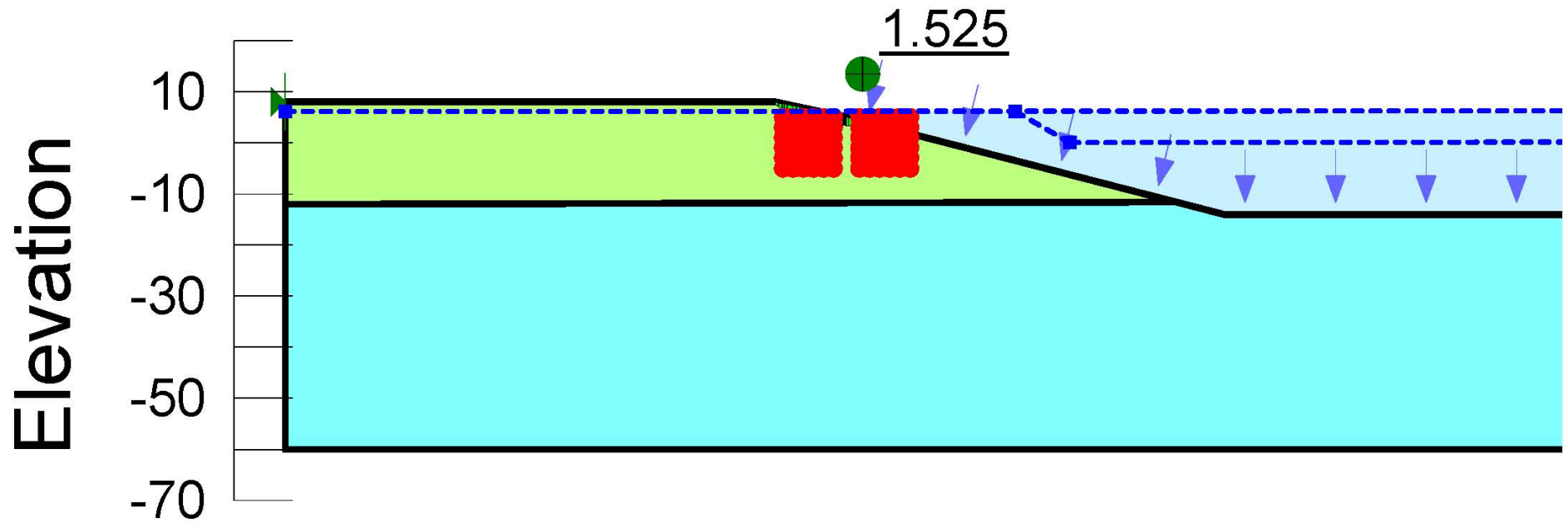


Figure L9 - WD3 Q-Case

Feature EC5 banks
Boring: CN99-3U
S Case



765 Figure L10 - EC5 Banks

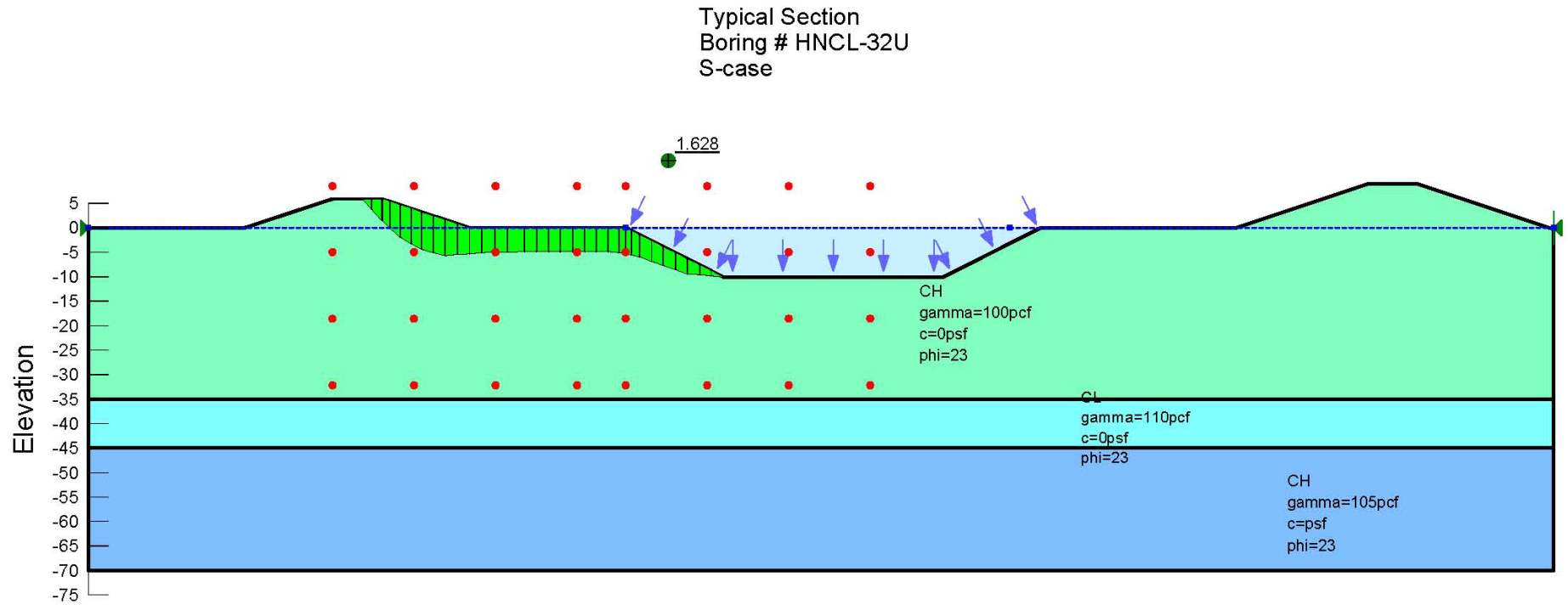
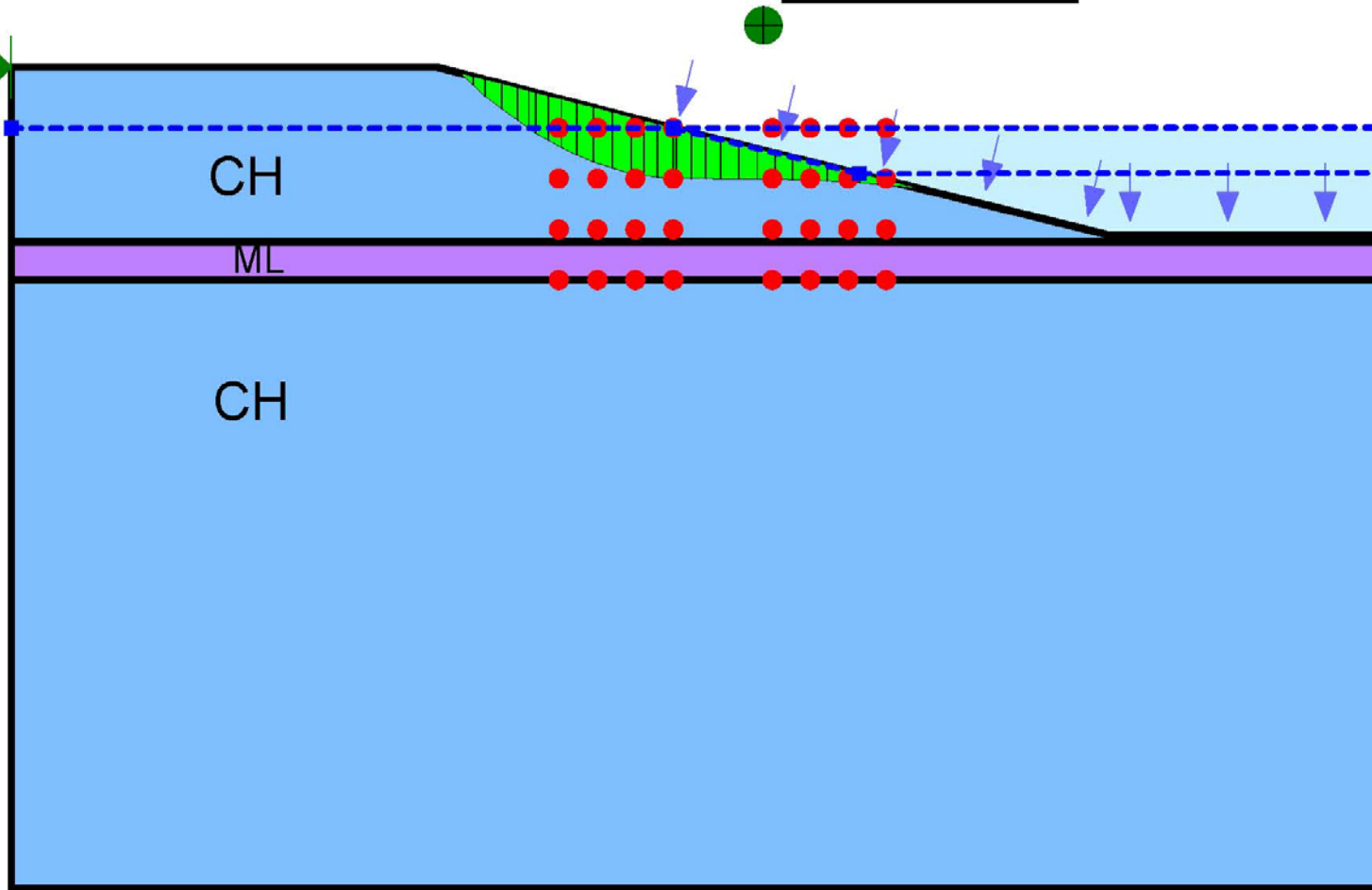


Figure L11 - S-Case

Figure 9
Feature ES2
S-case

1.405



770

Figure L12 - ES2 S-Case

Figure 10
EC3
Broing: 12-AIUT
Global Stability
S-case

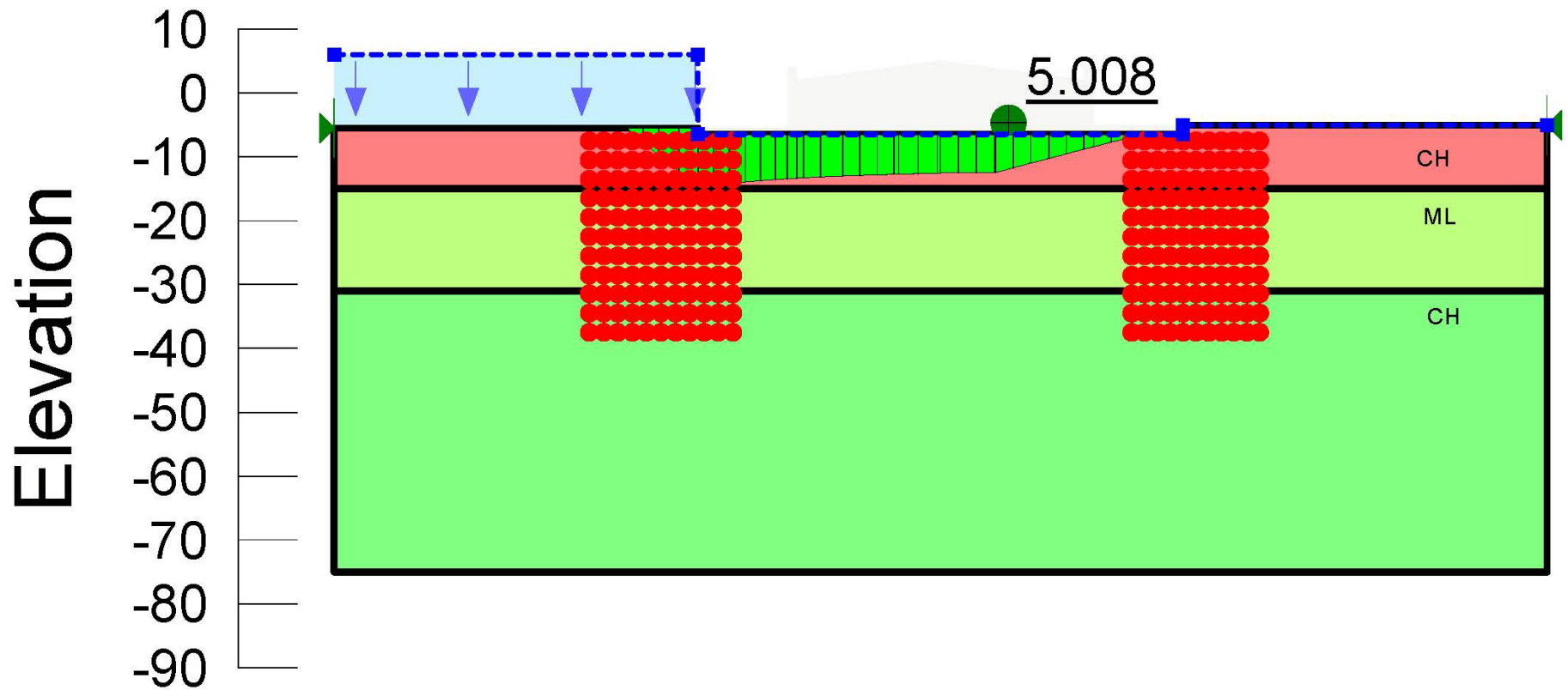


Figure L13 - EC3 Global Stability

775

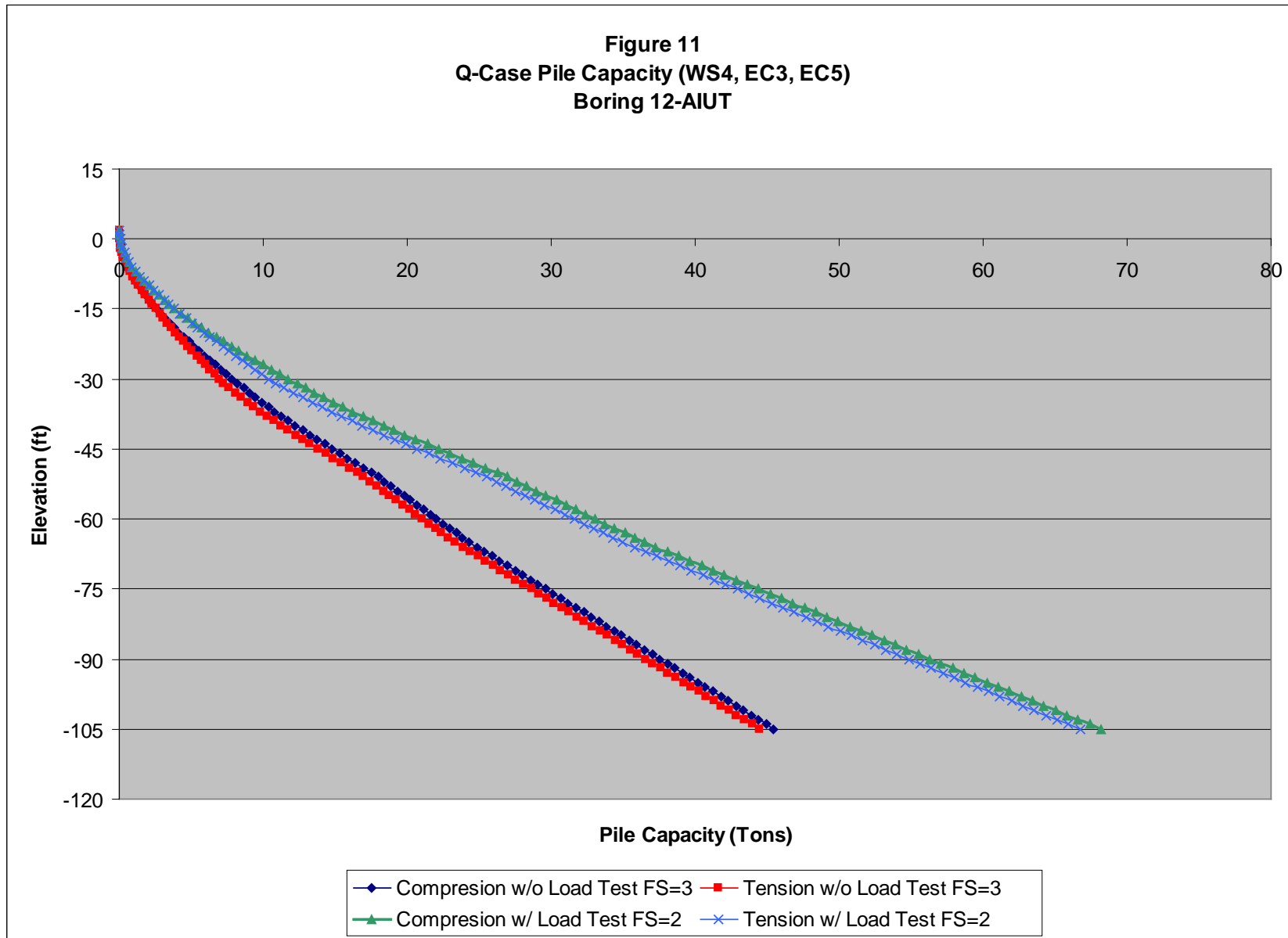


Figure L14 - Q-Case Pile Capacity

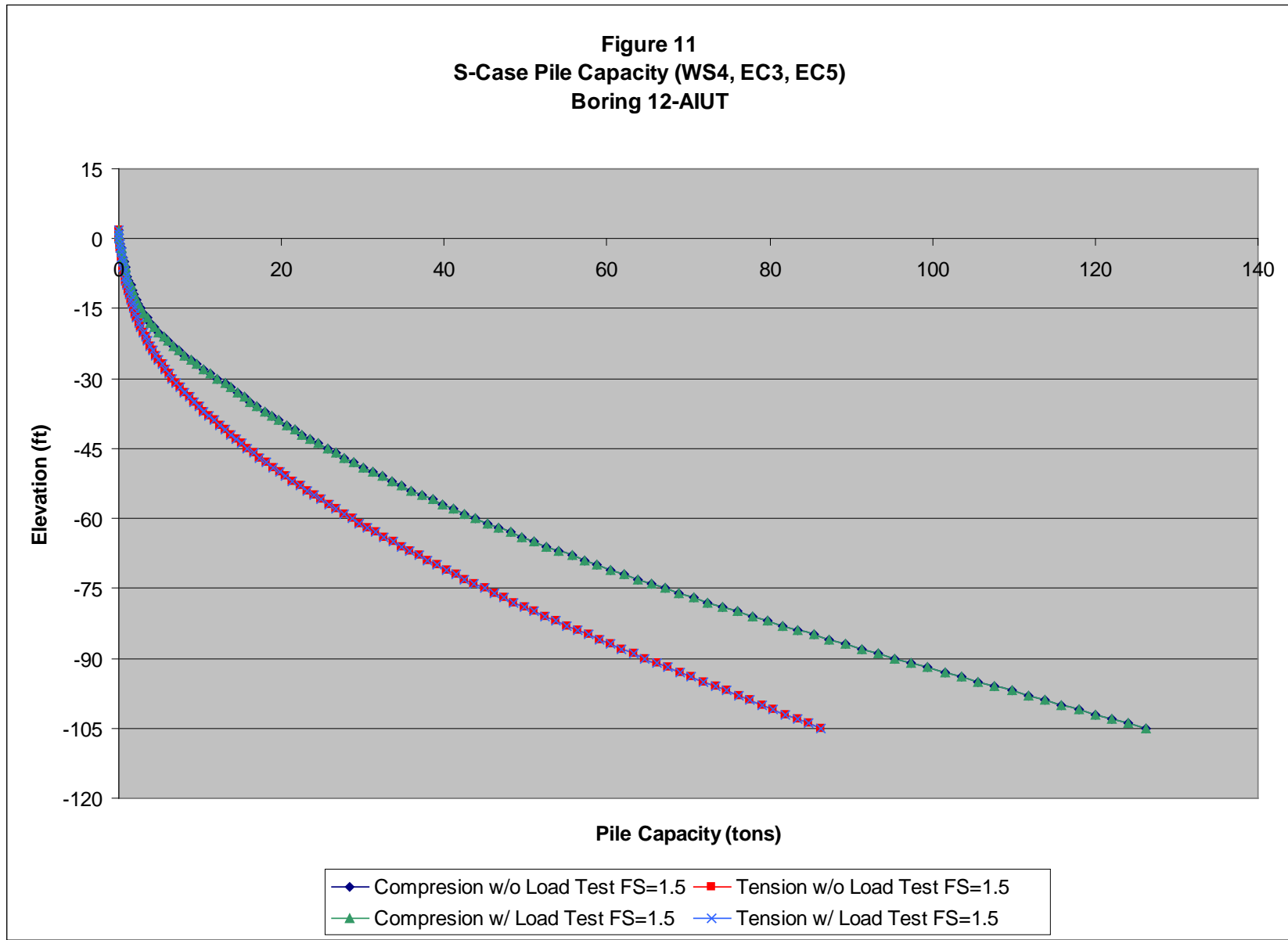
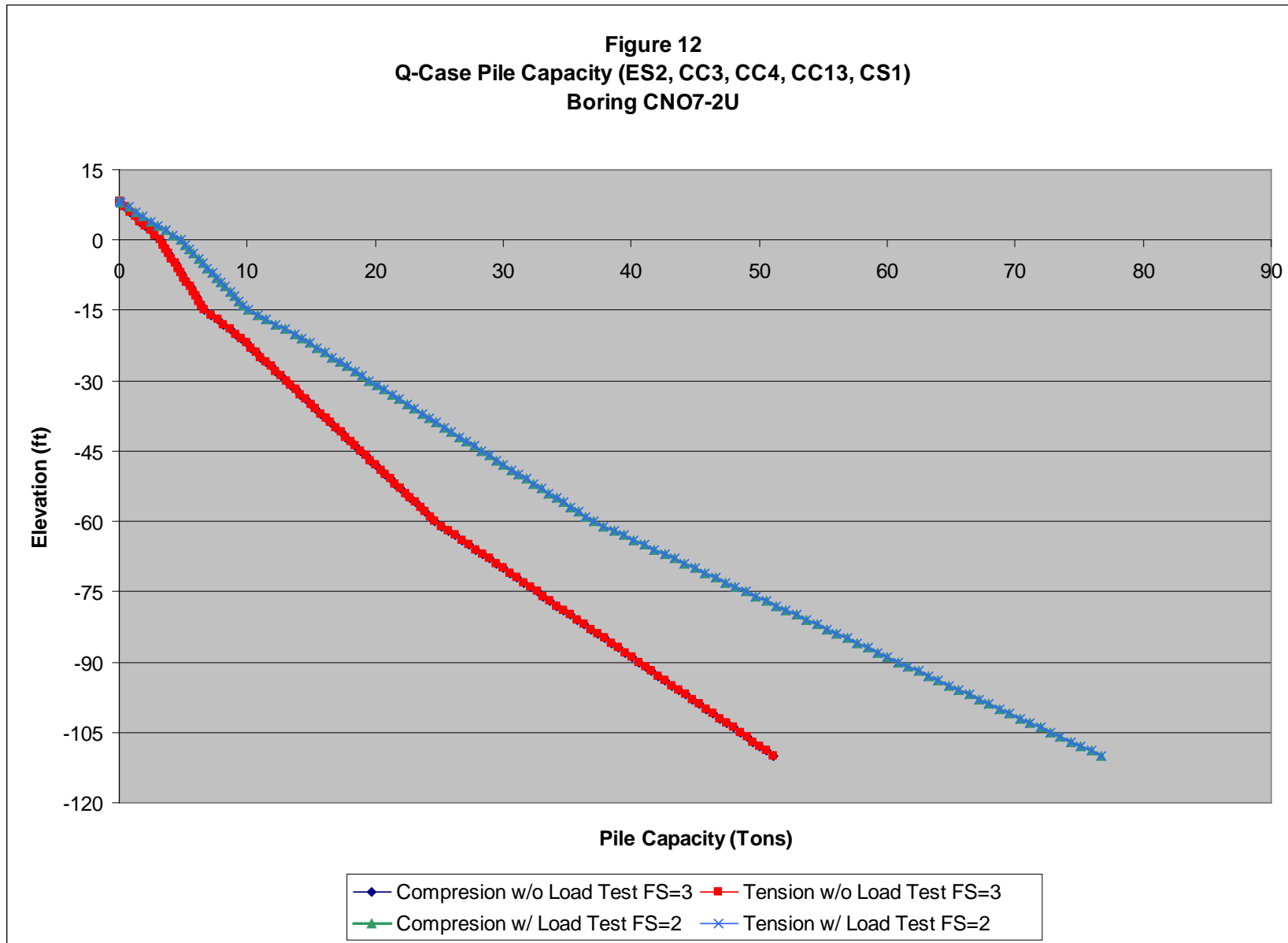


Figure L15 - S-Case Pile Capacity



780

Figure L16 - Q-Case Pile Capacity

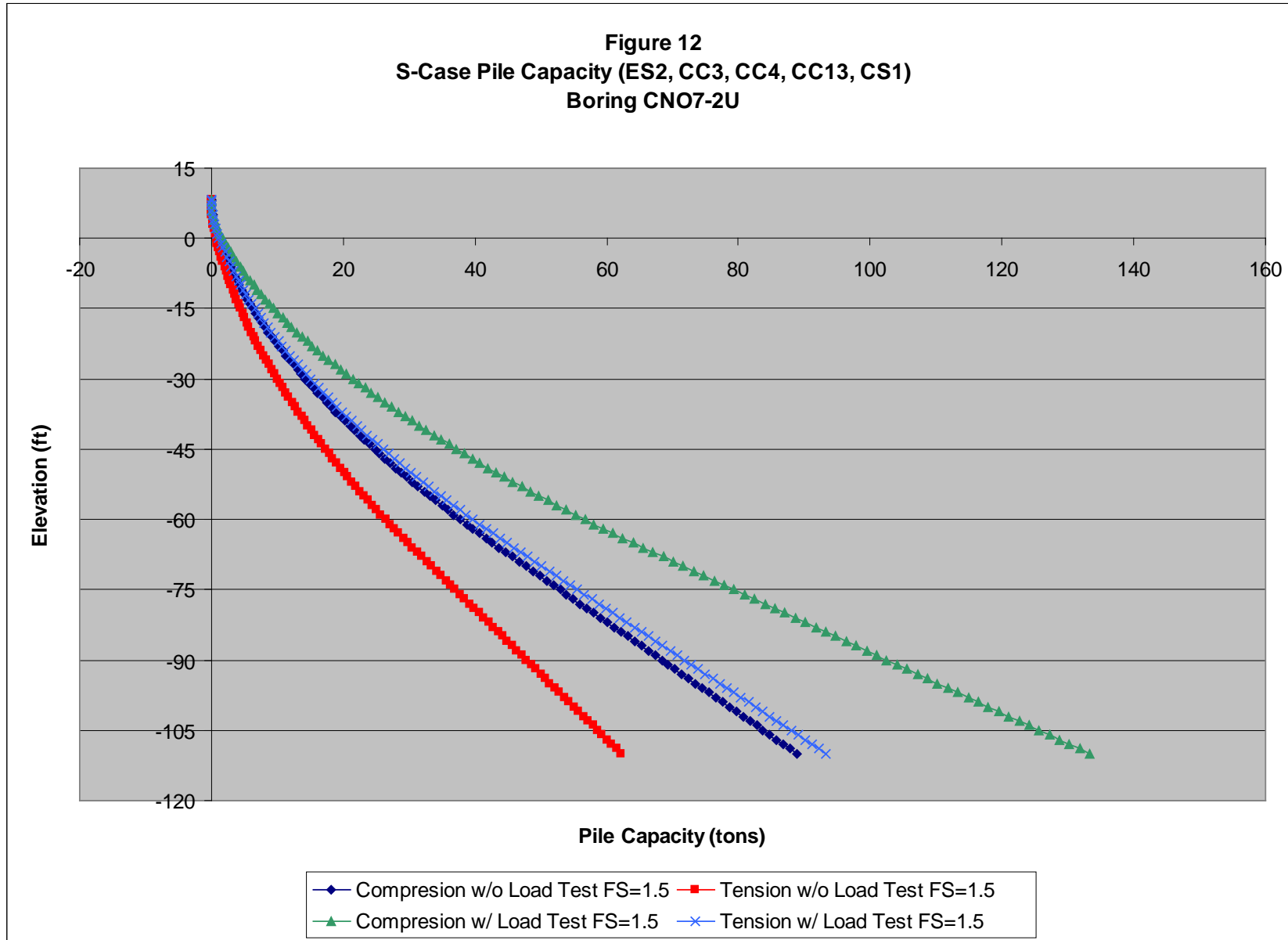


Figure L17 - S-Case Pile Capacity

785 **L6 ENVIRONMENTAL ENGINEERING**

L6.1 HAZARDOUS, TOXIC AND RADIOACTIVE WASTES

As reported in the Phase I ESA, during records research and site reconnaissance it was determined that areas adjacent to some of the project features contained REC's that presented a low to moderate risk of affecting potential project features, albeit that no REC's were noted within direct proximity of land associated with any of the potential project features.

Should at anytime during the project HTRW concerns arise, the CEMVN would take immediate actions to investigate the concerns. Should an HTRW issue be determined and the development of a response action required, CEMVN would coordinate with the appropriate Federal and state authorities to implement an approved response action.

For more information on the Phase I performed for the project see chapters four and five of the main report along with the HTRW appendix, Appendix N.

800

L7 CIVIL DESIGN CRITERIA

L7.1 General

All drawing sheet references in this section can be found in Annex 4 of this Appendix.

805 **L7.1.1 Surveys**

Very little survey information was available for this feasibility design. When available, existing ground elevations and bathymetry was used. If elevations were not available, reasonable assumptions were made based on elevations from similar projects located in the same area.

L7.1.2 Right-of-Way (ROW)

810 Temporary and Permanent ROW areas were estimated based on the feature footprint and construction limit requirements. A Real Estate plan showing actual Temporary and Permanent ROW requirements will be performed at the beginning of the Construction P&S phase.

- 1) Temporary ROW / Construction Limits – Temporary right-of-way will be minimized to reduce damage and mitigation to adjacent areas. Some features will require temporary ROW outside of the feature footprint for construction equipment access and movement.
- 2) Permanent ROW – Some features such as the Structures will require long term maintenance. Permanent ROW may be necessary for access to and around the feature. See the Real Estate section for more information.

820 **L7.2 Feature Designs**

L7.2.1 STRUCTURES

- 1) Site Design – The various structures will require excavation and backfilling. See the structural drawings for more details.
- 2) Construction – See the structural drawings for more details.
- 825 3) Assumptions - It is assumed that access to the sites is available or will be made available through "flotation channels".

L7.2.3 SHORELINE PROTECTION

- 830 1) Site Design – Several locations require the Shoreline Protection feature. This project utilizes Type B protection (see drawings sheet C-339) which consists of a rock berm placed parallel to the existing bank. The top of the berm shall be 5 ft above the elevation of the existing bank. The berm will be made of riprap material to resist wave action.
- 835 2) Construction – Construction of the Shoreline Protection will most likely take place from floating vessels. The rock will be placed from adjacent barges with a track-hoe bucket, by pushing the rock off the barge deck, or with a combination of both.
- 3) Assumptions – It is assumed that access to the sites is available or will be made available through “flotation channels”.

L7.2.4 DREDGED CHANNELS

- 840 1) Site Design – Deepening of existing channels and widening of existing channels will be performed in this project. See drawing C-339 for typical sections. Unless specifically specified, an adjacent berm will be constructed to contain the dredge spoil.
- 845 2) Construction – The dredged channels will be constructed using one of the available dredge types in the area (i.e., cutter head). For Type A Dredged Channels, the width of cut will be 5 ft or more away from the existing bankline to prevent sloughing of the bankline. The spoil area will be constructed in a fashion to add value to the environment (i.e., marsh creation). All exposed ground above the waterline will be seeded to prevent erosion.
- 850 3) Assumptions – Unless otherwise specified, it is assumed that all spoil material will be placed in an adjacent spoil area that is constructed by building berms from in-situ material. If it is determined that the dredge spoil can be used beneficially in other areas, the material will be used for Marsh Creation areas and will be designed and constructed as described in paragraph 7.2.12 (below).

L7.2.5 WEIRS

- 855 1) Site Design – Riprap type material will be placed across the channel approximately 10 ft below the water surface. Side slopes will be 1 vertical to 5 horizontal as shown on drawing C-339.
- 860 2) Construction – The riprap used to construct the weir will be placed with a track-hoe bucket and will most likely be brought to the site via a barge or other floating vessel.
- 3) Assumptions – It is assumed that access to the sites is available or will be made available through “flotation channels”.

L7.2.6 PLUGS

- 865 1) Site Design – Plugs will be constructed with a close-graded aggregate to an elevation 2 ft above the water surface. The plug will be tied-in with 1V:4H slopes extending to the existing ground. See drawing C-340 for a typical section.
- 870 2) Construction – The aggregate will be placed with a track-hoe bucket and will most likely be brought to the site via a barge or other floating vessel.
- 3) Assumptions - It is assumed that access to the sites is available or will be made available through “flotation channels”.

L7.2.7 TERRACING

- 875 1) Site Design – Terracing consists of a series of 10 ft wide parallel berms positioned approximately 90 degrees to the direction of surge. Existing terraced areas were looked at to determine spacing, lengths, and relative areas. See drawing C-341 for details.
- 2) Construction – The terracing berms are constructed by excavating adjacent in-situ material and piling the material until the berm is 2 ft above the water surface. The borrow trench is located a minimum of 25 ft away to prevent sloughing. The exposed ground above the water surface will be vegetated to reduce erosion.
- 880 3) Assumptions – It is assumed that access to the sites is available or will be made available through “flotation channels”.

L7.2.8 CULVERTS

- 885 1) Site Design – The culvert is placed to convey water from one area to the other. Depending upon the location, there may be multiple barrels and/or a flapgate.
- 2) Construction - The area to receive the culvert is excavated to 2 ft below the flowline. The trench is filled with 2 ft of bedding material and backfilled around the pipe.
- 3) Assumptions - It is assumed that access to the sites is available or will be made available through “flotation channels”.

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L7.2.9 REMOVALS

- 1) Site Design – N/A
- 2) Construction – The structures will be removed with equipment such as a track-hoe with a grapple, and the material will be removed off site and disposed at the contractor’s discretion.
- 895 3) Assumptions - It is assumed that access to the sites is available or will be made available through “flotation channels”.

L7.2.10 SPOIL GAP

- 900 1) Site Design – Existing dredge spoil banks will be excavated to allow water conveyance. Gaps will be excavated 50 ft long with 1V on 3H side slopes. See drawing C-342 for details.
- 2) Construction – The gaps will be excavated with track-hoes and sloped back to existing ground. The excavated material will be either hauled off by the contractor or placed adjacent to the gap. All exposed ground above the water surface will be seeded to reduce erosion.
- 905 3) Assumptions - It is assumed that access to the sites is available or will be made available through “flotation channels”.

L7.2.11 MARSH BERM

- 910 1) Site Design – The marsh berm has a 30 ft wide top width. All exposed ground above the water surface will be seeded for erosion protection.
- 2) Construction – The marsh berms will be constructed by borrowing adjacent in-situ material at least 25 ft from the berm toe. The material will be piled until the berm is at an elevation of +2.5.
- 915 3) Assumptions - It is assumed that access to the sites is available or will be made available through “flotation channels”.

L7.2.12 MARSH CREATION (BENEFICIAL USE OF DREDGE CHANNEL SPOIL)

- 920 1) Site Design – Marsh Creation areas consist of a containment berm surrounding the area to be filled. The berm borrow site will be located within the marsh creation area. The fill material borrow site will be located at some distance and location to be determined. Booster pumps and effluent pipes will run between the dredge borrow site and the marsh creation areas.
- 925 2) Construction – A containment berm is first created around the area by excavating in-situ material adjacent to the berm location. The material is piled until the berm is 2 ft above the dredge fill elevation. Spill boxes will be placed in the berm to allow water to drain. The area within the containment berms will be filled with dredge material up to an elevation of +2.5 to +3.0 in one or two lifts. Grading stakes will be placed throughout the areas for monitoring the fill elevations. The containment berm will require constant monitoring and maintenance to ensure no dredge material is allowed to escape.
- 930 3) Assumptions – It is assumed from similar jobs that the construction fill elevation of the dredged material will be +2.5 to +3.0 and will settle over several years to an elevation of approximately +1.0 to +1.5. Once this elevation is achieved, marsh vegetation can either be planted or establish naturally. The containment berms will be degraded to the adjacent dredge fill elevation after settling.

L8 STRUCTURAL DESIGN CRITERIA

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L8.1 FOUNDATION RECOMMENDATIONS

All drawings referenced in this section can be found in Annex 4 to this Appendix.

L8.1.1 GENERAL

940 Development of this proposed diversion project will require various proposed features to accomplish the intended purpose. Among those will be a variety of structures. A description of the foundations for each structural feature will be shown below. The pile founded structures will incorporate the use of steel H-piles and sheet piles, precast prestressed concrete (PPC) piles, and timber piles where indicated on the drawings. Preliminary assumptions of pile sizes, spacing, and

945 pile tip elevations were based on the design of similar structures found in the vicinity. Verification of the pile assumptions, along with any adjustments, was accomplished with the use of pile capacity curves that were developed for similar soils. A more accurate determination of soil properties was not possible due to the absence of reliable borings, therefore pile tip elevations may be adjusted in the next stage of design. All cast-in-place concrete structure

950 monoliths exposed to lateral loadings were analyzed using the COE CASE program “CPGA” (X0080), Pile Group Analysis Program to determine adequacy of pile pattern assumptions. Stability of sheet pile cellular structures was determined thru the use of the COE CASE program “CCELL” (X0040), Analysis of Circular Sheet Pile Cells. Precast concrete box culverts will be soil founded structures supported by a compacted base material of assumed varying thicknesses

955 which will be finalized in the next design stage. All designs were performed in accordance with applicable COE and technical publications, and industry codes. All structures will be constructed using conventional construction equipment and techniques. The contractor will be required to provide dewatering systems (where necessary) in order to construct foundations in a near dry atmosphere. The contractor will also be required to provide a system of shoring or open

960 excavation to safely facilitate construction procedures.

L8.1.2 DESCRIPTION OF FEATURE FOUNDATIONS

- 965 a. Project Feature WS4. – The proposed concrete monolithic structures at this location will be supported on a combination of steel HP14x73 piles, 12 in. x12 in. PPC piles, and 14 in. x14 in. PPC piles. Location, spacing, and pile tip elevations of the piling is shown on drawing S-201. A 4” stabilization slab will be placed between the concrete substructures and the soil foundation to act as a stable working surface during construction. A steel sheet pile seepage cut-off wall will be placed around the perimeter of the concrete substructures. The pile tip elevations of the cut-off walls are shown on drawing S-201.
- 970 b. Project Feature EC3. – The proposed cast-in-place concrete inlet and outlet monoliths of this diversion structure will be supported on 14 in. x14 in. PPC piles. Location, spacing, and pile tip elevations of the piling is shown on drawing S-210. A 4 in. stabilization slab will be placed between the cast-in-place concrete substructures and the soil foundation. A steel sheet pile scour wall will be placed around the perimeter of the inlet and outlet monoliths. The pile tip elevation of the scour walls will be El.-30.0. The precast concrete box culverts located
- 975 between the inlet and outlet monoliths will be supported on a 2.0 ft thick base of compacted material.
- 980 c. Project Feature ES2. – The proposed concrete pumping station, inlet, and outlet structures, bridge and gate structures, and piers for the service access area at this location will be supported on 14 in. x14 in. PPC piles. Location, spacing, and pile tip elevations of the piling is shown on drawing S-330. A 4 in. stabilization slab will be placed between the concrete substructures and the soil foundation. A steel sheet pile scour wall will be placed around the perimeter of the concrete substructures. The pile tip elevation of the scour walls will be El.-45.0.
- 985 d. Project Feature EC5. – The proposed concrete bridge substructure for this feature will be supported on 14 in. x14 in. PPC piles. Location, spacing, and pile tip elevations of the piling is shown on drawing S-140. A 4 in. stabilization slab will be placed between the concrete substructure and the soil foundation. A steel sheet pile scour wall will be placed around the perimeter of the concrete substructure. The pile tip elevation of the scour walls will be El.-
- 990 45.0.
- e. Project Feature WW2. – The proposed steel sheet pile cells and connecting arcs forming a straight line wall at this location will be founded at varying elevations, which are shown on drawing S-220.
- 995 f. Project Feature CC14. – The substructure for this proposed feature will be a combination of steel sheet piling and 12 in. diameter treated timber piles. Since reliable information pertaining to the existing ground elevations is not available, pile tip elevations will be determined in the next design stage.
- 1000 g. Project Feature CC3, CC4, CC13, and CS1. – The proposed cast-in-place concrete inlet and outlet monoliths of this diversion structure will be supported on 14 in. x14 in. PPC piles. Location of the piling is shown on drawings R-301 and R-302. Lateral spacing of the piles will be 6.0 ft on center, and the pile tips will be located at El.-28.0. A 4 in. stabilization slab will be placed between the cast-in-place concrete substructures and the soil foundation. A steel sheet pile scour wall will be placed around the perimeter of the inlet and outlet

1005 monoliths. The pile tip elevation of the scour walls will be El.-30.0. The precast concrete box culverts located between the inlet and outlet monoliths will be supported on a 3.0 ft thick base of compacted material. The steel sheet pile wingwalls connected to the inlet and outlet monoliths will extend from a top elevation of +4.0 to a tip elevation of -30.0.

1010 h. Project Feature CC15. – The substructure for this proposed feature will be a combination of treated timber piles and treated 2 in. x12 in. timber sheeting. Pile tip elevations could not be determined at this time due to the lack of reliable existing ground elevations.

L8.2 STRUCTURAL DESIGN FOR DIVERSION FACILITY

L8.2.1 GENERAL

1015 The general physical configuration of structures for this proposed diversion project were based on a variety of considerations, among them hydraulic requirements, similar structures performing the same function, and utilizing existing designs from other projects. Two types of box culverts are being presented in this project. The first type constructed of reinforced cast-in-place concrete, and the other reinforced precast concrete. All other concrete structures will be reinforced and
1020 cast-in-place, except the bridge beams for the highway, and trash handling bridges which will be PPC beams. Concrete member sizes were assumed based on similar structures of equivalent size with similar loadings, therefore, no stress analyses were performed in this design phase.

L8.2.2 DESCRIPTION OF STRUCTURAL FEATURES

1025 a. Project Feature WS4. – The proposed structures at this location will be a series of reinforced cast-in-place concrete box culverts constructed monolithically in conjunction with inflow, roller gate, bulkhead, and outflow monoliths. These structures will be located under an existing earth levee. There will be six box culvert barrels, each 15 ft high and 15 ft wide
1030 (inside dimensions). The flow line elevation inside the barrels will be El.-15.0. The box culverts base slab will be 4.0 ft thick, the top slab will be 3.0 ft thick, the interior vertical walls will be 2.5 ft thick, and the exterior vertical walls will be 3.0 ft thick. The length of the box culverts will be 120.0 ft. The concrete inflow monoliths on the upstream end of the structure will be comprised of a 4.0 ft thick base slab and two 3.0 ft thick vertical guidewalls providing a length of 90.0 ft. The roller gate monolith will be 60.0 ft long and 108.5 ft wide.
1035 The concrete bulkhead and outflow monoliths on the downstream end of the structure will also be comprised of a 4.0 ft thick base slab and two 3.0 ft thick vertical guidewalls providing a length of 95.0 ft and a width of 108.5 ft. The inflow channel bottom will be El.-15.0, with a width of 108.5 ft and side slopes of 1 vert. on 3 horiz. The total width of the concrete structure will be 108.5 ft. The outflow channel bottom will be El.-19.0, with a
1040 width of 108.5 ft and side slopes of 1 vert. on 3 horiz. Vertical slots and structural steel roller guides will be provided in the concrete walls at each end of the barrels for the placement of a bulkhead, when required. A 15 ft high and 15 ft wide fabricated structural steel roller gate will be located at the upstream end of each barrel. A concrete platform will be located at El.+21.0 to support the roller gate operators. A machinery building will be located adjacent
1045 to the support platform, also at El.+21.0. A 2.0 ft thick vertical concrete seepage cut-off wall extending from the top of the box culverts to El.+17.0 will be located on the roller gate monolith near the centerline of the earth levee. A 17.0 ft wide and 34.0 ft long timber pile

supported concrete bulkhead storage slab will be located on the landside of the levee. This feature can be seen in drawings S-101 and S-201 located in Annex 4 to this Appendix.

- 1050 b. Project Feature EC3. – The proposed structures at this location will be a series of precast reinforced concrete box culverts placed individually, with a common cast-in-place reinforced concrete inflow monolith located at each end of the culverts. This structure will be located under an existing earth levee. There will be ten box culverts placed parallel to each other, and spaced at 10.0 ft on center. Each culvert will be 5 ft high and 5 ft wide (inside dimensions).
1055 The flow line elevation inside each culvert will be El.-5.0. The flow line elevation inside each culvert will be El.-5.0. The thickness of the base slab, top slab, and side walls will be determined during detailed design using the applicable requirements of ASTM C 1433 and this design will be confirmed with a precast fabricator. The approximate length of the culverts will be 48.0 ft. The concrete inflow monoliths will be comprised of a 1.5 ft thick base slab, with 1.5 ft thick vertical divider walls located on each side of the culverts. A steel sheet pile wingwall will be connected to the vertical walls at each end of the monolith to retain the embankment. The inflow monolith adjacent to the Grand Bayou will be 15.0 ft long and 101.5 ft wide, and the inflow monolith at the other end of the culverts will be 13.0 ft long and 101.5 ft wide. A 1.5 ft thick vertical headwall will be located at each end of the culverts. The channel bottom at the Grand Bayou end of the structure will be El.-5.5, and the channel bottom at the other end will be El.-5.0. The channel bottom width at both ends will be 111.5 ft with side slopes of 1 vert. on 3 horiz. Slots will be provided in the divider walls at each end of the structure for the placement of stoplog needles, when required. A 5 ft high and 5 ft wide flap valve will be attached to the Grand Bayou end of each culvert. Since operation of this structure considers flows in both directions, provisions will be made to retain the flap valves in a full open position. Slots will also be provided in the divider walls at the structure end opposite the Grand Bayou for the placement of an adjustable weir. A 10.0 ft wide and 10.0 ft long timber pile supported concrete dewatering needles storage slab will be located on the Grand Bayou side of the levee. This feature can be seen in drawings S-110 and S-210
1060 located in Annex 4 to this Appendix.
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- c. Project Feature ES2. – This proposed feature will be a combination of multiple structures. A proposed channel thru this location will provide a channel bottom width of 470.0 ft with an elevation of El.-14.0. Side slopes of the channel will be 1 vert. on 4 horiz. A new highway bridge will be constructed to span the channel. A stormwater pumping station will be constructed adjacent to the bridge, and located on the centerline of the channel. A series of pneumatic spillway gates will be constructed in conjunction with the bridge substructure on each side of the pumping station. This feature can be seen in drawings S-130, S-230, and S-330 located in Annex 4 to this Appendix.
- 1080
1085 1) Pumping Station – The proposed pumping station will be located on the upstream side of the highway bridge. This structure will be made up of the following four parts; the structure containing the pumps, the inlet structure containing the trash racks and trash handling bridge, the outlet structure which will also be the highway bridge footing, and the service access and parking area. The cast-in-place reinforced concrete pump structure will be 93.0 ft long and 160.0 ft wide. The flow line elevation of the pump intake ports will be El.-14.0, and the flow line elevation of the discharge outlets will be El.-12.5. The founding elevation of the pump structure will be El.-21.0. Sluice gates and gate operators
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will be provided for the intake ports at the upstream end of the structure, and for the discharge outlets at the downstream end. Two 9 ft high and 15 ft wide access tunnels will be located in the substructure, extending the width of the structure. The floor elevation of the upper tunnel will be El.+9.0, and the floor elevation of the lower tunnel will be El.-2.0. The operating floor elevation will be El.+23.0. An 80.0 ft wide rigid frame steel building, extending the width of the pump structure, will be provided. The building will extend vertically from the operating floor to El.+60.0. A bridge crane will be located inside the building and will be sized during a later design phase based on the weight of the pump and prime mover. A 6 ft wide walkway at El.+29.0 will be located on the downstream side of the building, and extend the width of the pump structure. The cast-in-place reinforced concrete inlet structure containing the structural steel trash racks will be 55.0 ft long and 160.0 ft wide. The flow line elevation of the inlet structure will be El.-14.0, and the founding elevation will be El.-21.0. A trash raking machine will be utilized to remove trash from the trash racks. A 26.0 ft wide vehicle bridge at El.+15.0, extending the width of the inlet structure, will be located adjacent to the trash racks to provide access for trash removal. The bridge will be comprised of a reinforced concrete deck supported by 5-27 in. PPC beams spaced at 5.5 ft on center. Slots will be provided in the upstream end of the 4.0 ft thick vertical divider walls located on the inlet structure for the placement of stop logs, when required. The cast-in-place reinforced concrete outlet structure, which also serves as the footing for the proposed highway bridge piers, will be 40.0 ft long and extend the width of the pump structure. The flow line elevation of the outlet structure will be El.-14.0, and the founding elevation will be El.-18.0. The service access and parking area, located adjacent to the East end of the pump structure, will be 40.0 ft wide and approximately 124.0 ft long. The area will be a reinforced concrete deck at El.+15.0 supported by 36 in. PPC beams spaced at 7.0 ft on center. The beams will be supported at the pump structure on concrete haunches. The other end of the PPC beams will be supported on a continuous cast-in-place reinforced concrete beam which will be supported on 2.5 ft diameter concrete columns spaced at 20.0 ft on center. The columns will be resting on 10.0 ft square concrete footings. The footings will be 4.0 ft thick and founded at El.-21.0.

2) Highway Bridge – The centerline of the proposed highway bridge will be located approximately 20.0 ft downstream from and parallel to the pumping station. The bridge will span the proposed 470.0 ft wide channel. The roadway width will be 32.0 ft, assuming two 12.0 ft wide driving lanes and two 4.0 ft wide shoulders. A concrete barrier wall and aluminum single tube guardrail will be placed on each side of the roadway. The roadway deck will be cast-in-place reinforced concrete with a top El.15.0. The approximate length of the bridge deck between the end abutments will be 552.0 ft. The bridge deck will be supported with 5-36 in. PPC beams spaced at 7.0 ft on center. The end abutments will be cast-in-place reinforced concrete with turned back wings, and supported with a 3.0 ft thick foundation wall. The wall will rest on a 4.0 ft thick common footing which will also support the intermediate piers and spillway gates. The 3.0 ft thick intermediate piers will be 40.0 ft long and extend from a top elevation of approx. El.11.0 to the common footing, which will be founded at El.-18.0. The span lengths between piers are shown on drawing S-130.

- 1140 3) Pneumatic Spillway Gates – The proposed spillway gates will be located within the
confines of the first two highway bridge spans on each side of the pumping station.
Design and operation of the gates will be similar to those produced by Obermeyer Hydro,
Inc. The 10 ft wide gates are raised and lowered by means of air bladders, controlled with
1145 compressed air. The 83.0 ft bridge spans will contain eight gates, and the 63.0 ft bridge
spans will contain six. The gates will be hinged at the bottom to a common 4.0 ft thick
concrete foundation, 40.0 ft wide and the length of the highway bridge. The hinge point
will be approx. El.-14.0, and the top of the gate will terminate at El.+6.0. The air bladders
provided will be in 20.0 ft lengths. Restraining straps will be used to ensure the gates stay
1150 within their intended operating range. UHMW polyethylene abutment plates are attached
to the bridge piers to provide a rubbing surface and seal for the gates as recommended by
the gate manufacturer. The first bridge spans adjacent to the abutments will utilize a 2.0 ft
thick concrete wall oriented parallel to the bridge, in lieu of the spillway gates. The top of
the walls will be at El.+6.0 and rest on the common foundation at El.-14.0.
- 1150 d. Project Feature EC5. – This proposed feature will be a combination of two structures. A
proposed channel thru this location will provide a channel bottom width of 470.0 ft with an
elevation of El.-14.0. Side slopes of the channel will be 1 vert. on 4 horiz. A new highway
bridge will be constructed to span the channel, and a series of pneumatic spillway gates will
1155 be constructed in conjunction with the bridge substructure. This feature can be seen in
drawings S-140 and S-240 located in Annex 4 to this Appendix.
- 1160 1) Highway Bridge – The centerline of the proposed highway bridge will be oriented normal
to the centerline of the proposed channel, and span the overall width of the channel. The
roadway width will be 32.0 ft, assuming two 12.0 ft wide driving lanes and two 4.0 ft
wide shoulders. A concrete barrier wall and aluminum single tube guardrail will be
placed on each side of the roadway. The roadway deck will be cast-in-place reinforced
concrete with a top El.15.0. The approx. length of the bridge deck between the end
abutments will be 552.0 ft. The bridge deck will be supported with 5-36 in. PPC beams
spaced at 7.0 ft on center. The end abutments will be cast-in-place reinforced concrete
1165 with turned back wings, and supported with a 3.0 ft thick foundation wall. The wall will
rest on a 4.0 ft thick common footing which will also support the intermediate piers and
spillway gates. The 3.0 ft thick intermediate piers will be 40.0 ft long and extend from a
top elevation of approx. El.11.0 to the common footing, which will be founded at El.-
18.0. The span lengths between piers are shown on drawing S-140
- 1170 2) Pneumatic Spillway Gates – The proposed spillway gates will be located within the
confines of all five 83.0 ft bridge spans. Design and operation of the gates will be similar
to those produced by Obermeyer Hydro, Inc. The 10 ft wide gates are raised and lowered
by means of air bladders, controlled with compressed air. Each 83.0 ft bridge span will
contain eight gates. The gates will be hinged at the bottom to a common 4.0 ft thick
concrete foundation, 40.0 ft wide and the length of the highway bridge. The hinge point
1175 will be approx. El.-14.0, and the top of the gate will terminate at El.6.0. The air bladders
provided will be in 20,0 ft lengths. Restraining straps will be used to ensure the gates
remain within their intended operating range. UHMW polyethylene abutment plates are
attached to the bridge piers to provide a rubbing surface and seal for the gates. The first
bridge spans adjacent to the abutments will utilize a 2.0 ft thick concrete wall oriented

- 1180 parallel to the bridge, in lieu of the spillway gates. The top of the walls will be at El.+6.0
and rest on the common foundation at El.-14.0.
- e. Project Feature WW2. – The proposed structure at this location will be a water control
structure oriented normal to the centerline of the existing waterway. The structure will
require the placement of twenty circular steel sheet pile cells and eighteen steel sheet pile
1185 connecting arc cells, and the placement of a rock weir. Construction of this feature will
provide a straight line structure with a 100.0 ft opening in the sheet pile cell wall. The
opening will allow the passage of watercraft. A rock weir will be placed in the opening with
a top elevation of El.-12.0, and side slopes of 1 vert. on 2 horiz.. The circular cells will have
a diameter of 37.5 ft spaced at 42.5 ft on center, and a top elevation of El.+3.0. The
1190 connecting arc cells will have a radius of approx. 11.0 ft and a top elevation of El.+3.0. Total
length of the proposed structure will be 940.0 ft. Design of the circular and connecting arc
cells was based on the assumption that PS27.5 sheet piles will be used. Determination of
factors of safety of the cell failure modes were calculated with the use of the COE CASE
program “CCELL” (X0040). All circular and connecting arc cells will be filled with sand or
1195 shells. This feature can be seen in drawing S-220 located in Annex 4 to this Appendix.
- f. Project Feature CC14. – The proposed structure at this location will also be a water
control structure oriented normal to the centerline of the existing waterway. A straight line
wall will be constructed in the waterway with the use of 12 in. diameter treated timber piling,
and steel sheet piling. The timber piling will be spaced at 5.0 ft on center, and provide a
1200 cluster of piles at each location. The cluster will have one vertical pile, and two battered
away from the wall. 20.0 ft long sheet pile will be placed between the vertical timber piles. A
4.0 ft wide timber walkway will be located on top of the battered piles and extend the total
length of the structure. Three fabricated structural steel boxlike enclosures will be attached at
the center of the structure. A 4.0 ft square flap valve will be attached to each enclosure on the
1205 same side. An adjustable weir will be attached to each enclosure on the opposite side. Timber
stoplogs will be used to adjust the water level at the weir. The top elevation and total length
of the proposed structure will be determined in the next design phase, since reliable
topography information was not available at this time. This feature can be seen in drawing S-
220 located in Annex 4 to this Appendix.
- g. Project Features CC3, CC4, CC13, and CS1. – The proposed structures at these locations will
be a series of precast reinforced concrete box culverts placed individually, with a common
cast-in-place reinforced concrete inflow monolith on the upstream end of the culverts and a
common cast-in-place reinforced concrete outflow monolith on the downstream end of the
culverts. These structures will be located under existing roadways. There will be six culverts
1215 placed parallel to each other, and spaced at 14.0 ft on center. Each culvert will be 10 ft high
and 10 ft wide (inside dimensions). The flow line elevation inside each culvert will be El.-
10.0. The thickness of the base slab, top slab, and side walls will be determined at a later time
when a specific fabricator for the precast culverts is considered. The approx. length of the
culverts will be 100.0 ft. The concrete inflow monoliths will be comprised of a 2.5 ft thick
1220 base slab with 3.0 ft thick vertical divider walls located on each side of the culverts. A steel
sheet pile wingwall will be connected to the vertical walls at each end of the monoliths to
retain the embankment. The inflow monoliths will be 53.0 ft long, and the outflow monoliths
will be 22.0 ft long. A 1.5 ft thick vertical headwall will be located at each end of the

1225 culverts. The flow line elevation of the inflow and outflow monoliths will be El.-10.0. The
channel bottom at each end of the structures will be El.-10.0. Slots will be provided in the
divider walls at each end of the inflow and outflow structures for the placement of stoplog
needles, when required. Galvanized structural steel trash racks will be placed between the
divider walls of the inflow monoliths for all four structures. Trash handling access bridges
1230 will be constructed adjacent to the trash racks. The bridges which will be constructed with
precast concrete deck slabs, will allow for a 15.0 ft wide vehicle clearance. A 4 ft wide
galvanized steel walkway and vehicle guardrail will be erected parallel and adjacent to the
bridges. A 10 ft square sluice gate and gate operator will be installed on a concrete headwall
at the inlet end of all culverts. Reinforced concrete platforms will be constructed at the inlet
end of all culverts to support the sluice gate operators. The top of the platforms will be
1235 El.+13.5. A 15.0 ft wide and 10.0 ft long timber pile supported concrete slab for dewatering
needles storage will be located near the inflow end of all four structures. The handling of
traffic during construction has not been addressed at this time, therefore it has not been
determined whether a temporary road relocation will be necessary at each or any of the four
structure locations. These features can be seen in drawings R101, R301 and R302 located in
1240 Annex 4 to this Appendix.

h. Project Feature CC15. – The proposed structure at this location will be a water control
structure oriented normal to the centerline of the existing waterway. A straight line wall with
three openings (boat bays) will be constructed in the waterway with the use of 12 in.
diameter treated timber piling, and 2 in. x12 in. treated timber sheeting. The timber piling
1245 will be spaced at 8.0 ft on center, and provide a pair of piles at each location. The pair will be
one vertical, and one battered away from the wall. The vertical piling will be 40.0 ft long,
and the battered piling will be 45.0 ft long. Two layers of 2 in x12 in sheeting will be placed
vertically and parallel to the vertical piling, for the entire length of the proposed structure.
The piling and sheeting will be separated with 10 in x 10 in treated timber wales attached
1250 horizontally. The three boat bay openings will be centered on the waterway, and will allow
the passage of watercraft. The clear openings of the boat bays will be 7.0 ft horizontal, and
approx. 5.0 ft vertically. The top elevation and total length of the proposed structure will be
determined in the next design phase, since reliable topography information was not available
at this time.

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L9 ELECTRICAL AND MECHANICAL REQUIREMENTS

L9.1.1 ELECTRICAL SOURCES AND SUPPLY REQUIREMENTS

GENERAL

1260 Development of this proposed diversion project will require various proposed structural features
to accomplish the intended purpose. Specific structural features will require an electrical power
source depending on the operational requirements at each site. The ability to furnish electrical
power to each structural feature from an offsite location has not been determined at this time, and
will be investigated in another design stage. The possible electrical requirements at each feature
1265 site have been presented below.

L9.1.2 ELECTRICAL REQUIREMENTS PER SITE

- 1270 a. Project Feature WS4. – An electrical power supply will be required to operate the roller gate operators. Whether the operators will be electrically or hydraulically operated has not been determined at this time. In either case an electrical power source will be required for the operator motors, or for the electrical motors driving the hydraulic pumps for the operators. In addition, a power source will be required for the machinery building lighting, and switchboard equipment in the building.
- 1275 b. Project Feature ES2. – An electrical power supply will be required to operate the intake, and discharge sluice gate operators located inside the pumping station. Whether the operators will be electrically or hydraulically operated has not been determined at this time. In either case an electrical power source will be required for the operator motors, or for the electrical motors driving the hydraulic pumps for the operators. In addition, a power source will be required to operate the trash raking machine located on the inlet structure for the pumping station. An electrical power source will also be required inside the building located on top of the pumping station for the lighting, exhaust fans, and bridge crane. The air compressor and electrical control panel for the pneumatic spillway gates may be located inside the pumping station building, and if that is the case a permanent electrical power source will be required. Otherwise, a portable air compressor could be utilized.
- 1280 c. Project Feature EC5. – The electrical control panel and air lines termination point for the pneumatic spillway gates will be located near one of the highway bridge abutments. It has been assumed a portable air compressor will be used to activate the spillway gates, and therefore an offsite electrical power supply will not be required.
- 1285 d. Project Features CC3, CC4, CC13, and CS1. – An electrical power supply will be required to operate the sluice gate operators. Whether the operators will be electrically or hydraulically operated has not been determined at this time. In either case an electrical power source will be required for the operator motors, or for the electrical motors driving the hydraulic pumps for the operators. In addition, a power source will be required for the control house building, lighting and switchboard equipment.
- 1290 e. Electric Power Source(s). – Electric power source(s) can be either commercial utility electric power or diesel engine generators. Location of commercial utility power and the cost to supply this power will be compared to the cost of a diesel engine generator set, including estimated O&M costs to determine the recommended source of the required electrical power.
- 1295

1300 **L9.2 SOLAR POWER SUPPLY SYSTEMS**

L9.2.1 GENERAL

At this phase of the design it has been determined no structural features will incorporate a solar power supply system.

1305 **L9.3 ELECTRICAL AND MECHANICAL DESIGN FOR DIVERSION FACILITY**

L9.3.1 GENERAL

The size and type of electrical and mechanical components for the project features were selected based on a variety of considerations, among them hydraulic requirements, similar features performing the same function, and utilizing existing designs from other projects.

1310

L9.3.2 ELECTRICAL/MECHANICAL REQUIREMENTS PER SITE

- 1315 a. Project Feature WS4. – Regulation of flow thru the culverts will be controlled with the use of six 15’x15’ fabricated structural steel roller gates. The gates will be raised/lowered with the use of a gate hoist supplied by a known and acceptable gate manufacturer. Selection of either electric motor operated or hydraulically operated gate hoists will be determined in a later project design stage. Two fabricated structural steel bulkheads approximately 15 ft square will be provided and stored on site when not in use. The bulkheads will be fitted with rollers, and vertical steel roller guides will be cast in slots in the concrete walls.
- 1320 b. Project Feature ES2. – Regulation of flow in the proposed channel at this site will be controlled with the combined use of a stormwater pumping station, and pneumatic spillway gates.
- 1325 1) Pumping Station - The pump station will utilize ten 9’ wide by 8’ high cast iron sluice gates at the intake end, and five 10’x10’ cast iron sluice gates at the discharge end of the structure. The gates will be raised/lowered with the use of a gate hoist supplied by the sluice gate manufacturer. The gates will be mounted to a cast iron wall thimble cast in the concrete. The gates located at the intake end of the structure will be positioned to provide a flush bottom closure, and the gates located at the discharge end will be positioned to provide a raised bottom closure. Selection of either electric motor operated or hydraulically operated gate hoists will be determined in a later project design stage. The pumping station will be provided with five vertical lineshaft stormwater pumps. Each pump will provide a rated capacity of 800 cubic ft per second. Each pump will be driven with an individual diesel engine thru a right angle speed reducer. The diesel engine exhaust system will extend outside the steel building enclosure. In addition, exhaust fans will be provided in the building. A rail mounted bridge crane with a movable trolley will also be provided inside the building. The lifting capacity of the bridge crane will be determined at a later time when requirements are known. A mechanical trash rake will be provided at the concrete intake structure to remove trash and debris from trash racks. The mechanical rake will be on a movable trolley extending the width of the trash racks.
- 1330
- 1335
- 1340 2) Pneumatic Spillway Gates – Operation of the gates requires a compressed air source. Distribution of the compressed air to the gate bladders will be attained thru stainless steel pneumatic pipes embedded in the concrete structure supporting them. All the pipes will terminate at one central location where the compressed air source is introduced, which will be either a portable or permanently installed air compressor. Location of the air compressor will be finalized at a later time.
- 1345 c. Project Feature EC5. – Regulation of flow in the proposed channel at this site will be controlled with pneumatic spillway gates. Operation of the gates requires a compressed air source. Distribution of the compressed air to the gate bladders will be attained thru stainless steel pneumatic pipes embedded in the concrete structure supporting them. All the pipes will terminate at one central location where the compressed air source is introduced, which will be either a portable or permanently installed air compressor. Location of the air compressor will be finalized at a later time.
- 1350
- d. Project Features CC3, CC4, CC13, and CS1. – Regulation of flow thru the culverts at each of these sites will be controlled with the use of six 10’x10’ cast iron sluice gates. The gates will

1355 be raised/lowered with the use of a gate hoist supplied by the sluice gate manufacturer. The gates will be mounted to a cast iron wall thimble cast in the concrete, and positioned to provide a flush bottom closure. Selection of either electric motor operated or hydraulically operated gate hoists will be determined in a later project design stage.

L10 OPERATIONS AND MAINTENANCE

1360 All features were considered for Operational Cost and Maintenance Cost. Items that require painting, periodic inspections and debris removal were considered features that will have annual cost to them and have been priced accordingly. Features that consist of dredging or berm type work are considered as having no maintenance cost.

1365 Operation of the HNC lock and sector gate will involve closure of the sector gate year round. Normal vessel traffic will pass through the lock. A few times each year, large vessels that will not fit in the lock will need to pass through the structure. These vessels will schedule openings of the sector gate portion of the structure. After the vessel passes, the sector gates will again be closed. The sluice gates located within the HNC lock structure will be open year round with the
1370 exception of storm event conditions.

For the purposes of benefits analysis, all structures, with the exception of the HNC Lock, were assumed to be in the open position year-round.

L11 COST ESTIMATES

L11.1 Basis of Cost Estimate

1375 Two types of estimates were developed for this study, a preliminary cost estimate and a detailed cost estimate. The preliminary cost estimate for this Feasibility Study is based upon unit price method. The detailed cost estimate has been developed for all features identified in each of the
1380 study alternatives. Most of the construction quantities and estimates, for both type of estimates, were based upon historical costing data for this area and have been developed using the most recent and accurate information available. This data was developed specifically for this area and for this type of construction practice. The unit price cost estimates have been developed for all features identified in each study alternative. The detailed cost estimate is based upon developed
1385 crews of equipment and labor for only new type construction for the base year of 2010. The unit price estimates are based on the current design concepts, data and quantities for each study alternative, and site information available to date. Unit pricing for both types of estimates, come from data that was developed from using the most recent cost information for similar type construction.

1390

L11.1.1 Equipment Cost

The equipment cost, used in the detailed estimate to perform this work is specific for this area and for this type of construction practice

L11.1.2 Labor Cost

1395 The labor rates used in the detailed estimate to perform this work are specific for this type of construction and specifically for the State of Louisiana.

L11.1.3 Dredge Cost

1400 All dredging cost used in this study were developed from previous information and data, from previous dredging contracts from this area. Costs for the preliminary type estimate were developed from unit price method from previous dredging contracts. Costs for the detailed estimates were developed by the CEDEP calculation method.

1405 **L11.1.4 Estimate Documents**

The preliminary cost estimates for the various components were developed from conceptual exhibits, schematics, sketches and in limited cases, from drawings for the components. Many of the exhibits were GIS-based and should be considered less accurate than detailed construction plans.

1410 **L11.1.5 Areas of Consideration for Cost**

The estimate includes considerations for the following factors:

- Permanent constructed facility
- Ancillary site improvements, such as driveway, drainage, fencing, security, lighting, etc.
- 1415 • Existing Site Conditions
- Construction Access
- Construction Techniques
- Major construction facilities
- Temporary Road Relocations and detours
- 1420 • Temporary railroad Relocations
- Pipeline Relocations
- Utility Adjustments and Relocations
- Construction Sequences
- Storm Water pollution
- 1425 • Dewatering and water management
- Net Earth Quantities
- Availability of Material

1430 **L11.2 Contingencies**

Contingencies of 39% were used for all features identified in each study alternative for the preliminary cost estimates. These estimates were used for alternatives comparison.

1435 Contingencies for the MCACES detailed estimate are based on a Cost and Schedule Risk Analysis completed using Crystal Ball software. This analysis resulted in a 34% contingency, which was used for the MCACES detailed estimate performed on the RP. Further discussion of the Cost and Schedule Risk Analysis are discussed in the Risk Analysis Section below.

L11.3 Planning, Engineering and Design

1440 A rate of 10 percent was applied for all features identified in each study alternative that contained construction type activity. This percentage is based on MVN's average cost for planning, engineering and design for a Feasibility Report.

L11.4 Construction Management

1445 A rate of 8 percent applied to all features identified in each study alternative that contained construction type activity. This percentage is based on MVN’s average expenditures for construction management on a typical contract of this magnitude.

L11.5 Detailed Estimate

1450 A detailed cost estimate was performed on the Recommended Plan (RP). The detailed cost was developed using MCACES program as required under MVD direction. Included in the estimate is the category for ‘Lands and Damages’, ‘Cultural Resource Preservation’, ‘Planning, Engineering and Design’ and ‘Construction Management’. All construction type work is divided in the Relocations, Roads, Railroads and Bridges, Channels and Canals, Floodway Control & Diversion Structures and Bank Stabilization. This estimate can be seen in Volume III Appendix O.

L11.6 Risk Analysis

1460 A cost risk analysis was performed for this project in accordance with ER 1110-2-1302 paragraph 7.3.2 and ER 1110-2-1302, Appendix P, paragraph 4. The results of the cost risk analysis are shown in the Project Cost and Schedule Risk Analysis Report included in Volume III Appendix P.

L12 CONSTRUCTION SEQUENCE

L12.1 General

1465 The anticipated construction schedule for the RP is shown in Figure L18 and Figure L19. This schedule assumes a two year preconstruction engineering and design period and a five year construction period. Dates on the schedule are for demonstration purposes and do not represent actual planned dates. The schedule assumes that various types of work will be performed concurrently. This schedule may be shortened by adding crews for more concurrent construction.

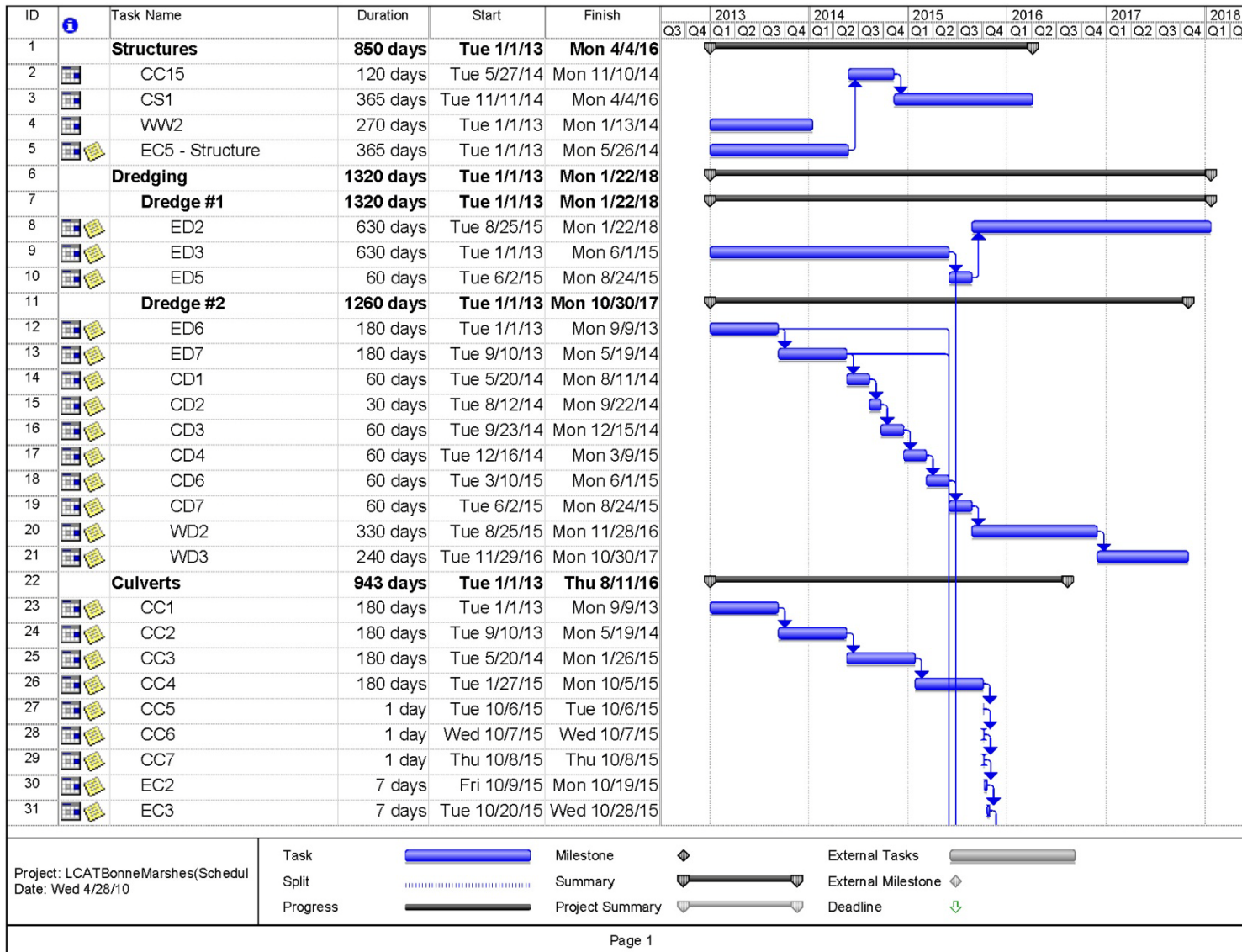
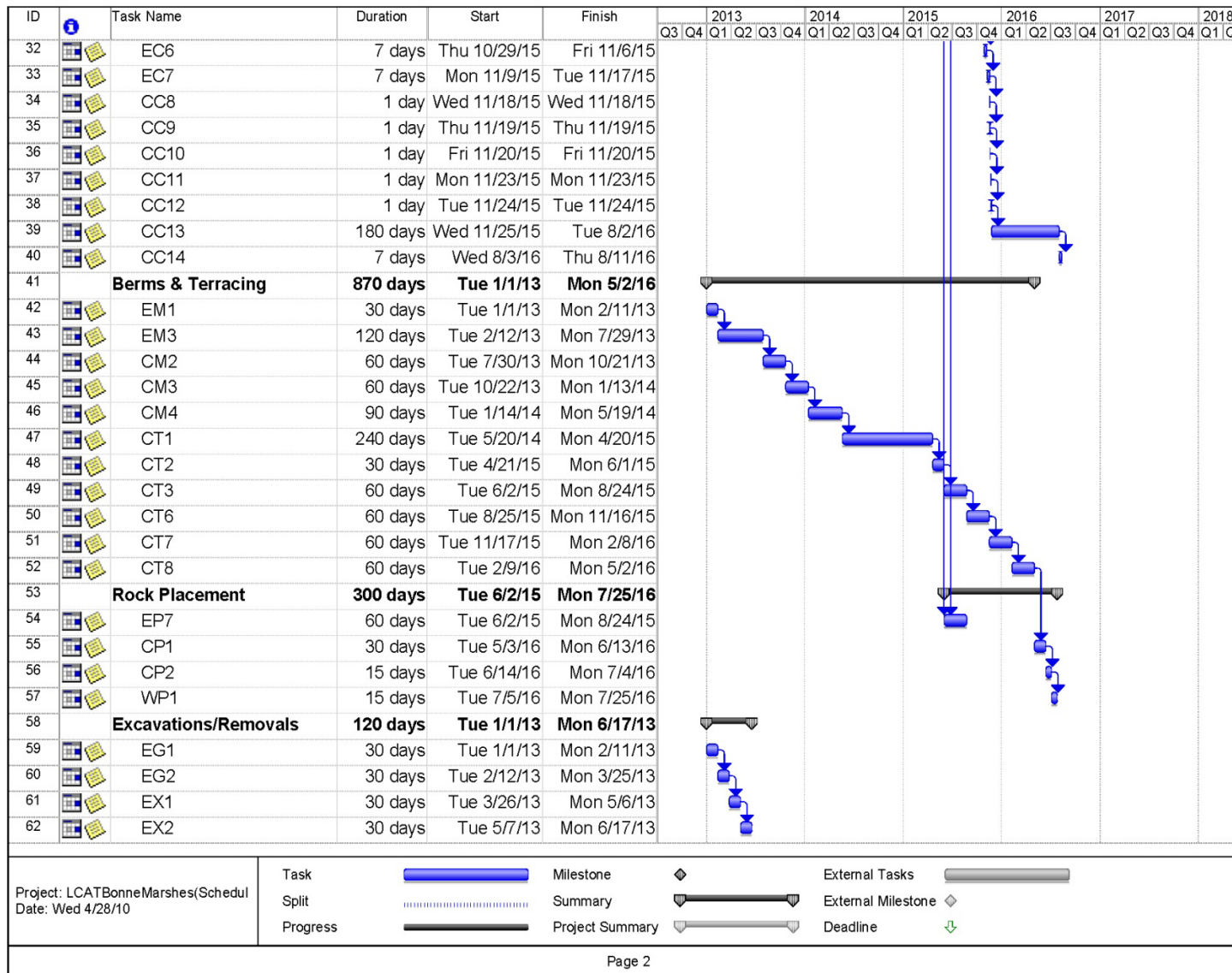


Figure L18 - Construction Schedule



1475

Figure L19 - Construction Schedule (Page 2)

L13 ANNEXES

1480 Annexes are provided as separate document.

Annex 1 – Detailed Construction Cost Estimates for Studied Alternatives

Annex 2 – Detailed Hydraulic Modeling Studies

Annex 3 – Hydraulic Model Sensitivity Analysis Report

1485 Annex 4 – Engineering Drawings

Annex 5 – Total Project Cost Summary

Annex 6 – Reserved

Annex 7 – Reserved

Annex 8 – Reserved

1490