Attachment 2: 1980's Feasibility Report

Geotechnical Appendix

APPENDIX C

GEOTECHNICAL INVESTIGATION
McCOOK LEVEE, McCOOK, ILLINOIS
FEASIBILITY STUDY

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APPENDIX C

GEOTECHNICAL INVESTIGATION
McCOOK LEVEE

GENERAL SITE DESCRIPTION

1. The Des Plaines River flows southward to the town of McCook and then curves southwestward along the southeast boundary of the city. McCook levee parallels the river in McCook. The river flows south from near the Wisconsin-Illinois border between moraines of the Valparaiso Moraine System until it reaches it's confluence with the Lake Chicago outlet channel which it then follows to the Illinois River. The river parallels the Chicago Sanitary and Ship Canal which both occupy an old glacial outlet channel. The river and canal merge in Joliet.

2. The McCook Levee is approximately 4,800 feet long, fifteen feet above original ground and has a crest width of approximately twenty feet. In 1979 following 2 breaches, a 4,000 foot continuous line of 18-foot long steel sheet pile was driven along the axis of the levee between Lawndale Avenue and the Atchison, Topeka and Santa Fe Railroad tracks. The landside and riverside slopes of the dike are relatively steep and vegetated with mature trees and brush. Overall, the slopes are steeper than two horizontal to one vertical. Generally, the landside slope appears stable. Both slopes and the crest are densely vegetated with brush and mature trees except along the crest roadway. The crest and riverside slope are in poor to fair condition. There are several depressions and deep ruts in the crest. The riverside slope shows signs of surface sloughing and continuing erosion. A drainage ditch at the toe of the landside slope is used to divert water from the low lying area into a siphon near Station 16+00 which then flows under the Des Plaines River and discharges into the Sanitary and Ship Canal.

GLACIAL GEOLOGY

3. The McCook Levee site is located in northeast Illinois within the Chicago Lake Plain deposit. This deposit is associated with the Great Lakes section of the Central Lowlands Province and is the youngest part of the Wisconsinan drift. The surficial deposits and bedrock surface in this area have been affected by several glacial advances and retreats. The layer of soil resting on the bedrock surface consists typically of glacial till interbedded with layers of sand, silt and gravel outwash. The till is overlain in this area and particularly toward the east by the post glacial Chicago Lake Plain alluvial deposits. Along the Des Plaines River channel and the tributaries, the lake bed deposits and part of the till have been locally eroded and the old channels backfilled with recent alluvium. The alluvium is predominantly sandy silt derived from slopewash and erosion of weathered glacial till.

4. The Des Plaines River and the Chicago Sanitary and Ship Canal are the primary drainage features in the area. The Des Plaines River Valley is approximately one mile wide with the river being on the order of 200 to 250 feet wide.

Soil Conditions

5. The unconsolidated natural surficial deposits in the McCook Levee area all belong to the Cahokia Alluvium Group. The Cahokia Alluvium is found along the channels of modern rivers including the Des Plaines River. These deposits consist of poorly graded silt and sand containing local deposits of sandy gravel. These soils have replaced the old Lake Chicago Alluvium which was eroded by the Des Plaines River and the old glacial sluiceways.

6. The glacial tills lying directly on top of the bedrock were probably deposited during the Woodfordian substage of the Wisconsin stage of glaciation. These soils are part of the Wedron Formation and consist of brown to gray silty clays to clayey silts with relatively few cobbles and boulders. Within the till deposits are isolated beds and lenses of silty sand.

Bedrock Conditions

7. The glacial till overlies Silurian-aged dolomite rock which in turn overlies Ordivician-aged shales, sandstones and carbonate rocks. The Silurian dolomite extends to depths greater than 300 feet and rests on the Maquoketa Group of shales. The uppermost five feet of bedrock is typically weathered, consisting of dolomite fragments and slabs ranging from gravel to boulders in size. Below this weathered zone, the bedrock is generally massive with widely spaced vertical joints. Bedrock in the area is sedimentary in origin and is rarely exposed except in localized outcrops along the Des Plaines River Valley and where man has carved channels and quarries in the rock. The bedrock formations were deposited on very gentle slopes with slight inclinations. Except for the gentle slopes of the Kankakee Arch, the bedding planes are essentially flat. Prior to the deposition of the glacial sediments, the bedrock had undergone erosion which resulted in an undulating surface. The erosion channels in the bedrock surface developed along entrenched valleys which are now masked by the surficial deposits. Through mapping of borings which penetrate to bedrock, the bedrock surface topography can be defined. In the vicinity of the McCook levee, the bedrock surface was encountered in three borings at depths from 16 to 27 feet. The bedrock surface in the levee area appears to dip from 0° to 5° and is generally between elevation 500 and 600.

Groundwater

8. The hydrogeology of the McCook Area is complicated by the variation in thickness and permeability of the surficial deposits, the altered surficial drainage patterns, and the pattern of recharge created by the major changes in the local drainageways, namely the Des Plaines River and Chicago Sanitary and Ship Canal. The groundwater conditions are also affected by the dewatering at nearby quarries and the heavy pumpage of well water by local industries. These factors affect the occurrence of groundwater and the patterns of groundwater flow in the substrata.

9. There are two types of shallow aquifers in the Northeastern Illinois Area, the dolomite bedrock zone above the Maquoketa Shale and the glacial drift or outwash zones above the dolomite. The Maquoketa Formation acts as a barrier or aquiclude separating these upper "shallow" aquifers from the deeper zones of groundwater trapped in the Cambrian-Ordivician rocks. Removal of water from both the dolomite aquifer and the Cambrian-Ordivician aquifer has been taking place for 100 years in the McCook Area seriously depleting both.

10. The groundwater in the McCook vicinity is generally flowing from the uppermost perched water zones within the glacial drift towards lower perched zones such as the Des Plaines River and the Chicago Sanitary and Ship Canal. Conditions are favorable for the groundwater from the overlying drift to migrate into the weathered top of bedrock except where very dense silt or highly overconsolidated clays prevent this downward migration. For this investigation the groundwater conditions of most significance are the perched groundwater conditions in the glacial drift, which are dependent on the soil permeability and whether the zone is connected to the surface water system.

EXISTING LEVEE

11. The section of McCook Levee studied extends from the east edge of Lawndale Avenue, Station 0+00, to the south end of 47th Street, approximately Station 48+00. This section was reportedly constructed in the 1890's. The crest has a width of approximately twenty feet and varies in elevation from 599.5 at boring CBM-4-84 to elevation 604.1 at boring CBM-5-84, plate C-1. The resulting embankment height is approximately fifteen feet above the original ground line as shown. The crest is partially gravel surfaced for an access roadway north from the railroad tracks. In other areas the crest is unsurfaced but clearly for vehicular passage, soft in some areas and is deeply rutted at places. A sheet pile cutoff wall comprised of PSA23 Sections is located on the riverward side of the crest and extends from Station 1+00 to Station 41+00. The top elevation of the cutoff wall varies from elevation 602.5 at Station 1+00 to elevation 603.5 at Station 41+00. The riverside and landside slopes are densely vegetated with tall grass, brush, and mature trees. This levee lying along the north and west bank of the Des Plaines River, is owned by the Metropolitan Sanitary District of Greater Chicago. The McCook Levee reach from Sta. 8+00 and 14+00 was breached in 1979 resulting in substantial damage to the industrial areas lying north and west of the levee.

SUBSURFACE INVESTIGATION

12. Nine 20 foot deep borings by Walter H. Flood and Company sampled the levee in Sept. 1979. The levee material may have been dredged from the river and fill depths ranged from 10 to 17 feet and averaged 11.3' deep above natural ground. This fill consisted chiefly of dark brown, gray or black silty clays with some gravel. The natural ground consisted of silts and clays above limestone bedrock (see boring logs, plates C2-10). Patrick Engineering, Inc. took six borings to 30 foot depths or bedrock pinpointing problem levee areas in November, 1984. Water recharge testing confirmed zones existing through the fill and the underlying natural ground, plate C-11.

13. All borings were sampled at 2.5 foot intervals with a split spoon sampler or thinwall Shelby tubes. If bedrock or a boulder were encountered, it was penetrated not less than five feet with a roller rock bit. Rock coring was not required. Borehole recharge testing was performed in three of the borings to provide an indication of the magnitude of the potential for seepage and piping. The seepage potential of three areas within the levee was measured by simple recharge tests. Eight tests were attempted and four tests are considered useful. These tests provide an indication of the relative permeability of the soil surrounding the open area below the temporary borehole casing or hollow

stem auger. The test procedure consists of pumping water into the casing and allowing it to flow out through the bottom cross sectional area of the cased holes into a zone of soil. The hydraulic head, when testing saturated zones, is the difference between the top of the casing and the natural groundwater table.

14. Coarse-grained soil samples were tested for particle size distribution for classification purposes and to help estimate the seepage potential of granular soil zones. The allocated shear strength tests were performed only on fine-grained soils, the reason being that the shear strength of the coarse-grained soil can be more easily predicted on the basis of particle size distribution tests, visual examination and relatively density as measured by the "N" values. Laboratory testing was performed in the Patrick Engineering laboratory in accordance with the Corps of Engineers, Engineering Manual, "Laboratory Soil Testing," EM 1110-2-1906 and select ASTM procedures, plate C-12 summarizes the tests.

15. This exploration program determined the depth and type of fill, natural soil and bedrock materials. The recharge tests provided parameters for seepage analysis and the laboratory tests determined the engineering properties of the soils and the combined results were used for the A/E preliminary and the "In House" seepage analyses. The borings were plotted in a profile and divided into four zones of fill materials, four of natural foundation soils and the highly competent dolomite bedrock.

EMBANKMENT-FOUNDATION MATERIALS

16. A study of the soil boring logs, borehole recharge test results, and laboratory test results reveals the embankment fill is composed primarily of four material types referred to as Zones as summarized below:

Zone 1. Random rubble fill consisting of brown to black silty clay, old bricks, concrete, wood, slag, limestone residual and stone. The material properties and consistency within this Zone are extremely variable. This material was found in the reach between Station 0+00 and Station 8+00.

Zone 2. This material is a very stiff to hard silty clay fill, typical of a well compacted, modern highway fill. The moisture content is approximately twenty percent which appears to be near to slightly higher than the optimum moisture content of the material. This Zone appears to have good shear strength characteristics, is relatively consistent and is confined to Reach B, between Stations 8+00 and 14+00. This material may be recent fill replacing a breached zone.

Zone 3. This material is predominantly silty clayey sand to sandy silty clay fill. This Zone consists of medium dense silty clayey sands generally having greater than thirty percent fines. The material is reasonably uniform in density and particle size distribution. The material was only encountered in Reach D, Station 32+00 to 48+00.

Zone 4. The bulk of the levee appears to be constructed of moderately organic silty clay. This Zone consists of organic material having moisture contents ranging from 20 percent to 35 percent depending on the organic content and the location relative to the river level. The dry density of the material is approximately 95 to 100 pcf with the natural density approximately 120 to 130 pcf. Generally, the shear strength decreases with depth. The range of unconfined compressive strength (estimated using a pocket penetrometer) varies from greater than 4.5 tons per square foot to as little as 1.5 tons per square foot. Because of the organic content, this material is moderately compressible and minor consolidation is possible under increased loading conditions. The organic fill was encountered in Reaches B, C, and D.

17. The foundation or natural ground consists of four generalized soil Zones overlying dolomitic limestone bedrock. The four soil Zones are summarized as follows:

Zone 5: Loose well graded silty to clean sand was encountered immediately below the levee fill from Station 32+00 to Station 48+00, Reach D. This sand is saturated and has moderate shear resistance based on the "N" values, plate C-1. Due to the loose nature of this Zone, additional loading may produce minor additional settlements. Borehole recharge test results indicate this material has a coefficient of permeability ranging from 10^{-4} to 10^{-2} cm/sec with moderate seepage and piping potential.

Zone 5A: Similar to Zone 5 but with saturated gravel layers and cobbles.

Zone 6: Zone 6 consists of normally consolidated soft brown and gray silty clay. The material within this Zone is saturated and has moisture contents of approximately 30 percent to 35 percent. Unconfined compressive strengths estimated with the aid of a pocket penetrometer range between 0.7 and 1.0 tons per square foot. The relatively high water content and low shear strength suggest this material is moderately compressible and would develop high pore pressures under rapid loading. This material appears to be common to all reaches, and is the weakest material encountered.

Zone 7. This soil is very stiff to hard silty clay borderline clayey silt. The Zone is saturated and extremely dense (overconsolidated) with an average moisture content of fifteen percent. This material yields dry densities of approximately 120 pcf and natural wet densities of approximately 137 pcf. The estimated unconfined compressive strength of the material is in excess of 4.5 tons per square foot. Zone 7 material underlies Zone 6 material and was encountered in all reaches.

Zone 8. The layer immediately overlying bedrock over most of the length of the levee except part of Reach C is an extremely dense silty sand. This Zone consistently has greater than twenty percent silt and "N" values greater than 100. The high "N" values indicate the shear strength of the silty sand is high and the silty nature of the soil indicates this layer will have limited seepage potential and is virtually incompressible.

18. The bedrock underlying the site is Racine Formation of Ordovician age. This is a moderately jointed, pure to locally argillaceous, reef forming dolomite with scattered chert and thin to thick bedding. Near surface joints and bedding plane fractures are open and so permeable but close and become less permeable with depth. Rock was not cored so local conditions are not known.

DESIGN ANALYSIS

DESIGN SHEAR STRENGTHS

19. The design shear strengths used for the existing material in the levee were based on the recommended parameters for engineering analyses listed in the Patrick report (Ref. 1). The design parameters for the new clay fill were shown based on Table 9-1, "Typical Properties of Compacted Materials" from the Naval Facilities Engineering Command Design Manual, NAVFAC-DM-7 (Ref. 2) and Joseph E. Bowles, Foundation Analysis and Design, Table 2-5 "Representative values for angle of internal friction, Ø" (Ref. 3). Once the area for borrow material is chosen, laboratory tests will have to be run to provide the actual soil parameters for the clay fill. The stability analysis will be finalized when the final borrow site and soil parameters are determined. The design parameters used in the stability analysis are indicated on Table C-1.

Table C-1 Design Parameters for Engineering Analysis

Zone	Moist (pcf)	Sat. (pcf)	Q Total Stress End of Raised Construction Condition Analyses C (psf)	Ø	$\overline{R}$ Effective Stress Steady State Seepage Condition Analyses C'(psf)	Ø'
New clay fill	125	130	1000.0	0°	200.0	32°
1	120	130	0.0	27°	0.0	27°
2	125	130	1000.0	0°	0.0	30°
3	120	130	0.0	28°	0.0	28°
4	120	125	250.0	0°	0.0	24°
5	125	125	0.0	30°	0.0	30°
5A	130	130	0.0	30°	0.0	30°
6	120	120	300.0	0°	0.0	26°
7	140	140	0.0	28°	0.0	28°
8	145	145	0.0	40°	0.0	40°
Bedrock	130	135	0.0	35°	0.0	35°

STABILITY ANALYSIS

20. The slope stability was analyzed using both a circular arc and wedge method program. The circular arc program is WES no. 741-F3-F5030 entitled "Slip Circle Slope Stability With Side Forces (I0013)". This program performs slip circle slope stability calculations on embankment slopes in accordance with EM 1110-2-1902. The factor of safety against sliding is calculated for a series of trial arcs tangent to a horizontal plane, and locates the circle with the minimum factor of safety.

21. The necessary input was the minimum elevation of all trial arcs for a given run; center of the initial circle; search increment to locate the circle with the minimum factor of safety; embankment and foundation profile; seismic coefficient; soil data, including moist and saturated unit weights (pcf), cohesion strength parameter "c" (psf), and angle of internal friction strength parameter "phi" (deg); and piezometric surface data. Also, the side force direction was assumed equal to the average of the embankment slopes adjacent to the slice interface.

22. The output consists of the center and radius of the critical arc with the minimum factor of safety, and a detailed listing of the slice data for the critical arc including: slice number, x-coordinate of the centerline of the slice; slice width; total weight of the slice as determined by the moist unit weights above the piezometric surface and saturated unit weights below the surface; acting on the bottom of the slice: the direction and magnitude of the water force, the developed cohesive force and "phi" angle, and the normal stress and force; acting on the sides of the slice: the angles of the side force direction and the developed side forces.

23. In some cases the search procedure found the local minimum safety factor and not the actual minimum. It was necessary to study the results sufficiently to determine if the critical safety factor had been found, and to determine if the failure surface as defined by the computer were realistic. Occasionally minimum factors of safety had to be rejected due to the proximity of the failure surface to the embankment slope.

24. The cross section was also analyzed using WES wedge program no. 741-F3-F5020 entitled "Slope Stability Utilizing a Generalized Failure Surface (I0014)". This program also performs slope stability calculations in accordance with EM 1110-2-1902. Factors of safety are determined for failure surfaces defined by either a series of straight line segments or an upslope wedge, network block and downslope wedge. Straight line segments were employed for all the runs made on this project.

25. The input for the wedge program consists of the same as the arc with the exception of using coordinates to define the failure surface rather than the center and tangent elevation of the arc. The direction of the side forces was assumed to be equal to the average of the top slope of the slice and the slope of the failure surface at the location of the slice. The output is also the same as the arc program except for the failure surface.

26. Input for both programs can be entered interactively from the terminal (in this case, a Texas Instrument 700) or read from a previously prepared data file. The output may come directly back to the terminal or be stored in a file for later use.

RESULTS OF ANALYSES

27. The results of these computer runs are condensed on Table C-2, and discussed in more detail in the following paragraphs. Although numerous computer runs were performed, only the more critical cases will be discussed here. For the ease of comparison, only the 100-year flood level protection discussed in this report; although only levels of protection were examined.

Table C-2 Summary of Computer Cases Analyzed

Reach	Sta.	Levee Height	Side Slopes River	Side Slopes Land	Case	Side Analyzed	Method	Minimum required factor of safety	Factor of safety	Figure
B	11+25	Existing (El. 600)	2:1 1/	1:1 1/	Infinite slope	River	-	1.0	1.15	1
B	11+25	100-year (El. 604.5)	2:1 Stone	2:1 Clay	Partial pool	River	Wedge	1.4	1.61	3
B	11+25	100-year (El. 604.5)	2:1 Stone	2:1 Clay	E.O.C.	River	Wedge	1.3	2.79	4
B	11+25	100-year (El. 604.5)	2:1 Stone	2:1 Clay	E.O.C.	Land	Wedge	1.3	2.63	5
B	11+25	100-year (El. 604.5)	2:1 Clay	2:1 Clay	Partial pool	River	Wedge	1.4	2.06	7
B	11+25	100-year (El. 604.5)	2:1 Clay	2:1 Clay	E.O.C.	River	Arc	1.3	2.01	8
B	11+25	100-year (El. 604.5)	2:1 Clay	2:1 Clay	E.O.C.	Land	Arc	1.3	2.04	9
C	23+00	Existing (El. 599.5)	2:1 1/ Clay	3:2 1/ Clay	Infinite slope	River	-	1.0	0.89 2/	10
C	23+00	100-year (El. 605)	2:1 Clay	2:1 Clay	Partial pool	River	Wedge	1.4	1.86	12
C	23+00	100-year (El. 605)	2:1 Clay	2:1 Clay	E.O.C.	River	Arc	1.3	1.18 2/	13
C	23+00	100-year (El. 605)	2:1 Clay	2:1 Clay	E.O.C.	Land	Arc	1.3	1.07 2/	14
C	23+00	100-year (El.605)	5:2 Clay	5:2 Clay	Partial pool	River	Arc	1.4	1.73	15

Table C-2 Summary of Computer Cases Analyzed (Cont'd)

C	23+00	100-year (El. 605)	5:2 Clay	5:2 Clay	E.O.C.	River	Arc	1.3	1.65	16
C	23+00	100-year (El. 605)	5:2 Clay	5:2 Clay	E.O.C.	Land	Arc	1.3	1.23 2/	17
B,C 3/	11+25 23+00	100-year (El. 604.5)	2:1 Stone	2:1 Clay	Partial pool	River	Wedge	1.4	2.97	18
B,C 3/	11+25	100-year (El. 604.5)	2:1 Stone	2:1 Clay	E.O.C.	River	Wedge	1.3	3.70	19
B,C	11+25	100-year (El. 604.5)	2:1 Stone	2:1 Clay w/Berm	E.O.C.	Land	Arc	1.3	1.84	20

1/ Approximate slope.
2/ Insufficient factor of safety.
3/ For levee area between Sta. 11+25 and 23+00, Sta. 11+25 cross section was used with Reach C soils for worst case.

28. Conditions Requiring Analysis. The various loading conditions to which a levee and its foundation may be subjected and which should be considered in analyses are designated as follows: Case I, end of construction, Case II, sudden drawdown from full flood stage; Case III, critical flood stage; Case IV, steady seepage from full flood stage, fully developed phreatic surface; Case V, steady seepage from full flood stage, partially developed phreatic surface; Case VI, earthquake. The design condition considered most applicable for the levee on the Des Plaines River was case III, critical flood stage (also known as Partial Pool, and referred as such in this report), although Case I, end of construction, was also studied for the rehabilitated levee when new clay fill is added to the top of the dike to increase the level of protection. Case II, sudden drawdown, occurs when a prolonged flood stage saturates at least the major part of the riverside embankment and then falls faster than the soil can drain. This condition is unlikely to occur on the Des Plaines River and; therefore, not considered in this stability analysis. Cases IV and V, steady seepage, refer to conditions when the water remains at or near full flood stage long enough to saturate the embankment, resulting in a steady seepage condition.

29. The flood stages on the Des Plaines River have short duration periods, thus eliminating the possibility of steady seepage occurring over the entire embankment. Earthquake loading, Case VI, is not considered due to the low probability of an earthquake coinciding with periods of high water. Cast 3 - End of Construction (E.O.C.) represents undrained conditions for impervious embankment and foundation soils. Excess pore water pressure may be present because the soil does not have sufficient time to drain after loading. Results for the unconsolidated-undrained "Q" tests (see 1st column in Table C-1) are applicable to fine-grained soils while results of the consolidated-drained "S" tests can be used for pervious soils that drain fast enough during loading so that no excess pore water pressure is present at the end of construction. Both riverside and

landside slopes are examined for the End of Construction case and must have a minimum factor of safety of 1.3 as stated in Table 6-1, Minimum Factors of Safety - Levee Slope Stability, in EM 1110-2-1913 (Ref. 4). Case III, Partial Pool, refers to the condition whereby some intermediate prolonged flood stage saturates the embankment and a condition of steady seepage is established. The Effective Stress Steady State Seepage Condition Analyses "$\overline{R}$" (see 2nd column in Table C-1) lists the soil parameters for this condition. The riverside slope is studied for this case and must meet the minimum factor of safety requirement of 1.4.

30. Stability analyses were run for two areas of McCook Levee, Reaches B and C (see plate C-1). These reaches were considered the most critical for slope stability. Reach B is where the breaches occurred in 1979 and due to the proximity of the levee to the river and access road there is a severe constraint on the base width and location of any proposed levee rehabilitation. Reach C is composed of a very weak organic silty clay with a soft organic silty clay foundation.

31. Figure C-1 shows the existing condition at Sta. 11+25, which is typical for Reach B. It was decided that the river side slope should be checked by the infinite slope method due to the small area of levee riverward of the steel sheet piling. The factor of safety for the slope, assuming no horizontal seepage is 1.15. A factor of safety of 1.0 is sufficient for infinite slope, so the existing condition of the slope on the riverside is satisfactory. However, the existing crown at this section of the levee is EL. 600 which would be overtopped by a storm with a 25-year frequency or more (see table C-3).

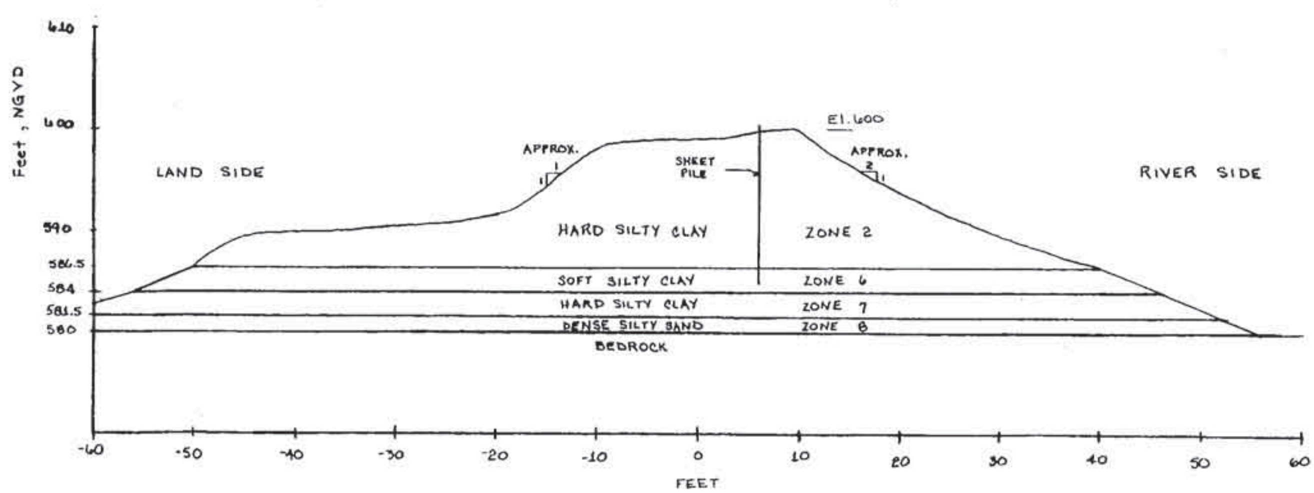


Figure C-1
Sta 11+25 - Existing Conditions
McCook Levee

Table C-3 Des Plaines River Stages at McCook Levee

River				Frequency Storm					
Mile	1	2	5	10	25	50	100	500	SPF
40.96	592.44	597.05	598.46	599.19	600.18	600.94	601.47	602.86	605.73
40.97	592.44	597.05	598.46	599.19	600.19	600.95	601.47	602.90	605.92
40.98	592.44	597.06	598.46	599.19	600.19	600.95	601.47	602.96	605.97
41.21	592.56	597.16	598.57	599.31	600.31	601.07	601.60	603.10	606.02
41.98	593.06	597.48	598.88	599.61	600.62	601.39	601.92	603.44	606.20
42.00	593.08	597.50	598.91	599.64	600.65	601.42	601.95	603.47	606.19

32. Figure C-2 shows the cross section recommended by the Geotechnical Branch to rehabilitate this section of the levee to withstand a river height up to the 100-year flood level. For the ease of comparison, only the 100-year flood level protection is discussed in this report; although other levels were examined. This plan, Alternative I, includes three feet of freeboard, a 2H:1V stone filled river slope and a 5H:2V land side slope. The cross section was analyzed for the Partial Pool case and End of Construction case.

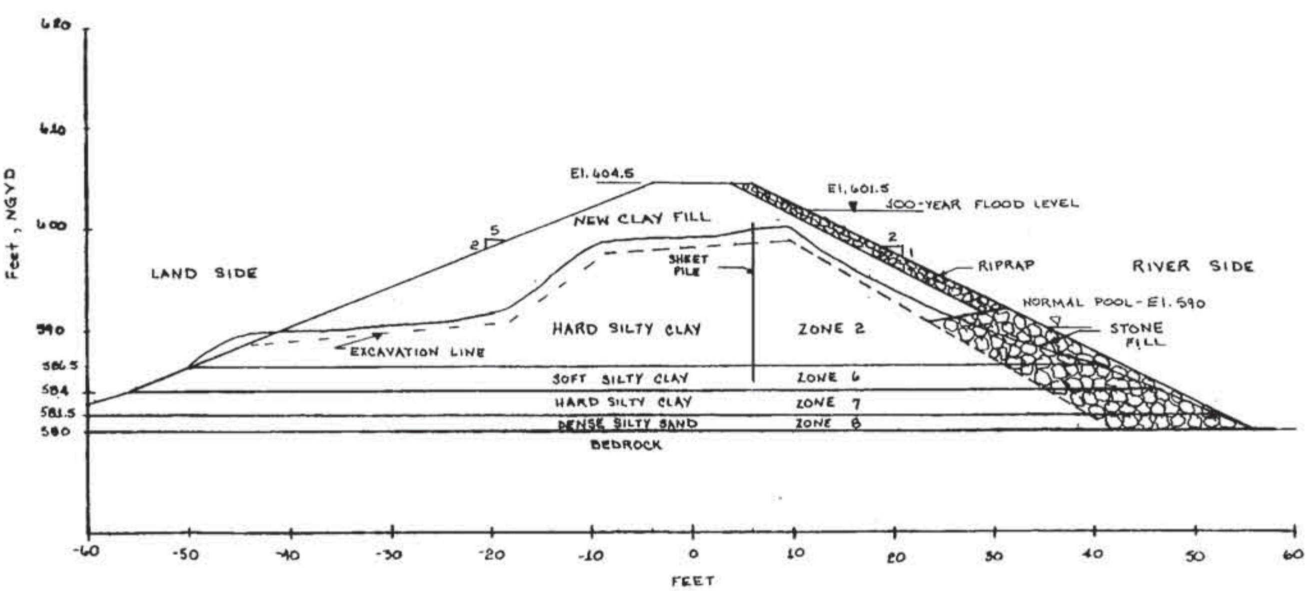


Figure C-2
Sta. 11+25 - Rehabilitation - Alternative I
McCook Levee

33. The wedge method was used to check the river slope for Partial Pool and the factor of safety was found to be 1.61, as can be seen in figure C-3. The section checked for End of Construction (E.O.C.) on the river side, assuming the river was at normal pool (EL. 590), gave a factor of safety of 2.79, figure C-4 and land side a factor of safety of 2.63, figure C-5. Thus, this proposed cross section is stable.

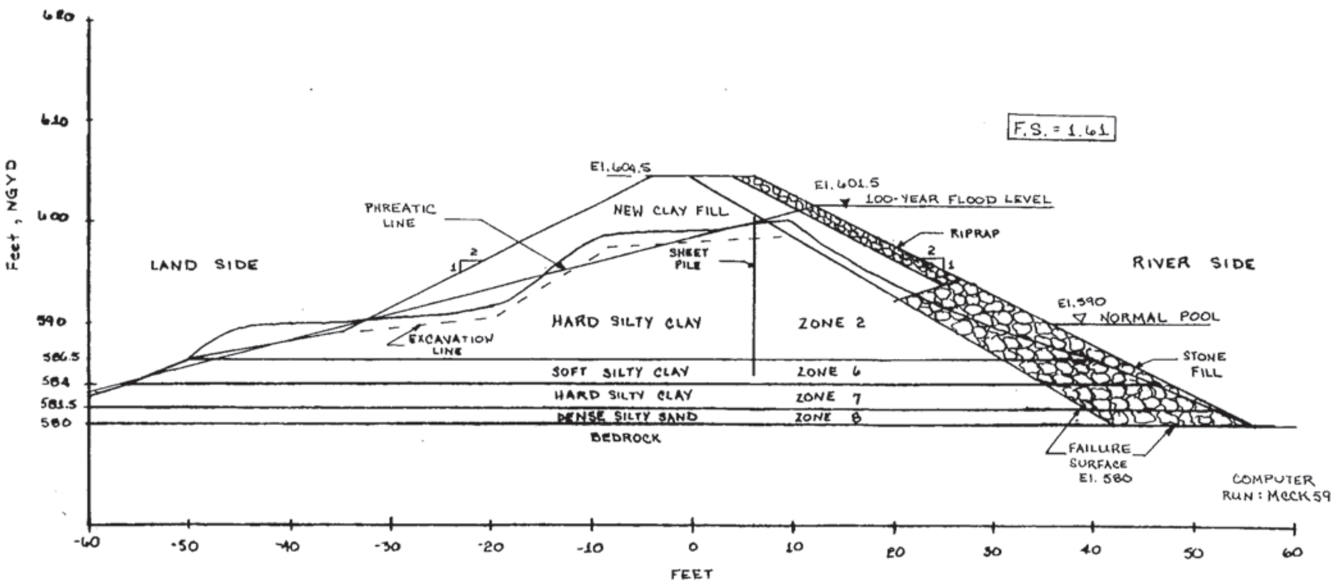


Figure C-3
Sta. 11+25 - Partial Pool - Alternative I
McCook Levee

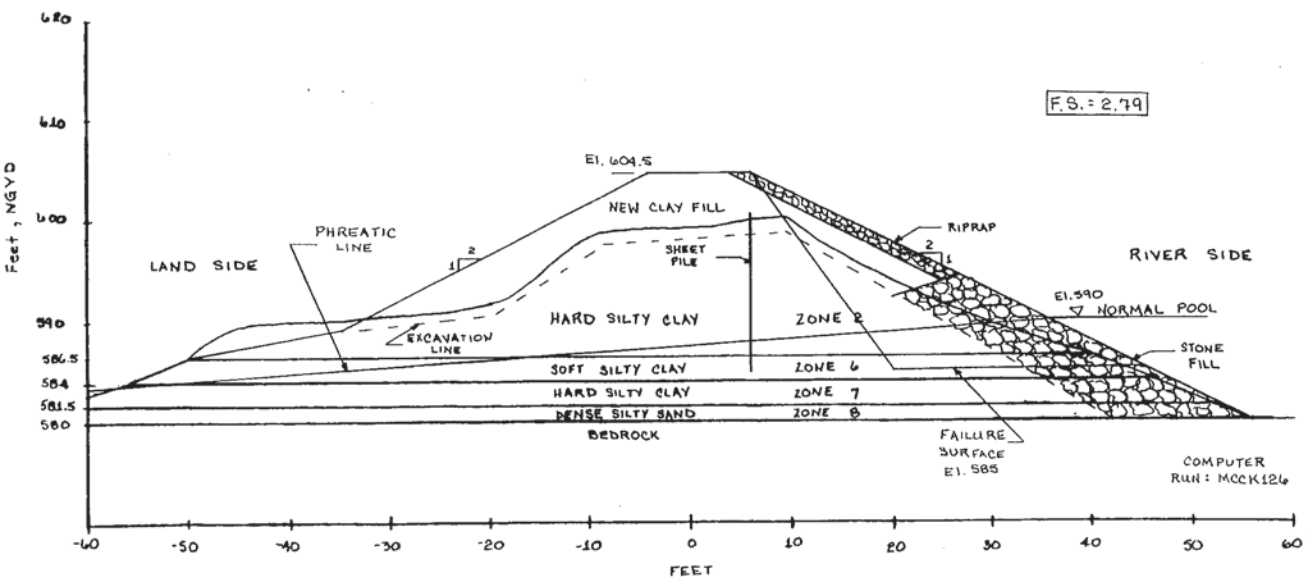


Figure C-4
Sta. 11+25 - End of Construction - River Side - Alt. I
McCook Levee

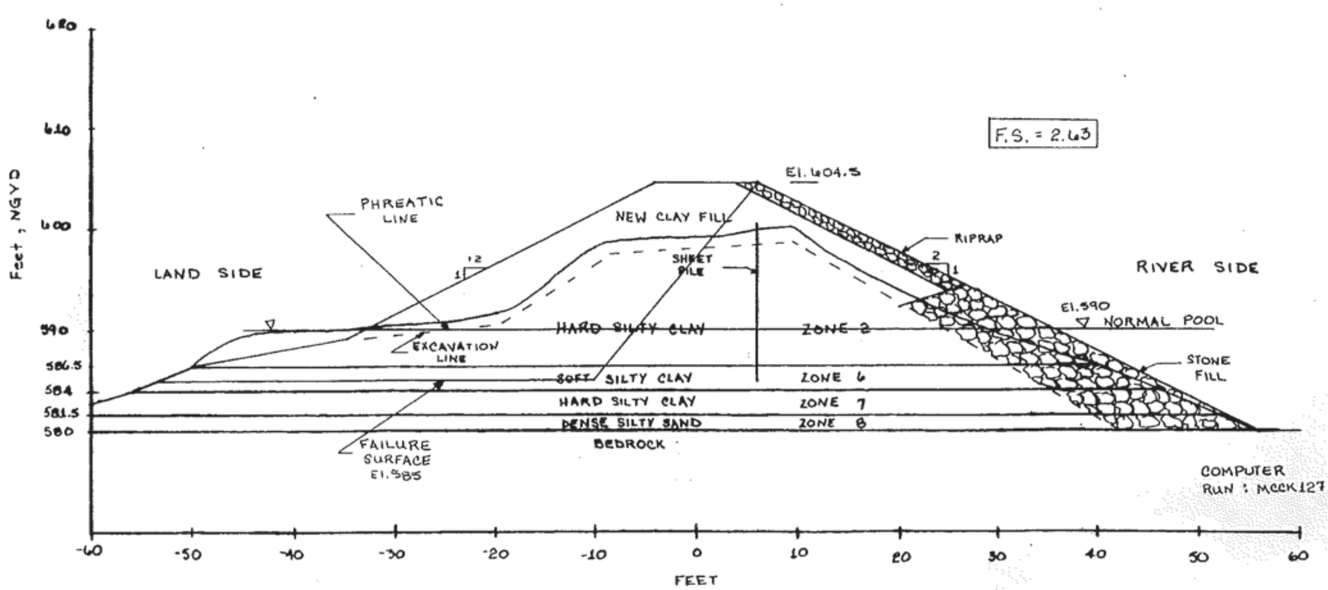


Figure C-5
Sta. 11+25 - End of Construction - Land Side - Alt. I
McCook Levee

34. Figure C-6 shows a plan proposed by NCCPD-P in an attempt to avoid the rehabilitated levee from encroaching on the river channel. This proposed section, Alternative II, was analyzed the same as Alternative I. The factors of safety determined were as follows: Partial Pool - 2.06, figure C-7; EOC - River - 2.01, figure C-8 and EOC - Land - 2.04, figure C-9. This alternative is also considered to be satisfactory.

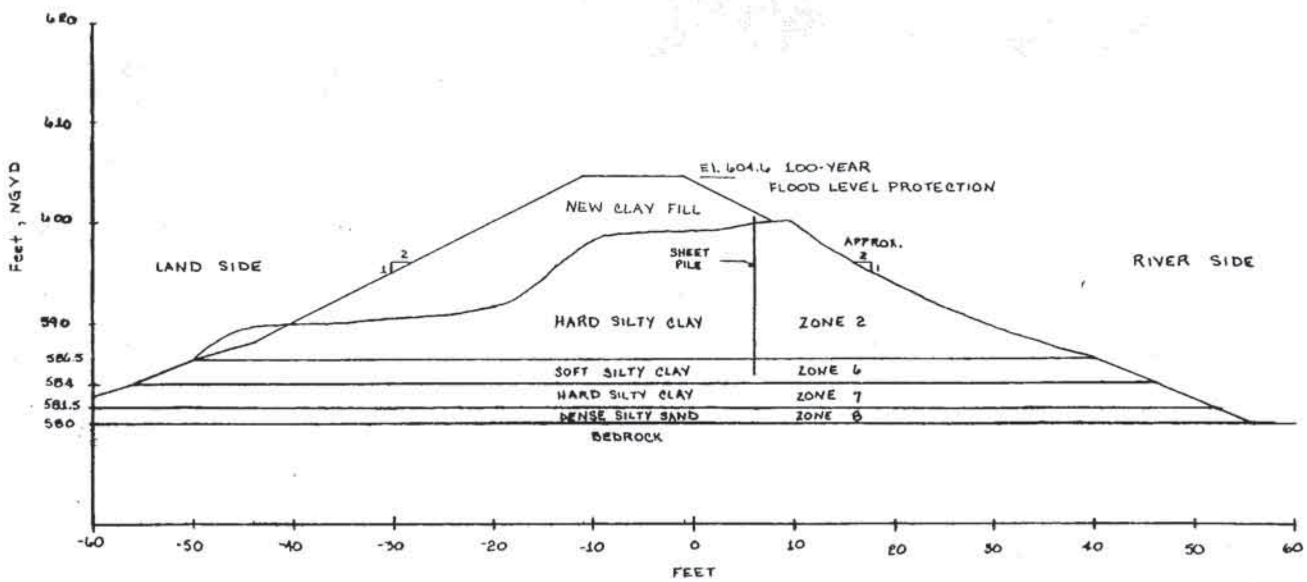


Figure C-6
Sta. 11+25 - Rehabilitation - Alternative II
McCook Levee

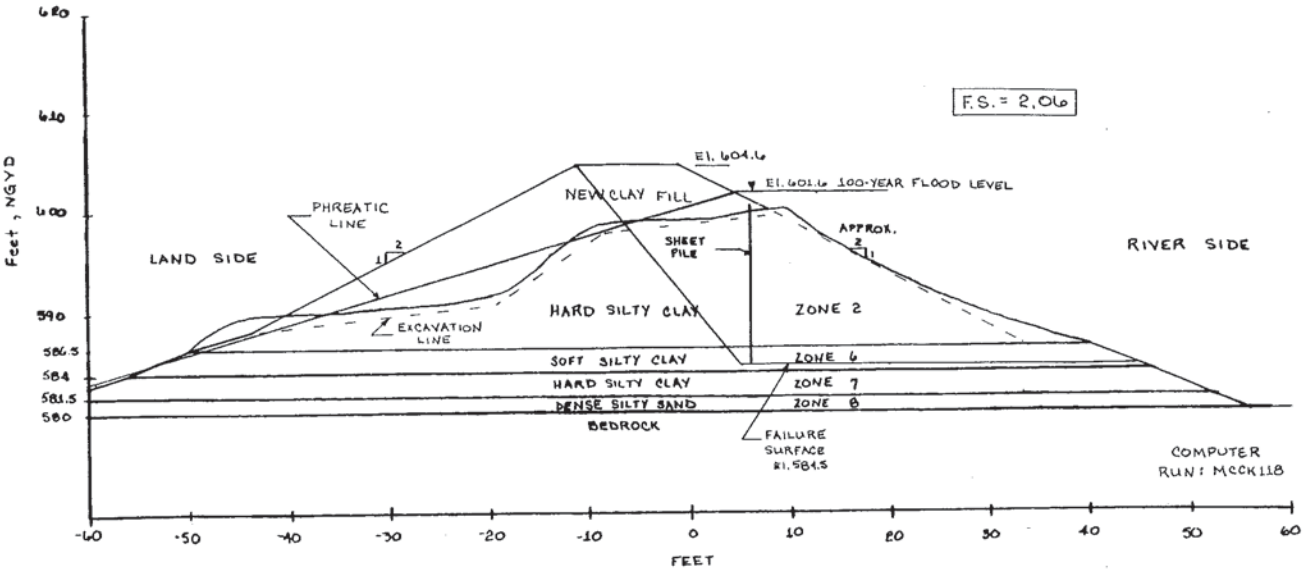


Figure C-7
Sta. 11+25 - Partial Pool - Alternative II
McCook Levee

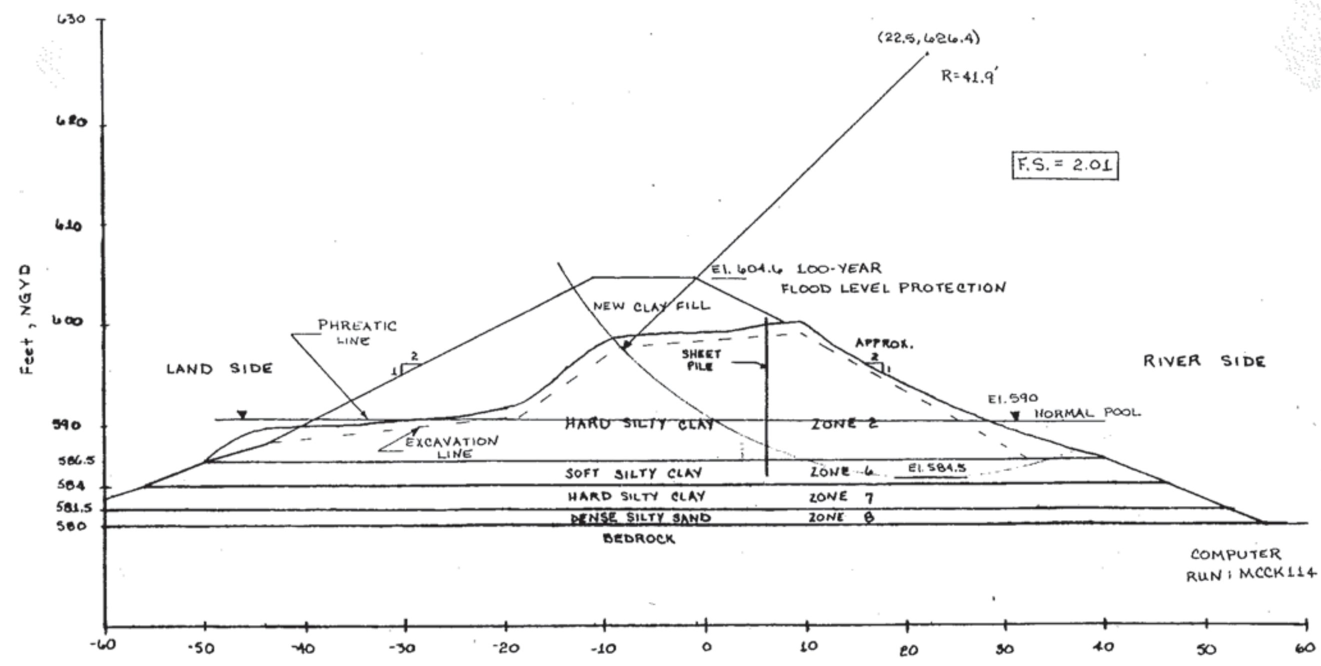


Figure C-8
Sta. 11+25 - End of Construction - River Side - Alt. II

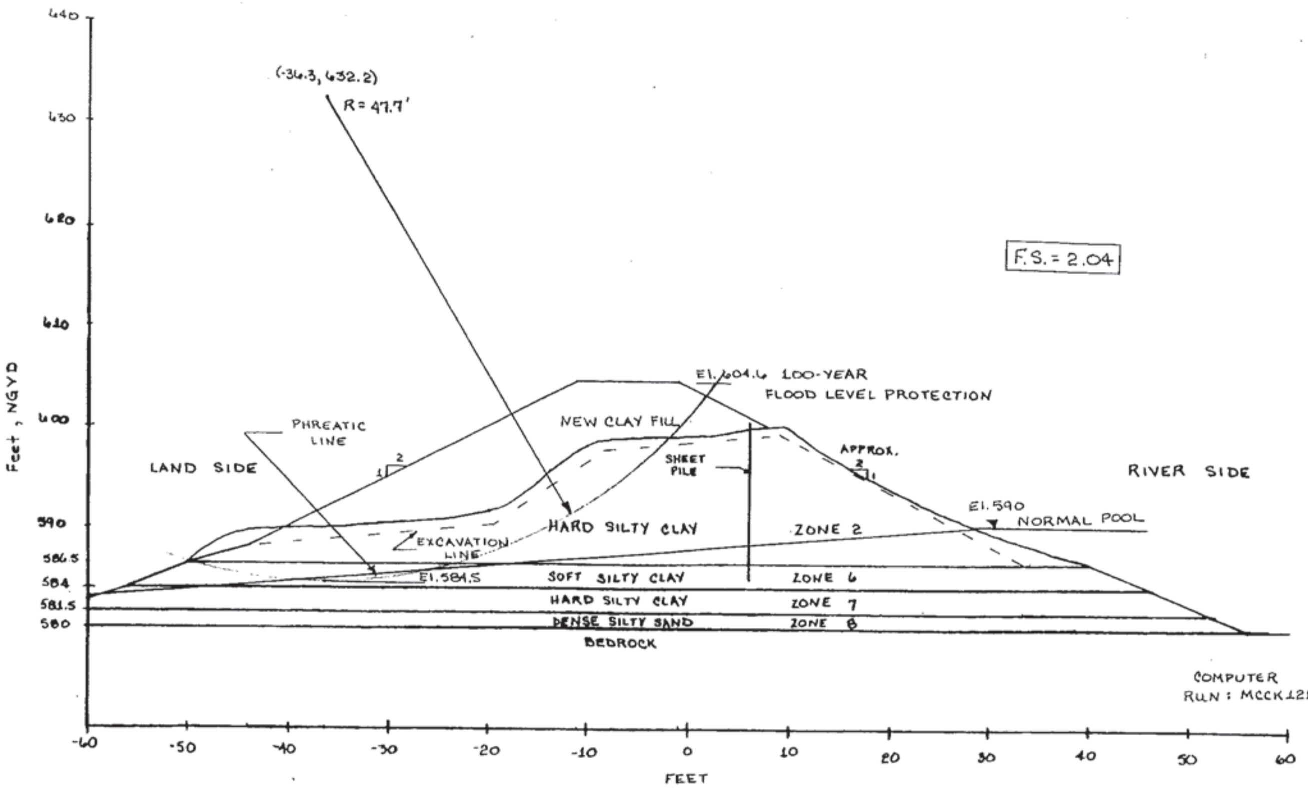


Figure C-9
Sta. 11+25 - End of Construction - Land Side - Alt. II
McCook Levee

35. Figure C-10 shows the existing condition of Sta. 23+00, which is typical for Reach C. The river side slope was also checked on this section by the infinite slope method. The factor of safety equals 0.89, which is insufficient. Figure C-11 shows the proposed rehabilitation for this reach. The minimum factor of safety for the Partial Pool case was found to be 1.86, see figure C-12. Figures C-13 and C-14 show the End of Construction cases for river side and land side and the factors of safety were 1.18 and 1.07, respectively. 2H:1V slopes are therefore insufficient for Reach C.

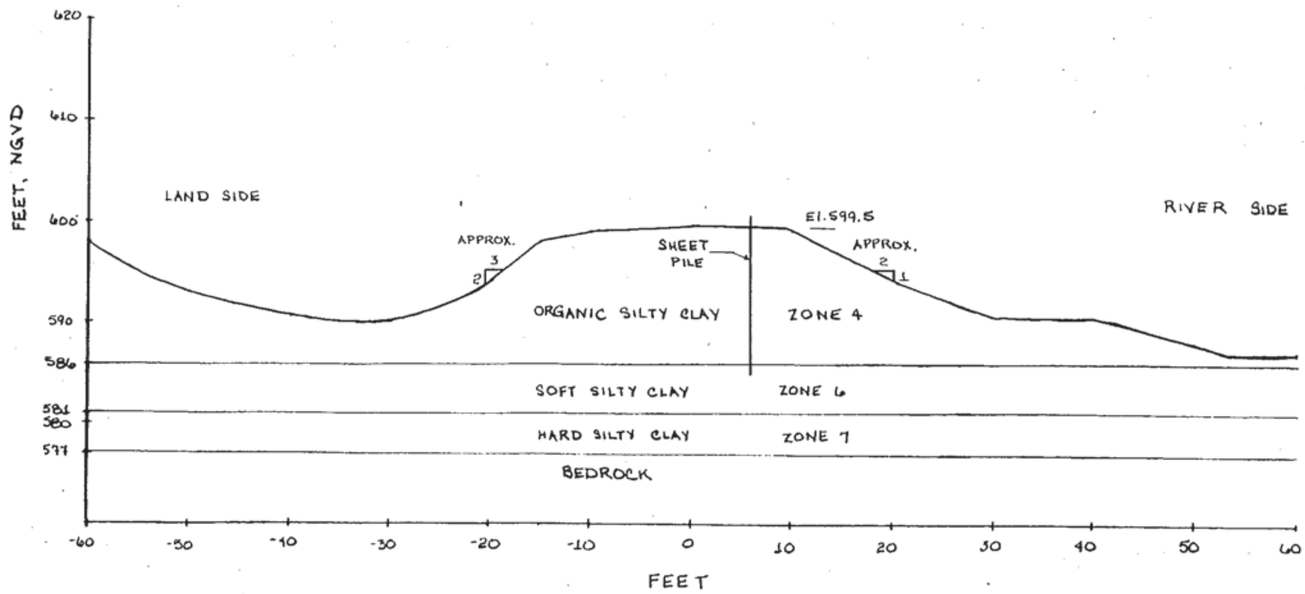


Figure C-10
Sta. 23+00 - Existing Conditions
McCook Levee

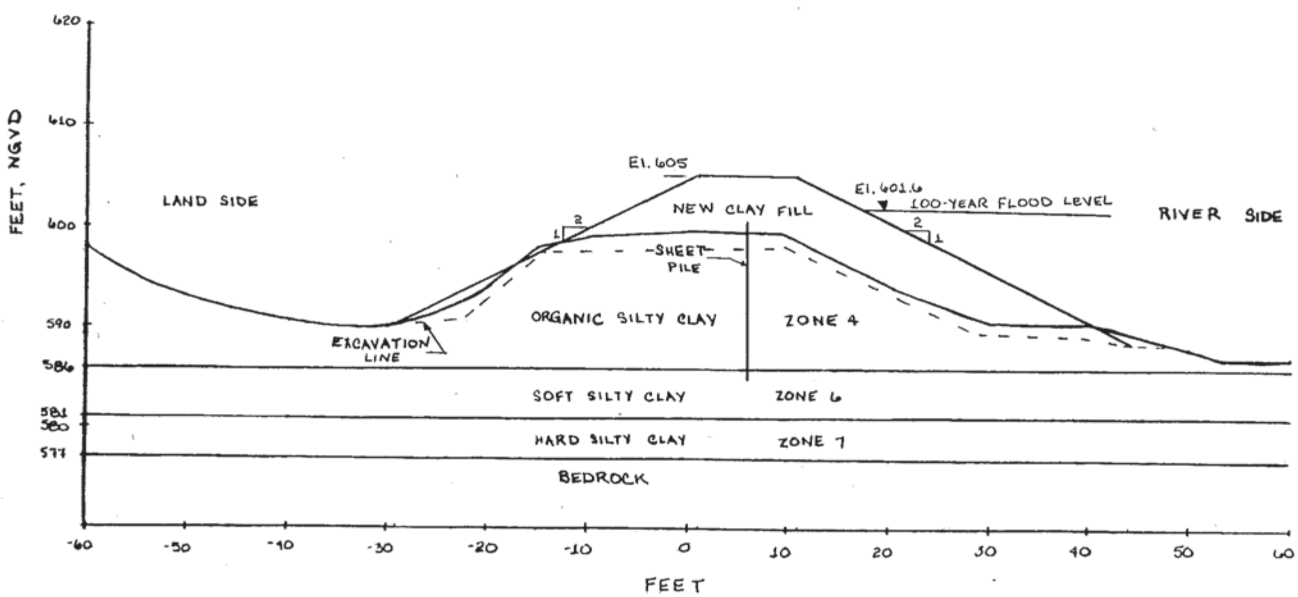


Figure C-11
Sta. 23+00 - Rehabilitattion - 2H:1V Slopes
McCook Levee

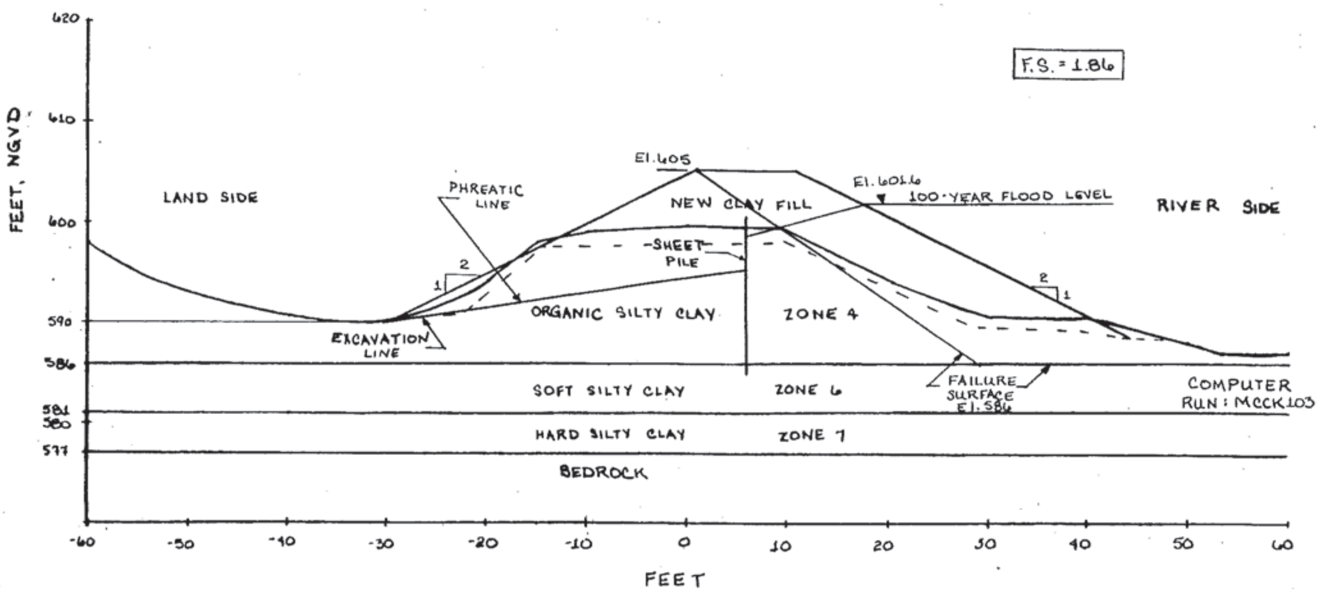


Figure C-12
Sta. 23+00 - Partial Pool - 2H:1V slopes
McCook Levee

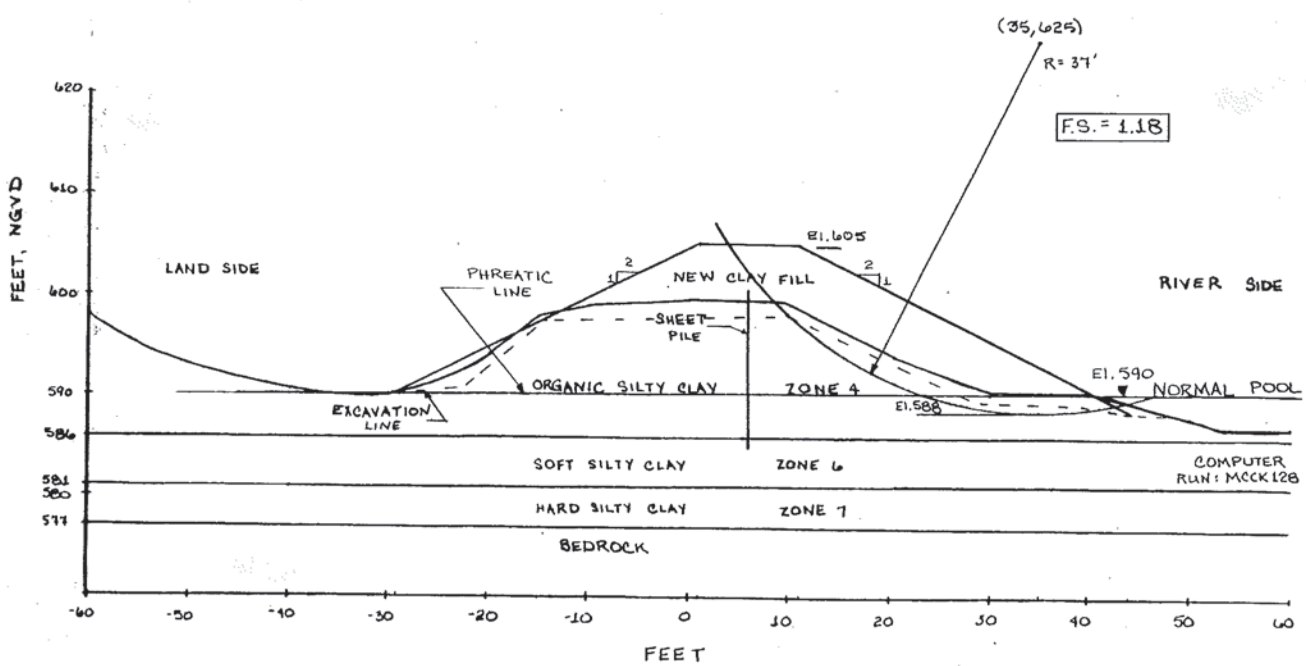


Figure C-13
Sta. 23+00 - End of Construction - River Side
McCook Levee

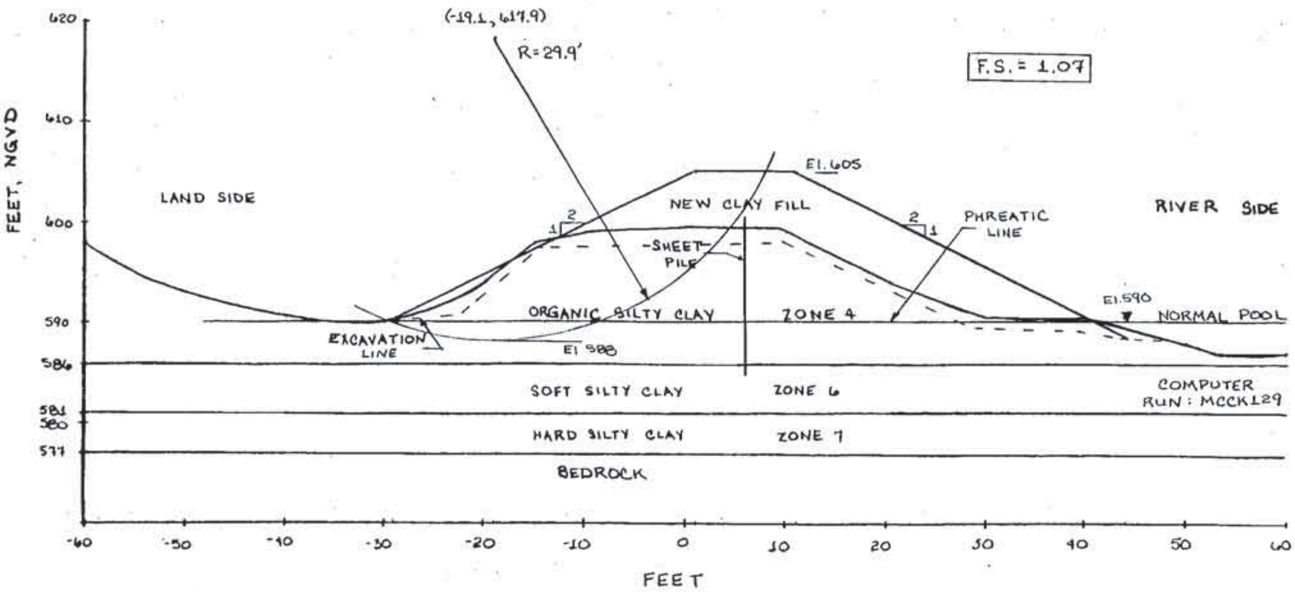


Figure C-14
Sta. 23+00 - End of Construction - Land Side
McCook Levee

36. The cross section for Sta. 23+00 was revised to contain 5H:2V slopes both river and land side. Figure C-15 shows how this cross section withstands a Partial Pool condition with a river side circular arc failure surface. The factor of safety was found to be 1.73.

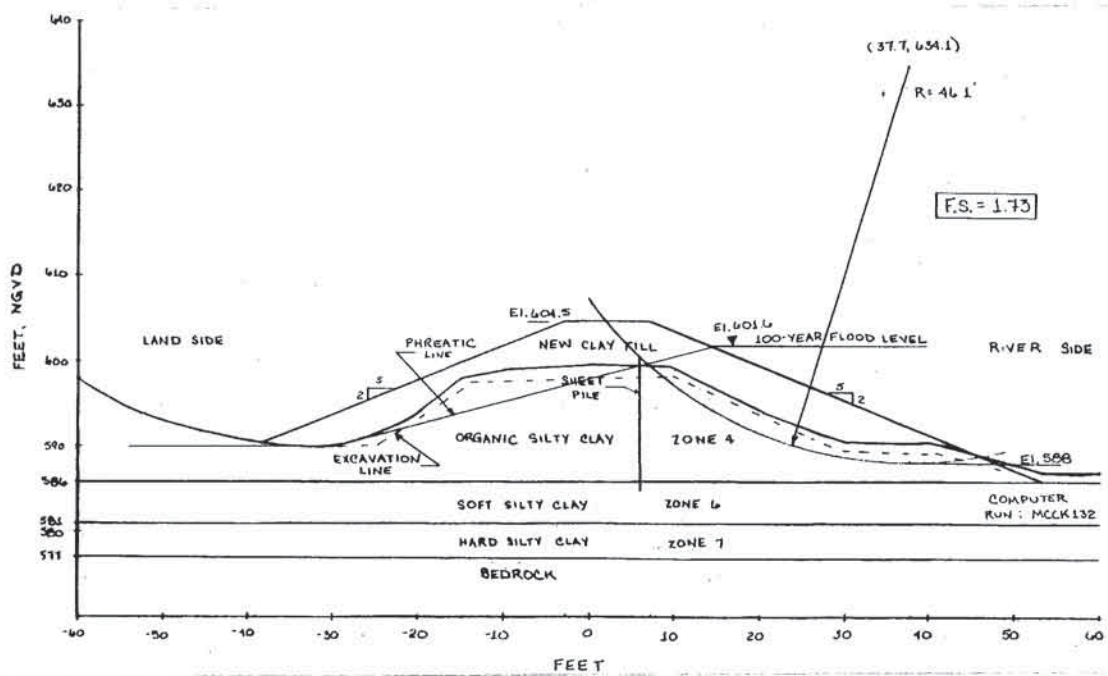


Figure C-15
Sta. 23+00 - Partial Pool - 5H:2V Slopes
McCook Levee

37. The revised cross section was also checked for End of Construction as in figures C-16 and C-17. The river side failure surface produced a 1.65, but the land side arc was determined to have an insufficient factor of safety of 1.23. Therefore a 5H:2V is needed for the river side slope of Sta. 23+00, but the land side slope will need a 3H:1V slope or an alternative rehabilitation.

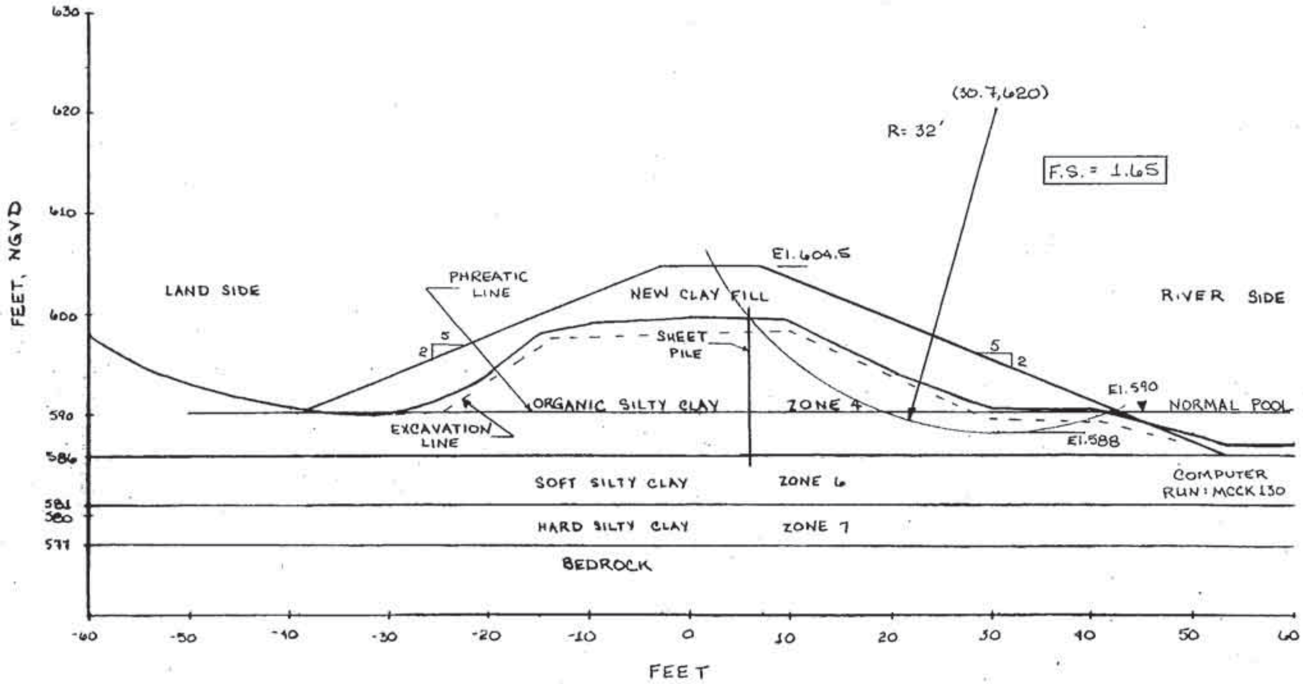


Figure C-16
Sta. 23+00 - End of Construction - River Side
McCook Levee

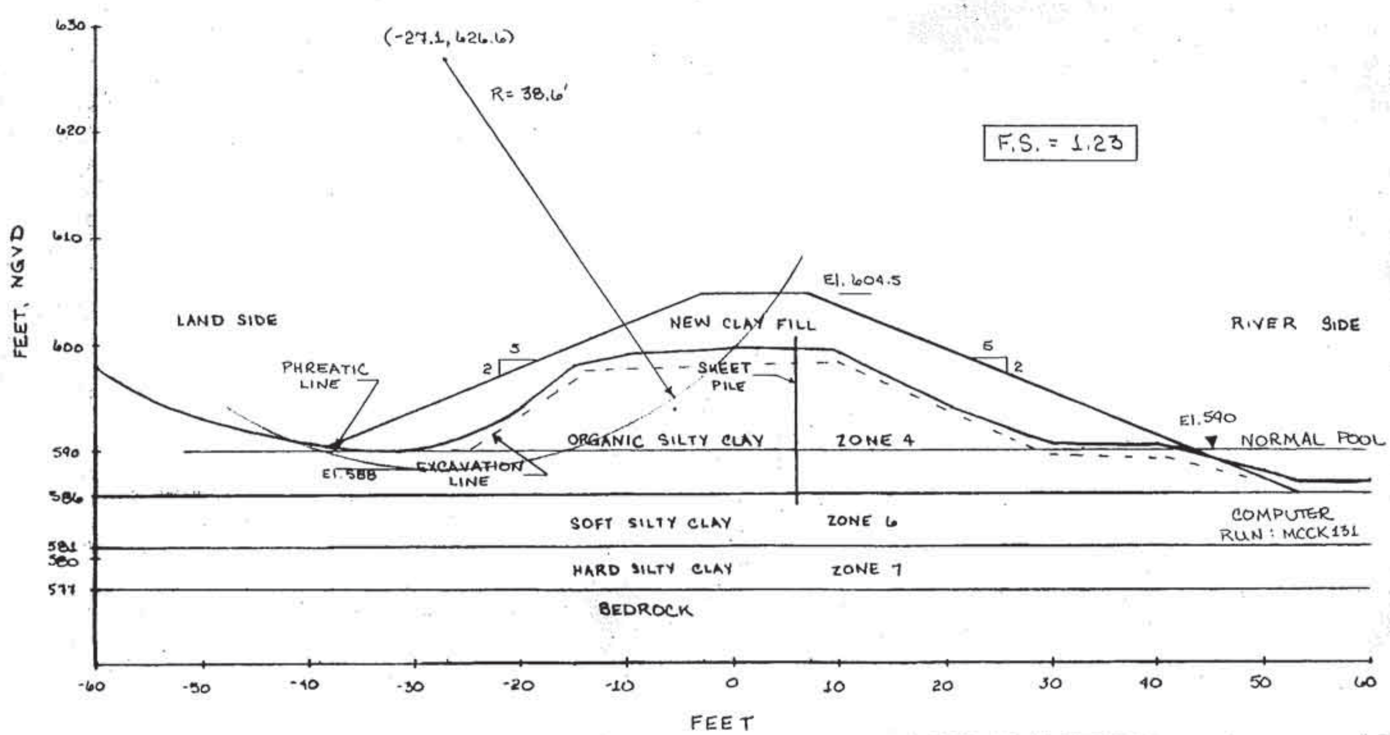


Figure C-17
Sta. 23+00 - End of Construction - Land Side
McCook Levee

38. The levee between Sta. 11+25 and Sta. 23+00 was not surveyed, but from the aerial photographs it appears that the cross section taken at Sta. 11+25 is very similar. This section is also very close to the river and, thereby restricted the same as Sta. 11+25 as discussed in paragraph 29. The soils are not the same as in Sta. 11+25 (see plate C-1), so as a worst case condition; the hard silty clay was replaced in the computer run with organic silty clay. Figure C-18 shows the rehabilitated levee with a 2H:1V stone-filled river side slope and the partial pool condition. A factor of safety of 2.97 was found for this case. A land side berm was added to the section to increase the stability of the levee on the land side and figures C-19 and C-20 show the levee with the End of Construction case for river side - F.S. = 3.70 and land side - F.S. = 1.84.

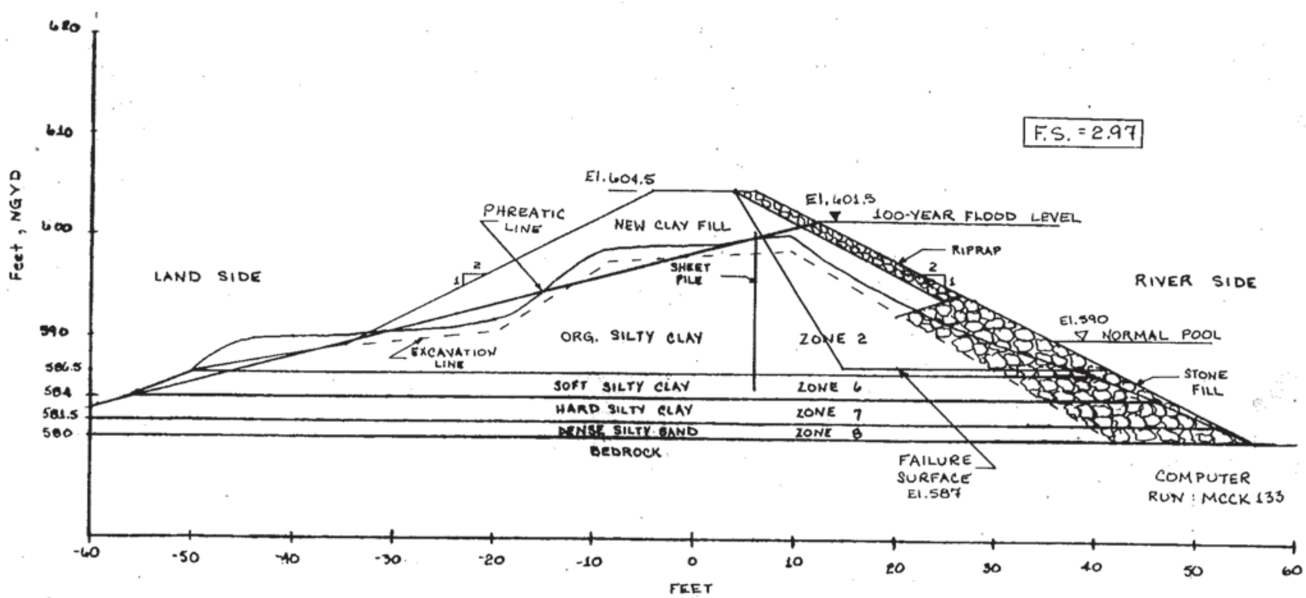


Figure C-18
Sta. 17+00 (Approx.) - Partial Pool
McCook Levee

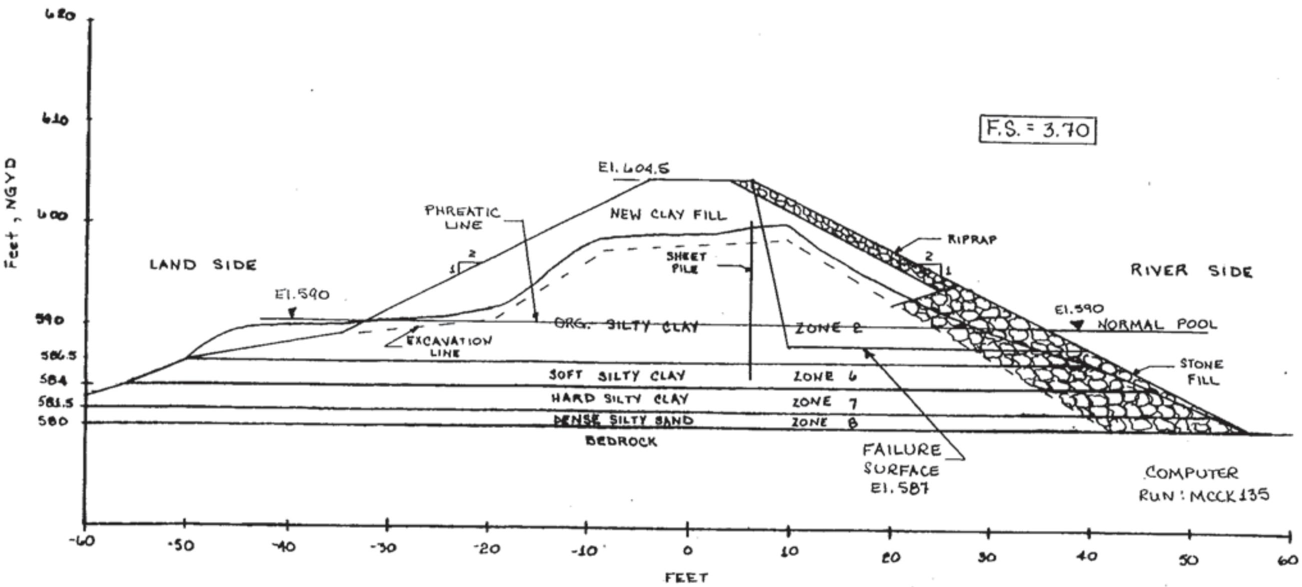


Figure C-19
Sta. 17+00 (Approx.) - End of Construction - River Side
McCook Levee

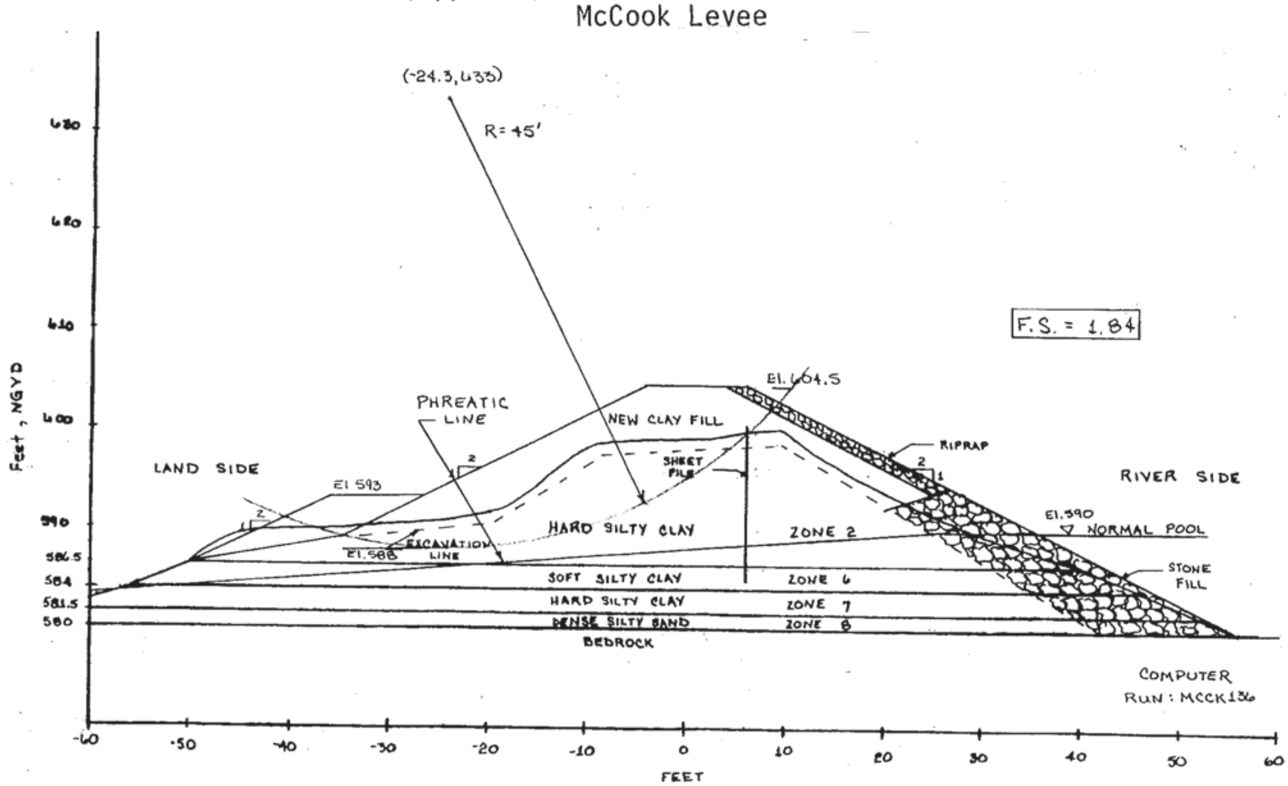


Figure C-20
Sta. 17+00 (Approx.) - End of Construction - Land Side (Berm Added)
McCook Levee

39 Reach A presents a special problem that was unable to be determined by a computer slope stability analysis. The embankment in this reach is composed of Zone 1 material-random rubble fill consisting of brown to black silty clay, old bricks, concrete, wood, slag, limestone, residual and stone. The sheet piling is not considered a positive cut-off for water. Further study would have to be performed on this section. One possible rehabilitation of this reach would be to grout this section of the levee and tie it into the more stable material in Reach B.

40. Reach D has a potential for seepage. There is a pocket of loose well graded silty to clean sand 15 to 20 feet below the levee crest. Further borings would have to be made to determine the extent of this problem. Standing water was noted inland of this section of the levee on a field trip last spring, which may be attributed to underseepage.

CONSTRUCTION MATERIALS

41. Soils suitable for impermeable levee cores are not readily available in close proximity to the project due chiefly to the densely populated nearby countryside having preempted possible source areas, and leaving little low cost open land suitable for borrow sources. Will County to the immediate south and west amd DuPage County areas to the northwest offer the closest available sources of cohesive materials. Haul distances of 10 miles are likely. Rock suitable for riprap is readily available in nearby McCook Quarry within 2-3 miles of the site. Limited available supply of the proper stone gradation may result in higher costs.

BENEFIT DETERMINATION INVOLVING EXISTING LEVEE

GENERAL

42. The ensuing analysis was extracted from a Planning Draft EC 1105-2-XXX which expired 31 October 1985; however, it still provides an engineering approach to evaluating the level of protection of an existing non-Federal levee for use in the benefit-cost analysis.

43. The McCook levee performance record prior to 1980 contains numerous accounts of levee embankment failures and high damages which prompted immediate action be taken to preclude future failures by MSD. After a preliminary soils investigation by Walter H. Flood, MSD personnel designed a steel sheet pile cut-off embedded into the riverward shoulder of the levee crown, station 0+00 to about station 41+00. River water levels have since risen to that experienced in the 1979 flood, and the existing levee has not had a major embankment failure, with exception of some land side water ponding. This can be attributed to either embankment seepage or interior drainage or possibly both. However, this water level only represents the 7.5 year flood and does not consider the 100-year design flood stage. Because the levee has been stable to-date since construction of the piling it does provide some degree of protection and therefore some benefits.

RELIABILITY

44. A determination of the benefit of the existing levee as predicted on engineering judgement, can be made based on the reliability of the levee to sustain various flood level elevations. Where there are many unknown parameters that effect the performance of an existing non-Federal levee some procedure must be used to determine the effectiveness of a given levee from a standpoint of economic value of the structure in a "without project" condition. If the integrity of the levee from an engineering view point can be reasonably established for at least two conditions of the levee, then a straight line assumption can be made for "reliability" versus "water surface elevation". This is based on the fact that the higher the water surface elevation on an embankment the greater the induced load with consequently greater chance for failure. In the engineering judgment by this procedure such things as stage-duration and repeated flood level stages at close intervals are also considered, in addition to as many parameters regarding the levee embankment and its foundation as are available.

45. In accordance with the draft EC the two points on the straight line shown on figure C-21 are defined as follows:

a. 1st point known as "PNP" - This represents the highest vertical elevation on the levee such that it is "highly likely" that the levee would not fail if the water surface were to reach that point.

b. 2nd point known as "PFP" - This is the lowest vertical elevation on the levee such that it is "highly likely" that the levee will fail.

c. "Highly likely", is defined as a 85+ percent confidence level.

46. By definition the PNP will be at a lower water surface elevation than the PFP. Where unresolved differences of opinion exist, the range of uncertainty extends from the lower of arguable PNP's to the higher arguable PFP's. This in effect says that the two points can be established on a logical basis for levee failure.

BENEFIT EVALUATION PROCEDURE

47. The preceding paragraphs discuss a general explanation of the benefit determination and is based on the fact that while an existing levee may afford little protection, it does none-the-less provide some degree of benefits. These benefits must be arbitrarily taken when definitive engineering judgment becomes difficult or becomes unreasonable to predict. If the probability distribution for levee failure is not known, then a straight line distribution known as a "default form" can be presumed. The straight line graph allows intermediate probabilities to be obtained for a given water surface elevation and the resulting damages determined. "At and below the PNP the existing levee will be assumed to be fully effective for benefit evaluation; i.e., a reliability of 100 percent. At and above the PFP no damage prevention by the existing levee shall be surmised; i.e., a reliability of zero."

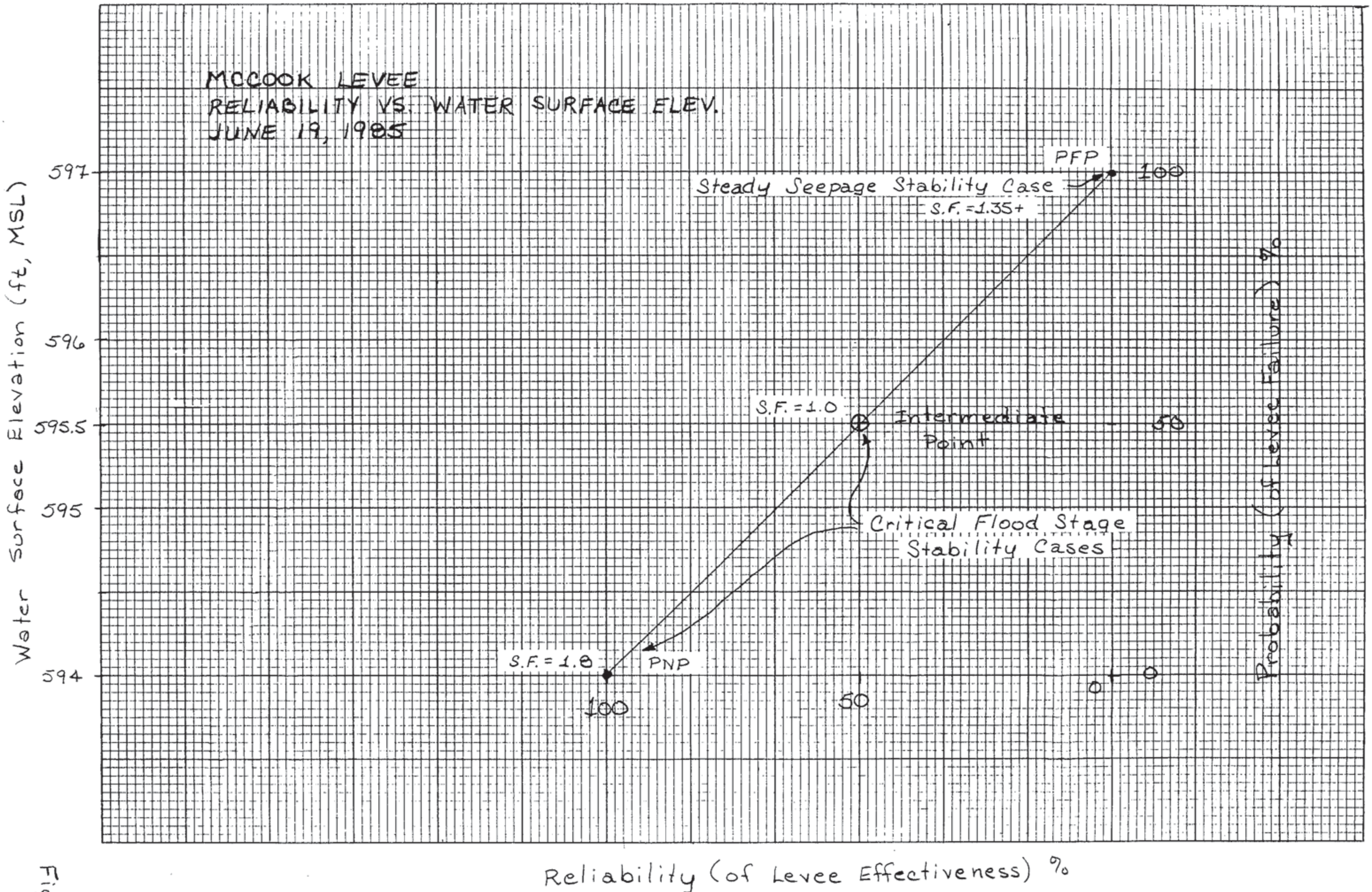


Figure C-21

Reliability of Levee Effectiveness vs Water Surface Elevation
McCook Levee

Figure C-21

DETERMINATION OF THE PNP AND PFP.

48. The open circle shown on Figure C-21 was determined by the methods of stability analysis as described in paragraph 28. The stability case involved herein is defined as the "Critical Flood Stage", wherein the water surface elevation is varied on the riverward levee slope until the lowest safety factor is obtained. This provided a safety factor of 1.01 for a water surface elevation of 595.5. At a water surface elevation of 593.5, the corresponding safety factor increased to 1.8. For a water surface elevation of 596 the factor of safety was found to be 1.6. The accepted minimum factor of safety for this condition is 1.4. The other condition for analyzing the riverward slope is the "sudden drawdown" case; however, this was not considered appropriate since the stage-frequency curve indicates slowly falling water for mid-embankment elevations which the outer soil slope could probably adjust to, partly due to the steel sheet piling and soil types. The normal pool elevation for the Des Plaines River ranges between 590 and 593.

49. As a means of evaluating the landside levee slope for stability, the case of "Steady Seepage" from full pool stage using a "partially develop phreatic surface" was used. Although it may be argued that the stage-frequency curve does not indicate that extremely high water stages would be sustained long enough to completely saturate the embankment, which is a requirement for this case. This case likewise requires a minimum safety factor of 1.4. The river stage on the riverward levee slope was assumed at elevation 600 which is close to the high water mark established by the flood of 3-9-79 that reached elevation 599.06. Two different phreatic lines were evaluated; a high straight line from 600 elevation to tailwater in the drainage ditch, and a lower broken line that considered the effect of the existing sheet piling. The higher line produced a safety factor of 1.11 and the lower line a safety factor of 1.35. The minimum safety factor required for this case is 1.4. As a consequence, it was decided to make the "PFP" point at elevation 597.0. By dropping the river stage to elevation 598 and assuming the lower phreatic line a safety factor of 1.4 would be achieved. It must be re-emphasized that at and above the PFP elevation there is a 100 percent "probability" for failure of the existing levee or the "effectiveness" of the levee is zero. For the PNP elevation and below the "probability" of failure is zero or the levee "effectiveness" is 100 percent which is why elevation 594 was selected. The intermediate point was established as the mid-point on the slope of the straight line and represents that confidence factor wherein the levee has a fifty-fifty chance of either failing or standing with a water surface at elevation 595.5. At this water surface elevation the lowest factor of safety was found to be unity.

50. The selection of these points as based on engineering judgement is argumentative; as to the range of water surfaces selected, scale of the "reliability" used, and the slope of the line determined. This analysis is basically an economic approach to determining the benefits derived from the existing McCook levee without predicting absolutely, under what conditions of water loading the levee will fail.

CONCLUSIONS

51. The McCook Levee between Lawndale Avenue and 47th Street is approximately fifteen feet high and appears to have been constructed of four types of fill; organic silty clay, inorganic silty clay, rubble fill and silty clayey sand. It appears most of the material was excavated from the adjacent Des Plaines River channel and placed with varying degrees of compactive effort to form the levee. Foundation preparation, that is removal of organic matter and extremely weak soil, if performed, was minimal.

52. The crest of the levee varies in elevation from approximately 599 to 604 feet NGVD. The levee crown is, in the lower area, three feet below the 100-year flood level. The depressed reach of the levee appears to be associated with the region where the fill is predominately organic silty clay overlying five feet of soft clay. The levee crown appears to have settled approximately three feet since construction was completed. The soft foundation soil and soft organic fill appears to have consolidated and although still weak, these materials are no doubt stronger now than at the time of construction.

53. In the area of the 1979 breaches the borings reveal the embankment fill to be a moderately well compacted silty clay. It is not known whether this material was placed to reconstruct the levee in 1979 or possibly following an earlier breach. The 1979 breach area now appears to be the most competent reach of the levee as the materials reflect a much higher shear strength.

54. The steel sheet pile cutoff wall erected on the basis of the 1979 borings does not penetrate deep enough or extend far enough to cutoff critical weak and permeable soil zones. The levee construction itself is not up to Corps standards both as to quality of soil materials, construction and being generally below minimum desired elevations, some of which is due to the settlement and consolidation of the unsuitable fill materials in some sections and overtopping erosion in other areas.

55. The embankment materials are generally saturated below elevation 590; the approximate normal river level. The shallow, weak layers of foundation soil are typically saturated and appear to be hydrogeologically connected with the river. The deeper denser foundation soils are of very low permeability and are estimated to be only partially saturated. Groundwater does not appear to be moving downward through these layers. The upper zone of bedrock appears to be only partially saturated. The river water and the groundwater trapped in the voids of the embankment fill and near surface foundation layers appears to be perched except where the dense silty sand and very stiff clay is locally absent.

56. The foundation soils immediately below the levee fill consist of soft organic clays, loose silty sand and interbedded sand, gravel and clay layers. These weaker and more permeable foundation soils are on the order of three feet to ten feet thick, and are underlain by more competent soils consisting of very stiff to hard silty clay and extremely dense silty fine sand. These more competent soils are on the order of four to more than ten feet thick and are resting on bedrock. The bedrock below these foundation soils is severely weathered to slightly weathered dolomitic limestone.

RECOMMENDATIONS

57. Prior to preparation of the Detailed Project Study, several additional borings should be made and a limited laboratory testing program completed. This would be required to verify present assumptions, particularly reaches where data are not available, such as Reach D. When the final centerline of the levee has been established and both slope toe-traces set, in addition to the approved top-of-protection and freeboard, another check will be made on the most critical levee section. It is envisioned at this point, that several levee sections will be required along the 4,000-foot of rehabilitated levee. The crown of the levee must be wide enough to allow emergency vehicle travel, including a "turn-around" and possible "access ramps." All levee slopes will require minimum slopes to allow for maintenance equipment. Based on the preliminary stability analysis presented in this appendix, the riverward slope should be 2H on 1V and the land-side slope 2H on 1V, for Reach B, and a 5H on 2V riverside and 3H on 1V land side for Reach C. Should underseepage prove to be a problem, then landside seepage berms may be required or pressure relief wells. For Reach A, it may be necessary to grout along the levee crown to form a positive cut-off against "thru-seepage" or provide a riverward impervious clay membrane.

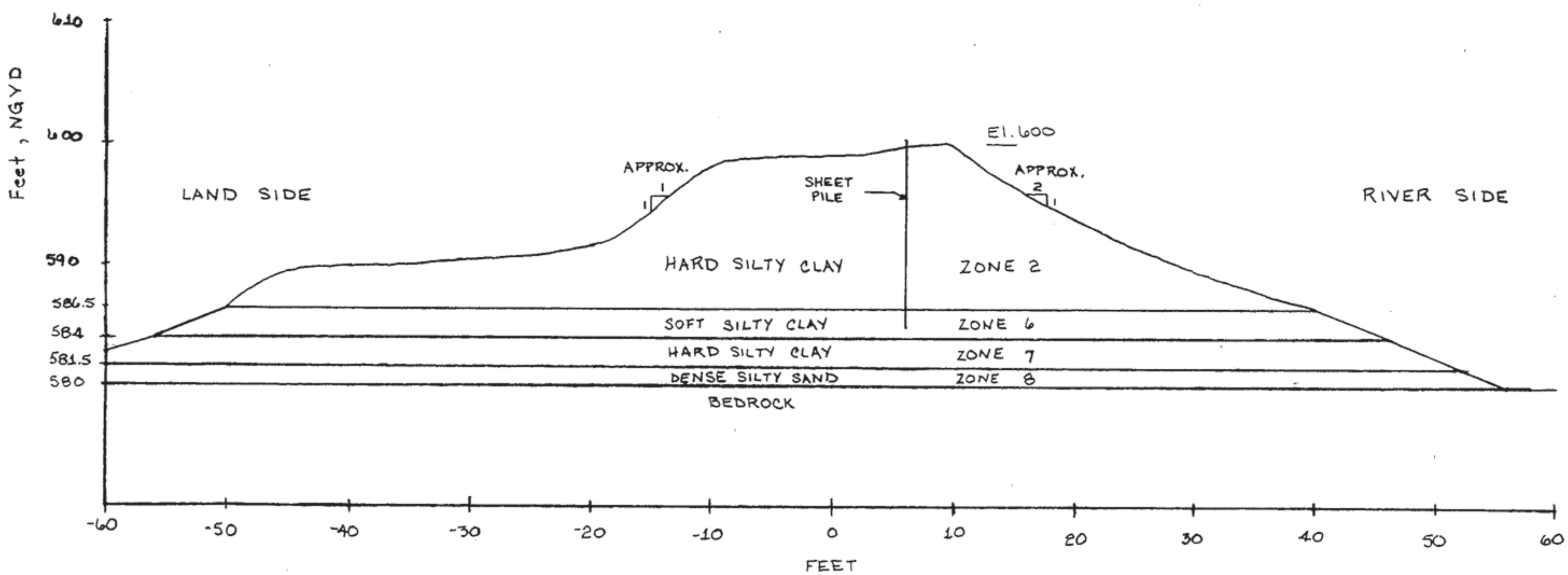
LIST OF REFERENCES

1. Patrick Engineering, Inc., Geotechnical Investigation McCook Levee, McCook, Illinois, March 1985 (available at Chicago District Office).

2. Bowles, Joseph E, 1982, Foundation Analysis and Design, McGraw Hill, New York.

3. Naval Facilities Engineering Command Design Manual, NAVFAC DM-7, Design Manual, Soil Mechanics, Foundations, and Earth Structures, Department of the Navy, 1971.

4. EM 1110-2-1913, 31 March 1978, Design and Construction of Levees, Department of the Army.



Feet , NGYD
610
600
590
586.5
584
581.5
580
LAND SIDE
APPROX.
1
1
SHEET
PILE
El. 600
APPROX.
2
1
RIVER SIDE
HARD SILTY CLAY
ZONE 2
SOFT SILTY CLAY
ZONE 6
HARD SILTY CLAY
ZONE 7
DENSE SILTY SAND
ZONE 8
BEDROCK
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-50
-40
-30
-20
-10
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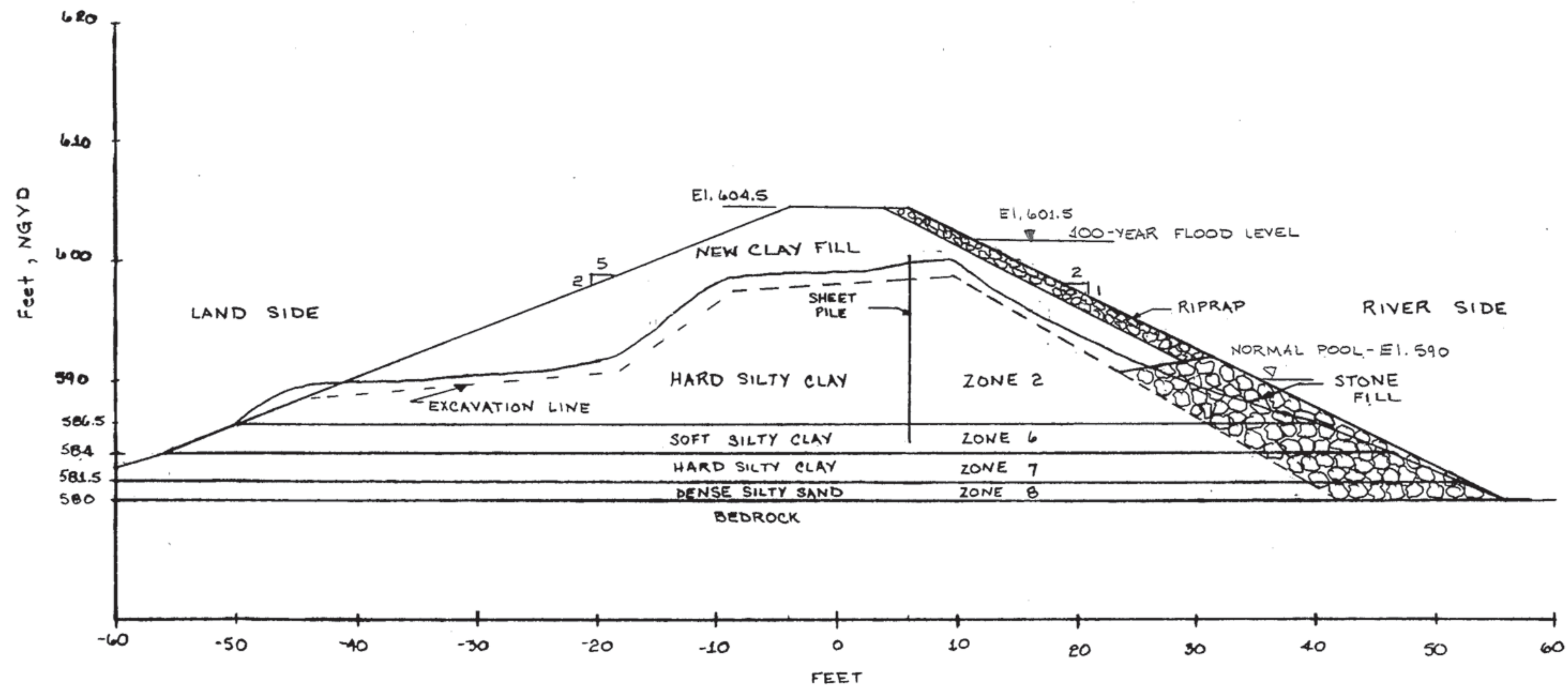
10
20
30
40
50
60
FEET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El. 601.5
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2
2
1
RIPRAP
RIVER SIDE
LAND SIDE
SHEET PILE
NORMAL POOL - El. 590
HARD SILTY CLAY
ZONE 2
STONE FILL
EXCAVATION LINE
SOFT SILTY CLAY
ZONE 6
HARD SILTY CLAY
ZONE 7
DENSE SILTY SAND
ZONE 8
BEDROCK
Feet, NGVD
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610
600
590
586.5
584
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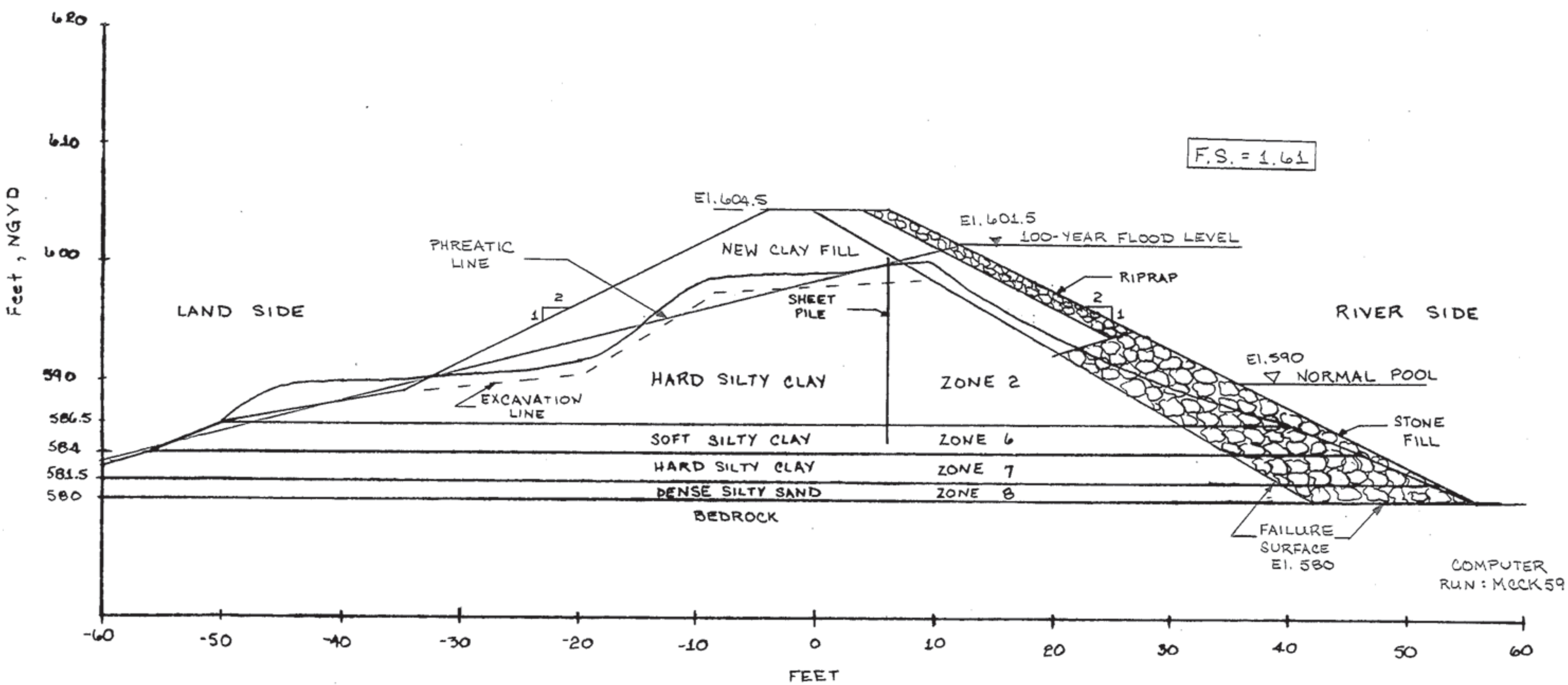
0
10
20
30
40
50
60
FEET



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600
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586.5
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580
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RIVER SIDE
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El. 601.5
100-YEAR FLOOD LEVEL
RIPRAP
PHREATIC LINE
NEW CLAY FILL
SHEET PILE
HARD SILTY CLAY
ZONE 2
El. 590
NORMAL POOL
EXCAVATION LINE
SOFT SILTY CLAY
ZONE 6
HARD SILTY CLAY
ZONE 7
DENSE SILTY SAND
ZONE 8
BEDROCK
STONE FILL
FAILURE SURFACE
El. 580
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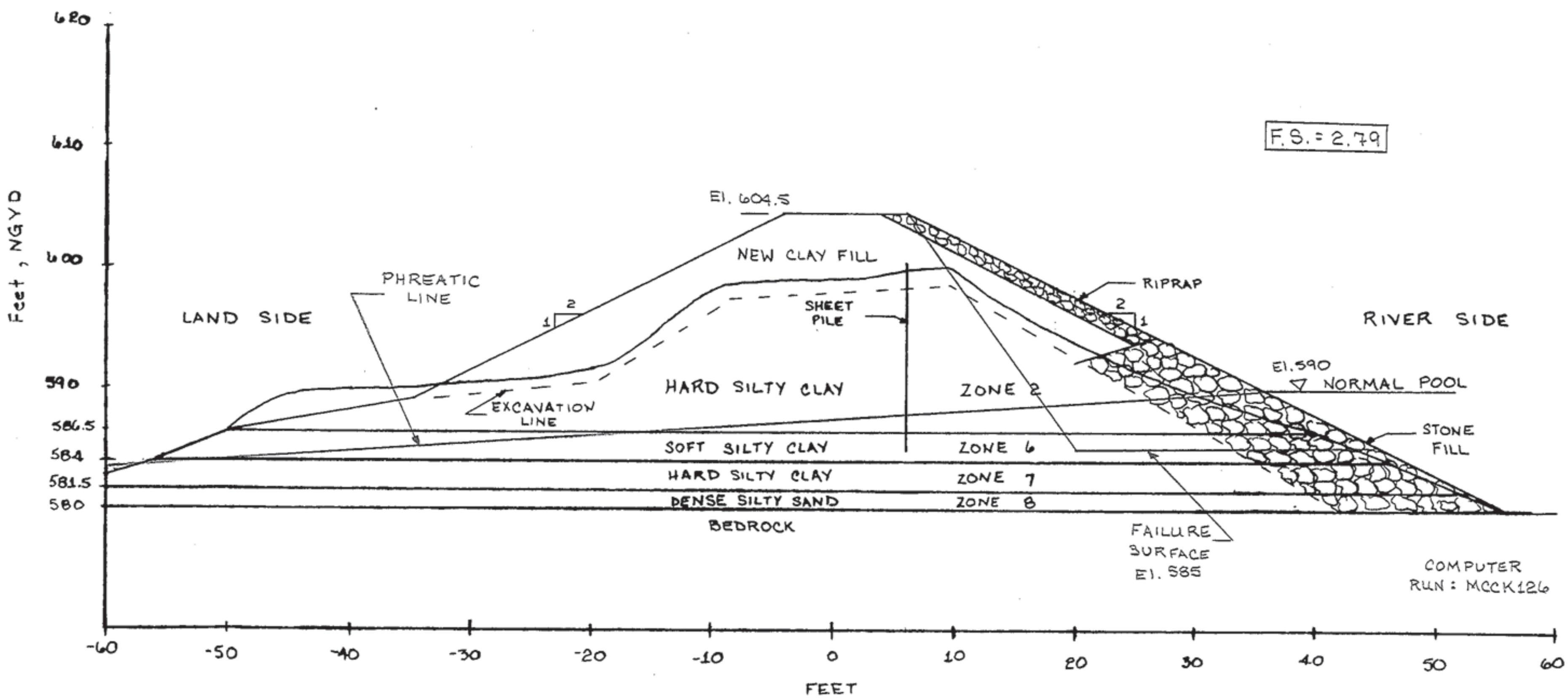
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ZONE 2
EXCAVATION LINE
SOFT SILTY CLAY
ZONE 6
HARD SILTY CLAY
ZONE 7
DENSE SILTY SAND
ZONE 8
BEDROCK
STONE FILL
FAILURE SURFACE El. 585
COMPUTER RUN: MCCK126
Feet, NGVD
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610
600
590
586.5
584
581.5
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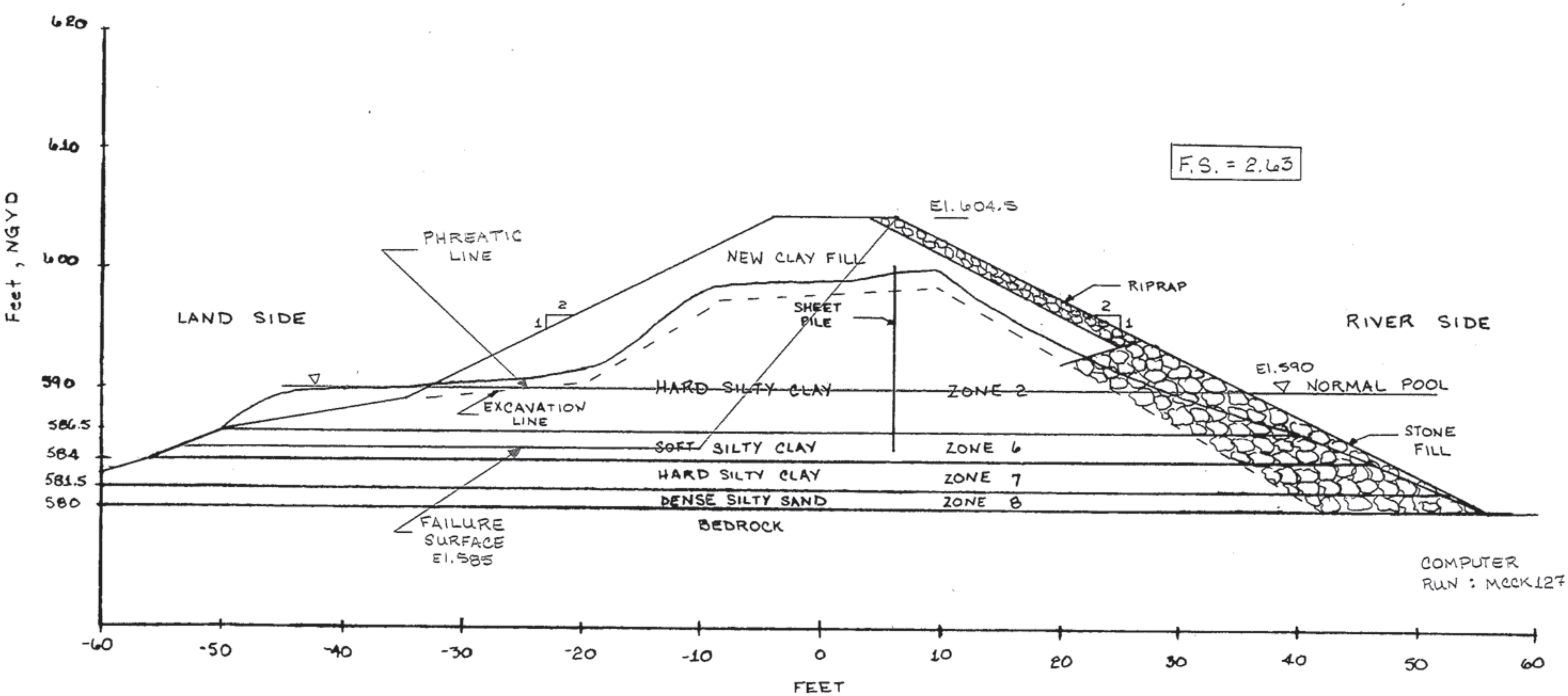
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ZONE 2
EXCAVATION LINE
STONE FILL
SOFT SILTY CLAY
ZONE 6
HARD SILTY CLAY
ZONE 7
DENSE SILTY SAND
ZONE 8
BEDROCK
FAILURE SURFACE El. 585
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Feet, NGYD
620
610
600
590
586.5
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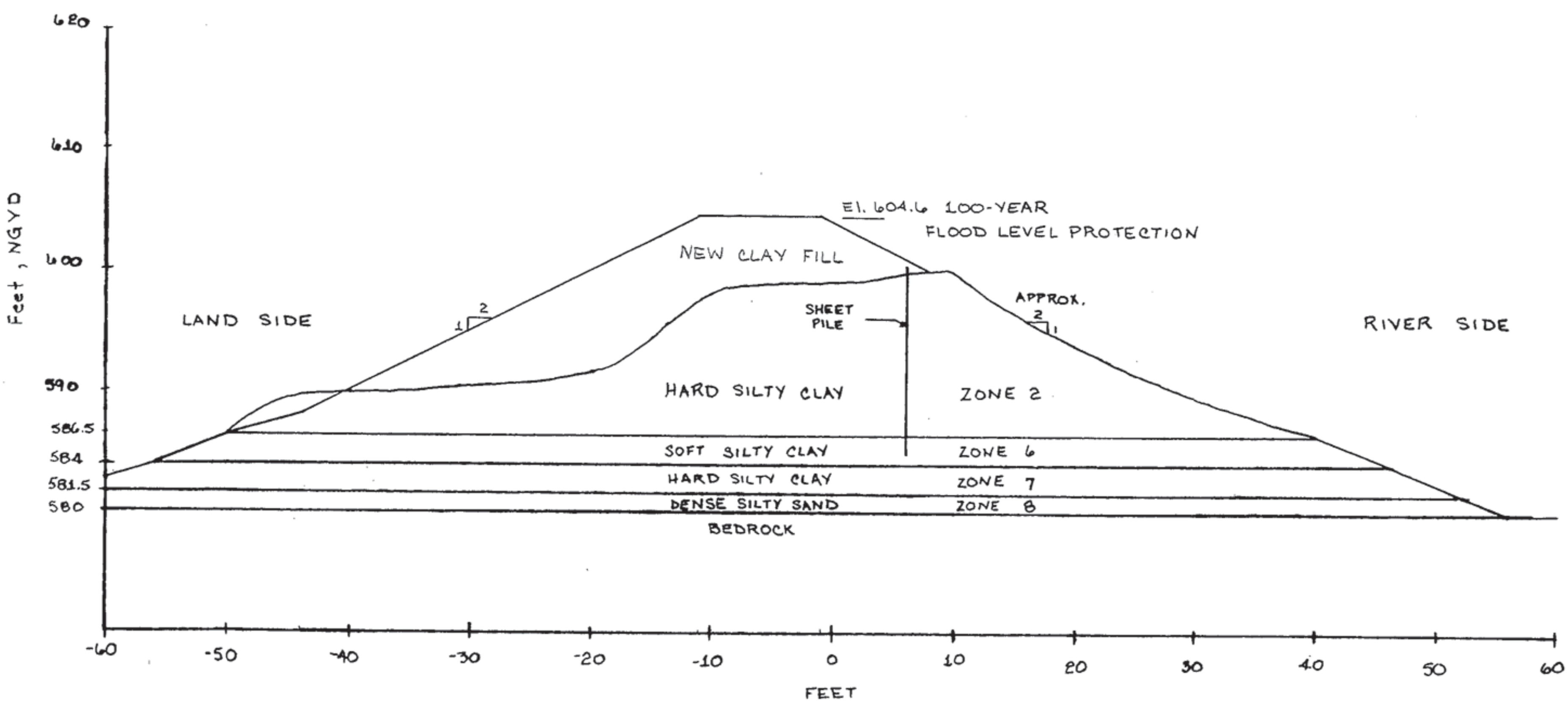
FEET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ZONE 2
SOFT SILTY CLAY
ZONE 6
HARD SILTY CLAY
ZONE 7
DENSE SILTY SAND
ZONE 8
BEDROCK
Feet, NGYD
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610
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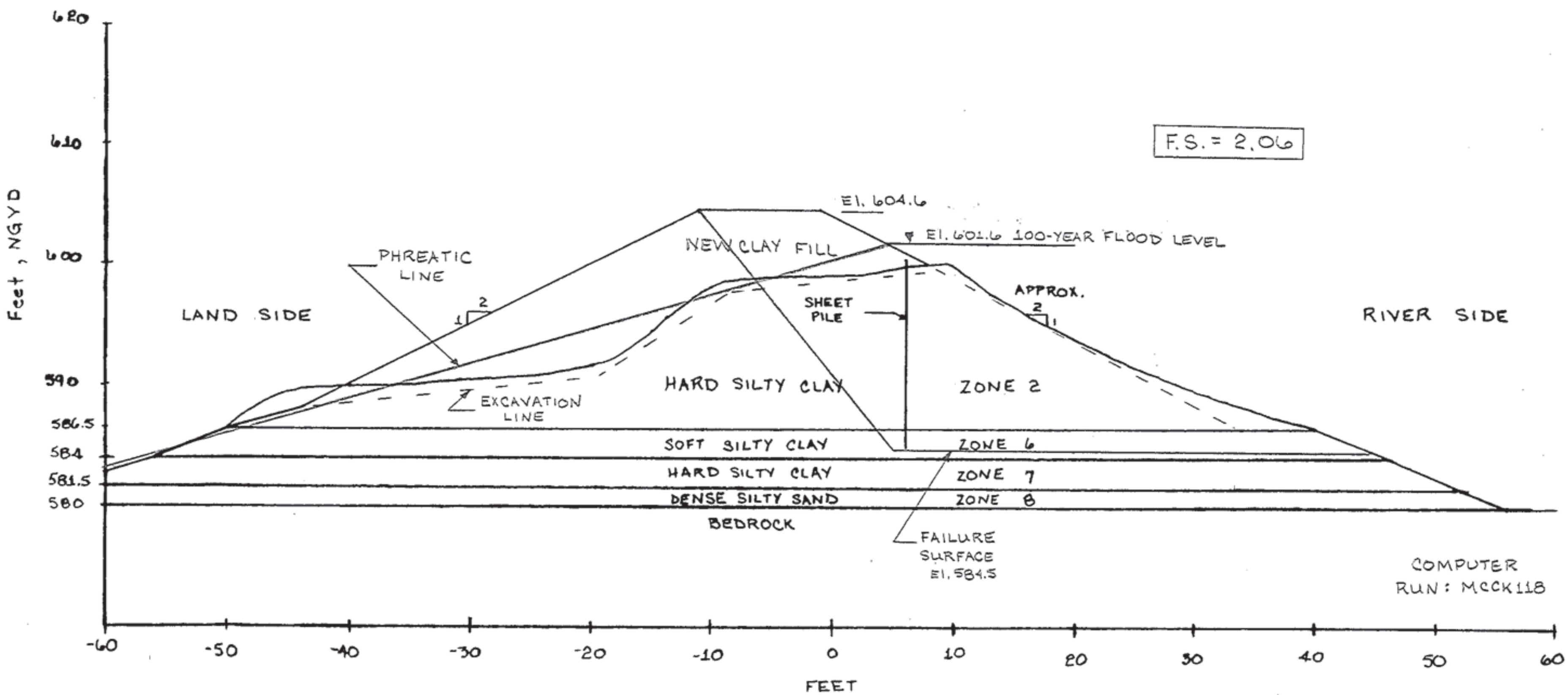
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El. 601.6 100-YEAR FLOOD LEVEL
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APPROX.
2
1
RIVER SIDE
HARD SILTY CLAY
ZONE 2
EXCAVATION LINE
SOFT SILTY CLAY
ZONE 6
HARD SILTY CLAY
ZONE 7
DENSE SILTY SAND
ZONE 8
BEDROCK
FAILURE SURFACE El. 584.5
COMPUTER RUN: MCCK118
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586.5
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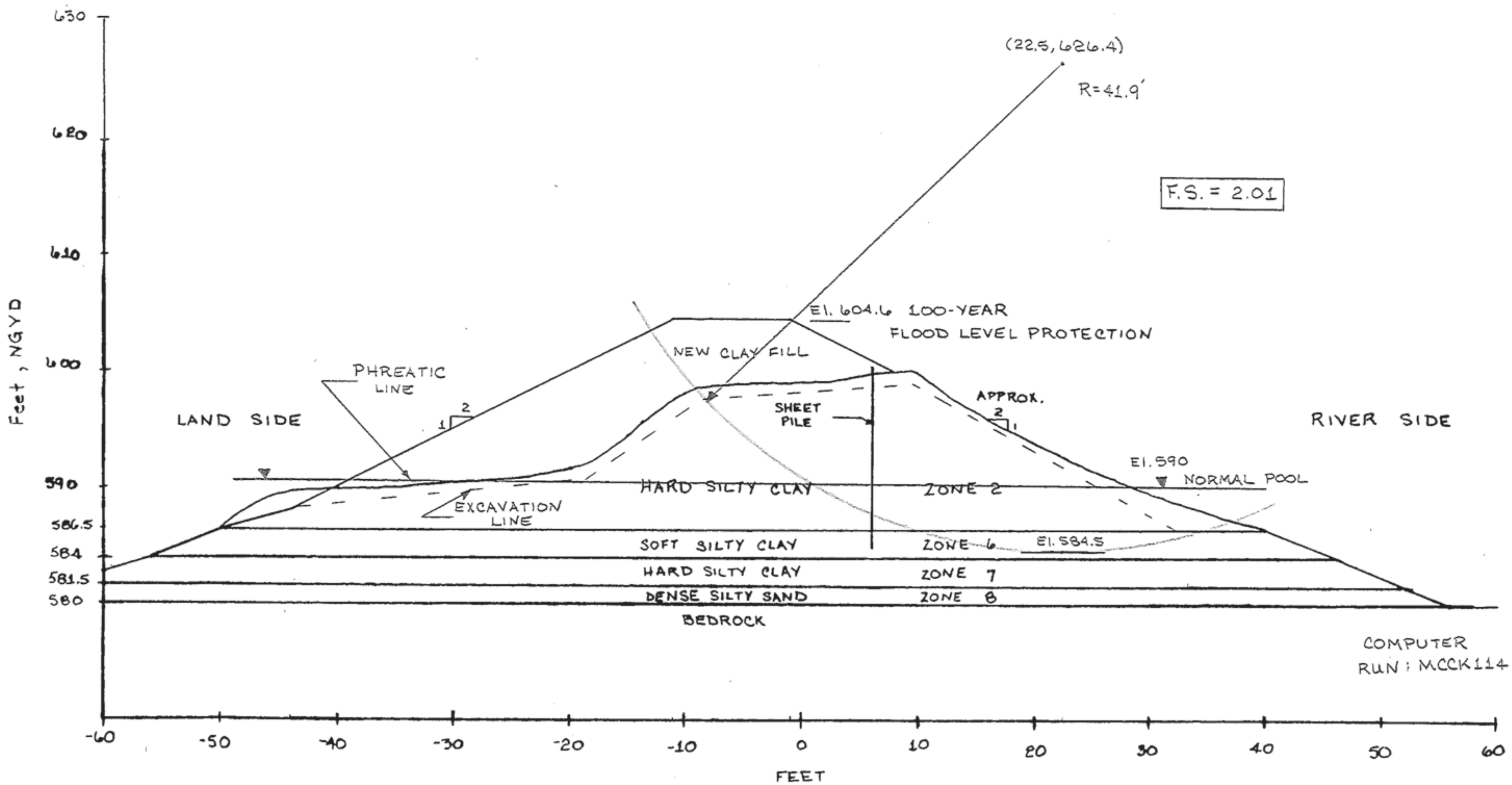
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FLOOD LEVEL PROTECTION
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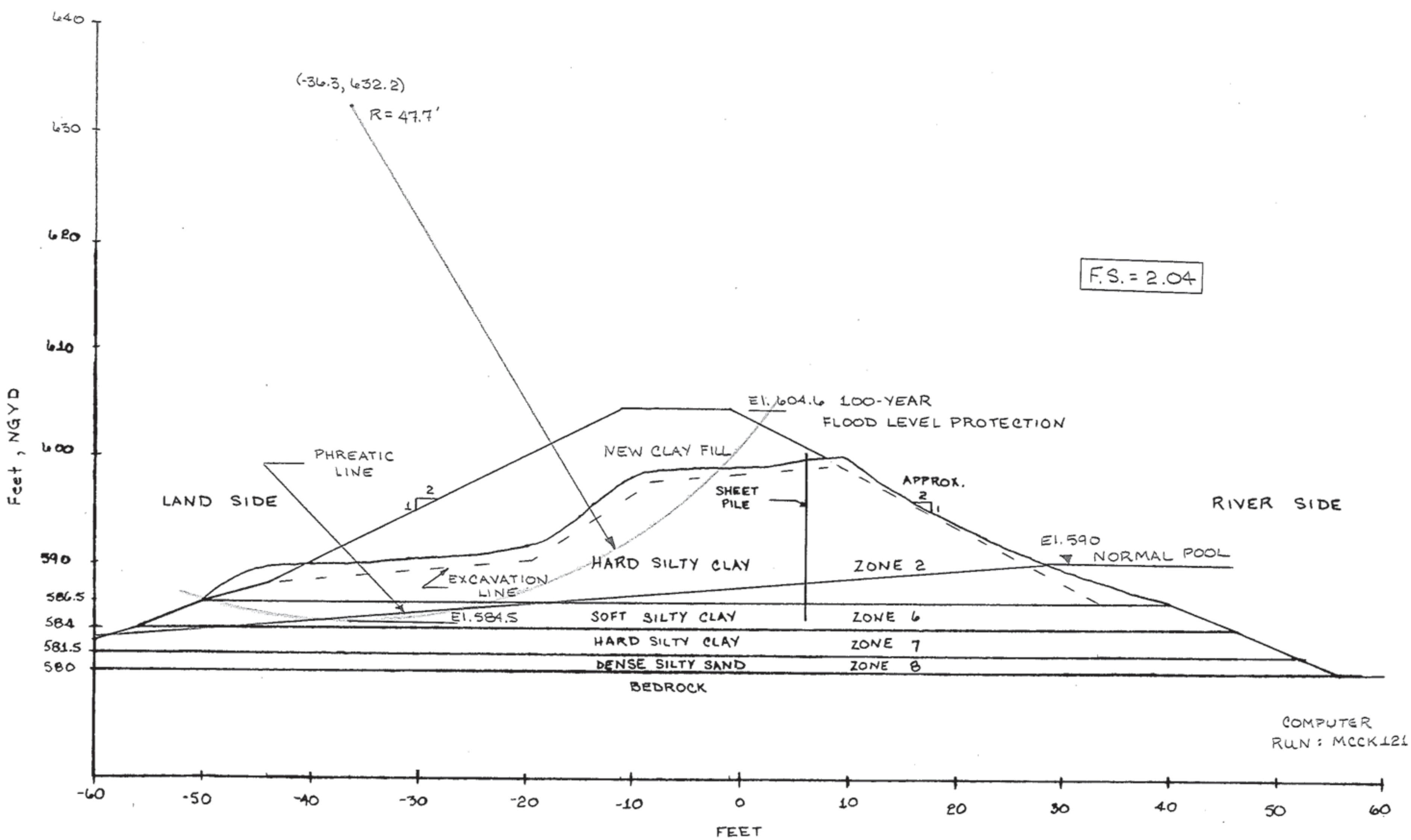
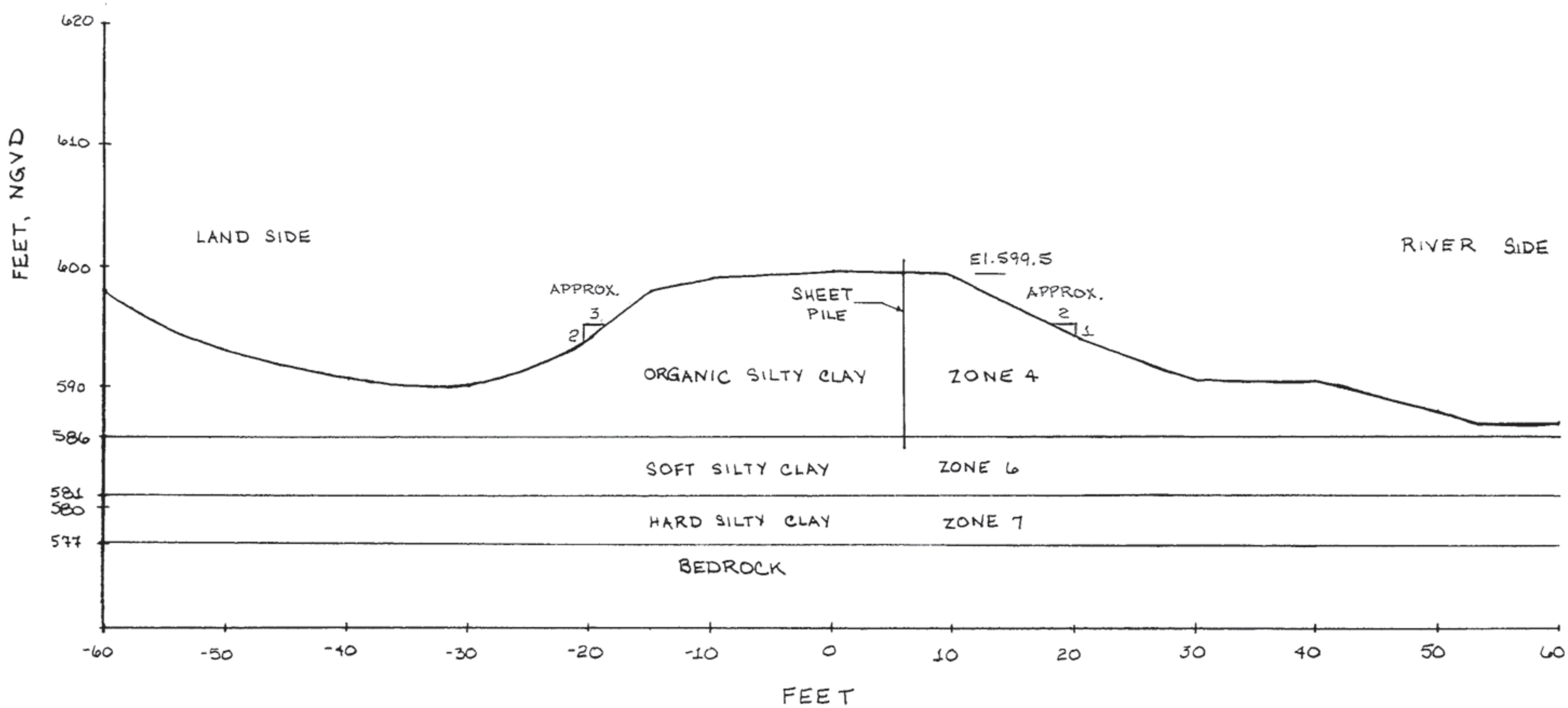
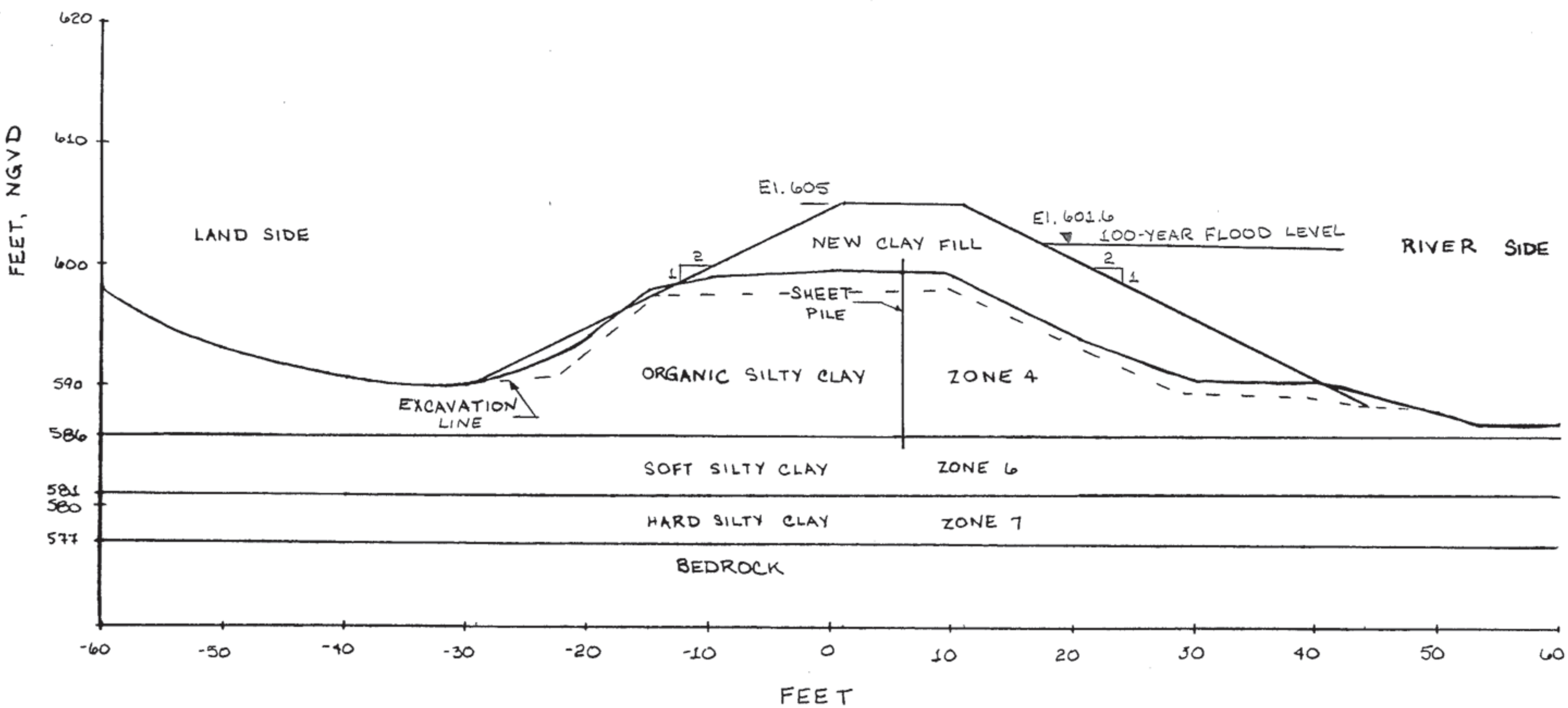
SHEET PILE
APPROX.
2
1
RIVER SIDE
El. 590
NORMAL POOL
HARD SILTY CLAY
ZONE 2
EXCAVATION LINE
SOFT SILTY CLAY
ZONE 6
El. 584.5
HARD SILTY CLAY
ZONE 7
DENSE SILTY SAND
ZONE 8
BEDROCK
COMPUTER
RUN: MCCK114
Feet, NGVD
630
620
610
600
590
586.5
584
581.5
580
-60
-50
-40
-30
-20
-10
0
10
20
30
40
50
60
FEET



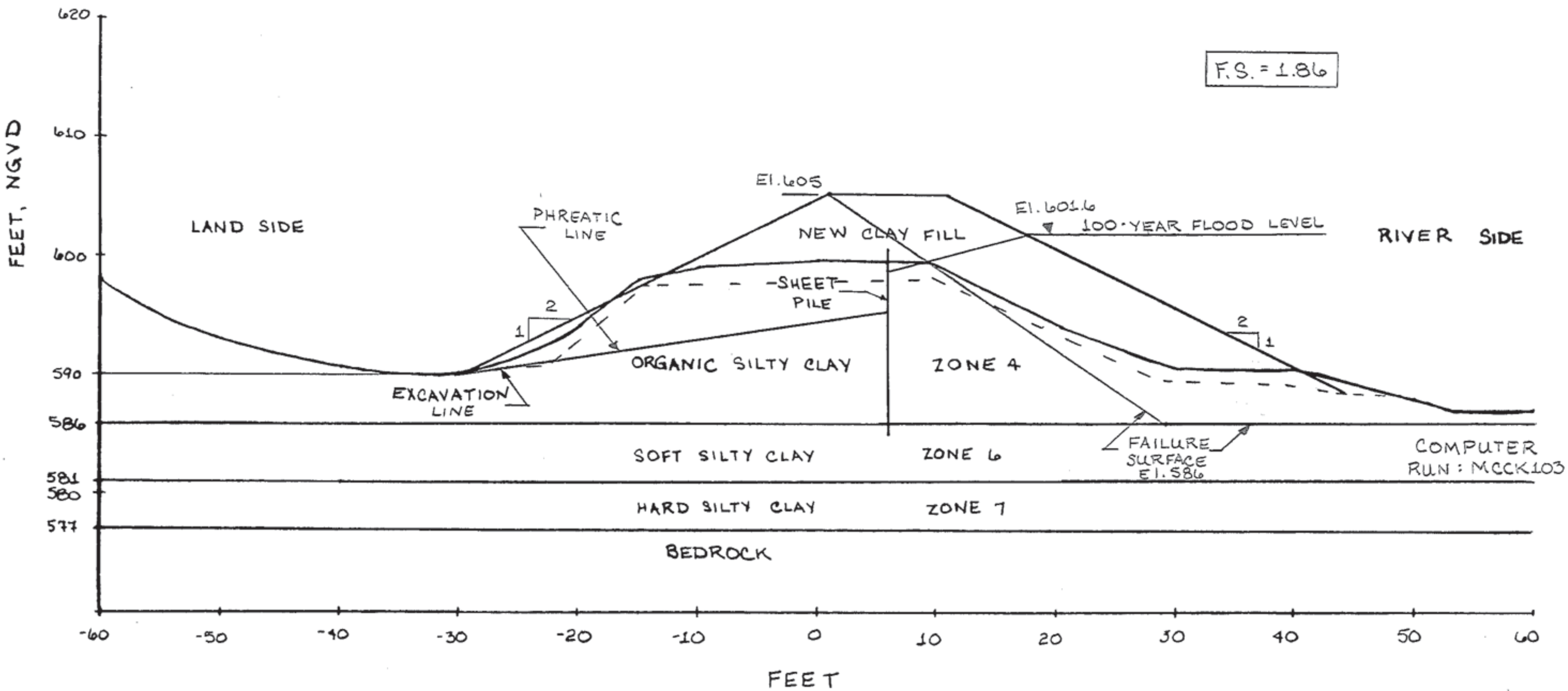
F.S. = 2.04
(-36.3, 632.2)
R = 47.7'
El. 604.6 100-YEAR
FLOOD LEVEL PROTECTION
NEW CLAY FILL
PHREATIC LINE
LAND SIDE
RIVER SIDE
SHEET PILE
APPROX.
2
1
El. 590
NORMAL POOL
HARD SILTY CLAY
ZONE 2
EXCAVATION LINE
El. 584.5
SOFT SILTY CLAY
ZONE 6
HARD SILTY CLAY
ZONE 7
DENSE SILTY SAND
ZONE 8
BEDROCK
COMPUTER RUN : MCCK121
Feet , NGVD
640
630
620
610
600
590
586.5
584
581.5
580
-60
-50
-40
-30
-20
-10
0
10
20
30
40
50
60
FEET



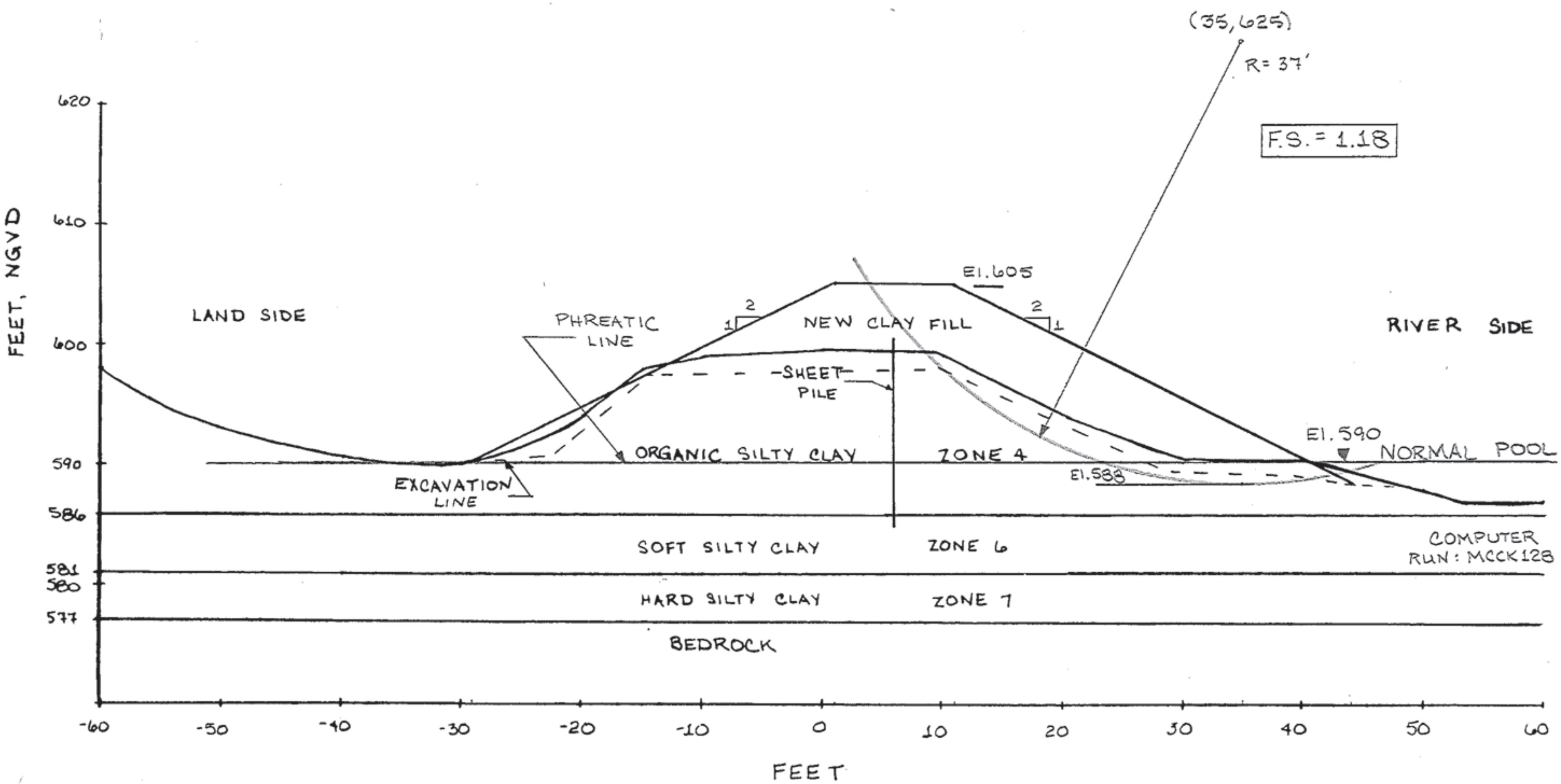
620
610
600
590
586
581
580
577
FEET, NGVD
LAND SIDE
RIVER SIDE
El.599.5
APPROX.
3
2
SHEET
PILE
APPROX.
2
1
ORGANIC SILTY CLAY
ZONE 4
SOFT SILTY CLAY
ZONE 6
HARD SILTY CLAY
ZONE 7
BEDROCK
-60
-50
-40
-30
-20
-10
0
10
20
30
40
50
60
FEET

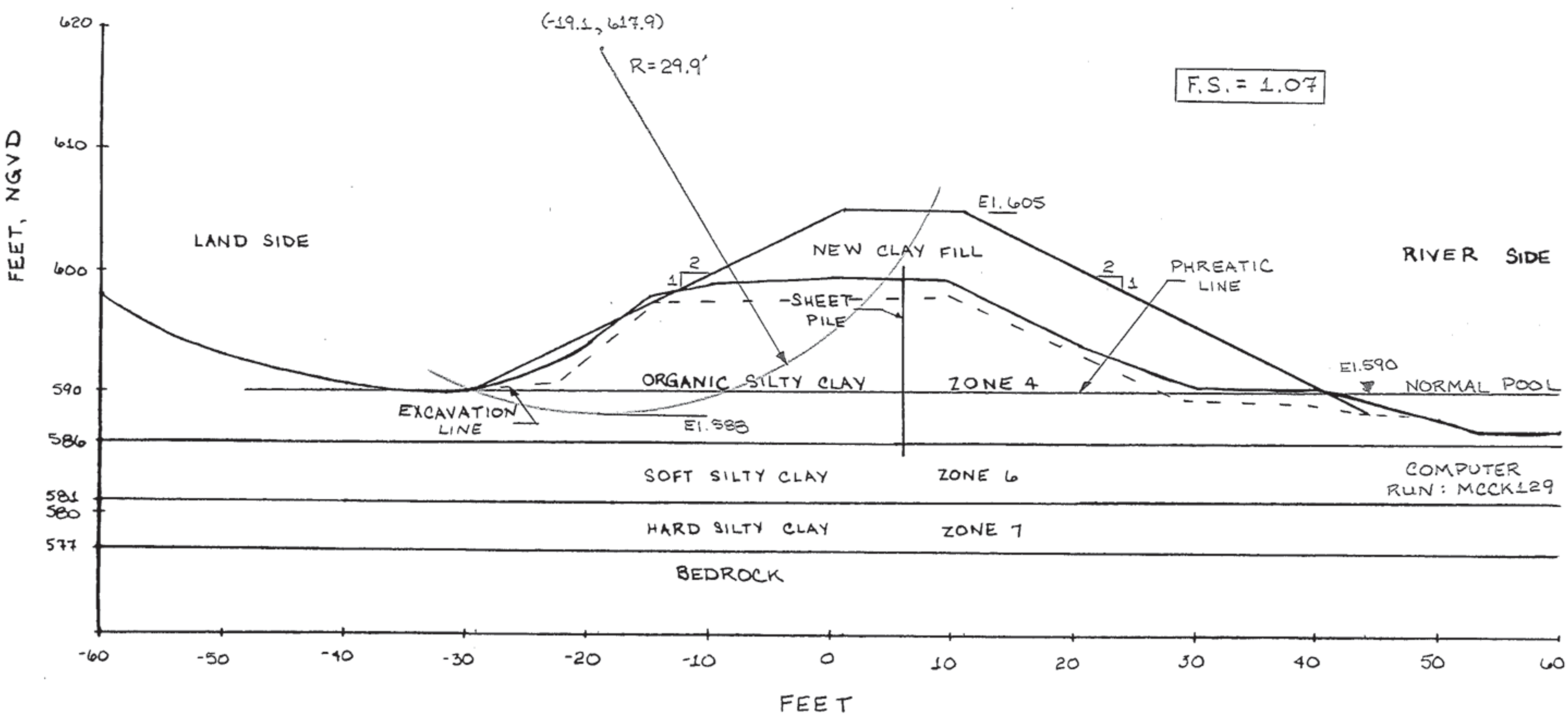


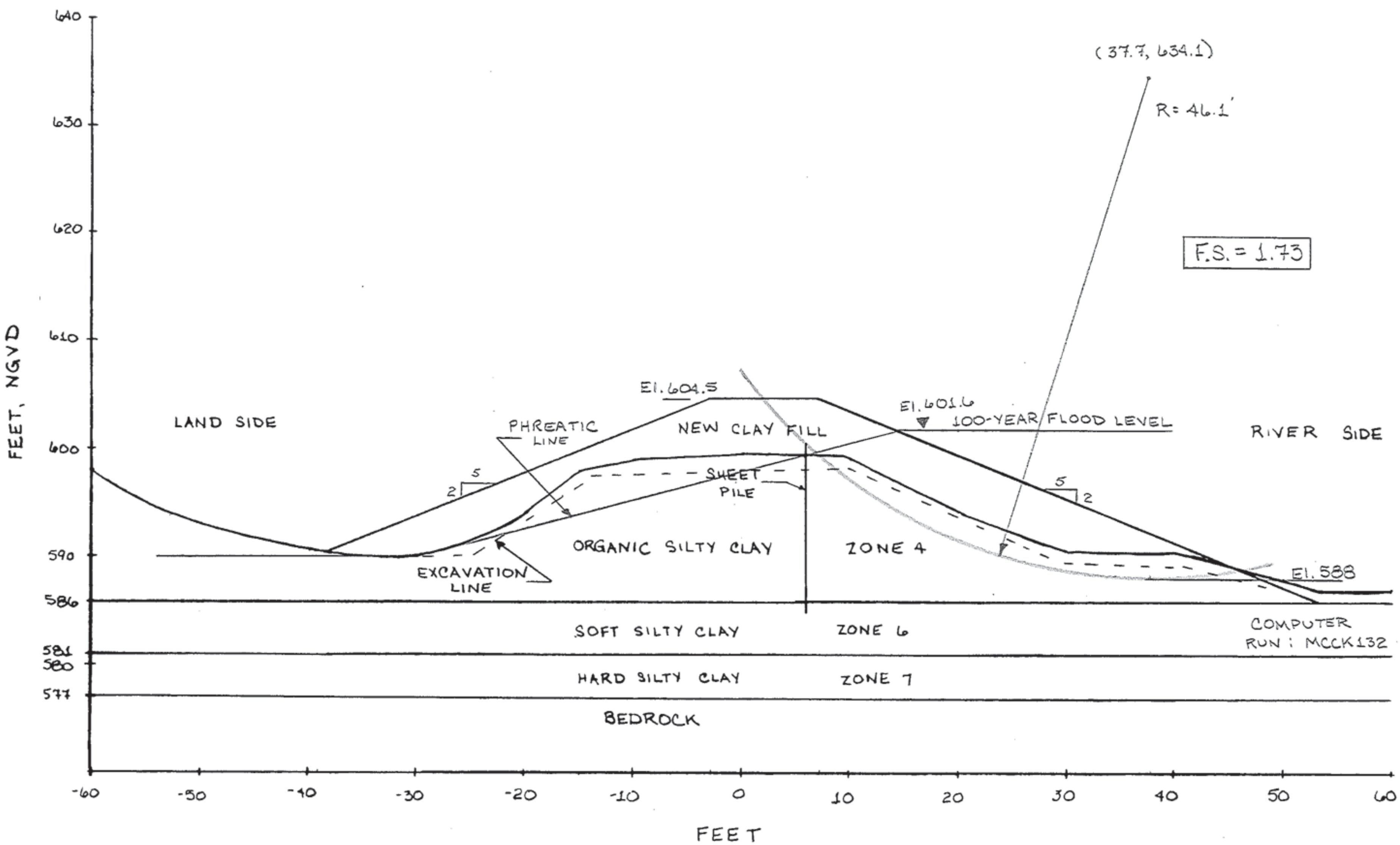
620
610
600
590
586
581
580
577
FEET, NGVD
LAND SIDE
El. 605
El. 601.6
100-YEAR FLOOD LEVEL
RIVER SIDE
NEW CLAY FILL
2
1
-SHEET-
PILE
ORGANIC SILTY CLAY
ZONE 4
EXCAVATION
LINE
SOFT SILTY CLAY
ZONE 6
HARD SILTY CLAY
ZONE 7
BEDROCK
-60
-50
-40
-30
-20
-10
0
10
20
30
40
50
60
FEET

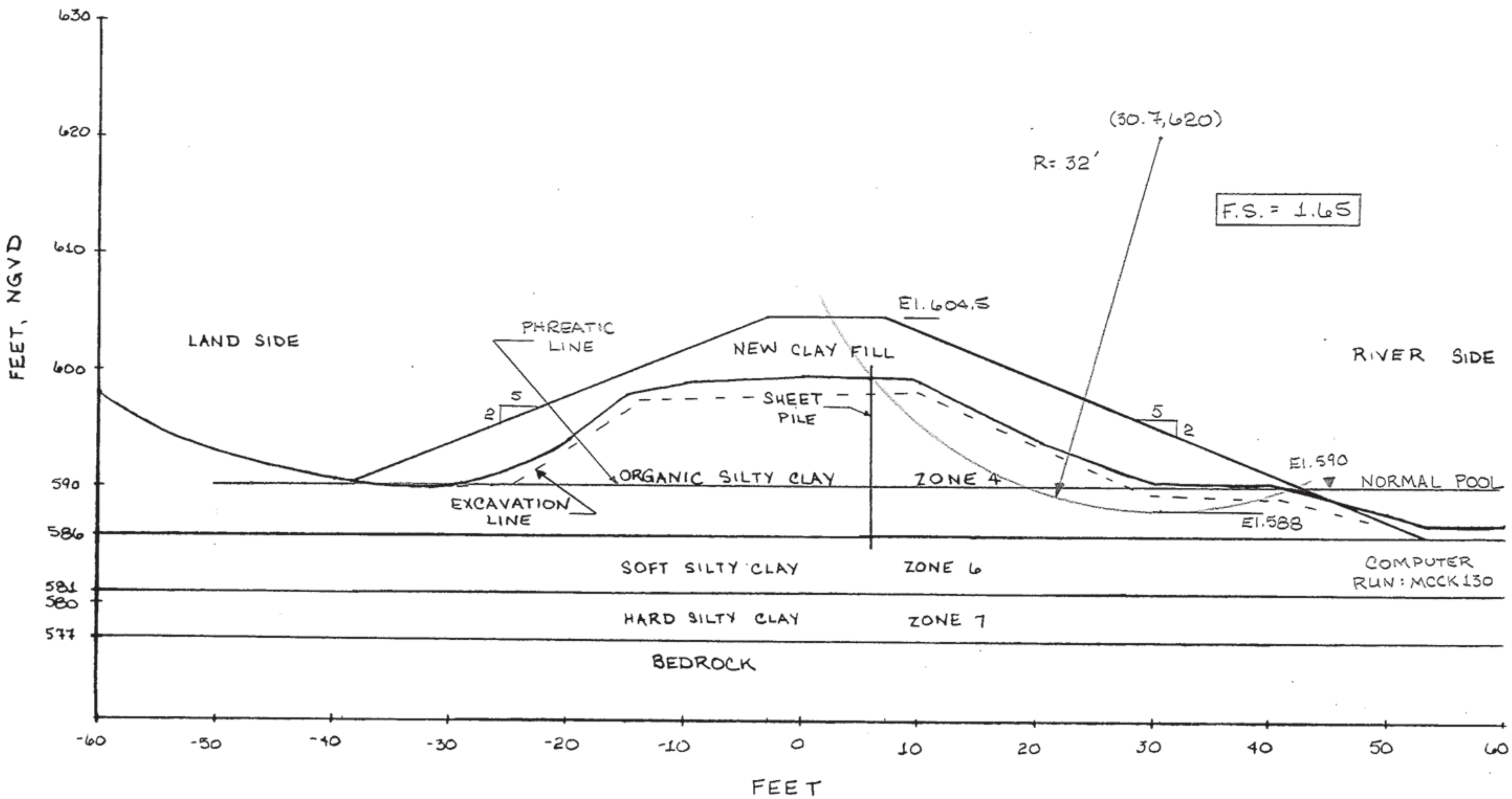


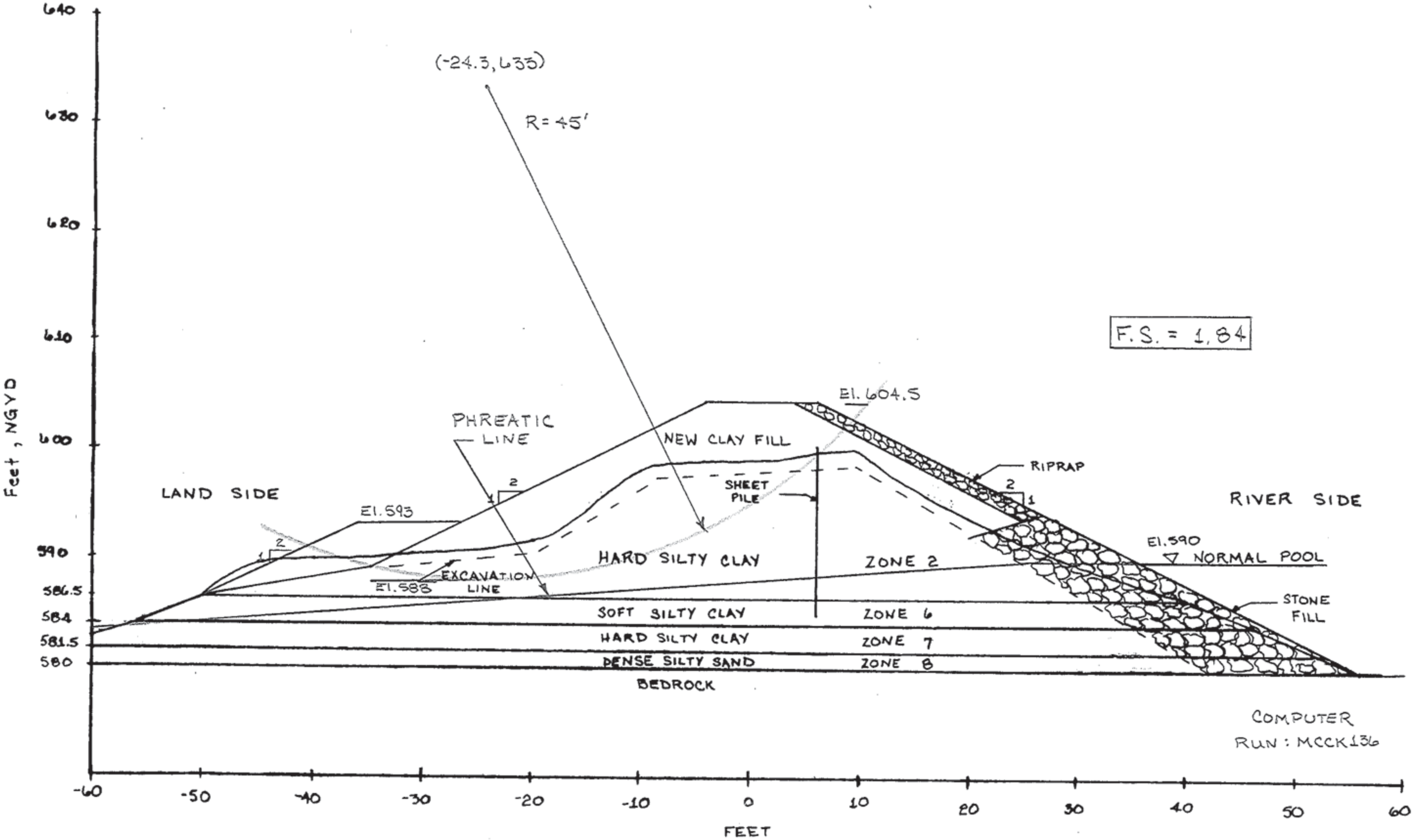
F.S. = 1.86
El. 605
El. 601.6
100-YEAR FLOOD LEVEL
LAND SIDE
RIVER SIDE
PHREATIC LINE
NEW CLAY FILL
SHEET PILE
ORGANIC SILTY CLAY
ZONE 4
EXCAVATION LINE
SOFT SILTY CLAY
ZONE 6
FAILURE SURFACE El. 586
COMPUTER RUN : MCCK103
HARD SILTY CLAY
ZONE 7
BEDROCK
FEET, NGVD
FEET
620
610
600
590
586
581
580
577
-60
-50
-40
-30
-20
-10
0
10
20
30
40
50
60


(35, 625)
R = 37'
F.S. = 1.18
El. 605
2
1
2
1
LAND SIDE
RIVER SIDE
PHREATIC LINE
NEW CLAY FILL
-SHEET- PILE
El. 590
NORMAL POOL
ORGANIC SILTY CLAY
ZONE 4
El. 588
EXCAVATION LINE
SOFT SILTY CLAY
ZONE 6
COMPUTER RUN: MCCK128
HARD SILTY CLAY
ZONE 7
BEDROCK
FEET, NGVD
620
610
600
590
586
581
580
577
-60
-50
-40
-30
-20
-10
0
10
20
30
40
50
60
FEET

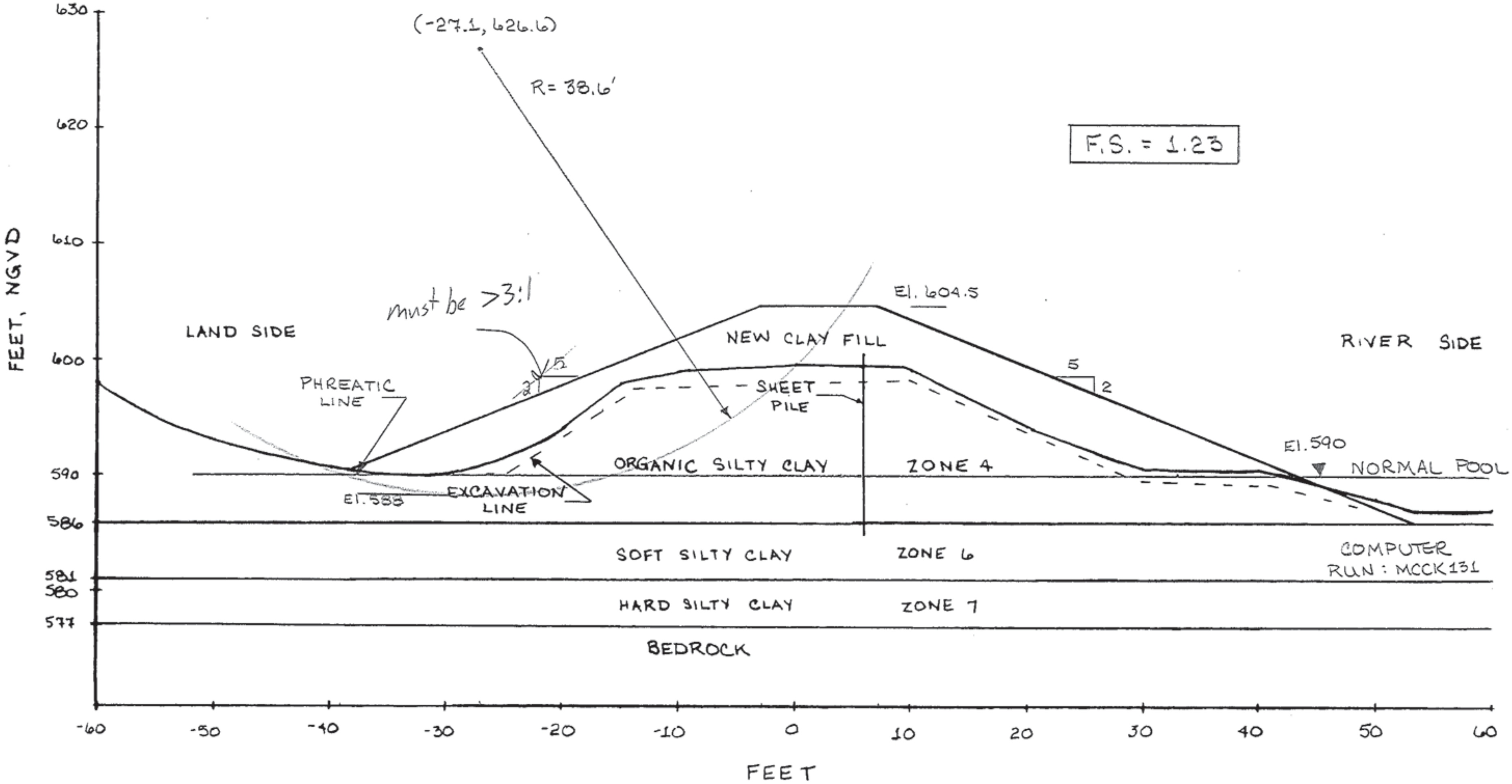

(-19.1, 617.9)
R=29.9'
F.S. = 1.07
LAND SIDE
RIVER SIDE
El. 605
NEW CLAY FILL
2
1
2
1
PHREATIC LINE
SHEET PILE
ORGANIC SILTY CLAY
ZONE 4
El. 590
NORMAL POOL
EXCAVATION LINE
El. 588
SOFT SILTY CLAY
ZONE 6
COMPUTER RUN: MCCK129
HARD SILTY CLAY
ZONE 7
BEDROCK
FEET, NGVD
620
610
600
590
586
581
580
577
-60
-50
-40
-30
-20
-10
0
10
20
30
40
50
60
FEET


(37.7, 634.1)
R = 46.1'
F.S. = 1.73
El. 604.5
El. 601.6
100-YEAR FLOOD LEVEL
LAND SIDE
RIVER SIDE
NEW CLAY FILL
PHREATIC LINE
SHEET PILE
5
2
ORGANIC SILTY CLAY
ZONE 4
EXCAVATION LINE
El. 588
SOFT SILTY CLAY
ZONE 6
COMPUTER RUN : MCCK132
HARD SILTY CLAY
ZONE 7
BEDROCK
FEET, NGVD
640
630
620
610
600
590
586
581
580
577
-60
-50
-40
-30
-20
-10
0
10
20
30
40
50
60
FEET

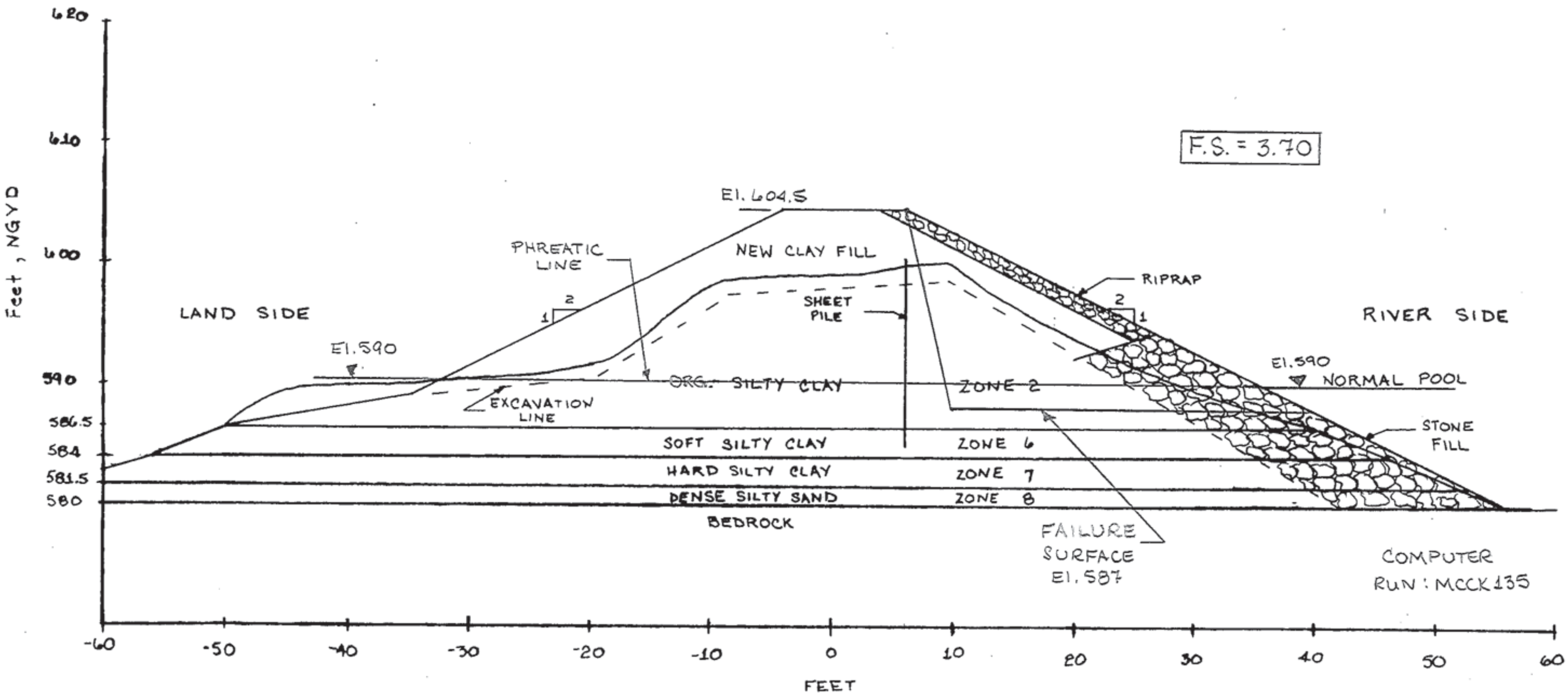

(30.7,620)
R= 32'
F.S. = 1.65
El. 604.5
PHREATIC LINE
LAND SIDE
NEW CLAY FILL
RIVER SIDE
SHEET PILE
ORGANIC SILTY CLAY
ZONE 4
El. 590
NORMAL POOL
EXCAVATION LINE
El. 588
SOFT SILTY CLAY
ZONE 6
COMPUTER RUN: MCCK130
HARD SILTY CLAY
ZONE 7
BEDROCK
FEET, NGVD
FEET
630
620
610
600
590
586
581
580
577
-60
-50
-40
-30
-20
-10
0
10
20
30
40
50
60

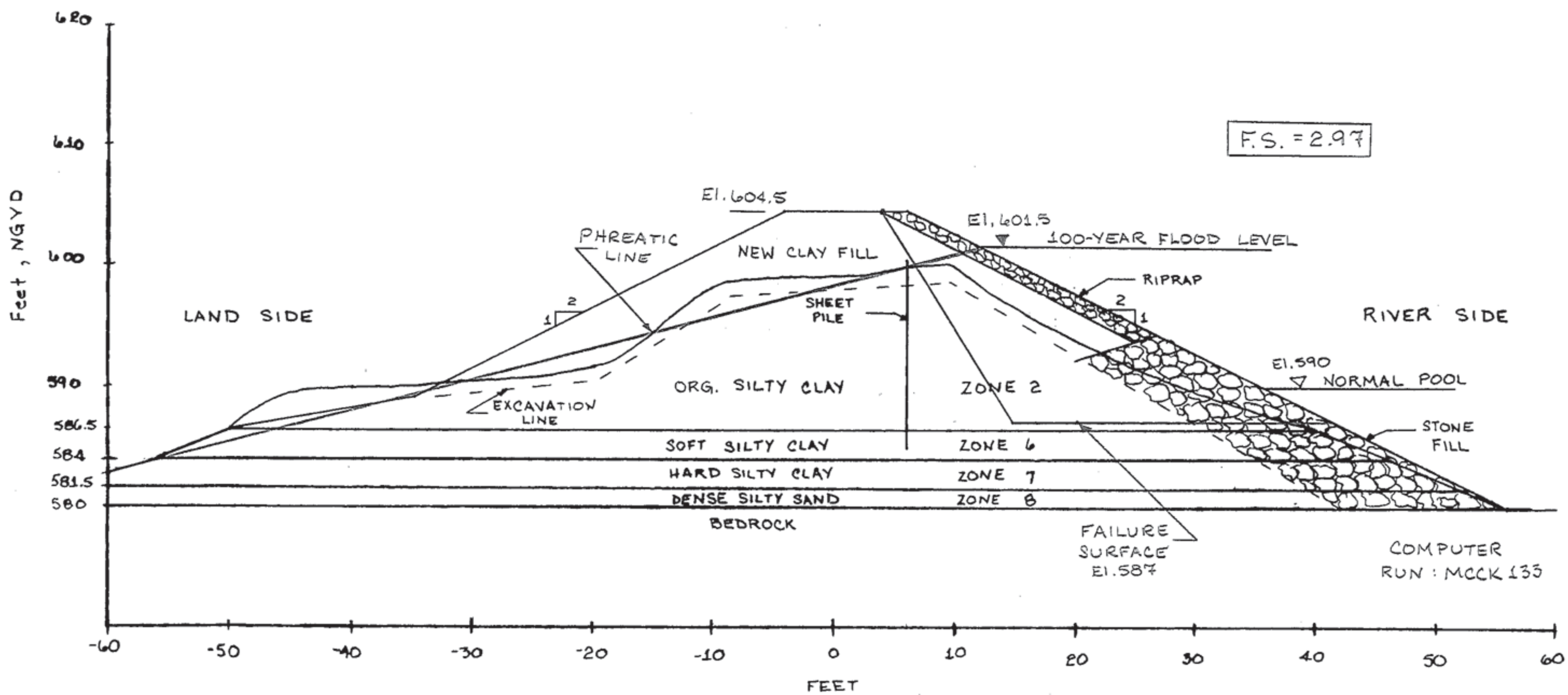


(-24.3, 635)
R = 45'
F.S. = 1.84
El. 604.5
PHREATIC LINE
NEW CLAY FILL
RIPRAP
LAND SIDE
SHEET PILE
RIVER SIDE
El. 593
El. 590
NORMAL POOL
HARD SILTY CLAY
ZONE 2
EXCAVATION LINE
El. 588
SOFT SILTY CLAY
ZONE 6
STONE FILL
HARD SILTY CLAY
ZONE 7
DENSE SILTY SAND
ZONE 8
BEDROCK
COMPUTER RUN : MCCK136
Feet, NGVD
640
630
620
610
600
590
586.5
584
581.5
580
-60
-50
-40
-30
-20
-10
0
10
20
30
40
50
60
FEET



F.S. = 1.23
(-27.1, 626.6)
R= 38.6'
must be >3:1
LAND SIDE
RIVER SIDE
PHREATIC LINE
NEW CLAY FILL
El. 604.5
SHEET PILE
EXCAVATION LINE
El. 588
El. 590
NORMAL POOL
ORGANIC SILTY CLAY
ZONE 4
SOFT SILTY CLAY
ZONE 6
HARD SILTY CLAY
ZONE 7
BEDROCK
COMPUTER RUN : MCCK131
FEET, NGVD
630
620
610
600
590
586
581
580
577
-60
-50
-40
-30
-20
-10
0
10
20
30
40
50
60
FEET


F.S. = 3.70
El. 604.5
NEW CLAY FILL
PHREATIC LINE
RIPRAP
2
1
LAND SIDE
RIVER SIDE
SHEET PILE
El. 590
El. 590
NORMAL POOL
ORG. SILTY CLAY
ZONE 2
EXCAVATION LINE
STONE FILL
SOFT SILTY CLAY
ZONE 6
HARD SILTY CLAY
ZONE 7
DENSE SILTY SAND
ZONE 8
BEDROCK
FAILURE SURFACE El. 587
COMPUTER RUN: MCCK135
Feet, NGVD
620
610
600
590
586.5
584
581.5
580
-60
-50
-40
-30
-20
-10
0
10
20
30
40
50
60
FEET



F.S. = 2.97
El. 604.5
El. 601.5
100-YEAR FLOOD LEVEL
PHREATIC LINE
NEW CLAY FILL
RIPRAP
SHEET PILE
LAND SIDE
RIVER SIDE
El. 590
NORMAL POOL
ORG. SILTY CLAY
ZONE 2
EXCAVATION LINE
STONE FILL
SOFT SILTY CLAY
ZONE 6
HARD SILTY CLAY
ZONE 7
DENSE SILTY SAND
ZONE 8
BEDROCK
FAILURE SURFACE El. 587
COMPUTER RUN : MCCK 133
Feet , NGYD
620
610
600
590
586.5
584
581.5
580
-60
-50
-40
-30
-20
-10
0
10
20
30
40
50
60
FEET

BORING LOGS