S&ME

October 25, 2000

[redacted]
Lockwood Greene Engineers
Post Office Box 491
Spartanburg, SC 29304

Re: Reliability of Existing Levees Against Underseepage Piping
Green Diamond Development
Columbia, SC
S&ME Project No. 1611-00-937

Dear [redacted]

On September 26, 2000 [redacted] of S&ME and [redacted] of Lockwood Greene discussed remarks made by a geotechnical engineer contracted by the Federal Emergency Management Authority relative to the current and proposed levee system. It was concluded that in-place permeability testing and further analysis of the existing levee system with respect to seepage, hydraulic gradient would assist in evaluating how the current levee system would respond to major flood events of the Congaree River. Of particular interest is the potential development of piping erosion below the current levee cross section, which was cited by FEMA as the most likely mode of failure.

Additional field work by S&ME incorporated in-situ hydraulic characteristic testing of the levee soils and natural base soils with hydraulic seepage analysis of critical cross sections identified by FEMA's geotechnical consultant. Accomplishment of the purpose of this investigation was achieved by the following phases of work.

1. A field program that included installing twelve piezometers at selected locations along the existing levee.

2. Field testing of the in-situ hydraulic characteristics of each of the piezometers installed.

3. Analysis of the seepage and hydraulic gradient of flow through the existing levee system based on the field data gathered, and the flood elevations provided by Lockwood Greene.

The investigation performed is intended to provide information for use in evaluating the existing levee system performance during major flood events. Our borings and tests were performed at large spacings. Though an effort was made to install piezometers and perform testing in typical soils of the levee and base material encountered during our

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original work to define as closely as reasonably possible the range of conditions present in the levee, subsurface conditions may vary somewhat between boring locations.

Levee Configuration and Profiles

S&ME was provided cross sections and survey data during our initial work. Surveyed cross sections for Levee Section 1 were performed by Surveying and Mapping, Inc. Cross sections were obtained at approximately 500-foot intervals for the entire length of both the river levee north of I-77 and the intermediate levee south of I-77. A total of 64 cross sections were surveyed. Cross section locations were marked in the field by a small plastic card on a stake with the Cross-Section number (#1A-#1D and #1 through #60) shown.

Installation and Development of Piezometers

Our field work was performed between October 3, 2000 and October 18, 2000. Field work consisted of installing piezometers and field pump tests to estimate the hydraulic conductivity of the soils within a short distance of the piezometer. S&ME performed 12 soil borings at or near locations of six previous borings drilled during our initial work. Piezometers were installed at the locations of these following previous borings, B141 (2 piezometes), B127 (1 piezometer), B126 (2 piezometers), B240 (2 piezometers), B253 (3 piezometers), and B102 (2 piezometers).

The piezometers were installed to depths ranging from approximately 6 feet to 30 feet below existing ground surface (either top of levee or base of levee). The borings were located in the field either by taping distances from existing site features or by identifying previous boring markers in the field. Figure 2 depicts the approximate locations of the borings. Elevations of the borings were estimated from previous surveys and estimates of elevations for the original boring.

Borings were drilled using a 4.25 inch I.D hollow stem auger. At each sample interval in the borings, a penetration test was performed by driving the sampler a set depth using a standard 140 pound hammer falling 30 inches in general accordance with ASTM D1586. The depth to groundwater was noted at the time of drilling. Stabilized groundwater levels in the auger borings were measured prior to performing hydraulic tests in the piezometers. Stabilized groundwater levels shown on the test boring records represent the conditions only at the time of the exploration and may fluctuate with seasonal variations in rainfall. Normally the highest seasonal groundwater levels occur in late winter and early spring and lowest levels in late summer and fall.

Piezometer completion records are presented in the Appendix . The piezometers were installed in soils ranging from natural silty sands, poorly graded sands and clays to levee silty sands and clays. Completion of piezometers consisted of placing 2 inch diameter, 0.01 inch slot, schedule 40 PVC pipe in the lower five foot of each location. Threaded two inch diameter riser pipe was then installed to above the ground surface. The slotted interval was measured and a sand pack was placed to the top of the screen as accurately as could be measured in the field. Two piezometers encountered flowing sand such that the sand pack extended above the screened interval 1 to 2 feet as noted on the piezometer records. A minimum 1-foot bentonite seal was placed above the sand pack and hydrated to form an impermeable zone above the screened interval. The riser pipes were initially extended above ground to as near as practical the elevation of the 100 year flood.

The following table summarizes the piezometers installed, completion depths and soil types within the screened intervals.

Piezometer No.	**Completion Depth Feet Below Top of Collar**	**Soil Type**	**Top of Casing Elev.**
B102A	18.7 to 23.7	Natural Poorly Graded Sand	Top of Levee
B102B	3.8 to 8.8	Levee Silty Sand	Top of Levee
B126A	1.5 to 6.5	Levee Clay	Top of Levee
B126B	0.9 to 5.9	Natural Clay	Base of Levee
B127A	17.1 to 22.1	Natural Silty Sand	Top of Levee
B141A	5.2 to 10.2	Levee Silty Sand	Top of Levee
B141B	17.4 to 22.4	Natural Silty Sand	Top of Levee
B240A	4.5 to 9.5	Levee Clay	Top of Levee
B240B	23.2 to 28.2	Natural Poorly Graded Sand	Top of Levee
B253A	21.6 to 26.6	Natural Poorly Graded Sand with Silt	Top of Levee
B253B	12.9 to 17.9	Natural Silty Sand	Top of Levee
B253C	3.9 to 8.9	Levee Clay	Top of Levee

Following completion of the testing the piezometer will be abandoned by pulling readily removable pipe and filling all voids with bentonite chips.

Field Permeability Testing

Prior to hydraulic characteristic testing in each piezometer the depth of the ground water was measured. Four piezometers encountered groundwater, B102A, B240B, B253A, B253B, and could thus be considered as fully saturated tests. The remaining 8 piezometers were dry and required inundation of the surrounding soils to a saturated or near saturated state prior to performing either falling head or constant head tests. Both falling head and constant head permeability tests were conducted in each piezometer to check the findings of either approach. Typically, five to six tests were performed to establish repeatable values and allow statistical treatment of the results.

Due to the variability of the soils and the current moisture content of the soils the amount of water added to each piezometer to achieve consistent hydraulic readings varied significantly. The following table summarizes each piezometer, the quantity of water added and the type of test performed.

Piezometer No.	Soil Type	Type of Test	Quantity of Water
B126A	Levee Clay	Constant Head, Unsaturated	75 Liters
B240A	Levee Clay	Constant Head, Unsaturated	44 Liters
B253C	Levee Clay	Constant Head Unsaturated	33 Liters
B126B	Natural Clay	Constant Head Unsaturated	118 Liters
B102B	Levee Silty Sand	Constant Head Unsaturated	130 Liters
B141A	Levee Silty Sand	Constant Head Unsaturated	550 Liters
B127A	Natural Silty Sand	Falling Head Unsaturated	945 Liters
B141B	Natural Silty Sand	Falling Head Unsaturated	1340 Liters
B253B	Natural Silty Sand	Falling Head Saturated	1010 Liters
B253A	Natural Poorly Graded Sand with Silt	Falling Head Saturated	1020 Liters
B102A	Natural Poorly Graded Sand	Falling Head Saturated	1260 Liters
B240B	Natural Poorly Graded Sand	Falling Head Saturated	1970 Liters

Constant head tests in unsaturated soils were modeled using the van Genuchten model regression analysis approach described by Daniel B. Stephens, Kevin Lambert, and David Watkins in Water Resources Research Bull. Vol. 23, No. 12, pp. 2207-2214 (Dec. 1987) to estimate the saturated hydraulic conductivity of the unsaturated soils. Falling head tests in unsaturated soils after inundation and in saturated soils were modeled using the procedure described in Figure 13 of volume 7.1 of the US Navy Facilities Engineering Command Design Manual (page 7.1-104) for a piezometer in isotropic soil.

Laboratory Testing

A limited laboratory testing program was performed to supplement earlier tests. These included grain size analysis and Atterberg limits to allow correlation of the piezometer completion depths to samples obtained during the original investigations. Laboratory test results are presented in Appendix D on the laboratory summary table.

Soil Permeability Values

The following table summarizes the in-situ permeability tests.

Piezometer No.	Soil Type	Permeability Range Feet/Seconds	Expected Value Feet/Second	Standard Deviation
B126A	Levee Clay	7.5×10^{-6} - 4.5×10^{-6}	6.0×10^{-6}	1.5×10^{-6}
B240A	Levee Clay	7.0×10^{-8} - 1.1×10^{-8}	9.2×10^{-8}	2.3×10^{-8}
B253C	Levee Clay	4.5×10^{-7} - 2.7×10^{-7}	3.6×10^{-7}	8.8×10^{-8}
B126B	Natural Clay	1.7×10^{-5} - 1.0×10^{-5}	1.4×10^{-5}	3.3×10^{-6}
B102B	Levee Silty Sand	8.7×10^{-6} - 5.3×10^{-6}	7.0×10^{-6}	1.7×10^{-6}
B141A	Levee Silty Sand	4.9×10^{-5} - 2.9×10^{-5}	3.9×10^{-5}	9.6×10^{-5}
B127A	Natural Silty Sand	3.8×10^{-3} - 1.3×10^{-5}	6.9×10^{-3}	1.1×10^{-2}
B141B	Natural Silty Sand	6.5×10^{-5} - 3.1×10^{-5}	4.9×10^{-5}	3.6×10^{-5}
B253B	Natural Silty Sand	2.0×10^{-4} - 1.6×10^{-4}	1.8×10^{-4}	1.3×10^{-4}
B253A	Natural Poorly Graded Sand with Silt	1.1×10^{-4} - 1.3×10^{-4}	1.2×10^{-4}	5.6×10^{-5}
B102A	Natural Poorly Graded Sand	9.9×10^{-5} - 7.4×10^{-5}	8.7×10^{-5}	7.1×10^{-5}
B240B	Natural Poorly Graded Sand	2.7×10^{-4} - 2.0×10^{-4}	2.2×10^{-4}	1.1×10^{-4}

Probability of Piping Occurrence

There is no specific requirement described in 44CFR 65.10 for protection against uplift pressures and piping. Excess seepage pressures and resultant piping failure is described in the literature as having an order of magnitude greater probability of occurrence than slope instability for short term conditions. Formation of seepage boils and subsequent piping of soils from the landside toe of the levee is described as the most likely mode of levee failure in 1976.

If uplift pressures in pervious deposits underlying impervious or semipervious top strata landward of the levee exceed the effective weight of the top stratum, heaving or rupturing of the top strata could result in formation of sand boils or piping below the levee foundation. An estimate of substratum pressures was made at selected cross sections along Levee 1 and the intermediate levee at locations where soil conditions were reasonably well defined by soil test borings conducted during earlier exploration. The equations used were those presented in Appendix B of US Army Corps of Engineers Engineering Manual 1110-2-1913, "Design and Construction of Levees" (April, 2000), which were in turn developed during a study (reported in U.S. Army Engineer Waterways Experiment Station TM 3-424 (Appendix A) of piezometric data and seepage measurements along the Lower Mississippi River and confirmed by model studies.

Under this approach the foundation is generalized into a pervious sand or gravel stratum with a uniform thickness and permeability and a semipervious or impervious top stratum with a uniform thickness and permeability (although the thickness and permeability of the riverside and landside top stratum may be different). This approach involves certain other simplifying assumptions:

- Seepage may enter the pervious substratum at any point in the foreshore (usually at riverside borrowpits) and/or through the riverside top stratum.

- Flow through the top stratum is vertical.
- Flow through the pervious substratum is horizontal.
- The levee (including impervious or thick berms) and the portion of the top stratum beneath it is impervious.
- All seepage is laminar.

The typical levee cross section analyzed is illustrated in Figure 3 in the Appendix. For this analyses, we assumed the top strata both landside and riverside of the levee to be semipervious. We assumed an open seepage entrance on the riverside at a distance from the levee equal to the distance to the river channel. A blocked seepage exit landward of the levee (due to a cutoff of the pervious layer by buried silt or clay deposits or by landward thickening of the top stratum) was assumed at a distance of 20 feet landward of the landside toe. This seepage geometry is given as Case VII in Appendix B of EM 1110-2-1913 referenced above.

The exit gradient landside of the levee, defined as the excess hydrostatic head at the landside toe divided by the thickness of the top stratum, was used to define the performance of the levee against piping occurrence. The value of the critical hydrostatic gradient for piping occurrence was assumed to be 1.0, based on an in-situ top stratum buoyant unit weight approximately equating to the unit weight of water. All levee cross sections were assumed to be dry, that is, no ponded water on the landside toe except in drainage ditches where these lie close to the levee.

For an existing levee subjected to a flood, the probability of piping failure can be expressed as a function of the floodwater elevation and other factors including soil permeability, embankment geometry, foundation stratigraphy, etc. This analysis focused on developing the *conditional* probability of failure function for the floodwater elevation, constructed using engineering estimates of the probability functions or moments of the relevant variables. This approach is described in US Army Corps of Engineers Technical Letter 1110-2-556 (1999).

Five random variables were considered. These included the horizontal permeability of the pervious substratum k_f, the vertical permeability of the semi-pervious top blanket k_r on the riverside, and the vertical permeability k_l on the landside. Permeability values were assumed to have a coefficient of variability of 30 percent. As there is some uncertainty regarding the thicknesses of the soil strata between boring locations, the thickness of the pervious strata *(d)* and the top elevation of the pervious strata landward of the levee were also modeled as random variables. Their deviations are set to match engineering judgment regarding the probable range of actual values. Assigning a standard deviation to the top elevation of the pervious layer of 2.5 ft models a high probability that the actual value lies between two consecutive split spoon samples in the borings. The thickness of the pervious layer was assumed to have a coefficient of variability of 50 percent.

To facilitate computation, a spreadsheet was developed that accomplishes the following for each stage of flooding at each input cross section:

- Solves for the exit gradient using the methods in EM 1110-2-1913 Appendix B.
- Repeats the solution for 11 combinations of the input parameters required in the Taylor's series method.
- Determines the expected value and standard deviation of the exit gradient.
- Calculates the expected value and standard deviation of the natural logarithm of the exit gradient.
- Calculates the probability that the exit gradient is above the critical value of 1.0 required to initiate boiling or piping.

Results from the spreadsheet for Cross Section 16 for a river stage of 140 feet are shown on the following page. The details of the calculations follow.

For the first analysis (Run 1), the random variables are all taken at their expected values. From EM 1110-2-1913, first the effective exit distance x_3 is calculated as:

$$X_3 = 1 / C \times \tanh(CL_3), \text{ where:}$$

$$C = \{k_{bl} / (k_f \times z_{bl} \times d)\}^{0.5}$$

L_3 = distance from the landside levee toe to an effective seepage block in the pervious layer

k_{bl} = permeability of top strata landward of the levee toe

k_f = permeability of pervious stratum

z_{bl} = thickness of top strata landward of the levee toe

d = thickness of pervious stratum

RIVER STAGE 140 FEET CROSS SECTION 16

TAYLORS SERIES EXPANSION FOR LEVEE UNDERSEEPAGE

CASE VII-PARTIALLY PERMEABLE SEEPAGE BERMS ON BOTH RIVER AND LAND SIDES

WITH SEEPAGE BLOCK 20 FEET FROM LANDSIDE TOE

Kbl	Kbr	Kf	d	z	zbr	zbl	cr	cl	X1		X3	Ho	ho/Zbl	var comp	% of var
0.000020	0.000197	0.000591	20	110.00	19.00	15.00	0.0296	0.0105	33		457	12.28	0.82		
0.000014	0.000197	0.000591	20	110.00	19.00	15.00	0.0296	0.0088	33		650	12.98	0.87		
2.559E-05	0.000197	0.000591	20	110.00	19.00	15.00	0.0296	0.0120	33		353	11.65	0.78	0.0019425	8.53
0.000020	0.000138	0.000591	20	110.00	19.00	15.00	0.0248	0.0105	39		457	12.15	0.81		
0.000020	0.0002559	0.000591	20	110.00	19.00	15.00	0.0338	0.0105	29		457	12.36	0.82	4.83E-05	0.21
0.000020	0.000197	0.000413	20	110.00	19.00	15.00	0.0354	0.0126	28		322	11.55	0.77		
0.000020	0.000197	0.0007677	20	110.00	19.00	15.00	0.0260	0.0092	37		592	12.73	0.85	0.0015574	6.84
0.000020	0.000197	0.000591	10	110.00	19.00	15.00	0.0419	0.0149	24		232	10.74	0.72		
0.000020	0.000197	0.000591	30	110.00	19.00	15.00	0.0242	0.0086	40		682	12.95	0.86	0.0054453	23.92
0.000020	0.000197	0.000591	20	107.50	21.50	17.50	0.0278	0.0098	35		532	12.56	0.72		
0.000020	0.000197	0.000591	20	112.50	16.50	12.50	0.0318	0.0115	31		382	11.91	0.95	0.0137666	60.49
													sum	0.0227601	100

crest elev		142 ft
elev of toe (inside)		125 ft
elev of toe (outside)		129 ft
crest width		11 ft
inside slope		1.6 H:V
outside slope		2.3 H:V
dist to river channel	L1	80 ft
dist to seepage block	L3	20 ft
Width of Levee at base	L2	68.1 ft
Tailwater Elevation		0 ft

	average	Coeff var	std dev	avg+std dev	avg - std dev
Substratum Permeability ft/sec	3.E-04	30	9.E-05	4.E-04	2.E-04
Top Blanket Permeability (outside)	1.E-04	30	3.E-05	1.E-04	7.E-05
Top Blanket Permeability (inside)	1.E-05	30	3.E-06	1.E-05	7.E-06
Thickness of Pervious Layer	20	50	10	30.0	10.0
Top elev of pervious	110	2.27	2.5	112.5	107.5

E(I)	0.82	E(lnI)	-0.21702
Var(I)	0.0228		
sig(I)	0.1509	sig(lnI)	0.1827869
Vx	0.1843		
z	0.75		
normsdist	0.7732		
prob > crit	0.2268		

The distance from the landside toe to the effective source of seepage entrance X_1 is:

$X_1 = \tanh CL_1 / C$, where:

L_I = distance from the landside levee toe to the river from topo map.

The net residual head under the top stratum at the levee toe h_0 is:

$h_0 = H \{ X_3 / (X_1 + L_2 + X_3)\}$, where:

H = total head above the landside levee toe to the flood elevation

L_2 = base width of the levee, determined by the levee crest elevation, width, and side slopes obtained from surveyed cross sections.

And the landside toe exit gradient i is:

$i = h_0 / z_{bl}$

For subsequent runs, the variables are in turn adjusted to their expected values plus and minus one standard deviation, holding the other variables at their expected values. These are used to determine the component of the total variance related to the variation of each variable. When the variance components are summed in the spreadsheet above, the total variance of the exit gradient is 0.0227. Taking the square root of the variance gives the standard deviation of 0.151.

The exit gradient was assumed to be a lognormally distributed random variable with probabilistic moments $E[i] = 0.82$ and $\sigma_i = 0.151$. Using the properties of the lognormal distribution, the equivalent normally distributed random variable has moments $E[\ln i] = -0.217$ and $\sigma_{\ln i} = .183$.

The critical exit gradient is assumed to be 1.0. The probability of failure is then:

$P_{rf} = P_r (\ln i > \ln 1.0)$

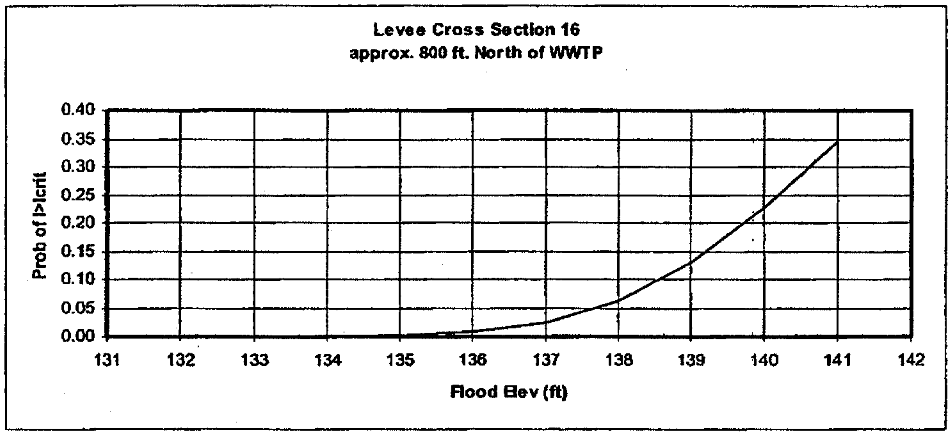
This probability was evaluated using a normal distribution function built into the Microsoft Excel spreadsheet. It can be solved using standard tables by first calculating the standard normalized variate z:

$z = (\ln i_{crit} - E(\ln i)) / \sigma_{\ln i}$

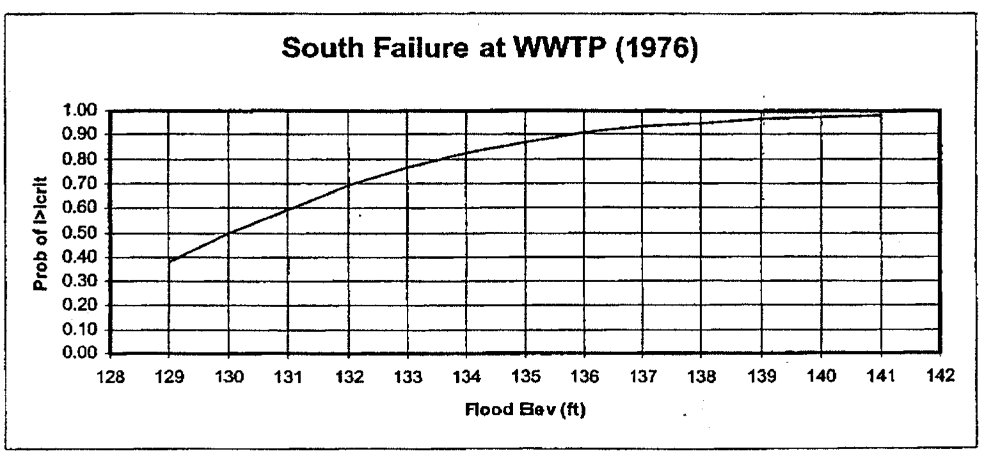
For the moment values given above, the standard normalized variate is 0.75. For this value, the cumulative distribution function (NORMSDIST) is 0.773, and represents the probability that the exit gradient is below the critical gradient of 1.0. The probability that the exit gradient exceeds the critical gradient is

$P_{rf} = 1 - NORMSDIST(z) = 1 - 0.773 = 0.227$, or 22.7 percent

The analysis was repeated for a range of flood stages up to the crest elevation. The resulting conditional probability of failure function was plotted as shown below for Levee Cross Section 16. The shape of the probability of failure function is similar to that suggested in ETL 1110-2-556 for "good" levees in that the probability of failure is very low until the flood stage on the levee exceeds about 139 ft, after which it curves up sharply. When the floodwater elevation is near the top of the levee, the conditional probability of failure approaches 35 percent.



To compare the remaining sections of the Manning Levee to locations where failure of the levee is known to have previously occurred, the analysis described above was performed for the geometry of the 1976 South Failure at the City of Columbia Wastewater Treatment Plant. Expected values and variabilities for permeability of the top strata and pervious strata were assigned to soil types described in the 1976 borings using the pump test data described in the preceding chapters. The conditional probability of failure function for this cross section is given below:



The shape of the probability of failure function here is similar to that described in ETL 1110-2-556 for a "poor" levee. The probability of failure function is concave upwards for flood elevations considerably below the crest elevation at that location (140 feet). Failure of the levee was noted to have occurred at flood elevation 134.9 feet. At that flood stage the probability of the exit gradient at the toe exceeding the critical value was approximately 85 percent.

Levee probability of failure by cross section was determined for three different flood stages and are tabulated below. The relative flood elevation at each cross section during each event was estimated from the profile of flood elevations shown on a drawing titled "Levee Profile, Levee Sections One and Two" dated August 19, 1999 by Lockwood Greene Engineers.

- The first stage considered was the 1976 event which resulted in failure at the City of Columbia Wastewater Treatment Plant at an flood elevation of 134.9 feet at the plant. Backwater in Gills Creek was estimated to be at approximately 130 feet and there was no water in Levee Section 2, so that the intermediate levee between Sections 1 and 2 was not subject to any head.

- Next we considered a flood approximating the 100 year BFE computed by Lockwood Greene for a 252,000 cfs flow. In this case Levee Section 2 was assumed flooded to elevation 134.4 feet along with Gills Creek.

- The final run considered a flood approximately 2 feet higher than the 100 year BFE profile shown on the Lockwood Greene plan of 1999, approximating the 2000 FEMA 100-year BFE for 292,000 cfs flow. At this point the flood elevation approaches the levee crest just north of the wastewater plant. Thus this would be the maximum elevation where piping would be the most likely mode of failure. The pool downstream of the intermediate levee south of I-77 is approximately 138 feet and the intermediate levee between Levee Section 1 and Section 2 has been overtopped.

Location	Cross Section	1976 Flood		100 Year BFE by Lockwood Greene 252,000 cfs		100 Year BFE by FEMA 292,000 cfs	
		Flood Elev.	P_{rf} %	Flood Elev.	P_{rf} %	Flood Elev.	P_{rf} %
North End of Main Levee	1A	138.0	0.0	141.1	2.0	142.0	4.0
Gregg Property	3	137.5	<1.0	141.5	5.0	142.0	10.0
Gregg Property	6	137.2	0.0	140.0	0.0	142.0	0.0
Columbia Venture	11	136.3	4.0	140.1	16.0	142.0	**32.0**
Heathwood Hall	13	136.0	1.0	139.2	12.0	141.2	28.0
Heathwood Hall	15	135.7	3.0	138.9	10.0	140.9	18.0
Heathwood Hall	16	135.5	1.0	138.5	10.0	140.5	28.0
City WWTP	WWTP	134.9	**85.0**	-	-	-	-
South Trib. Levee	30	No water	-	134.4	0.0	138	0.0
South Trib. Levee	36	No water	-	134.4	21.0	138	Overtop
South Trib. Levee	38	No water	-	134.4	18.0	138	Overtop
South Trib. Levee	41	No water	-	134.4	14.0	138	Overtop
Gills Creek	48	130.0	<1.0	134.4	16.0	138	Overtop

Conclusions

The above data suggest that the levee north of the wastewater treatment plant provides generally good reliability against development of excessive seepage exit gradients on the landside toe up to the 292,000 cfs 100-year base flood. At no cross section analyzed does the probability of failure approach the 85 percent value determined for the 1976 South Failure.

There is about a 1/3 probability of piping development at the worst location evaluated between cross sections 11 and the boundary of the wastewater treatment plant at flood levels slightly below the minimum crest elevation in that section. These estimates are likely conservative because we considered the protected area behind the levee to be dry up to the point of failure – that is, there is no ponded water landward of the levee due to internal drainage. A breach at any one location – either due to piping or overtopping - would result in ponding landward of the remaining levee, decreasing the net head across each section and decreasing the exit gradient, thus stabilizing the remaining sections.

One of the conveyance formulations presented in the FEMA presentation was based on formation of two breaches in the portion of Levee Section 1 north of the wastewater treatment plant. These breaches were simulated in the FEMA finite element analyses by removing a portion of the levee at locations corresponding roughly to cross sections 1A and 15. To evaluate the probability of breaches occurring in both locations during a single flood, we first defined the reliability of the levee reach defined by cross sections 1A through 6 and the reach defined by cross sections 11 through the WWTP to be the products of the individual reliabilities of the 3 to 4 cross sections analyzed for seepage gradient in each reach. Reliability is defined as unity minus the probability of failure. The joint probability of at least one breach occurring at any location, in both reaches at the same flood stage, is thus $\{1-(1-P_{rf1A})(1-P_{rf3})(1-P_{rf6})\}$ times $\{1-(1-P_{rf11})(1-P_{rf13})(1-P_{rf15})(1-P_{rf16})\}$, or approximately 9 percent for the FEMA 100-year BFE.

The 9 percent probability of two breaches occurring does not take into account the reduced potential for piping occurrence in the event that a tailwater condition exists landward of the levee once a breach has occurred. Reduction in net head across the levee due to ponding on the landside would greatly reduce the exit gradient and potential for formation of seepage erosion channels. If a breach formed on the levee near the wastewater plant and water were to pond behind levee cross sections 1A through 6 to a depth of 3 to 5 feet (to elevation 130), for example, the probability of a second breach forming at the north end of the protected area would be reduced to about 2 percent.

The reliability analysis above suggests a low probability of two widely separated breaches forming in the Manning Levee where it fronts the Congaree River during the 100 year BFE. While two breaches were observed in the City of Columbia levee in 1976, we feel that the second breach, which formed at a penetration, was due to poor backfilling around the conduit when the treatment plant was built about 1970. The reliability of the existing levee will be further improved by raising the crest and reshaping the side slopes, which will increase the length of the seepage path and further reduce the exit gradients.

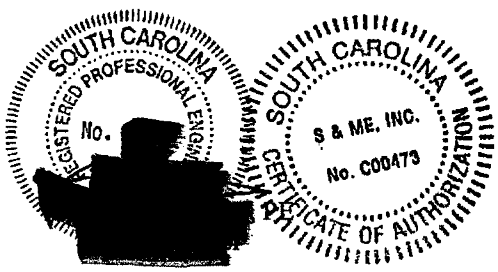
Please call if you have any questions.

Very Truly Yours,

S&ME, Inc.

[redacted]

Geotechnical Engineer



Attachments:

Appendix A – Figures
Appendix B – Boring Logs
Appendix C – Piezometer Logs
Appendix D – Laboratory Exhibit Sheets
Appendix E – Seepage Computation Spreadsheet Input Pages