Appendix 4B-13: Conceptual Design of a PSTA Demonstration Project in STA-3/4

Burns & McDonnell

Everglades Protection Area Tributary Basins

Conceptual Plan for Achieving Long-Term Water Quality Goals

Process Development and Engineering

Conceptual Design of a PSTA Demonstration Project in STA-3/4 (Final Draft)

Submitted to

South Florida Water Management District

August 18, 2003

Project No. 34273

Burns & McDonnell
SINCE 1898

August 18, 2003

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South Florida Water Management District
Conceptual Plan for Achieving Long-Term Quality Goals
Process Development and Engineering

Conceptual Design of a PSTA Demonstration Project in STA-3/4
B&McD Project No. 34273

Dear Dr. Chimney:

Burns & McDonnell is pleased to submit this Final Draft of a Conceptual Design for the addition of a Periphyton-Based Stormwater Treatment Area (PSTA) Demonstration Project in Stormwater Treatment Area 3&4 (STA-3/4). This document has been updated from the initial August 1, 2003 working draft to reflect the additional guidance and input received from the working group review meeting held in the District's offices on August 13, 2003. We understand that it is the District's intent to subject this Conceptual Design to a peer review. We look forward to the results of that review.

We gratefully acknowledge the contributions and guidance received from the District's staff and others in the development of this Conceptual Design. As always, it has been a distinct pleasure to be of service to the District in its continuing efforts to protect and restore the River of Grass, America's Everglades. Please feel free to contact me at (816)822-3099, or electronically at gmiller@burnsmcd.com, should you have any questions or desire further assistance.

Sincerely,

Galen E. Miller, P.E.
Associate Vice President

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1. SUMMARY

This document presents the conceptual design of a Periphyton-Based Stormwater Treatment Area (PSTA) demonstration project in Stormwater Treatment Area 3 & 4 (STA-3/4). The basic scope of the project is to develop an approximate 100-acre PSTA project within the footprint of STA-3/4 which would serve as the final "polishing" cell in the overall treatment process. This PSTA Demonstration Project is intended to augment and form an additional element of the Process Development and Engineering component of the *Conceptual Plan for Achieving Long-Term Water Quality Goals* (Burns & McDonnell, 2003) in discharges from basins tributary to the Everglades Protection Area (EPA). It is intended that the substrate for the PSTA cell consist of the natural limestone underlying the project area; the existing peat (muck) surficial soils would be removed from the PSTA cell interior to expose the limestone surface (caprock). It is anticipated that a parallel PSTA demonstration project will be constructed by the Jacksonville District, U.S. Army Corps of Engineers in Stormwater Treatment Area 1 East (STA-1E), in which the substrate would be formed by placement of an overlying layer of various materials.

The conceptual design has been structured to also permit continued detailed analysis of the performance of Submerged Aquatic Vegetation (SAV) in cells immediately parallel to the PSTA demonstration cell. The resultant design permits a direct side-by-side comparison of the performance of these two communities.

The preparation of this Conceptual Design was authorized by the South Florida Water Management District (SFWMD) through its July 21. 2003 issuance of Purchase Order No. PC P302468 to Burns & McDonnell Engineering Company, Inc.

2. STA-3/4 BACKGROUND INFORMATION

The South Florida Water Management District is presently constructing STA-3/4; construction completion and initial startup is presently scheduled for mid 2004. STA-3/4 will provide a total effective treatment area of 16,543 acres, situated generally between U.S. Highway 27 (on the east) and the Holey Land Wildlife Management Area (on the west), lying immediately north of the L-5 Borrow Canal. This stormwater treatment area is intended to treat inflows from the Miami Canal (via Pumping Station G-372) and the North New River Canal (via Pumping Station G-370).

The general boundaries of STA-3/4's primary tributary basins are shown in Figure 1.

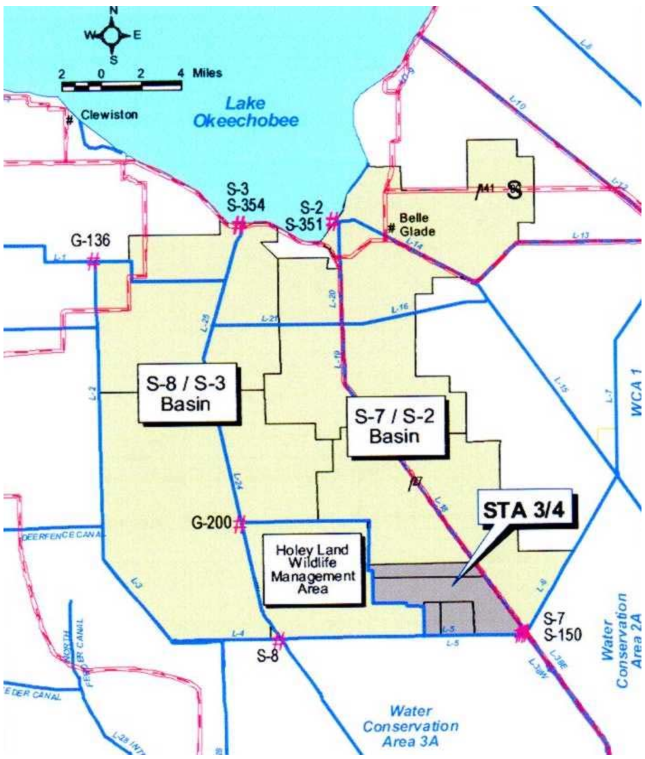
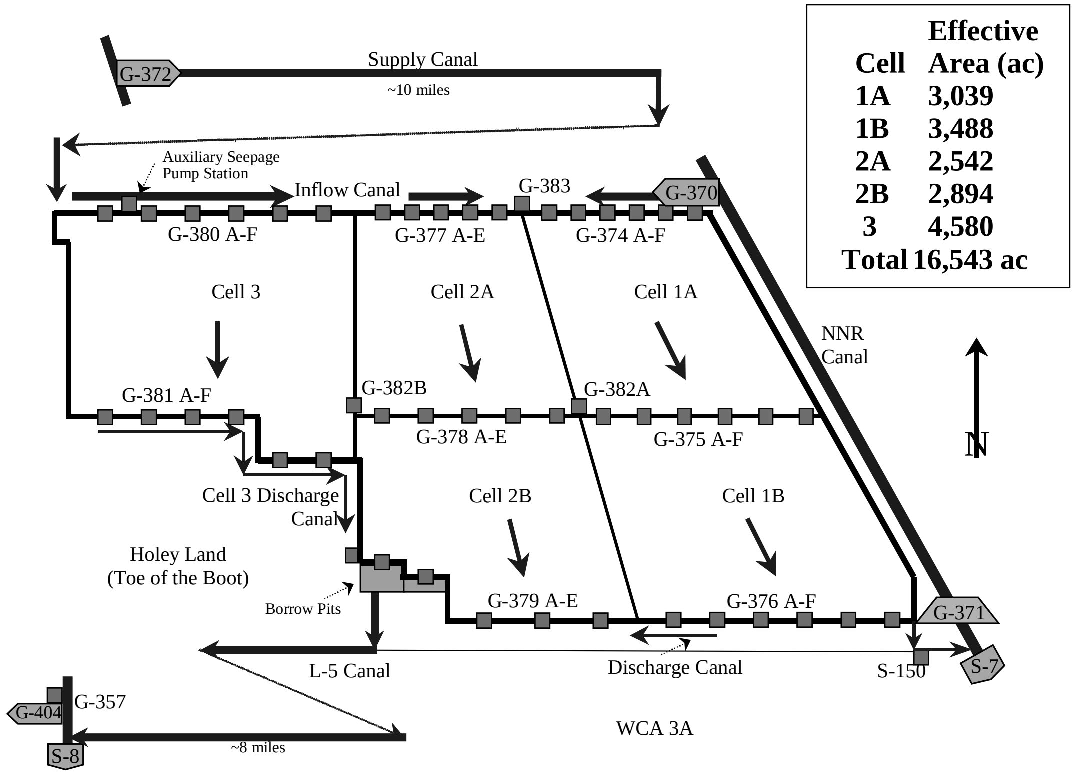


Figure 1: Areas Tributary to STA-3/4

STA-3/4 is being developed as three parallel flow paths. The most easterly flow path (Cells 1A and 1B in series) is intended to treat inflows from the North New River Canal. The two westerly flow paths (Cells 2A and 2B in series, Cell 3 in parallel) are intended to treat inflows from the Miami Canal. A schematic of the present design of STA-3/4 is shown in Figure 2.



Cell	Effective Area (ac)
1A	3,039
1B	3,488
2A	2,542
2B	2,894
3	4,580
Total	16,543 ac

Figure 2: Current Design Schematic, STA-3/4

Structure G-383, situated in the Inflow Canal at the northwesterly corner of Cell 1A, is intended to be normally closed (Burns & McDonnell, 2001), thus effectively separating North New River and Miami Canal inflows. In essence, STA-3/4 is intended to normally function as two separate but immediately adjacent stormwater treatment areas. The presence of Structure G-383 is included in the design to afford flexibility in the future operation. STA-3/4 is presently being developed in emergent macrophyte vegetation throughout its effective treatment area.

2.1. Previously Recommended Improvements and Enhancements

A number of improvements and enhancements to STA-3/4 have been recently recommended in connection with the development of a conceptual plan to achieve long-term water quality goals in hydrologic basins tributary to the Everglades Protection Area (Burns & McDonnell, 2003). A schematic diagram of the enhanced STA is presented in Figure 3:

Cell	Effective Area (ac)
1A	3,039
1B	3,488
2A	2,542
2B	2,894
3A	2,153
3B	2,427
Total	16,543 ac

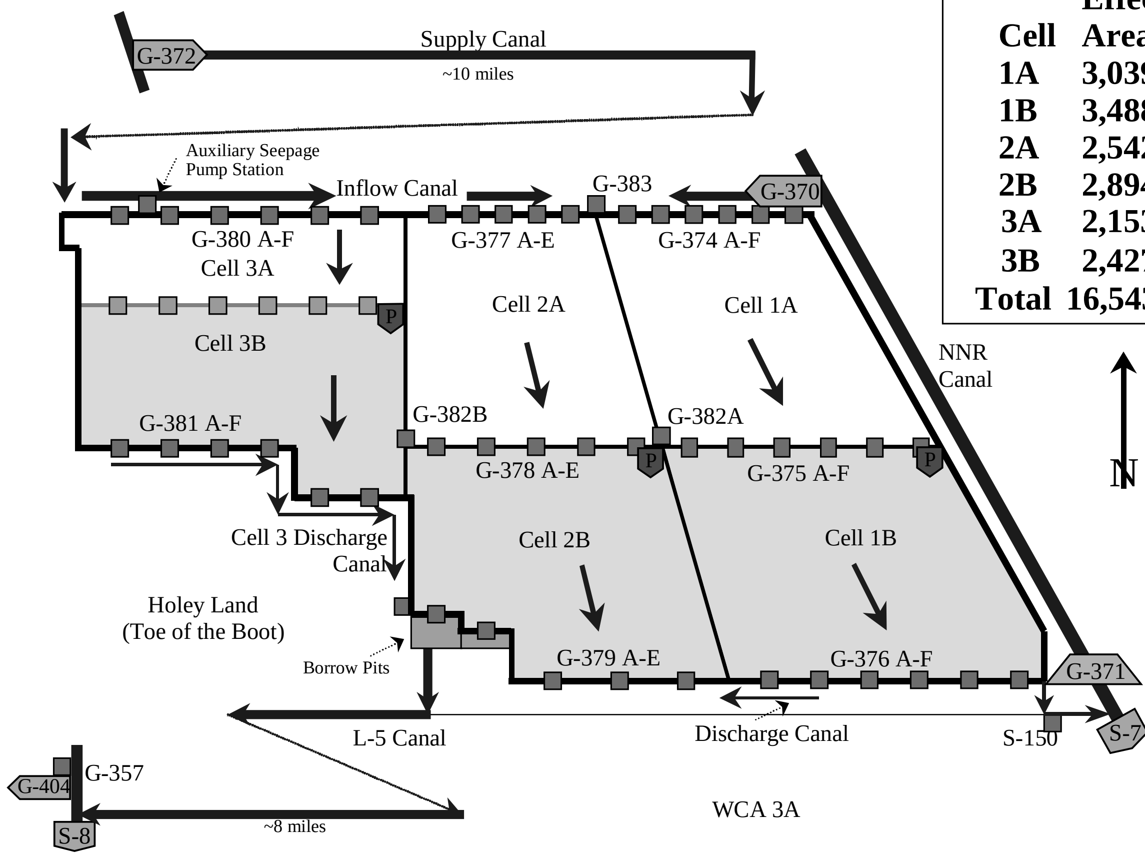


Figure 3: Schematic Diagram of Enhanced STA-3/4

The following improvements and enhancements, all to be completed prior to December 31, 2006, were recommended:

- Construction of approximately 3.3 miles of interior levee, subdividing Cell 3 into Cells 3A and 3B.
- Construction of additional water control structures through the new levee subdividing Cell 3 into Cells 3A and 3B.
- Extension of an overhead power distribution line from the intersection of Interior Levee 3 and Interior Levee 4, extending north along Interior Levee 4 to the new levee across Cell 3, and then west along the new levee across Cell 3.

- Addition of three small forward-pumping stations along the interior levees between cells in series to permit withdrawal from upstream emergent marsh cells to maintain stages in the downstream SAV cells.
- Conversion of Cells 1B, 2B and 3B from emergent macrophyte vegetation to Submerged Aquatic Vegetation (SAV).

Simulations of the performance of the enhanced STA-3/4 in reducing total phosphorus concentrations and loads conducted in connection with the SFWMD's Basin-Specific Feasibility Studies (Burns & McDonnell, 2002) suggested that the enhanced STA would afford at least the possibility of meeting water quality goals for total phosphorus concentrations (defined as a long-term geometric mean concentration of 10 ppb). However, substantial analytical uncertainties surrounded those projections. It was concluded that the phosphorus reduction performance of the STA is sensitive to the performance of the SAV community (range in long-term flow-weighted mean concentrations of 14 to 21 ppb, and in long-term geometric mean concentrations of 10 to 14 ppb).

2.2. Treatment Area Inflows from G-372

Inflow volumes and loads are taken from an Excel file ("sta34 inflow tp.xls" dated May 29, 2001) developed in connection with SFWMD's publication of Baseline Data (Goforth and Piccone, 2001). That data was for a 31-year simulation extending from January 1, 1965 through December 31, 1995. As summarized in previous documents (Goforth and Piccone, 2001; Burns & McDonnell, 2002), inflow volumes and TP loads to STA-3/4 included discharges from both Pumping Station G-370 and Pumping Station G-372. However, as noted above, the detailed design of STA-3/4 is developed to slave Cells 2A, 2B and 3 to discharges from G-372.

The PSTA Demonstration Project is intended to be located in Cell 2B of STA-3/4 (see Section 3 of this document). It was therefore necessary to develop an inflow time series to STA-3/4 that considers only simulated discharges from Pumping Station G-372. The SFWMD's Baseline Data file for STA-3/4 was entered and modified to include, for this Concept Design, only those discharges from G-372. Discharges from G-370 are assigned to Cells 1A and 2A, and are of no interest to this analysis.

A summary of annual discharges from G-372 for Water Years 1965-1994 (e.g., from May 1, 1965 through April 30, 1995) is presented in Table 1. That Water Year definition was selected for consistency with the reporting periods reflected in the various permits issued for the Everglades Construction Project.

Table 1: Summary of Annual Discharges from Pumping Station G-372

Water Year	G-372 Discharge (ac-ft)	TP Load (kg)	Mean TP Conc. (ppb)
1965	408,752	48,199	95.6
1966	481,681	52,858	89.0
1967	299,496	36,473	98.7
1968	696,034	75,032	87.4
1969	714,768	72,740	82.5
1970	325,351	34,085	84.9
1971	327,109	40,407	100.1
1972	186,576	23,700	103.0
1973	193,282	22,900	96.0
1974	288,892	35,754	100.3
1975	347,379	43,622	101.8
1976	198,397	22,708	92.8
1977	270,653	30,999	92.8
1978	445,537	48,230	87.8
1979	616,509	59,063	77.7
1980	149,251	15,217	82.6
1981	91,802	10,707	94.5
1982	581,907	67,209	93.6
1983	415,597	47,375	92.4
1984	280,565	29,942	86.5
1985	253,007	28,879	92.5
1986	386,408	47,516	99.7
1987	212,233	24,679	94.3
1988	234,120	26,903	93.2
1989	131,685	15,425	95.0
1990	159,842	17,065	86.5
1991	349,733	37,827	87.7
1992	486,964	50,811	84.6
1993	258,729	28,241	88.5
1994	454,218	51,874	92.6
Average	**341,549**	**38,215**	**90.7**
Minimum	**91,802**	**10,707**	**77.7**
Maximum	**714,768**	**75,032**	**103.0**

2.3. Cell 2B Inflows from Cell 2A

As all water quality performance simulations for this Conceptual Design are focused on Cell 2B, it was considered desirable to structure an inflow time series specific to Cell 2B to facilitate the analysis. The April 12, 2002 version of the Dynamic Model for Stormwater Treatment Areas, or DMSTA (Walker and Kadlec, 2003) was employed for that purpose.

Discharges from Pumping Station G-372 were assumed distributed to Cells 2A/2B and 3 on the basis of area (e.g., uniform hydraulic loading rate), with the result that 54.3% of the G-372 discharges were assigned to Cell 2A. A DMSTA simulation of Cell 2A was then conducted using that inflow time series. All parameters for Cell 2A (cell area, cell width, vegetative community as Emergent, daily rainfall, daily evapotranspiration, seepage gains and losses, and hydraulic parameters) were established identical to those employed in the Basin Specific Feasibility Studies (Burns & McDonnell, 2002). A summary of the resultant simulated daily discharges from Cell 2A (e.g., inflows to Cell 2B) is presented in Table 2.

Table 2: Summary of Cell 2A Discharges

Water Year	Cell 2A Outflow (ac-ft)	TP Load (kg)	Mean TP Conc. (ppb)
1965	217,293	15,115	56.4
1966	259,955	18,618	58.1
1967	158,358	11,210	57.4
1968	373,108	27,698	60.2
1969	384,971	28,810	60.7
1970	174,535	12,793	59.4
1971	171,733	12,188	57.5
1972	97,563	6,630	55.1
1973	100,044	5,785	46.9
1974	150,066	9,650	52.1
1975	182,293	12,019	53.4
1976	103,665	6,062	47.4
1977	140,403	8,634	49.9
1978	235,927	14,637	50.3
1979	329,263	21,514	53.0
1980	77,244	4,643	48.7
1981	43,123	2,393	45.0
1982	310,788	19,606	51.1
1983	219,839	14,764	54.4
1984	143,191	9,705	54.9
1985	136,800	8,319	49.3
1986	201,601	13,486	54.2
1987	112,740	7,009	50.4
1988	118,377	7,180	49.2
1989	68,240	3,643	43.3
1990	82,368	4,054	39.9
1991	187,422	10,031	43.4
1992	262,588	15,975	49.3
1993	139,350	8,217	47.8
1994	245,486	15,851	52.3
Average	**180,944**	**11,875**	**53.2**
Minimum	**43,123**	**2,393**	**39.9**
Maximum	**384,971**	**28,810**	**60.7**

The maximum daily rate of discharge from Cell 2A to Cell 2B resulting from the simulation was 2,100 cfs. The mean daily rate of discharge over the 30-year simulation was 250 cfs.

2.4. Simulated Performance of Cell 2B as Presently Designed

A series of DMSTA simulations were conducted to assess the projected performance of Cell 2B as it is presently designed (e.g., with no PSTA Demonstration Project) to serve as a baseline for assessment of the possible impact of the Demonstration Project on the overall performance of Cell 2B (which is an element of what will, throughout the course of the Demonstration Project, be an operating STA subject to permit compliance).

Those simulations were conducted with all Cell 2A parameters identical to those employed in the Basin Specific Feasibility Studies, but with the vegetative community varied to include all Emergent and Submerged Aquatic Vegetation calibration data sets available in the DMSTA model. The Submerged Aquatic Vegetation calibration data sets included those considered in the Basin Specific Feasibility Studies (e.g., SAV, NEWS, and SAV_C4), as well as additional calibration data sets developed subsequent to completion of those studies (Walker, personal communication dated July 16, 2003). Those additional calibration data sets include:

- SAV_2 (modified from the original SAV calibrations to assign C*=8 ppb).
- NEWS_2 (modified from the original NEWS calibrations to consider new data made available from all research platforms).
- SAV_C4_2 (modified from original SAV_C4 calibration, which considered only the best two years of ENRP Cell 4 performance, to reflect all Cell 4 data through March 2002).

The results of those analyses, all for the 30-year simulations, are presented in Table 3.

Table 3: Simulated Performance of Cell 2B as Presently Designed

Cell 2B "Community" (Calibration Set)	Simulated Cell 2B Outflow TP Concentration (ppb)	
	Flow-Weighted Mean (FWM)	Geometric Mean (GEO)
Emergent	33.7	32.4
SAV	23.3	14.0
NEWS	21.0	12.9
SAV_C4	13.6*	8.6**
SAV_2	19.3	11.6
NEWS_2	18.5	12.0
SAV_C4_2	14.4	9.3**
Ave. Performance of all SAV Calibration Sets	18.4	11.5
* Outside Calibration Range, Consider as 14.0 ppb		
** Outside Calibration Range, Consider as 10.0 ppb		

In Table 3, the estimated performance of Cell 2B using the "SAV_C4" calibration data set is that most consistent with simulations performed in connection with the Basin-Specific Feasibility Studies for an enhanced STA-3/4 (Burns & McDonnell, 2002).

3. LOCATION AND GENERAL CONFIGURATION OF PSTA CELL

The recommended location of the PSTA demonstration cell is in the southwest corner of Cell 2B, north and west of Structure G-379E. This location is recommended after consideration of the following:

- The demonstration cell would be placed at the downstream end of the flow path, where the lowest phosphorus concentrations can be anticipated, consistent with the intended function of the PSTA cell as a final "polishing" cell in an overall STA.
- In this area, the combination of relatively high ground surface elevations and thin layers of surficial peat (or muck) will minimize (but will not eliminate) the influence of upwelling seepage in the PSTA demonstration cell.
- It is anticipated that certain construction economies will result from:
  - The use of the existing Cell 2B exterior levee to form the perimeter along the west and south sides of the PSTA demonstration cell.

– The relatively thin layer of surficial muck will minimize the required volume of mass earthwork.

- This location is readily accessible from U.S. Highway 27 along the L-5 Access Road.

As indicated in Table 2, the estimated long-term flow-weighted mean TP concentration in discharges from Cell 2A (inflows to Cell 2B) is 53 ppb. That concentration is considered to be too high to permit the development and maintenance of a PSTA community, based on the results of the District's previous research (Kadlec and Walker, 2003). As a result, it is anticipated that it will be necessary to precede the PSTA demonstration cell with an SAV cell, placed between Cell 2A and the demonstration cell.

A series of preliminary DMSTA simulations were conducted to assess the probable total phosphorus gradient downstream of Cell 2A, with the intent to locate the upstream end of the PSTA demonstration at a point where a long-term flow-weighted mean TP concentration of roughly 25-30 ppb can be anticipated. Given the present uncertainty in the SAV performance that can eventually be realized, those simulations were conducted for all available SAV calibration data sets, and are summarized in Table 4.

Table 4: Estimated TP Concentration Gradient Downstream of Cell 2A

Distance from Inlet (% length)	Simulated FWM TP Concentration for Calibration Set Indicated					
	SAV (ppb)	NEWS (ppb)	SAV_C4 (ppb)	SAV_2 (ppb)	NEWS_2 (ppb)	SAV_C4_2 (ppb)
0	53.2	53.2	53.2	53.2	53.2	53.2
30	33.6	33.4	30.2	32.9	32.8	31.5
38	31.6	31.3	27.1	30.3	30.3	28.4
46	30.1	29.6	24.5	28.2	28.2	25.9
54	28.9	28.2	22.5	26.6	26.5	23.7
62	27.8	27.0	20.8	25.2	25.1	22.0
70	27.0	25.9	19.3	24.1	23.9	20.5
100	24.7	22.9	15.6	21.0	20.5	16.6

For those simulations, it was assumed that 20% of the daily discharges from Cell 2A (e.g., the discharge from one of the five Cell 2A outflow control structures) would be carried through a rectangular "cell" (designated as Cell 2B_1 for the analysis) approximately 2,460 feet in width and 7,800 feet in length. That cell would be formed through construction of a north-south levee along the general alignment of the existing agricultural canal bisecting what was once the Griffin property, extending along the entire flow length of Cell 2B at that location. The total area of Cell 2B_1 would then be approximately 458 acres, or 18% of the total area of Cell 2B. As a result, the above simulations were conducted for hydraulic loads roughly 10% above those that would result from a uniform hydraulic loading rate on the entire Cell 2B.

On the basis of those preliminary simulations, it is anticipated that roughly 50% of the flow path downstream of Cell 2A should be considered for conversion to the PSTA demonstration cell, with the remaining (upstream) 50% continued in Submerged Aquatic Vegetation as presently planned (Burns & McDonnell, 2003). Given that the target size of the PSTA demonstration cell is approximately 100 acres, it was assumed that Cell 2B_1 would be further subdivided into two parallel flow paths. The westerly flow path (229 acres) would consist of an SAV cell (designated 2B_1_1) followed in series by the PSTA demonstration cell (designated 2B_1_2). The easterly flow path would consist of a single 229-acre cell (designated 2B_1_3) developed entirely in Submerged Aquatic Vegetation. An additional benefit of that general arrangement is that it permits the concurrent and closely paralleled evaluation of both PSTA and SAV communities near the outlet of STA-3/4.

A second series of preliminary simulations was then prepared, in which 10% of the overall discharge from Cell 2A was assigned to the westerly flow path (Cells 2B_1_1 and 2B_1_2), and 10% assigned to the easterly flow path (Cell 2B_1_3). For those simulations, the width of the PSTA cell (and, as a result, its area) was reduced by 5% to reflect the loss of treatment area associated with construction of a new levee between Cell 2B_1_2 and 2B_1_3. Those simulations were conducted for all possible combinations of the six SAV calibration data sets and two PSTA calibration data sets available, and considered 40%, 50% and 60% of the flow path length dedicated to PSTA (e.g., demonstration cell areas of 87, 108, and 130 acres). The results of those simulations are summarized in Table 6.

Inspection of Table 6 suggests that, given the available calibration data sets, it is not presently projected that a long-term geometric mean concentration of 10 ppb would be reached at the downstream end of a PSTA cell for the hydraulic and total phosphorus loading rates considered to this point. It is therefore necessary to anticipate that, during operation of the PSTA demonstration cell, it may well become necessary to reduce the loads applied to these cells in order to demonstrate the capacity of this treatment technology to achieve that objective.

Table 6: Preliminary Simulations of PSTA Demonstration Cell Performance

Cell Designation	Vegetative "Community" (Calibration Data Set)	Simulated TP Conc. in Cell Outflows for Cell 2B_1_1 Length					
		40% of Total		50% of Total		60% of Total	
		FWM (ppb)	GEO (ppb)	FWM (ppb)	GEO (ppb)	FWM (ppb)	GEO (ppb)
Cell 2B_1_1	NEWS	30.9	19.7	28.9	18.1	27.3	16.9
Cell 2B_1_2	PSTA	23.6	17.5	23.3	17.0	23.1	16.5
Cell 2B_1_2	PSTA_2	22.0	15.9	21.9	15.6	22.0	15.4
Cell 2B_1_2	NEWS	22.4	14.0	22.3	13.9	22.3	13.8
Cell 2B_1_1	SAV	31.2	19.8	29.4	18.2	28.1	17.1
Cell 2B_1_2	PSTA	23.9	17.8	23.7	17.4	23.7	17.1
Cell 2B_1_2	PSTA_2	22.2	16.1	22.3	15.9	22.5	15.9
Cell 2B_1_2	SAV	25.7	16.8	25.6	16.6	25.5	16.5
Cell 2B_1_1	SAV_C4	26.4	19.6	23.4	16.9	21.2	14.9
Cell 2B_1_2	PSTA	20.4	15.1	19.2	14.0	18.2	13.1
Cell 2B_1_2	PSTA_2	19.1	13.7	18.2	12.8	17.4	12.2
Cell 2B_1_2	SAV_C4	15.0	11.1	14.9	10.9	14.9	10.8
Cell 2B_1_1	NEWS_2	29.7	20.4	27.3	18.4	25.4	16.8
Cell 2B_1_2	PSTA	22.7	16.9	22.1	16.1	21.6	13.1
Cell 2B_1_2	PSTA_2	21.2	15.3	20.8	14.8	20.6	14.4
Cell 2B_1_2	NEWS_2	19.9	13.3	19.8	13.1	19.8	13.0
Cell 2B_1_1	SAV_2	29.7	20.3	27.4	18.1	25.5	16.5
Cell 2B_1_2	PSTA	22.8	16.9	22.1	16.1	21.7	15.5
Cell 2B_1_2	PSTA_2	21.2	15.3	20.9	14.8	20.7	14.5
Cell 2B_1_2	SAV_2	21.2	14.3	21.1	14.0	21.0	13.8
Cell 2B_1_1	SAV_C4_2	27.7	21.0	24.7	18.2	22.4	16.1
Cell 2B_1_2	PSTA	21.2	15.9	20.0	14.8	19.1	13.9
Cell 2B_1_2	PSTA_2	19.8	14.4	18.9	13.6	18.3	12.9
Cell 2B_1_2	SAV_C4_2	16.0	12.2	15.9	11.9	15.9	11.8

The recommended general layout and configuration of the PSTA demonstration project is shown in Figure 4. As shown in Figure 4, Cell 2B_1_3 has been further subdivided into two SAV cells in series (designated as Cells 2B_1_3 and 2B_1_4), reflecting additional guidance received from the District's working group review meeting on August 13, 2003. All DMSTA simulations prepared for this Conceptual Design were conducted prior to that further subdivision, with the result that the combined performance of those two cells must be considered as represented by the simulation results for Cell 2B_1_3 as it existed prior to that further subdivision.

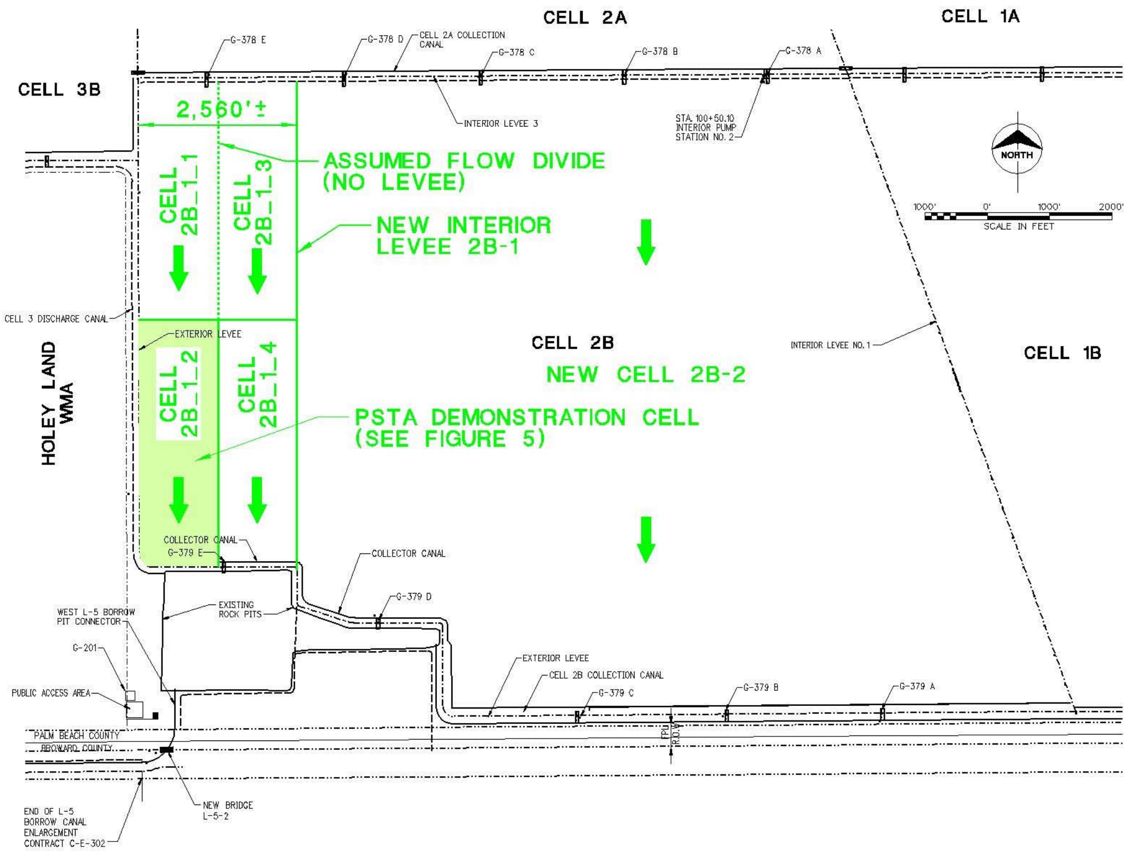


Figure 4: General Plan, PSTA Demonstration Project

4. CONCEPTUAL DESIGN PERFORMANCE SIMULATIONS

As noted in Section 3, it is anticipated that the operation of the PSTA Demonstration project may well require that the hydraulic and total phosphorus loading rates be reduced in order to demonstrate the capacity of this treatment technology to achieve a long-term geometric mean concentration of 10 ppb. As the demonstration project is to be constructed within the footprint of an operating STA subject to permit compliance, it was considered necessary to simulate the performance of Cell 2B, reconfigured as recommended herein, to assess the potential impact of the project on the overall performance of STA-3/4. Those simulations were also structured to permit, to the extent practicable, a direct comparison of the projected performance of the two parallel flow paths developed along the westerly border of Cell 2B.

For those simulations, the DMSTA hydraulic parameters were updated to reflect site-specific conditions in the PSTA demonstration cell, as well as projected seepage inflows to the demonstration cell from both the adjacent Cell 2B_1_4 (on the east) and the Cell 3 Discharge Canal (on the west).

An initial set of simulations was prepared on the base assumption that Cell 2B_1_1 and Cell 2B_1_3 are each assigned 10% of the daily discharges from Cell 2A. Simulations were conducted for each possible combination of the available SAV and PSTA calibration data sets. The results of those simulations (which, again, do not reflect the subsequent subdivision of Cell 2B_1_3) are summarized in Table 7.

Table 7: Project Performance at Full Design Inflows

SAV Calib. Data Set	Simulated Outlet TP Concentrations (ppb) for					
	PSTA in Cell 2B_1_2, all others SAV					
	2B_1_2		2B_1_3		Entire Cell 2B	
	FWM	GEO	FWM	GEO	FWM	GEO
SAV	23.6	17.2	25.2	15.3	23.3	14.4
NEWS	23.3	16.8	23.3	14.4	21.1	13.3
SAV_C4	19.2	14.0	15.9	10.4	14.1	9.3
SAV_2	22.1	16.0	21.4	13.2	19.5	12.1
NEWS_2	22.0	16.0	20.9	13.5	18.8	12.3
SAV_C4_2	20.0	14.8	16.8	11.3	14.9	10.0
	PSTA_2 in Cell 2B_1_2, all others SAV					
	2B_1_2		2B_1_3		Entire Cell 2B	
	FWM	GEO	FWM	GEO	FWM	GEO
SAV	22.3	15.8	25.2	15.3	23.2	14.2
NEWS	21.9	15.5	23.3	14.4	20.9	13.1
SAV_C4	18.1	12.8	15.9	10.4	14.0	9.1
SAV_2	20.9	14.7	21.4	13.2	19.4	11.9
NEWS_2	20.8	14.7	20.9	13.5	18.7	12.1
SAV_C4_2	18.9	13.5	16.8	11.3	14.8	9.7
Note: All above for 10% of Cell 2A Outflow assigned to Cell 2B_1_1, 10% assigned to Cell 2B_1_3, 80% assigned to balance of Cell 2B. Data for "Entire Cell 2B" is composite performance of all three parallel flow paths as taken from the DMSTA output.						

The above information does not present a truly balanced comparison of the simulated performance of PSTA to adjacent SAV, due primarily to the influence of cell-to-cell seepage on the overall hydraulic loading of the two cells. This condition can be expected to significantly influence measured performance during operation of the demonstration project, despite attempts to locate the PSTA demonstration cell where such influences are minimized.

A comparison of the simulated hydraulic performance of the various “new” cells in what is now Cell 2B is presented in Table 8. Inspection of that data confirms that the performance of Cells 2B_1_2 and 2B_1_3 will be substantially influenced by seepage (primarily from Cell 2B_1_3 to Cell 2B_1_2). It will be necessary to develop a monitoring and evaluation program for the demonstration project that properly accounts for the influence of seepage inflows in the PSTA demonstration cell.

Table 8: Simulated Hydraulic Performance at Full Design Inflows

Description	Units	Simulation Results by Cell			
		Cell 2B_1_1	Cell 2B_1_2	Cell 2B_1_3	Cell 2B_2
Total Outflow	hm^3/yr	22.2	22.4	20.9	176.8
Total Outflow	ac-ft/yr	18,000	18,200	16,900	143,300
Cell Depths					
Control Depth	cm	60	30	60	60
Average	cm	60.9	39.4	57.8	60.5
Minimum	cm	12.6	18.8	---	12.5
Maximum	cm	98.5	94.7	98.3	97.3
Frequency < 5 cm	%	0.0	0.0	0.3	0.0
Flow/Width	m^2/day	156	166	147	149
Hydraulic Loading Rate					
Mean	cm/day	13.2	13.8	6.6	5
Maximum	cm/day	84.3	91.5	42.2	31.7

4.1. Potential Reduced Loading and Impact on STA Performance

The results of the simulations summarized in Table 7 indicate that, if 10% of the total discharge from Cell 2A is assigned to Cells 2B_1_1 and 2B_1_2 in series, the planning objective of a long-term mean geometric concentration of 10 ppb in outflows from the PSTA demonstration cell cannot be presently forecast. As a result, it may be necessary to reduce the hydraulic and phosphorus loading on that flow path in order to demonstrate the ability of this particular technology to achieve the planning objective. The implementation of such reductions during operation of the implementation project would act to increase the loading on the balance of Cell 2B, leading to potential concerns over the impact of the demonstration project on the performance of the STA as a whole. Given those potential concerns, an additional set of simulations were prepared in which the proportion of Cell 2A discharges directed to Cell 2B_1_1 and Cell 2B_1_3 were varied. The analyses proceeded in an iterative fashion until a long-term geometric mean TP concentration in outflows from Cell 2B_1_2 (the PSTA demonstration cell) was obtained.

Those iterative analyses were prepared for three different Submerged Aquatic Vegetation calibration data sets (SAV_2, NEWS_2 and SAV_C4_2) in Cells 2B_1_1 and 2B_1_3, and the PSTA_2 calibration data set in Cell 2B_1_2.

For "SAV_2" in Cell 2B_1_1, the simulations suggest that it would be necessary to reduce the hydraulic loading to the cell from 10% of the Cell 2A outflows to 5.4% of the Cell 2A outflows. The estimated flow-weighted mean and geometric mean concentrations in inflows to the PSTA demonstration cell were 21.7 and 13.2 ppb, respectively. The estimated flow-weighted mean and geometric mean concentrations in outflows from the PSTA demonstration cell were 15.1 and 10.0 ppb, respectively. The estimated flow-weighted mean and geometric mean concentrations from Cell 2B as a whole were 19.3 and 11.4 ppb, respectively. From Table 3, the estimated flow-weighted mean and geometric mean concentrations from Cell 2B as it is presently designed, and if acting as "SAV_2", were 19.3 and 11.6 ppb, respectively.

For "NEWS_2" in Cell 2B_1_1, the simulations suggest that it would be necessary to reduce the hydraulic loading to the cell from 10% of the Cell 2A outflows to 5.5% of the Cell 2A outflows. The estimated flow-weighted mean and geometric mean concentrations in inflows to the PSTA demonstration cell were 21.5 and 13.8 ppb, respectively. The estimated flow-weighted mean and geometric mean concentrations in outflows from the PSTA demonstration cell were 15.0 and 10.0 ppb, respectively. The estimated flow-weighted mean and geometric mean concentrations from Cell 2B as a whole were 18.6 and 11.7 ppb, respectively. From Table 3, the estimated flow-weighted mean and geometric mean concentrations from Cell 2B as it is presently designed, and if acting as "NEWS_2", were 18.5 and 12.0 ppb, respectively.

For "SAV_C4_2" in Cell 2B_1_1, the simulations suggest that it would be necessary to reduce the hydraulic loading to the cell from 10% of the Cell 2A outflows to 6.7% of the

Cell 2A outflows. The estimated flow-weighted mean and geometric mean concentrations in inflows to the PSTA demonstration cell were 19.8 and 13.8 ppb, respectively. The estimated flow-weighted mean and geometric mean concentrations in outflows from the PSTA demonstration cell were 14.6 and 10.0 ppb, respectively. The estimated flow-weighted mean and geometric mean concentrations from Cell 2B as a whole were 14.6 and 9.4 ppb, respectively. From Table 3, the estimated flow-weighted mean and geometric mean concentrations from Cell 2B as it is presently designed, and if acting as "SAV_C4_2", were 14.4 and 9.3 ppb, respectively.

As a result of those simulations, it is concluded that construction and operation of the PSTA demonstration project in Cell 2B as generally described herein can be expected to have little influence on the overall operating results from Cell 2B of STA-3/4, even if the new cells operated under markedly reduced hydraulic loading rates. It is further concluded that the long-term flow-weighted mean and geometric mean TP concentrations in inflows to the PSTA cell is relatively insensitive to the performance of the upstream SAV community when the hydraulic loading rate is reduced to achieve a geometric mean concentration of 10 ppb in outflows from the PSTA cell.

5. CONCEPTUAL DESIGN OF FACILITIES

This conceptual design of the PSTA Demonstration Cell (Cell 2B_1_2) was developed using topographic and subsurface data developed during the detailed design of STA-3/4. While adequate for that purpose, the detailed design of the Demonstration Cell should include the acquisition of a substantially denser topographic and muck depth survey. All elevations reflected in this conceptual design should thus be considered as preliminary, as they are based on the best available information, and may require adjustment during detailed design. Preliminary design computations on which the information presented herein is based is included as an Appendix to this document to facilitate subsequent adjustment, and to more fully communicate the initial assumptions implicit in this conceptual design. A conceptual plan of the PSTA Demonstration Cell 2B_1_2 and the adjacent SAV Cell 2B_1_4 is presented in Figure 5.

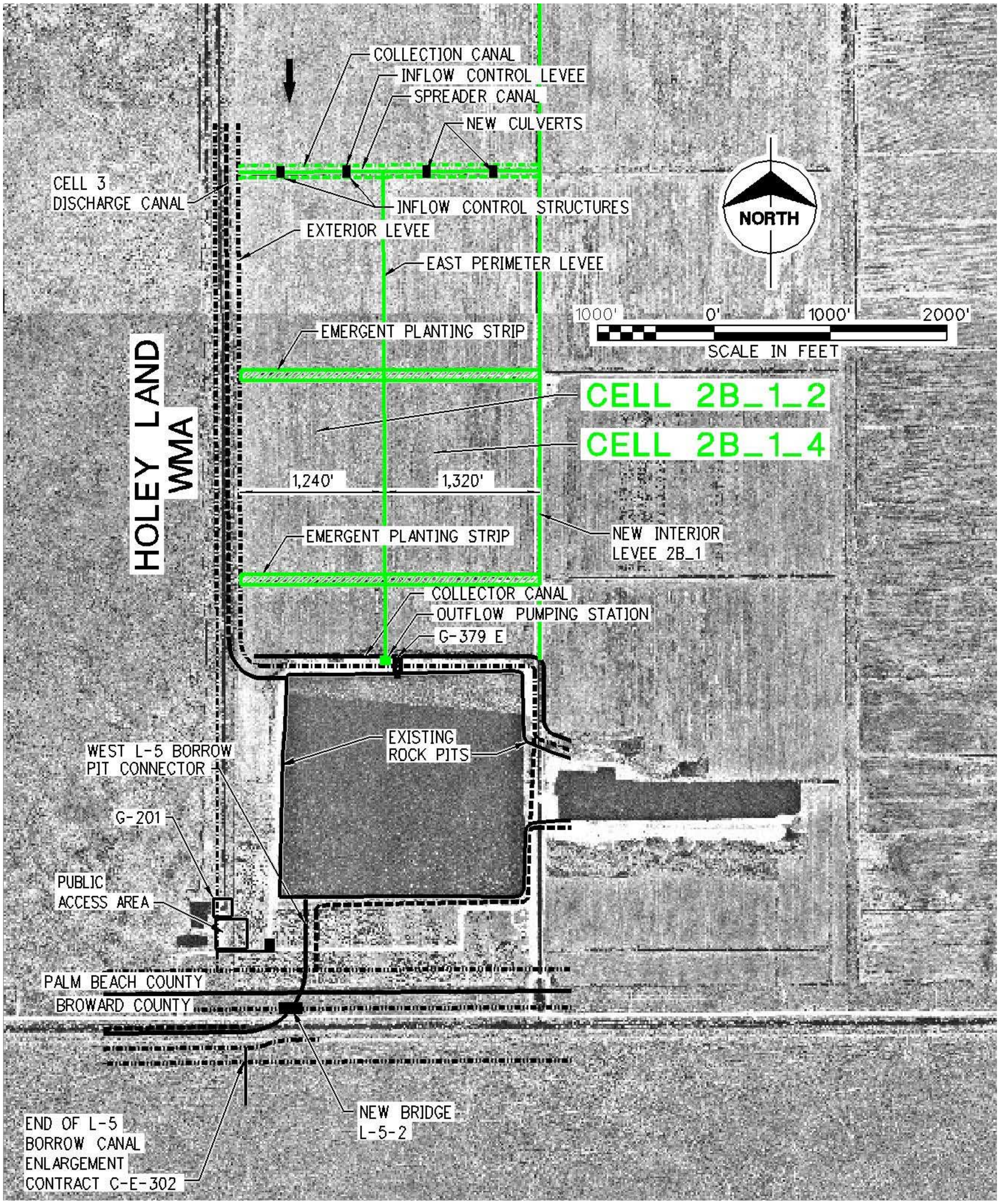


Figure 5: PSTA Demonstration Cell 2B_1_2

Table 9 presents a summary of estimated controlling data for the PSTA Demonstration Cell. Water surface profiles in the cell were estimated based on the results of ananalysis of water surface profiles in Shark River Slough (Bolster and Saiers, 2002).

Table 9: PSTA Cell 2B_1_2 Data

Design Element	Units	Values
Cell Dimensions		
Length (ctr-ctr levees)	Ft.	3,960
Width (ctr-ctr levees)	Ft.	1,260
Cell Area		
Gross (ctr-ctr levees)	Acres	114.5
Effective (toe-toe levees)	Acres	107.4
Estimated Existing Topography		
Maximum Ground Elevation*	Ft. NGVD	10.5
Average Ground Elevation	Ft. NGVD	10.2
Minimum Ground Elevation*	Ft. NGVD	9.9
Estimated Top of Caprock		
Maximum Elevation	Ft. NGVD	9.5
Average Elevation	Ft. NGVD	8.8
Minimum Elevation	Ft. NGVD	8.2
Static Water Surface		
Depth above Average Top of Caprock	Ft.	1.0
Elevation	Ft. NGVD	9.8
Design Inflow/Outflow Rate		
Overall Mean	cfs	25
Mean on Days with Outflow	cfs	34
Maximum	cfs	210
Estimated Water Surface Elevations		
Under Maximum Inflow/Outflow (210 cfs)		
Downstream End	Ft. NGVD	11.9
Cell Midpoint	Ft. NGVD	12.4
Upstream End	Ft. NGVD	12.7
Under Mean Inflow/Outflow (25 cfs)		
Downstream End	Ft. NGVD	9.8
Cell Midpoint	Ft. NGVD	10.2
Upstream End	Ft. NGVD	10.3
* Neglecting existing agricultural berms and canals		

5.1. Structure Data

It is anticipated that a total of three new structures will be required for the PSTA Cell 2B_1_2, two inflow control structures and one outflow pumping station. The inflow control structures would be situated at the northerly end of Cell 2B_1_2, and are expected to consist of gated reinforced concrete box (RCB) culverts equipped for remote control and monitoring through the District's telemetry system. A summary of the anticipated controlling design data for these structures is presented in Table 10.

Table 10: PSTA Cell Inflow Control Structure Data

Design Element	Units	Values
Total Inflow Rate		
Overall Mean	cfs	25
Mean on Days with Outflow	cfs	34
Maximum	cfs	210
Static (Control) Water Surface Elevation		
Headwater	Ft. NGVD	10.9
Tailwater	Ft. NGVD	9.8
Design Discharge Conditions		
Normal Range of Operation		
Total Discharge (cfs)	cfs	0-210
Headwater Range	Ft. NGVD	10.0-13.2
Tailwater Range	Ft. NGVD	9.2-12.7
Estimated Std. Project Flood Elev. (for structure protection)	Ft. NGVD	16.1
Structure Geometry		
Number of Structures	---	2
Number of Barrels Per Structure	---	1
Peak Inflow Rate Per Structure	cfs	105
Barrel Type & Size	RCB	6'x6'
Invert Elevation	Ft. NGVD	2.0
Inlet Type	Headwall w/ Slide Gate	
Outlet Type	Headwall or Projecting	

As it is intended that inflows to the PSTA Demonstration Cell be flow-proportional to discharges from Cell 2A, it is anticipated that the slide gates on the inflow control structures be operated in response to measured discharges from Structure G-378E. The proportion of G-378E discharges assigned to the PSTA Demonstration Cell would normally be established at 50%. Should it be found necessary to reduce the hydraulic loading on the demonstration project to achieve an outflows geometric mean concentration of 10 ppb, that reduction would be effected by reducing the gate opening at G-378E as compared to that at G-378A through G-378D. This operational need suggests the desirability of modulating the gate openings as at G-378E and the new inflow control structures to the PSTA cell as necessary to accomplish the desired hydraulic loading rates and flow distribution.

A new outflow pumping station will be required to lift discharges from the PSTA Demonstration Cell. This pump station would be situated in the existing Cell 2B Collection Canal at the southeasterly corner of the Cell, immediately west of the existing Cell 2B outflow control structure G-379E. The pumping station will draw from the Cell 2B Collection Canal west of the structure, and could discharge to either to the Cell 2B Collection Canal east of the structure (e.g., headwater pool for Structure G-379E) or across the South Exterior Levee directly to the rock pit. The latter point of discharge (e.g., across the South Exterior Levee to the rock pits) is recommended, so that discharges from Cell 2B_1_2 and Cell 2B_1_4 may be independently monitored.

Table 11: Outflow Pump Station Design Data

Design Element	Units	Values
Total Nominal Discharge Capacity	cfs	210
Intake Water Surface Elevations		
Start Pumping	Ft. NGVD	9.9
Normal Draw-Down Pumping	Ft. NGVD	9.7
Minimum Draw-Down Pumping	Ft. NGVD	8.0
Maximum Pumping, Normal Operation	Ft. NGVD	12.2
Std. Project Flood Elevation	Ft. NGVD	15.6
Discharge Water Surface Elevations		
Std. Project Flood Elevation	Ft. NGVD	15.6
Maximum Pumping, Normal Operation	Ft. NGVD	12.6
Minimum Pumping, Normal Operation	Ft. NGVD	10.0
Minimum Non-Pumping	Ft. NGVD	9.0
Total Number of Pumps	---	4
Pump No. 1 Data		
Nominal Capacity	cfs	35
Intake Elevation, Start	Ft. NGVD	9.9
Intake Elevation, Stop	Ft. NGVD	9.7
Pump No. 2 Data		
Nominal Capacity	cfs	35
Intake Elevation, Start	Ft. NGVD	10.3
Intake Elevation, Stop	Ft. NGVD	10.1
Pump No. 3 Data		
Nominal Capacity	cfs	70
Intake Elevation, Start	Ft. NGVD	11.3
Intake Elevation, Stop	Ft. NGVD	11
Pump No. 4 Data		
Nominal Capacity	cfs	70
Intake Elevation, Start	Ft. NGVD	12.2
Intake Elevation, Stop	Ft. NGVD	11.9
Cell 2B Collection Canal		
Invert Elevation	Ft. NGVD	0.0
Bottom Width	Ft.	10
Side Slopes	H:V	2:1

The pump stop elevations listed in Table 11 were established to limit the mean flow velocity at the downstream end of the treatment cell to 2 cm/sec (0.066 fps) or less. The pump start elevations were established to limit the number of pump starts to approximately 1 every 2 hours. It is presently assumed that the pumps will be driven by electric motors in the interest of overall economy and operational ease; that assumption must be confirmed during detailed design of the demonstration project.

The District may wish to consider an alternate to the pump array suggested above in which pumps salvaged from existing Pump Station G-201 (seepage return pumping station to the Holey Land Wildlife Management Area) are relocated for use in the outflow pump station, replacing Pump No. 3 and Pump No. 4. That station is to be retired from service, and is equipped with three hydraulic pumps affording a total nominal capacity of 165 cfs (Burns & McDonnell, 2000). Specific evaluation of that alternative array is beyond the scope of this Conceptual Design.

There will also be a need for new culverts at the north end of Cell 2B_1_4 to pass flows to the south from Cell 2B_1_3. These culverts need to be designed to minimize head loss, with the result that it will probably not be practicable to effect any true flow control at these structures. Their primary purpose is to provide a location at which water quality samples representative of the overall flow can be obtained.

A summary of the anticipated controlling design criteria for these structures is presented in Table 12.

Table 12: Cell 2B_1_4 Inflow Structure Data

Design Element	Units	Values
Total Inflow Rate		
Overall Mean	cfs	25
Mean on Days with Outflow	cfs	34
Maximum	cfs	210
Static (Control) Water Surface Elevation		
Headwater	Ft. NGVD	10.9
Tailwater	Ft. NGVD	10.9
Design Discharge Conditions		
Normal Range of Operation		
Total Discharge (cfs)	cfs	0-210
Headwater Range	Ft. NGVD	10.0-13.45
Tailwater Range	Ft. NGVD	10.0-13.2
Estimated Std. Project Flood Elev. (for structure protection)	Ft. NGVD	16.1
Structure Geometry		
Number of Structures	---	2
Number of Barrels Per Structure	---	1
Peak Inflow Rate Per Structure	cfs	105
Barrel Type & Size	CMP	84" Dia.
Invert Elevation	Ft. NGVD	3.0
Inlet Type	Flared End Section	
Outlet Type	Flared End Section	

5.2. Levees

The westerly and southerly levees for the PSTA Demonstration Project will consist of the South Exterior Levee constructed for STA-3/4. That levee is being constructed to a top elevation of 18.5 ft. NGVD, a minimum top width of 14 feet, and side slopes of 3H:1V.

The north levee for both Cell 2B_1_2 and 2B_1_4 will comprise the Inflow Control Levee, and is expected to be constructed to a top elevation of 18.0 ft. NGVD, a top width of 14 feet, and side slopes of 3H:1V. It will be flanked on the north by a collection canal, and on the south by a spreader canal. Each canal is expected to consist of a trapezoidal section having a bottom width of 10 feet at elevation 0.0 ft. NGVD, and side slopes of 2.5H:1V. This levee will consist of an engineered fill (controlled compaction) constructed of materials blasted from the two canals.

Both the East Perimeter Levee for Cell 2B_1_2 and new Interior Levee 2B_1 are expected to consist of compacted peat excavated from the cell interior. It is not intended that these levees accommodate vehicular traffic. The minimum required levee section at both levees would have a top width of 10 feet, a top elevation of 16.5 ft. NGVD, and side slopes of 3H:1V. In order to balance the embankment volume with the estimated volume of peat excavation, it is anticipated that the top elevation of both levees will be at a maximum elevation of 18.0 ft. NGVD. It is also anticipated that it will be desirable to revet (rock face) the westerly slope of the East Perimeter Levee of Cell 2B_1_2 in order to reduce the potential for leaching of phosphorus from the peat embankment, and to more closely approximate the desired substrate in this cell.

5.3. Emergent Planting Strips

Experience in previous and ongoing PSTA research projects suggests the desirability of including sparse emergent macrophytes in the demonstration cell; the preferred species is reported to be *Eleocharis.* The PSTA Demonstration project area is traversed east-west by existing agricultural drainage ditches excavated into and possibly below the caprock. Those ditches are flanked on the south side by an agricultural roadway constructed from the material excavated for the ditch. Those ditches will be filled and subsequently planted in emergents, forming two planting strips transverse to the general flow path. The presence of those planting strips is expected to improve the hydraulic efficiency of the PSTA demonstration cells through flow redistribution; reduce the effective wind fetch across the cell surface; and assist in controlling the potential mass movement of periphytic algae or other biomass in the water column.

It is anticipated that the roadway fill above the surrounding prevailing grade will be used to fill the ditch to the approximate top of caprock elevation. This fill material is expected to consist primarily of mineral soils excavated for the ditch, which should have a relatively low TP content. Material from the road bed below the elevation of the surrounding prevailing grade is expected to consist of a mixture of peat and mineral soils, and will be removed and handled as is intended for the overall excavation of peat from the cell interior.

For this Conceptual Design, it is assumed that the *Eleocharis* will be planted on roughly a two-foot grid, requiring a total of approximately 12,000 plantings in the two planting strips in Cell 2B_1_2, each of which is expected to roughly 20 feet in width. Based on guidance received from the August 13, 2003 of the District's working group, it was determined to extend those planting strips across Cell 2B_1_4 as well, roughly doubling the total required number of plantings.

5.4. Electrical Power Supply

It will be necessary to extend electrical power supply to the new inflow control structures and the outflow pumping station.

The present design of STA-3/4 includes the construction of a 3-phase overhead power supply line along Interior Levee 2 and 3, extending eventually to the new forward-pumping station between Cells 3A and 3B of STA-3/4. Power supply to the inflow control structures is projected to consist of a single-phase service fed from that line, extending south from Levee 3 along the South Exterior Levee and then along the PSTA Demonstration Cell inflow control levee to serve those two structures.

The present design of STA-3/4 also includes the construction of a 3-phase overhead power line along the South Exterior Levee. Overall, this line, which replaces an existing line along the L-5 Access Road, will extend from U.S. Highway 27 west to Pumping Station S-8 at the Miami Canal. It is assumed for this Conceptual Design that the line provides sufficient capacity to serve the new outflow pumping station ; that assumption must be verified during detailed design, as it directly influences the selection of pump drivers. It is presently anticipated that the total installed horsepower at this station may approach 300 hp; for analysis of power supply potential, it should be anticipated that one motor up to 100 hp will be started with 200 hp in operation.

6. OPINION OF COST

An opinion of the probable capital cost for the initial design and construction of the PSTA Demonstration Project as outlined in this Conceptual Design is presented in Table 13. That opinion of cost is based on Burns & McDonnell's experience, qualifications and judgment as design professionals. Since Burns & McDonnell has no control over weather; cost and availability of labor, material, and equipment; labor productivity; construction contractors' procedures, methods, or pricing strategies; competitive bidding and market conditions; and other factors affecting such cost opinions or projections, Burns & McDonnell does not guarantee that actual rates, costs, schedules and related items will not vary from the cost opinions and projections presented in this Conceptual Design.

Table 13: Opinion of Capital Cost

Item No.	Description	Estimated Quantity	Unit	Unit Cost	Total Estimated Cost
1	Clearing (light brush)	115	Acres	$300	$34,500
2	Peat Excavation				
	Excavate and Load	234,000	Cu. Yd.	$0.50	$117,000
	Haul to East Perimeter Levee, ave. 600' one-way haul	78,000	Cu. Yd.	$1.50	$117,000
	Place, Spread and Compact at East Perimeter Levee	78,000	Cu. Yd.	$0.35	$27,300
	Haul to Levee 2B_1, ave. 3,800' one-way haul	156,000	Cu. Yd.	$2.50	$390,000
	Place, Spread and Compact in Levee 2B_1	156,000	Cu. Yd.	$0.35	$54,600
3	Degrade Existing Roads along Levee 2B_1, Fill Existing Canal (Note 1)	34,000	Cu. Yd.	$0.50	$17,000
4	Degrade Existing Roads along east-west ditches in Cells 2B_1_2, 2B_1_4 (Note 1)	15,200	Cu. Yd.	$0.50	$7,600
5	Inflow Control Levee				
	Drill and Blast for Canal Excavation	44,800	Cu. Yd.	$0.75	$33,600
	Canal Excavation	44,800	Cu. Yd.	$1.50	$67,200
	Place, Spread and Compact in Levee Embankment	41,600	Cu. Yd.	$0.75	$31,200
	Haul to East Perimeter Levee for Revetment, ave. 1,900' one-way	3,200	Cu. Yd.	$2.00	$6,400
6	East Perimeter Levee Revetment				
	Geotextile Fabric	9,600	Sq.Yd.	$1.50	$14,400
	Place Rock	3,200	Cu. Yd.	$2.00	$6,400
7	Cell 2B_1_2 Inflow Control Structures, 6'x6' RCB with Slide Gates	2	Ea.	$125,000	$250,000
8	Electrical and Controls for Cell 2B_1_2 Inflow Control Structures	2	Ea.	$43,000	$86,000
9	Single-phase Power Line to Cell 2B_1_2 Inflow Control Structures	1.0	Mi.	$80,000	$80,000
10	Cell 2B_1_4 Inflow Structures, 84" CMP	2	Ea.	$25,000	$50,000
11	Outflow Pumping Station (Note 2)	210	cfs	5,000	$1,050,000
12	Three-phase Power Line to Outflow Pumping Station (Note 3)	0.4	Mi.	$100,000	$40,000
13	Stilling Wells (HW & TW at Cell 2B_1_2 Inflow Structures, HW at Pump Station, HW & TW at G-379E and G-378E)	7	Ea.	$9,000.00	$63,000
14	Dewatering During Construction	Job	Lump	Allow	$75,000
15	Emergent Plantings	24,000	Ea.	$2.00	$48,000
	Subtotal, Estimated Construction Cost				**$2,666,200**
	Planning, Engineering & Design @ 10%				$266,620
	Contingency @ 30%				$879,846
	Total Estimated Capital Cost				**$3,812,666**
Notes	1. This item of construction is presently included in the scope of Contract C-E307 2. Assumes temporary construction 3. Replaces single-phase power to G-379E currently planned				

District staff is preparing a separate plan and opinion of the probable cost for operation, monitoring and maintenance of the PSTA Demonstration Project in STA-3/4.

7. SCHEDULE

The *Conceptual Plan for Achieving Long-Term Water Quality Goal*s (Burns & McDonnell, 2003) includes the submittal to the Governor and Legislature of a comprehensive report on the status and results of the Process Development and Engineering component on December 31, 2008. That document is intended to define those major additional steps then considered necessary to achieve the long-term water quality goals in all discharges to the Everglades Protection Area. The schedule for construction and operation of the PSTA Demonstration Project in STA-3/4 is developed to maximize the amount of information that can be gained relative to this treatment technology for inclusion in that document.

It is anticipated that operation and monitoring of the PSTA demonstration project will extend at least through Fiscal Year 2008 (e.g., through September 30, 2008). However, development of any conceptual plans for subsequent efforts that may be included in the December 31, 2008 report will of necessity be primarily developed on the basis of data obtained prior to the end of calendar year 2007, updated as may be necessary with the additional information obtained in calendar year 2008. Given the anticipated 12-month or longer startup period anticipated for establishment of the PSTA community in the demonstration project, it is imperative that construction of the project be completed at the earliest practicable date.

It is anticipated that preparation of the detailed design can be completed on a schedule that will permit the receipt of bids in advance of the February, 2004 Governing Board meeting, with award of the construction contract(s) at that time. Construction completion is projected in January, 2005. Initial cell flooding could begin as early as late August, 2004. Allowing roughly one year for vegetation grow in after cell flooding, it is projected that the PSTA demonstration project could be in full operation commencing in early October, 2005. Three full years of operation would then be available prior to completion of the December 31, 2008 report to the Governor and Legislature. A preliminary Gantt chart of the projected implementation schedule is presented on the following page; task duration on that chart is in working days.

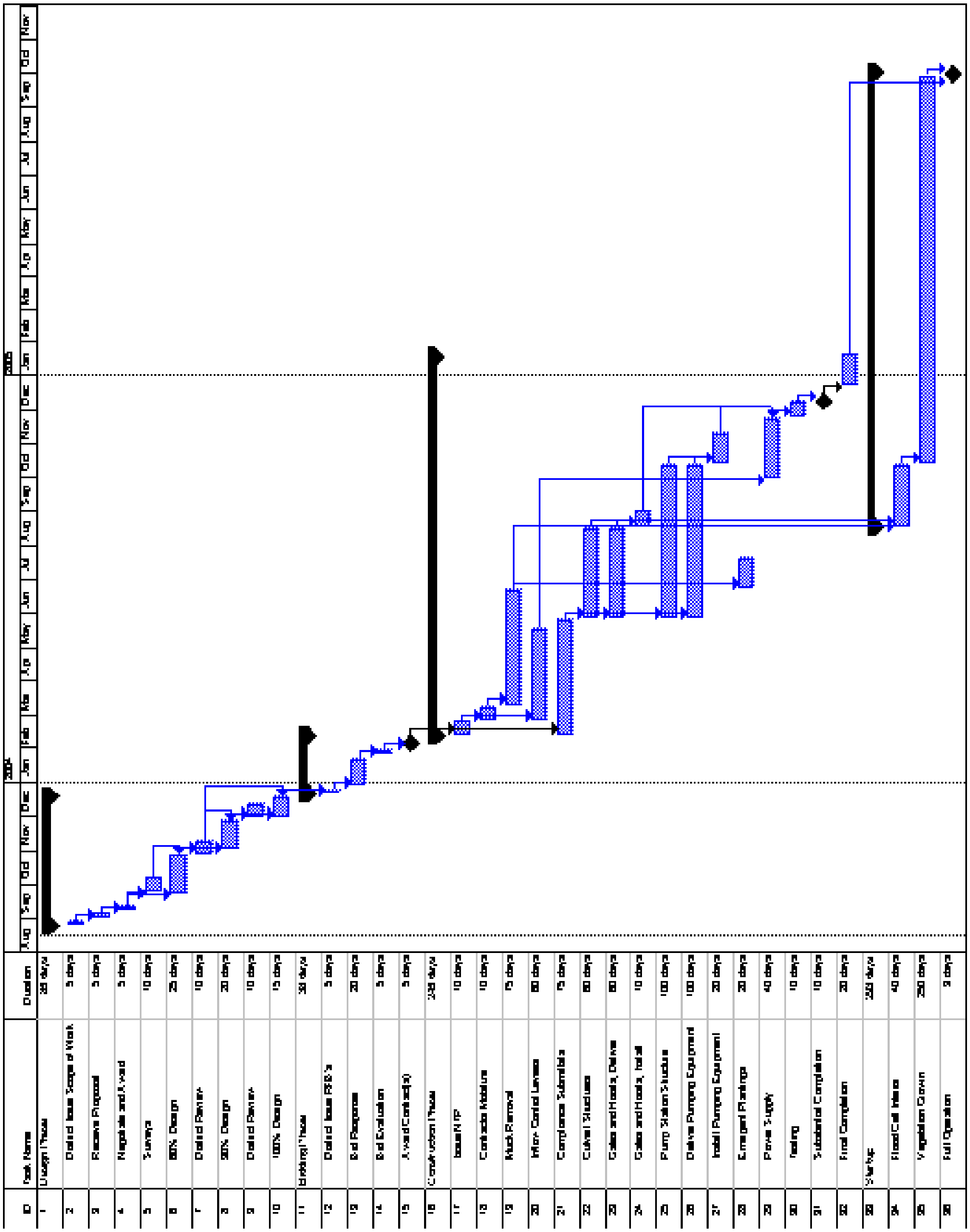


Figure 6: Demonstration Project Gantt Chart

The schedule was prepared in Microsoft Project, and that file is separately furnished to the District. A full listing of the tasks and anticipated early start and completion dates is included as the last page in Appendix A. The following is a summary of key dates taken from that schedule.

- Award contract for detailed design: September 12, 2003 (assumes issuance of a work order under an existing District continuing services contract).
- Design complete, ready to advertise for bids: December 19, 2003
- Receive bids for construction: January 23, 2004
- Award construction contracts: February 11, 2004.
- Complete primary earthwork activities: June 23, 2004
- Begin cell flooding for startup: August 19, 2004
- All construction complete: January 19, 2005
- Commence full operation: October 3, 2005 (assumes approximately one year of vegetation grow in required).

8. BIBLIOGRAPHY

Bolster, C.H., and Saiers, J.E. 2002. *Development and Evaluation of a Mathematical Model for Surface-Water Flow within the Shark River Slough of the Florida Everglades.* Journal of Hydrology, vol. 259, pp. 221-235

Burns & McDonnell, 2000. *Stormwater Treatment Area-3/4 and East WCA-3A Hydropattern Restoration, Plan Formulation.* Prepared for SFWMD, June 30, 2000.

Burns & McDonnell, 2001. *Draft Operation Plan, Stormwater Treatment Area-3/4*. Prepared for SFWMD, October 2001.

Burns & McDonnell, 2002. *Basin-Specific Feasibility Studies, Everglades Protection Area Tributary Basins, Evaluation of Alternatives for the ECP Basins.* Final report to SFWMD, October 23, 2002.

Burns & McDonnell, 2003. *Everglades Protection Area Tributary Basins, Conceptual Plan for Achieving Long-Term Water Quality Goals.* Final report to SFWMD, March 17, 2003.

Goforth, G.F. and Piccone, T., 2001. *Baseline Data for the Basin-Specific Feasibility Studies to Achieve the Long-Term Water Quality Goals for the Everglades.* Prepared by SFWMD, May, 2001.

Kadlec, R.H. and Walker, W.W., 2003. *Technology Review of Periphyton Stormwater Treatment (Draft).* August 8, 2003.

Walker, W.W. and R. Kadlec, 2003. *Dynamic Model for the Stormwater Treatment Areas*. http://wwwalker.net/dmsta/.

Appendix A
Conceptual Design Notes

Index

Burns & McDonnell
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011200 Form GCO-29B

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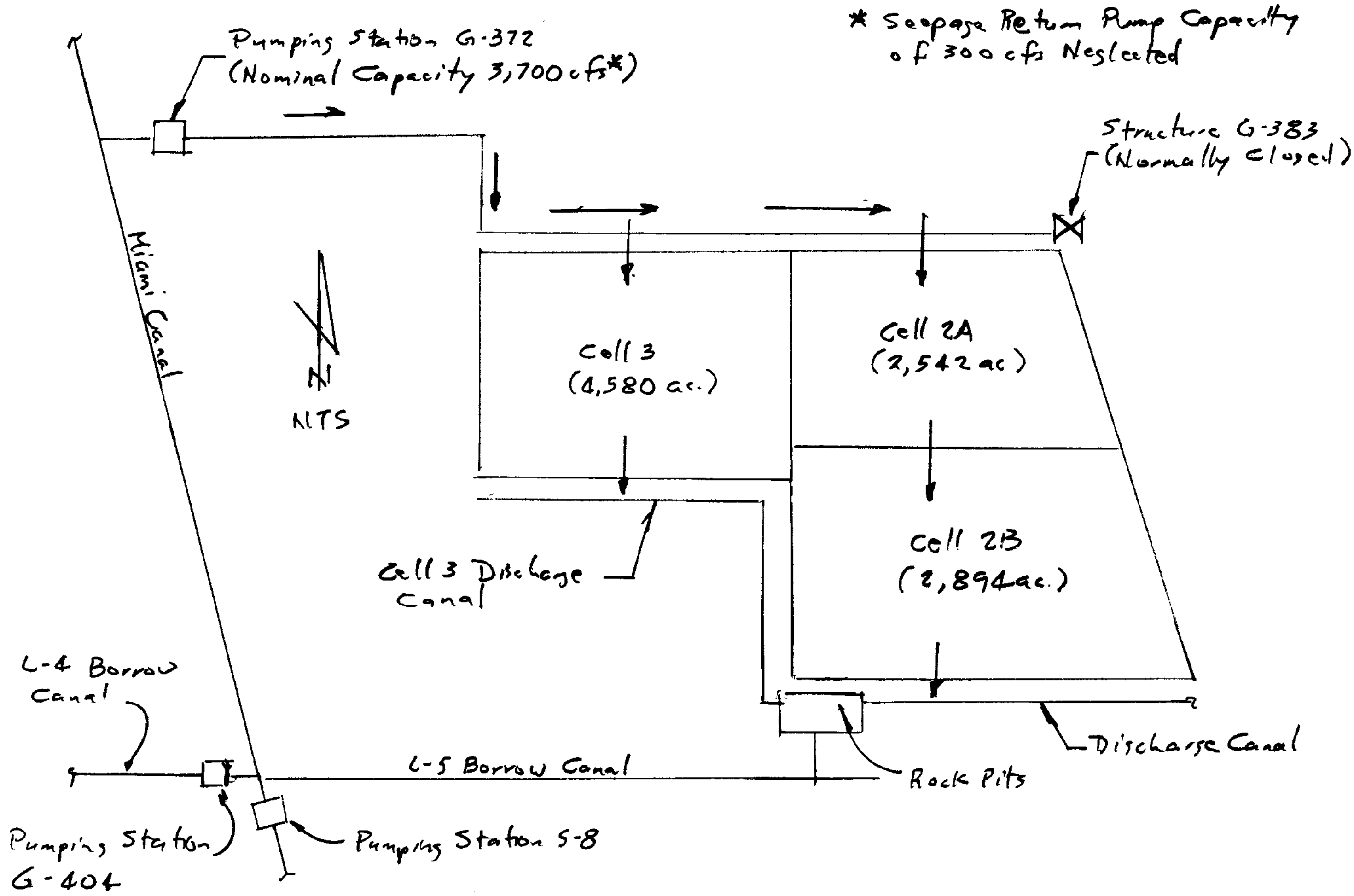
Project 34273 Date 8/01/03 Made By G. Miller

STA-3/4 Flow Distribution Checked By

Cells 2A, 2B & 3 Preliminary ✓ Final



Apportion G-372 Discharges on basis of cell area (e.g., maintain uniform distribution of HLR to each flow path):

Total area of 3 cells = 10,016 ac.

Assign $\frac{4{,}580}{10{,}016}$ = 0.457 to Cell 3,

0.543 to Cells 2A & 2B

1.000

Conclude that 54.3% of G-372 Discharges should be Assigned to Cells 2A & 2B in series

Max. Design Inflow to Cell 2A = 0.543 (3,700) = 2,009 cfs

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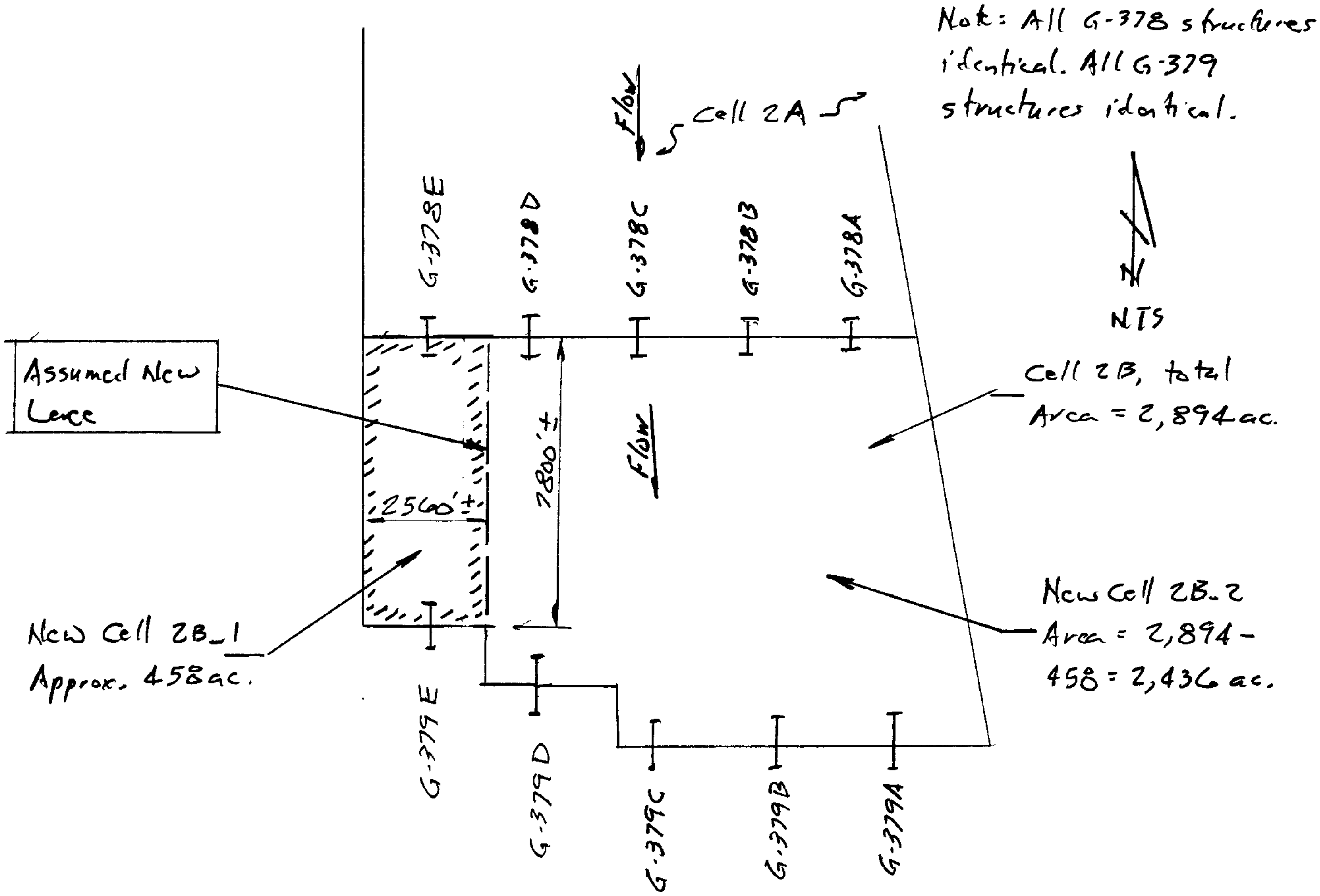
STA-3/4 Flow Distribution Checked By

Preliminary Final



Assign 20.0% of total Cell 2A outflows to Each cell 2B inflow & outflow Structure; for max. design inflow = max. design outflow in Cell 2A, max. design rate of flow to each structure = 0.20 (2,009) = $\boxed{402 \text{ cfs}}$

Then consider peak design rate of inflow to new Cell 2B_1 to be 402 cfs. Note: from DMSTA simulation for water years 1965-1994, simulated maximum rate of outflow from Cell 2A = 2099.7 cfs, with result that maximum rate of inflow to Cell 2B_1 = 2,100 (0.2) = $\boxed{420 \text{ cfs}}$ Mean daily discharge from Cell 2A = 249.8 cfs; then mean daily rate of inflow to Cell 2B_1 = 250 (0.2) = $\boxed{50 \text{ cfs}}$

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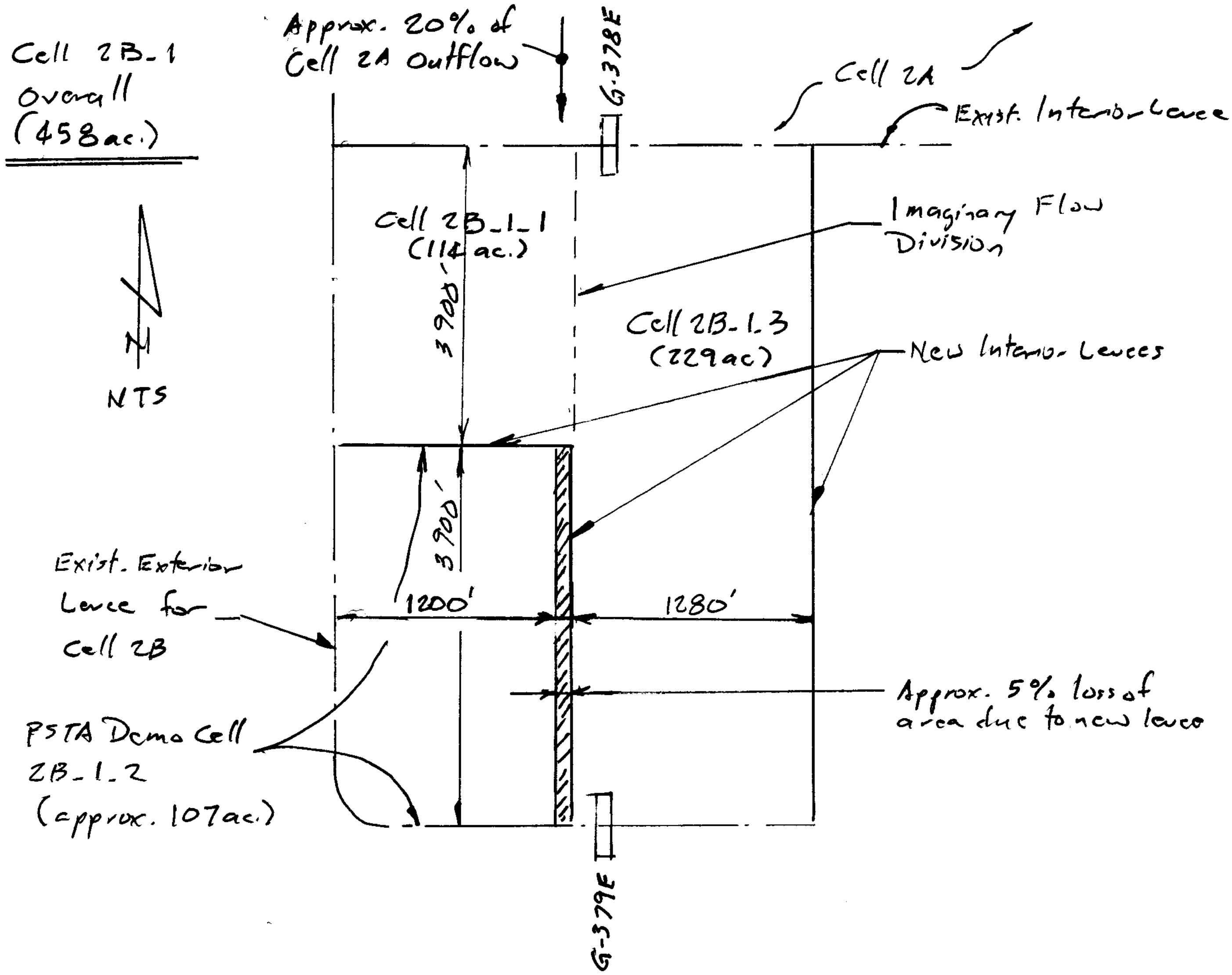
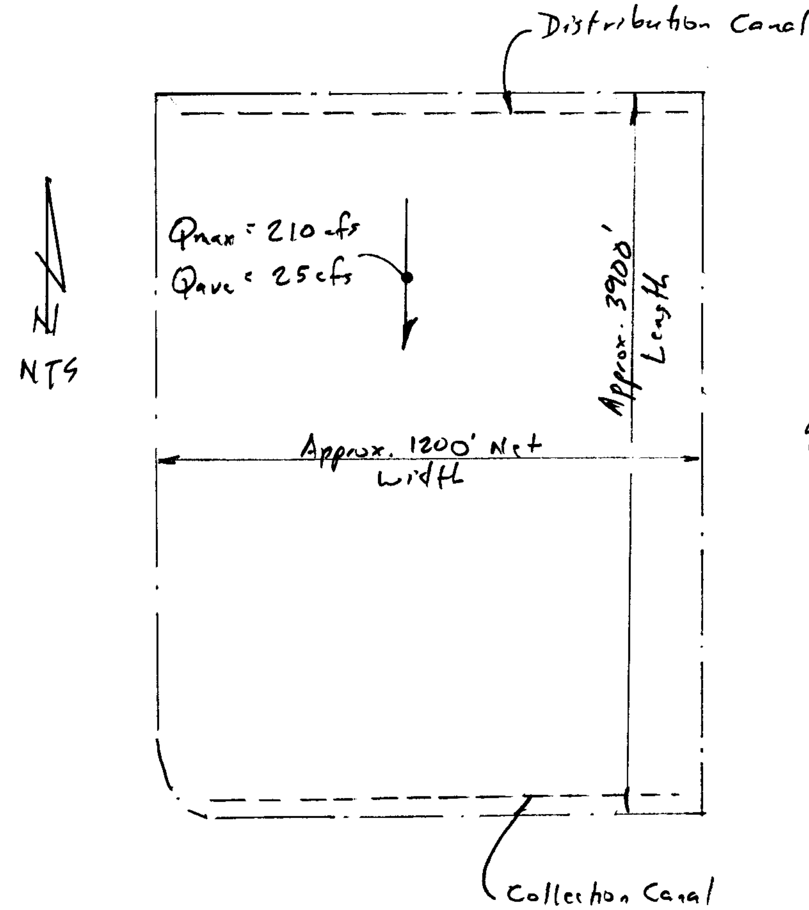


Cell 2B-1 further subdivided into two flow paths. Westerly path is SAV cell 2B-1-1 & then PSTA cell 2B-1.2 in series; easterly flow path is single cell 2B-1-3.

Assign 50% of Cell 2B-1 inflow to each flow path.

Then max. rate of inflow to Cell 2B-1-1 = 0.5(420) = 210 cfs mean rate of inflow to Cell 2B-1.1 = 0.5(50) = 25 cfs



$q_{max} = \frac{210}{1200} = 0.175$ cfs/ft

$q_{min} = \frac{25}{1200} = 0.02083$ cfs/ft

For preliminary analysis, assume cell floor at uniform elevation, no slope, analyze potential flow profile after Bolster 2002.

$v = K_f d^{\beta} S_f^{\lambda}$

v = velocity
d = characteristic depth
K_f = flow conductance coefficient
S_f = friction slope
λ = slope exponent

For this analysis, use $\lambda = 1.0$ (laminar or transitional flow)
$\lambda = 0.5$ (turbulent flow)

best-fit estimates with 95% conf. intervals $\left\{ \begin{array}{l} K_f = 6.2\times10^{6} \pm 3.8\times10^{5}\ m^{0.56} d^{-1} \\ \beta = 0.44 \pm 0.017 \end{array} \right.$

(use best-fit values)

Consider max. flow rate; assign control depth @ downstream end = 1.0 ft (30 cm)

velocity @ downstream end = q/d = $\frac{0.175 \text{ cfs/ft}}{1.0 \text{ ft}}$ = 0.175 fps = **5.3 cm/s**

For mean flow rate, $V = q/d = \frac{0.02083}{1.0}$ = 0.02083 fps = **0.63 cm/s**

Note that V = 5.3 cm/s >> than any velocity experienced in experimental platforms; for this analysis, assign V_{max} = 2.0 cm/s (0.0656 fps) then req'd. min. depth @ downstream end = 81 cm (2.67'); initiate analysis @ 2.7' (82 cm) depth for q_{max}. $K_f = 6.2 \times 10^6$; $\beta = 0.44$, $\lambda = 1.0$

Std. step-backwater method

d (m)	V (m/d)	For λ = 1.0, S =	Ave. Slope	Length (m)	Length (ft)	Σ L (ft)
0.82	1,707	0.00030				
			0.00028	286	937	937
0.90	1,555	0.00026				
			0.00025	200	656	1593
0.95	1,473	0.00024				
			0.000233	215	704	2297
1.00	1,400	0.000226				
			0.000218	229	752	3049
1.05	1,333	0.000210				
			0.000204	246	806	3855
1.10	1,272	0.000197				

Close enough for overall length = 3900'

Then, at Q_{max} of 210 cfs, assign downstream depth = 82 cm
upstream depth = 110 cm
Overall S = 0.00015 mean depth ≅ 93 cm @ mid-point

Under mean discharge of 25 cfs, assume d.s. control @ 30 cm depth,
V = 0.02083 fps = 549 m/day

Again, for $\beta = 0.44$, $K_f = 6.2\times10^6$, $\lambda = 1.0$

d (m)	V (m/day)	S	Ave. slope	Length (m)	Length (ft)	Σ Length (ft)
0.30	549	0.000150				
			0.000135	370	1213	1213
0.35	471	0.000121				
			0.000110	454	1488	2701
0.40	412	0.000099				
			0.0000945	317	1041	3743
0.43	383	0.000090				

For mean discharge rate, est. downstream depth < 30 cm
upstream depth ≈ 43 cm
ave. depth ≈ 37 cm
Overall S = 0.00011

For low flow ≈ consistent with 30 cm depth, assume 30 cm depth @ outlet, 35 cm depth @ inlet; use 33 cm ave. depth.

$$\bar{S} = \frac{0.35-.30}{3900(.304801)} = 0.000042$$

$$V = 6.2\times10^6 \times (0.33)^{0.44}(.000042)^{1.0} = 160\ \text{m/day}$$

For V = 160 m/day, $A = Bd = (1200' \times 0.304801)(0.33) = 120.7\ \text{m}^2$

Then $Q = VA = 160(120.7) = 19{,}312\ \text{m}^3/\text{day} = 7.9$ cfs

NOTE: All above for level cell floor, will be superceded by analysis incorporating estimated cell topography

Existing Ground Surface:

Reference Fig. 3.2 of 6/2000 "Plan Formulation" for STA-3/4.
By map inspection, estimate prevailing grade elevations vary from 10.0 to 10.8 ft. NGVD (slight down gradient from west to east, highest elevation in immediate vicinity of SE corner).
Estimate average grade elevation ≈ 10.1 in cell as a whole.

Muck Depth

Reference Fig. 3.1 of 6/2000 "Plan Formulation" for STA-3/4.
By map inspection, estimate muck depth varies from approx. 1.0' (vic. SE corner of cell) to approx 1.8' (vic. NE corner of cell)
Estimate average muck depth in cell ≈ 1.3'

Check By Points

At NW corner, assign ground surface ≈ 10.2', muck ≈ 1.3', caprock el. ≈ 8.9'

At NE corner, assign ground elev. ≈ 10.0, muck ≈ 1.8', caprock el. ≈ 8.2'

At SE corner, assign ground elev. ≈ 10.5, muck ≈ 1.0', caprock el. ≈ 9.5'

At SW corner, assign ground elev. ≈ 10.3, muck ≈ 1.4', caprock el. ≈ 8.9'

At E midpoint, assign ground elev. ≈ 9.9, muck ≈ 1.2', caprock el. ≈ 8.7'

At W midpoint, assign ground elev. ≈ 10.1, muck ≈ 1.3', caprock el. ≈ 8.8'

Averaging above, ground = el. 10.17, muck d = 1.33', caprock el. = 8.83'

$\boxed{\text{Use Ave. muck depth = 1'-4", Ave. caprock elev. = 8.8 ft. NGVD}}$

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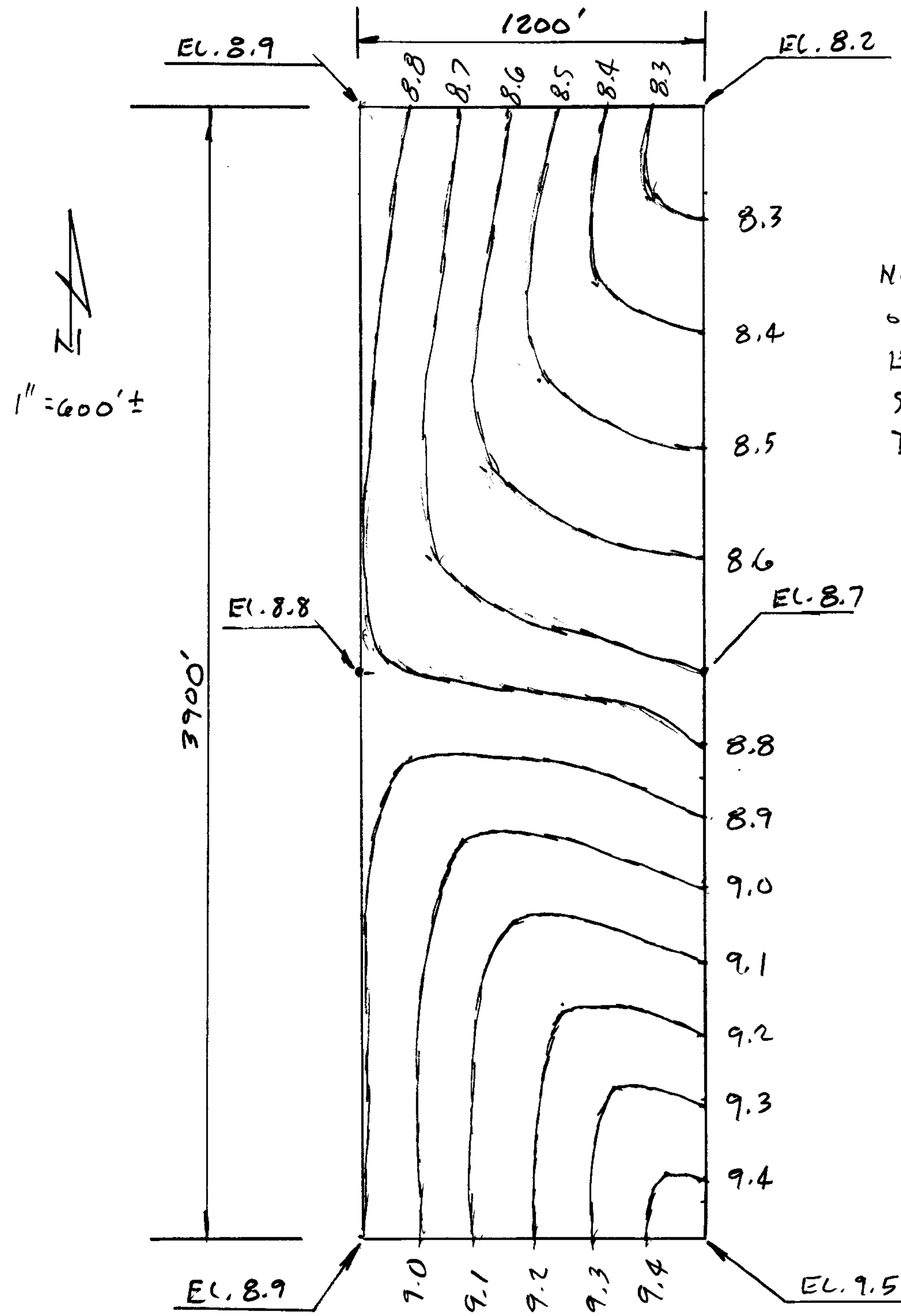
Estimated Top of Caprock Elevations in Cell ZB-1.2:



Note: Initial Approximation of Top of Caprock Elevations Only; Add'l. Survey Req'd. to Finally Define.

Upon inspection of initial estimated cell floor topography, anticipate bulk of flow will generally proceed from NE corner to SW corner.

① Set D.S. Control Elevation @ max. flow;

From cell hydraulic notes, desire mean depth @ max. flow (210 cfs) to be ≈ 82 cm = 2.7'. Anticipate mean caprock elevation along south line ≅ 9.2' NGVD. Then consider max. design water surface elevation @ outlet = 9.2 + 2.7 = $\boxed{11.9'\ \text{NGVD}}$

② Estimate U.S. Control Elevation @ max. flow; q = .175 cfs = 1404.6 m³/d/m

@ midpoint of cell, mean top of caprock el. ≈ 8.75 ft. NGVD

@ north line of cell, mean top of caprock el. ≈ 8.55 ft. NGVD

Loc.	Caprock El. (ft NGVD)	d (m)	V (m/d)	S (λ=1.0)	$\bar{S}$	L (m)	L (ft)	ΔEl. (ft)	WSEL. (ft NGVD)
D.S. End	9.2	0.82	1713	0.0003015					11.90
					0.00025	579	1900	0.48	
Mid-Pt.	8.75	1.10	1227	0.000198					12.38 ✓
					0.00018	"	"	0.34	
U.S. End	8.55	1.27	1106	0.000161					$\boxed{12.72}$ ✓

(Above calcs after Bolster, 2002 ; $V = 6.2\times10^{6}\, d^{0.44} S^{1.0}$)

③ Set Control Elevation ; desired nominal depth = 30 cm = 1.0'

For mean grade el. 8.8', control elevation nominally 9.8' NGVD

For that control elevation, mean depth @ downstream end would be 9.8 − 9.2 = 0.6' = 18 cm

For maximum velocity = 2.0 cm/sec = 0.066 fps, max. allowable q = 0.0656 (1200)(0.6) = 47 cfs (mean q = 25 cfs)

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Based on results of prelim. DMSTA simulations, can expect reduction in stage during dry periods of ≈ 45 cm. For that condition, drawdown WS EL = 9.8 − 1.5 = 8.3 ft NGVD ; conclude that entire cell would dry out at least once during the period of simulation, south end can be expected to dry out much more frequently.

Estimate WS Profile for Q_{mean} = 25 cfs ; q = 0.0208 cfs/ft = 167.2 $m^3/d/m$

Loc.	Caprock El. (ft NGVD)	d (m)	V (m/d)	S (λ=1.0)	$\bar{S}$	L (m)	L (ft)	Δ EL. (ft)	WS EL. (ft NGVD)
DS End	9.2	0.18	929	0.000319					9.80
					0.000203	579	1900	0.39	
Mid-Pt.	8.75	0.44	380	0.000088					10.19 ✓
					0.000077	"	"	0.15	
U.S. End	8.55	0.54	310	0.000066					10.34 ✓

See sketch on p. 11 for estimated profiles under mean & max. discharge.

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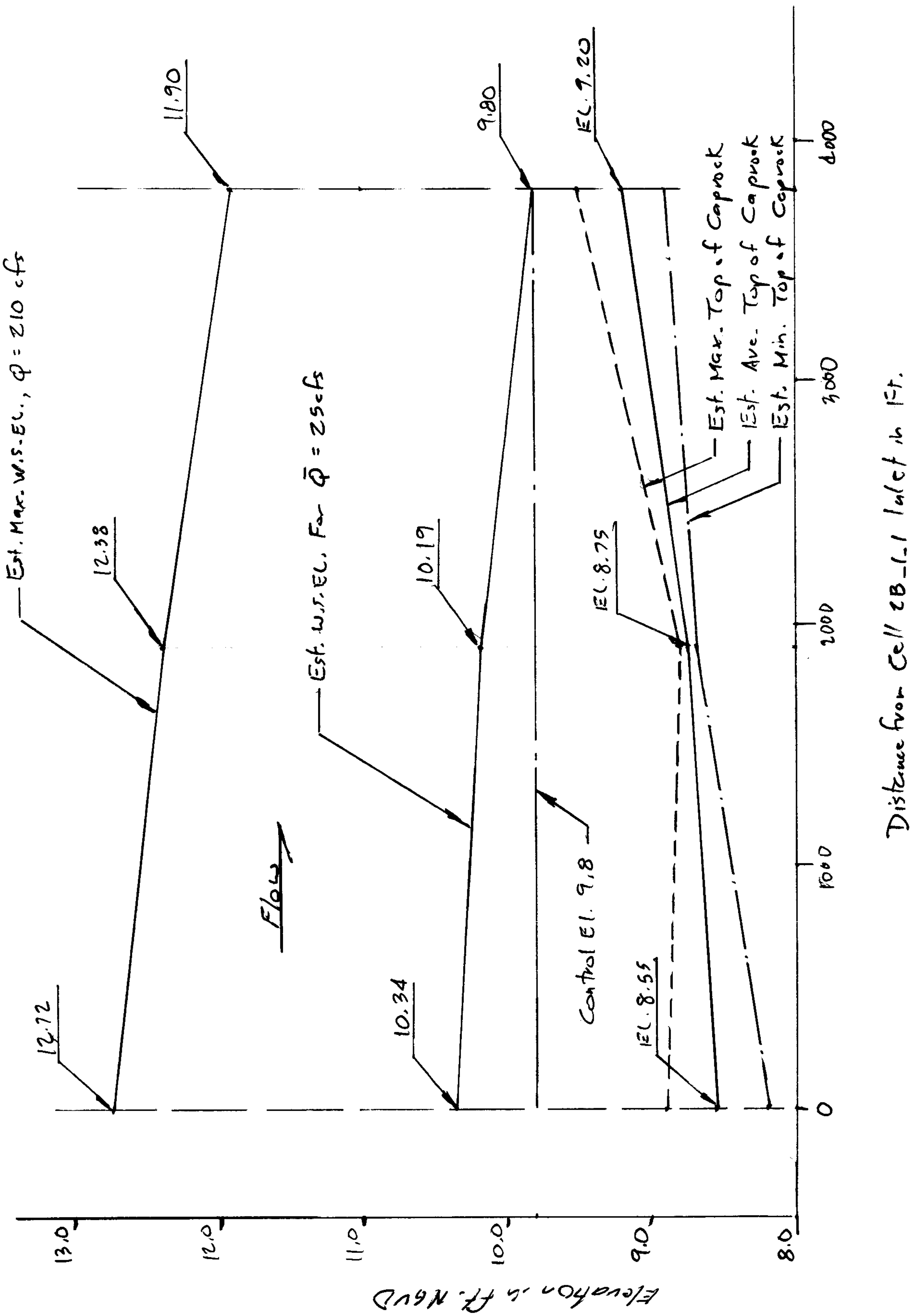
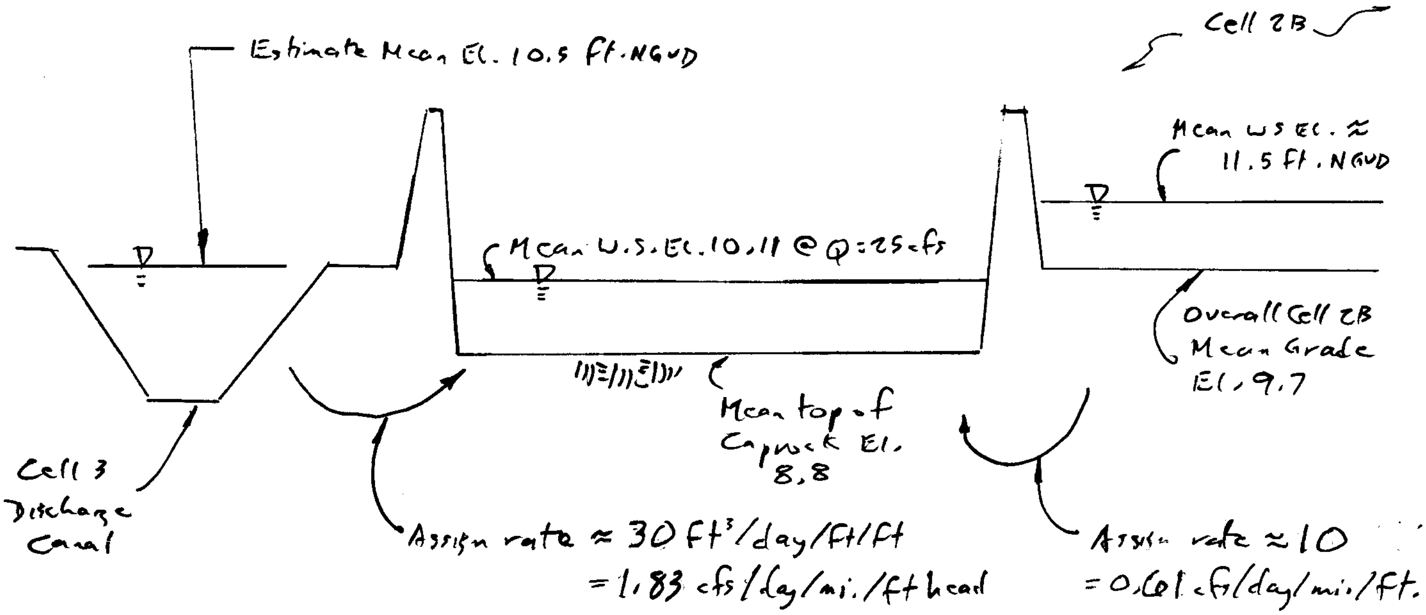
Estimated Profile Through Checked By

PSTA Demo Cell Preliminary ✓ Final



Base Analysis on Estimated Mean Conditions.

Note that PSTA cell will be abutted by Cell 3 Discharge Canal on the west, balance of Cell 2A on the east. Conceptualized cross section below, assuming approx. mean conditions throughout. Applicable to a length of 3,900'.



From Cell 3 Discharge Canal Estimate Mean Daily Inflow to PSTA Demo Cell = 30 (3900)(10.7 - 10.11) = 69,030 ft³

From adjacent Cell 2B, Estimate Mean Daily Inflow to PSTA Demo Cell = 10 (3900)(11.5 - 10.11) = 54,210 ft³

Then total estimated mean seepage inflow = 123,240 ft³/day (1.4 cfs)

Over cell area of 1200' (3900') = 4,680,000 ft², ave. seepage inflow = 123,240 / 4,680,000 = 0.026 ft/day (0.80 cm/day)

Given:
1. $Q_{max} = 210\,cfs = 0.5138\ hm^3/day$
2. $Q_{ave} = 25\,cfs = 0.0612\ hm^3/day$
3. Cell Width $= 1200' = 0.3658\ Km$
4. From pp. 9-11, estimate cell mean depths as follows:
   a. For Q_{max}, mean depth $= (2.7' + 3.63' + 4.17')/3 = 3.5'$ (1.067 m)
   b. For Q_{ave}, mean depth $= (0.60 + 1.44' + 1.79')/3 = 1.28'$ (0.389 m)

In DMSTA, $Q/W = a z^b$

$Q/W = (hm^3/day)/Km$
a = discharge @ 1 m depth
z = depth (m)
b = exponent

Then from above,

$$0.5138/0.3658 = a(1.067)^b \qquad 0.0612/0.3658 = a(0.389)^b$$

$$a = a$$

$$\frac{1.4046}{1.067^b} = \frac{0.1673}{0.389^b}$$

$$1.4046(0.389)^b = 0.1673(1.067)^b$$

$$8.3957 = 2.743^b$$

$$\log(8.3957) = (b)(\log(2.743))$$

$$0.924 = 0.438\,b, \quad b = 2.1096, \text{ use } b = 2.1$$

For $b = 2.1$, $1.4046 = a(1.067)^{2.1} \rightarrow a = 1.226$, use $a = 1.2$

Check Reasonableness: For $a = 1.2$, $b = 2.1$,

when $Q/W = 1.4046$, $z = 1.078\,m$ (target = 1.067, Δ = 0.04', say OK)

when $Q/W = 0.1673$, $z = 0.391\,m$ (target: 0.389, $\Delta < 0.01'$, say OK)

$$\boxed{\begin{array}{l}\text{Use } a = 1.2,\ b = 2.1\\ \text{Control Depth} = 30\text{ cm}\end{array}}$$

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Estimate DMSTA Seepage Constants Checked By
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Given: 1. From p. 12, estimated ave. daily inflow = 0.80 cm/day, total from two sources.
a. From Cell 2B, w.s. differential = 1.39' ≈ +42 cm
b. From Cell 3 Discharge Canal, w.s. differential = 0.39' = 12 cm

Cell 2-B-1-2

Base DMSTA input on differential to Cell 2B = 42 cm

Inflow Seepage Control Elevation = 42 cm

Inflow Seepage Rate = $0.80/42 = 0.019$ cm/day/cm

Cell 2-B-1.3

"Cell" adjacent to PSTA cell on the east. All seepage inflows to PSTA cell are considered to come from this cell.
From p. 12, ave. daily seepage out = 54,210 ft^3/day

Cell area ≈ 1280 (7900) = 10,112,000 ft^2

Then loss = $\frac{54210}{10,112,000} = 0.0054$ ft/day = 0.16 cm/day

For 42 cm differential, rate = $0.16/42 = 0.0038$ cm/day/cm

Discharge duration data taken from DMSTA output file "Demo Cell_Out.xls" file contains a daily time series for outflows from the demo cell, when 10% of overall discharge from Cell 2A is assigned to Cells 2B_1_1 and 2B_1_2 (the PSTA demo cell); DMSTA input file is "Demo_Data.xls", design case is "PSTA Daily".

(for 5% Cell 2A outflows) Mean Daily Discharge (cfs)	Cum. % of Time	✓(for 10% Cell 2A Outflows) Mean Daily Discharge (cfs)	Exceedance Frequency (%)
0	0	0	0
0	1	0	0
0	5	0	0
0	10	0	0
0	20	0	0
0	25	0.1	75
6.0	50	14.7	50
19.8	75	37.4	25
23.9	80	46.8	20
36.7	90	72.3	10
43.8	95	86.2	5
68.5	99	135.3	1
107.2	100	212.0	0

(Mean = 12.6) Mean Discharge = 25.1 cfs (entire period)

Approx. Mean Discharge on Days w/ Outflow = $\frac{25.1}{0.75}$ = 33.5 cfs

Again, all above for 10% of Cell 2A outflow assigned to PSTA flow path. Note potential during operation to reduce assigned inflows to as low as 5% of Cell 2A outflow. (e.g., 50% reduction in inflow volume & rates). Resultant discharge frequency added in left-hand column above.

Following inspection of discharge duration data, initially assign total installed pump capacity of 210 cfs distributed to four pumps as follows:

Pump No.	Nominal Capacity (cfs)
1	35
2	35
3	70
4	70
Total	210

Note that each of pumps 1 & 2 would provide capacity ≈ mean daily discharge on outflow pumping days, for max. inflow rate. Also note that pumps 1 and 2 in combination would provide adequate capacity approx. 90% of the time for max. inflow rates, and 99% of the time for min. inflow rates.

Establish min. headwater elevations for pumps on; note that control elevation = 9.8 ft. NGVD, max. velocity = 0.066 fps (2 cm/s), ave. grade elev. @ downstream end of cell ≈ el. 9.2, flow width ≈ 1200'

Q_{TOT} (cfs)	Min. A. (ft²) for V = 0.066 fps	Ave. D (ft)	WS El. (ft NGVD)	Use WS El.
35	530	0.44	9.64	9.7
70	1061	0.88	10.08	10.1
105	1591	1.33	10.53	10.6
140	2121	1.77	10.97	11.0
210	3182	2.65	11.85	11.9

Above elevations are to be considered as "pump stop" elevation for total capacities showing as operation to lower elevations would violate assigned max. velocity of 0.066 fps.

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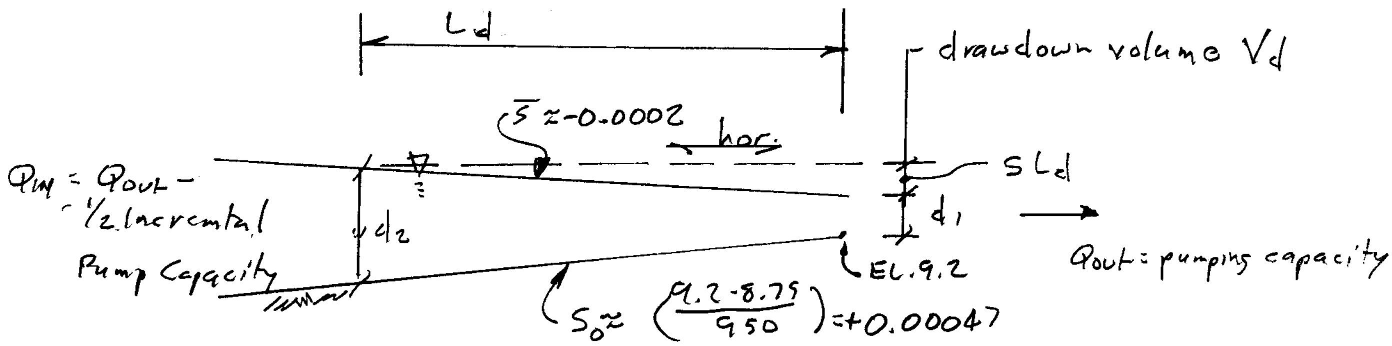
Outflow Pump Station Checked By

Pump Capacities & Control Elevations Preliminary ✓ Final

Estimate desirable pump start elevations ~ attempt to limit pump starts to approx. 1 start / 2 hrs. min, anticipate min. cycle time will occur for net outflow ≈ ½ inc. pump Q. For prelim. analysis, assign nominal WS slope ≈ 0.0002 ft/ft



For 1200' flow width,

$$\text{Drawdown Volume } V_d = \tfrac{1}{2} S L_d^2 (1200) = 600\,S\,L_d \;;\; \text{If } S \approx 0.0002,$$

$$V_d = 0.12\,L_d^2 \;(ft^3) = \tfrac{1}{2}\text{ incremental pumping capacity for 60 min.} = \tfrac{3600}{2} Q_{pump} = 1800\,Q_{pump} \;(ft^3)$$

Q_{pump} (cfs)	Q_{inc} (cfs)	V_d (ft^3)	L_d (ft)	SL_d* (ft) drawdown depth	Use drawdown of
35	35	63,000	725	0.14	0.20
70	35	"	725	0.14	0.20
105	35	"	725	0.14	0.20
140	35	"	725	0.14	0.20
210	70	126,000	1025	0.21	0.30

use 0.3' if inc. from 70 to 140

* Rough approximation for assigned WS slope = 0.0002; slope should increase somewhat at higher flow rates, up to max. of ≈ 0.0003

Then,

Pump No.	Capacity (cfs)	Total Q (cfs)	Start El. (ft NGVD)	Stop El. (ft NGVD)
1	35	35	9.9	9.7
2	35	70	10.3	10.1
3	70	140	11.3	11.0
4	70	210	12.2	11.9

Above is for simplest pump start/stop arrangement, assuming ≈ fixed capacity (e.g., constant speed) pumps.

Review need for full outflow pump capacity

Ref: October 2001 Draft Operation Plan for STA-3/4

From Table 2.8, for structures G-379 (includes G-379E adjacent to SE corner of PSTA demonstration cell)

Static HW El. (Q=0) = 10.9 ft. NGVD.
Static TW El. (Q=0) = 10.0 ft. NGVD.
Design Max. HW El. (Q=100% of design) = 12.9 ft. NGVD
Design Max. TW El. = 12.6 ft. NGVD

From Fig. 8.7 of July 2000 "Plan Formulation", estimate mean (50% duration) stage in Rock Pit ranges from 10.25 to 10.7 ft. NGVD.

Conclude that pumping over full range of discharge is necessary, whether the pump station discharges upstream or downstream of G-379E.

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Given:
1. Design Total Rate of Inflow to PSTA Cell = 210 cfs (see p. 3)
2. Design TW El. @ structures = 12.72 ft. NGVD (see p. 11)
3. Design Static WS El. (TW) ≅ 9.8 ft. NGVD. (see p. 9)
4. Minimum Grade El. D.S. ≅ 8.2 ft. NGVD (see p. 8)

Estimate Design HW El.

Ref: October 2001 Draft Operation Plan, STA-3/4

Assign Design HW El. = ave. of design HW El. @ G-379
design TW El. @ G-378

From Table 2.8 of reference, G-379 design HW = 12.9 ft. NGVD
From Table 2.6 of reference, G-378 design TW = 13.5 ft. NGVD

Then estimated design HW El. for PSTA Inflow Structures =
(12.9+13.5)/2 = 13.2 ft. NGVD.

Basic Size of Structures

Desire min. of 2 structures for op'l. flexibility & flow distribution → $Q_{ea.}$ = 210/2 = 105 cfs

Estimated ΔH = 13.2 - 12.72 = 0.48'

For initial sizing, assign $K_{exit} = 1.0\, v^2/2g$
$K_{ent.} = 0.5\, v^2/2g$
$K_{form} = 0.2\, v^2/2g$

$K_{total} = 1.7\, v^2/2g$

Then $0.48 = 1.7\, V^2/2g \rightarrow V = 4.26$ fps; req'd. minimum area = Q/A = 105/4.26 = 24.6 ft²/structure.

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Given: Reqd. A each structure = 24.63 ft² ; rectangular openings preferred to facilitate flow measurement at partial gate openings.

For square RCB, would need 5'x5' min. (25 ft²); given uncertainty in actual HW & TW elevation, consider it prudent to increase size to <u>6'x6' RCB</u>

$A_{ea.} = 36\,ft^2, \quad \Delta H @ Q = 105\,cfs \cong 1.7\left(\frac{105}{36}\right)^2/2g = 0.22\,ft$

Use 2 - 6'x6' RCB's, gated

To facilitate flow measurement over full range of operation, <u>set max. crown elev. @ 8.0 ft. NGVD, Inv. El. 2.0 ft. NGVD</u>

Given: 1. Est. Mean Top of Caprock El. ≅ 8.5 ft. NGVD
2. Set Design Top of Levee = est. SPF elev + 1.5 ft.

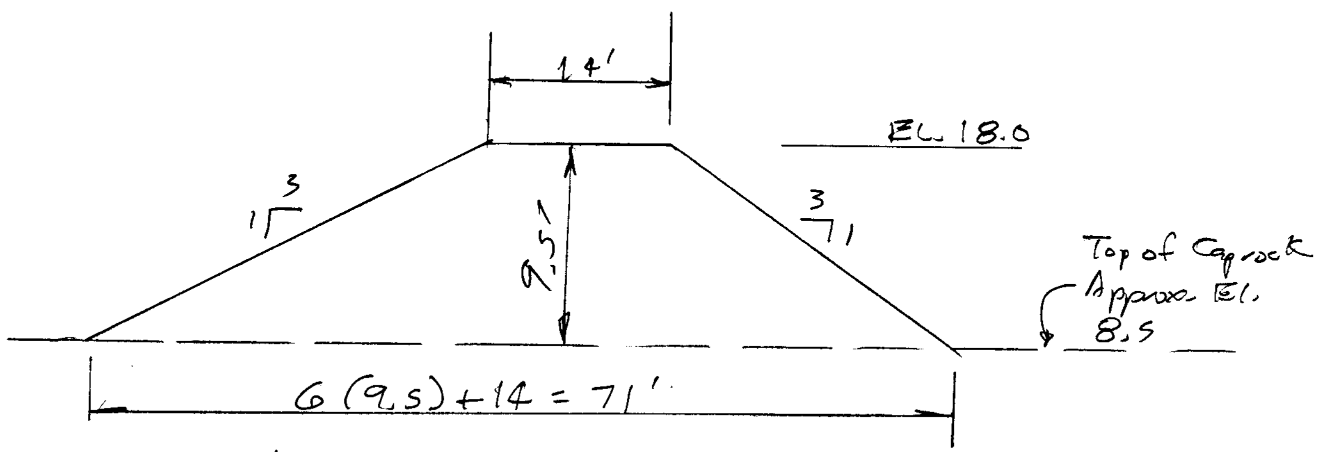
From Oct. 2001 Draft Op's. Plan,
Table 2.8 SPF @ G-379 = 15.6 ft. NGVD
Table 2.6 SPF @ G-378 = 16.5 ft. NGVD

Estimate SPF El. @ Levee = $\frac{16.5+15.6}{2}$ = 16.05 ft. NGVD

Plus 1.5' to maintain access trafficability → 17.55 ft. NGVD

<u>Set top of levee at ≈ 18.0 ft. NGVD</u>

<u>Estimate Req'd. Fill Volume</u>



End Area = $\frac{1}{2}$ (14+71)(9.5) = 404 ft²

Will be constructed of excavated caprock & marls, etc.; conservatively, allow no bulking, 10% shrinkage loss, then req'd. excavation = 444 ft² for levee construction

For L ≅ 1260' total, V = 1260 (444)/27 = 20,720 yd³

Increase excavation by approx. 3,000 yd³ to serve as borrow for facing on new east perimeter levee (muck).

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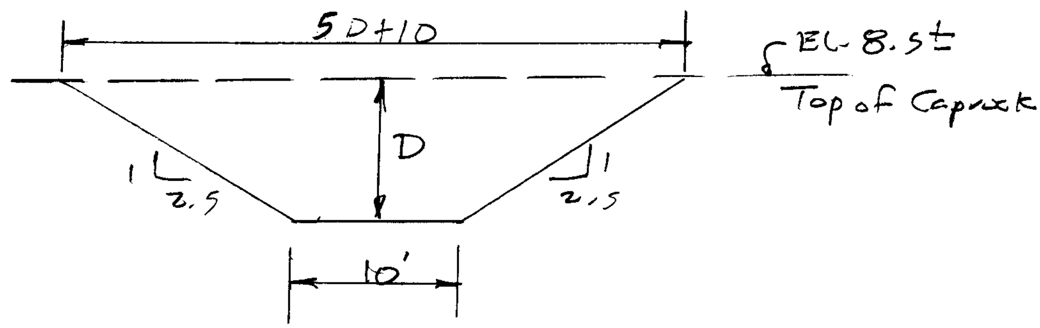
Typical Section, Inflow Control Levee Checked By

Preliminary ✓ Final

Then total desired excavation $\approx$ 20,720 + 3,000 = 23,720 yd^3

For $\pm$ 1200' collector & spreader canal length, req'd. excavation = $\frac{23{,}720\ (27)}{1200}$ = 534 ft^2.

Assign 1/2 total excavation to both collector & spreader canal, then A_{ea} = 534/2 = 267 ft^2. Assume trapezoidal channel, 10' bottom width, 2.5H:1V side slopes.



$A = 267 = \frac{1}{2}(10 + 5D + 10)(D) = (10 + 2.5D)(D) = 10D + 2.5D^2$

$2.5D^2 + 10D - 267 = 0 \rightarrow D^2 + 4D - 106.8 = 0$

$D = \frac{-4 \pm \sqrt{16 + 427}}{2} = 8.5$ ft.

Set Invert EL. = 0.0 ft. NGVD, both canals, 10' BW, 2.5H:1V side slopes

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011200 Form GCO-29B

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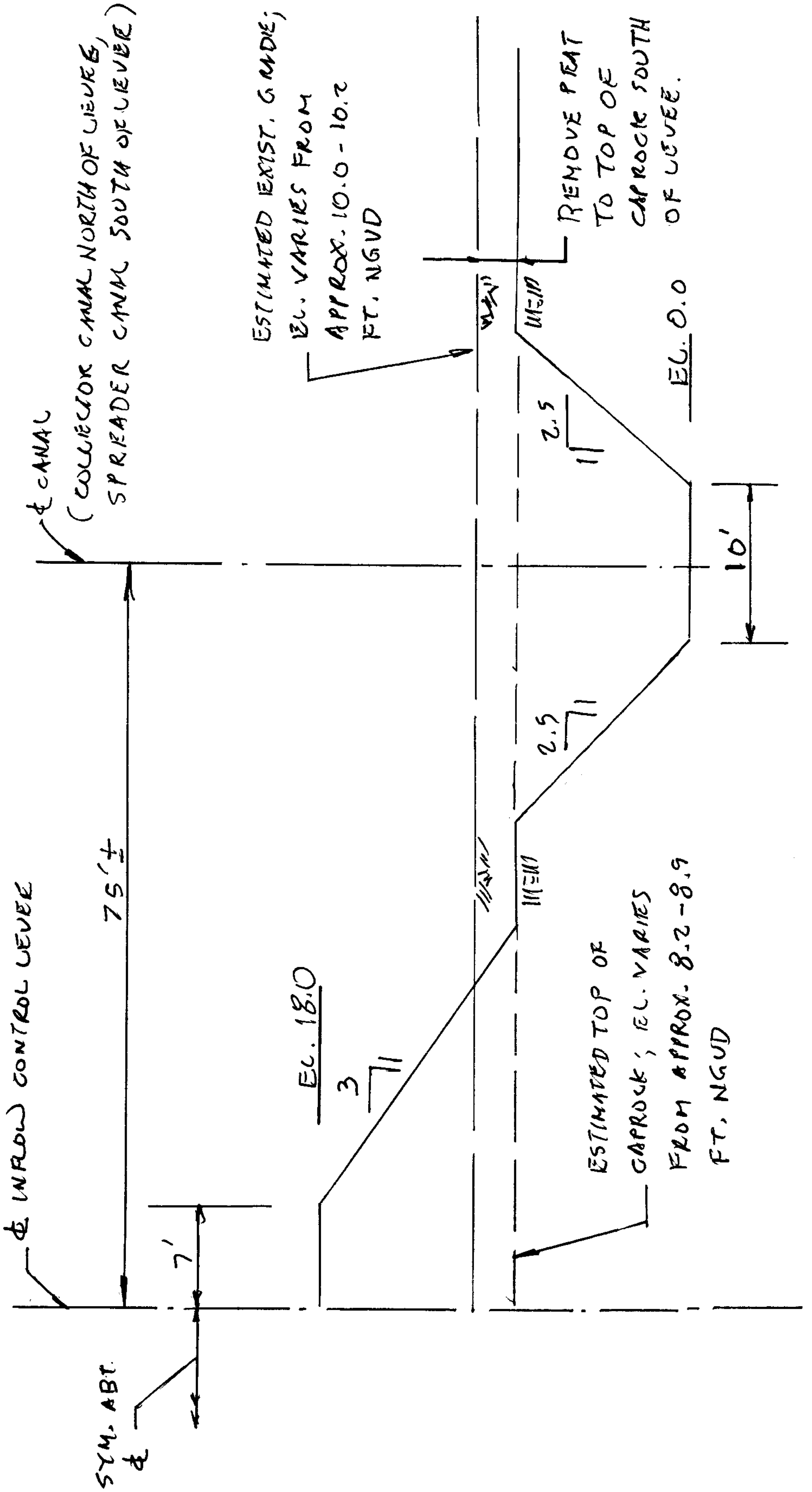
Project 34273 Date 8/07/03 Made By G. Miller

Typical Section, Inflow Control Levee Checked By

Preliminary ✓ Final



TYPICAL SECTION
INFLOW CONTROL LEVEE
Not to Scale

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Project 34273 Date 8/01/03 Made By G. Miller

Typical Section, Cell 2B-1-2 Checked By

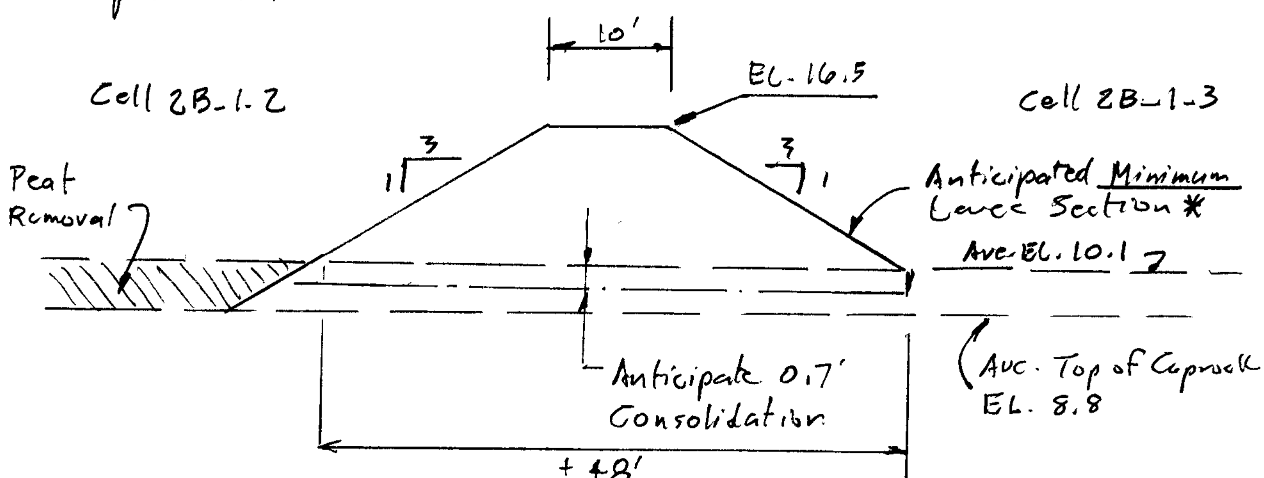
East Perimeter Levee Preliminary ✓ Final

Given:
1. Approximate Length = 3,900'
2. Ave. Existing Grade Approx. 10.1 ft. NGVD
3. Ave. Top of Caprock Approx. 8.8 ft. NGVD
4. In Cell 2B-1-3,
   a. Design WSEL @ max. inflow varies from 12.9-13.2 ft. NGVD
   b. SPF EL. varies from approx. 15.6 - 16.0
5. In Cell 2B-1-2 (PSTA Cell), max. design WSEL varies from 11.9-12.7 ft. NGVD (see p. 11)

Assume that this levee is not intended to support vehicular traffic, and will be constructed entirely of peat stripped from the cell interior.

Assign min. top elevation at higher of
① 3' above design WSEL (13.2+3.0 = 16.2 ft. NGVD)
② 0.5' above SPF elevation (16.0+0.5 = 16.5 ft. NGVD)
Use min. EL. 16.5 ; set min. top width = 10' for constructability

Consider ± 50% shrinkage from in-situ to compacted embankment, & approx. 50% consolidation of underlying peat layer. Set side slopes @ 3H:1V.



$A_{fill} \simeq \frac{1}{2}(10+48)(16.5-10.1) = 186\ ft^2$

$A_{cons.} \simeq \frac{1}{2}(48+50.1)(0.7) = 18\ ft^2$

Total Fill $= 204\ ft^2$

* See p. 28 for estimated final section for mat'l. balance.

Use 30,000 for estimate

For L = 3900', V = 204 (3900) / 27 = 29,470 compacted cubic yards

Req'd. Excavation = 29,470 / 0.5 = 58,940 bank cubic yards

Use 60,000 for estimate

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Project 34273 Date 8/01/03 Made By G. Miller

Typical Section, East Perimeter Levee. Checked By

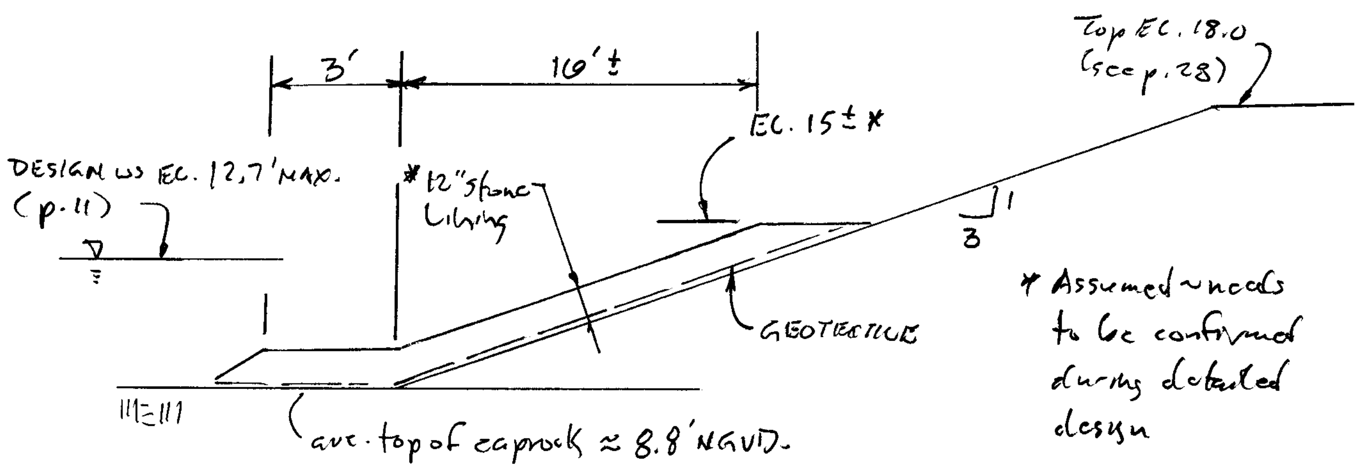
Preliminary ✓ Final

Note that the east perimeter levee will be composed of peat assumed to be high in TP content; if not otherwise addressed, leaching of TP from the soil could exacerbate difficulty in attempting to achieve extremely low concentrations ≤10 ppb. Would also serve as a potential source for propagation & spread of undesirable emergents in PSTA cell.

Consider it desirable to face the west slope (Cell 2B-1-2) side with limerock excavated from collector & spreader canals (see pp. 21-23) ~

For concept design, assign $D_{50} \cong 8''$, layer thickness = $1.5 D_{50} \approx 12''$
$D_{100} \cong 12''$

Underlay stone w/ geotextile fabric.

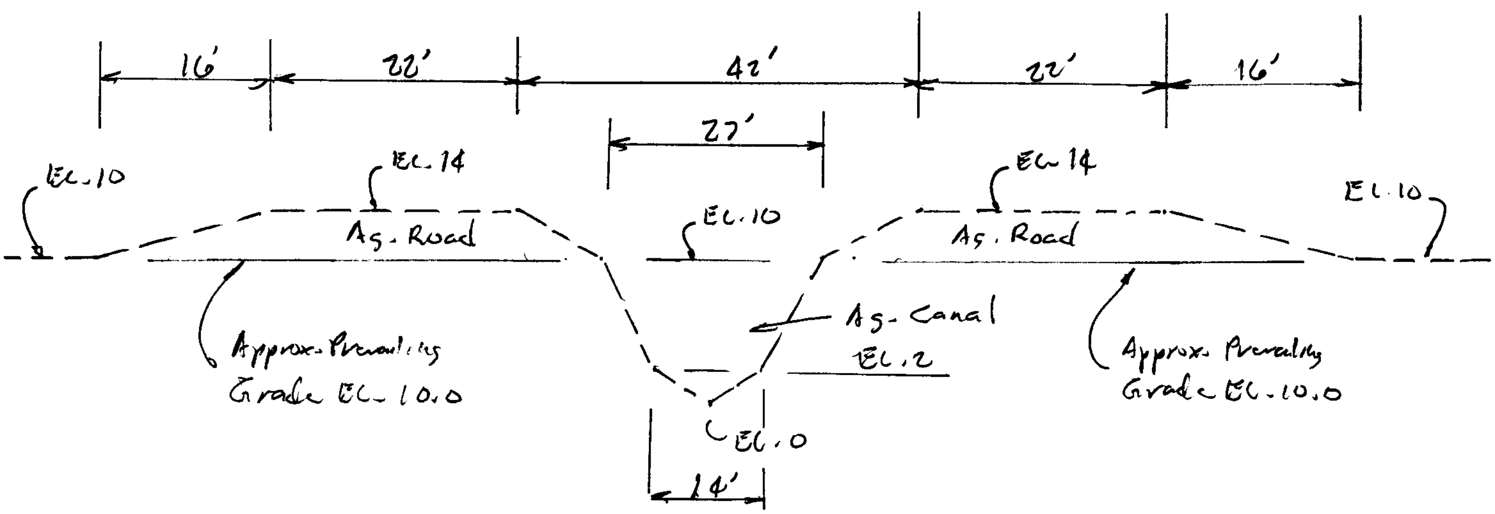


Estimated total volume of rock lining ≈ 3900' (22') (1.0) / 27 = 3,178 yd³
Say 3,200 yd³

Estimate total quantity of geotextile = 3900' (22') / 9 = 9,533 yd²
Say 9,600 yd²

This levee is intended to follow the alignment of the main north-south drainage canal in the center of what was the Griffin Bros. property. Assume representative existing grade and cross section as represented by average of cross sections XS-44 and XS-44 in Contract C-E307 construction drawings (see Shts. C102, C107).

Use for existing;



Overall length of levee ≈ 7800'

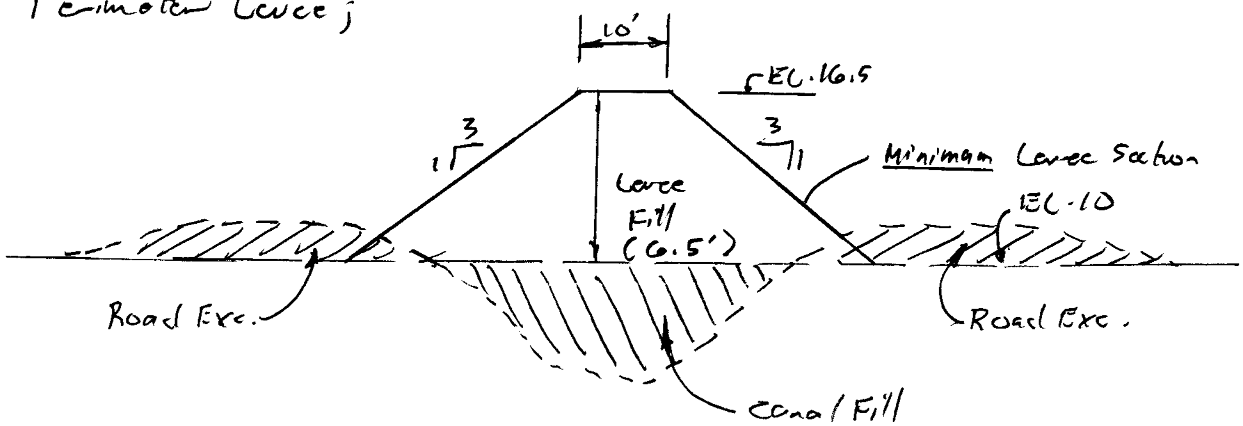
Approx. Area in Ag. Roads $= (2)(\frac{1}{2})(22+38)(4) = 240\ ft^2$ (exist. fill)
Approx. Area in Canal Below EL.10 $= \frac{1}{2}(27+14)(10-2) + \frac{1}{2}(14)(2-0)$
$= 209 + 14 = 223\ ft^2$

≈ equal, say OK

Assume ag. canal used for dewatering work area ~ anticipate filling beginning at north end and proceeding south, generally paralleling progress of muck excavation in Cell 2B-1-2.

For sequence, assume canal filled with material from road degradation, resulting in an average grade elev. of 10.0 ft. NGVD prior to placement of overlying levee.

Consider levee to be constructed of peat soils, no intended vehicular traffic, minimum req'd. levee section similar to that for East Perimeter Levee;



Levee Fill = $\frac{1}{2}(10+49)(6.5) = 192\ ft^2$

Compacted Fill Volume $\approx$ 192(7800)/27 = 55,500 yd^3

Req'd. Peat Excavation for 50% shrinkage = 55,500/0.5 = 111,000 yd^3

From p. 24, Req'd. Excavation for East Perimeter Levee = 60,000 yd^3
From above, Req'd. Excavation for Levee 2B-1 = 111,000 yd^3

Total Levee Fill = 171,000 yd^3

Note that total peat excavation $\approx$ 3960' × 1200' × 1.33'/27 $\cong$ 234,000 yd^3

∴ Need to enlarge levees to accommodate additional 63,000 yd^3;

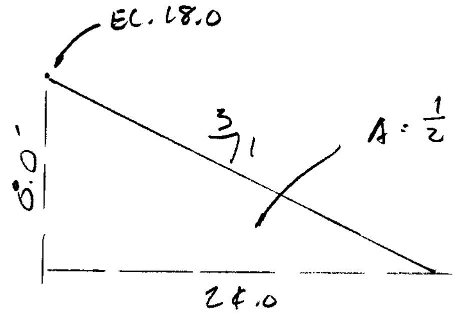
For total length of 3900' + 7800' = 11,700 ft, ave. increase of 63,000 (27)/11,700 = 145 sq. ft.

If ave. surface width of levee $\approx$ 51', would require approx. 2.5-3' increase in top elevation, for no other section changes.

Note that design elevation of top of exterior levee = 18.50 ft. NGVD.

Set maximum top of levee = elev. 18.0; maintain side slopes @ 3:1; compute needed top width for average end area ≈ !

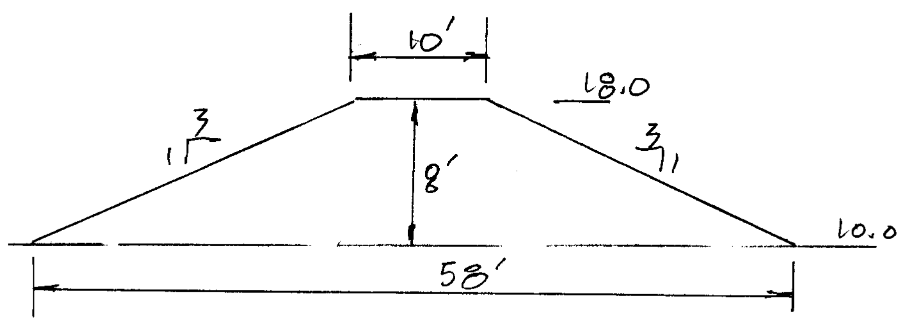
$$(234{,}000\ yd^3)(27)/11{,}700' = 540\ ft^2 \times 0.5\ \text{shrinkage} = \underline{270\ ft^2}$$



$A = \frac{1}{2}(8)(24) = 96\ ft^2$, ea. side

Then req'd. top width $= \dfrac{270\ ft^2 - 2(96)}{8} = 9.75'$, $\underline{\underline{\text{Use } 10'}}$

Check:

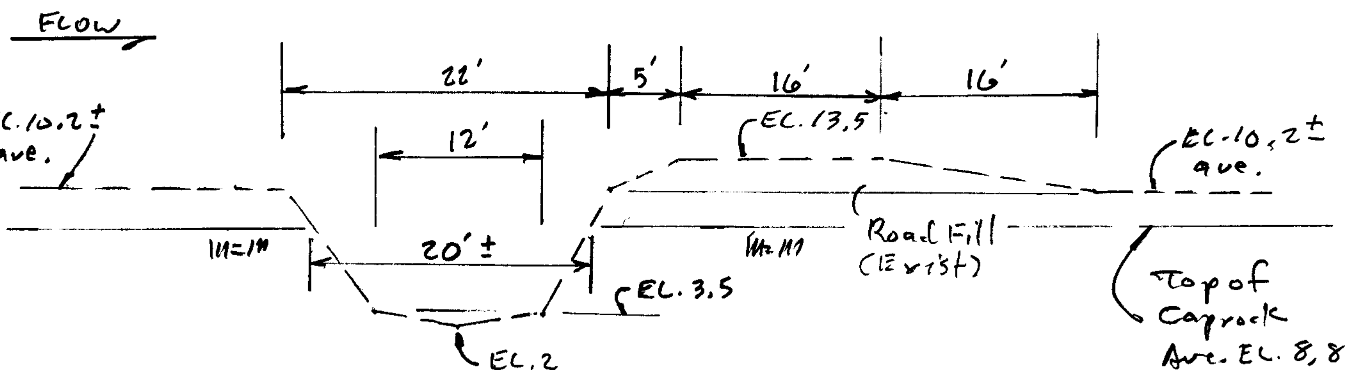


$A = \frac{1}{2}(10+58)(8) = 272\ ft^2$ ✓

For both East Perimeter Levee & Levee 2B-1, anticipate 10' top width at elev. 18.0, 3:1 side slopes

The PSTA demo. cell is traversed east-west by two agricultural drainage ditches that penetrate the caprock. Consider those as appropriate locations for emergent planting strips.

From Dwgs. C102 & C107 of Contract C-E307 construction drawings, for XS-II and XS-KK, characterize typical existing section as:



Total previous excavation $\approx \frac{1}{2}(12+22)(10.2-3.5) + \frac{1}{2}(12)(3.5-2) \approx 114+9 = 123\ ft^2$

Total previous fill $\approx \frac{1}{2}(16+37)(13.5-10.2) = 85\ ft^2$ above grade.

If peat consolidated 50% in "upstream" 21', tapering to zero at "downstream" end, add'l. fill $\cong 21\left(\frac{1.4}{2}\right) + \frac{1}{2}(16)\left(\frac{1.4}{2}\right) = 20\ ft^2$,
total fill $\approx 85+20 = 105\ ft^2$

Of previous excavation, approx. $20'(1.4') = 28\ ft^2$ was peat;
if 50% shrinkage, net fill would have been $123 - 0.5(28) = 109\ ft^2$

∴ Consider above approximation as reasonably representative of the east-west ditches & adjacent ag. road

Assume canal is filled to ave. elev. 8.8' (e.g., top of caprock);

required fill area $\cong \frac{1}{2}(20+12)(8.8-3.5) = 85^2$

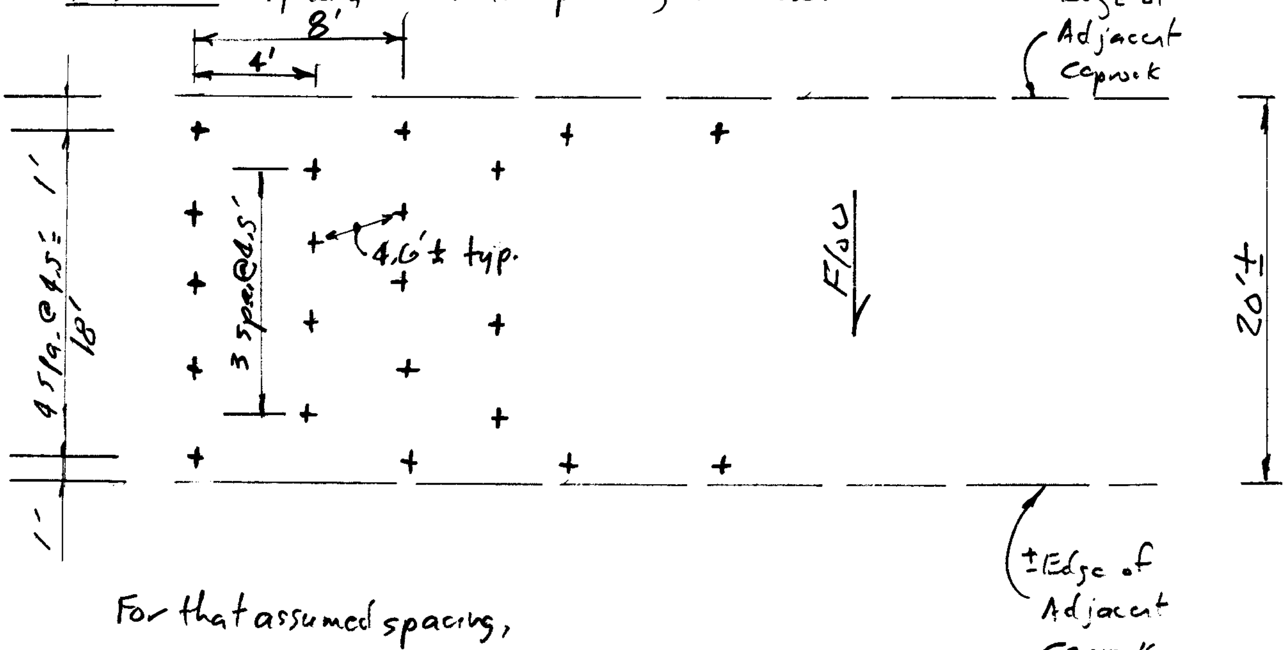
Note Approx. Balance

Then assume roadway fill above prevailing grade used to fill exist. drainage ditches, should consist primarily of material originally excavated from caprock & any underlying marls, etc., so should be relatively low in TP content.

Balance of roadway subgrade below prevailing grade probably a mix of mineral soils and peat - assume included in overall peat excavation & placed in East Perimeter Levee & new Interior Levee 2B_1.

Then approx. width of emergent planting strip (transverse to flow direction) ≈ 20'. Length of each planting strip ≈ 1,200'.

<u>Assumed</u> spacing for initial planting as follows:



For that assumed spacing,

ave. of 9 plants / 8' length = 1.125 plantings / ft.

For 2,400' total length, 2400 (1.125) = 2,700 plants ~ project that spacing can be reduced within reason relative to cost; spacing of ± 4.5' may not be adequate to ensure successful grow-in.

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Project 34273
Date 8/01/03
Made By G. Miller
Emergent Plantings Strips
Checked By
Preliminary ✓ Final

Assume spacing reduced by ±50% to 2' ctrs.

Anticipate 10 + 9 = 19 plants every 4' → 4.75 plants/ft length

Total No. of plantings = 4.75 (2400') = 11,400 plantings.

If spacing further reduced to ± 1' ctrs., no. of plants would increase to approx. 4 (11,400) = 45,600 plantings.

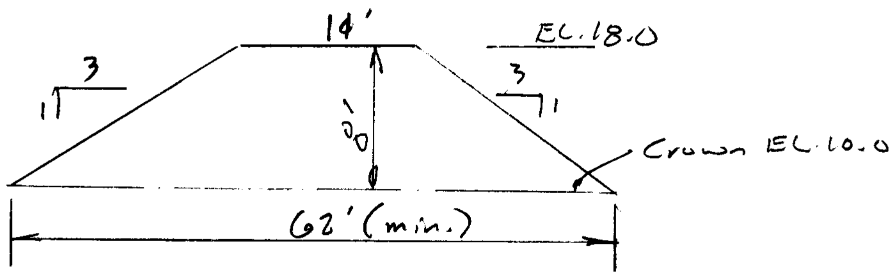
Initially project plantings @ approx. 2' ctrs., roughly 12,000 plants req'd.

Given: 1. Anticipated max. rate of discharge = 210 cfs (total of all structures)
2. At peak discharge, anticipated TW elev. = 13.2 ft. NGVD

Desire to minimize head loss, say 0.2' total allowed at peak rate.

Assume 2 structures, $Q_{ea.}$ = 105 cfs, corrugated aluminum pipe.
Set crown @ approx. el. 10.0 ft. NGVD.

Estimate min. length of structure:



Assume use of 84" ϕ culvert, $A = 38.48\ ft^2$, $P = 21.99'$
$R = A/P = 1.75$

$K = \frac{1.486}{n} A R^{2/3}$, use $n = 0.023 \rightarrow K = \frac{1.486}{.023}(38.48)(1.75)^{2/3} = 3{,}610$

Assume overall length = <u>80'</u>

$Q = KS^{1/2} \rightarrow 105 = 3610\,S^{1/2}$
$S = 0.00085$
$h_f = SL = 0.00085(80) = 0.07'$

$V = Q/A = 105/38.48 = 2.73$ fps, $V^2/2g = 0.12'$

Use entrance K = 0.5, exit K = 1.0 → minor losses = $1.5\,V^2/2g$

$1.5\,(0.12) = 0.18$ ft. ; then total loss = $0.07' + 0.18' = 0.25'$

$\boxed{\text{Use 2-84" } \phi \text{ CMPs}}$

ID		Task Name	Duration	Start	Finish	Predecessors	Resource Names
2		District Issue Scope of Work	5 days	Mon 8/25/03	Fri 8/29/03		
3		Receive Proposal	5 days	Mon 9/1/03	Fri 9/5/03	2	
4		Negotiate and Award	5 days	Mon 9/8/03	Fri 9/12/03	3	
5		Surveys	10 days	Wed 9/24/03	Tue 10/7/03	4FS+7 days	
6		60% Design	25 days	Tue 9/23/03	Mon 10/27/03	4,5FF+14 days	
7		District Review	10 days	Tue 10/28/03	Mon 11/10/03	6	
8		90% Design	20 days	Mon 11/3/03	Fri 11/28/03	7FF+14 days,6	
9		District Review	10 days	Mon 12/1/03	Fri 12/12/03	8	
10		100% Design	15 days	Mon 12/1/03	Fri 12/19/03	8,7FF+7 days	
12		District Issue RFB's	5 days	Mon 12/22/03	Fri 12/26/03	10	
13		Bid Response	20 days	Mon 12/29/03	Fri 1/23/04	12	
14		Bid Evaluation	5 days	Mon 1/26/04	Fri 1/30/04	13	
15		Award Contract(s)	5 days	Thu 2/5/04	Wed 2/11/04	14	
17		Issue NTP	10 days	Thu 2/12/04	Wed 2/25/04	15	
18		Contractor Mobilize	10 days	Thu 2/26/04	Wed 3/10/04	17	
19		Muck Removal	75 days	Thu 3/11/04	Wed 6/23/04	18	
20		Inflow Control Levees	60 days	Thu 2/26/04	Wed 5/19/04	17	
21		Compliance Submittals	75 days	Thu 2/12/04	Wed 5/26/04	15	
22		Culvert Structures	60 days	Thu 5/27/04	Wed 8/18/04	21	
23		Gates and Hoists, Deliver	60 days	Thu 5/27/04	Wed 8/18/04	21	
24		Gates and Hoists, Install	10 days	Thu 8/19/04	Wed 9/1/04	23,22	
25		Pump Station Structure	100 days	Thu 5/27/04	Wed 10/13/04	21	
26		Deliver Pumping Equipment	100 days	Thu 5/27/04	Wed 10/13/04	21	
27		Install Pumping Equipment	20 days	Thu 10/14/04	Wed 11/10/04	26,25	
28		Emergent Plantings	20 days	Thu 6/24/04	Wed 7/21/04	19	
29		Power Supply	40 days	Thu 9/30/04	Wed 11/24/04	24FF+10 days,27FF+1	
30		Testing	10 days	Thu 11/25/04	Wed 12/8/04	29	
31		Substantial Completion	10 days	Thu 12/9/04	Wed 12/22/04	30	
32		Final Completion	20 days	Thu 12/23/04	Wed 1/19/05	31	
34		Flood Cell Interior	40 days	Thu 8/19/04	Wed 10/13/04	19,22	
35		Vegetation Growin	250 days	Thu 10/14/04	Wed 9/28/05	34	
36		Full Operation	3 days	Thu 9/29/05	Mon 10/3/05	35,32	