NEW JERSEY
SHORE PROTECTION STUDY

US Army Corps
of Engineers
Philadelphia District

New Jersey Department
of Environmental
Protection

Hereford Inlet to Cape May Inlet

Volume - 2 Engineering and Technical Appendix

VOLUME 2: ENGINEERING TECHNICAL APPENDIX

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Section 1

General

This Engineering and Technical Appendix was prepared in accordance with ER 1110-2-1150, Engineering and Design for Civil Works Projects. Information in this appendix supplements data in the Feasibility report to satisfy criteria in 1110-2-1150

Section 2

Hydrology and Hydraulics/Coastal Processes

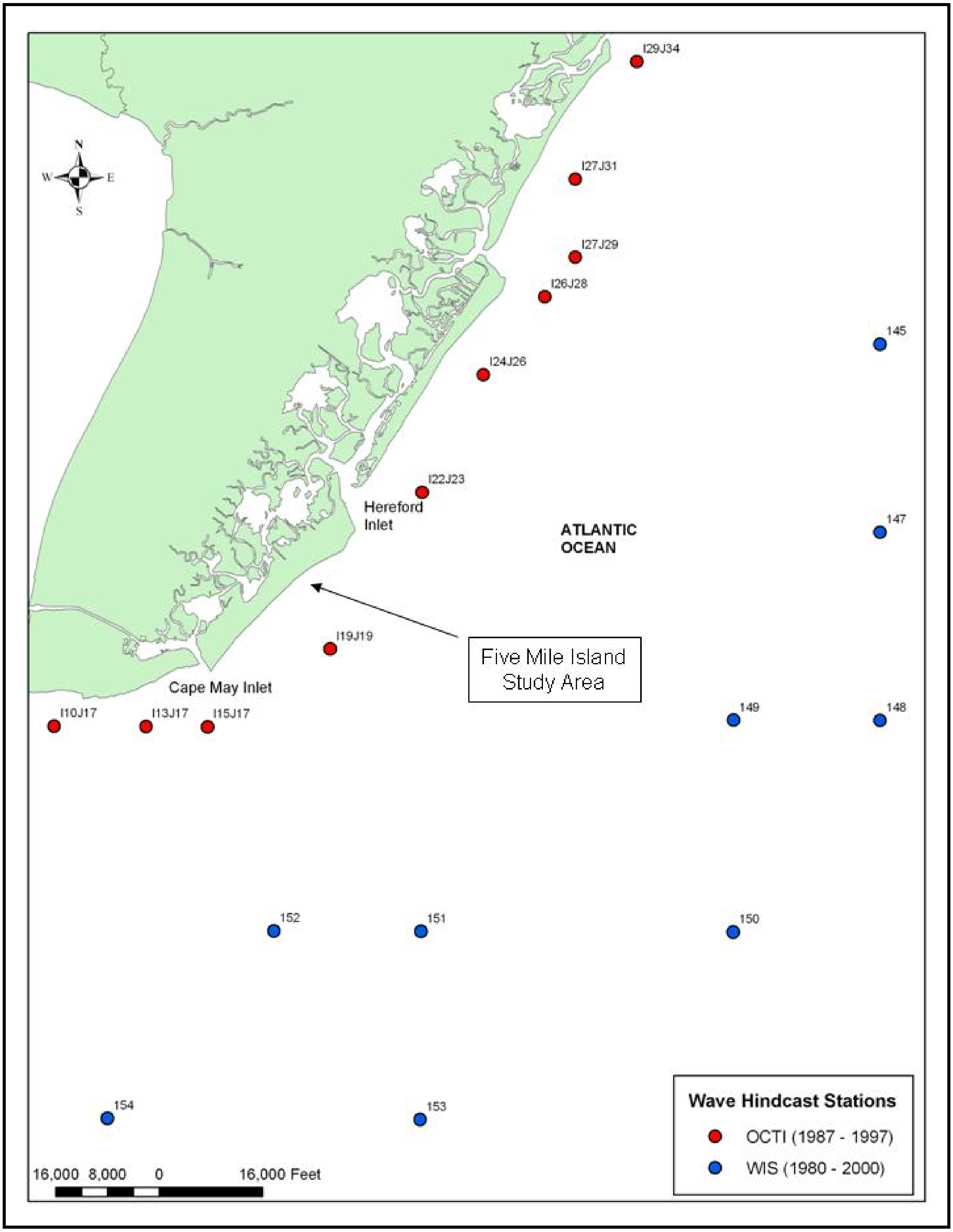


Figure 1. Wave Hindcast Stations

Table 1. Percent Occurrences of Wave Height by Month for WIS Station 147 (1980 – 2000)

Hmo (meters)	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	CASES	PCT
0.00 - 0.49	0.81	0.93	1.3	1.74	1.61	2.11	2.55	1.91	1.23	1.17	0.91	1	30259	**17.3**
0.50 - 0.99	2.97	2.62	2.98	3.21	4.21	4.39	4.53	4.72	3.49	3.55	2.96	2.92	74569	**42.5**
1.00 - 1.49	2.64	2.42	2.39	2.02	1.82	1.29	1.1	1.27	2.34	2.38	2.42	2.61	43266	**24.7**
1.50 - 1.99	1.16	1.03	1.05	0.77	0.52	0.33	0.23	0.34	0.68	0.83	1.12	1.2	16207	**9.2**
2.00 - 2.49	0.47	0.42	0.41	0.33	0.21	0.08	0.06	0.1	0.24	0.32	0.44	0.46	6204	**3.5**
2.50 - 2.99	0.22	0.19	0.19	0.1	0.09	0.01	0.01	0.07	0.14	0.11	0.21	0.18	2679	**1.5**
3.00 - 3.49	0.12	0.08	0.12	0.03	0.03	0	0.01	0.04	0.05	0.06	0.08	0.05	1185	**0.7**
3.50 - 3.99	0.04	0.02	0.02	0.02	0.01	.	0	0.02	0.04	0.03	0.05	0.04	541	**0.3**
4.00 - 4.49	0.02	0.01	0.01	.	0	.	0	0.02	0.01	0.01	0.03	0.01	224	**0.1**
4.50 - 4.99	0.01	0.01	0.01	.	.	.	0	0	0	0.02	0	0.01	117	**0.1**
5.00 - GREATER	0.01	0.01	0	.	.	.	.	.	0	0	.	0.01	43	**0**

Table 2. Percent Occurrences of Peak Period by Month for WIS Station 147 (1980 – 2000)

Tp(sec)	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	CASES	PCT
3.0 - 3.9	2.24	1.86	1.97	1.79	1.39	1.74	1.88	1.7	1.6	2.28	2.31	2.37	40554	**23.1**
4.0 - 4.9	2.01	1.65	1.61	1.35	1.41	1.64	1.8	1.62	1.48	1.76	1.73	2.11	35332	**20.2**
5.0 - 5.9	0.68	0.66	0.86	0.92	1.31	1.64	1.81	1.87	1.14	0.88	0.71	0.64	22993	**13.1**
6.0 - 6.9	0.62	0.62	0.67	0.86	1.63	1.64	1.57	1.66	1.04	0.76	0.73	0.53	21603	**12.3**
7.0 - 7.9	0.69	0.66	0.7	0.96	1.46	1	0.96	0.75	0.85	0.76	0.78	0.56	17783	**10.1**
8.0 - 8.9	0.61	0.7	0.71	1	0.74	0.36	0.22	0.22	0.45	0.73	0.65	0.62	12303	**7**
9.0 - 9.9	0.61	0.63	0.7	0.6	0.29	0.12	0.14	0.12	0.27	0.46	0.46	0.54	8662	**4.9**
10.0 - 10.9	0.46	0.43	0.52	0.35	0.1	0.05	0.03	0.09	0.19	0.28	0.36	0.38	5704	**3.3**
11.0 - 13.9	0.53	0.47	0.68	0.37	0.15	0.01	0.07	0.31	0.85	0.45	0.43	0.66	8737	**5**
14.0 - LONGER	0.03	0.06	0.07	0	0.01	.	0.01	0.14	0.34	0.12	0.06	0.08	1623	**0.9**

Table 3. Percent Occurrences of Mean Direction by Month for WIS Station 147 (1980 – 2000)

Direction Band (deg)	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	CASES	PCT
348.75 - 11.24	0.42	0.4	0.36	0.17	0.11	0.09	0.06	0.1	0.17	0.27	0.34	0.5	5260	3
11.25 - 33.74	0.46	0.4	0.39	0.25	0.18	0.13	0.08	0.16	0.27	0.46	0.39	0.46	6379	3.6
33.75 - 56.24	0.66	0.64	0.66	0.46	0.43	0.26	0.11	0.43	0.44	0.75	0.51	0.59	10434	6
56.25 - 78.74	0.55	0.67	0.7	0.68	0.79	0.51	0.27	0.71	0.85	0.8	0.57	0.55	13389	7.6
78.75 - 101.24	0.5	0.63	0.77	0.81	1.08	0.67	0.54	0.88	1.17	0.96	0.69	0.61	16321	9.3
101.25 - 123.74	0.55	0.48	0.54	0.84	0.94	0.88	0.73	1.09	1.16	0.92	0.63	0.52	16272	9.3
123.75 - 146.24	0.54	0.61	0.87	0.91	1.01	1.15	1.23	1.42	1.26	0.74	0.55	0.45	18810	10.7
146.25 - 168.74	0.77	0.61	0.91	1.28	1.52	1.62	1.73	1.49	1	0.74	0.67	0.5	22536	12.9
168.75 - 191.24	0.76	0.83	0.99	1.37	1.62	2.01	2.63	1.41	0.83	0.79	0.79	0.77	25959	14.8
191.25 - 213.74	0.69	0.54	0.53	0.43	0.33	0.51	0.75	0.47	0.43	0.54	0.67	0.74	11629	6.6
213.75 - 236.24	0.4	0.24	0.22	0.19	0.13	0.14	0.14	0.11	0.18	0.28	0.48	0.51	5268	3
236.25 - 258.74	0.4	0.26	0.2	0.16	0.1	0.07	0.07	0.06	0.1	0.23	0.37	0.45	4314	2.5
258.75 - 281.24	0.49	0.31	0.31	0.2	0.06	0.05	0.06	0.04	0.06	0.27	0.41	0.5	4842	2.8
281.25 - 303.74	0.48	0.34	0.34	0.16	0.06	0.04	0.03	0.04	0.08	0.23	0.41	0.51	4758	2.7
303.75 - 326.24	0.42	0.36	0.34	0.17	0.05	0.05	0.03	0.04	0.09	0.26	0.39	0.4	4541	2.6
326.25 - 348.74	0.38	0.41	0.36	0.14	0.09	0.03	0.04	0.04	0.12	0.25	0.33	0.42	4582	2.6

Table 4. Summary of Mean Hmo (feet) by Month and Year for WIS Station 147 (1980 – 2000)

YEAR	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	MEAN
1980	4.23	3.12	4.00	3.31	1.94	2.23	2.33	2.26	2.59	3.31	3.51	3.81	**3.05**
1981	2.92	4.23	3.28	2.99	3.05	2.10	2.46	2.59	3.08	3.51	3.97	3.81	**3.15**
1982	4.10	4.20	2.95	3.15	1.84	2.40	1.84	2.03	2.69	3.18	3.71	3.28	**2.95**
1983	4.07	4.17	4.49	3.25	2.62	2.33	1.64	2.23	2.99	3.87	3.45	4.36	**3.28**
1984	3.54	4.40	4.40	2.92	3.81	2.76	2.69	2.03	3.41	3.90	3.77	3.25	**3.41**
1985	3.61	3.45	2.95	2.92	3.02	2.17	2.43	2.43	2.69	3.22	4.46	3.67	**3.08**
1986	3.77	3.18	3.02	2.89	2.46	3.02	1.87	2.76	2.66	2.85	3.41	3.90	**2.99**
1987	3.74	2.79	3.41	4.49	3.41	2.30	1.97	2.36	2.72	3.48	4.20	3.22	**3.18**
1988	3.28	3.87	3.08	3.31	2.69	2.56	2.20	2.26	2.43	3.31	3.48	3.41	**2.99**
1989	3.22	3.67	4.20	2.69	2.95	2.20	2.20	2.72	3.81	2.99	3.81	3.71	**3.18**
1990	2.89	3.38	3.15	2.92	2.72	2.26	2.13	2.43	3.28	4.10	3.02	4.04	**3.02**
1991	4.20	3.35	3.77	3.41	2.36	2.26	2.53	2.92	3.51	3.64	3.94	3.67	**3.28**
1992	4.23	3.67	3.87	2.89	3.77	2.43	2.53	2.59	4.20	3.58	3.71	4.59	**3.51**
1993	4.23	4.17	3.81	3.77	2.56	2.17	2.20	2.69	3.05	3.35	3.90	3.94	**3.31**
1994	4.17	3.31	3.51	2.92	3.08	2.79	2.36	2.33	2.79	2.56	4.82	4.00	**3.22**
1995	4.49	3.74	2.92	2.76	2.69	2.82	2.62	4.69	4.92	3.67	4.27	3.74	**3.61**
1996	4.72	4.10	4.04	4.07	3.25	2.66	3.18	2.62	4.20	4.07	3.51	4.33	**3.74**
1997	4.17	4.13	3.84	3.08	3.22	2.76	2.72	2.30	2.99	3.02	4.13	3.28	**3.28**
1998	4.40	5.09	4.10	2.92	3.38	2.36	2.00	3.41	3.12	3.35	3.08	3.22	**3.35**
1999	4.43	3.64	4.00	2.49	3.54	3.31	2.43	3.45	5.18	3.31	4.23	3.84	**3.64**
MEAN	**3.90**	**3.77**	**3.64**	**3.15**	**2.92**	**2.49**	**2.33**	**2.66**	**3.31**	**3.41**	**3.81**	**3.74**	

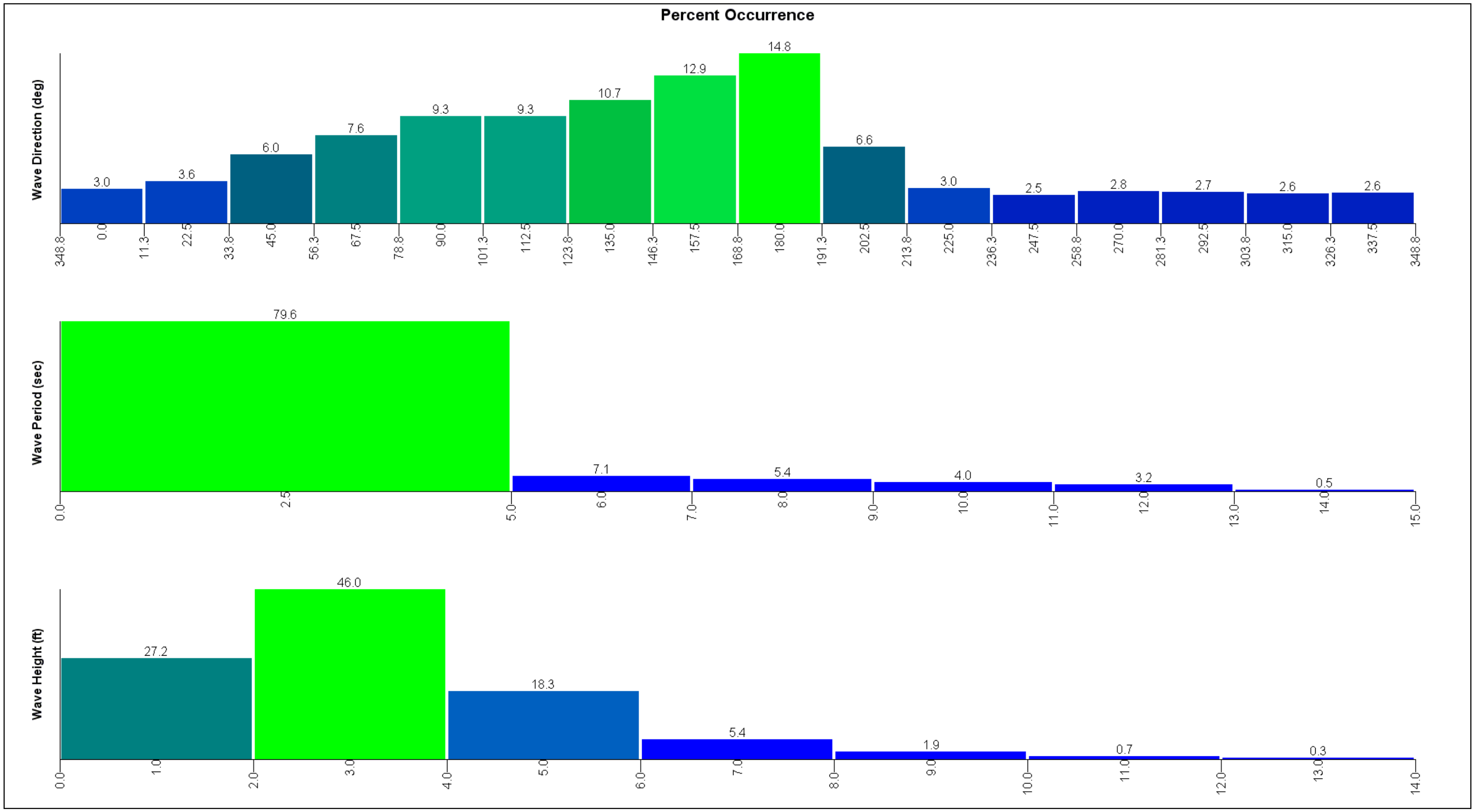
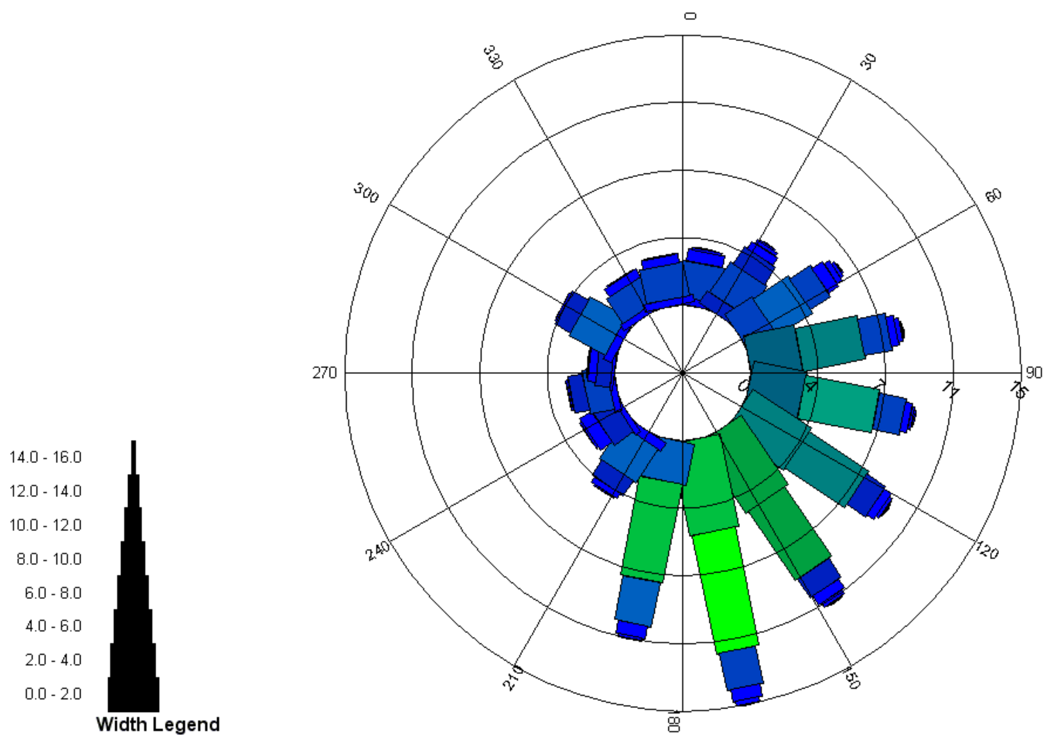
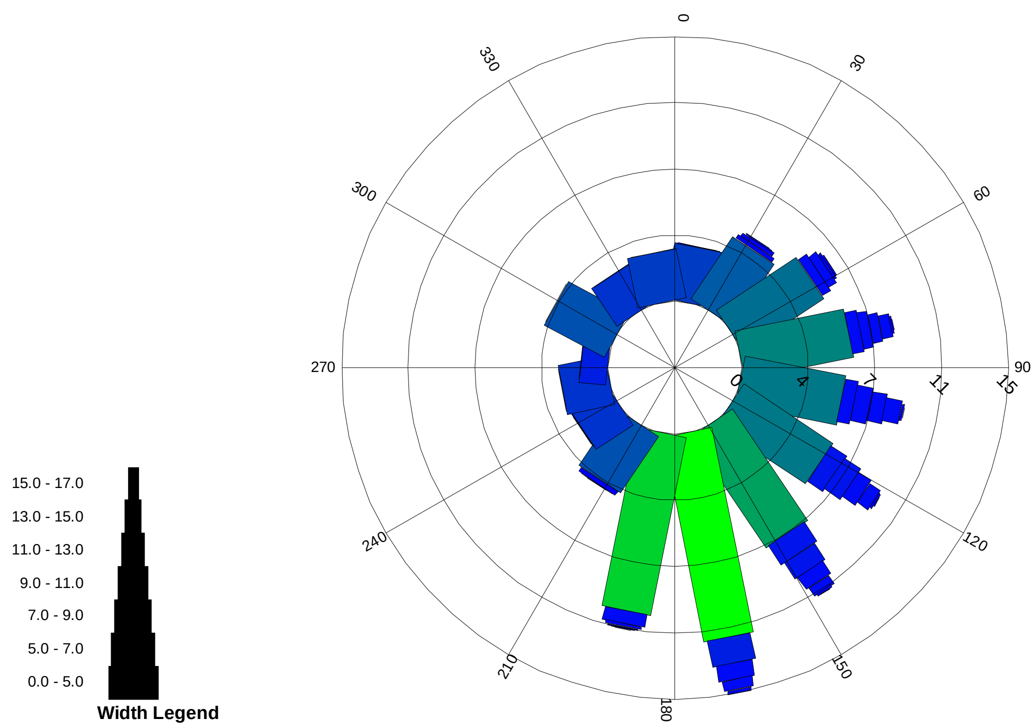


Figure 2. WIS Station 147 Percent Occurrence Wave Direction, Period, and Height Histograms (1980-2000)



(a) Percent Occurrence of Wave Direction vs. Wave Height (feet)



(b) Percent Occurrence of Wave Direction vs. Wave Period (sec)

Figure 3. Wave Roses of WIS Station 147 (1980-2000)

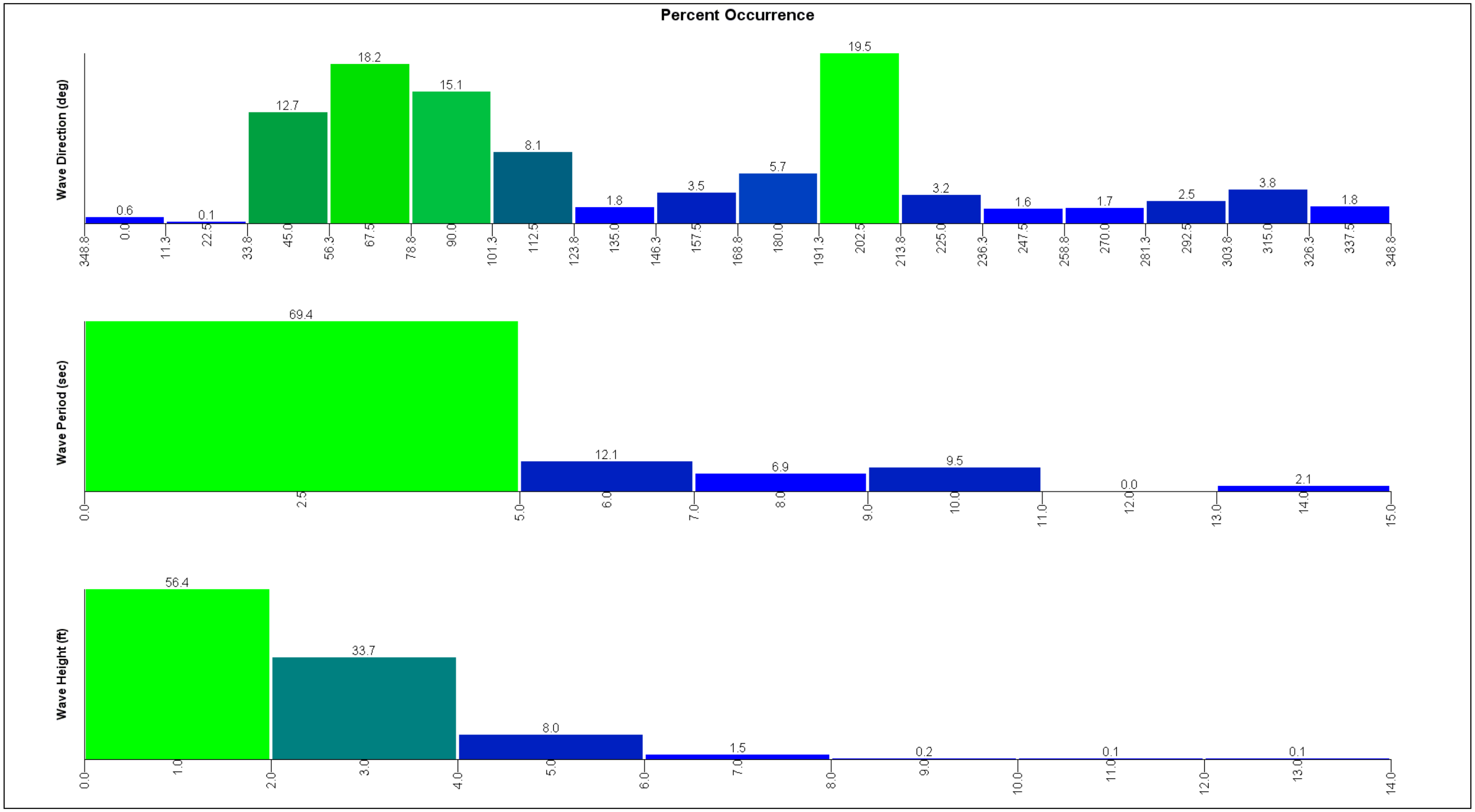
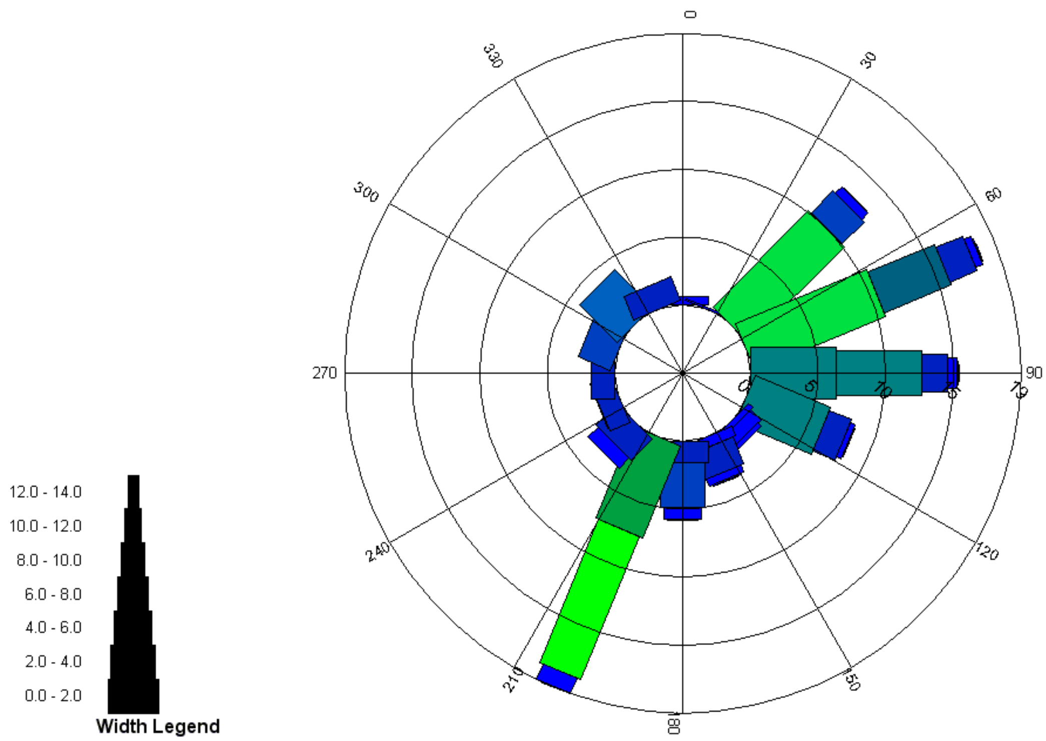
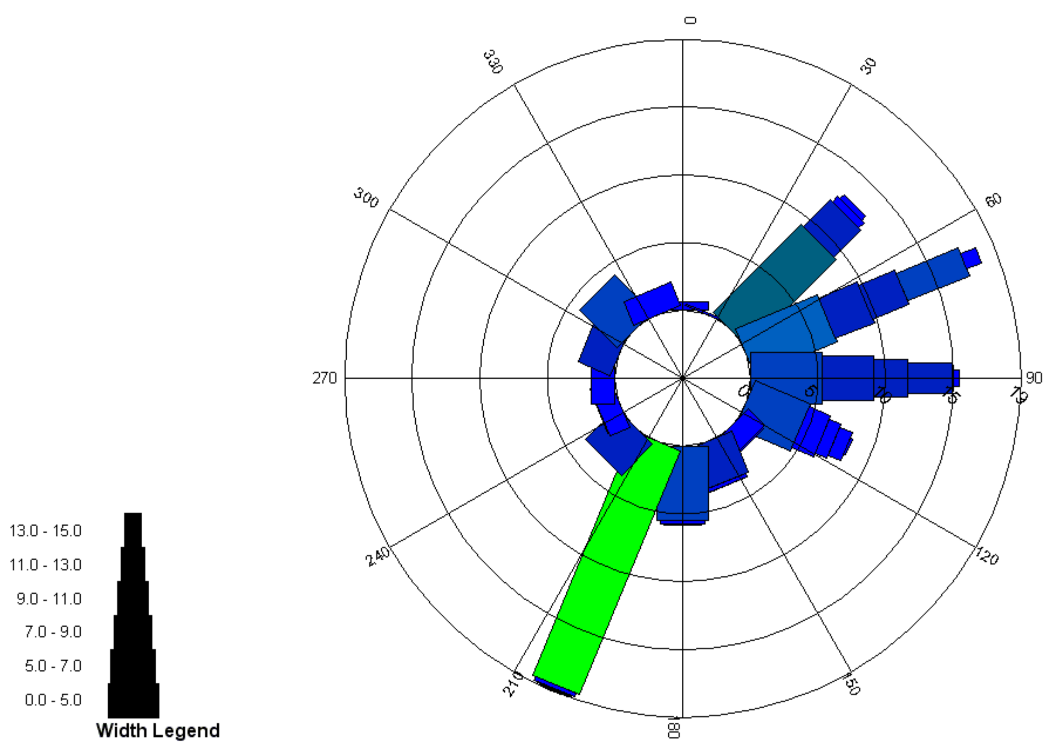


Figure 4. OCTI Station I19J19 Percent Occurrence Wave Direction, Period, and Height Histograms (1987-1997)



(a) Percent Occurrence of Wave Direction vs. Wave Height (feet)



(b) Percent Occurrence of Wave Direction vs. Wave Period (sec)

Figure 5. Wave Roses of OCTI Station I19J19 (1987-1997)

Table 5. Percent Occurrences of Wind Speed by Month for WIS Station 147 (1980 – 2000)

WS(m/sec)	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	CASES	PCT
0.00 - 1.99	0.07	0.16	0.25	0.41	0.69	0.72	0.62	0.55	0.29	0.24	0.11	0.09	7342	**4.20**
2.00 - 3.99	0.84	1.03	1.44	1.95	2.60	2.84	3.16	2.93	2.04	1.48	0.89	0.88	38726	**22.10**
4.00 - 5.99	1.62	1.61	1.98	2.25	2.60	2.68	2.93	2.86	2.59	2.16	1.69	1.76	46855	**26.70**
6.00 - 7.99	1.93	1.65	1.68	1.79	1.50	1.31	1.31	1.36	1.76	1.96	2.00	1.82	35176	**20.10**
8.00 - 9.99	1.74	1.45	1.39	1.00	0.65	0.51	0.36	0.57	0.96	1.51	1.57	1.57	23277	**13.30**
10.00 - 11.99	1.14	0.88	0.95	0.50	0.32	0.14	0.09	0.12	0.40	0.70	1.04	1.18	13079	**7.50**
12.00 - 13.99	0.71	0.56	0.49	0.22	0.11	0.02	0.01	0.06	0.14	0.28	0.60	0.74	6914	**3.90**
14.00 - 15.99	0.29	0.24	0.20	0.07	0.01	.	0.00	0.03	0.03	0.10	0.22	0.30	2623	**1.50**
16.00 - 17.99	0.09	0.11	0.08	0.02	0.00	.	0.00	0.00	0.01	0.03	0.08	0.12	950	**0.50**
18.00 - 19.99	0.04	0.03	0.02	0.01	.	.	0.00	0.00	0.00	0.01	0.02	0.03	290	**0.20**
20.00 - GREATER	0.01	0.01	0.01	.	.	.	0.00	0.00	0.00	.	0.00	0.00	62	**0.00**

Table 6. Percent Occurrences of Wind Direction by Month for WIS Station 147 (1980 – 2000)

Direction Band (deg)	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	CASES	PCT
348.75 - 11.24	0.62	0.61	0.72	0.47	0.40	0.40	0.34	0.50	0.61	0.66	0.54	0.71	11519	**6.60**
11.25 - 33.74	0.49	0.46	0.44	0.38	0.54	0.30	0.26	0.56	0.53	0.50	0.33	0.39	9074	**5.20**
33.75 - 56.24	0.45	0.50	0.47	0.51	0.57	0.48	0.33	0.61	0.82	0.67	0.40	0.38	10842	**6.20**
56.25 - 78.74	0.25	0.23	0.35	0.37	0.36	0.33	0.23	0.45	0.54	0.39	0.32	0.18	7027	**4.00**
78.75 - 101.24	0.23	0.24	0.25	0.31	0.36	0.36	0.29	0.43	0.47	0.40	0.24	0.24	6695	**3.80**
101.25 - 123.74	0.14	0.17	0.20	0.32	0.32	0.25	0.27	0.29	0.29	0.28	0.18	0.16	5039	**2.90**
123.75 - 146.24	0.21	0.22	0.34	0.41	0.38	0.37	0.34	0.36	0.34	0.31	0.26	0.16	6500	**3.70**
146.25 - 168.74	0.23	0.21	0.36	0.47	0.50	0.53	0.45	0.47	0.34	0.33	0.26	0.17	7602	**4.30**
168.75 - 191.24	0.50	0.59	0.76	0.99	1.28	1.25	1.23	1.07	0.86	0.70	0.67	0.41	18072	**10.30**
191.25 - 213.74	0.55	0.51	0.67	0.71	0.93	1.29	1.45	1.11	0.74	0.72	0.67	0.63	17471	**10.00**
213.75 - 236.24	0.56	0.42	0.45	0.50	0.62	0.90	1.24	0.88	0.67	0.63	0.61	0.76	14437	**8.20**
236.25 - 258.74	0.35	0.31	0.28	0.37	0.41	0.43	0.56	0.45	0.41	0.34	0.45	0.45	8437	**4.80**
258.75 - 281.24	0.75	0.58	0.59	0.59	0.49	0.37	0.48	0.39	0.32	0.49	0.60	0.76	11261	**6.40**
281.25 - 303.74	1.12	0.84	0.78	0.64	0.43	0.33	0.33	0.29	0.33	0.65	0.95	1.02	13482	**7.70**
303.75 - 326.24	1.29	1.13	1.09	0.70	0.51	0.38	0.38	0.35	0.51	0.79	1.12	1.23	16625	**9.50**
326.25 - 348.74	0.73	0.71	0.75	0.47	0.41	0.24	0.27	0.29	0.44	0.62	0.62	0.84	11211	**6.40**

Table 7. Beach Profile Locations in Study Area

Profile Line #	Municipality	Nearby Location Reference	Station (wrt Baseline)	Profile Origin (on Baseline) Easting NJSP NAD 83 (ft)	Northing NJSP NAD 83 (ft)	Notes
WW 1	N. Wildwood	2nd Ave	0+20	410,609.74	61,439.60	loc of LRP H-11 prof
WW 1A	N. Wildwood	5th Ave	8+32	410,050.71	60,850.02	
WW 1B	N. Wildwood	8th Ave	16+79	409,389.08	60,331.73	
WW 2	N. Wildwood	10th Ave	21+68	409,045.92	59,983.76	loc of LRP NP-114 prof
WW 2A	N. Wildwood	12th Ave	27+36	408,646.95	59,579.19	
WW 2B	N. Wildwood	15th Ave	35+10	408,103.39	59,028.00	
CM 111	N. Wildwood	15th Ave	35+92	407,991.49	59,027.56	
WW 3	N. Wildwood	18th Ave	43+40	407,520.72	58,437.17	
WW 3A	N. Wildwood	23rd Ave	57+31	406,388.97	57,628.34	
WW 4	N. Wildwood	26th Ave	65+82	405,633.22	57,246.81	loc of LRP NP-115 prof
WW 5	Wildwood	Pine Ave	79+40	404,461.33	56,570.57	
WW 6	Wildwood	Lincoln Ave	92+41	403,456.58	55,752.41	
WW 7	Wildwood	Baker Ave	107+15	402,385.58	54,739.20	loc of LRP NP-116 prof
WW 8	Wildwood	Taylor Ave	121+30	401,215.08	53,946.88	
WW 9	Wildwood	Cresse Ave	136+84	400,077.35	52,887.38	
CM 110	Wildwood	Cresse Ave	136+87	400,242.56	52,727.56	
WW 10	Wildwood Crest	Crocus Rd	149+31	399,165.24	52,037.99	loc of LRP NP-117 prof
WW 11	Wildwood Crest	Fern Rd	169+88	397,659.59	50,635.86	
WW 12	Wildwood Crest	Stanton Rd	189+96	396,238.94	49,218.95	loc of LRP NP-118 prof
WW 13	Wildwood Crest	Toledo Ave	209+25	394,921.00	47,810.09	
WW 14	Wildwood Crest	Trenton Ave	228+42	393,571.43	46,450.07	
WW 15	Lower Township	Seapoint Blvd	245+68	392,307.33	45,275.17	loc of LRP NP-119 prof
CM 109	Lower Township	Raleigh Ave	249+97	392,197.68	44,797.79	
WW 16	Lower Township	Coast Guard Base	258+70	391,374.95	44,367.10	
WW 17	Lower Township	Coast Guard Base	273+57	390,308.20	43,331.09	

WW 18	Lower Township	Coast Guard Base	286+72	389,322.73	42,460.97	loc of LRP NP-120 prof

Table 7 (Continued). Beach Profile Locations in Study Area

Profile Line #	Municipality	Nearby Location Reference	Station (wrt Baseline)	Profile Origin (on Baseline)		Notes
				Easting NJSP NAD 83 (ft)	Northing NJSP NAD 83 (ft)	
CM 208	Lower Township	Coast Guard Base	287+09	389,950.36	41,936.55	
WW 19	Lower Township	Coast Guard Base	301+63	388,406.87	41,300.91	
WW 20	Lower Township	CM Inlet North Jetty	314+04	387,741.09	40,255.00	loc of LRP CS-1 prof

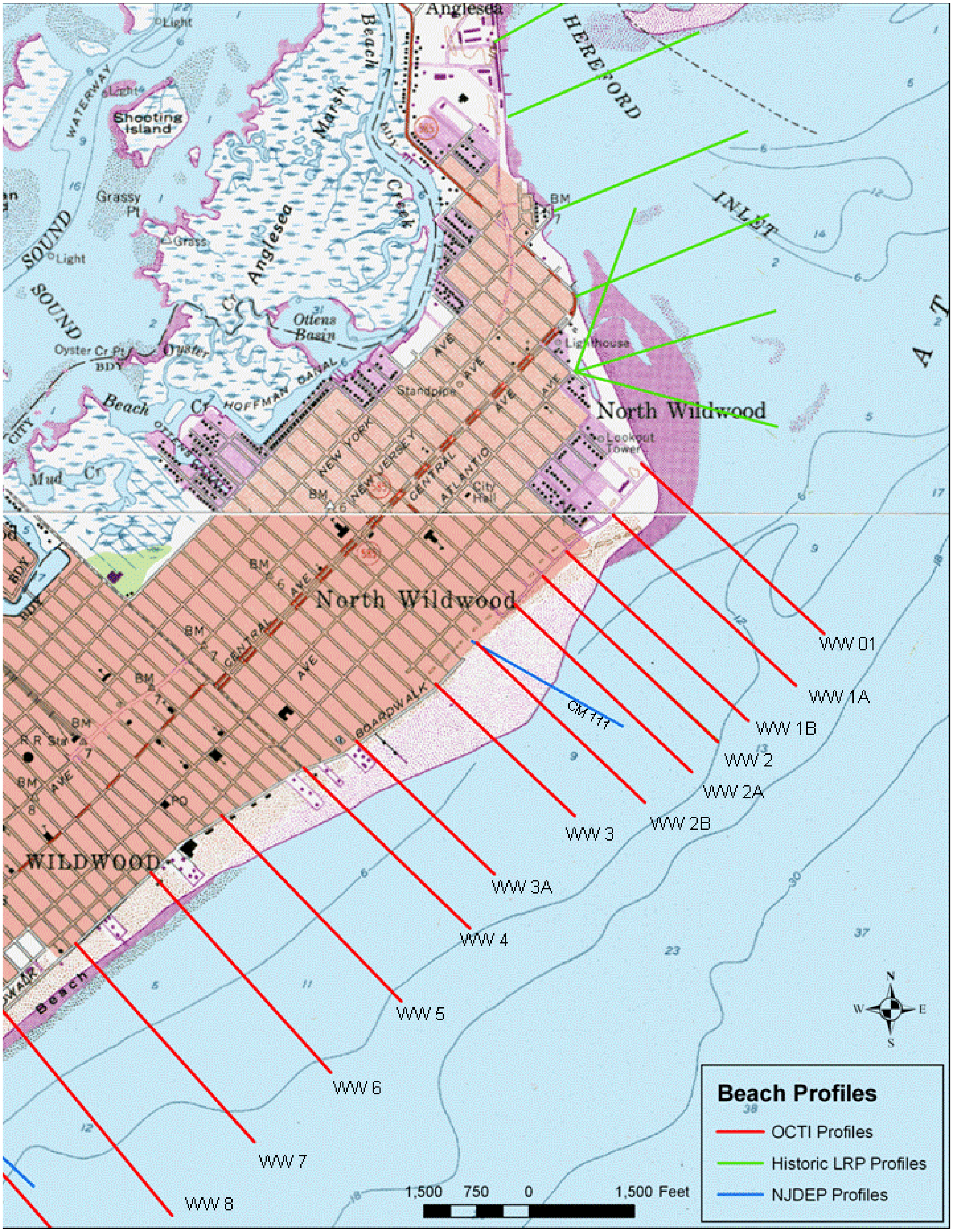


Figure 6. Beach Profile Locations Hereford Inlet to Wildwood

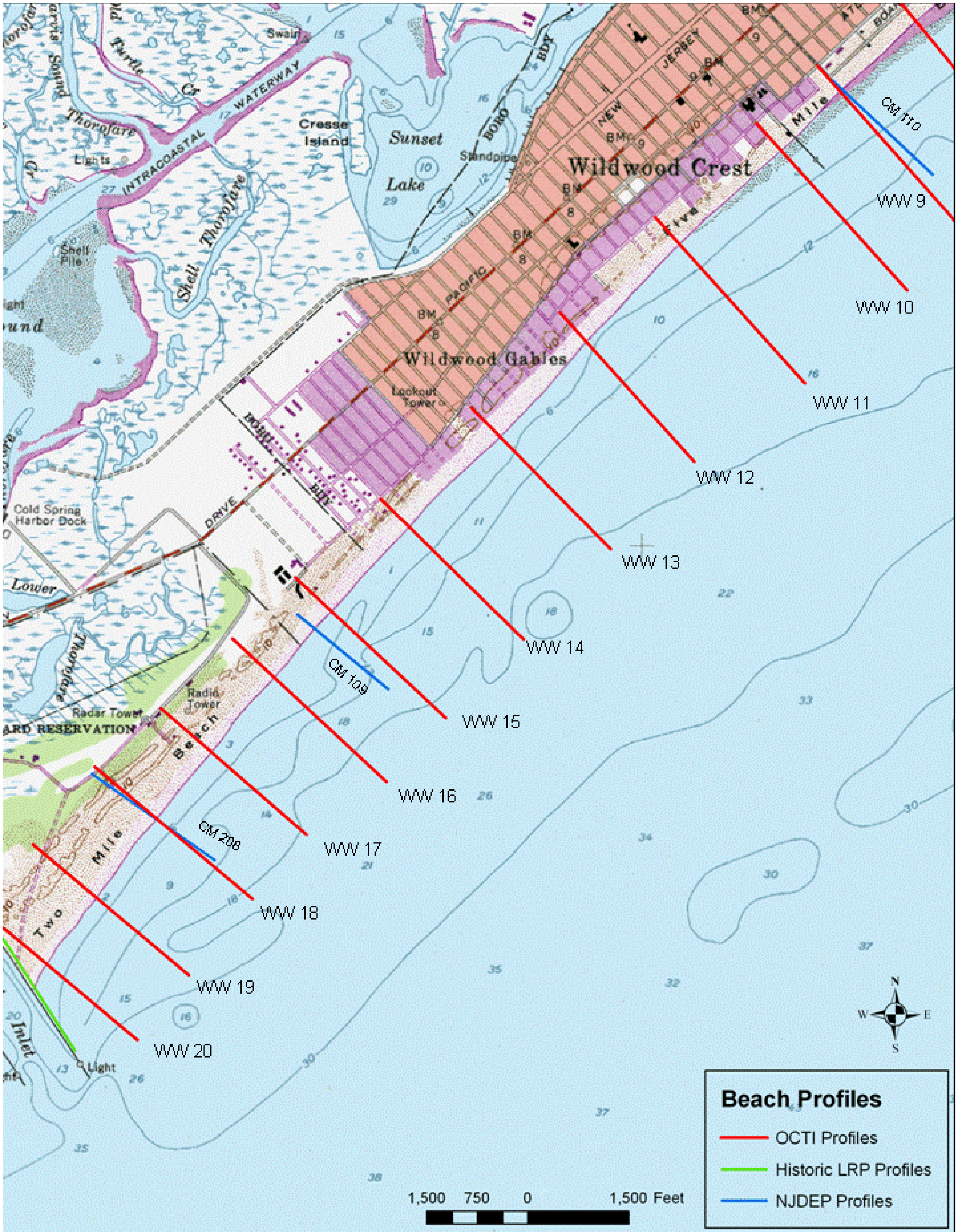


Figure 7. Beach Profile Locations from Wildwood to Cape May Inlet

Table 8. Elevation Parameters of the Beach Profiles Collected By OCTI in 2001 and 2003

		Dune Crest Elev. (ft. NAVD 88)			Avg. Berm Elev. (ft. NAVD88)		
Profile	Town	Sept. 2001	Oct. 2003	Diff.	Sept. 2001	Oct. 2003	Diff.
WW01	North Wildwood	10.3	10.2	-0.1	4.4	5.8	1.4
WW1A	North Wildwood		10.3			5.3	
WW1B	North Wildwood		10.4			5.4	
WW02	North Wildwood	9.8	10.4	0.6	4.2	5.0	0.8
WW2A	North Wildwood		10.4			5.6	
WW2B	North Wildwood		none			5.5	
WW03	North Wildwood	10.8	9.5	-1.3	4.7	5.4	0.7
WW3A	North Wildwood		13.5			6.1	
WW04	North Wildwood	none	12.0		5.5	5.8	0.3
WW05	Wildwood	none	none		4.5	4.5	0.0
WW06	Wildwood	none	none		4.8	5.4	0.6
WW07	Wildwood	none	none		4.4	4.6	0.2
WW08	Wildwood	none	none		4.4	4.6	0.2
WW09	Wildwood	12.6	12.5	-0.1	4.8	4.8	0.0
WW10	Wildwood Crest	10.4	10.6	0.2	4.6	4.6	0.0
WW11	Wildwood Crest	14.2	16.0	1.8	4.5	4.8	0.3
WW12	Wildwood Crest	none	none		5.1	5.4	0.3
WW13	Wildwood Crest	none	none		5.0	5.2	0.2
WW14	Wildwood Crest	none	none		5.4	5.8	0.4
WW15	Lower Township	11.6	11.6	0.0	5.9	5.9	0.0
WW16	Lower Township	14.1	14.4	0.3	4.9	5.1	0.2
WW17	Lower Township	14.7	15.0	0.3	5.5	6.1	0.6
WW18	Lower Township	21.4	22.3	0.9	5.3	6.1	0.8
WW19	Lower Township	18.9	18.6	-0.3	5.6	5.9	0.3
WW20	Lower Township	14.4	15.7	1.3	4.9	6.2	1.3

Table 9. Contour Locations of the Beach Profiles Collected By OCTI in 2001 and 2003

Profile	Town	0.0 ft. ft. NAVD 88 Location			-10 ft. NAVD 88 Location		
		Sept. 2001	Oct. 2003	Diff.	Sept. 2001	Oct. 2003	Diff.
WW01	North Wildwood	298.0	244.0	-54.0	1221.0		
WW1A	North Wildwood		398.0				
WW1B	North Wildwood		411.0			1212.0	
WW02	North Wildwood	495.0	391.0	-104.0	929.0	1082.0	153.0
WW2A	North Wildwood		403.0			976.0	
WW2B	North Wildwood		597.0			1075.0	
WW03	North Wildwood	908.0	653.0	-255.0	1155.0	1334.0	179.0
WW3A	North Wildwood		1129.0			1715.0	
WW04	North Wildwood	1455.0	1379.0	-76.0	1914.0	1919.0	5.0
WW05	Wildwood	1759.0	1641.0	-118.0	2060.0	2229.0	169.0
WW06	Wildwood	1736.0	1728.0	-8.0	2314.0	2324.0	10.0
WW07	Wildwood	1563.0	1581.0	18.0	2160.0	2218.0	58.0
WW08	Wildwood	1578.0	1608.0	30.0	2200.0	2307.0	107.0
WW09	Wildwood	1382.0	1386.0	4.0	1996.0	2156.0	160.0
WW10	Wildwood Crest	1260.0	1300.0	40.0	1888.0	2069.0	181.0
WW11	Wildwood Crest	1138.0	1128.0	-10.0	1748.0	1952.0	204.0
WW12	Wildwood Crest	1062.0	1034.0	-28.0	1699.0	1920.0	221.0
WW13	Wildwood Crest	946.0	946.0	0.0	1569.0	1841.0	272.0
WW14	Wildwood Crest	943.0	919.0	-24.0	1552.0	1815.0	263.0
WW15	Lower Township	1045.0	1026.0	-19.0	1602.0	1886.0	284.0
WW16	Lower Township	1099.0	1062.0	-37.0	1727.0	1968.0	241.0
WW17	Lower Township	1210.0	1176.0	-34.0	1752.0	1979.0	227.0
WW18	Lower Township	1375.0	1365.0	-10.0	1842.0	1934.0	92.0
WW19	Lower Township	1363.0	1333.0	-30.0	1863.0	1915.0	52.0
WW20	Lower Township	1271.0	1232.0	-39.0	1857.0	1759.0	-98.0

Table 10. Shoreline Change Grid Compartments

Analysis Segment	Compartment Number	Compartment Start	Compartment End	Compartment Length (ft)
WW1	1	0	800	800
	2	800	1500	700
	3	1500	2250	750
	4	2250	3150	900
	5	3150	3850	700
	6	3850	4600	750
	7	4600	5400	800
	8	5400	6000	600
	9	6000	6840	840
WW2	1	0	1000	1000
	2	1000	1900	900
	3	1900	2500	600
	4	2500	3200	700
	5	3200	4200	1000
	6	4200	5200	1000
	7	5200	6200	1000
	8	6200	6830	630
WW3	1	0	1000	1000
	2	1000	2000	1000
	3	2000	3000	1000
	4	3000	4000	1000
	5	4000	5000	1000
	6	5000	6000	1000
	7	6000	7000	1000
	8	7000	7700	700
	9	7700	8700	1000
	10	8700	9630	930
WW4	1	0	800	800
	2	800	1600	800
	3	1600	2600	1000
	4	2600	3600	1000
	5	3600	4600	1000
	6	4600	5600	1000
	7	5600	6600	1000
	8	6600	7350	750
	9	7350	7850	500
	10	7850	8350	500
TOTALS	**37**			**31,650**

Table 11. Mean Shoreline Positions (feet) by Compartment

Segment	Comp	1899	1932	1943	1971	1977	1986	1994	1998	2003	Avg	Std Dev
1	1.00	1711.50	1859.10	2056.70		2178.70	2615.40		2321.30	2166.00	2129.81	325.04
	2.00	1845.90	1938.80	1938.70		2749.30	3294.90		2467.00	2180.80	2345.06	575.38
	3.00	1987.20	1996.50	1872.30		3442.80	3441.20		2590.90	2138.10	2495.57	731.47
	4.00	2063.00	2064.70	1879.30		3479.80	3471.30		2678.20	2218.20	2550.64	725.73
	5.00	2031.00	2076.20	1958.40		3355.40	3393.80		2777.50	2375.00	2566.76	669.63
	6.00	1835.50	2086.70	2049.60		3200.00	3294.40		2877.30	2488.90	2547.49	641.72
	7.00	1239.80	2103.40	2000.10		3048.70	3129.70		2936.10	2639.30	2442.44	752.87
	8.00	869.50	2050.40	1920.60		2901.10	2943.70		2909.90	2728.50	2331.96	823.80
	9.00	715.90	2026.10	1897.30		2755.90	2806.10		2864.10	2743.30	2258.39	829.31
2	1.00	632.40	1908.20	1855.30		2616.00	2710.00		2861.80	2802.10	2197.97	832.97
	2.00	672.90	1860.00	1921.60		2481.90	2595.10		2860.90	2866.80	2179.89	785.98
	3.00	743.00	1843.60	2001.20		2380.20	2486.50		2842.60	2896.40	2170.50	732.26
	4.00	768.60	1813.60	2012.50		2301.60	2454.60		2845.90	2920.10	2159.56	715.26
	5.00	774.90	1808.00	2023.80		2223.20	2373.10		2781.10	2917.60	2128.81	683.72
	6.00	831.30	1806.10	2066.70		2160.00	2347.90		2724.70	2894.20	2118.70	644.84
	7.00	946.40	1805.90	2063.60		2121.70	2355.00		2686.70	2862.10	2120.20	593.99
	8.00	1073.90	1797.30	2120.00	2031.10	2117.30	2342.90		2649.20	2851.20	2122.86	494.04
3	1.00	1134.60	1766.70	2099.60	2042.00	2091.90	2354.80		2604.90	2797.80	2111.54	467.97
	2.00	1146.70	1740.90	2076.20	2070.50	2066.70	2276.60		2516.40	2689.90	2072.99	438.19
	3.00	1177.90	1699.70	2012.90	2053.70	2025.60	2238.00		2420.60	2592.70	2027.64	404.88
	4.00	1188.50	1695.10	1897.00	2075.70	2033.20	2201.30		2354.90	2535.50	1997.65	385.47
	5.00	1151.70	1708.40	1889.30	2103.10	2088.10	2197.40		2340.00	2475.30	1994.16	398.11
	6.00	1080.80	1740.20	1892.00		2155.60	2241.60		2352.60	2459.90	1988.96	465.38
	7.00	942.60	1770.50	1891.90		2237.70	2289.40		2362.90	2481.60	1996.66	531.39
	8.00	620.00	1841.00	1931.70		2272.20	2325.30		2382.70	2475.40	1978.33	662.60
	9.00		1874.40	1944.90	2245.40	2292.20	2347.20		2394.50	2480.70	2225.61	218.83
	10.00		1961.50	1998.40	2277.00	2333.20	2404.90		2421.80	2476.50	2267.61	202.94

Table 11 (Continued). Mean Shoreline Positions (feet) by Compartment

Segment	Comp	1899	1932	1943	1971	1977	1986	1994	1998	2003	Avg	Std Dev
4	1.00		2036.60	2040.60	2290.70	2353.80	2472.10		2436.20	2496.30	2303.76	191.31
	2.00		2056.40	2050.60	2276.90	2340.90	2454.10	2386.60	2403.90	2485.40	2306.85	165.00
	3.00	1795.80	2069.80	2140.10	2272.20	2327.60	2396.30	2396.00	2404.10	2456.70	2250.96	213.45
	4.00	2203.50	2085.00	2167.60	2325.00	2358.00	2397.70	2411.70	2407.60	2455.70	2312.42	125.59
	5.00	2078.90	2074.20	2199.50	2376.10	2389.80	2410.80	2412.00	2408.60	2469.90	2313.31	151.25
	6.00	1644.30	2119.90	2248.70	2399.60	2430.10	2480.40	2437.10	2437.80	2492.50	2298.93	282.10
	7.00	1122.50	2203.70	2347.90	2439.20	2469.70	2546.00	2483.30	2508.80	2554.80	2297.32	474.23
	8.00	775.90	2296.30	2478.30	2519.90	2515.40	2589.20	2544.40	2553.80	2621.50	2321.63	615.89
	9.00	504.10	2362.30	2538.90	2606.10	2587.30	2682.20	2604.40	2608.70	2678.00	2352.44	736.34
	10.00		2364.50	2612.30	2687.30	2663.60	2783.60	2648.20	2666.40	2709.80	2641.96	129.38

Table 12. Shoreline Change Rates (feet/year) by Epochs

Segment	Comp #	Length	1899-1932	1932-1943	1943-1971	1943-1977	1977-1986	1986-1994	1994-1998	1986-1998	1998-2003
1	1	800	4.47	17.96		3.59	48.52			-23.85	-30.57
	2	700	2.82	-0.01		23.84	60.62			-67.15	-56.34
	3	750	0.28	-11.29		46.19	-0.18			-68.96	-89.13
	4	900	0.05	-16.85		47.07	-0.94			-64.32	-90.55
	5	700	1.37	-10.71		41.09	4.27			-49.98	-79.23
	6	750	7.61	-3.37		33.84	10.49			-33.83	-76.46
	7	800	26.17	-9.39		30.84	9.00			-15.70	-58.43
	8	600	35.78	-11.80		28.84	4.73			-2.74	-35.71
	9	840	39.70	-11.71		25.25	5.58			4.70	-23.78
2	1	1000	38.66	-4.81		22.37	10.40			12.65	-11.75
	2	900	35.97	5.60		16.48	12.58			21.56	1.16
	3	600	33.35	14.33		11.15	11.81			28.88	10.59
	4	700	31.67	18.08		8.50	17.00			31.74	14.61
	5	1000	31.31	19.62		5.86	16.66			33.09	26.87
	6	1000	29.54	23.69		2.74	20.88			30.56	33.37
	7	1000	26.05	23.43		1.71	25.92			26.90	34.53
	8	630	21.92	29.34		-0.08	25.07			24.84	39.76
3	1	1000	19.15	30.26	-2.07	8.33	29.21			20.28	37.97
	2	1000	18.01	30.48	-0.20	-0.63	23.32			19.45	34.15
	3	1000	15.81	28.47	1.46	-4.68	23.60			14.81	33.88
	4	1000	15.35	18.35	6.38	-7.08	18.68			12.46	35.55
	5	1000	16.87	16.45	7.64	-2.50	12.14			11.57	26.63
	6	1000	19.98	13.80			9.56			9.00	21.12
	7	1000	25.09	11.04			5.74			5.96	23.37
	8	700	37.00	8.25			5.90			4.66	18.25
	9	1000		6.41	10.73	7.80	6.11			3.84	16.97
	10	930		3.35	9.95	9.37	7.97			1.37	10.77

Table 12 (Continued). Shoreline Change Rates (feet/year) by Epochs

Segment	Comp #	Length	1899-1932	1932-1943	1943-1971	1943-1977	1977-1986	1986-1994	1994-1998	1986-1998	1998-2003
4	1	800		0.36	8.93	10.52	13.14			-2.91	11.83
	2	800		-0.53	8.08	10.67	12.58	-7.95	4.51	-4.07	16.04
	3	1000	8.30	6.39	4.72	9.23	7.63	-0.04	2.11	0.63	10.35
	4	1000	-3.59	7.51	5.62	5.50	4.41	1.65	-1.07	0.80	9.47
	5	1000	-0.14	11.39	6.31	2.28	2.33	0.14	-0.89	-0.18	12.07
	6	1000	14.41	11.71	5.39	5.08	5.59	-5.10	0.18	-3.45	10.77
	7	1000	32.76	13.11	3.26	5.08	8.48	-7.39	6.64	-3.02	9.06
	8	750	46.07	16.55	1.49	-0.75	8.20	-5.28	2.45	-2.87	13.33
	9	500	56.31	16.05	2.40	-3.13	10.54	-9.16	1.12	-5.96	13.64
	10	500		22.53	2.68	-3.95	13.33	-15.95	4.74	-9.51	8.54

Table 13. Shoreline Change Rates (feet/year) Relative to 2003

Segment	Comp #	Length	1899-2003	1932-2003	1943-2003	1971-2003	1977-2003	1986-2003	1998-2003
1	1	800	6.26	5.92	3.83		-3.27	-25.45	-30.57
	2	700	8.59	9.41	7.22		-27.52	-64.58	-56.34
	3	750	9.31	10.77	7.75		-51.40	-73.75	-89.13
	4	900	9.37	11.25	9.02		-49.25	-70.55	-90.55
	5	700	10.09	11.92	10.01		-38.06	-56.93	-79.23
	6	750	11.63	12.12	10.41		-26.80	-43.95	-76.46
	7	800	16.11	13.08	13.10		-14.53	-25.85	-58.43
	8	600	18.65	13.94	15.03		-5.29	-10.57	-35.71
	9	840	19.17	13.64	15.25		0.84	-2.06	-23.78
2	1	1000	19.89	14.84	16.43		8.36	5.29	-11.75
	2	900	19.55	15.04	16.13		15.93	16.71	1.16
	3	600	18.56	14.34	14.93		21.15	24.54	10.59
	4	700	18.94	15.27	13.80		24.86	27.67	14.61
	5	1000	17.72	13.84	14.39		27.29	31.61	26.87
	6	1000	16.74	12.95	13.11		28.08	31.23	33.37
	7	1000	15.60	12.51	12.66		27.72	28.71	34.53
	8	630	15.00	14.18	12.10	25.20	27.07	28.39	39.76
3	1	1000	13.68	11.69	10.90	23.32	25.51	24.49	37.97
	2	1000	12.57	10.65	10.08	19.69	22.73	22.94	34.15
	3	1000	11.68	10.08	9.36	17.15	20.19	19.34	33.88
	4	1000	11.33	10.03	9.88	14.43	17.66	17.94	35.55
	5	1000	11.33	9.47	9.11	11.59	13.92	15.14	26.63
	6	1000	11.89	9.31	8.97		10.92	11.88	21.12
	7	1000	13.15	9.44	9.18		8.42	10.10	23.37
	8	700	15.29	8.70	8.60		7.00	7.88	18.25
	9	1000		8.45	8.36	6.50	6.42	6.96	16.97
	10	930		7.59	7.69	5.50	4.70	3.60	10.77
4	1	800		7.03	7.44	5.46	4.01	0.59	11.83
	2	800		6.45	6.95	5.25	3.80	0.71	16.04
	3	1000	6.07	5.32	5.16	5.00	4.13	2.94	10.35
	4	1000	3.16	5.00	4.53	3.49	3.13	2.86	9.47
	5	1000	4.20	5.04	3.99	2.27	2.34	2.73	12.07
	6	1000	7.46	4.83	3.73	2.04	1.30	-0.08	10.77
	7	1000	11.44	4.42	3.35	2.95	2.16	-0.15	9.06
	8	750	13.98	3.52	2.10	2.65	2.77	0.98	13.33
	9	500	16.39	3.50	1.99	1.73	1.80	-1.30	13.64
	10	500		3.87	1.50	0.35	-0.09	-5.22	8.54

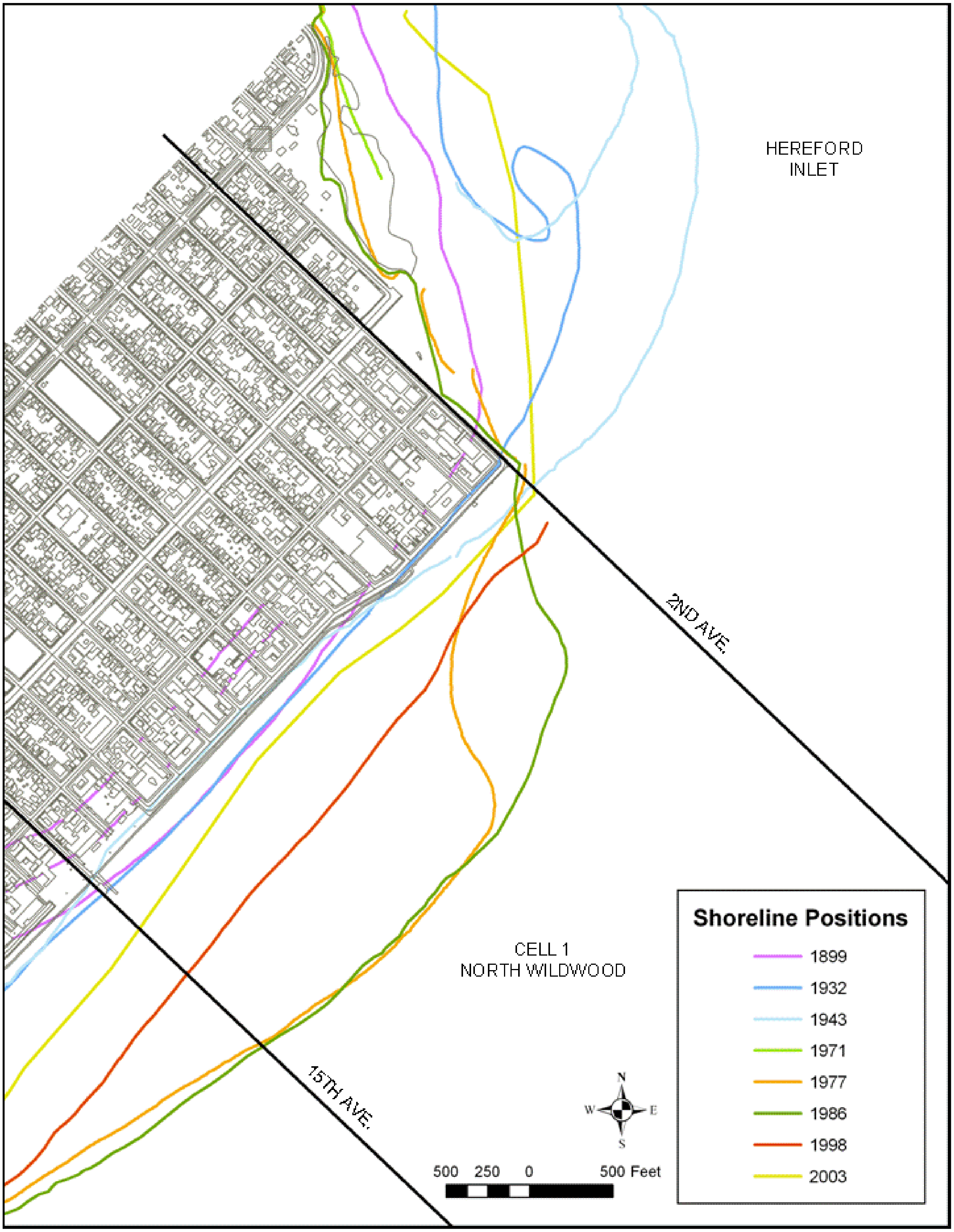


Figure 8. North Wildwood Shoreline Positions

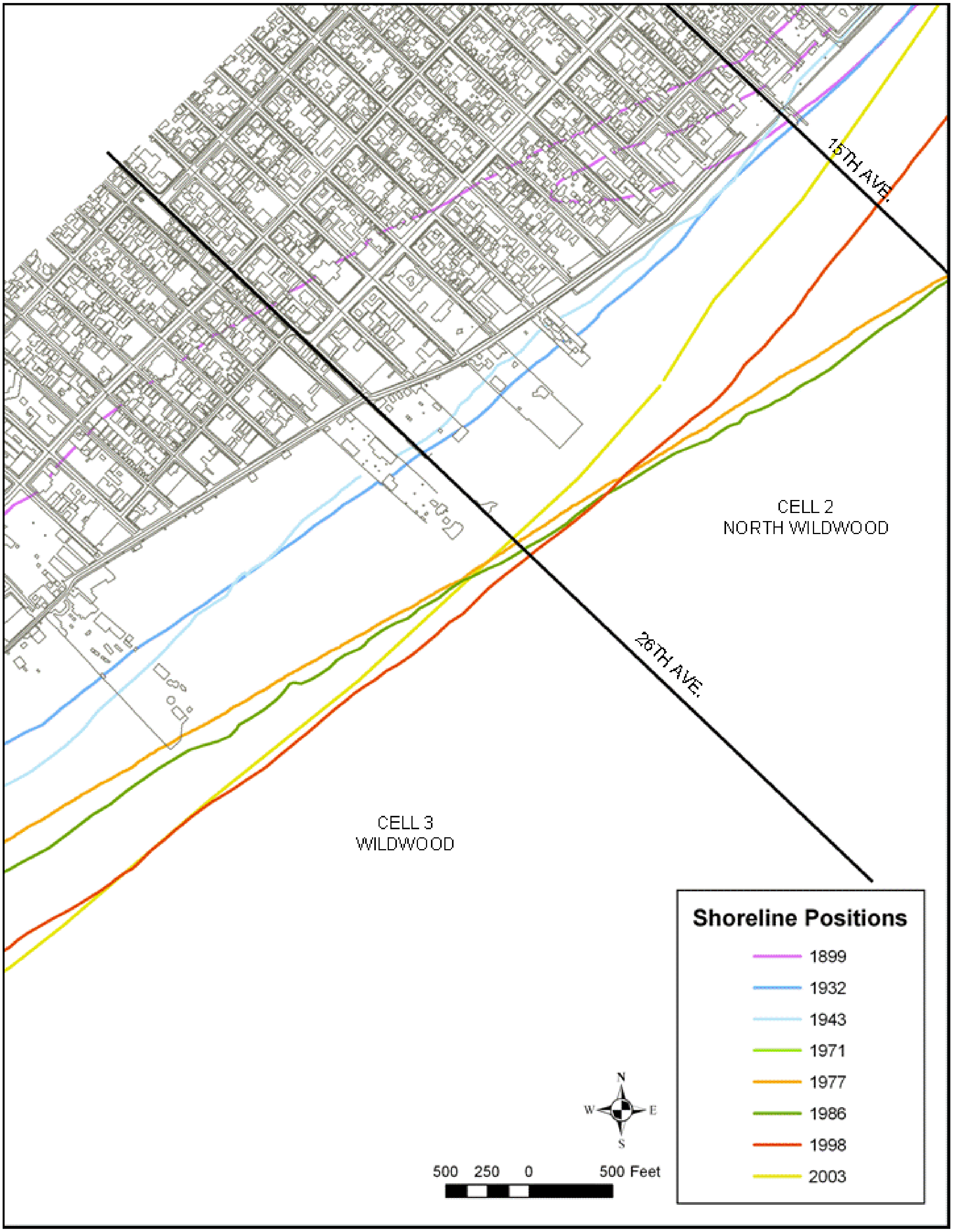


Figure 9. North Wildwood and Wildwood Shoreline Positions

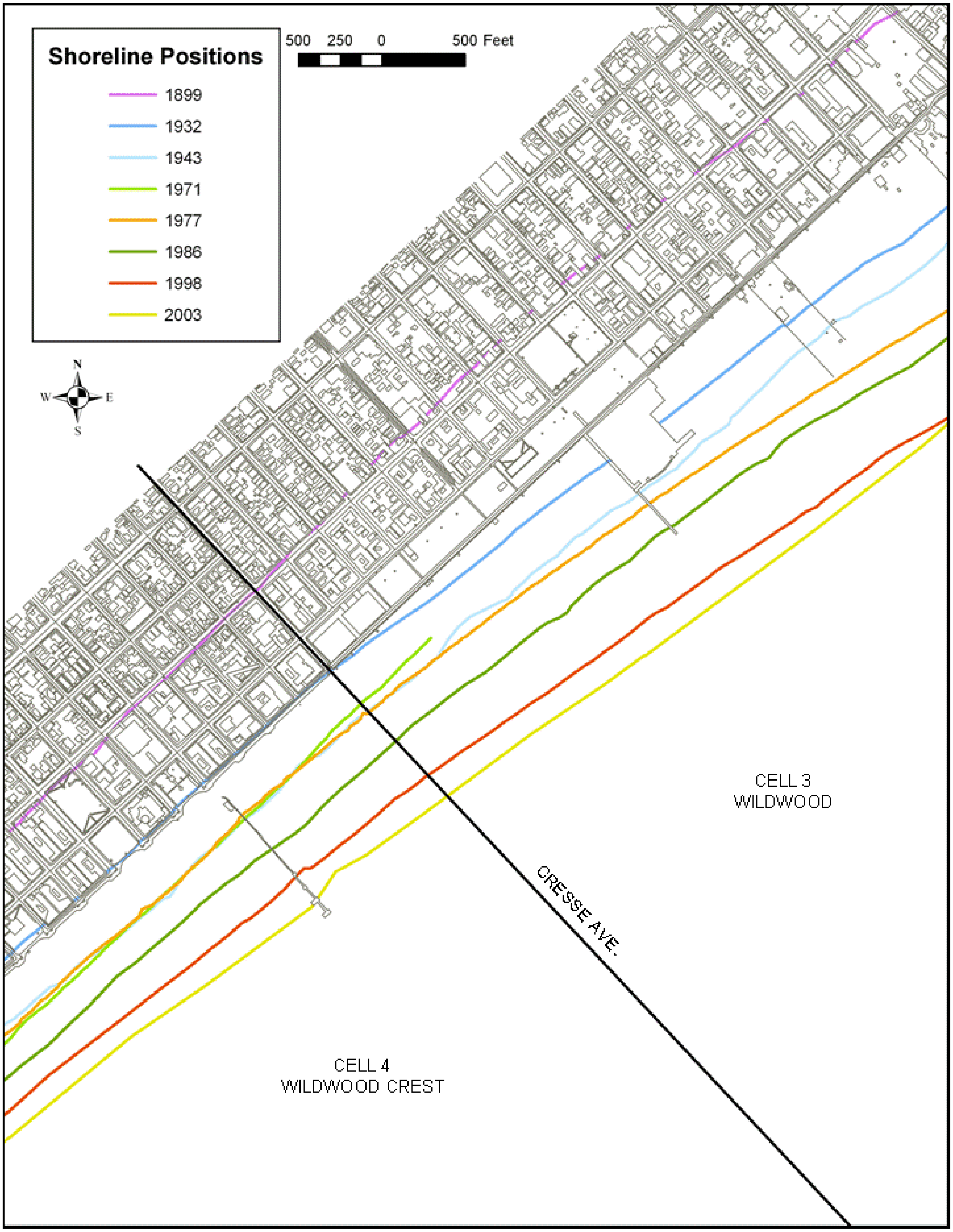


Figure 10. Wildwood and Wildwood Crest Shoreline Positions

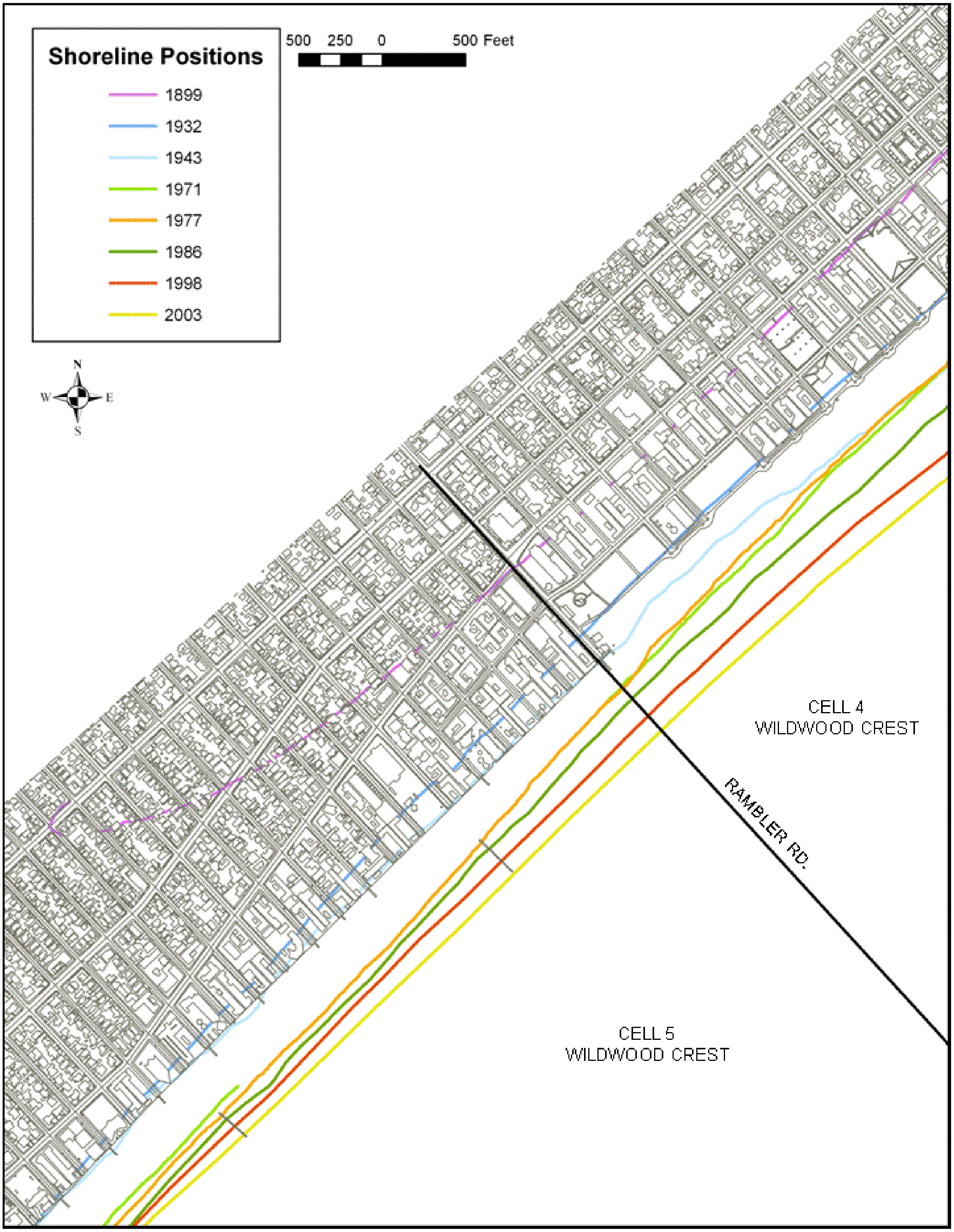


Figure 11. Wildwood Crest Shoreline Positions

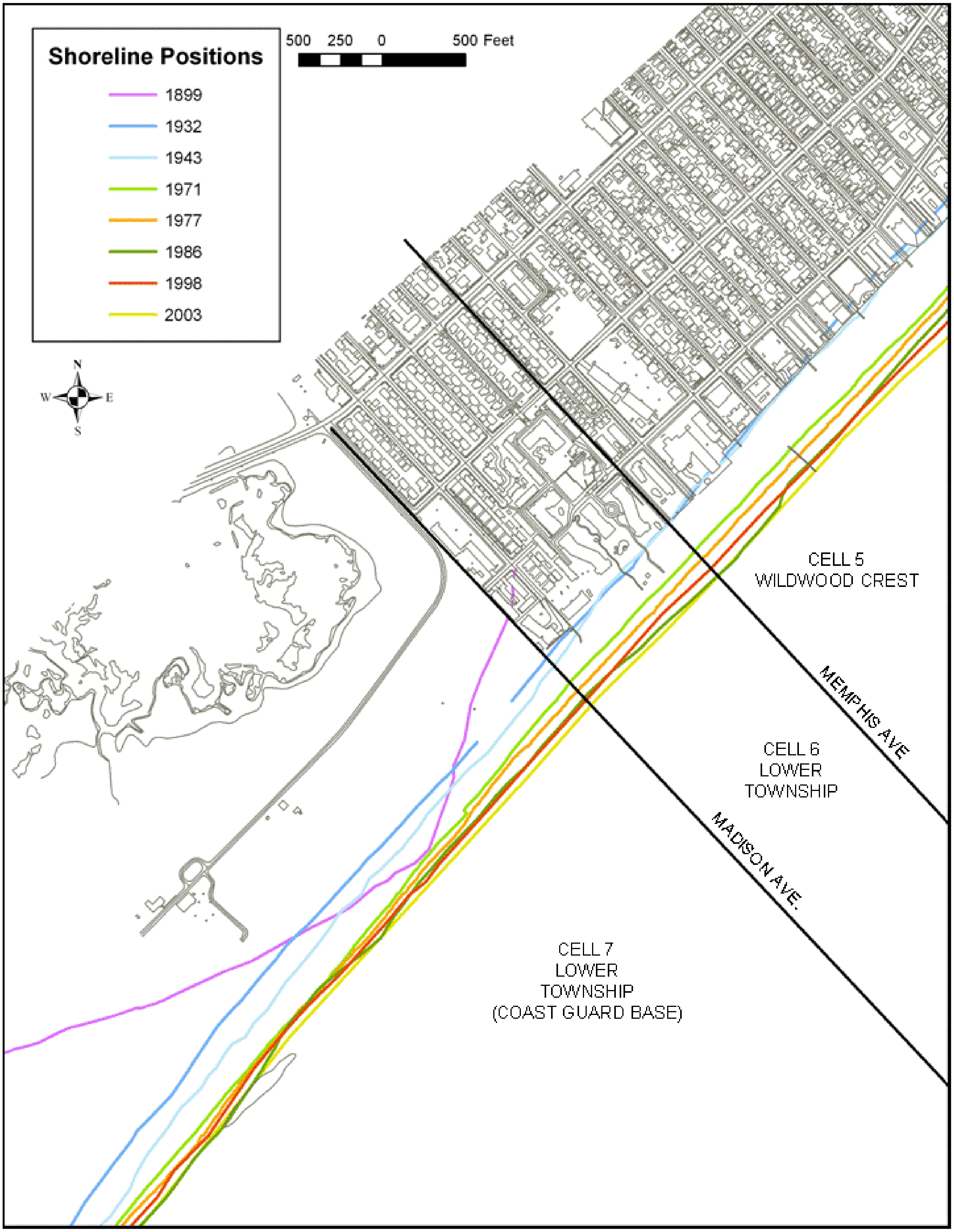


Figure 12. Lower Township Shoreline Positions

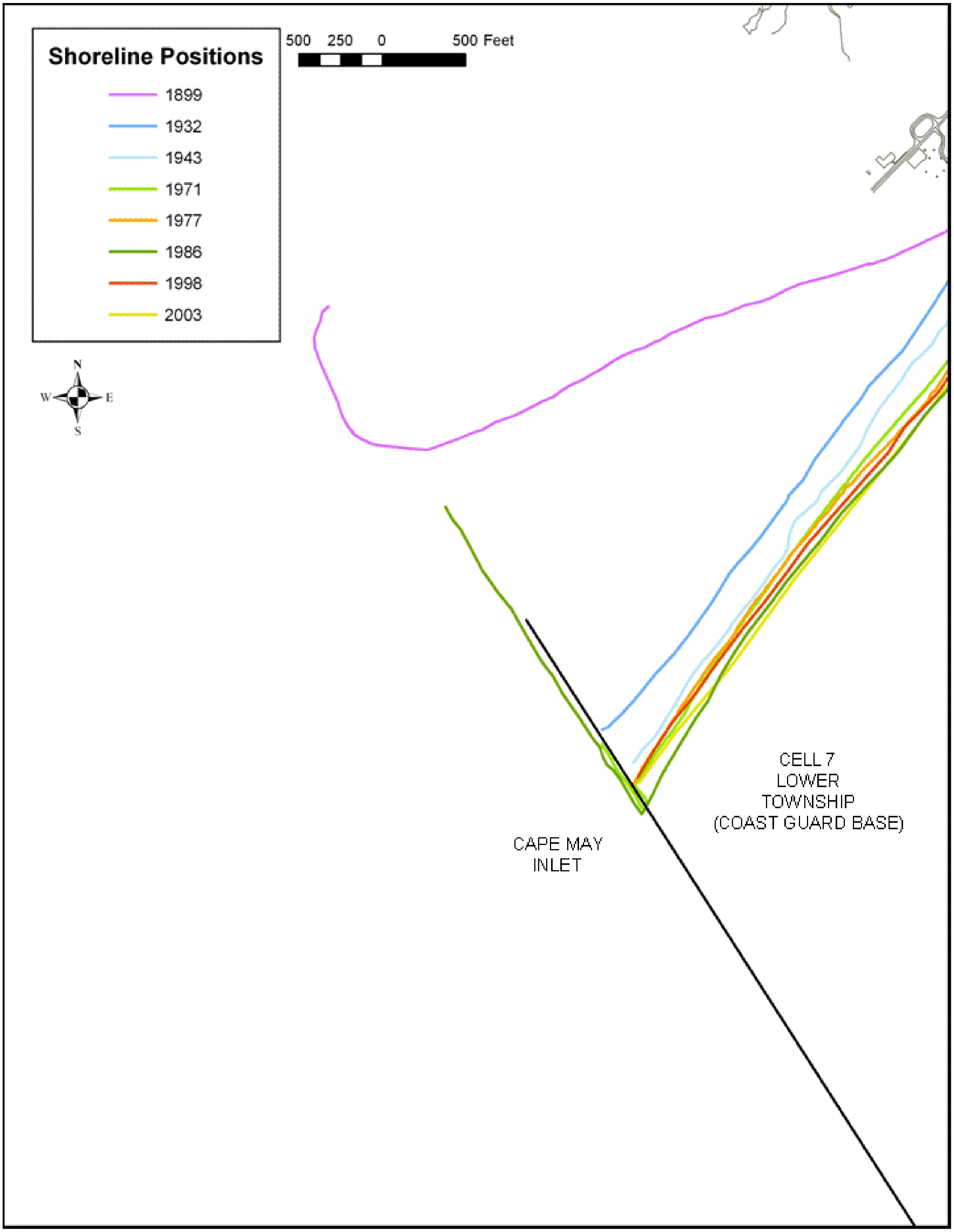


Figure 13. Cape May Inlet Vicinity Shoreline Positions

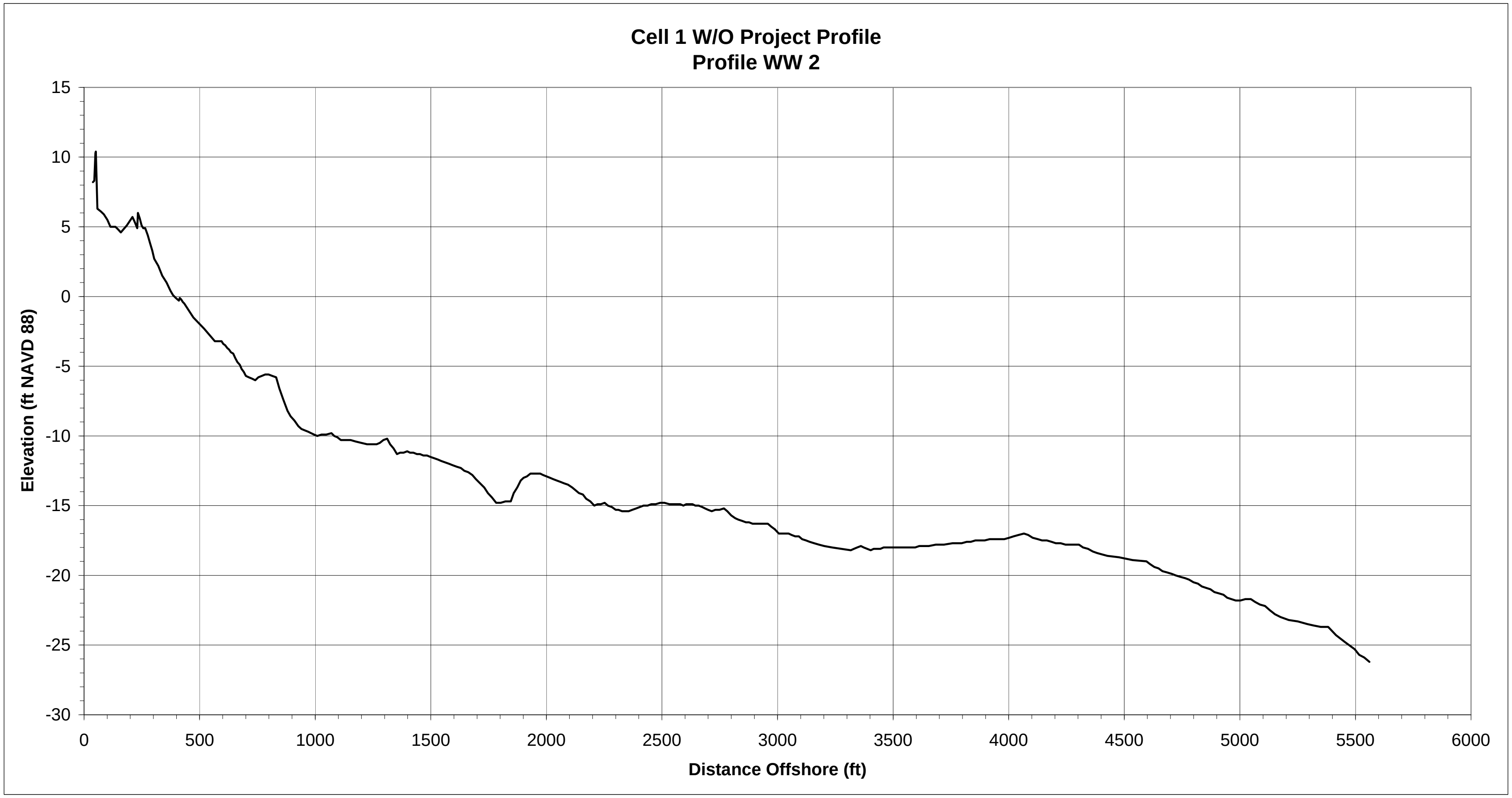


Figure 14. W/O Project Conditions Profile for Cell 1

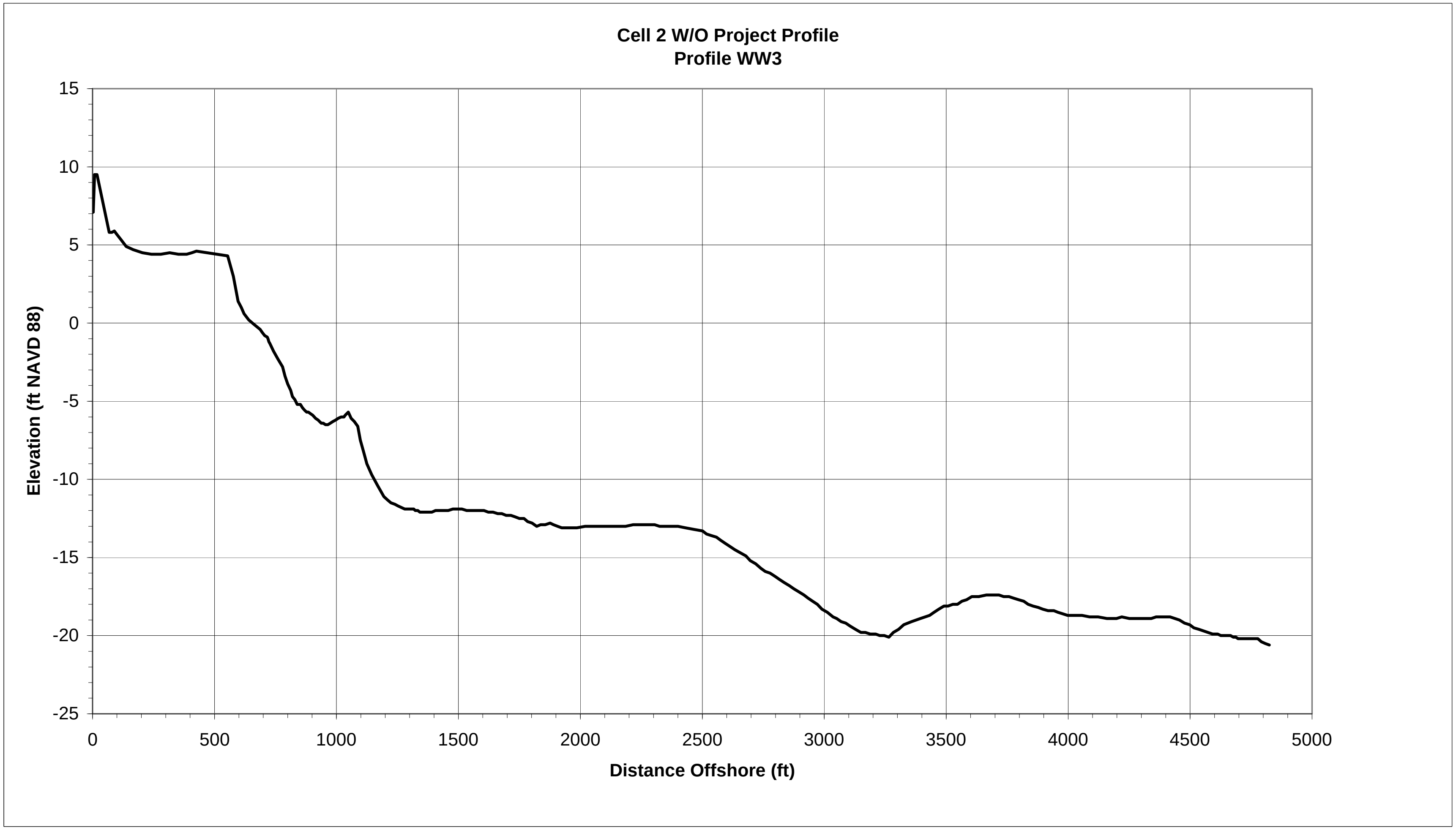


Figure 15. W/O Project Conditions Profile for Cell 2

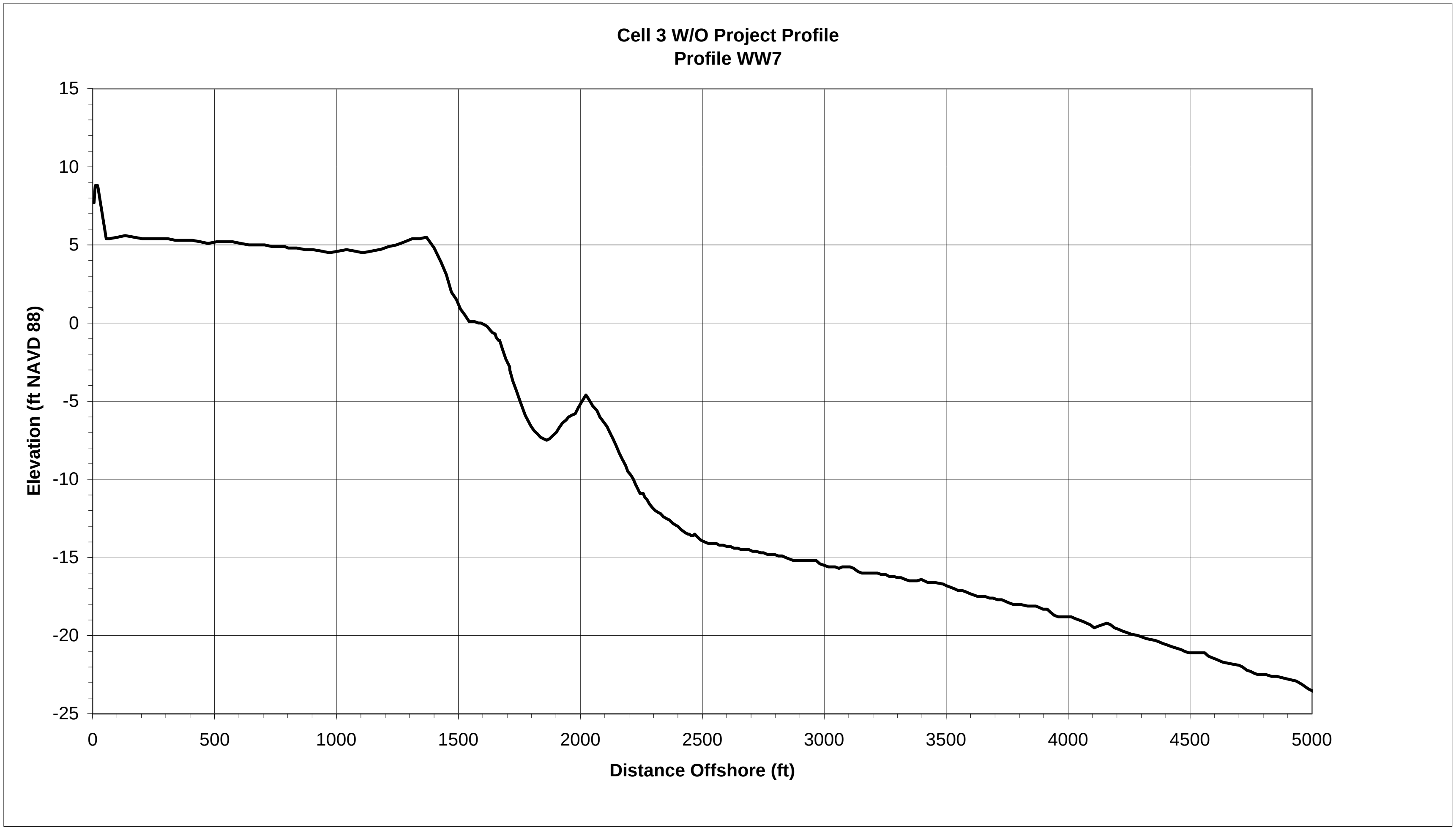


Figure 16. W/O Project Conditions Profile for Cell 3

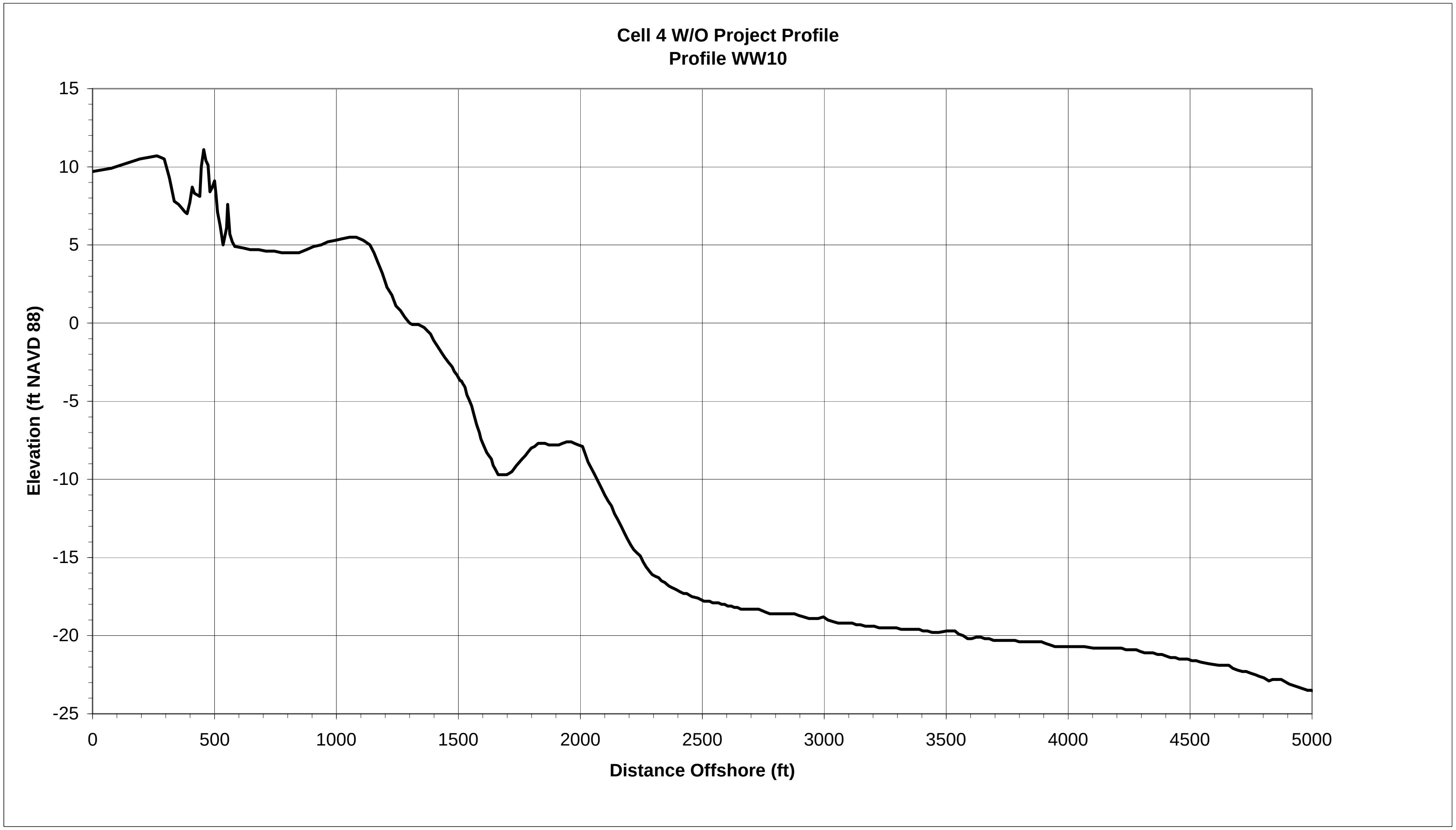


Figure 17. W/O Project Conditions Profile for Cell 4

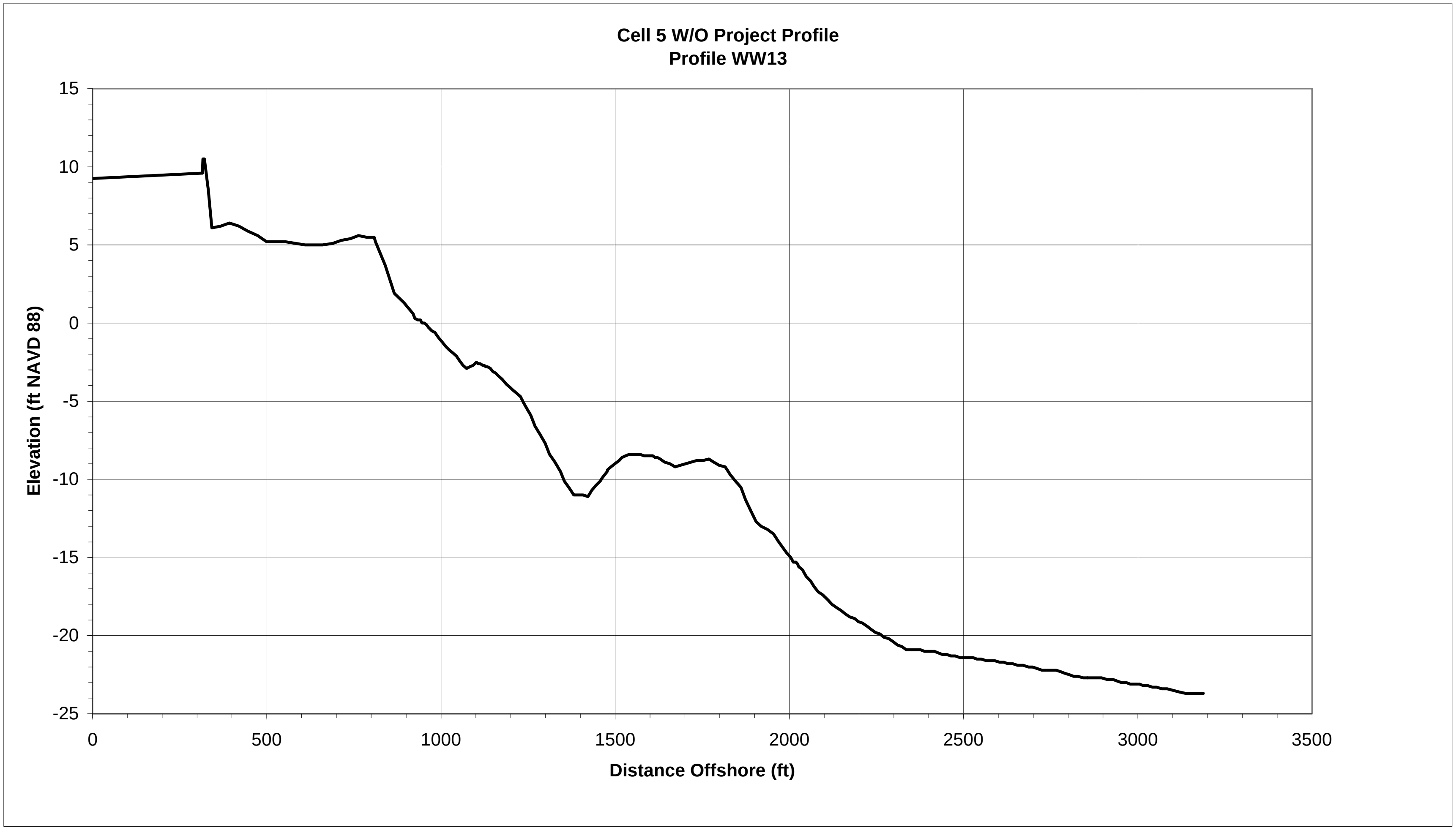


Figure 18. W/O Project Conditions Profile for Cell 5

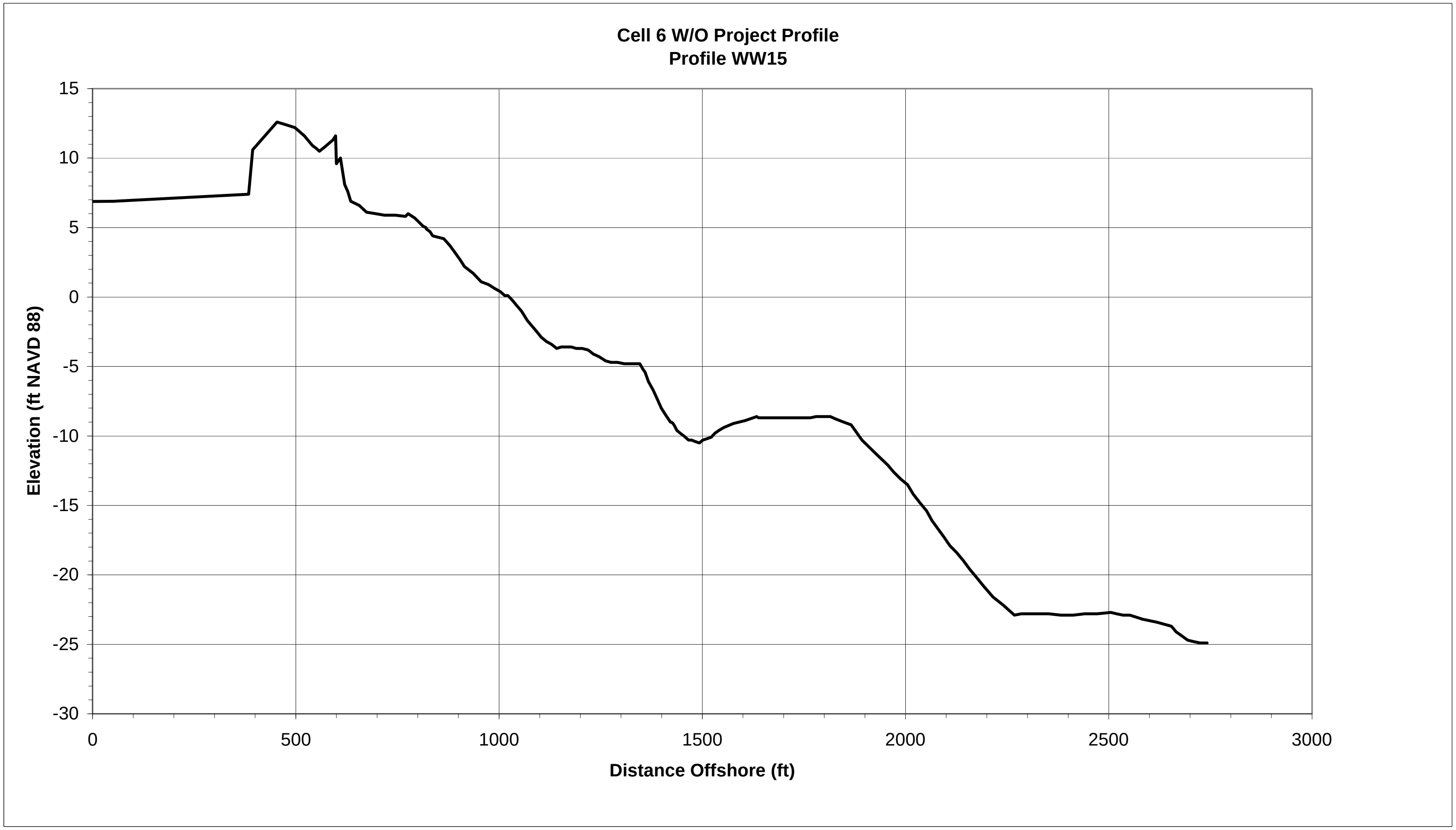


Figure 19. W/O Project Conditions Profile for Cell 6

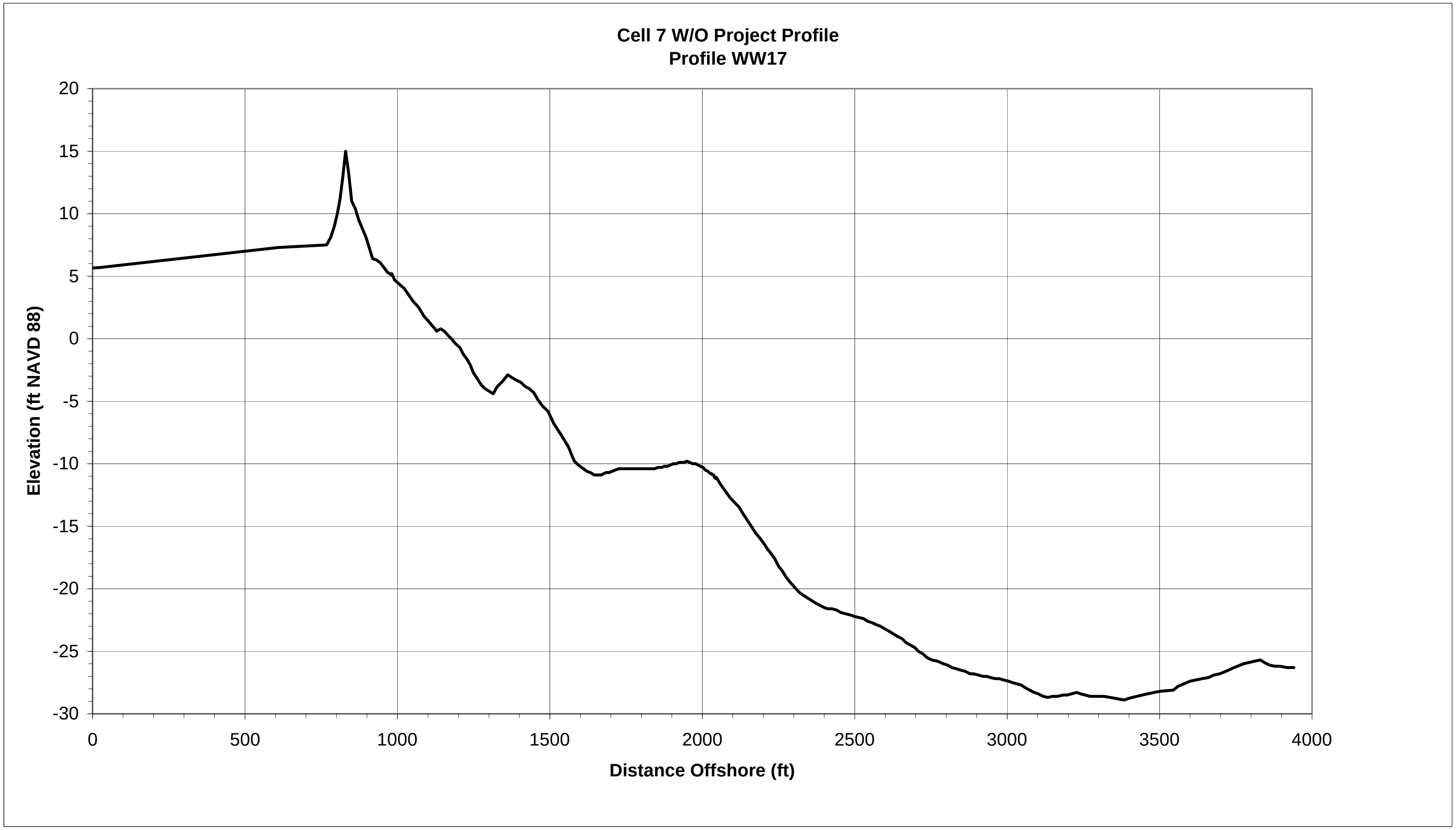


Figure 20. W/O Project Conditions Profile for Cell 7

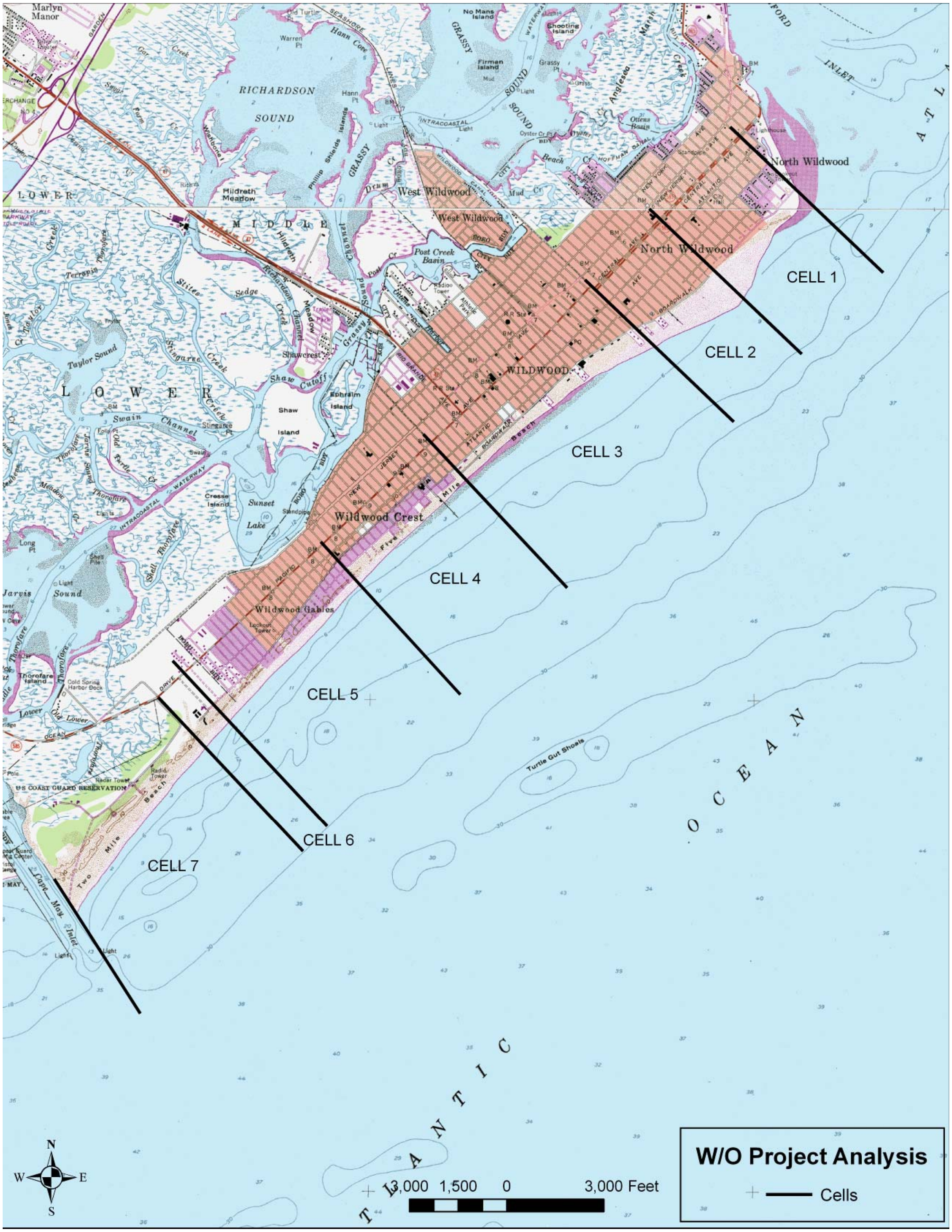


Figure 21. Cell Limits

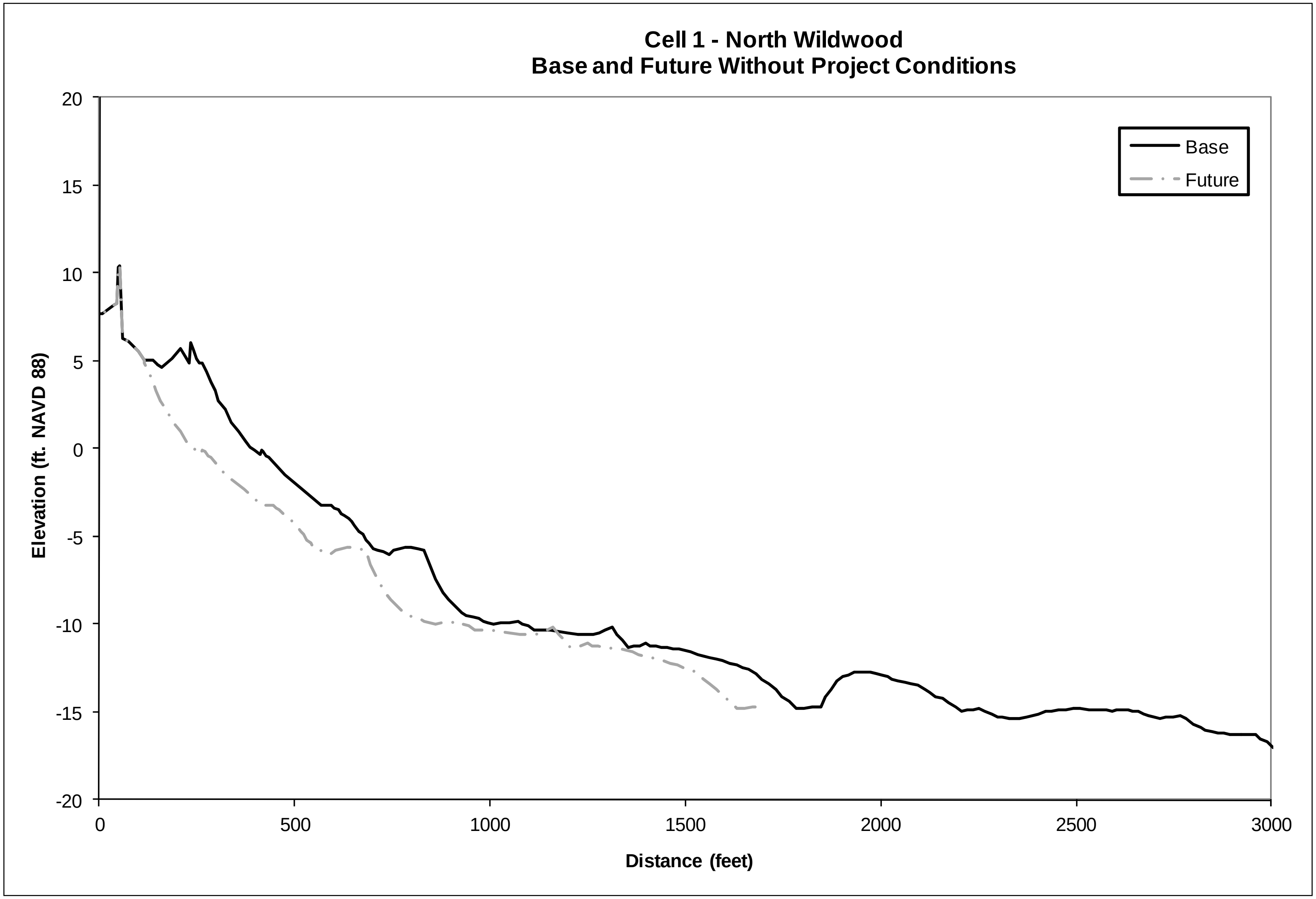


Figure 22. Base and Future Without Project Conditions for Cell 1

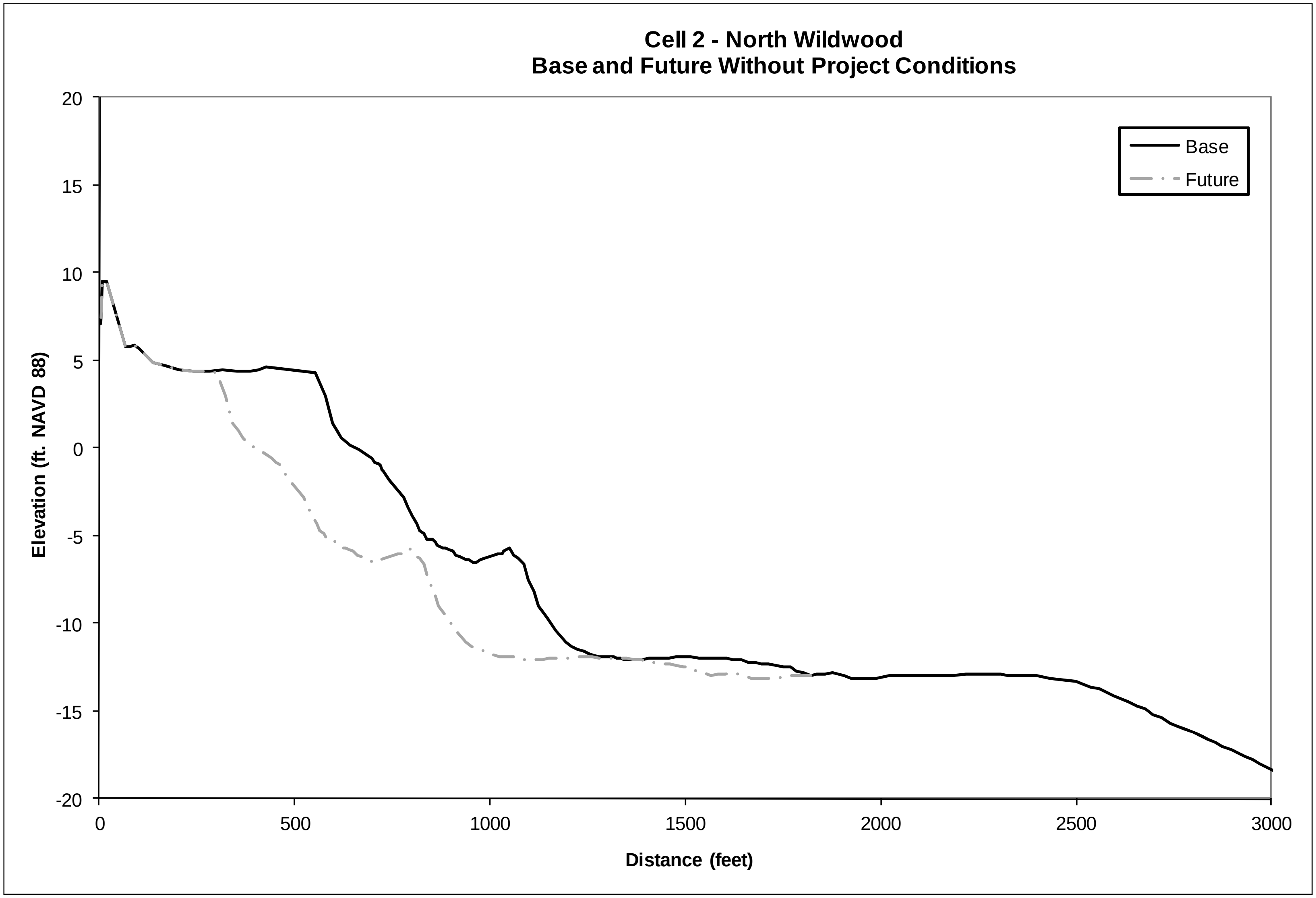


Figure 23. Base and Future Without Project Conditions for Cell 2

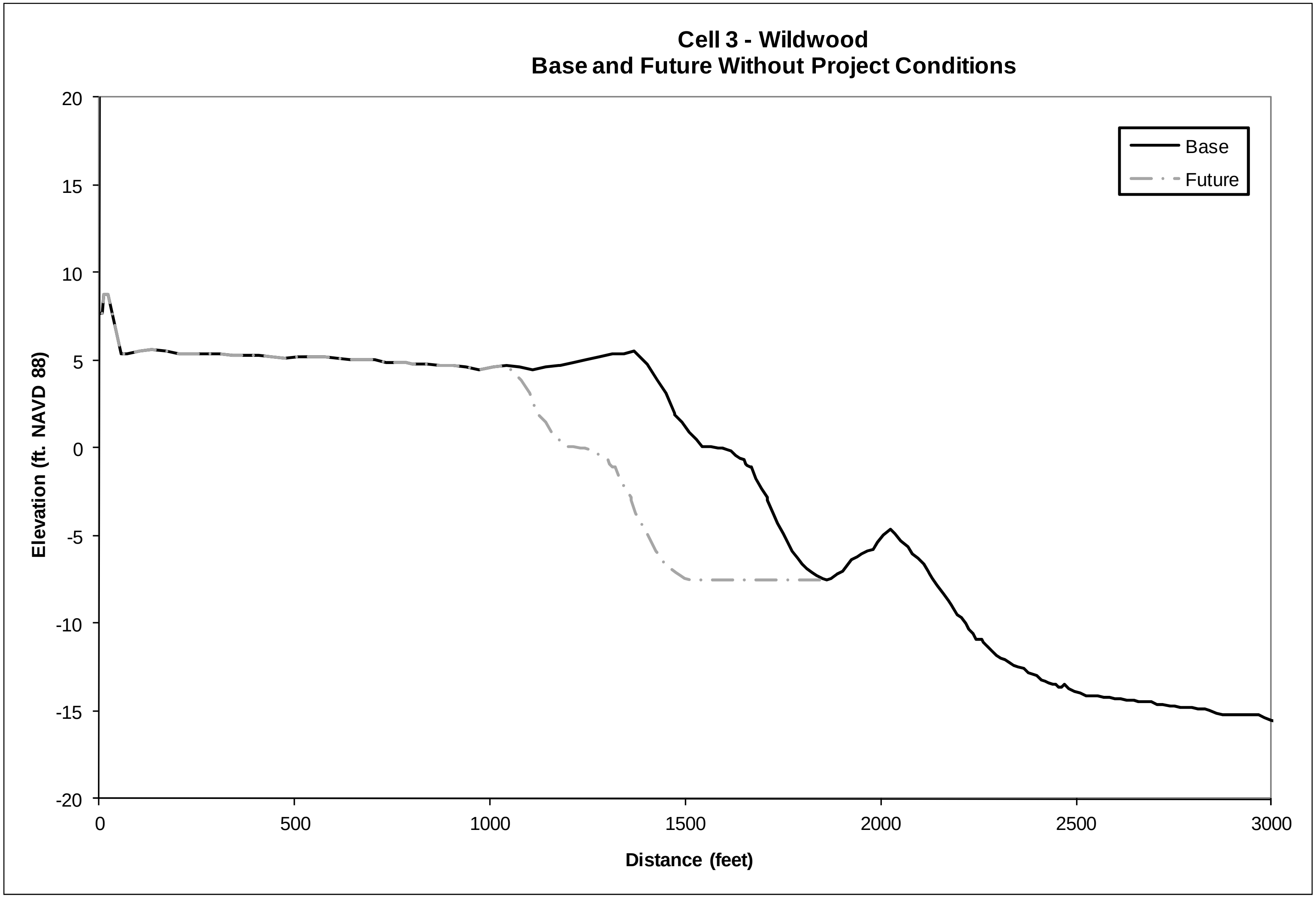


Figure 24. Base and Future Without Project Conditions for Cell 3

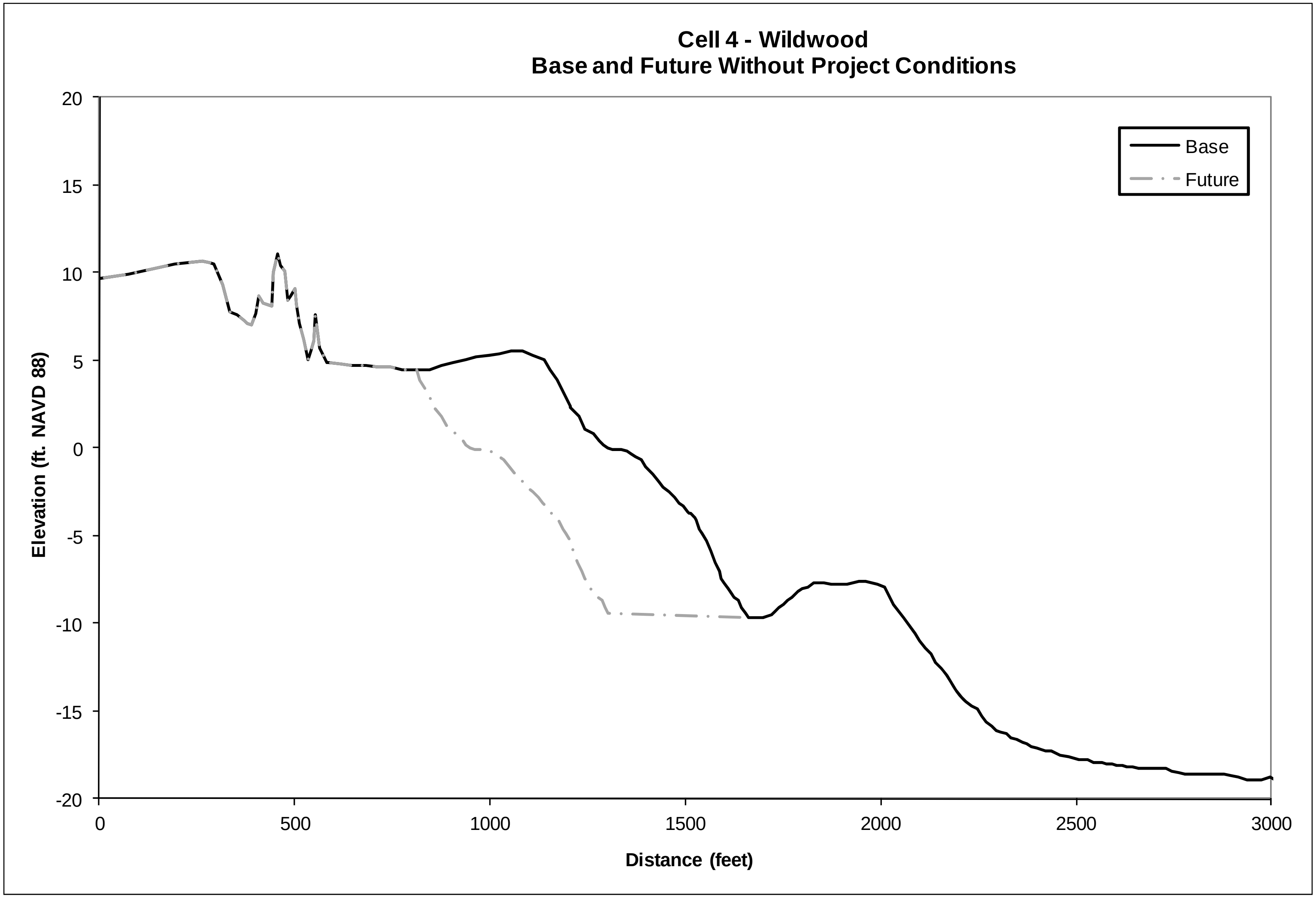


Figure 25. Base and Future Without Project Conditions for Cell 4

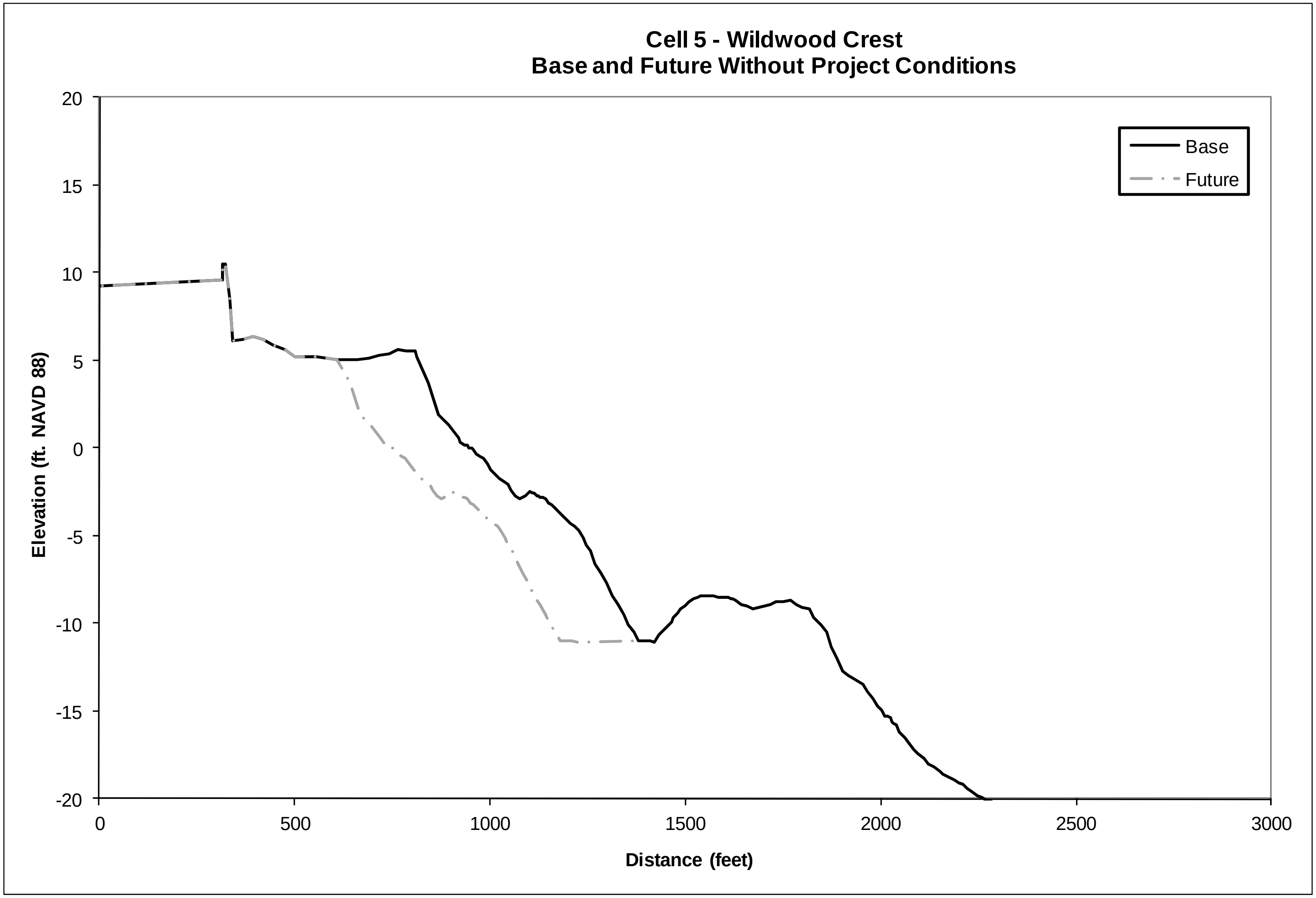


Figure 26. Base and Future Without Project Conditions for Cell 5

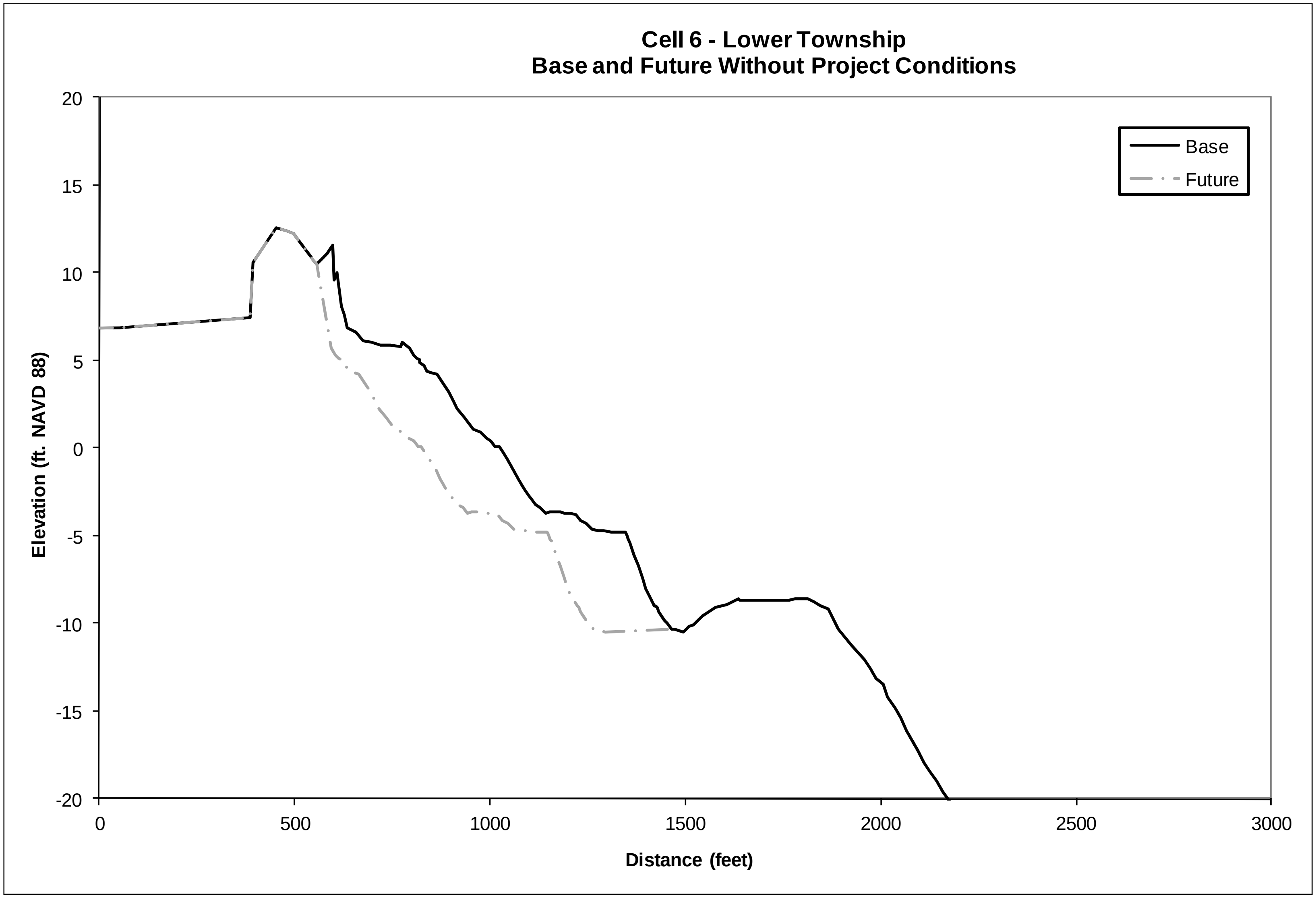


Figure 27. Base and Future Without Project Conditions for Cell 6

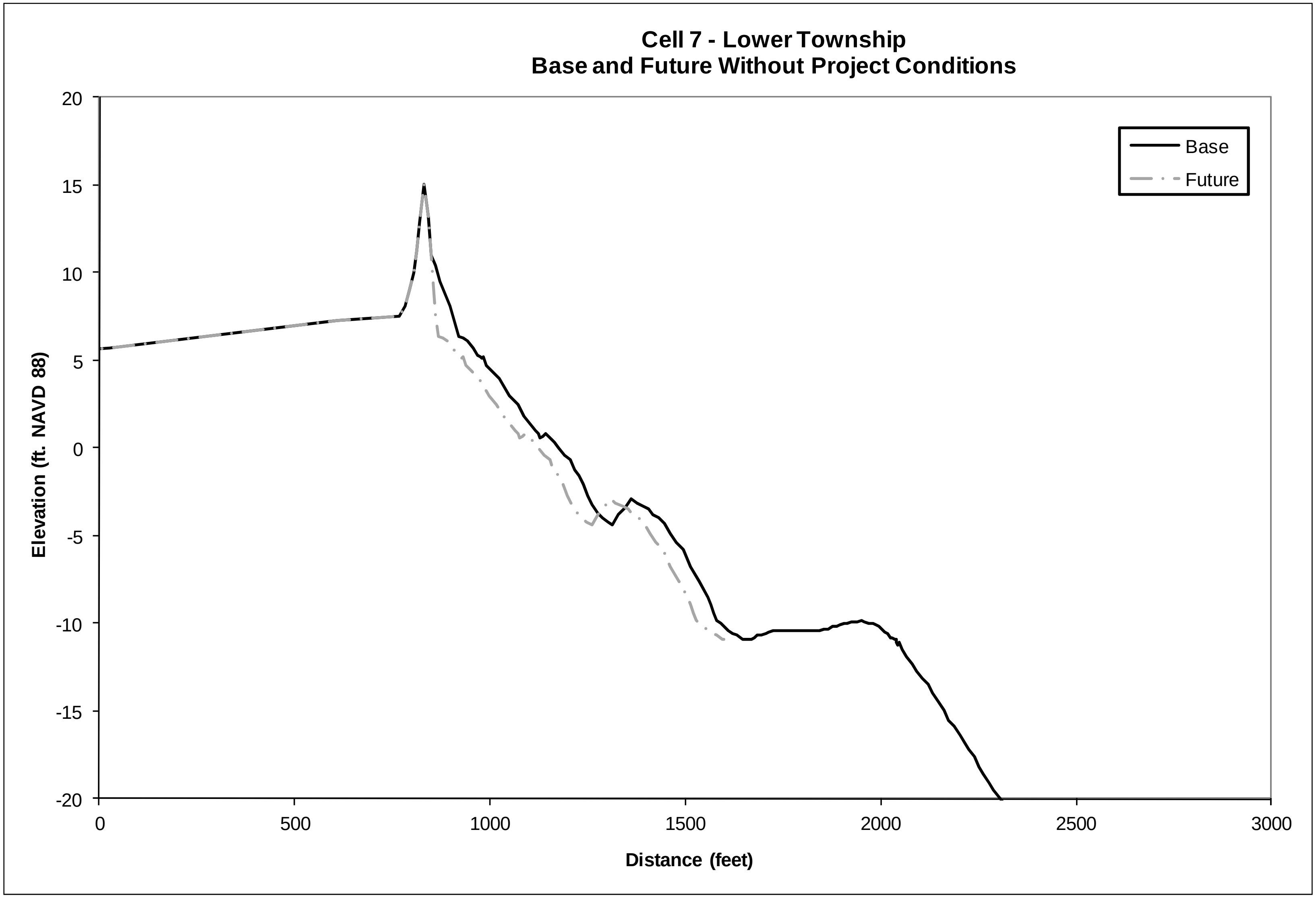


Figure 28. Base and Future Without Project Conditions for Cell 7

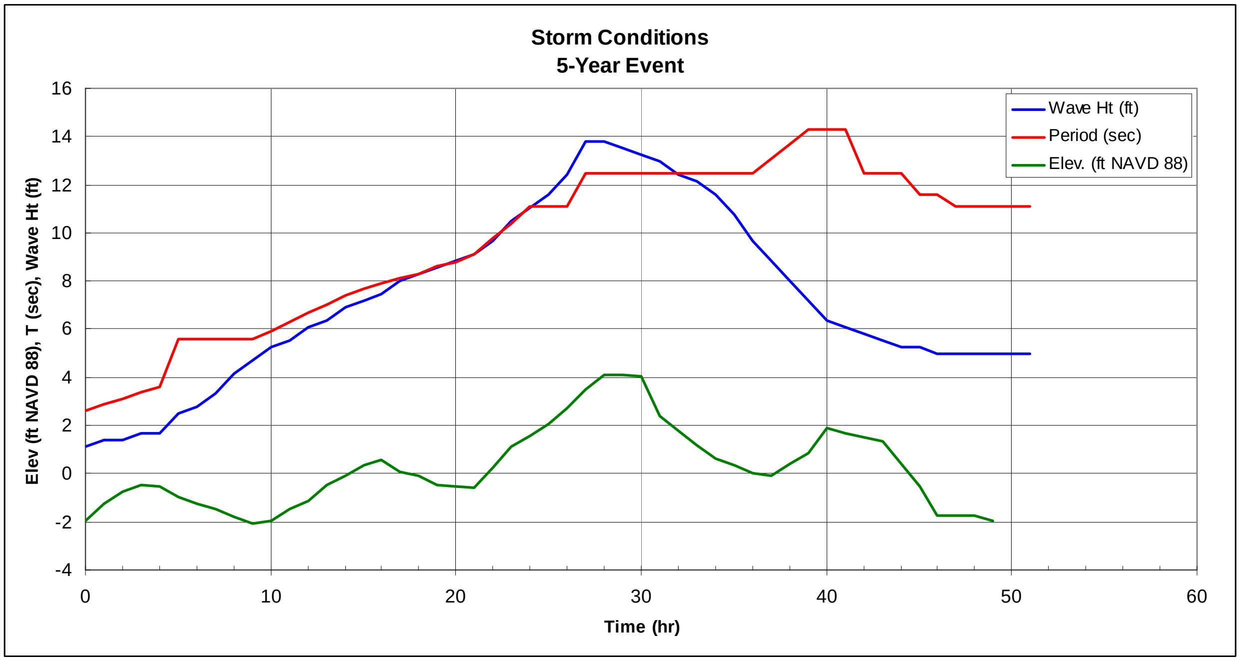


Figure 29. "5-yr" Storm Conditions used in Storm Damage Analysis

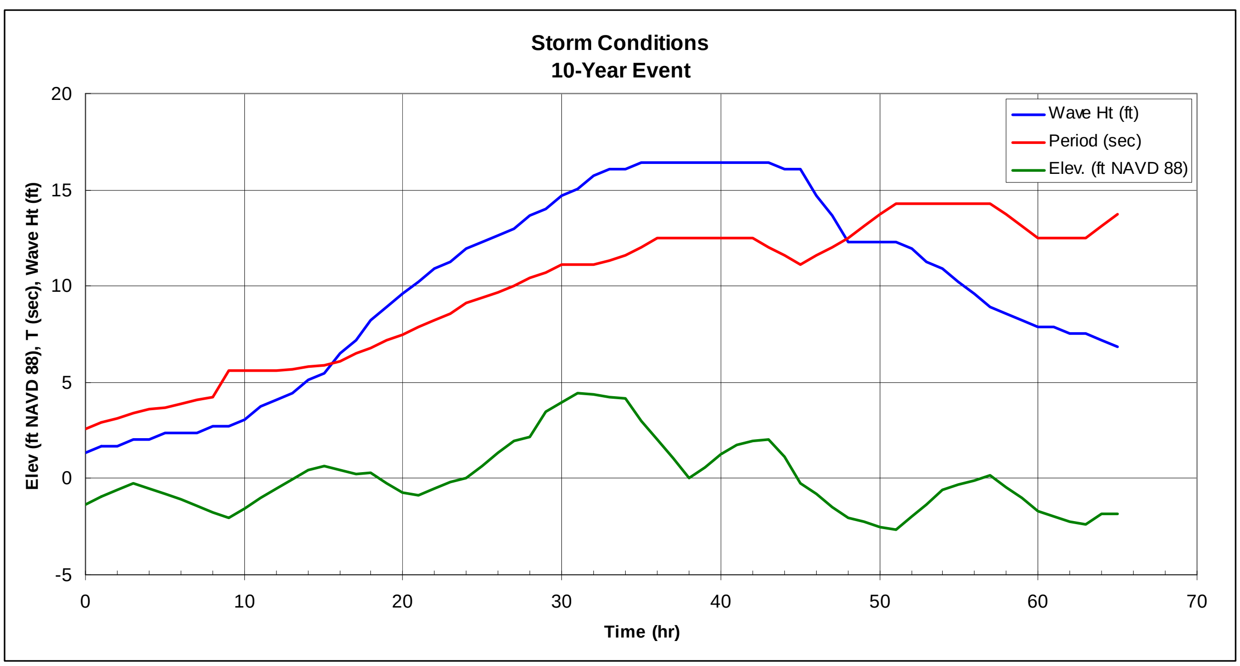


Figure 30. "10-yr" Storm Conditions used in Storm Damage Analysis

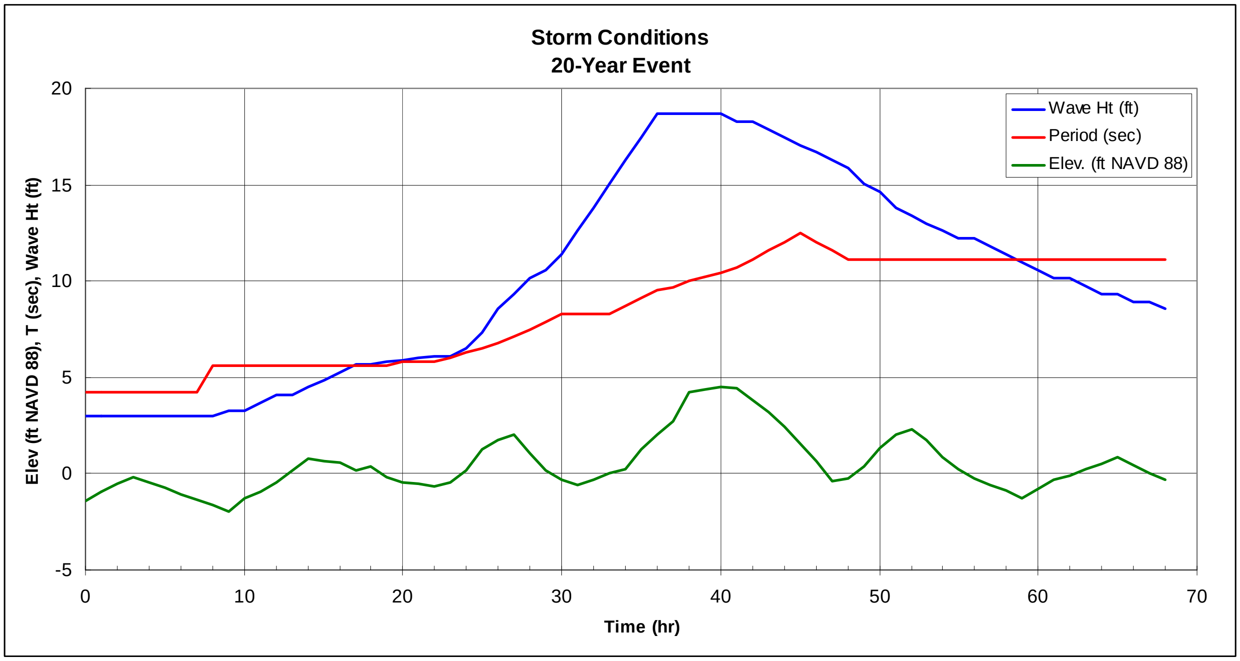


Figure 31. "20-yr" Storm Conditions used in Storm Damage Analysis

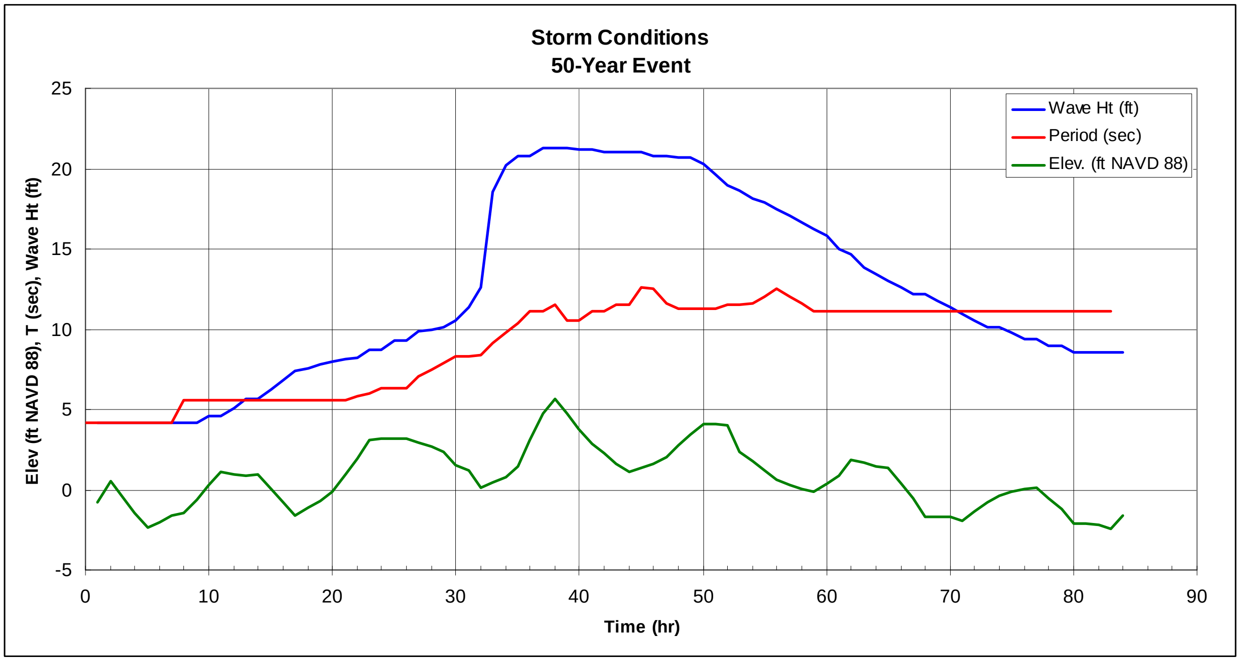


Figure 32. "50-yr" Storm Conditions used in Storm Damage Analysis

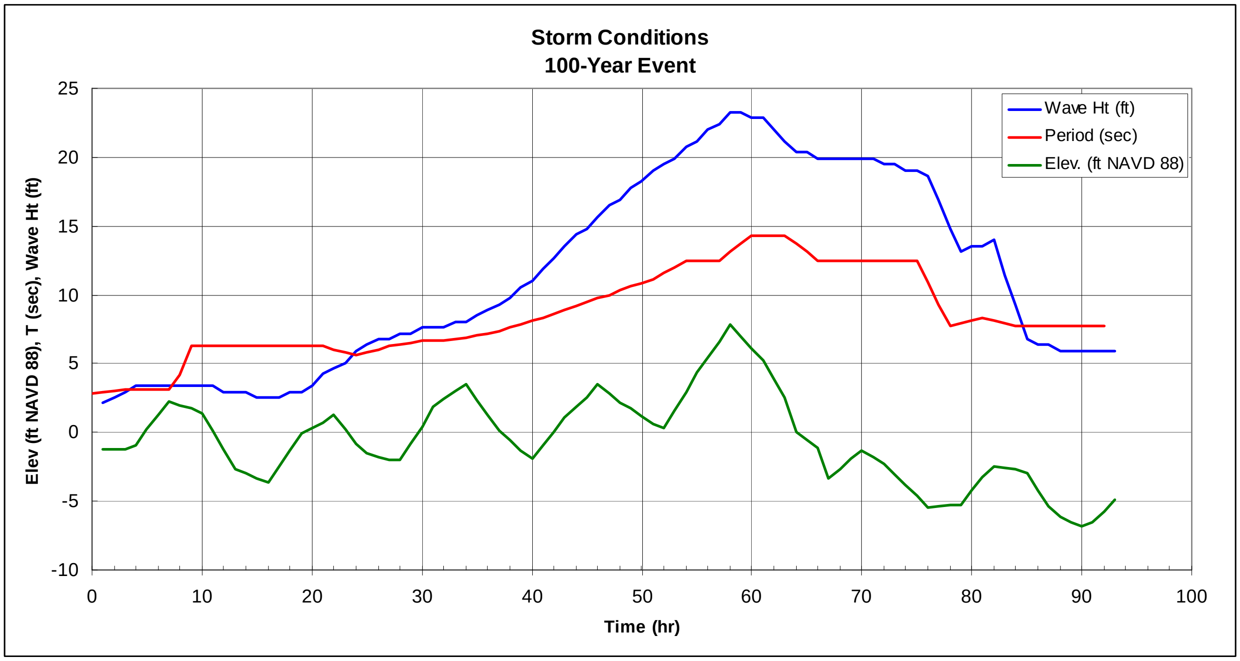


Figure 33. "100-yr" Storm Conditions used in Storm Damage Analysis

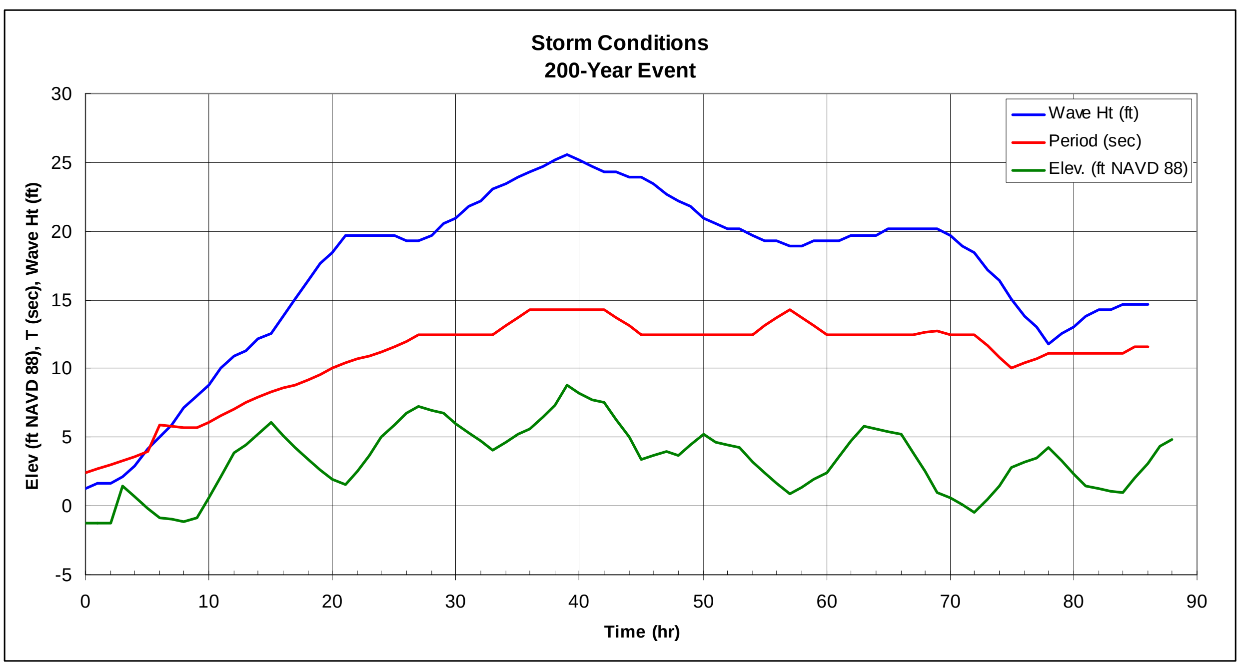


Figure 34. "200-yr" Storm Conditions used in Storm Damage Analysis

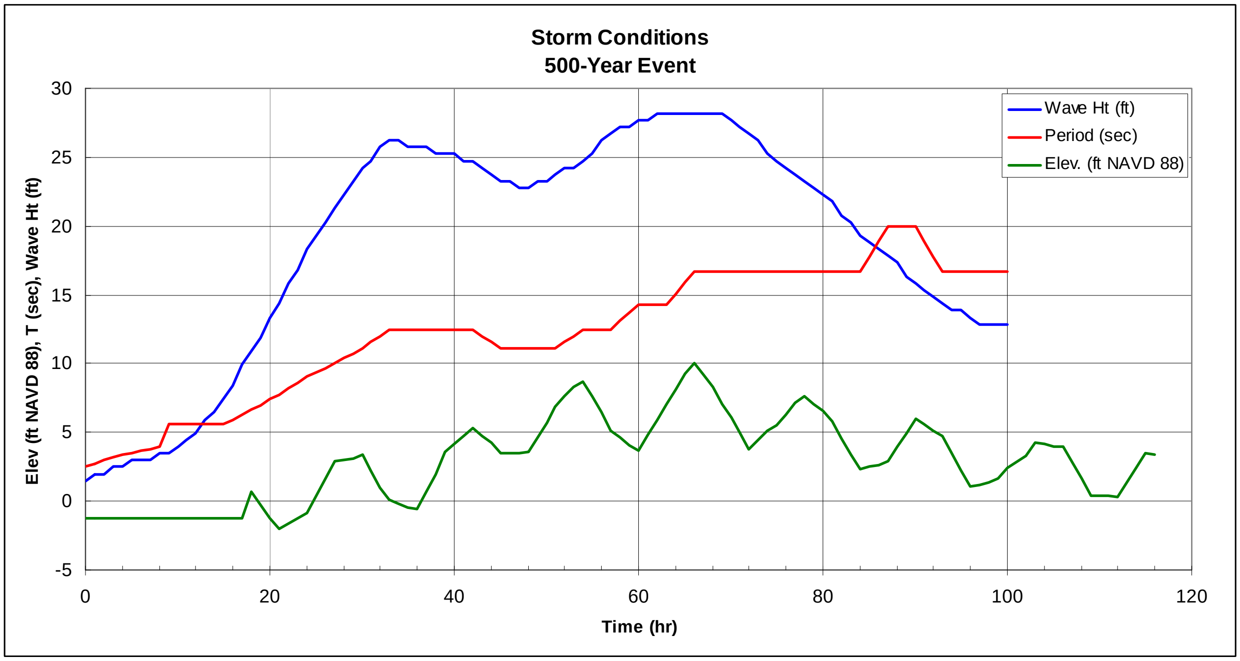


Figure 35. "500-yr" Storm Conditions used in Storm Damage Analysis

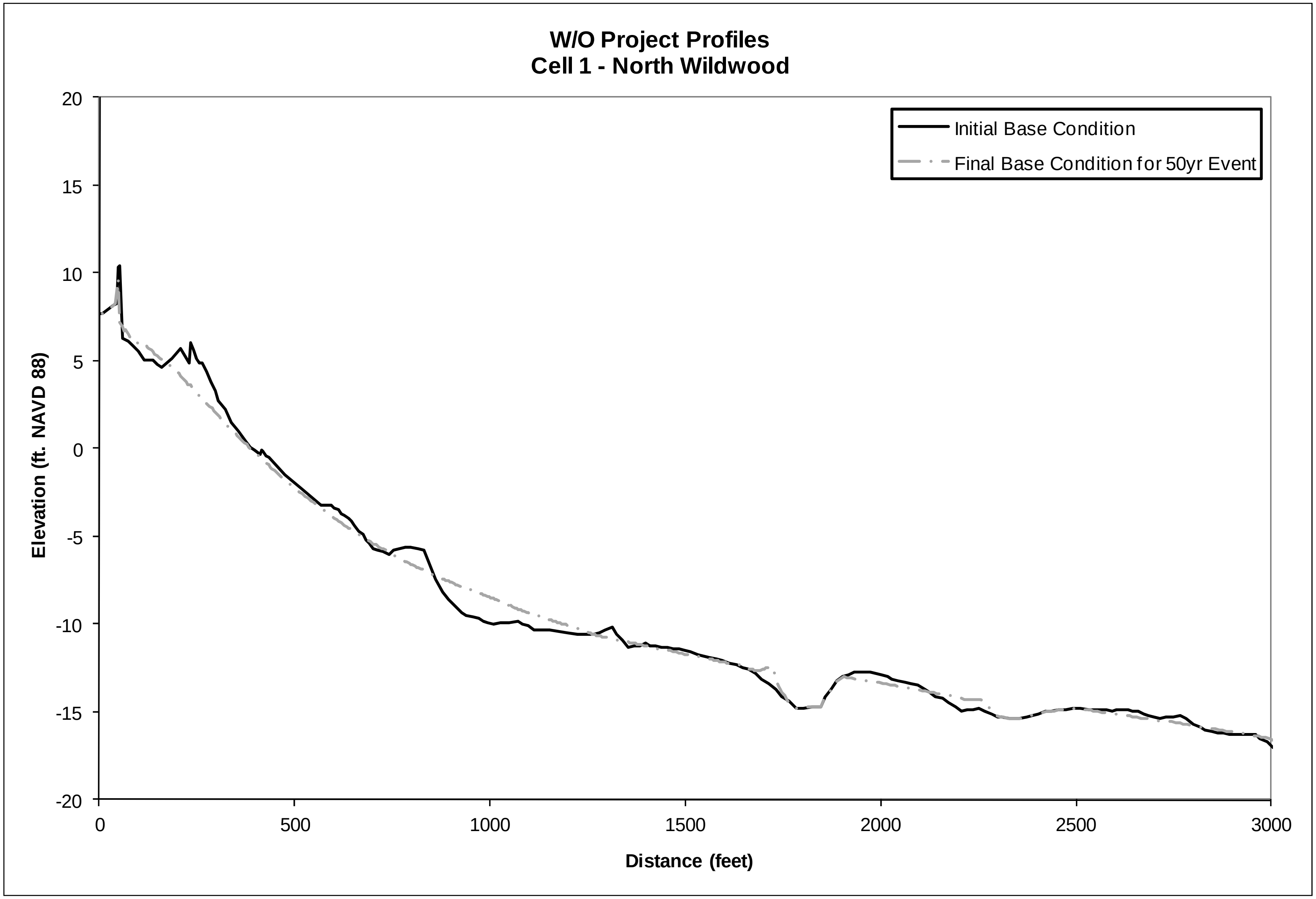


Figure 36. Pre- and Post "50-yr" Storm Beach Profiles for Without Base Conditions in Cell 1

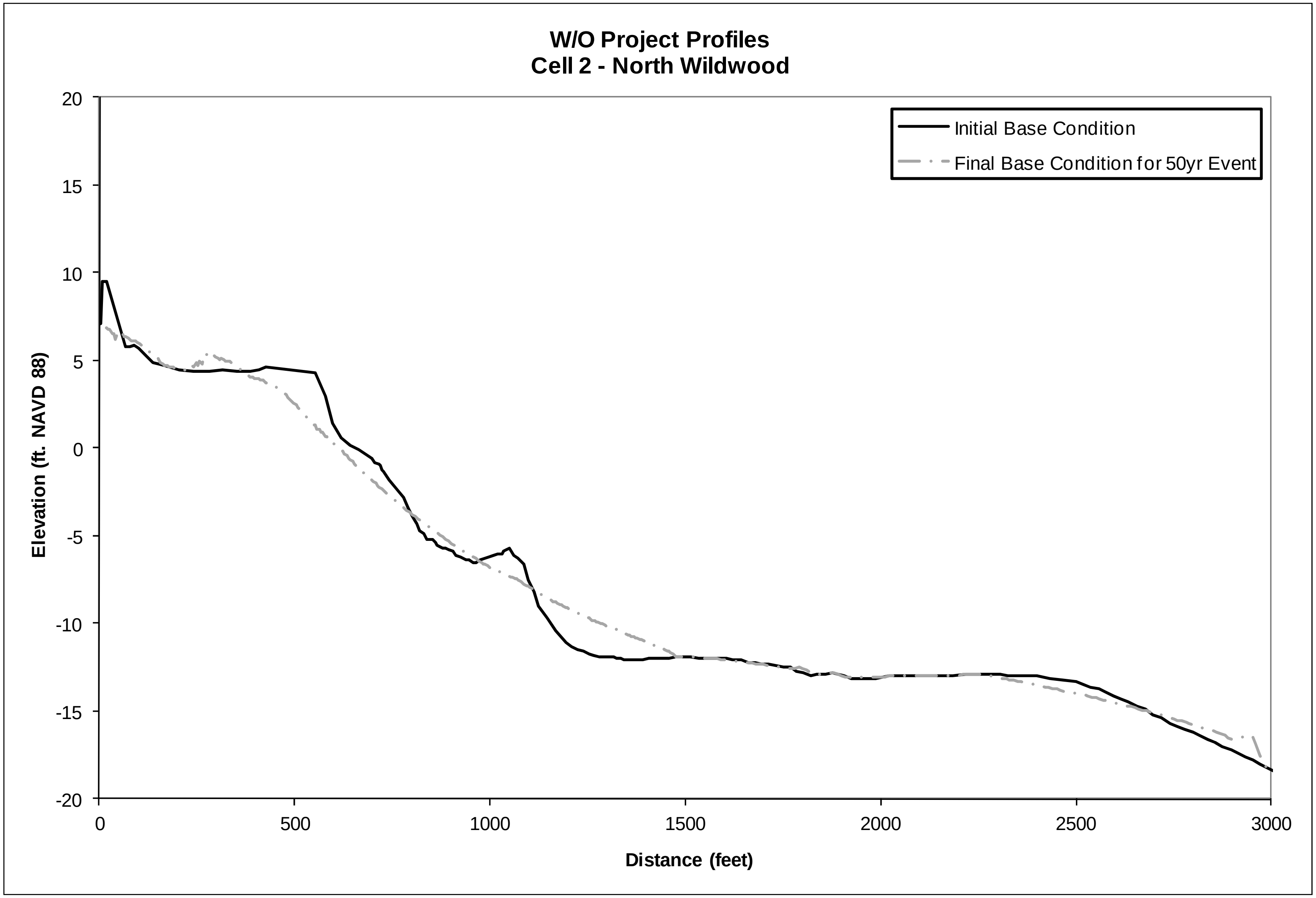


Figure 37. Pre- and Post "50-yr" Storm Beach Profiles for Without Base Conditions in Cell 2

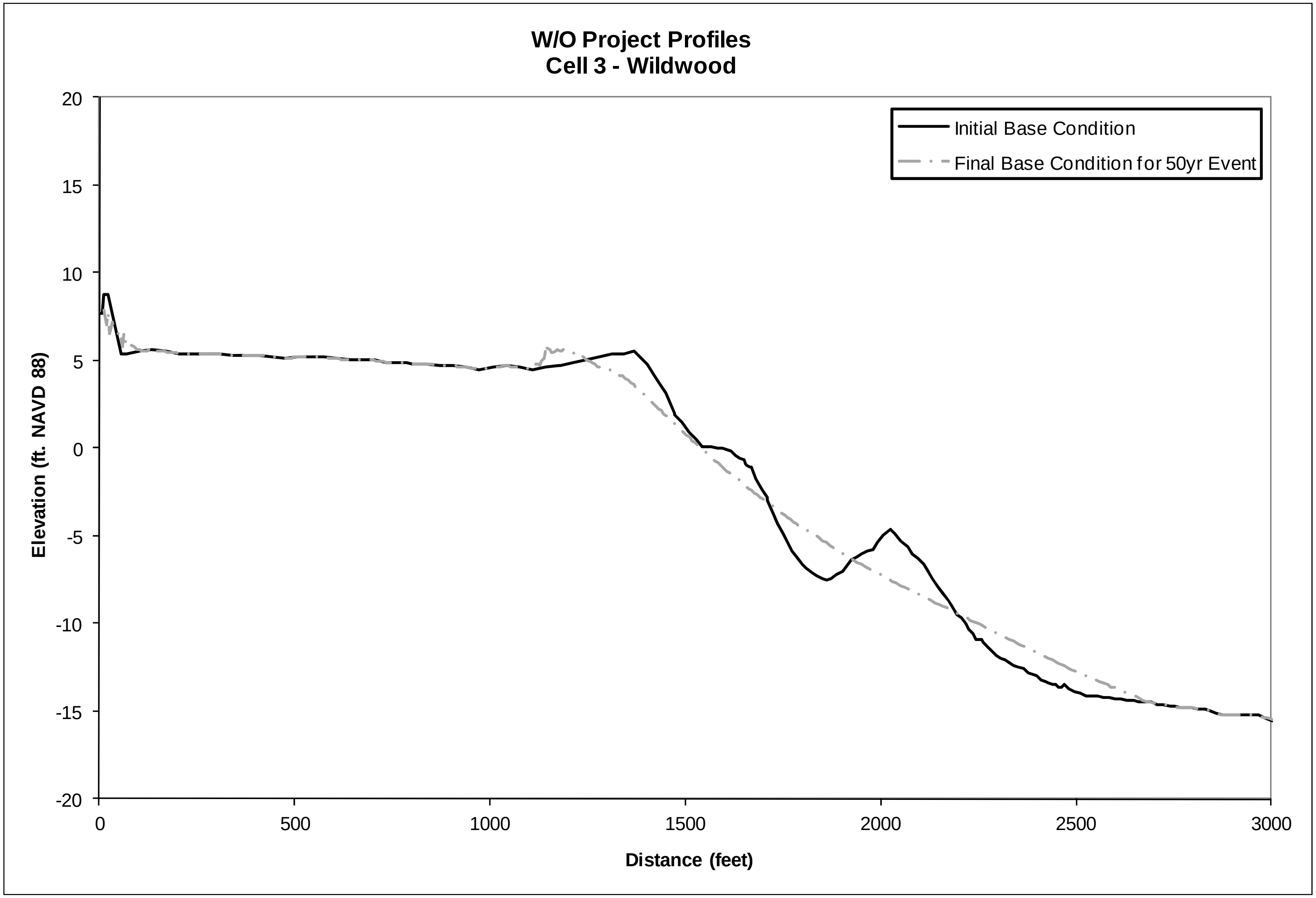


Figure 38. Pre- and Post "50-yr" Storm Beach Profiles for Without Base Conditions in Cell 3

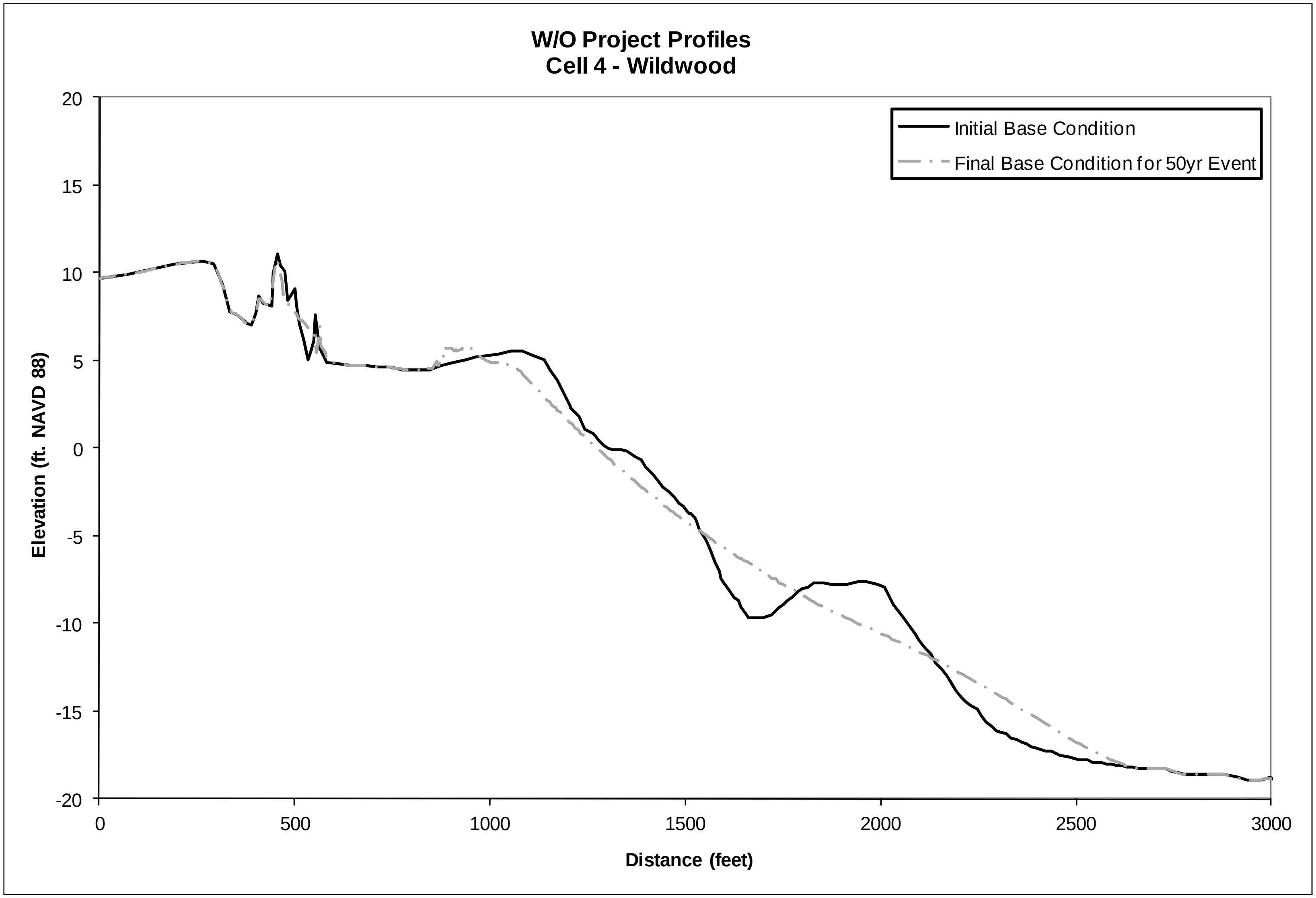


Figure 39. Pre- and Post "50-yr" Storm Beach Profiles for Without Base Conditions in Cell 4

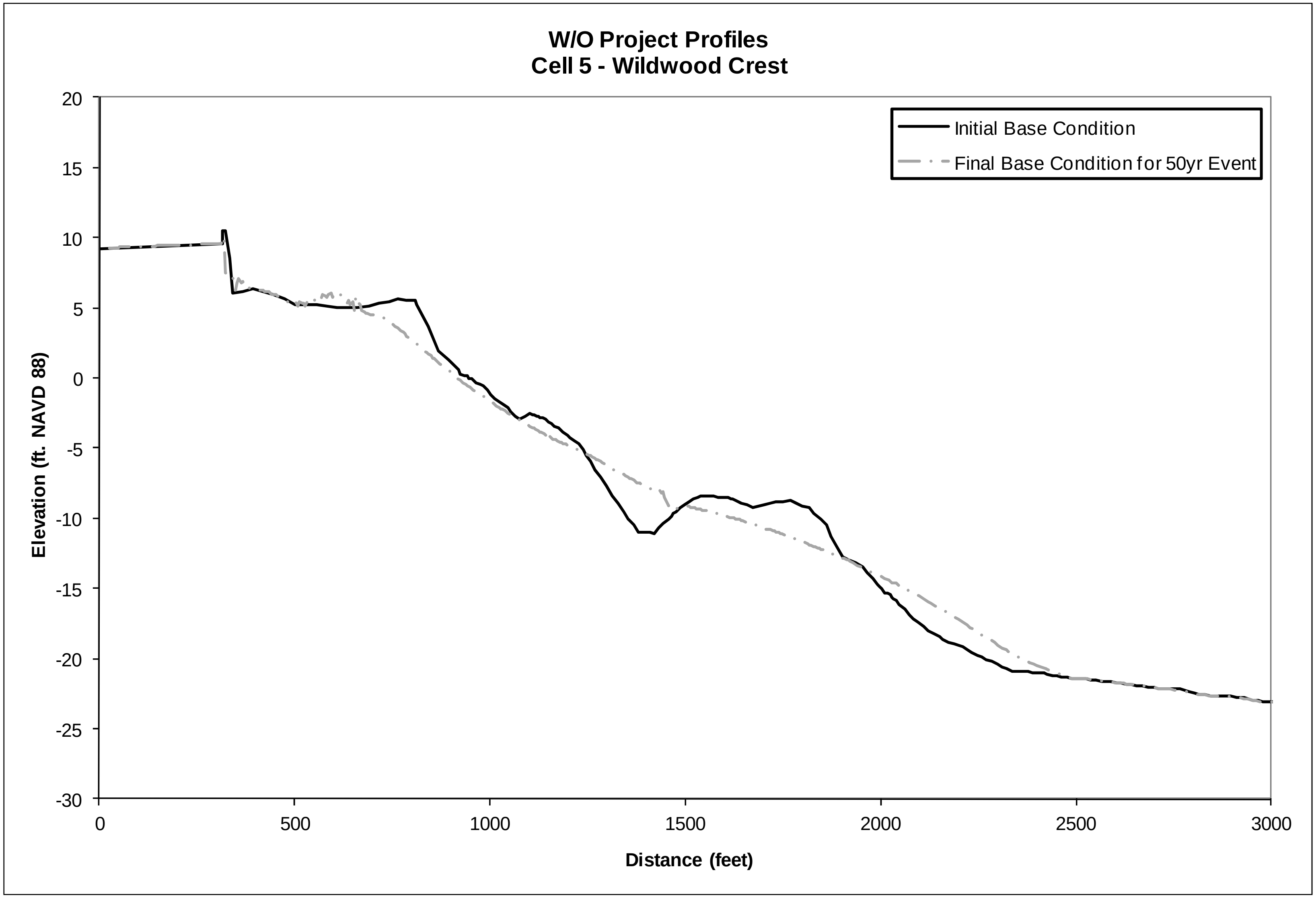


Figure 40. Pre- and Post "50-yr" Storm Beach Profiles for Without Base Conditions in Cell 6

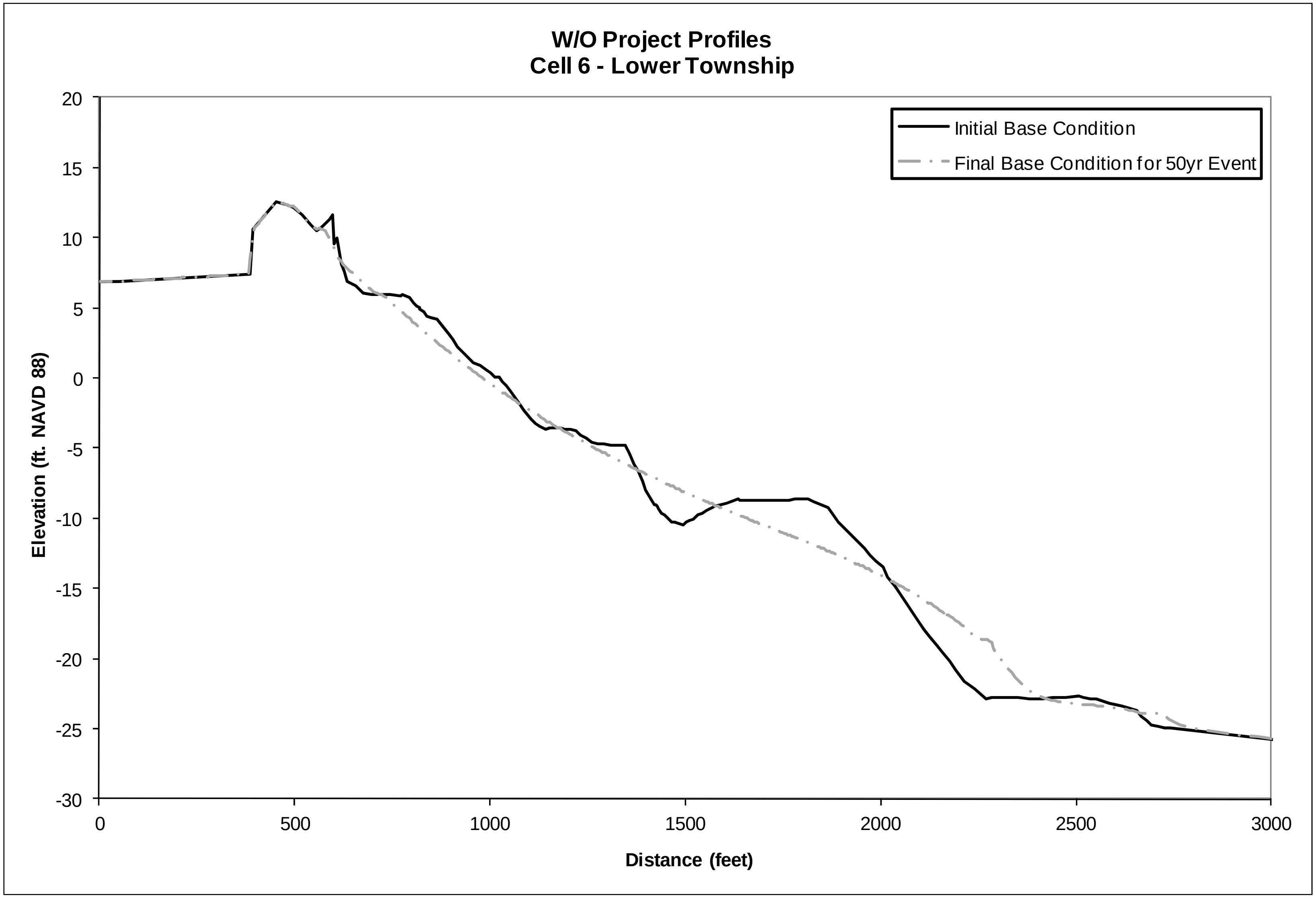


Figure 41. Pre- and Post "50-yr" Storm Beach Profiles for Without Base Conditions in Cell 6

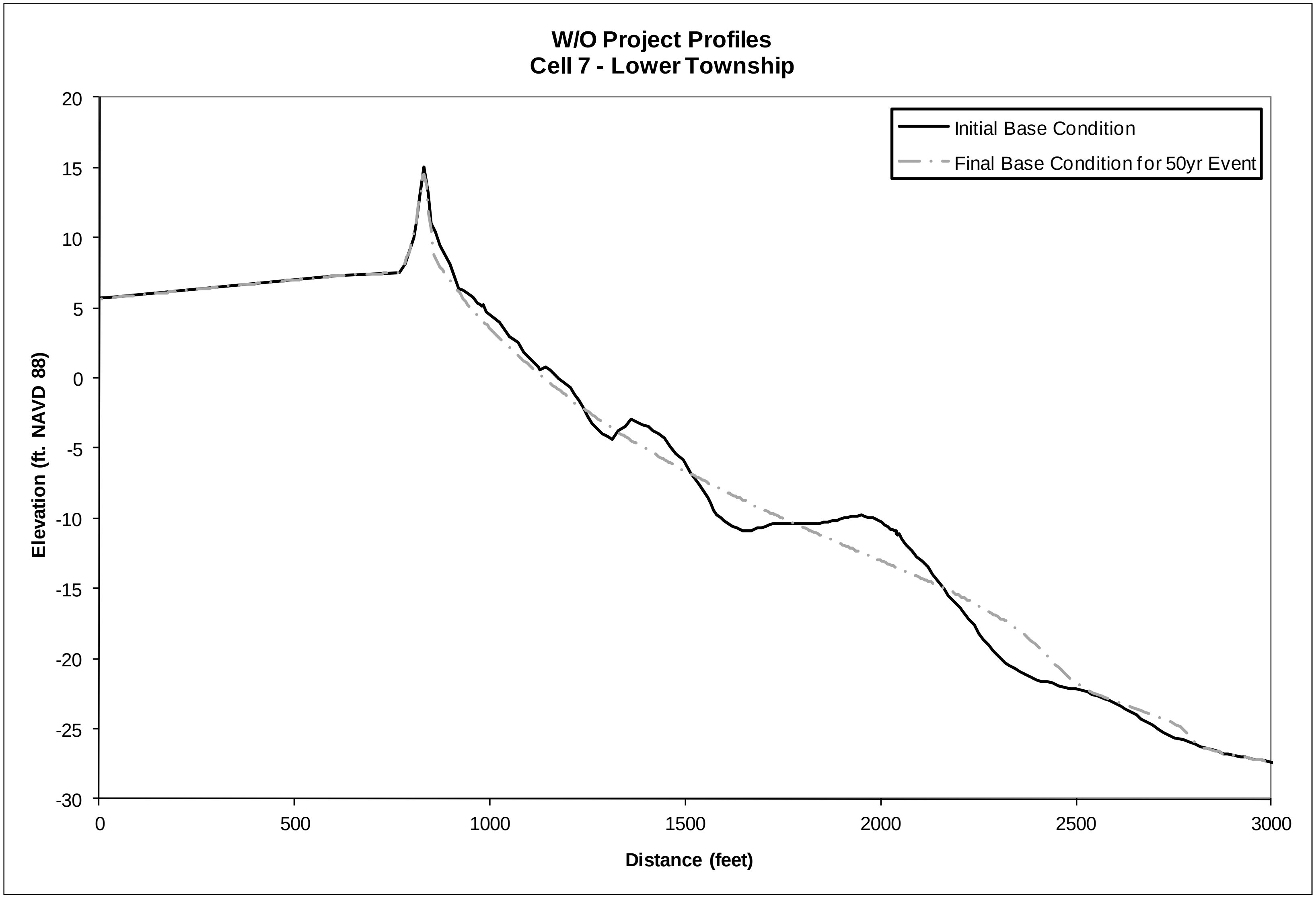


Figure 42. Pre- and Post "50-yr" Storm Beach Profiles for Without Base Conditions in Cell 7

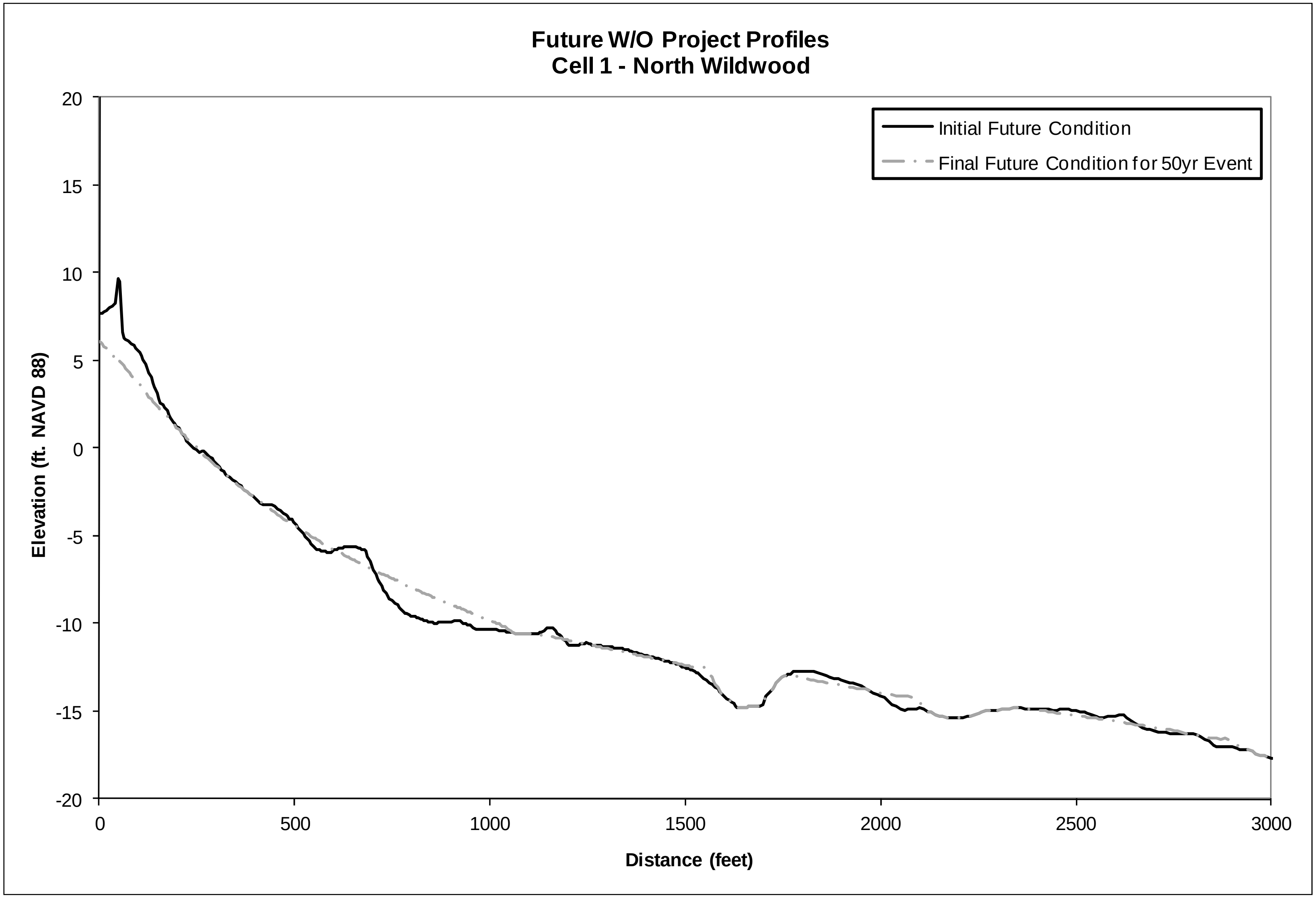


Figure 43. Pre- and Post "50-yr" Storm Beach Profiles for Without Future Conditions in Cell 1

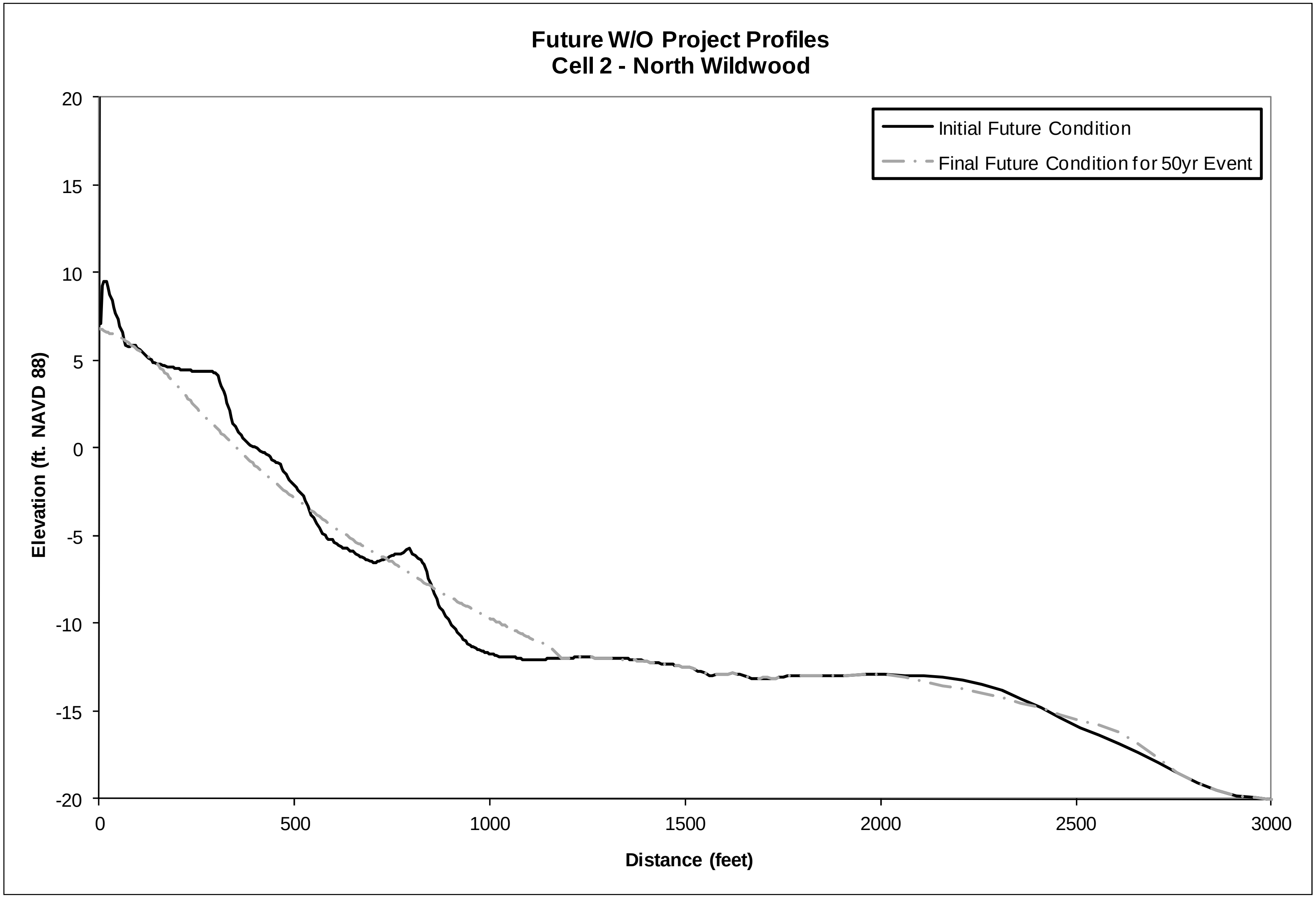


Figure 44. Pre- and Post "50-yr" Storm Beach Profiles for Without Future Conditions in Cell 2

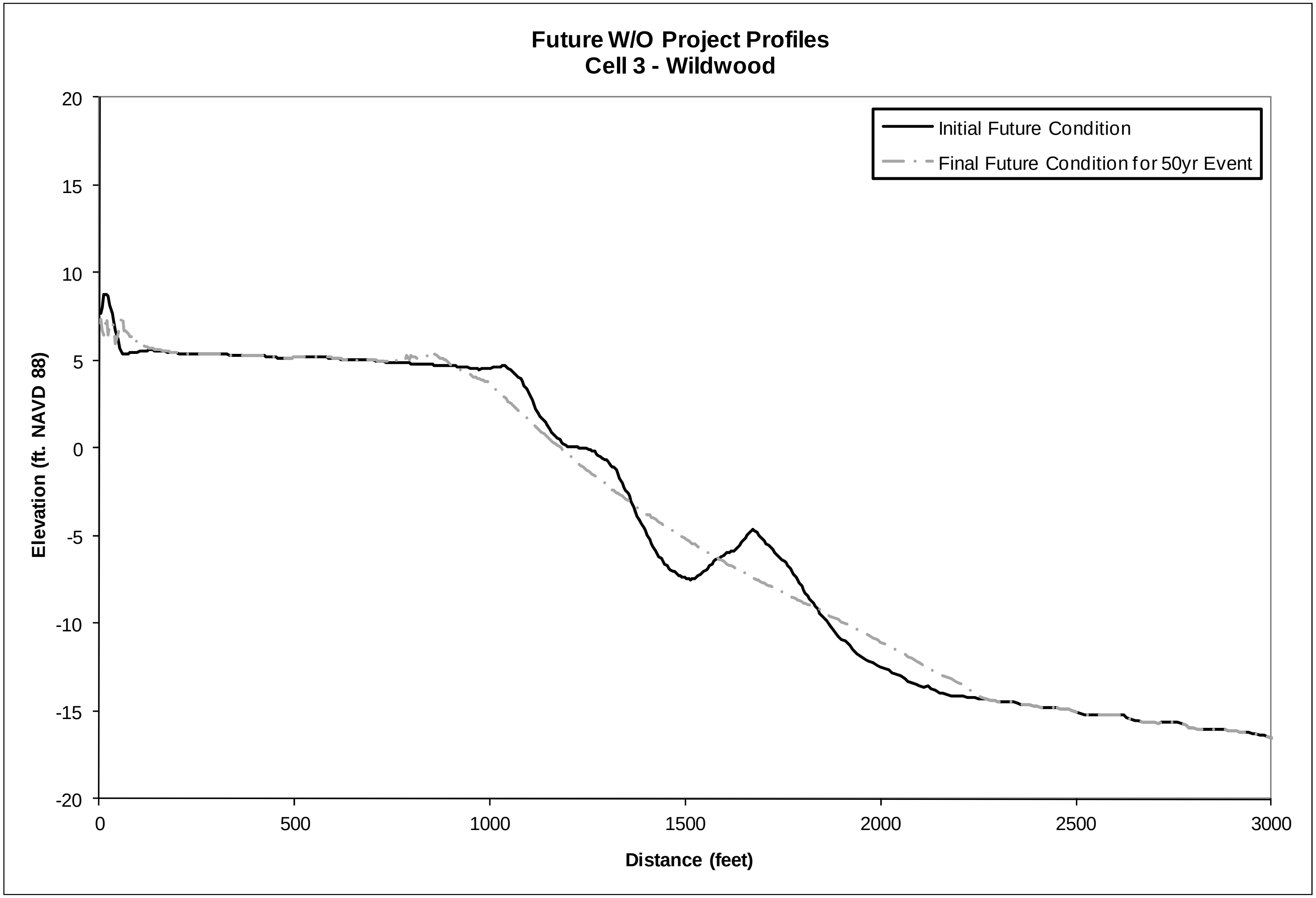


Figure 45. Pre- and Post "50-yr" Storm Beach Profiles for Without Future Conditions in Cell 3

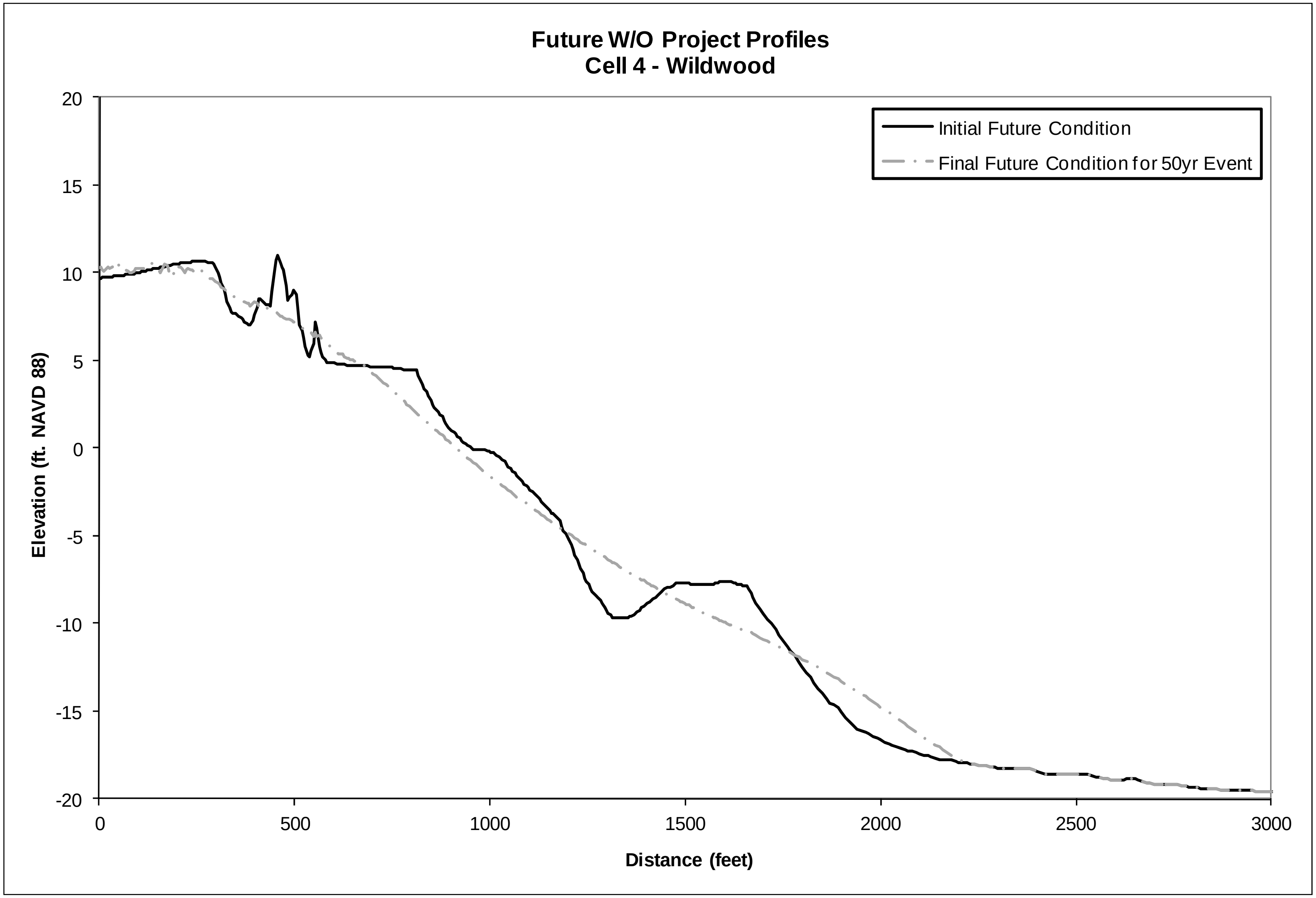


Figure 46. Pre- and Post "50-yr" Storm Beach Profiles for Without Future Conditions in Cell 4

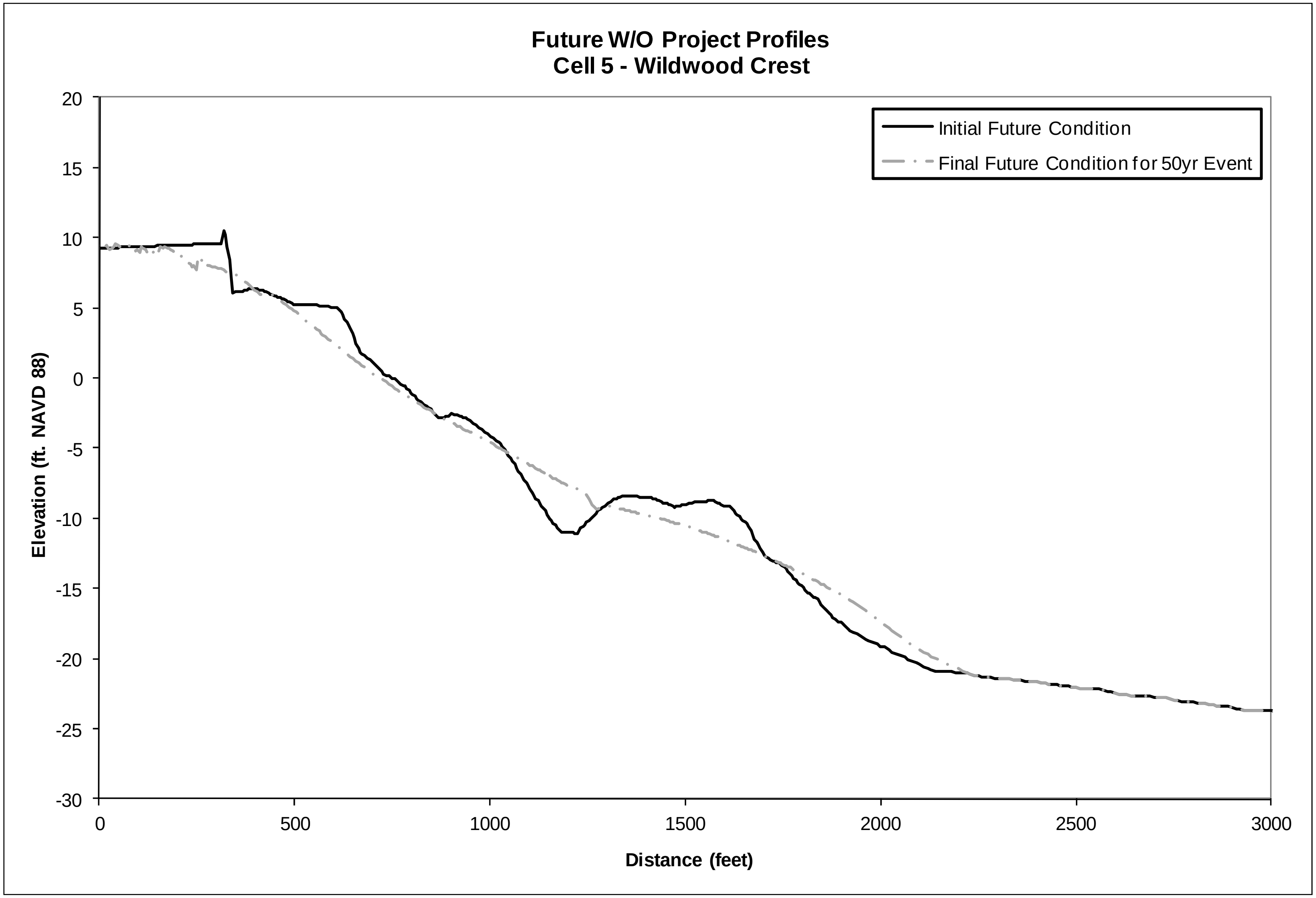


Figure 47. Pre- and Post "50-yr" Storm Beach Profiles for Without Future Conditions in Cell 5

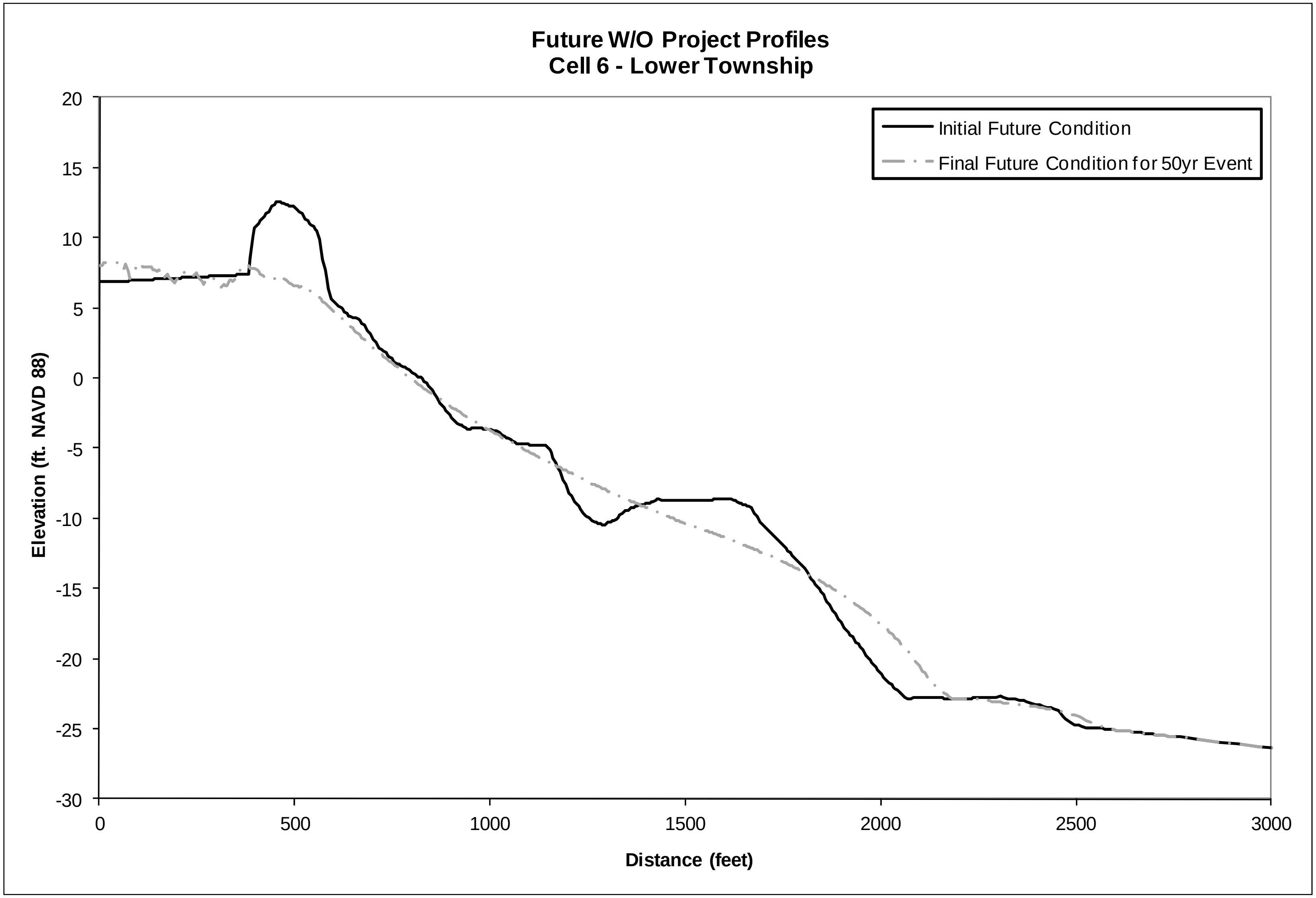


Figure 48. Pre- and Post "50-yr" Storm Beach Profiles for Without Future Conditions in Cell 6

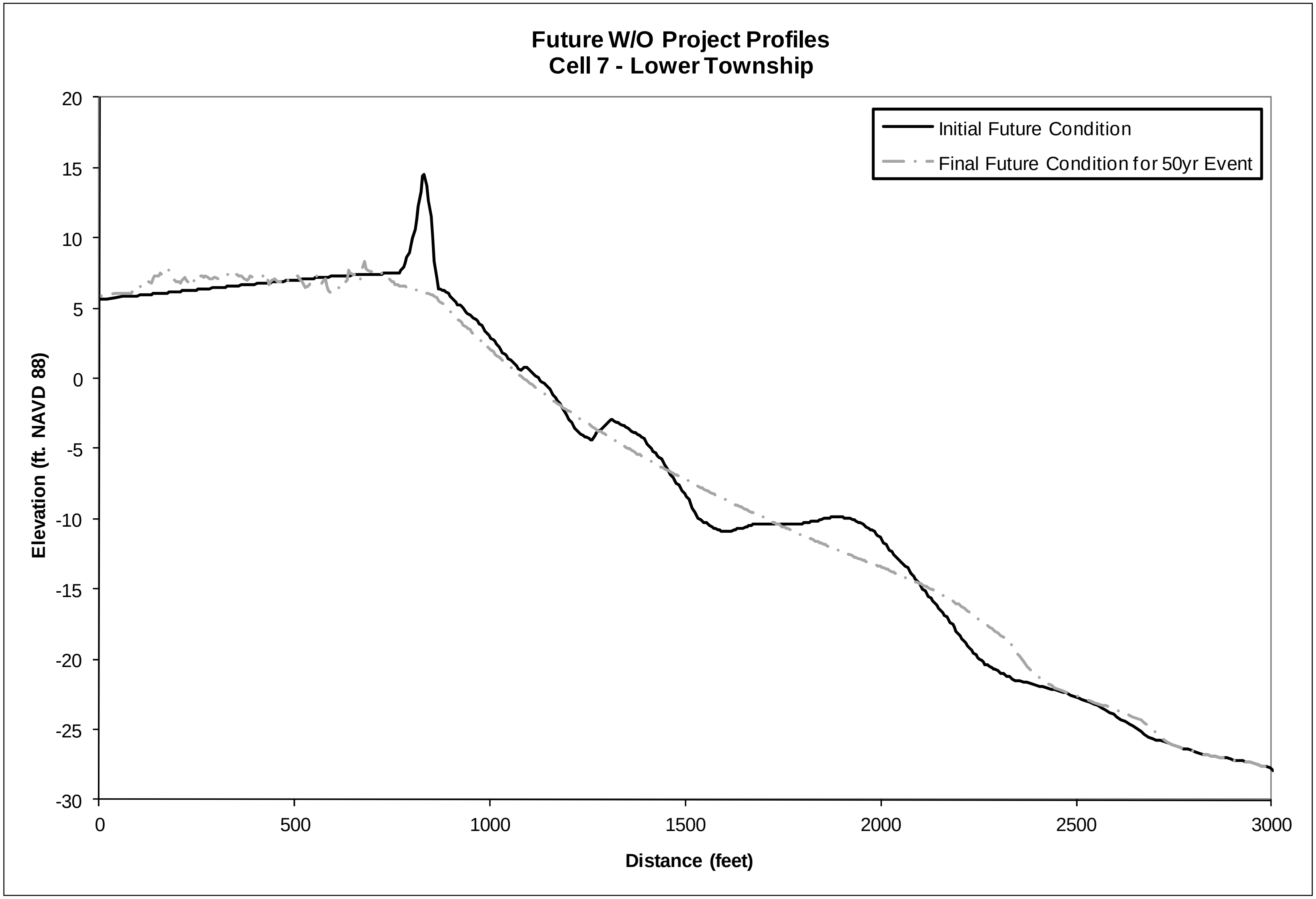


Figure 49. Pre- and Post "50-yr" Storm Beach Profiles for Without Future Conditions in Cell 7

Costdam file explanation.

Location of the 0.5 ft erosion depth for the 5-yr through 500-yr storm events

Location of the 3 foot wave impact zone for the 5-yr through 500-yr storm events

Inundation profiles shown as distance, elevation pairs for the 5-yr through 500-yr storm events

```
REACH LENGTH                    ME
1000.,0.0
DISTANCE SHORELINE TO REF                    EROSION RATE
0.0      0.0
DEFAULT PERSONAL PROPERTY %
0.25
STORM EROSION,WAVE IMPACT,WAVE DAMAGE ELEVATION, ZONES 1 & 2
5yr     0.0      .0     0.0     0.   0000.
10yr    0.0      .0     0.0     0.   0000.
20yr    0.0      .0     0.0     0.   0000.
50yr    0.0      .0     0.0     0.   0000.
100yr  45.0   955.0     0.0     0.   0000.
200yr  60.0   940.0     0.0     0.   0000.
500yr  65.0   935.0     0.0    00.   0000.
NUMBER OF POINTS IN WATER ELEVATION PROFILE
           7
 INUNDATION PROFILE:  DISTANCE FROM BASELINE AND TOTAL
WATERLEVEL
```

5yr	-200.0	10.7	-60.0	10.7	0.0	5.0	500.0	5.0
	700.0	5.0	800.0	5.0	1000.0	5.0		
10yr	-200.0	12.1	-30.0	11.8	0.0	5.5	500.0	5.5
	700.0	5.5	800.0	5.5	1000.0	5.5		
20yr	-200.0	12.4	-30.0	11.8	0.0	6.1	150.0	6.1
	250.0	6.1	500.0	6.1	1000.0	6.1		
50yr	-200.0	15.2	-5.0	13.5	45.0	8.9	150.0	8.0
	250.0	7.5	500.0	7.1	1000.0	7.1		
100yr	-200.0	16.9	-30.0	15.5	45.0	15.5	295.0	15.2
	545.0	14.2	795.0	11.1	1000.0	7.9		
200yr	-200.0	19.8	-20.0	17.8	60.0	17.8	310.0	17.3
	560.0	16.3	810.0	12.6	1000.0	8.9		
500yr	-200.0	22.6	-30.0	20.5	65.0	20.5	315.0	19.9
	565.0	18.9	780.0	14.4	1000.0	10.0		

Table 14. Without Project Conditions COSTDAM File for Cell 1

```
REACH LENGTH AND X-SECTION VOLUME
  3549,0.0
DISTANCE SHORELINE TO REF LINE / LONG TERM EROSION RATE
     0.0      0.0
DEFAULT PERSONAL PROPERTY PERCENTAGE
0.35
STORM EROSION,WAVE IMPACT,WAVE DAMAGE ELEVATION, ZONES 1 & 2
   1080.00       0.00       0.0     0.      0.
   1180.00       0.00       0.0     0.      0.
   1235.00       0.00       0.0     0.      0.
   1285.00       0.00       0.0     0.      0.
   1345.00     285.00       0.0     0.      0.
   1450.00    2190.00       0.0     0.      0.
   1535.00    2105.00       0.0     0.      0.
NUMBER OF POINTS IN WATER ELEVATION PROFILE
          10
INUNDATION PROFILE.  DISTANCE FROM BASELINE AND TOTAL WATERLEVEL
    0.00    9.20  650.00    9.20 1302.00    9.20 1307.00    0.00 1700.00    0.00
 2100.00    0.00 2500.00    0.00 2800.00    0.00 3200.00    0.00 3640.00    0.00
    0.00    9.40  650.00    9.40 1302.00    9.40 1307.00    0.00 1700.00    0.00
 2100.00    0.00 2500.00    0.00 2800.00    0.00 3200.00    0.00 3640.00    0.00
    0.00    9.70  650.00    9.70 1302.00    9.70 1307.00    0.00 1700.00    0.00
 2100.00    0.00 2500.00    0.00 2800.00    0.00 3200.00    0.00 3640.00    0.00
    0.00   11.50 1302.00   11.50 1352.00    8.70 1680.00    8.60 2008.00    8.60
 2336.00    8.30 2664.00    8.00 2992.00    7.10 3320.00    7.10 3640.00    7.10
    0.00   13.60  650.00   13.60 1302.00   13.60 1320.00   13.60 1335.00   13.60
 1352.00   13.60 1845.00   11.70 2339.00    9.80 2832.00    7.90 3640.00    7.90
    0.00   15.00  700.00   15.00 1420.00   15.00 1450.00   15.00 1470.00   15.00
 1492.00   15.00 1998.00   13.00 2505.00   10.90 3012.00    8.90 3640.00    8.90
    0.00   17.10  750.00   17.10 1490.00   17.10 1520.00   17.10 1550.00   17.10
 1583.00   17.10 2170.00   14.70 2758.00   12.40 3346.00   10.00 3640.00   10.00
```

Table 15. Without Project Conditions COSTDAM File for Cell 2

```
REACH LENGTH AND X-SECTION VOLUME
  2959,0.0
DISTANCE SHORELINE TO REF LINE / LONG TERM EROSION RATE
     0.0      0.0
```

```
DEFAULT PERSONAL PROPERTY PERCENTAGE
0.35
STORM EROSION,WAVE IMPACT,WAVE DAMAGE ELEVATION, ZONES 1 & 2
   1040.00       0.00       0.0     0.      0.
   1100.00       0.00       0.0     0.      0.
   1170.00       0.00       0.0     0.      0.
   1265.00       0.00       0.0     0.      0.
   1330.00     145.00       0.0     0.      0.
   1385.00    2170.00       0.0     0.      0.
   1440.00    2115.00       0.0     0.      0.
NUMBER OF POINTS IN WATER ELEVATION PROFILE
          10
INUNDATION PROFILE.  DISTANCE FROM BASELINE AND TOTAL WATERLEVEL
    0.00    8.60  650.00    8.60 1333.00    8.60 1338.00    0.00 1700.00    0.00
 2100.00    0.00 2500.00    0.00 2800.00    0.00 3200.00    0.00 3555.00    0.00
    0.00    8.90  650.00    8.90 1333.00    8.90 1338.00    0.00 1700.00    0.00
 2100.00    0.00 2500.00    0.00 2800.00    0.00 3200.00    0.00 3555.00    0.00
    0.00    9.30  650.00    9.30 1343.00    9.30 1348.00    0.00 1700.00    0.00
 2100.00    0.00 2500.00    0.00 2800.00    0.00 3200.00    0.00 3555.00    0.00
    0.00   10.80  650.00   10.80 1337.00   10.80 1580.00    9.80 1823.00    8.80
 2006.00    8.00 2316.00    7.80 2626.00    7.60 3250.00    7.10 3555.00    7.10
    0.00   14.20  650.00   14.20 1480.00   14.20 1500.00   14.20 1520.00   14.20
 1538.00   14.20 2051.00   12.10 2564.00   10.00 3077.00    7.90 3555.00    7.90
    0.00   15.90  650.00   15.90 1480.00   15.90 1500.00   15.90 1520.00   15.90
 1556.00   15.90 2128.00   13.60 2700.00   11.20 3272.00    8.90 3555.00    8.90
    0.00   18.10  650.00   18.10 1480.00   18.10 1500.00   18.10 1520.00   18.10
 1560.00   18.10 2072.00   16.10 2566.00   14.10 3061.00   12.10 3555.00   10.10
```

Table 16. Without Project Conditions COSTDAM File for Cell 3

```
REACH LENGTH AND X-SECTION VOLUME
  6965,0.0
DISTANCE SHORELINE TO REF LINE / LONG TERM EROSION RATE
     0.0      0.0
DEFAULT PERSONAL PROPERTY PERCENTAGE
0.35
STORM EROSION,WAVE IMPACT,WAVE DAMAGE ELEVATION, ZONES 1 & 2
     85.00       0.00       0.0     0.      0.
    250.00       0.00       0.0     0.      0.
    415.00       0.00       0.0     0.      0.
    665.00       0.00       0.0     0.      0.
    875.00       0.00       0.0     0.      0.
   1075.00    2245.00       0.0     0.      0.
   1375.00    1945.00       0.0     0.      0.
NUMBER OF POINTS IN WATER ELEVATION PROFILE
```

```
         10
INUNDATION PROFILE.  DISTANCE FROM BASELINE AND TOTAL WATERLEVEL
    0.00     7.90  650.00     7.90 1338.00     7.90 1343.00     0.00 1700.00     0.00
 2100.00     0.00 2500.00     0.00 2800.00     0.00 3200.00     0.00 3320.00     0.00
    0.00     8.30  650.00     8.30 1338.00     8.30 1343.00     0.00 1700.00     0.00
 2100.00     0.00 2500.00     0.00 2800.00     0.00 3200.00     0.00 3320.00     0.00
    0.00     8.60  650.00     8.60 1338.00     8.60 1343.00     0.00 1700.00     0.00
 2100.00     0.00 2500.00     0.00 2800.00     0.00 3200.00     0.00 3320.00     0.00
    0.00     9.30  650.00     9.30 1338.00     9.30 1343.00     0.00 1700.00     0.00
 2100.00     0.00 2500.00     0.00 2800.00     0.00 3200.00     0.00 3320.00     0.00
    0.00    13.60  650.00    13.60  900.00    13.60 1200.00    13.60 1300.00    13.60
 1388.00    13.60 1891.00    11.50 2399.00     9.50 2800.00     7.90 3320.00     7.90
    0.00    15.20  650.00    15.20  900.00    15.20 1200.00    15.20 1300.00    15.20
 1473.00    15.20 2040.00    12.90 2608.00    10.70 3050.00     8.90 3320.00     8.90
    0.00    17.20  650.00    17.20  900.00    17.20 1200.00    17.20 1300.00    17.20
 1528.00    17.20 1976.00    15.40 2400.00    13.60 3150.00    10.70 3320.00    10.00
```

Table 17. Without Project Conditions COSTDAM File for Cell 4

```
REACH LENGTH AND X-SECTION VOLUME
  4585,0.0
DISTANCE SHORELINE TO REF LINE / LONG TERM EROSION RATE
     0.0      0.0
DEFAULT PERSONAL PROPERTY PERCENTAGE
0.35
STORM EROSION,WAVE IMPACT,WAVE DAMAGE ELEVATION, ZONES 1 & 2
    325.00       0.00       0.0     0.      0.
    485.00       0.00       0.0     0.      0.
    655.00       0.00       0.0     0.      0.
    875.00       0.00       0.0     0.      0.
   1045.00       0.00       0.0     0.      0.
   1200.00    1915.00       0.0     0.      0.
   1415.00    1700.00       0.0     0.      0.
NUMBER OF POINTS IN WATER ELEVATION PROFILE
         10
INUNDATION PROFILE.  DISTANCE FROM BASELINE AND TOTAL WATERLEVEL
    0.00     8.00  650.00     8.00  893.00     8.00  898.00     0.00 1200.00     0.00
 1600.00     0.00 2000.00     0.00 2400.00     0.00 2800.00     0.00 3115.00     0.00
    0.00     8.60  650.00     8.60  893.00     8.60  898.00     0.00 1200.00     0.00
 1600.00     0.00 2000.00     0.00 2400.00     0.00 2800.00     0.00 3115.00     0.00
    0.00     9.20  650.00     9.20  893.00     9.20  898.00     0.00 1200.00     0.00
 1600.00     0.00 2000.00     0.00 2400.00     0.00 2800.00     0.00 3115.00     0.00
    0.00    10.20  650.00    10.20  898.00    10.20  903.00    10.70 1200.00     0.00
 1600.00     0.00 2000.00     0.00 2400.00     0.00 2800.00     0.00 3115.00     0.00
    0.00    12.30  650.00    12.30  900.00    12.30  920.00    12.30  940.00    12.30
```

```
  948.00   12.30 1368.00   10.80 1789.00    9.40 2210.00    7.90 3115.00    7.90
    0.00   14.60  650.00   14.60  900.00   14.60  920.00   14.60  940.00   14.60
 1168.00   14.60 1644.00   12.70 2120.00   10.80 2596.00    8.90 3115.00    8.90
    0.00   16.80  650.00   16.80  900.00   16.80  920.00   16.80  940.00   16.80
 1210.00   16.80 1783.00   14.50 2355.00   12.30 2928.00   10.00 3115.00   10.50
```

Table 18. Without Project Conditions COSTDAM File for Cell 5

```
REACH LENGTH AND X-SECTION VOLUME
  5835,0.0
DISTANCE SHORELINE TO REF LINE / LONG TERM EROSION RATE
     0.0      0.0
DEFAULT PERSONAL PROPERTY PERCENTAGE
0.35
STORM EROSION,WAVE IMPACT,WAVE DAMAGE ELEVATION, ZONES 1 & 2
    655.00       0.00       0.0     0.      0.
    715.00       0.00       0.0     0.      0.
    775.00       0.00       0.0     0.      0.
    870.00       0.00       0.0     0.      0.
   1275.00       0.00       0.0     0.      0.
   1400.00       0.00       0.0     0.      0.
   1475.00      45.00       0.0     0.      0.
NUMBER OF POINTS IN WATER ELEVATION PROFILE
          10
INUNDATION PROFILE.  DISTANCE FROM BASELINE AND TOTAL WATERLEVEL
    0.00    8.30  650.00    8.30 1029.00    8.30 1034.00    0.00 1400.00    0.00
 1800.00    0.00 2200.00    0.00 2600.00    0.00 3000.00    0.00 3315.00    0.00
    0.00    9.00  650.00    9.00 1034.00    9.00 1039.00    0.00 1400.00    0.00
 1800.00    0.00 2200.00    0.00 2600.00    0.00 3000.00    0.00 3315.00    0.00
    0.00    9.50  650.00    9.50 1034.00    9.50 1039.00    0.00 1400.00    0.00
 1800.00    0.00 2200.00    0.00 2600.00    0.00 3000.00    0.00 3315.00    0.00
    0.00   11.20 1034.00   11.20 1084.00   10.60 1402.00   10.20 1721.00    9.80
 2040.00    9.40 2359.00    9.10 2677.00    8.50 2996.00    8.10 3315.00    7.70
    0.00   13.10  650.00   13.10 1000.00   13.10 1020.00   13.10 1040.00   13.10
 1084.00   13.10 1545.00   11.70 2006.00   10.30 2467.00    8.90 3315.00    7.90
    0.00   14.70  650.00   14.70 1000.00   14.70 1020.00   14.70 1040.00   14.70
 1226.00   14.70 1697.00   12.80 2168.00   10.80 2640.00    8.90 3315.00    8.90
    0.00   17.40  650.00   17.40 1000.00   17.40 1200.00   17.40 1220.00   17.40
 1246.00   17.40 1871.00   14.90 2497.00   12.50 3123.00   10.00 3315.00   10.00
```

Table 19. Without Project Conditions COSTDAM File for Cell 6

```
REACH LENGTH AND X-SECTION VOLUME
  1090,0.0
DISTANCE SHORELINE TO REF LINE / LONG TERM EROSION RATE
     0.0      0.0
DEFAULT PERSONAL PROPERTY PERCENTAGE
0.35
STORM EROSION,WAVE IMPACT,WAVE DAMAGE ELEVATION, ZONES 1 & 2
    525.00       0.00       0.0     0.      0.
    575.00       0.00       0.0     0.      0.
    645.00       0.00       0.0     0.      0.
    730.00       0.00       0.0     0.      0.
    830.00       0.00       0.0     0.      0.
    970.00       0.00       0.0     0.      0.
   1145.00       0.00       0.0     0.      0.
NUMBER OF POINTS IN WATER ELEVATION PROFILE
          10
INUNDATION PROFILE.  DISTANCE FROM BASELINE AND TOTAL WATERLEVEL
    0.00    9.90  650.00    9.90  893.00    9.90  898.00    0.00 1200.00    0.00
 1600.00    0.00 2000.00    0.00 2400.00    0.00 2800.00    0.00 3150.00    0.00
    0.00   10.20  650.00   10.20  893.00   10.20  898.00    0.00 1200.00    0.00
 1600.00    0.00 2000.00    0.00 2400.00    0.00 2800.00    0.00 3150.00    0.00
    0.00   10.40  650.00   10.40  893.00   10.40  898.00    0.00 1200.00    0.00
 1600.00    0.00 2000.00    0.00 2400.00    0.00 2800.00    0.00 3150.00    0.00
    0.00   12.40  650.00   12.40  893.00   12.40  898.00    0.00 1200.00    0.00
 1600.00    0.00 2000.00    0.00 2400.00    0.00 2800.00    0.00 3150.00    0.00
    0.00   13.40  650.00   13.40  893.00   13.40  909.00   13.10  926.00   12.90
  943.00   12.60 1320.00   11.00 1698.00    9.50 2075.00    7.90 3150.00    7.90
    0.00   15.10  650.00   15.10  903.00   15.10  919.00   15.00  936.00   14.90
  953.00   14.70 1468.00   12.80 1983.00   10.90 2498.00    8.90 3150.00    8.90
    0.00   17.10  650.00   17.10 1000.00   17.10 1020.00   17.10 1040.00   17.10
 1088.00   17.10 1727.00   14.80 2367.00   12.40 3007.00   10.00 3150.00   10.00
```

Table 20. Without Project Conditions COSTDAM File for Cell 7

```
REACH LENGTH AND X-SECTION VOLUME
  6267,0.0
DISTANCE SHORELINE TO REF LINE / LONG TERM EROSION RATE
     0.0      0.0
DEFAULT PERSONAL PROPERTY PERCENTAGE
0.35
STORM EROSION,WAVE IMPACT,WAVE DAMAGE ELEVATION, ZONES 1 & 2
    110.00       0.00       0.0     0.      0.
    215.00       0.00       0.0     0.      0.
    310.00       0.00       0.0     0.      0.
    435.00       0.00       0.0     0.      0.
    535.00       0.00       0.0     0.      0.
    685.00       0.00       0.0     0.      0.
    925.00     630.00       0.0     0.      0.
NUMBER OF POINTS IN WATER ELEVATION PROFILE
          10
INUNDATION PROFILE.  DISTANCE FROM BASELINE AND TOTAL WATERLEVEL
    0.00    9.60  200.00    9.60  518.00    9.60  523.00    0.00  800.00    0.00
 1000.00    0.00 1100.00    0.00 1200.00    0.00 1400.00    0.00 1650.00    0.00
    0.00    9.90  200.00    9.90  518.00    9.90  523.00    0.00  800.00    0.00
 1000.00    0.00 1100.00    0.00 1200.00    0.00 1400.00    0.00 1650.00    0.00
    0.00   10.80  200.00   10.80  518.00   10.80  523.00    0.00  800.00    0.00
 1000.00    0.00 1100.00    0.00 1200.00    0.00 1400.00    0.00 1650.00    0.00
    0.00   12.80  200.00   12.80  518.00   12.80  523.00    0.00  800.00    0.00
 1000.00    0.00 1100.00    0.00 1200.00    0.00 1400.00    0.00 1650.00    0.00
    0.00   13.60  200.00   13.60  518.00   13.60  534.00   13.30  551.00   12.90
  568.00   12.60  838.00   11.50 1109.00   10.40 1379.00    9.30 1650.00    8.20
    0.00   15.30  200.00   15.30  300.00   15.30  400.00   15.30  500.00   15.30
  603.50   15.30  864.00   14.20 1126.00   13.20 1388.00   12.20 1650.00   11.20
    0.00   17.90  200.00   17.90  300.00   17.90  400.00   17.90  500.00   17.90
  683.00   17.90  924.00   17.20 1166.00   16.50 1408.00   15.90 1650.00   15.20
```

Table 21. Future Low Risk Without Project Conditions COSTDAM File for Cell 1

```
REACH LENGTH AND X-SECTION VOLUME
```

```
  3549,0.0
DISTANCE SHORELINE TO REF LINE / LONG TERM EROSION RATE
     0.0      0.0
DEFAULT PERSONAL PROPERTY PERCENTAGE
0.35
STORM EROSION,WAVE IMPACT,WAVE DAMAGE ELEVATION, ZONES 1 & 2
   1045.00       0.00       0.0     0.      0.
   1141.00       0.00       0.0     0.      0.
   1188.00       0.00       0.0     0.      0.
   1226.00       0.00       0.0     0.      0.
   1262.00      58.00       0.0     0.      0.
   1310.00     341.00       0.0     0.      0.
   1371.00    1250.00       0.0     0.      0.
NUMBER OF POINTS IN WATER ELEVATION PROFILE
          10
INUNDATION PROFILE.  DISTANCE FROM BASELINE AND TOTAL WATERLEVEL
    0.00    8.00  650.75    8.00 1301.50    8.00 1306.50    0.00 1695.42    0.00
 2084.33    0.00 2473.25    0.00 2862.17    0.00 3251.08    0.00 3640.00    0.00
    0.00    8.80  650.75    8.80 1301.50    8.80 1306.50    0.00 2062.67    0.00
 2818.83    0.00 3575.00    0.00 1350.00    0.00 2495.00    0.00 3640.00    0.00
    0.00    9.60  650.75    9.60 1301.50    9.60 1306.50    0.00 1969.33    0.00
 2632.17    0.00 3295.00    0.00 1350.00    0.00 2495.00    0.00 3640.00    0.00
    0.00   10.10  650.75   10.10 1301.50   10.10 1306.50    0.00 1769.33    0.00
 2232.17    0.00 2695.00    0.00 1350.00    0.00 2495.00    0.00 3640.00    0.00
    0.00   11.74 1302.00   11.74 1352.00    8.90 1680.00    8.80 2008.00    8.80
 2336.00    8.50 2664.00    8.20 2992.00    7.90 3320.00    7.90 3640.00    7.90
    0.00   12.70  680.25   12.70 1360.50   12.70 1377.17   12.70 1393.83   12.70
 1410.50   12.70 1725.90   11.44 2041.30   10.17 2356.70    8.90 3640.00    8.90
    0.00   14.18  702.85   14.18 1405.70   14.18 1430.47   14.18 1455.23   14.18
 1480.00   14.18 1828.00   12.78 2176.00   11.39 2524.00   10.00 3640.00   10.00
```

Table 22. Future Low Risk Without Project Conditions COSTDAM File for Cell 2

```
REACH LENGTH AND X-SECTION VOLUME
  2959,0.0
DISTANCE SHORELINE TO REF LINE / LONG TERM EROSION RATE
     0.0      0.0
DEFAULT PERSONAL PROPERTY PERCENTAGE
0.35
STORM EROSION,WAVE IMPACT,WAVE DAMAGE ELEVATION, ZONES 1 & 2
   1002.00       0.00       0.0     0.      0.
   1059.00       0.00       0.0     0.      0.
   1121.00       0.00       0.0     0.      0.
   1208.00       0.00       0.0     0.      0.
   1256.00       0.00       0.0     0.      0.
```

```
   1297.00      55.00       0.0     0.      0.
   1329.00     340.00       0.0     0.      0.
NUMBER OF POINTS IN WATER ELEVATION PROFILE
          10
INUNDATION PROFILE.  DISTANCE FROM BASELINE AND TOTAL WATERLEVEL
    0.00    8.00  666.25    8.00 1332.50    8.00 1337.50    0.00 1707.08    0.00
 2076.67    0.00 2446.25    0.00 2815.83    0.00 3185.42    0.00 3555.00    0.00
    0.00    8.20  666.25    8.20 1332.50    8.20 1337.50    0.00 1707.08    0.00
 2076.67    0.00 2446.25    0.00 2815.83    0.00 3185.42    0.00 3555.00    0.00
    0.00    8.50  666.25    8.50 1332.50    8.50 1337.50    0.00 1821.67    0.00
 2305.83    0.00 2790.00    0.00 1350.00    0.00 2452.50    0.00 3555.00    0.00
    0.00   10.10  666.25   10.10 1337.00   10.10 1580.00    9.10 1823.00    8.00
 1920.00    7.60 2200.00    7.40 2400.00    7.30 2700.00    7.10 3555.00    7.10
    0.00   12.06  721.85   12.06 1443.70   12.06 1460.37   12.06 1477.03   12.06
 1493.70   12.06 1826.80   10.67 2159.90    9.29 2493.00    7.90 3555.00    7.90
    0.00   12.82  727.00   12.82 1454.00   12.82 1470.67   12.82 1487.33   12.82
 1504.00   12.82 1815.90   11.51 2127.80   10.21 2439.70    8.90 3555.00    8.90
    0.00   15.20  730.35   15.20 1460.70   15.20 1477.37   15.20 1494.03   15.20
 1510.70   15.20 1930.63   13.47 2350.57   11.73 2770.50   10.00 3555.00   10.00
```

Table 23. Future Low Risk Without Project Conditions COSTDAM File for Cell 3

```
REACH LENGTH AND X-SECTION VOLUME
  6965,0.0
DISTANCE SHORELINE TO REF LINE / LONG TERM EROSION RATE
     0.0      0.0
DEFAULT PERSONAL PROPERTY PERCENTAGE
0.35
STORM EROSION,WAVE IMPACT,WAVE DAMAGE ELEVATION, ZONES 1 & 2
      0.00       0.00       0.0     0.      0.
     70.00       0.00       0.0     0.      0.
    205.00       0.00       0.0     0.      0.
    424.00       0.00       0.0     0.      0.
    508.00       0.00       0.0     0.      0.
    590.00     838.00       0.0     0.      0.
    842.00     956.00       0.0     0.      0.
NUMBER OF POINTS IN WATER ELEVATION PROFILE
          10
INUNDATION PROFILE.  DISTANCE FROM BASELINE AND TOTAL WATERLEVEL
    0.00    6.70  668.75    6.70 1337.50    6.70 1342.50    0.00 1672.08    0.00
 2001.67    0.00 2331.25    0.00 2660.83    0.00 2990.42    0.00 3320.00    0.00
    0.00    7.00  668.75    7.00 1337.50    7.00 1342.50    0.00 1672.08    0.00
 2001.67    0.00 2331.25    0.00 2660.83    0.00 2990.42    0.00 3320.00    0.00
    0.00    7.10  668.75    7.10 1337.50    7.10 1342.50    0.00 1672.08    0.00
 2001.67    0.00 2331.25    0.00 2660.83    0.00 2990.42    0.00 3320.00    0.00
```

```
   0.00    8.40  668.75    8.40 1337.50    8.40 1342.50    0.00 1846.67    0.00
 2350.83    0.00 2855.00    0.00 1350.00    0.00 2335.00    0.00 3320.00    0.00
   0.00   12.00  668.75   12.00 1337.50   12.00 1354.17   12.00 1370.83   12.00
 1387.50   12.00 1712.50   10.63 2037.50    9.27 2362.50    7.90 3320.00    7.90
   0.00   13.12  668.75   13.12 1337.50   13.12 1364.17   13.12 1390.83   13.12
 1417.50   13.12 1768.83   11.71 2120.17   10.31 2471.50    8.90 3320.00    8.90
   0.00   14.69  708.75   14.69 1417.50   14.69 1434.17   14.69 1450.83   14.69
 1467.50   14.69 1856.50   13.13 2245.50   11.56 2634.50   10.00 3320.00   10.00
```

Table 24. Future Low Risk Without Project Conditions COSTDAM File for Cell 4

```
REACH LENGTH AND X-SECTION VOLUME
  4585,0.0
DISTANCE SHORELINE TO REF LINE / LONG TERM EROSION RATE
     0.0      0.0
DEFAULT PERSONAL PROPERTY PERCENTAGE
0.35
STORM EROSION,WAVE IMPACT,WAVE DAMAGE ELEVATION, ZONES 1 & 2
    247.00       0.00       0.0     0.      0.
    349.00       0.00       0.0     0.      0.
    455.00       0.00       0.0     0.      0.
    593.00       0.00       0.0     0.      0.
    691.00       0.00       0.0     0.      0.
    791.00       0.00       0.0     0.      0.
    909.00    2206.00       0.0     0.      0.
NUMBER OF POINTS IN WATER ELEVATION PROFILE
          10
INUNDATION PROFILE.  DISTANCE FROM BASELINE AND TOTAL WATERLEVEL
   0.00    7.60  432.25    7.60  864.50    7.60  869.50    0.00 1243.75    0.00
 1618.00    0.00 1992.25    0.00 2366.50    0.00 2740.75    0.00 3115.00    0.00
   0.00    8.00  432.25    8.00  864.50    8.00  869.50    0.00 1243.75    0.00
 1618.00    0.00 1992.25    0.00 2366.50    0.00 2740.75    0.00 3115.00    0.00
   0.00    8.50  432.25    8.50  864.50    8.50  869.50    0.00 1243.75    0.00
 1618.00    0.00 1992.25    0.00 2366.50    0.00 2740.75    0.00 3115.00    0.00
   0.00    9.90  434.25    9.90  868.50    9.90  873.50    0.00 1545.67    0.00
 2217.83    0.00 2890.00    0.00 1350.00    0.00 2232.50    0.00 3115.00    0.00
   0.00   10.50  528.75   10.50 1057.50   10.50 1062.50    0.00 1425.00    0.00
 1787.50    0.00 2150.00    0.00 1350.00    0.00 2232.50    0.00 3115.00    0.00
   0.00   12.52  528.75   12.52 1057.50   12.52 1074.17   12.52 1090.83   12.52
 1107.50   12.52 1399.33   11.31 1691.17   10.11 1983.00    8.90 3115.00    8.90
   0.00   14.70  539.00   14.70 1078.00   14.70 1094.67   14.70 1111.33   14.70
 1128.00   14.70 1517.00   13.14 1906.00   11.57 2295.00   10.00 3115.00   10.00
```

Table 25. Future Low Risk Without Project Conditions COSTDAM File for Cell 5

```
REACH LENGTH AND X-SECTION VOLUME
  5835,0.0
DISTANCE SHORELINE TO REF LINE / LONG TERM EROSION RATE
     0.0      0.0
DEFAULT PERSONAL PROPERTY PERCENTAGE
0.35
STORM EROSION,WAVE IMPACT,WAVE DAMAGE ELEVATION, ZONES 1 & 2
    355.00       0.00       0.0     0.      0.
    450.00       0.00       0.0     0.      0.
    495.00       0.00       0.0     0.      0.
    550.00       0.00       0.0     0.      0.
   1010.00       0.00       0.0     0.      0.
   1175.00       0.00       0.0     0.      0.
   1290.00       0.00       0.0     0.      0.
NUMBER OF POINTS IN WATER ELEVATION PROFILE
          10
INUNDATION PROFILE.  DISTANCE FROM BASELINE AND TOTAL WATERLEVEL
    0.00    7.60  514.25    7.60 1028.50    7.60 1033.50    0.00 1413.75    0.00
 1794.00    0.00 2174.25    0.00 2554.50    0.00 2934.75    0.00 3315.00    0.00
    0.00    8.30  514.25    8.30 1033.50    8.30 1038.50    0.00 1061.83    0.00
 1078.50    0.00 1521.83    0.00 1965.17    0.00 2408.50    0.00 3315.00    0.00
    0.00    8.60  516.75    8.60 1033.50    8.60 1038.50    0.00 1066.83    0.00
 1083.50    0.00 1516.83    0.00 1950.17    0.00 2383.50    0.00 3315.00    0.00
    0.00   10.10 1038.50   10.10 1043.50    0.00 1402.00    0.00 1721.00    0.00
 2040.00    0.00 2359.00    0.00 2677.00    0.00 2996.00    0.00 3315.00    0.00
    0.00   12.20  592.50   12.20  700.00   12.20  800.67   12.20 1000.33   12.20
 1084.00   12.20 1545.00   10.80 2006.00    9.60 2820.00    7.90 3315.00    7.90
    0.00   13.30  657.50   13.30 1015.00   13.30 1131.67   13.30 1248.33   13.30
 1250.00   13.30 1650.00   11.90 2480.00    8.90 2640.00    8.90 3315.00    8.90
    0.00   15.80  667.50   15.80 1335.00   15.80 1368.33   15.80 1401.67   15.80
 1435.00   15.80 1871.00   13.40 2720.00   10.00 3123.00   10.00 3315.00   10.00
```

Table 26. Future Low Risk Without Project Conditions COSTDAM File for Cell 6

```
REACH LENGTH AND X-SECTION VOLUME
  1090,0.0
DISTANCE SHORELINE TO REF LINE / LONG TERM EROSION RATE
     0.0      0.0
DEFAULT PERSONAL PROPERTY PERCENTAGE
0.35
STORM EROSION,WAVE IMPACT,WAVE DAMAGE ELEVATION, ZONES 1 & 2
    468.00       0.00       0.0     0.      0.
    509.00       0.00       0.0     0.      0.
    567.00       0.00       0.0     0.      0.
    621.00       0.00       0.0     0.      0.
    697.00       0.00       0.0     0.      0.
    803.00       0.00       0.0     0.      0.
    949.00       0.00       0.0     0.      0.
NUMBER OF POINTS IN WATER ELEVATION PROFILE
          10
INUNDATION PROFILE.  DISTANCE FROM BASELINE AND TOTAL WATERLEVEL
    0.00     9.30  437.25    9.30  874.50    9.30  879.50    0.00 1257.92    0.00
 1636.33     0.00 2014.75    0.00 2393.17    0.00 2771.58    0.00 3150.00    0.00
    0.00     9.60  437.25    9.60  874.50    9.60  879.50    0.00 1594.67    0.00
 2309.83     0.00 3025.00    0.00 1350.00    5.50 2250.00    5.50 3150.00    5.50
    0.00     9.80  437.25    9.80  874.50    9.80  879.50    0.00 1528.00    0.00
 2176.50     0.00 2825.00    0.00 1350.00    6.10 2250.00    6.10 3150.00    6.10
    0.00    11.50  437.25   11.50  874.50   11.50  879.50    0.00 1228.00    0.00
 1576.50     0.00 1925.00    0.00 1350.00    7.10 2250.00    7.10 3150.00    7.10
    0.00    12.30  440.25   12.30  880.50   12.30  885.50    0.00 1096.17    0.00
 1306.83     0.00 1517.50    0.00 1350.00    7.90 2250.00    7.90 3150.00    7.90
    0.00    13.70  440.25   13.70  880.50   13.70  897.17   13.17  913.83   12.64
  930.50    12.11 1219.17   11.04 1507.83    9.97 1796.50    8.90 3150.00    8.90
    0.00    14.52  497.25   14.52  994.50   14.52 1011.17   14.52 1027.83   14.52
 1044.50    14.52 1454.50   13.01 1864.50   11.51 2274.50   10.00 3150.00   10.00
```

Table 27. Future Low Risk Without Project Conditions COSTDAM File for Cell 7

```
REACH LENGTH AND X-SECTION VOLUME
  6267,0.0
DISTANCE SHORELINE TO REF LINE / LONG TERM EROSION RATE
     0.0      0.0
DEFAULT PERSONAL PROPERTY PERCENTAGE
0.35
STORM EROSION,WAVE IMPACT,WAVE DAMAGE ELEVATION, ZONES 1 & 2
     78.00       0.00       0.0     0.      0.
    146.00       0.00       0.0     0.      0.
    237.00       0.00       0.0     0.      0.
    332.00       0.00       0.0     0.      0.
    399.00       0.00       0.0     0.      0.
    506.00       0.00       0.0     0.      0.
    716.00       0.00       0.0     0.      0.
NUMBER OF POINTS IN WATER ELEVATION PROFILE
          10
INUNDATION PROFILE.  DISTANCE FROM BASELINE AND TOTAL WATERLEVEL
    0.00     9.10  258.75     9.10  517.50     9.10  522.50     0.00  710.42     0.00
  898.33     0.00 1086.25     0.00 1274.17     0.00 1462.08     0.00 1650.00     0.00
    0.00     9.30  258.75     9.30  517.50     9.30  522.50     0.00  830.00     0.00
 1137.50     0.00 1445.00     0.00 1350.00     0.00 1500.00     0.00 1650.00     0.00
    0.00    10.10  258.75    10.10  517.50    10.10  522.50     0.00  776.67     0.00
 1030.83     0.00 1285.00     0.00 1350.00     0.00 1500.00     0.00 1650.00     0.00
    0.00    11.90  258.75    11.90  517.50    11.90  522.50     0.00  750.83     0.00
  979.17     0.00 1207.50     0.00 1350.00     0.00 1500.00     0.00 1650.00     0.00
    0.00    12.50  200.00    12.50  300.00    12.50  350.00    12.50  375.00    12.50
  553.50    12.50  838.00    11.17 1109.00    10.10 1600.00     7.90 1650.00     7.90
    0.00    13.41  262.25    13.41  524.50    13.41  541.17    13.41  557.83    13.41
  574.50    13.41  843.38    12.34 1112.25    11.28 1381.13    10.21 1650.00     9.14
    0.00    15.13  291.75    15.13  583.50    15.13  600.17    15.13  616.83    15.13
  633.50    15.13  887.63    14.40 1141.75    13.67 1395.88    12.93 1650.00    12.20
```

Table 28. Future High Risk Without Project Conditions COSTDAM File for Cell 1

```
REACH LENGTH AND X-SECTION VOLUME
  3549,0.0
DISTANCE SHORELINE TO REF LINE / LONG TERM EROSION RATE
     0.0      0.0
DEFAULT PERSONAL PROPERTY PERCENTAGE
0.35
STORM EROSION,WAVE IMPACT,WAVE DAMAGE ELEVATION, ZONES 1 & 2
   1175.00       0.00       0.0     0.      0.
   1255.00       0.00       0.0     0.      0.
```

```
   1330.00       0.00       0.0     0.      0.
   1435.00      95.00       0.0     0.      0.
   1555.00    1050.00       0.0     0.      0.
   1645.00    1995.00       0.0     0.      0.
   1690.00    1950.00       0.0     0.      0.
NUMBER OF POINTS IN WATER ELEVATION PROFILE
          10
INUNDATION PROFILE.  DISTANCE FROM BASELINE AND TOTAL WATERLEVEL
    0.00    9.50  650.75    9.50 1301.50    9.50 1306.50    0.00 1695.42    0.00
 2084.33    0.00 2473.25    0.00 2862.17    0.00 3251.08    0.00 3640.00    0.00
    0.00   10.87  650.75   10.87 1301.50   10.87 1318.17   10.87 1334.83   10.87
 1351.50   10.87 1782.43    9.08 2213.37    7.29 2644.30    5.50 3640.00    5.50
    0.00   11.60  650.75   11.60 1301.50   11.60 1318.17   11.58 1334.83   11.56
 1351.50   11.54 1788.17    9.73 2224.83    7.91 2661.50    6.10 3640.00    6.10
    0.00   14.52  650.75   14.52 1301.50   14.52 1386.00   14.52 1470.50   14.52
 1555.00   14.52 2173.06   12.04 2791.11    9.57 3409.17    7.10 3640.00    7.10
    0.00   16.37  777.50   16.37 1555.00   16.37 1585.00   16.37 1615.00   16.37
 1645.00   16.37 2143.75   14.38 2642.50   12.38 3141.25   10.39 3640.00    8.39
    0.00   17.98  822.50   17.98 1645.00   17.98 1668.33   17.98 1691.67   17.98
 1715.00   17.98 2196.25   16.06 2677.50   14.13 3158.75   12.21 3640.00   10.28
    0.00   20.38  857.50   20.38 1715.00   20.38 1741.67   20.38 1768.33   20.38
 1795.00   20.38 2256.25   18.53 2717.50   16.69 3178.75   14.84 3640.00   13.00
```

Table 29. Future High Risk Without Project Conditions COSTDAM File for Cell 2

```
REACH LENGTH AND X-SECTION VOLUME
  2959,0.0
DISTANCE SHORELINE TO REF LINE / LONG TERM EROSION RATE
     0.0      0.0
DEFAULT PERSONAL PROPERTY PERCENTAGE
0.35
STORM EROSION,WAVE IMPACT,WAVE DAMAGE ELEVATION, ZONES 1 & 2
   1150.00       0.00       0.0     0.      0.
   1235.00       0.00       0.0     0.      0.
   1315.00       0.00       0.0     0.      0.
   1392.00       0.00       0.0     0.      0.
   1448.00     802.00       0.0     0.      0.
   1502.00    2053.00       0.0     0.      0.
   1578.00    1977.00       0.0     0.      0.
NUMBER OF POINTS IN WATER ELEVATION PROFILE
          10
INUNDATION PROFILE.  DISTANCE FROM BASELINE AND TOTAL WATERLEVEL
    0.00   11.27  857.50   11.27 1000.00   11.27 1200.00   11.27 1400.00   11.27
 1455.00   11.27 1955.00    9.18 2480.00    7.09 2995.83    5.00 3555.00    5.00
    0.00   11.85  713.75   11.85 1427.50   11.85 1444.17   11.85 1460.83   11.85
```

```
 1477.50   11.85 2002.78    9.74 2528.06    7.62 3053.33    5.50 3555.00    5.50
    0.00   12.57  732.50   12.57 1465.00   12.57 1481.67   12.57 1498.33   12.57
 1515.00   12.57 2048.06   10.41 2581.11    8.26 3114.17    6.10 3555.00    6.10
    0.00   14.77  748.75   14.77 1497.50   14.77 1520.83   14.77 1544.17   14.77
 1567.50   14.77 2206.94   12.22 2846.39    9.66 3485.83    7.10 3555.00    7.10
    0.00   16.81  783.75   16.81 1567.50   16.81 1584.17   16.81 1600.83   16.81
 1617.50   16.81 2101.88   14.85 2586.25   12.89 3070.63   10.93 3555.00    8.97
    0.00   18.43  797.75   18.43 1595.50   18.43 1612.17   18.43 1628.83   18.43
 1645.50   18.43 2122.88   16.50 2600.25   14.58 3077.63   12.66 3555.00   10.74
    0.00   20.86  816.25   20.86 1632.50   20.86 1649.17   20.86 1665.83   20.86
 1682.50   20.86 2150.63   18.98 2618.75   17.09 3086.88   15.21 3555.00   13.33
```

Table 30. Future High Risk Without Project Conditions COSTDAM File for Cell 3

```
REACH LENGTH AND X-SECTION VOLUME
  6965,0.0
DISTANCE SHORELINE TO REF LINE / LONG TERM EROSION RATE
     0.0      0.0
DEFAULT PERSONAL PROPERTY PERCENTAGE
0.35
STORM EROSION,WAVE IMPACT,WAVE DAMAGE ELEVATION, ZONES 1 & 2
    258.00       0.00       0.0     0.      0.
    430.00       0.00       0.0     0.      0.
    625.00       0.00       0.0     0.      0.
    906.00       0.00       0.0     0.      0.
   1095.00    1466.00       0.0     0.      0.
   1245.00    1760.00       0.0     0.      0.
   1500.00    2190.00       0.0     0.      0.
NUMBER OF POINTS IN WATER ELEVATION PROFILE
          10
INUNDATION PROFILE.  DISTANCE FROM BASELINE AND TOTAL WATERLEVEL
    0.00   11.38  816.25   11.38 1349.00   11.38 1365.00   11.38 1382.00   11.38
 1399.00   11.38 1905.00    9.25 2445.00    7.13 2943.50    5.00 3320.00    5.00
    0.00   11.99  674.25   11.99 1348.50   11.99 1365.17   11.99 1381.83   11.99
 1398.50   11.99 1922.50    9.83 2446.50    7.66 2970.50    5.50 3320.00    5.50
    0.00   12.75  674.25   12.75 1348.50   12.75 1365.17   12.75 1381.83   12.75
 1398.50   12.75 1935.83   10.53 2473.17    8.32 3010.50    6.10 3320.00    6.10
    0.00   14.74  674.25   14.74 1348.50   14.74 1365.17   14.74 1381.83   14.74
 1398.50   14.74 2018.43   12.19 2638.37    9.65 3258.30    7.10 3320.00    7.10
    0.00   16.76  674.25   16.76 1348.50   16.76 1365.17   16.76 1381.83   16.76
 1398.50   16.76 1878.88   14.79 2359.25   12.82 2839.63   10.85 3320.00    8.87
    0.00   18.34  674.25   18.34 1348.50   18.34 1397.33   18.34 1446.17   18.34
 1495.00   18.34 1951.25   16.52 2407.50   14.69 2863.75   12.86 3320.00   11.04
    0.00   21.42  747.50   21.42 1495.00   21.42 1515.00   21.42 1535.00   21.42
 1555.00   21.42 1996.25   19.65 2437.50   17.89 2878.75   16.13 3320.00   14.36
```

Table 31. Future High Risk Without Project Conditions COSTDAM File for Cell 4

```
REACH LENGTH AND X-SECTION VOLUME
  4585,0.0
DISTANCE SHORELINE TO REF LINE / LONG TERM EROSION RATE
     0.0      0.0
DEFAULT PERSONAL PROPERTY PERCENTAGE
0.35
STORM EROSION,WAVE IMPACT,WAVE DAMAGE ELEVATION, ZONES 1 & 2
    450.00       0.00       0.0     0.      0.
    674.00       0.00       0.0     0.      0.
    903.00       0.00       0.0     0.      0.
   1228.00     890.00       0.0     0.      0.
   1428.00    1687.00       0.0     0.      0.
   1642.00    1473.00       0.0     0.      0.
   1921.00    1194.00       0.0     0.      0.
NUMBER OF POINTS IN WATER ELEVATION PROFILE
          10
INUNDATION PROFILE.  DISTANCE FROM BASELINE AND TOTAL WATERLEVEL
    0.00   10.50  502.25   10.50 1004.50   10.50 1009.50    0.00 1360.42    0.00
 1711.33    0.00 2062.25    0.00 2413.17    0.00 2764.08    0.00 3115.00    0.00
    0.00   11.50  502.25   11.50 1004.50   11.50 1021.17   11.40 1037.83   11.30
 1054.50   11.20 1512.83    9.30 1971.17    7.40 2429.50    5.50 3115.00    5.50
    0.00   12.10  502.25   12.10 1004.50   12.10 1021.17   11.94 1037.83   11.77
 1054.50   11.61 1497.00    9.77 1939.50    7.94 2382.00    6.10 3115.00    6.10
    0.00   13.99  502.25   13.99 1004.50   13.99 1021.17   13.99 1037.83   13.99
 1054.50   13.99 1685.50   11.69 2316.50    9.40 2947.50    7.10 3115.00    7.10
    0.00   16.02  612.50   16.02 1225.00   16.02 1241.67   16.02 1258.33   16.02
 1275.00   16.02 1735.00   14.16 2195.00   12.31 2655.00   10.46 3115.00    8.60
    0.00   17.83  630.00   17.83 1260.00   17.83 1276.67   17.83 1293.33   17.83
 1310.00   17.83 1761.25   16.01 2212.50   14.19 2663.75   12.37 3115.00   10.55
    0.00   19.59  647.50   19.59 1295.00   19.59 1311.67   19.59 1328.33   19.59
 1345.00   19.59 1787.50   17.81 2230.00   16.03 2672.50   14.25 3115.00   12.47
```

Table 32. Future High Risk Without Project Conditions COSTDAM File for Cell 5

```
REACH LENGTH AND X-SECTION VOLUME
  5835,0.0
DISTANCE SHORELINE TO REF LINE / LONG TERM EROSION RATE
     0.0      0.0
DEFAULT PERSONAL PROPERTY PERCENTAGE
0.35
STORM EROSION,WAVE IMPACT,WAVE DAMAGE ELEVATION, ZONES 1 & 2
   1028.00       0.00       0.0     0.      0.
   1125.00       0.00       0.0     0.      0.
   1225.00       0.00       0.0     0.      0.
   1430.00       0.00       0.0     0.      0.
   1758.00       0.00       0.0     0.      0.
   1845.00      45.00       0.0     0.      0.
   1925.00    1390.00       0.0     0.      0.
NUMBER OF POINTS IN WATER ELEVATION PROFILE
          10
INUNDATION PROFILE.  DISTANCE FROM BASELINE AND TOTAL WATERLEVEL
    0.00   10.80  647.50   10.80 1028.00   10.80 1206.17   10.00 1344.00    9.40
 1400.00    9.10 1500.03    8.77 1971.57    6.89 2443.10    5.00 3315.00    5.00
    0.00   11.40  514.25   11.40 1028.50   11.40 1045.17   11.28 1061.83   11.16
 1078.50   11.04 1525.17    9.19 1971.83    7.35 2418.50    5.50 3315.00    5.50
    0.00   12.00  516.75   12.00 1033.50   12.00 1050.17   11.83 1066.83   11.66
 1083.50   11.49 1516.00    9.69 1948.50    7.90 2381.00    6.10 3315.00    6.10
    0.00   14.39  516.75   14.39 1033.50   14.39 1050.17   14.39 1066.83   14.39
 1083.50   14.39 1724.39   11.96 2365.28    9.53 3006.17    7.10 3315.00    7.10
    0.00   15.87  592.25   15.87 1184.50   15.87 1201.17   15.87 1217.83   15.87
 1234.50   15.87 1925.50   13.21 2616.50   10.56 3307.50    7.90 3315.00    7.90
    0.00   17.59  657.50   17.59 1315.00   17.59 1331.67   17.59 1348.33   17.59
 1365.00   17.59 1852.50   15.61 2340.00   13.63 2827.50   11.65 3315.00    9.67
    0.00   19.72  667.50   19.72 1335.00   19.72 1368.33   19.72 1401.67   19.72
 1435.00   19.72 1905.00   17.84 2375.00   15.96 2845.00   14.08 3315.00   12.20
```

Table 33. Future High Risk Without Project Conditions COSTDAM File for Cell 6

```
REACH LENGTH AND X-SECTION VOLUME
  1090,0.0
DISTANCE SHORELINE TO REF LINE / LONG TERM EROSION RATE
     0.0      0.0
DEFAULT PERSONAL PROPERTY PERCENTAGE
0.35
STORM EROSION,WAVE IMPACT,WAVE DAMAGE ELEVATION, ZONES 1 & 2
    715.00       0.00       0.0     0.       0.
    774.00       0.00       0.0     0.       0.
    872.00       0.00       0.0     0.       0.
   1005.00       0.00       0.0     0.       0.
   1135.00       0.00       0.0     0.       0.
   1312.00       0.00       0.0     0.       0.
   1540.00       0.00       0.0     0.       0.
NUMBER OF POINTS IN WATER ELEVATION PROFILE
          10
INUNDATION PROFILE.  DISTANCE FROM BASELINE AND TOTAL WATERLEVEL
    0.00   12.30  456.25   12.30  912.50   12.30  917.50    0.00 1289.58    0.00
 1661.67    0.00 2033.75    0.00 2405.83    0.00 2777.92    0.00 3150.00    0.00
    0.00   12.60  913.00   12.60  918.00    0.00 1287.86    0.00 1598.21    0.00
 1908.57    0.00 2218.93    0.00 2529.29    0.00 2839.64    0.00 3150.00    0.00
    0.00   13.60  938.00   13.60  988.00    8.37 1296.86    7.91 1605.71    7.81
 1914.57    7.72 2223.43    7.64 2532.29    7.55 2841.14    7.13 3150.00    6.20
    0.00   15.10  469.00   15.10  938.00   15.10  954.67   14.97  971.33   14.83
  988.00   14.70 1607.00   12.17 2226.00    9.64 2845.00    7.11 3150.00    7.10
    0.00   16.24  472.50   16.24  945.00   16.24  961.67   16.24  978.33   16.24
  995.00   16.24 1686.67   13.46 2378.33   10.68 3070.00    7.90 3150.00    7.90
    0.00   18.07  492.50   18.07  985.00   18.07 1001.67   18.07 1018.33   18.07
 1035.00   18.07 1563.75   16.03 2092.50   13.98 2621.25   11.94 3150.00    9.89
    0.00   20.69  552.50   20.69 1105.00   20.69 1121.67   20.69 1138.33   20.69
 1155.00   20.69 1653.75   18.86 2152.50   17.03 2651.25   15.20 3150.00   13.36
```

Table 34. Future High Risk Without Project Conditions COSTDAM File for Cell 7

```
REACH LENGTH AND X-SECTION VOLUME
  6267,0.0
DISTANCE SHORELINE TO REF LINE / LONG TERM EROSION RATE
     0.0      0.0
DEFAULT PERSONAL PROPERTY PERCENTAGE
0.35
```

```
STORM EROSION,WAVE IMPACT,WAVE DAMAGE ELEVATION, ZONES 1 & 2
    315.00        0.00        0.0     0.      0.
    438.00        0.00        0.0     0.      0.
    548.00        0.00        0.0     0.      0.
    715.00        0.00        0.0     0.      0.
    873.00       69.00        0.0     0.      0.
    983.00      668.00        0.0     0.      0.
   1182.00      618.00        0.0     0.      0.
NUMBER OF POINTS IN WATER ELEVATION PROFILE
         10
INUNDATION PROFILE.  DISTANCE FROM BASELINE AND TOTAL WATERLEVEL
    0.00   11.70  322.25   11.70  644.50   11.70  649.50    0.00  816.25    0.00
  983.00    0.00 1149.75    0.00 1316.50    0.00 1483.25    0.00 1650.00    0.00
    0.00   12.30  329.75   12.30  659.50   12.30  664.50    0.00  924.67    0.00
 1184.83    0.00 1445.00    0.00 1350.00    0.00 1500.00    0.00 1650.00    0.00
    0.00   12.60  329.75   12.60  659.50   12.60  676.17   12.50  692.83   12.40
  709.50   12.30  944.63   11.33 1179.75   10.35 1414.88    9.38 1650.00    8.40
    0.00   14.66  337.25   14.66  674.50   14.66  691.17   14.66  707.83   14.66
  724.50   14.66  955.88   13.70 1187.25   12.75 1418.63   11.79 1650.00   10.83
    0.00   16.50  347.25   16.50  694.50   16.50  711.17   16.50  727.83   16.50
  744.50   16.50  970.88   15.65 1197.25   14.80 1423.63   13.95 1650.00   13.10
    0.00   18.15  400.00   18.15  800.00   18.15  816.67   18.15  833.33   18.15
  850.00   18.15 1050.00   17.43 1250.00   16.70 1450.00   15.98 1650.00   15.25
    0.00   20.35  462.50   20.35  925.00   20.35  941.67   20.35  958.33   20.35
  975.00   20.35 1143.75   19.79 1312.50   19.22 1481.25   18.66 1650.00   18.09
```

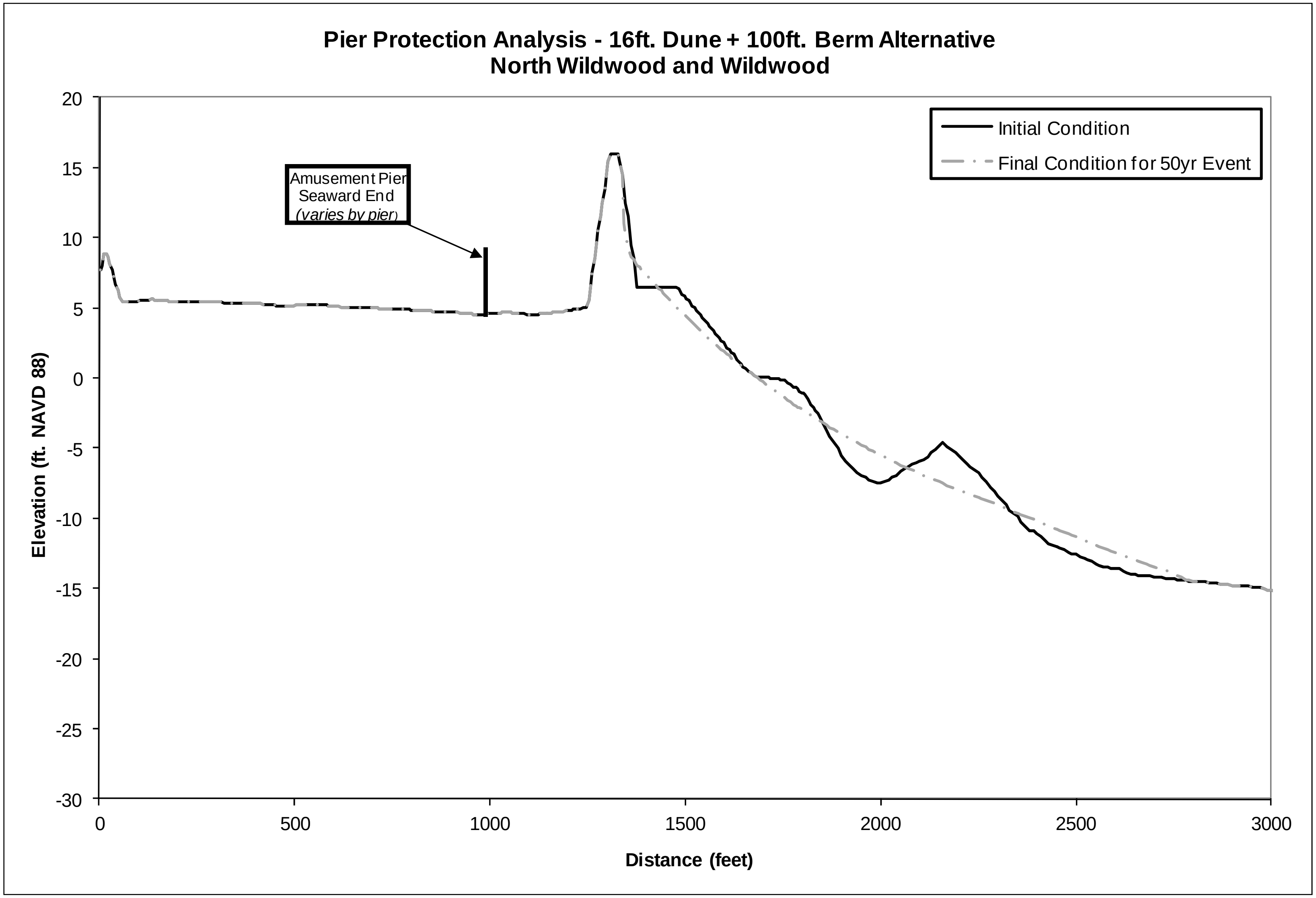


Figure 50. Pre- and Post "50-yr" Storm Beach Profiles for 16 ft. Dune & 100 ft. Berm Pier Protection Alternative

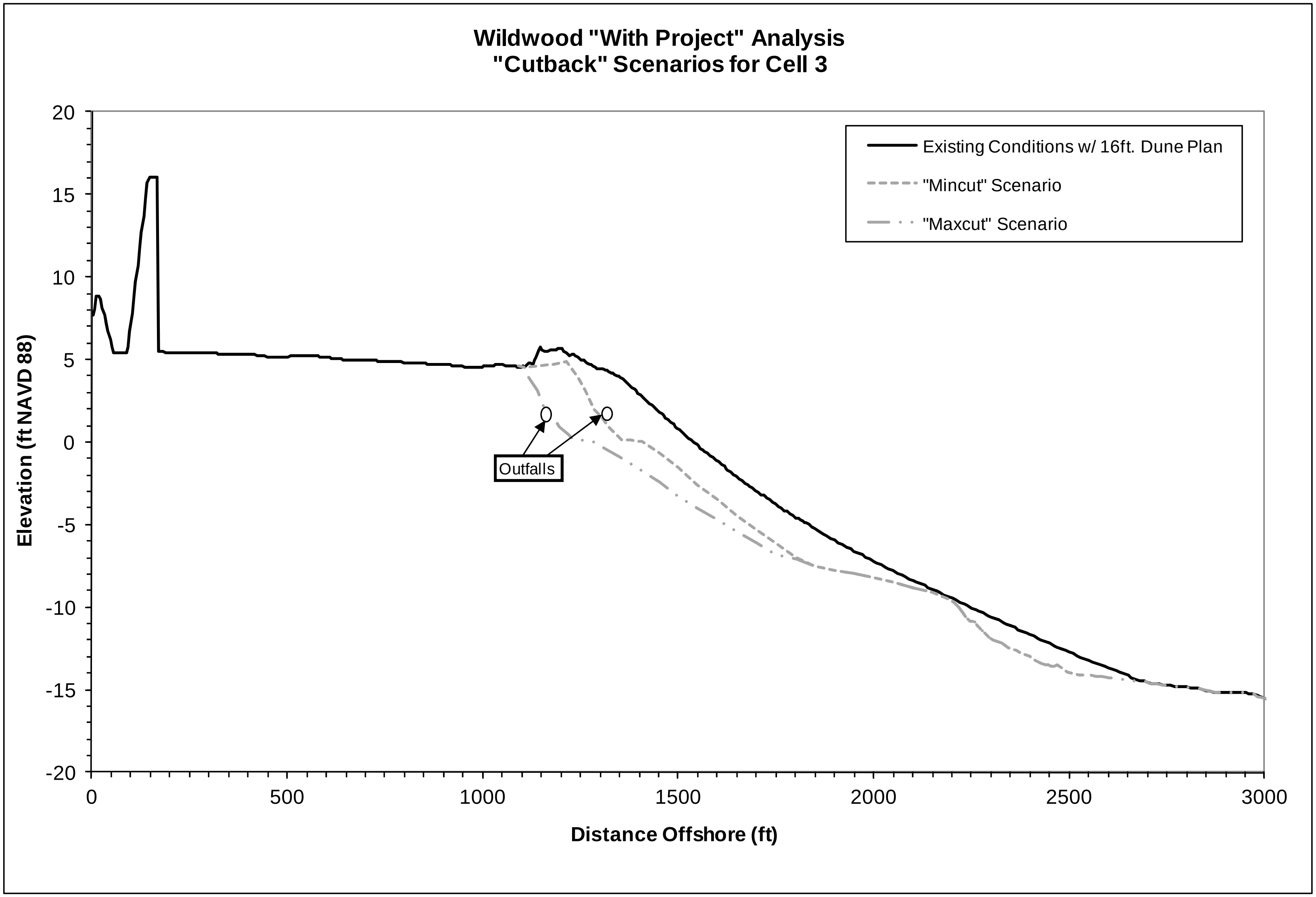


Figure 51. Berm Cutback Alternatives Examined for With Project Analysis in Cell 3

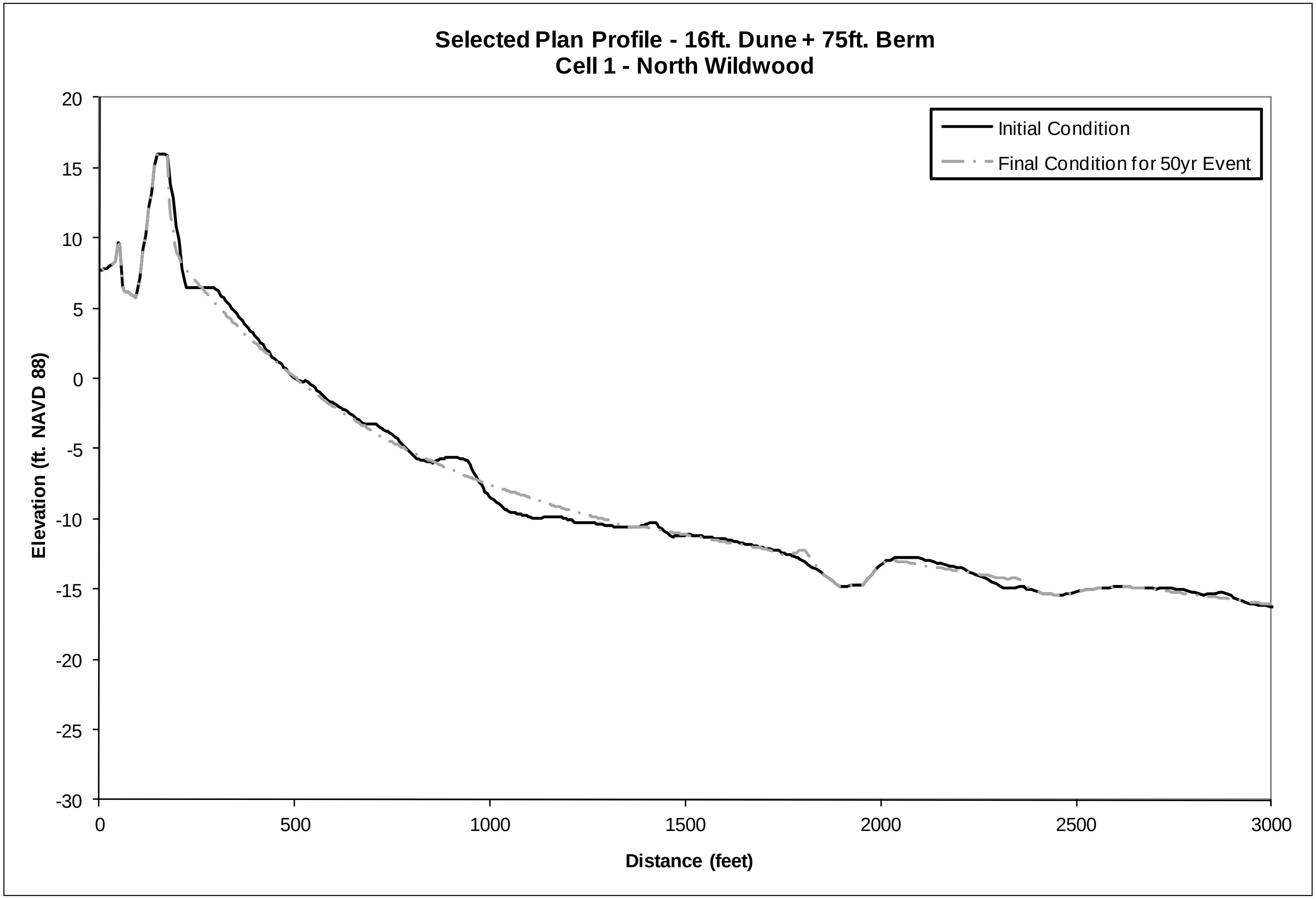


Figure 52. Pre- and Post "50-yr" Storm Beach Profiles for Selected Plan in Cell 1

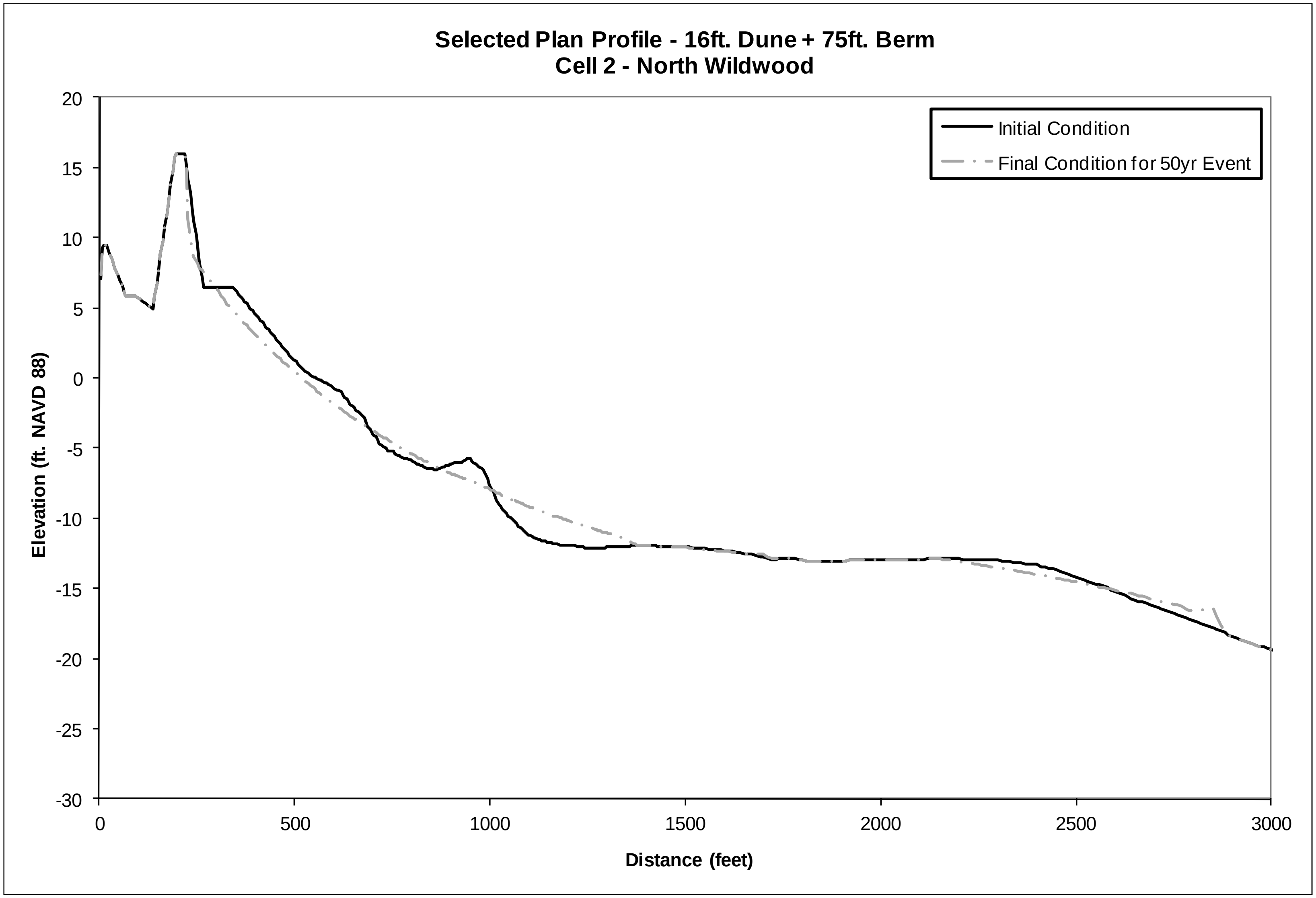


Figure 53. Pre- and Post "50-yr" Storm Beach Profiles for Selected Plan in Cell 2

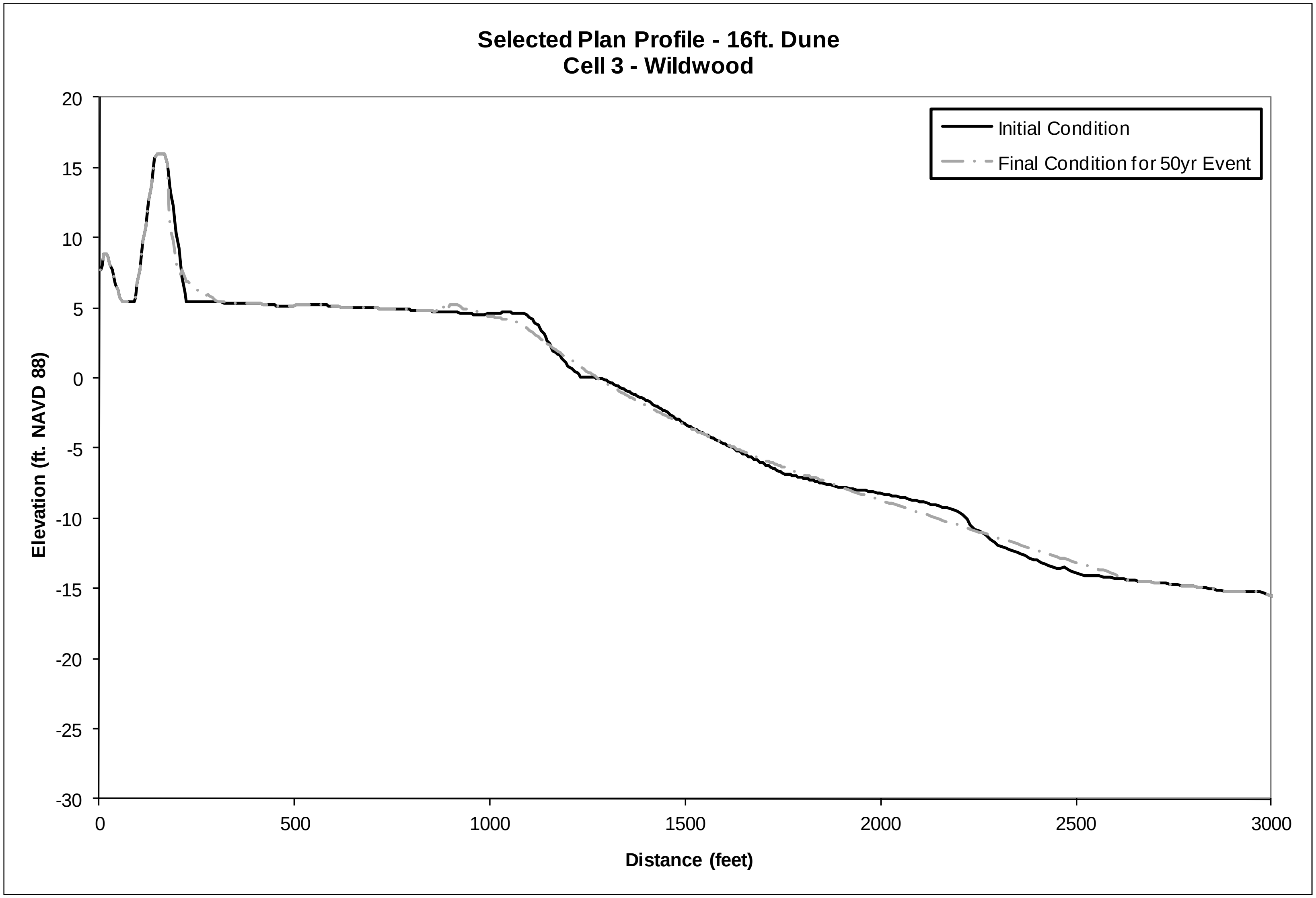


Figure 54. Pre- and Post "50-yr" Storm Beach Profiles for Selected Plan in Cell 3

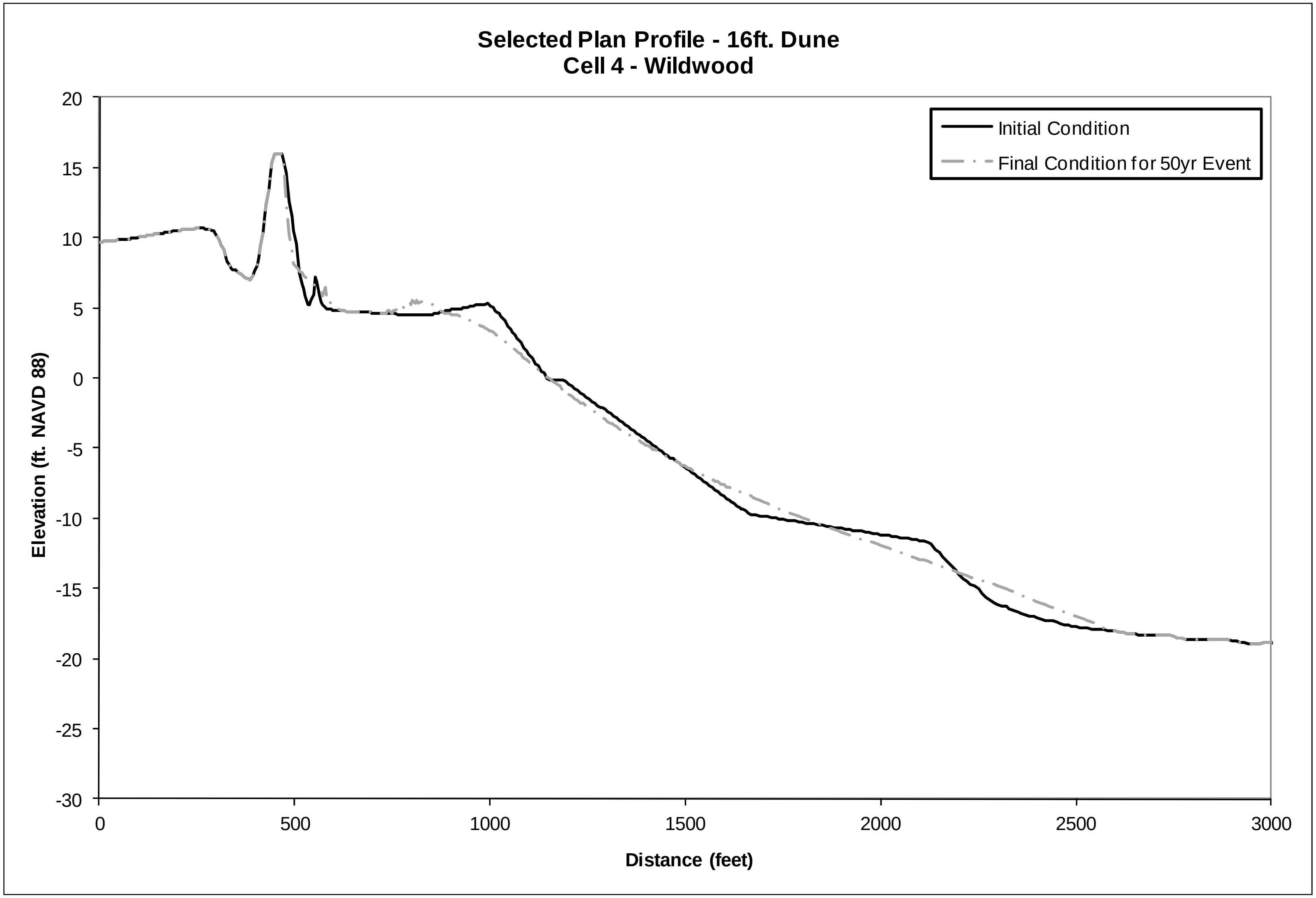


Figure 55. Pre- and Post "50-yr" Storm Beach Profiles for Selected Plan in Cell 4

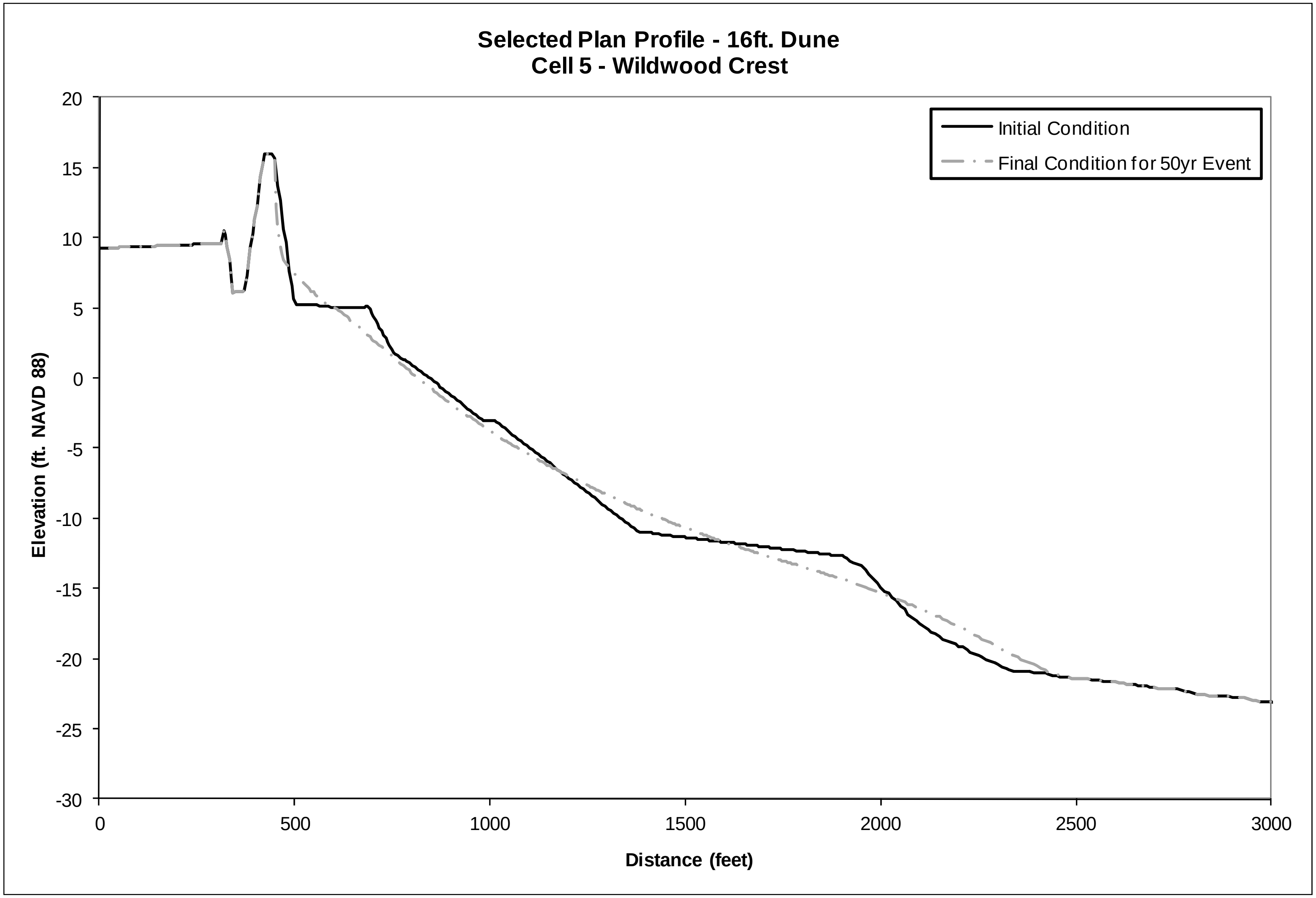


Figure 56. Pre- and Post "50-yr" Storm Beach Profiles for Selected Plan in Cell 5

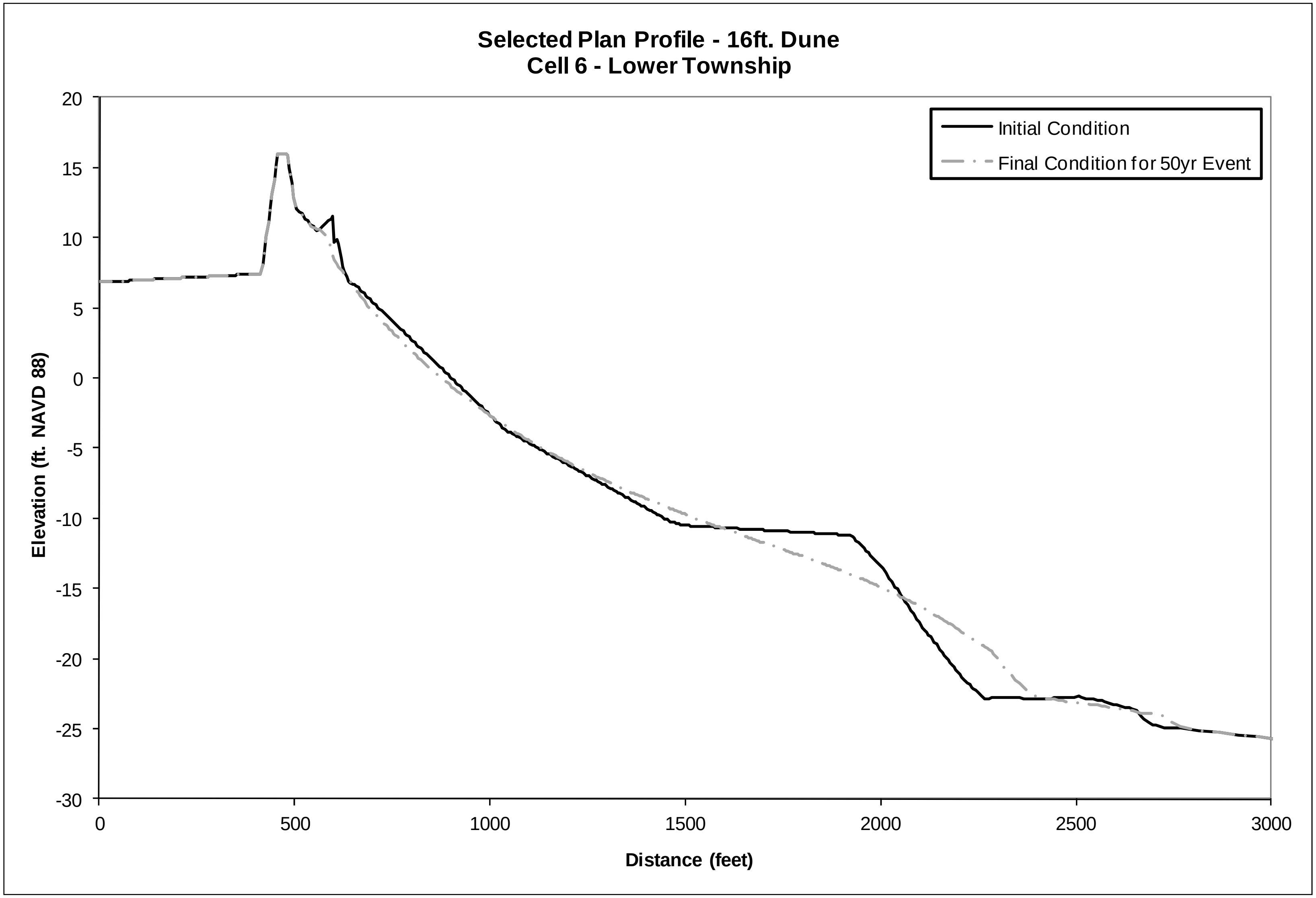


Figure 57. Pre- and Post "50-yr" Storm Beach Profiles for Selected Plan in Cell 6

Table 35. Selected Plan of 16 ft. Dune + 75 ft. Berm COSTDAM File for Cell 1

```
REACH LENGTH AND X-SECTION VOLUME
  3549,0.0
DISTANCE SHORELINE TO REF LINE / LONG TERM EROSION RATE
     0.0      0.0
DEFAULT PERSONAL PROPERTY PERCENTAGE
0.35
STORM EROSION,WAVE IMPACT,WAVE DAMAGE ELEVATION, ZONES 1 & 2
   1065.00       0.00       0.0     0.      0.
   1155.00       0.00       0.0     0.      0.
   1160.00       0.00       0.0     0.      0.
   1175.00       0.00       0.0     0.      0.
   1185.00       0.00       0.0     0.      0.
   1215.00       0.00       0.0     0.      0.
   1250.00    2390.00       0.0     0.      0.
NUMBER OF POINTS IN WATER ELEVATION PROFILE
          10
INUNDATION PROFILE.  DISTANCE FROM BASELINE AND TOTAL WATERLEVEL
    0.00    8.80  600.00    8.80 1182.00    8.80 1187.00    0.00 1300.00    0.00
 1500.00    0.00 1900.00    0.00 2300.00    0.00 2900.00    0.00 3640.00    0.00
    0.00    9.00  600.00    9.00 1182.00    9.00 1187.00    0.00 1300.00    0.00
 1500.00    0.00 1900.00    0.00 2300.00    0.00 2900.00    0.00 3640.00    0.00
    0.00    9.40  600.00    9.40 1182.00    9.40 1187.00    0.00 1300.00    0.00
 1500.00    0.00 1900.00    0.00 2300.00    0.00 2900.00    0.00 3640.00    0.00
    0.00   11.20  600.00   11.20 1182.00   11.20 1187.00    0.00 1300.00    0.00
 1500.00    0.00 1900.00    0.00 2300.00    0.00 2900.00    0.00 3640.00    0.00
    0.00   13.20  600.00   13.20 1187.00   13.20 1192.00    0.00 1300.00    0.00
 1500.00    0.00 1900.00    0.00 2300.00    0.00 2900.00    0.00 3640.00    0.00
    0.00   14.90  600.00   14.90 1187.00   14.90 1204.00   14.90 1221.00   14.90
 1238.00   14.70 1749.00   12.80 2260.00   10.80 2771.00    8.90 3640.00    8.90
    0.00   16.90  600.00   16.90 1313.00   16.90 1329.00   16.90 1346.00   16.90
 1363.00   16.90 1954.00   14.90 2546.00   12.50 3175.00   10.00 3640.00   10.00
```

Table 36. Selected Plan of 16 ft. Dune + 75 ft. Berm COSTDAM File for Cell 2

```
REACH LENGTH AND X-SECTION VOLUME
  2959,0.0
DISTANCE SHORELINE TO REF LINE / LONG TERM EROSION RATE
     0.0      0.0
DEFAULT PERSONAL PROPERTY PERCENTAGE
0.35
STORM EROSION,WAVE IMPACT,WAVE DAMAGE ELEVATION, ZONES 1 & 2
```

```
   1045.00       0.00       0.0     0.      0.
   1110.00       0.00       0.0     0.      0.
   1160.00       0.00       0.0     0.      0.
   1200.00       0.00       0.0     0.      0.
   1260.00       0.00       0.0     0.      0.
   1305.00       0.00       0.0     0.      0.
   1380.00     335.00       0.0     0.      0.
NUMBER OF POINTS IN WATER ELEVATION PROFILE
          10
INUNDATION PROFILE.  DISTANCE FROM BASELINE AND TOTAL WATERLEVEL
    0.00    9.40  600.00    9.40 1132.00    9.40 1137.00    0.00 1200.00    0.00
 1400.00    0.00 2000.00    0.00 2500.00    0.00 3000.00    0.00 3555.00    0.00
    0.00    9.80  600.00    9.80 1132.00    9.80 1137.00    0.00 1200.00    0.00
 1400.00    0.00 2000.00    0.00 2500.00    0.00 3000.00    0.00 3555.00    0.00
    0.00   10.00  600.00   10.00 1132.00   10.00 1137.00    0.00 1200.00    0.00
 1400.00    0.00 2000.00    0.00 2500.00    0.00 3000.00    0.00 3555.00    0.00
    0.00   12.00  600.00   12.00 1132.00   12.00 1137.00    0.00 1200.00    0.00
 1400.00    0.00 2000.00    0.00 2500.00    0.00 3000.00    0.00 3555.00    0.00
    0.00   13.00  600.00   13.00 1142.00   13.00 1147.00    0.00 1200.00    0.00
 1400.00    0.00 2000.00    0.00 2500.00    0.00 3000.00    0.00 3555.00    0.00
    0.00   14.50  600.00   14.50 1142.00   14.50 1159.00   14.50 1176.00   14.50
 1193.00   14.50 1723.00   12.80 2252.00   10.70 2765.00    8.90 3555.00    8.90
    0.00   17.40  600.00   17.40 1200.00   17.40 1220.00   17.40 1230.00   17.40
 1240.00   17.40 2014.00   14.50 2703.00   11.70 3150.00   10.00 3555.00   10.00
```

Table 37. Selected Plan of 16 ft. Dune COSTDAM File for Cell 3

```
REACH LENGTH AND X-SECTION VOLUME
  6965,0.0
DISTANCE SHORELINE TO REF LINE / LONG TERM EROSION RATE
     0.0      0.0
DEFAULT PERSONAL PROPERTY PERCENTAGE
0.35
STORM EROSION,WAVE IMPACT,WAVE DAMAGE ELEVATION, ZONES 1 & 2
     85.00       0.00       0.0     0.      0.
    250.00       0.00       0.0     0.      0.
    420.00       0.00       0.0     0.      0.
    675.00       0.00       0.0     0.      0.
    885.00       0.00       0.0     0.      0.
    995.00     845.00       0.0     0.      0.
   1275.00    2045.00       0.0     0.      0.
NUMBER OF POINTS IN WATER ELEVATION PROFILE
          10
INUNDATION PROFILE.  DISTANCE FROM BASELINE AND TOTAL WATERLEVEL
    0.00    7.90  650.00    7.90 1182.00    7.90 1187.00    0.00 1700.00    0.00
```

```
 2100.00    0.00 2500.00    0.00 2800.00    0.00 3200.00    0.00 3320.00    0.00
    0.00    8.30  650.00    8.30 1182.00    8.30 1187.00    0.00 1700.00    0.00
 2100.00    0.00 2500.00    0.00 2800.00    0.00 3200.00    0.00 3320.00    0.00
    0.00    8.60  650.00    8.60 1182.00    8.60 1187.00    0.00 1700.00    0.00
 2100.00    0.00 2500.00    0.00 2800.00    0.00 3200.00    0.00 3320.00    0.00
    0.00    9.30  650.00    9.30 1182.00    9.30 1187.00    0.00 1700.00    0.00
 2100.00    0.00 2500.00    0.00 2800.00    0.00 3200.00    0.00 3320.00    0.00
    0.00   13.60  650.00   13.60 1187.00   13.60 1192.00    0.00 1251.00    0.00
 1268.00    0.00 1571.00    0.00 1874.00    0.00 2178.00    0.00 3320.00    0.00
    0.00   15.80  650.00   15.80 1200.00   15.80 1250.00   15.80 1255.00   15.80
 1258.00   15.80 1589.00   14.40 1920.00   13.00 2252.00    8.90 3320.00    8.90
    0.00   17.90  650.00   17.90 1200.00   17.90 1250.00   17.90 1260.00   17.90
 1300.00   17.90 1385.00   17.90 1895.00   15.80 2518.00   13.30 3320.00   10.10
```

Table 38. Selected Plan of 16 ft. Dune COSTDAM File for Cell 4

```
REACH LENGTH AND X-SECTION VOLUME
  4585,0.0
DISTANCE SHORELINE TO REF LINE / LONG TERM EROSION RATE
     0.0      0.0
DEFAULT PERSONAL PROPERTY PERCENTAGE
0.35
STORM EROSION,WAVE IMPACT,WAVE DAMAGE ELEVATION, ZONES 1 & 2
    330.00       0.00       0.0     0.      0.
    490.00       0.00       0.0     0.      0.
    665.00       0.00       0.0     0.      0.
    805.00       0.00       0.0     0.      0.
    955.00       0.00       0.0     0.      0.
   1145.00     855.00       0.0     0.      0.
   1425.00    1690.00       0.0     0.      0.
NUMBER OF POINTS IN WATER ELEVATION PROFILE
          10
INUNDATION PROFILE.  DISTANCE FROM BASELINE AND TOTAL WATERLEVEL
    0.00    8.00  650.00    8.00  877.00    8.00  882.00    0.00 1200.00    0.00
 1600.00    0.00 2000.00    0.00 2400.00    0.00 2800.00    0.00 3115.00    0.00
    0.00    8.60  650.00    8.60  877.00    8.60  882.00    0.00 1200.00    0.00
 1600.00    0.00 2000.00    0.00 2400.00    0.00 2800.00    0.00 3115.00    0.00
    0.00    9.20  650.00    9.20  882.00    9.20  887.00    0.00 1200.00    0.00
 1600.00    0.00 2000.00    0.00 2400.00    0.00 2800.00    0.00 3115.00    0.00
    0.00   10.50  650.00   10.50  887.00   10.50  892.00    0.00 1200.00    0.00
 1600.00    0.00 2000.00    0.00 2400.00    0.00 2800.00    0.00 3115.00    0.00
    0.00   12.80  650.00   12.80  902.00   12.80  907.00    0.00  930.00    0.00
  947.00    0.00 1294.00    0.00 1640.00    0.00 2120.00    0.00 3115.00    0.00
    0.00   14.90  650.00   14.90  900.00   14.90  915.00   14.90  930.00   14.90
 1050.00   14.90 1343.00   13.70 1734.00   12.10 2540.00    8.90 3115.00    8.90
```

```
    0.00    17.00  650.00    17.00  900.00    17.00  915.00    17.00 1063.00    17.00
 1200.00    17.00 1680.00    14.90 2309.00    12.40 2939.00    10.00 3115.00    10.00
```

Table 39. Selected Plan of 16 ft. Dune COSTDAM File for Cell 5

```
REACH LENGTH AND X-SECTION VOLUME
  5835,0.0
DISTANCE SHORELINE TO REF LINE / LONG TERM EROSION RATE
     0.0      0.0
DEFAULT PERSONAL PROPERTY PERCENTAGE
0.35
STORM EROSION,WAVE IMPACT,WAVE DAMAGE ELEVATION, ZONES 1 & 2
    655.00       0.00       0.0     0.      0.
    715.00       0.00       0.0     0.      0.
    775.00       0.00       0.0     0.      0.
    870.00       0.00       0.0     0.      0.
   1035.00       0.00       0.0     0.      0.
   1195.00       0.00       0.0     0.      0.
   1315.00       0.00       0.0     0.      0.
NUMBER OF POINTS IN WATER ELEVATION PROFILE
          10
INUNDATION PROFILE.  DISTANCE FROM BASELINE AND TOTAL WATERLEVEL
    0.00     8.30  650.00     8.30  917.00     8.30  922.00     0.00 1400.00     0.00
 1800.00     0.00 2200.00     0.00 2600.00     0.00 3000.00     0.00 3315.00     0.00
    0.00     9.00  650.00     9.00  922.00     9.00  927.00     0.00 1400.00     0.00
 1800.00     0.00 2200.00     0.00 2600.00     0.00 3000.00     0.00 3315.00     0.00
    0.00     9.50  650.00     9.50  932.00     9.50  937.00     0.00 1400.00     0.00
 1800.00     0.00 2200.00     0.00 2600.00     0.00 3000.00     0.00 3315.00     0.00
    0.00    11.40  650.00    11.40  932.00    11.40  937.00     0.00 1721.00     0.00
 2040.00     0.00 2359.00     0.00 2677.00     0.00 2996.00     0.00 3315.00     0.00
    0.00    13.40  650.00    13.40  932.00    13.40  937.00     0.00  976.00     0.00
  993.00     0.00 1348.00     0.00 1703.00     0.00 2058.00     0.00 3315.00     0.00
    0.00    14.90  650.00    14.90  700.00    14.90  800.00    14.90 1000.00    14.90
 1020.00    14.90 1492.00    13.00 1963.00    11.00 2474.00     8.90 3315.00     8.90
    0.00    17.50  650.00    17.50  900.00    17.50 1000.00    17.50 1100.00    17.50
 1150.00    17.50 1711.00    15.40 2273.00    13.10 3064.00    10.00 3315.00    10.00
```

Table 40. Selected Plan of 16 ft. Dune COSTDAM File for Cell 6

```
REACH LENGTH AND X-SECTION VOLUME
  1090,0.0
DISTANCE SHORELINE TO REF LINE / LONG TERM EROSION RATE
     0.0      0.0
DEFAULT PERSONAL PROPERTY PERCENTAGE
0.35
STORM EROSION,WAVE IMPACT,WAVE DAMAGE ELEVATION, ZONES 1 & 2
    525.00       0.00       0.0     0.      0.
    575.00       0.00       0.0     0.      0.
    645.00       0.00       0.0     0.      0.
    730.00       0.00       0.0     0.      0.
    830.00       0.00       0.0     0.      0.
    920.00       0.00       0.0     0.      0.
   1120.00       0.00       0.0     0.      0.
NUMBER OF POINTS IN WATER ELEVATION PROFILE
          10
INUNDATION PROFILE.  DISTANCE FROM BASELINE AND TOTAL WATERLEVEL
    0.00   10.30  650.00   10.30  893.00   10.30  898.00    0.00 1200.00    0.00
 1600.00    0.00 2000.00    0.00 2400.00    0.00 2800.00    0.00 3150.00    0.00
    0.00   10.60  650.00   10.60  893.00   10.60  898.00    0.00 1200.00    0.00
 1600.00    0.00 2000.00    0.00 2400.00    0.00 2800.00    0.00 3150.00    0.00
    0.00   10.90  650.00   10.90  893.00   10.90  898.00    0.00 1200.00    0.00
 1600.00    0.00 2000.00    0.00 2400.00    0.00 2800.00    0.00 3150.00    0.00
    0.00   12.70  650.00   12.70  893.00   12.70  898.00    0.00 1200.00    0.00
 1600.00    0.00 2000.00    0.00 2400.00    0.00 2800.00    0.00 3150.00    0.00
    0.00   13.70  650.00   13.70  893.00   13.70  898.00    0.00 1200.00    0.00
 1600.00    0.00 2000.00    0.00 2400.00    0.00 2800.00    0.00 3150.00    0.00
    0.00   15.50  650.00   15.50  903.00   15.50  919.00   15.40  936.00   15.30
  953.00   15.10 1468.00   13.00 1983.00   10.90 2435.00    8.90 3150.00    8.90
    0.00   17.60  650.00   17.60 1000.00   17.60 1020.00   17.60 1040.00   17.60
 1088.00   17.60 1727.00   15.20 2367.00   12.70 3100.00   10.00 3150.00   10.00
```

Section 3.

Surveying and Mapping Requirements

Profile data was developed from offshore and onshore survey lines collected in 1955, 1965, 2001, 2003 and 2012. These profiles were used to perform storm damage and volume calculations. A total of 25 survey lines were occupied in North Wildwood, Wildwood, Wildwood Crest and Lower Township. These profiles consisted of surveys from the back dune, over the berm to the offshore area and to the depth of closure. Profiles from the Richard Stockton Coastal Research Center were also used for this analysis.

Data collection for the study mapping effort including surveys, historic aerial photography from 1920, 1933, 1944, 1962, 1970, 2003, and 2006. Planimetric data was also collected from the 2003 survey data. Auto CAD and Arcmap were used to store and interpret the survey data. Mapping for the Feasibility study is sufficient for the plans and specifications phase, but new survey data will be acquired. Beach profile surveys every 200 feet from the dune/bulkhead line to the depth of closure will be required to accurately determine the quantities in developing the plans and specifications. These profiles will also include shore protection structures and groins in the study area.

Section 4

Geotechnical Appendix

GEOTECHNICAL APPENDIX

03/6/14 Version

INTRODUCTION

The geotechnical appendix provides detailed information and records of the data and procedures utilized in the preparation of the Geotechnical Analysis presented in this report. The information and data presented in the following sections which include the following:

Section 1 – Summary of Boring and Vibracore Utilized
Section 2 – Design Value Computations and Tables
Section 3 – Description of the Design Methodology
Section 4 – Investigation Reports (DVD)
Section 5 – Bibliography of Referenced Documents

GEOTECHNICAL APPENDIX

SECTION 1
Summary of Boring and Vibracore Utilized

This section presents the boring and vibracore logs and gradation curves utilized in the geotechnical analysis presented in this report. It is noted that there are additional beach, vibracore and SPT sampling locations along the beach and near the proposed borrow areas that are not included in the analysis because they were either outside the area under consideration, or they represent material that was examined and found to be unsatisfactory for the proposed North Wildwood beach replenishment. In some cases, potential borrow areas had been dedicated to other beach replenishment projects and were therefore not considered in this analysis. The additional geotechnical information is available in the geotechnical investigation reports included for reference in Section 4, on the attached DVD. The following presents a summary of the borings and vibracores used for the geotechnical analysis.

<table>
<tr><th>Boring ID</th><th>Boring Type</th><th>Sample Area</th><th>Report</th></tr>
<tr><td>NVB-1, NVB-2 & NVB-3</td><td>Borings</td><td rowspan="2">Native Beach Material - North Wildwood</td><td>Geotechnical Data Report Wildwood Beach Investigation (4-17-2007)</td></tr>
<tr><td>WW-1, WW-2 & WW-2B</td><td>Beach Samples</td><td>Atlantic Coast of New Jersey
Beach Profile Data Collection (12-17-2003)</td></tr>
<tr><td></td><td></td><td>Hereford Inlet Borrow Area</td><td></td></tr>
<tr><td>NJV-745, NJV-452 & NJV-799</td><td rowspan="4">Vibracore</td><td>H-1</td><td rowspan="4">Geotechnical Data Report – Vibrational Coring – Wildwood to Hereford Inlet (06-30-2006) and
Geotechnical Investigation Vibrocoring along the New Jersey Coast – Townsend/Hereford Inlet Study Area (12/1998)</td></tr>
<tr><td>NJV-185 & NJV-746</td><td>H-2</td></tr>
<tr><td>NJV-747 & NJV-797</td><td>H-3</td></tr>
<tr><td>NJV-187 & NJV-800</td><td>H-4</td></tr>
<tr><td>NVB-5, 7, 9 & 11</td><td>Borings</td><td rowspan="2">WW/WWC Borrow Area</td><td>Geotechnical Data Report Wildwood Beach Investigation (4-17-2007)</td></tr>
<tr><td>WW-4, WW-7, WW-10, WW-2B, WW-13 & WW-15</td><td>Beach Samples</td><td>Atlantic Coast of New Jersey
Beach Profile Data Collection (12-17-2003)</td></tr>
<tr><td>NJGS-158 & 159</td><td>Vibracore</td><td>Offshore #1</td><td rowspan="2">Logs provided by NJGS</td></tr>
<tr><td>NJGS-147 & 148</td><td>Vibracore</td><td>Offshore #2</td></tr>
<tr><td>NJV-34, 45, 48, 49 & 51</td><td>Vibracore</td><td>Offshore #3</td><td>No report was available, information taken from grain size analysis curves</td></tr>
</table>

GEOTECHNICAL APPENDIX

SECTION 2
Design Value Computations, Plots and Tables

This section presents the computations utilized in the geotechnical analysis, including the millimeter to phi conversion and design value computations, cumulative grain size distribution (GSD) plots and composite distribution plot, for the borrow areas, and tables utilized to tabulate and calculate the overfill factors and re-nourishment factors for each of the borrow areas.

NORTH WILDWOOD BEACH RESTORATION PROJECT

CURRENT SUMMARY SHEETS

North Wildwood Native Beach parameters established using composite of surface and SPT samples values - See Table 2.5

A. Native Beach Material Parameters

From Beach Lines - WW-1, WW-2 and WW-2B, surface samples for total line - BC+200 to El.-18, and 0 to 4' SPT samples for NJB -1, 2 and 3
(See **Table 2.5** for additional information and details)

$M_{\Phi n}$ (Average) = 2.34 — Subscript "n" indicates native beach material property.

$\sigma_{\Phi n}$ (Average) = 0.46 — Subscript "b" indicates borrow material property.

B. Individual Borrow Area Summaries

Table 2.1A Hereford Inlet Borrow Areas (Considering only depth of borrow and compositing)

Borrow Area & Comments	Vibracores	Mean Median Dia. in Phi Units $M_{\Phi b}$	Mean Standard Deviation in Phi units $\sigma_{\Phi b}$	Est. Borrow Depth D	$M_{\Phi b}$*D	D*$\sigma_{\Phi b}$	Avg. $M_{\Phi b}$	Avg. $\sigma_{\Phi b}$	$\sigma_{\Phi b}/\sigma_{\Phi n}$	$(M_{\Phi b}-M_{\Phi n})/\sigma_{\Phi n}$	Overfill Factor & Quadrant (R_a)	Renourish-ment Factor (R_j)
H-1	NJV- 745	2.41	0.36	12.5	30.10	4.46			0.78	0.15		
	NJV- 799	2.22	0.66	14	31.14	9.26			1.44	-0.25		
	NJV- 452	2.61	0.39	12	31.34	4.63			0.84	0.59		
H-1 Totals	NJV-745,799&452	7.24	1.40	38.5	92.58	18.35						
H-1 Average	NJV-						2.40	0.48	1.04	0.14	1.25/Q1	1.2/1
H-1 Average*	745,799&452						2.41	0.47	1.02	0.16	1.25/Q1	1.2/1
H-2	NJV-185	2.25	0.65	14	31.50	9.03			1.40	-0.20		
	NJV-746	2.51	0.33	17	42.74	5.61			0.72	0.38		
H-2 Totals	NJV-185&746	4.76	0.98	31	74.24	14.64						
H-2 Average	NJV-185&746						2.39	0.47	1.03	0.12	1.2/1	1.2/1
H-2 Average*							2.38	0.49	1.06	0.09	1.15/Q1	1/1
H-3	NJV-747	2.38	0.41	16	38.02	6.62			0.90	0.08		
	NJV-797	2.57	0.43	11	28.27	4.71			0.93	0.50		
H-3 Totals	NJV-747&797	4.95	0.84	27	66.29	11.33						
H-3 Average	NJV-747&797						2.46	0.42	0.91	0.25	1.6/Q4	1.4/1
H-3 Average*							2.47	0.42	0.92	0.29	1.75/Q4	1.5/1
H-4	NJV-187	2.43	0.67	16	38.85	10.69			1.45	0.19		
	NJV-800	2.42	0.66	12	28.98	7.86			1.42	0.16		
H-4 Totals	NJV-187&800	4.84	1.32	28	67.83	18.55						
H-4 Average	NJV-187&800						2.42	0.66	1.44	0.18	1.3/Q1	0.7/1
H-4 Average*							2.42	0.66	1.44	0.18	1.3/Q1	0.9/1
Average H-1 to H-4 (Weighted by depth)							2.42	0.51	1.10	0.17	1.25/Q1	1.2/1
Average H-1 to H-4*(No weighting)							2.42	0.51	1.11	0.18	1.25/Q1	1.2/1

Notes:
1 - Overfill (R_a) and Renourishment (R_j)Factors calculated using isolines for adjusted overfill and renourishment, (Shore Protection Manual, 1984).
2 - Overfill (R_a) and Renourishment (R_j) Factors calculated using "Beach Nourishment Overfill Ratio and Volume" equations contained in Coastal Engineering Manual, 2004.

Overfill/Renourishment Factors[2]	
R_a	1.17
R_j	1.06

NORTH WILDWOOD BEACH RESTORATION PROJECT

CURRENT SUMMARY SHEETS

North Wildwood Native Beach parameters established using composite of surface and SPT samples values - See Table 2.5

A. Native Beach Material Parameters

From Beach Lines - WW-1, WW-2 and WW-2B, surface samples for total line - BC+200 to El.-18, and 0 to 4' SPT samples for NJB -1, 2 and 3

(See **Table 2.5** for additional information and details)

$M_{\Phi n}$ (Average) = 2.34 Subscript "n" indicates native beach material property.

$\sigma_{\Phi n}$ (Average) = 0.46 Subscript "b" indicates borrow material property.

B. Individual Borrow Area Summaries

Table 2.1B Hereford Inlet Borrow Areas (Considering compositing by volume using depth based M_{Φ} & σ_{Φ} values)

Borrow Area & Comments	Vibracores	Mean Median Diameter in Phi units (weighted) $M_{\Phi b}$ Mean Median	Mean Standard Deviation in Phi units (weighted) $\sigma_{\Phi b}$	Est. Area (SF*10^6)	Est. Borrow Depth D	Estimated Volume (V)	V*$M_{\Phi b}$	V*$\sigma_{\Phi b}$	$\sigma_{\Phi b}/\sigma_{\Phi n}$	$\frac{(M_{\Phi b}-M_{\Phi n})}{\sigma_{\Phi n}}$	Overfill Factor & Quadrant (R_a)	Renourish-ment Factor (R_j)
H-1	NJV-745, 452 & 799	2.40	0.48	2.55	15	1415000	3396000	674955	1.04	0.13	1.25/Q1	1.2/1
H-2	NJV-185 & 746	2.39	0.47	4.25	16	2516741	6015010	1187902	1.03	0.11	1.2/Q1	1.2/1
H-3	NJV-747 & 797	2.46	0.42	1.80	14	932296	2293449	391564	0.91	0.26	1.6/Q4	1.4/1
H-4	NJV-187 & 800	2.42	0.66	1.71	14	884074	2139459	586141	1.44	0.17	1.25/Q1	0.7/1
			ΣVolumetric Values		59	5748111	13843919	2840562				
							Avg M_{Φ} = $\Sigma V*M_{\Phi}/\Sigma V$ =	2.41				
							Avg σ_{Φ} = $\Sigma V*\Phi_{\sigma}/\Sigma V$=	0.49	1.07	0.15	1.2/Q1	1.2/1

Overfill/Renourishment Factors[2]	
R_a	1.16
R_j	1.09

Notes:

1 - Overfill (R_a) and Renourishment (R_j)Factors calculated using isolines for adjusted overfill and renourishment, (Shore Protection Manual, 1984).

2 - Overfill (R_a) and Renourishment (R_j) Factors calculated using "Beach Nourishment Overfill Ratio and Volume" equations contained in Coastal Engineering Manual, 2004.

NORTH WILDWOOD BEACH RESTORATION PROJECT

CURRENT SUMMARY SHEETS

North Wildwood Native Beach parameters established using composite of surface and SPT samples values - See Table 2.5

A. Native Beach Material Parameters

From Beach Lines - WW-1, WW-2 and WW-2B, surface samples for total line - BC+200 to El.-18, and 0 to 4' SPT samples for NJB -1, 2 and 3

(See **Table 2.5** for additional information and details)

$M_{\Phi n}$ (Average) 2.34 — Subscript "n" indicates native beach material property.

$\sigma_{\Phi n}$ (Average) 0.46 — Subscript "b" indicates borrow material property.

B. Individual Borrow Area Summaries

Table 2.1C Hereford Inlet Borrow Areas (Considering compositing by volume with no weighting by depth)

Borrow Area & Comments	Vibracores	Mean Median Diameter in Phi units (weighted) $M_{\Phi b}$	Mean Standard Deviation in Phi units (weighted) $\sigma_{\Phi b}$	Est. Area ($SF*10^6$)	Est. Borrow Depth D	Estimated Volume (V)	$V*M_{\Phi b}$	$V*\sigma_{\Phi b}$	$\sigma_{\Phi b}/\sigma_{\Phi n}$	$\frac{(M_{\Phi b}-M_{\Phi n})}{\sigma_{\Phi n}}$	Overfill Factor & Quadrant (Ra)	Renourish-ment Factor (Rj)
H-1	NJV-745, 452 & 799	2.41	0.47	2.55	15	1415000	3416753	662126	1.02	0.16	1.25/Q1	1.3/1
H-2	NJV-185 & 746	2.38	0.49	4.25	16	2516741	5994876	1226911	1.06	0.09	1.2/Q1	1.2/1
H-3	NJV-747 & 797	2.47	0.42	1.80	14	932296	2305569	392497	0.92	0.29	1.75/Q4	1.5/1
H-4	NJV-187 & 800	2.42	0.66	1.71	14	884074	2140785	584948	1.44	0.18	1.3/Q1	0.8/1
		ΣVolumetric Values		10.30	59	5748111	13857984	2866481				
						Avg $M_{\Phi b} = \Sigma V*M_{\Phi b}/\Sigma V$ =		2.41				
						Avg $\sigma_{\Phi b} = \Sigma V*\sigma_{\Phi b}/\Sigma V$=		0.50	1.08	0.15	1.2/Q1	1.2/1

Notes:
1 - Overfill (R_a) and Renourishment (R_j)Factors calculated using isolines for adjusted overfill and renourishment, (Shore Protection Manual, 1984).
2 - Overfill (R_a) and Renourishment (R_j) Factors calculated using "Beach Nourishment Overfill Ratio and Volume" equations contained in Coastal Engineering Manual, 2004.

Overfill/Renourishment Factors[2]	
R_a	1.15
R_j	1.06

Table 2.1D - Summary - Total values for Tables 2.1A, 2.1B and 2.1C

Description	Avg. $M_{\Phi b}$	Avg. $\sigma_{\Phi b}$	$\sigma_{\Phi b}/\sigma_{\Phi n}$	$\frac{(M_{\Phi b}-M_{\Phi n})}{\sigma_{\Phi n}}$	Overfill Factor & Quadrant (Ra)	Renourish-ment Factor (Rj)
H-1 thru H-4 Values by method of Note 1	2.42	0.51	1.10	0.17	1.25/Q1	1.2/1
H-1 thru H-4 Values by method of Note 2	2.42	0.51	1.11	0.18	1.25/Q1	1.2/1
H-1 thru H-4 Values by volumetric method with weighted (Note 1) values for individual areas	2.41	0.49	1.07	0.15	1.2/Q1	1.2/1
H-1 thru H-4 Values by volumetric method with unweighted (Note 2) values for individual areas	2.41	0.50	1.08	0.15	1.2/Q1	1.2/1

NOTES:

1 In **Table 2.1A** shows results which employed the weighted values by height to determine the average values of M_{Φ} & σ_{Φ}.

2 In **Table 2.1A** shows results which employed the unweighted values by height to determine the average values of M_{Φ} & σ_{Φ}.

TABLE 2.2 OFFSHORE BORROW AREA - OS#1
Summary Sheet for Borrow Area Characteristics

Vibracore Designation	Mean Median Diameter in Phi Units (weighted $M_{\Phi b}$)	Mean Standard Deviation in Phi Units (wgt'd $\sigma_{\Phi b}$)	Depth of Suitable Material (feet)	Assumed Borrow Depth	$M_{\Phi b}$*D	D*$\sigma_{\Phi b}$	($M_{\Phi b}$*D)/D	(D*$\sigma_{\Phi b}$)/D	$\sigma_{\Phi b}/\sigma_{\Phi n}$	$\frac{(M_{\Phi b}-M_{\Phi n})}{\sigma_{\Phi n}}$	Overfill Factor & Quadrant (Ra)	Renourish-ment Factor (Rj)
NJGS-158	2.15	1.35	10	10	21.5	13.50	2.15	1.35	2.93	-0.41	----	----
NJGS-159	2.36	0.61	10	10	23.6	6.10	2.36	0.61	1.33	0.04	----	----
	Totals -Borrow Area OS#1			20	45.1	19.60						
	Wtd. Average-Borrow Area OS#1						2.26	0.98	2.13	-0.18	1.35/Q2	1/10
	Check - using individual weighted averages for each vibrocore						2.26	0.98	2.13	-0.18	1.35/Q2	1/10

Overfill/Renourishment Factors[2]	
R_a	1.03
R_j	0.14

TABLE 2.3 OFFSHORE BORROW AREA - OS#2
Summary Sheet for Borrow Area Characteristics

Vibracore Designation	Mean Median Diameter in Phi Units (weighted $M_{\Phi b}$)	Mean Standard Deviation in Phi Units (wgt'd $\sigma_{\Phi b}$)	Depth of Suitable Material (feet)	Assumed Borrow Depth	$M_{\Phi b}$*D	D*$\sigma_{\Phi b}$	($M_{\Phi b}$*D)/D	(D*$\sigma_{\Phi b}$)/D	$\sigma_{\Phi b}/\sigma_{\Phi n}$	$\frac{(M_{\Phi b}-M_{\Phi n})}{\sigma_{\Phi n}}$	Overfill Factor & Quadrant (Ra)	Renourish-ment Factor (Rj)
NJGS-147	1.64	1.07	10	10	16.36	10.72	1.64	1.07	2.33	-1.53	----	----
NJGS-148	1.42	1.43	10	10	14.20	14.34	1.42	1.43	3.12	-2.00	----	----
	Totals -Borrow Area OS#2			20	30.56	25.06						
	Wtd. Average-Borrow Area OS#2						1.53	1.25	2.72	-1.77	1.22/Q2	Stable
	Check - using individual weighted averages for each vibrocore						1.53	1.25	2.72	-1.77	1.22/Q2	Stable

Overfill/Renourishment Factors[2]	
R_a	1.12
R_j	0.007

TABLE 2.4 OFFSHORE BORROW AREA - OS#3
Summary Sheet for Borrow Area Characteristics

Vibracore Designation	Mean Median Diameter in Phi Units (weighted $M_{\Phi b}$)	Mean Standard Deviation in Phi Units (wgt'd $\sigma_{\Phi b}$)	Depth of Suitable Material (feet)	Assumed Borrow Depth	$M_{\Phi b}$*D	D*$\sigma_{\Phi b}$	($M_{\Phi b}$*D)/D	(D*$\sigma_{\Phi b}$)/D	$\sigma_{\Phi b}/\sigma_{\Phi n}$	$\frac{(M_{\Phi b}-M_{\Phi n})}{\sigma_{\Phi n}}$	Overfill Factor & Quadrant (Ra)	Renourish-ment Factor (Rj)
NJV-34	1.29	0.54	9	9	11.59	4.89	1.29	0.54	1.18	-2.29	----	----
NJV-45	2.08	1.78	9	9	18.75	16.05	2.08	1.78	3.88	-0.56	----	----
NJV-48	1.28	0.26	12.2	10	12.79	2.57	1.28	0.26	0.56	-2.31	----	----
NJV-49	1.64	0.59	14	10	16.42	5.93	1.64	0.59	1.29	-1.52	----	----
NJV-51	1.19	0.87	14	10	11.92	8.74	1.19	0.87	1.90	-2.50	----	----
	Totals -Borrow Area OS#3			48	71.47	38.19						
	Wtd. Average-Borrow Area OS#3						1.49	0.80	1.73	-1.85	1.02/Q2	1/18
	Check - using individual weighted averages for each vibracore						1.50	0.81	1.76	-1.83	1.02/Q2	1/18

Overfill/Renourishment Factors[2]	
R_a	1.01
R_j	0.06

Notes:
1 - Overfill (R_a) and Renourishment (R_j)Factors calculated using isolines for adjusted overfill and renourishment, (Shore Protection Manual, 1984).
2 - Overfill (R_a) and Renourishment (R_j) Factors calculated using "Beach Nourishment Overfill Ratio and Volume" equations contained in Coastal Engineering Manual, 2004.

STANDARD PENETRATION TEST (SRT) BORING TO BEACH SAMPLE DESIGN VALUE COMPARISON AND DETERMINATION OF COMPOSITE DESIGN VALUES FOR NORTH WILDWOOD BEACH RESTORATION PROJECT

Table 2.5 - NATIVE BEACH DESIGN ANALYSIS FOR NORTH WILDWOOD

Beach Sampling Line	Average Value BC + 200 to EL -18		SPT Boring Number	Depth (Feet)	Average Value SPT Sample Depth Range		Remarks	Depth (Feet)	Composite Value	
	Geometric Mean (M_Φ)	Inclusive Graphic Deviation (σ_Φ)			Geometric Mean (M_Φ)	Inclusive Graphic Deviation (σ_Φ)			M_Φ	σ_Φ
WW-1	2.36	0.46	NVB-1	0-4	2.38	0.40	Combine NVB-1 values with WW-1 values	0-4	2.37	0.43
				0-8	2.47	0.40		0-8	2.41	0.40
				0-12	2.54	0.41		0-12	2.45	0.41
WW-2	2.24	0.49	NVB-2	0-4	2.37	0.40	Combine NVB-2 values with WW-2 values	0-4	2.30	0.44
				0-8	2.60	0.48		0-8	2.42	0.48
				0-12	2.54	0.52		0-12	2.39	0.52
WW-2B	2.23	0.48	NVB-3	0-4	2.46	0.52	Combine NVB-3 values with WW-2B values	0-4	2.35	0.50
				0-8	2.57	0.53		0-8	2.40	0.53
				0-12	2.70	0.51		0-12	2.46	0.51
Average Values	2.27	0.47			**North Wildwood - Native Beach - Overall Composite using WW-1 to WW-2B Surface Samples & NVB-1, NV-2 & NVB-3 (0-4')**				2.34	0.46

Table 2.6 - BORROW AREA DESIGN ANALYSIS (AREA WW/WC)

Beach Sampling Line	Average Value BC + 200 to EL -18		SPT Boring	Depth (feet)	Average Value SPT Sample Depth Range		Remarks	Depth (feet)	Composite Value	
	Geometric Mean (M_Φ)	Inclusive Graphic Deviation (σ_Φ)			Geometric Mean (M_Φ)	Inclusive Graphic Deviation (σ_Φ)			M_Φ	σ_Φ
WW-15	2.38	0.53	NVB-11	0-4	2.53	0.36	Combine NVB-	0-4	2.44	0.43
AVG. WW-15 & -13	2.35	0.51		0-8	2.56	0.38	11 values with	0-8	2.45	0.45
WW-13	2.33	0.48		0-12	2.56	0.47	avg. for WW-15	0-12	2.45	0.49
				0-16	2.46	0.47	& WW-13	0-16	2.41	0.49
WW-13	2.33	0.48	NVB-9	0-4	2.46	0.45	Combine NVB-9	0-4	2.40	0.47
AVG. WW-13 & -10	2.34	0.48		0-8	2.46	0.44	values with avg.	0-8	2.40	0.46
WW-10	2.36	0.48		0-12	2.44	0.50	for WW-13 &	0-12	2.39	0.49
				0-16	2.48	0.48	WW-10	0-16	2.41	0.48
WW-10	2.36	0.48	NVB-7	0-4	2.40	0.41	Combine NVB-7	0-4	2.37	0.44
AVG. WW-10 & -07	2.33	0.48		0-8	2.46	0.36	values with avg.	0-8	2.40	0.42
WW-07	2.30	0.47		0-12	2.44	0.38	for WW-10 &	0-12	2.39	0.43
				0-16	2.45	0.39	WW-07	0-16	2.39	0.43
WW-07	2.30	0.47	NVB-5	0-4	2.47	0.32	Combine NVB-5	0-4	2.37	0.41
AVG. WW-07 & -04	2.28	0.50		0-8	2.47	0.37	values with avg.	0-8	2.37	0.42
WW-04	2.27	0.52		0-12	2.48	0.34	for WW-07 &	0-12	2.38	0.41
				0-16	2.63	0.39	WW-04	0-16	2.45	0.43
WW/WC Overall Composite using Only Beach Samples Calculations & M_Φ=2.34 & σ_Φ=0.46			**Composite using only SPT Samples**				**Composite using WW-15 to WW-4 Surface Samples and NVB-11 to NVB-5 SPT Samples**			
	2.33	0.49		0-4	2.46	0.38		0-4	2.40	0.44
	2.33	0.50		0-8	2.48	0.39		0-8	2.41	0.44
				0-12	2.48	0.42		0-12	2.40	0.45
	$(M_{\Phi b}-M_{\Phi n})/\sigma_{\Phi n}$ =	-0.03	@12' Excavation	$(M_{\Phi b}-M_{\Phi n})/\sigma_{\Phi n}$ =	0.30	Ra=1.7	@12' Excavation	$(M_{\Phi b}-M_{\Phi n})/\sigma_{\Phi n}$ =	0.14	Ra=1.25
	$\sigma_{\Phi b}/\sigma_{\Phi n}$ =	1.07		$\sigma_{\Phi b}/\sigma_{\Phi n}$ =	0.92	Rj=1.5/1		$\sigma_{\Phi b}/\sigma_{\Phi n}$ =	0.99	Rj=1.2/1
		Ra=1.05		0-16	2.51	0.43		0-16	2.42	0.47
WW/WC Beach		Rj=1/1	@16' excavation	$(M_{\Phi b}-M_{\Phi n})/\sigma_{\Phi n}$ =	0.36	Ra=1.9	@16' excavation	$(M_{\Phi b}-M_{\Phi n})/\sigma_{\Phi n}$ =	0.17	Ra=1.25
Sample Composite &				$\sigma_{\Phi b}/\sigma_{\Phi n}$ =	0.93	Rj=1.7/1		$\sigma_{\Phi b}/\sigma_{\Phi n}$ =	1.02	Rj=1.25/1
Using only Beach	$(M_{\Phi b}-M_{\Phi n})/\sigma_{\Phi n}$ =	0.11	Overfill/ Renourishment Factors[2]	0-12'	R_a	1.61	Overfill/ Renourishment Factors[2]	0-12'	R_a	1.18
Samples to compute	σφb/σφn=	1.05			R_j	1.47			Rj	1.16
Avg. σ_Φ& M_Φ for		Ra=1.17		0-16'	R_a	1.76		0-16'	R_a	1.21
Native beach		Rj=1.1/1			R_j	1.54			R_j	1.16

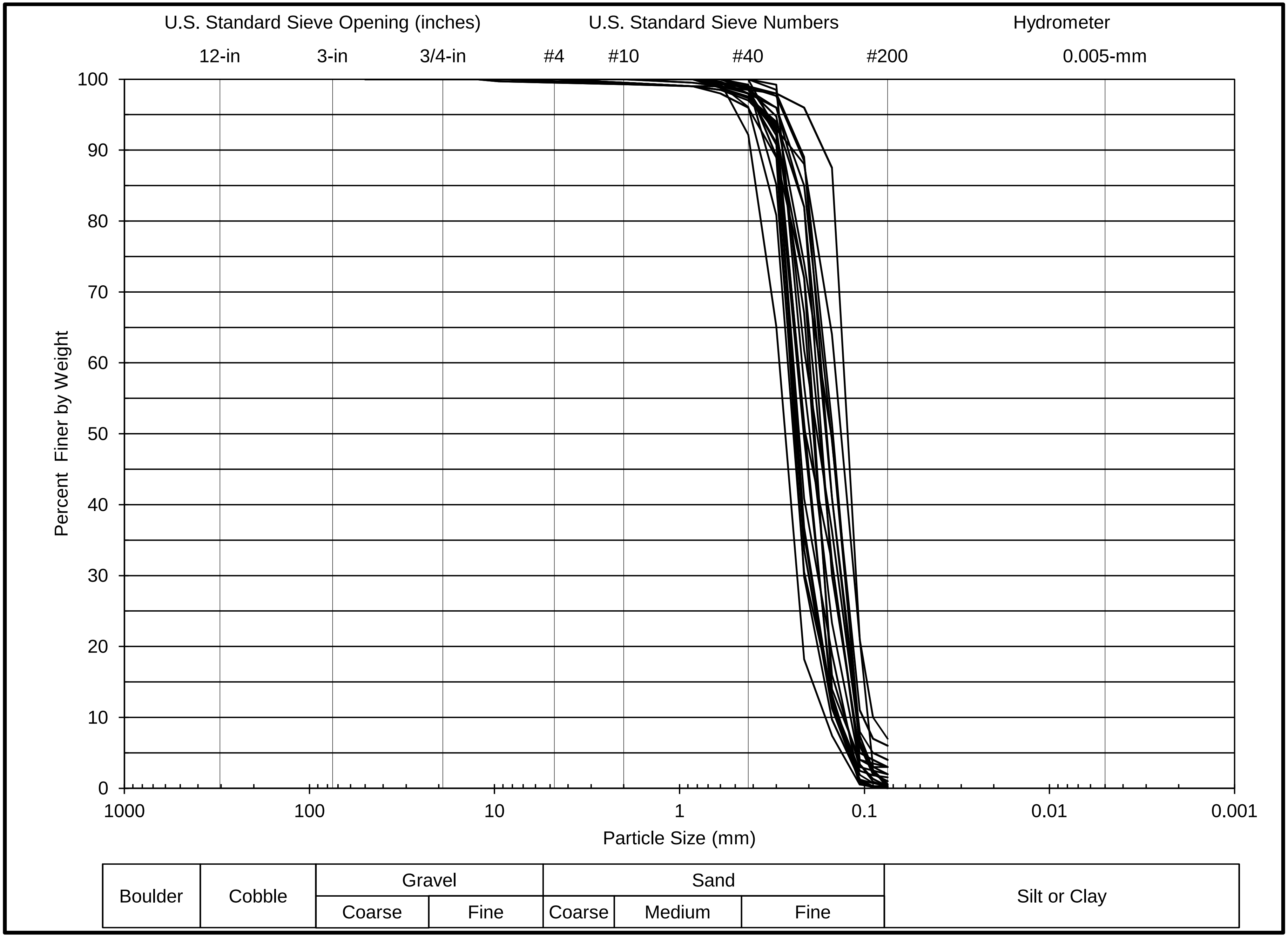
Notes:
1 - Overfill (R_a) and Renourishment (R_j)Factors calculated using isolines for adjusted overfill and renourishment, (Shore Protection Manual, 1984).
2 - Overfill (R_a) and Renourishment (R_j) Factors calculated using "Beach Nourishment Overfill Ratio and Volume" equations contained in Coastal Engineering Manual, 2004.

Statistical Analysis of Gradation Data

Native Beach - North Wildwood

Boring ID	Sample No. Sample Depth	Gravel (%)	Sand (%)			Fines (%)	c_u	Soil Description
			Coarse	Medium	Fine			
WW-1	S1	0.0	0.0	0.0	99.8	0.2		SP
WW1	S3	0.0	0.0	2.5	97.5	0.1		SP
WW1	S4	0.0	0.0	2.1	97.9	0.0		SP
WW1	S5	0.0	0.0	2.0	98.0	0.0		SP
WW2	S6	0.0	0.0	1.1	98.3	0.6		SP
WW2	S7	0.0	0.0	0.8	99.0	0.3		SP
WW2	S9	0.0	0.0	0.8	99.1	0.1		SP
WW2	S10	0.0	0.0	1.5	98.4	0.2		SP
WW2	S11	0.0	0.0	3.9	96.0	0.1		SP
WW2B	S12	0.0	0.0	1.4	98.6	0.0		SP
WW2B	S13	0.0	0.0	0.0	99.6	0.4		SP
WW2B	S15	0.0	0.0	0.0	98.5	1.5		SP
WW2B	S16	0.0	0.0	1.2	98.3	0.5		SP
WW2B	S17	0.0	0.0	7.8	91.9	0.3		SP
WW2B	S18	0.0	0.0	2.7	97.0	0.3		SP
NVB-1	0-4	0.0	0.0	2.0	96.0	2.0		SP
NVB-1	4-8	0.0	0.0	2.0	95.0	3.0		SP
NVB-1	8-12	0.0	0.0	1.0	95.0	4.0		SP
NVB-2	0-4	0.5	0.2	1.8	94.5	3.0		SP
NVB-2	4-8	0.5	0.2	2.3	94.0	3.0		SP
NVB-2	8-12	0.3	0.2	2.0	96.5	1.0		SP
NVB-3	0-4	0.0	0.5	3.5	94.0	2.0		SP
NVB-3	4-8	0.0	0.5	1.0	92.5	6.0		SP-SC
NVB-3	8-12	0.0	0.5	1.0	91.5	7.0		SP-SC
Minimum		0.0	0.0	0.0	91.5	0.0		
Maximum		0.5	0.5	7.8	99.8	7.0		
Median		0.0	0.0	1.6	97.2	0.5		
Mean		0.1	0.1	1.8	96.5	1.5		
Standard Deviation		0.2	0.2	1.6	2.5	2.0		
Variance		0.0	0.0	2.6	6.1	3.8		

Cummulative Grain Size Distribution (GSD) Plot
North Wildwood Beach - Native Beach

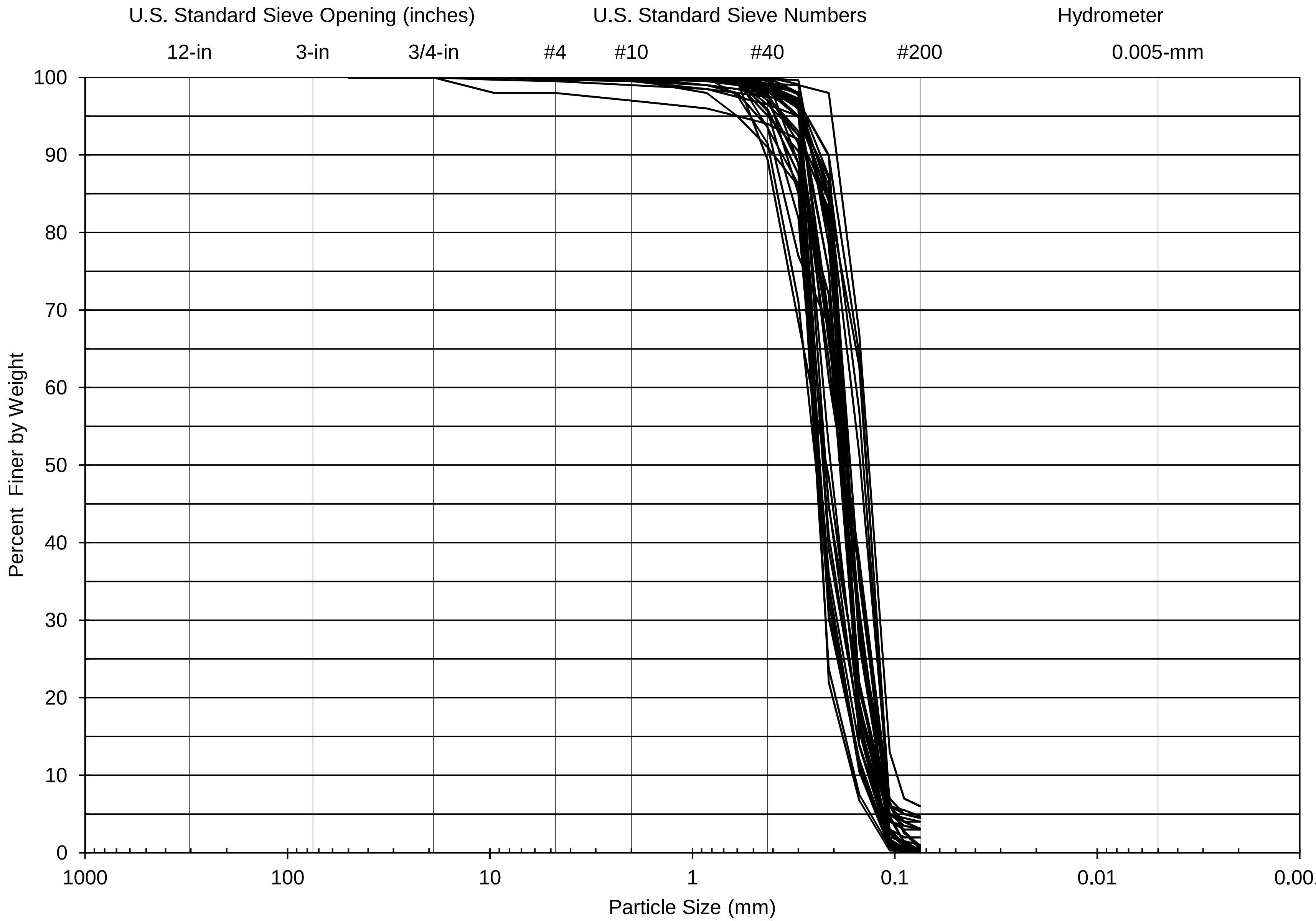


Statistical Analysis of Gradation Data

Borrow Area - Wilwood/Wildwood Crest

Boring ID	Sample No./ Sample Depth	Gravel (%)	Sand (%)			Fines (%)	c_u	Soil Description
			Coarse	Medium	Fine			
WW-4	S19	0.0	0.0	0.5	99.0	0.5		SP
WW-4	S21	0.0	0.0	0.8	99.1	0.1		SP
WW-4	S22	0.0	0.0	2.1	97.8	0.1		SP
WW-4	S23	0.0	0.0	8.5	91.3	0.2		SP
WW-4	S24	0.0	0.0	1.8	98.1	0.2		SP
WW-7	S25	0.0	0.0	0.0	99.8	0.2		SP
WW-7	S27	0.0	0.0	6.5	92.8	0.7		SP
WW-7	S28	0.0	0.0	4.4	95.3	0.4		SP
WW-7	S29	0.0	0.0	10.7	89.0	0.3		SP
WW-7	S30	0.0	0.0	1.8	97.4	0.8		SP
WW-10	S31	0.0	0.0	0.0	99.9	0.2		SP
WW-10	S33	0.0	0.0	0.0	99.8	0.2		SP
WW-10	S34	0.0	0.0	2.1	97.7	0.2		SP
WW-10	S35	0.0	0.0	4.9	94.7	0.4		SP
WW-10	S36	0.0	0.0	3.5	95.5	1.0		SP
WW-13	S37	0.0	0.0	0.2	99.8	0.0		SP
WW-13	S39	0.0	0.0	3.0	96.8	0.2		SP
WW-13	S40	0.0	0.0	4.1	95.7	0.2		SP
WW-13	S41	0.0	0.0	2.5	97.4	0.1		SP
WW-13	S42	0.0	0.0	1.4	98.2	0.4		SP
WW-15	S43	0.0	0.0	0.2	99.6	0.2		SP
WW-15	S45	0.0	0.0	1.3	98.5	0.1		SP
WW-15	S46	0.0	0.0	2.3	97.5	0.1		SP
WW-15	S47	0.0	0.0	1.0	98.7	0.3		SP
NVB-05	0-4	0.0	0.2	0.8	97.0	2.0		SP
NVB-05	4-8	2.0	1.0	3.0	92.0	2.0		SP
NVB-05	8-12	0.5	0.5	1.5	94.5	3.0		SP
NVB-05	12-16	0.0	0.2	0.6	93.2	6.0		SP-SC
NVB-07	0-4	0.0	0.0	1.0	96.0	3.0		SP
NVB-07	4-8	0.0	0.3	1.7	97.0	1.0		SP
NVB-07	8-12	0.0	0.5	2.0	93.5	4.0		SP
NVB-07	12-16	0.0	0.0	1.5	95.5	3.0		SP
NVB-09	0-4	0.0	0.0	0.2	95.3	4.5		SP
NVB-09	4-8	0.0	0.2	1.1	95.7	3.0		SP
NVB-09	8-12	0.0	0.3	8.7	87.0	4.0		SP
NVB-09	12-16	0.3	0.2	3.0	92.0	4.5		SP
NVB-11	0-4	0.0	0.0	0.0	96.9	3.1		SP
NVB-11	4-8	0.0	0.0	1.0	96.0	3.0		SP
NVB-11	8-12	0.0	0.1	0.7	94.7	4.5		SP
NVB-11	12-16	0.0	0.0	6.4	88.9	4.7		SP
Minimum		0.0	0.0	0.0	87.0	0.0		
Maximum		2.0	1.0	10.7	99.9	6.0		
Median		0.0	0.0	1.6	96.4	0.5		
Mean		0.1	0.1	2.4	95.9	1.6		
Standard Deviation		0.3	0.2	2.6	3.2	1.8		
Variance		0.1	0.0	6.7	10.1	3.1		

Cummulative Grain Size Distribution (GSD) Plot
Wildwood/Wildwood Crest Borrow Area



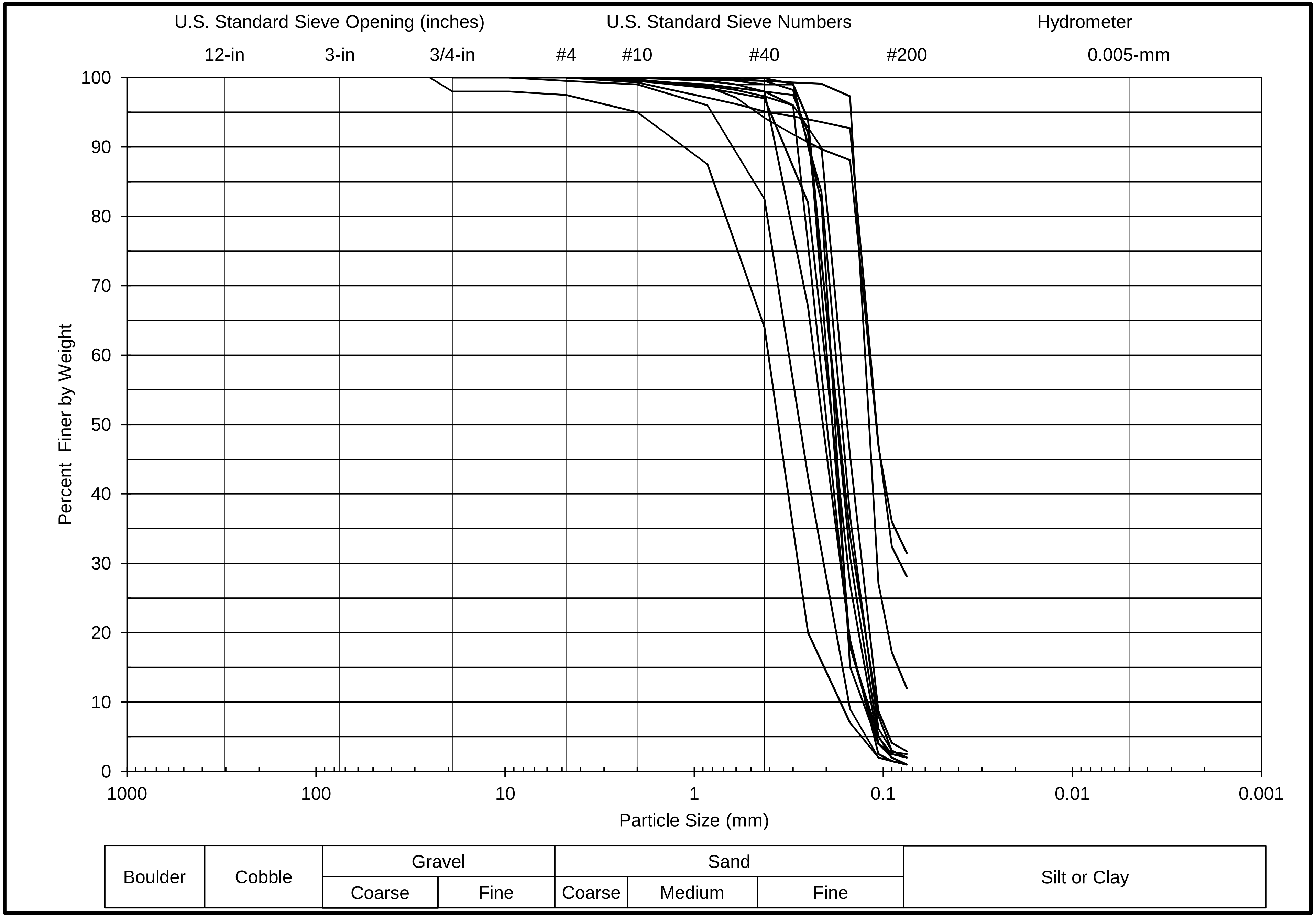
Boulder	Cobble	Gravel		Sand			Silt or Clay
		Coarse	Fine	Coarse	Medium	Fine	

Statistical Analysis of Gradation Data

Borrow Area - Hereford Inlet - H-1

Boring ID	Sample No.	Gravel (%)	Sand (%)			Fines (%)	c_u	Soil Description
			Coarse	Medium	Fine			
NJV-799	0-5	0.0	0.5	2.5	96.0	1.0		SP
NJV-799	5-10	0.5	0.5	16.5	81.5	1.0		SP
NJV-799	10-15	0.0	0.5	1.5	97.0	1.0		SP
NJV-799	15-20	2.5	2.5	31.0	63.0	1.0		SP
NJV-745	0-5	0.0	0.5	1.5	96.0	2.0		SP
NJV-745	5-10	0.0	0.0	2.0	96.0	2.0		SP
NJV-745	10-15	0.0	0.0	1.0	98.0	1.0		SP
NJV-745	15-20	0.0	0.0	1.0	97.0	2.0		SP
NJV-452	0-5	0.0	0.3	2.4	94.4	2.9		SP
NJV-452	5-10	0.0	0.0	0.1	97.4	2.5		SP
NJV-452	10-11.3	0.0	0.0	0.5	97.5	2.0		SP
NJV-452	11.3-14.1	0.0	0.1	0.4	87.5	12.0		SC
NJV-452	15-16.8	0.0	0.7	4.2	67.0	28.1		SC
NJV-452	16.8-18.6	0.0	0.3	5.5	62.7	31.5		SC
Minimum		0.0	0.0	0.1	62.7	1.0		
Maximum		2.5	2.5	31.0	98.0	31.5		
Median		0.0	0.3	1.8	96.0	2.0		
Mean		0.2	0.4	5.0	87.9	6.4		
Standard Deviation		0.7	0.6	8.6	13.6	10.3		
Variance		0.5	0.4	73.3	186.3	106.4		

Cummulative Grain Size Distrubtion (GSD) Plot
Hereford Inlet - H-1

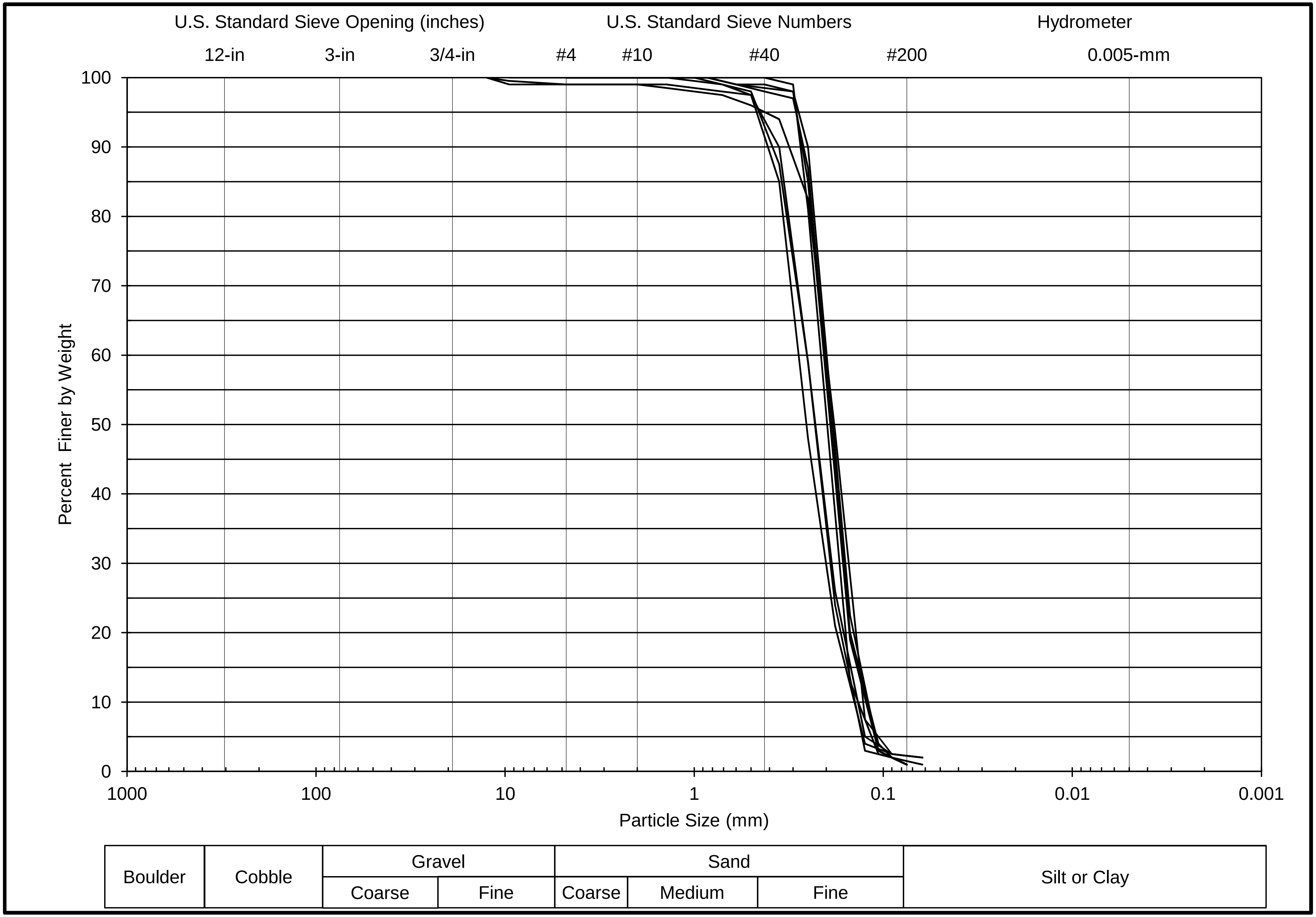


Statistical Analysis of Gradation Data

Borrow Area - Hereford Inlet - H-2

Boring ID	Sample Depth	Gravel (%)	Sand (%) Coarse	Sand (%) Medium	Sand (%) Fine	Fines (%)	c_u	Soil Description
NJV-185	2.8-3.3	0.0	0.0	5.6	92.9	1.5		SP
NJV-185	6.3-6.5	0.0	0.0	7.0	91.5	1.5		SP
NJV-185	7.8-8.3	0.0	0.0	8.4	89.3	2.3		SP
NJV-185	10.6-10.9	1.0	0.0	5.1	91.7	2.3		SP
NJV-185	13.1-13.7	1.0	0.0	3.9	92.8	2.3		SP
NJV-746	0-5	0.0	0.0	2.0	97.0	1.0		SP
NJV-746	5-10	0.0	0.0	0.0	99.0	1.0		SP
NJV-746	10-15	0.0	0.0	1.5	97.5	1.0		SP
NJV-746	15-20	0.0	0.0	1.0	98.0	1.0		SP
Minimum		0.0	0.0	0.0	89.3	1.0		
Maximum		1.0	0.0	8.4	99.0	2.3		
Median		0.0	0.0	3.9	92.9	1.5		
Mean		0.2	0.0	3.8	94.4	1.5		
Standard Deviation		0.4	0.0	2.9	3.5	0.6		
Variance		0.2	0.0	8.4	12.1	0.3		

Cummulative Grain Size Distrubtion (GSD) Plot
Hereford Inlet - H-2

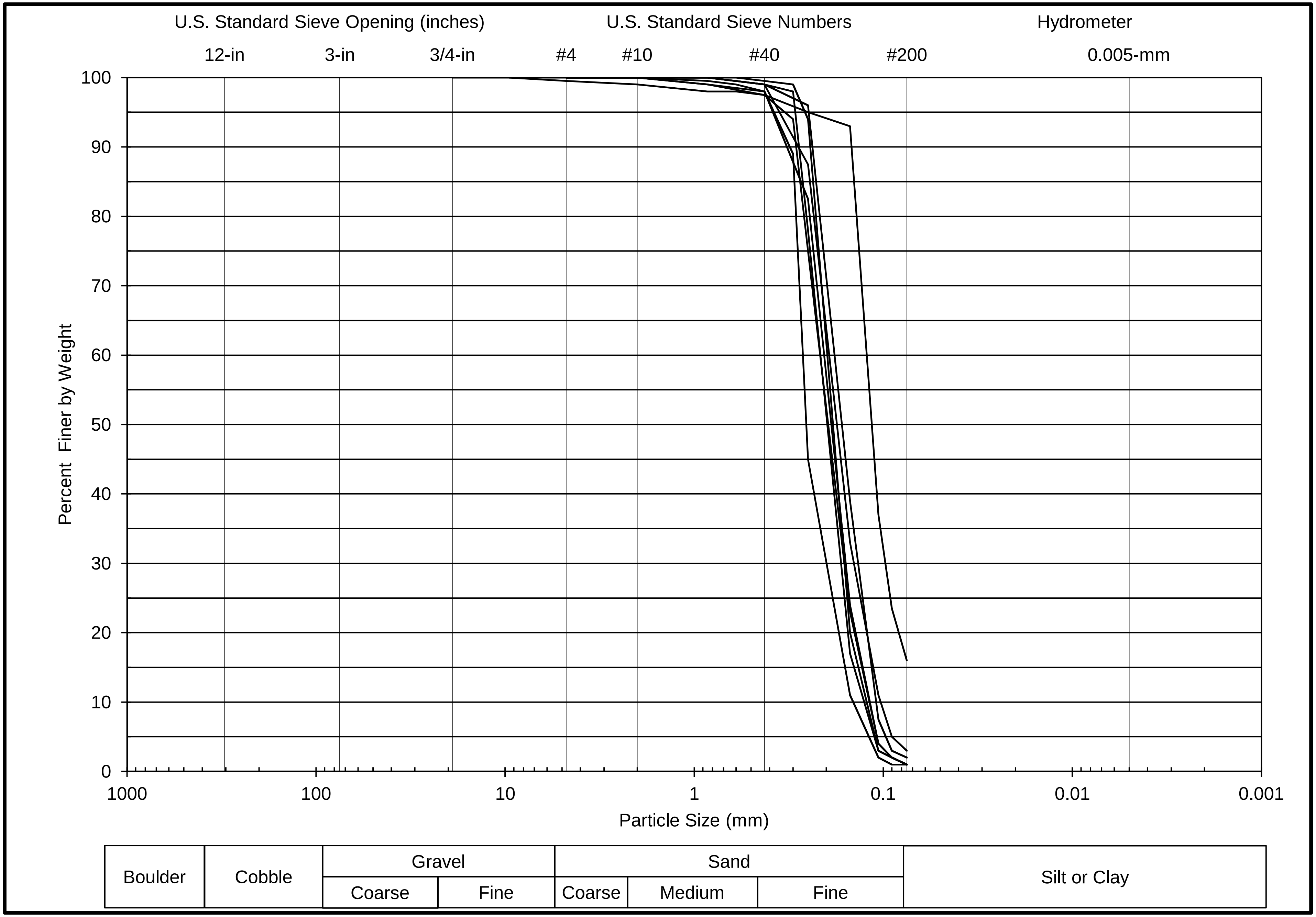


Statistical Analysis of Gradation Data

Borrow Area - Hereford Inlet - H-3

Boring ID	Sample Depth	Gravel (%)	Sand (%)			Fines (%)	c_u	Soil Description
			Coarse	Medium	Fine			
NJV-797	0-5	0.0	0.0	2.0	97.0	1.0		SP
NJV-797	5-10	0.0	0.0	1.0	97.0	2.0		SP
NJV-797	10-15	0.0	0.0	1.0	96.0	3.0		SP
NJV-797	15-20	0.0	0.0	2.5	81.5	16.0		SC
NJV-747	0-5	0.5	0.5	1.5	96.5	1.0		SP
NJV-747	5-10	0.0	0.0	2.0	97.0	1.0		SP
NJV-747	10-15	0.0	0.0	1.0	98.0	1.0		SP
NJV-747	15-20	0.0	0.0	0.5	98.5	1.0		SP
Minimum		0.0	0.0	0.5	81.5	1.0		
Maximum		0.5	0.5	2.5	98.5	16.0		
Median		0.0	0.0	1.3	97.0	1.0		
Mean		0.1	0.1	1.4	95.2	3.3		
Standard Deviation		0.2	0.2	0.7	5.6	5.2		
Variance		0.0	0.0	0.5	31.2	27.1		

Cummulative Grain Size Distrubtion (GSD) Plot
Hereford Inlet - H-3

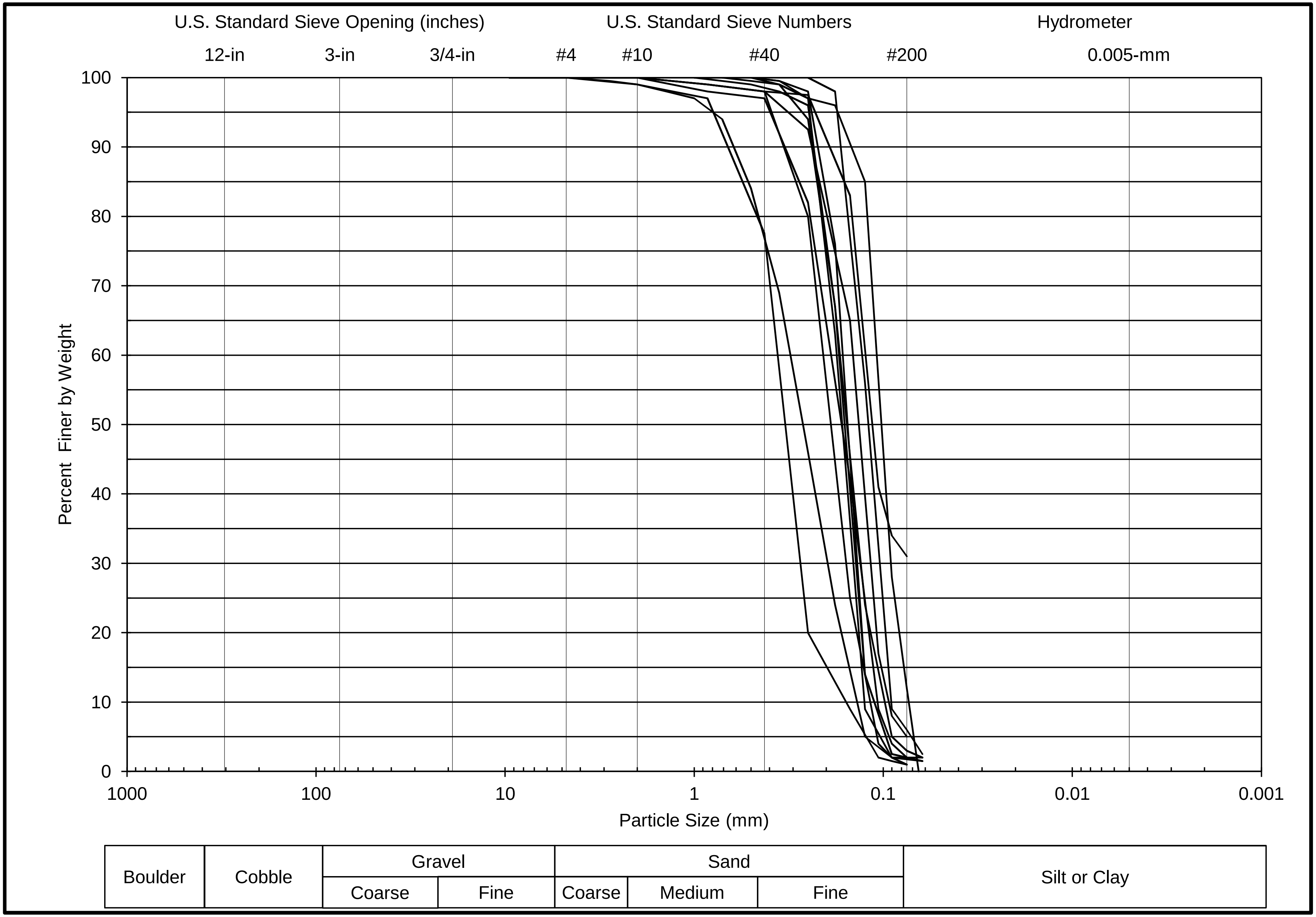


Statistical Analysis of Gradation Data

Borrow Area - Hereford Inlet (H-4)

Boring ID	Sample Depth.	Gravel (%)	Sand (%) Coarse	Sand (%) Medium	Sand (%) Fine	Fines (%)	c_u	Soil Description
NJV-187	2-2.5	0.0	0.0	0.7	87.3	12.0		SP-SC
NJV-187	4.5-4.8	0.0	0.0	1.5	95.5	3.0		SP
NJV-187	8-8.5	0.0	0.0	0.2	98.0	1.8		SP
NJV-187	10.2-10.5	0.0	0.0	0.2	97.8	2.0		SP
NJV-187	13.2-13.7	0.0	0.0	0.5	97.5	2.0		SP
NJV-187	15-15.3	0.0	1.0	22.1	74.9	2.0		SP
NJV-187	18.2-18.7	0.0	0.0	0.0	94.0	6.0		SP-SC
NJV-800	0-5	0.0	0.0	2.0	97.0	1.0		SP
NJV-800	5-7.5	0.0	0.0	3.0	95.0	2.0		SP
NJV-800	7.5-10	0.0	1.0	21.5	76.5	1.0		SP
NJV-800	10-15	0.0	0.0	2.0	93.0	5.0		SP-SC
NJV-800	15-20	0.0	0.0	2.0	67.0	31.0		SC
Minimum		0.0	0.0	0.0	67.0	1.0		
Maximum		0.0	1.0	22.1	98.0	31.0		
Median		0.0	0.0	1.7	94.5	2.0		
Mean		0.0	0.2	4.6	89.5	5.7		
Standard Deviation		0.0	0.4	8.1	10.7	8.5		
Variance		0.0	0.2	65.1	114.0	72.9		

Cummulative Grain Size Distrubtion (GSD) Plot
Hereford Inlet - H-4

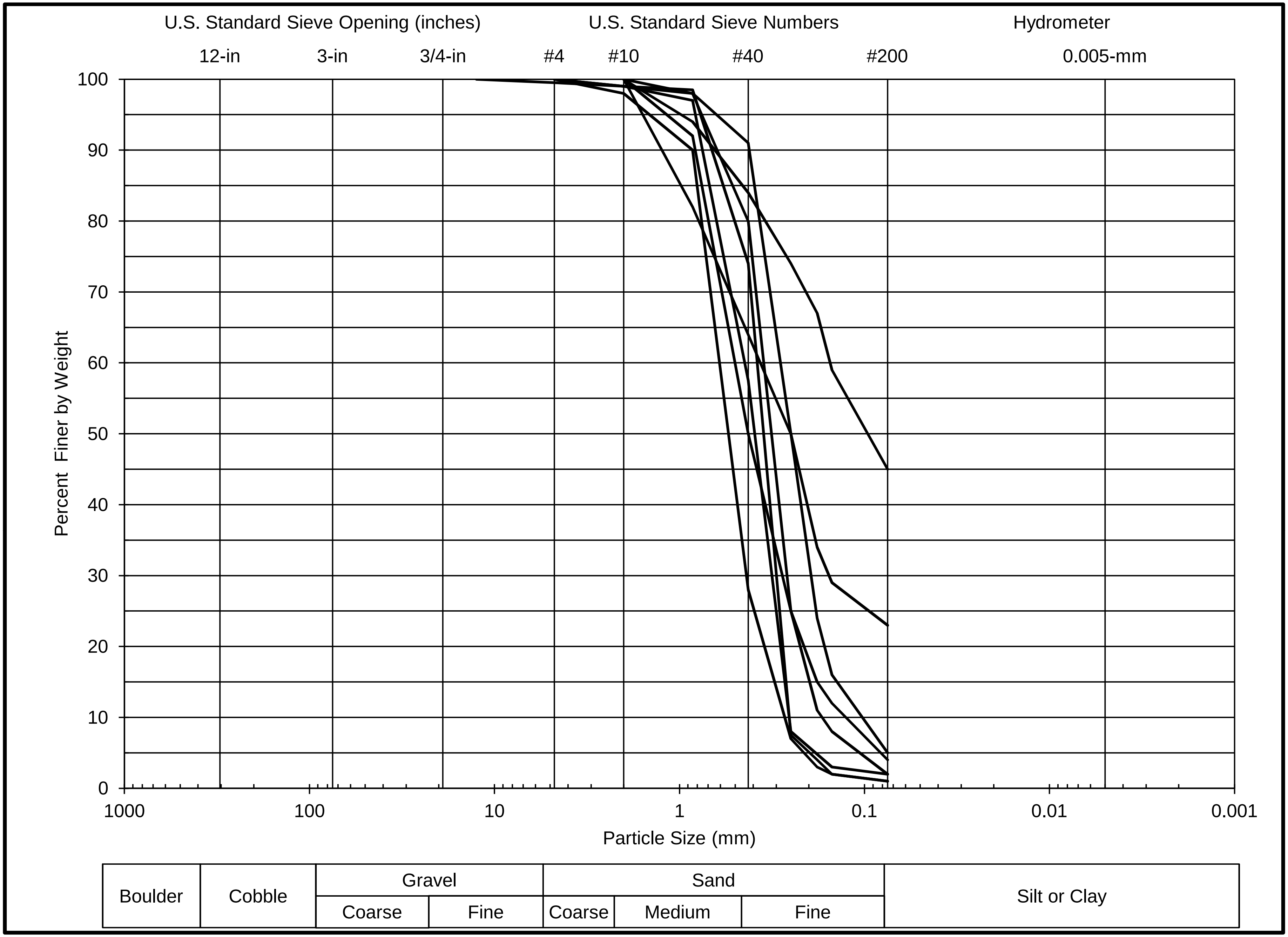


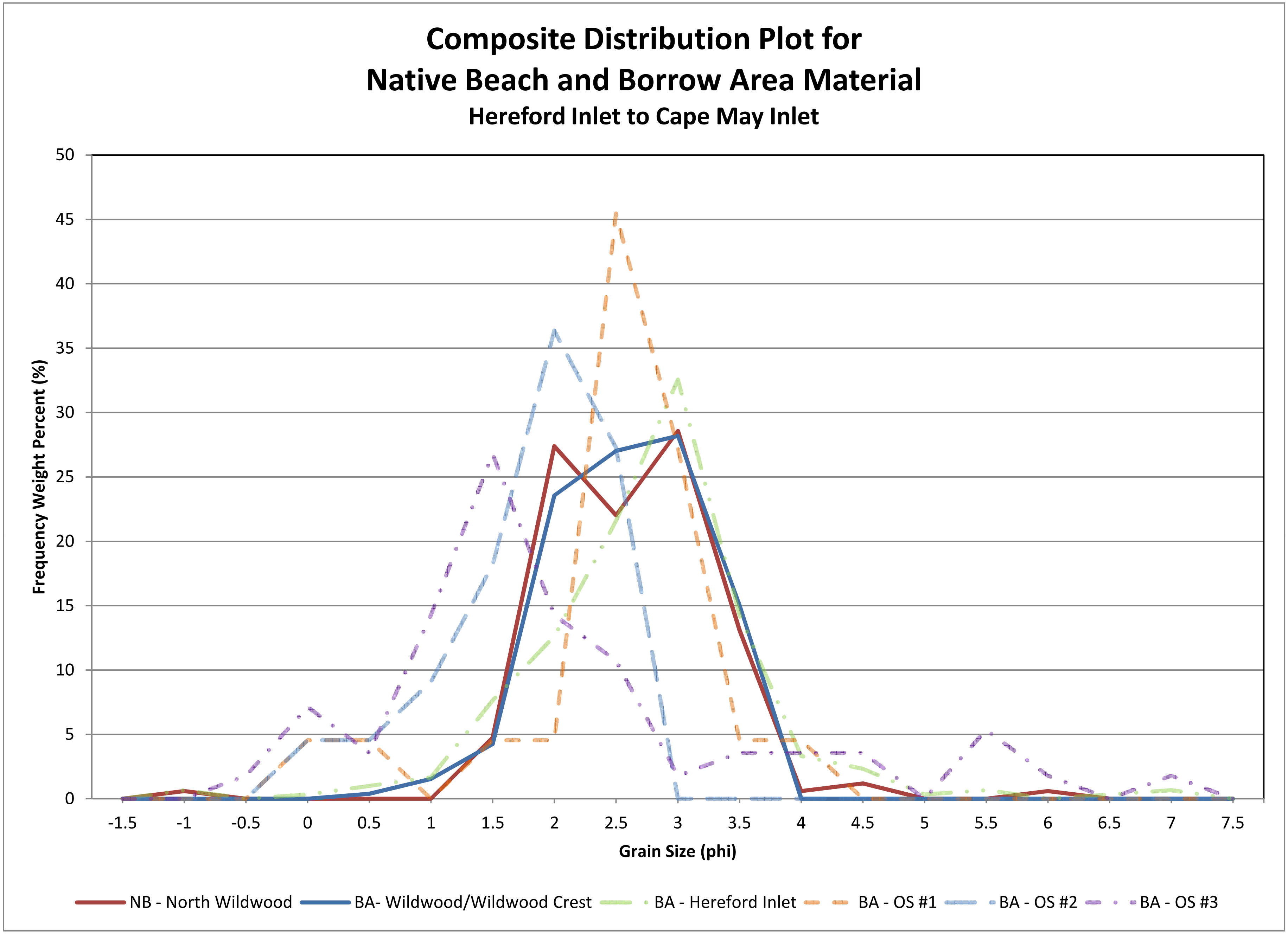
Statistical Analysis of Gradation Data

Off Shore Borrow Area - OS #3

Boring ID	Sample No.	Gravel (%)	Sand (%)			Fines (%)	c_u	Soil Description
			Coarse	Medium	Fine			
NJV-34	S-1	0.5	0.5	41.5	55.5	2.0		SP
NJV-48	S-1	0.0	1.0	25.0	73.0	1.0		SP
NJV-45	S-1	0.0	0.0	36.0	41.0	23.0		SC-SM
NJV-45	S-2	0.0	0.0	9.0	86.0	5.0		SP-SC
NJV-45	S-3	0.0	0.0	16.0	39.0	45.0		SC
NJV-51	S-1	0.0	0.0	50.0	46.0	4.0		SP
NJV-51	S-2	0.0	2.0	70.0	27.0	1.0		SP
NJV-49	S-1	0.0	1.0	19.0	78.0	2.0		SP
Minimum		0.0	0.0	9.0	27.0	1.0		
Maximum		0.5	2.0	70.0	86.0	45.0		
Median		0.0	0.3	30.5	50.8	3.0		
Mean		0.1	0.6	33.3	55.7	10.4		
Standard Deviation		0.2	0.7	20.2	21.1	15.8		
Variance		0.0	0.5	409.1	446.8	249.1		

Cummulative Grain Size Distribution (GSD) Plot
Off-Shore Borrow Area #3





Composite Distribution Plot for
Native Beach and Borrow Area Material
Hereford Inlet to Cape May Inlet
Frequency Weight Percent (%)
50
45
40
35
30
25
20
15
10
5
0
-1.5
-1
-0.5
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
5.5
6
6.5
7
7.5
Grain Size (phi)
NB - North Wildwood
BA- Wildwood/Wildwood Crest
BA - Hereford Inlet
BA - OS #1
BA - OS #2
BA - OS #3

GEOTECHNICAL APPENDIX

SECTION 3

Description of the Design Methodology

This section presents additional information and details on the method developed by the Philadelphia District to convert available gradation data to design values for comparison of native beach and borrow materials.

Method for Determining Median Grain Size (M_{Φ}) and Mean Standard Deviation (σ_{φ})

The method described herein was developed by the Geotechnical Section of the Philadelphia District of the U.S. Army Corps of Engineers (NAP-EC-EG) during the period 2005 to 2006. At the time, NAP-EC-EG was tasked to perform a borrow area investigation for the proposed beach restoration in North Wildwood, New Jersey.

A limited amount of geotechnical data was available from past investigations of the North Wildwood, Wildwood, Wildwood Crest and Lower Township beaches and borrows areas located offshore and in Hereford Inlet adjacent to the North Wildwood beach. Additional investigations were preformed to obtain supplemental information required for the geotechnical analysis of the target site and potential source areas. Recovered soil samples were subjected to visual identification and laboratory gradation analyses to determine the type of soil material present and grain size distribution of the recovered samples.

The data evaluated was primarily in the form of gradation curves plotted on 5 Cycle semi-log paper (ENG FORM 2087). The gradation curves were available for beach samples which had been taken along selected lines and that had been obtained in a number of potential borrow areas considered for the North Wildwood beach restoration site. As the study progressed it was determined that additional sampling was required to accurately determine the native beach and borrow area material design parameters. These investigations consisted of several additional vibracores taken in the Hereford Inlet borrow area and 12 Standard Penetration Test (SPT) borings drilled along the beach between North Wildwood and Lower Township.

The available data was converted to phi grain size designations in order to develop design parameters for the native beach and borrow area materials. This was accomplished using the following procedure:

1. The grain size in millimeters (mm) for the 95, 84, 75, 50, 25, 16, and 5 percentages coarser by weight of each sample (D_{95}, D_{84}, D_{75}, etc.) were determined from the individual grain size curve.

2. These values were converted to phi sizes using the relationship: $\varphi = -\log_2 D$.

3. The mean grain size and standard deviation for each sample were then computed as follows:

a. The mean grain size (M_φ) of each sample was determined using the relationship:

$M_\varphi = (\varphi_{16}+\varphi_{50}+\varphi_{84})/3$ where φ_{16}, φ_{50} and φ_{84} are the 16%, 50% and 84% coarser by weight phi sizes, respectively.

b. The standard deviation for each sample was determined using the relationship:

$\sigma_\varphi = (\varphi_{84}-\varphi_{16})/4 + (\varphi_{95}+\varphi_{5})/6$.

4. The mean grain size and standard deviation for each group of samples can then be determined by averaging the values of each test, i.e. $M_{\varphi 1to4} = (M_\varphi 1+ M_\varphi 2+ M_\varphi 3+ M_\varphi 4)/4$ and σ_1 to $\sigma_4 = (\sigma_1+\sigma_2+\sigma_3+\sigma_4)/4$. The weighted averages of these values can also be determined by using the height or volume of the sample portion represented by the individual result as a multiplier of that result then summing and averaging to obtain the weighted value for a given sample or area. The following example shows this methodology for an individual vibracore with varying heights of samples:

$$\text{Weighted } M_{\varphi 1to4} = (h_1*M_{\varphi 1}+h_2*M_{\varphi 2}+ h_3* M_{\varphi 3}+ h_4* M_{\varphi 4})/ (h_1 +h_2 + h_3 + h_4).$$

The EXCEL program developed to accomplish these calculations is illustrated on **Tables 3.1** and **3.2**. **Table 3.1** provides typical output from the program for Vibracore NJV-454. **Table 3.2** provides the input for the individual cells for the same vibracore. It is noted that the program, once set up for an individual sample, can be copied and pasted in subsequent sections of the worksheet for new samples with minimal effort. However, care must be taken in entering new data and spot checking of computed values is required to assure valid results.

The method employed to determine the overall median grain sizes and standard deviations for complete sampling lines, vibracores, STP boring borings, native beach area and complete borrow sources are obtained in a similar manner as was shown on the **Table 3.1**.

Once the values were obtained for the native beach material and each individual borrow area investigated, the values for the overfill (R_a) and renourishment (R_j) factors were determined using the nomograph method of the 2008 Edition of the Coastal Engineering Manual and 1984 Edition of the Shore Protection Manual. A further refinement in determining the R_a and R_j factors consisted of using the applicable portion of the Automated Coastal Engineering System (ACES) methodology with inputs of the M_Φ and σ_Φ values for the native beach and borrow areas to obtain the values of R_a and R_j for each individual borrow area. These values were then checked against those obtained using the nomograph method. Agreement between the two methods is not exact, but is considered adequate for the scope of this study. It is noted that the higher values of the renourishment factors (those obtained by the nomograph method) were used to determine the beachfill costs for the cost analysis.

Table 3.1

Calculated Values of Φ, MΦ, σΦ and Wtd. Values of same from mm grain size Values

Vibrocore Number	Sample Number and Depth	Grain Size	D - % Coarser	Phi Diameter	Geometric Mean (phi16+phi50+phi84)/3 (M_{Φ})	Inclusive Graphic Deviation (phi84-phi16)/4+phi95-phi05)/6 (σ_{Φ})	Depth represented by sample - D (ft)	Wtd, GM GM*D D*M_{Φ}	Wtd. IGD IGD*D D*σ_{Φ}
NJV - 455	S -1 0.0 - 5.0.	0.12	95	3.059					
		0.13	84	2.943					
		0.15	75	2.737					
		0.17	50	2.556	2.540	0.392	5	12.700	1.958
		0.2	25	2.322					
		0.23	16	2.120					
		0.26	5	1.943					
NJV - 455	S - 2 5.5 - 10.0	0.13	95	2.943					
		0.16	84	2.644					
		0.17	75	2.556					
		0.21	50	2.252	2.211	0.485	5	11.054	2.423
		0.26	25	1.943					
		0.3	16	1.737					
		0.38	5	1.396					
NJV - 455	S - 3 10.0 - 15.0	0.13	95	2.943					
		0.15	84	2.737					
		0.17	75	2.556					
		0.24	50	2.059	2.077	0.737	5	10.384	3.686
		0.35	25	1.515					
		0.37	16	1.434					
		0.72	5	0.474					
NJV - 455	S - 4 15.0 - 19.1	0.14	95	2.837					
		0.19	84	2.396					
		0.22	75	2.184					
		0.27	50	1.889	1.906	0.505	5	9.532	2.523
		0.33	25	1.599					
		0.37	16	1.434					
		0.42	5	1.252					
			SUM		8.734	2.118			
			Average		2.184	0.530			
						SUM	20	43.670	10.590
						WTD. AVG.		2.184	0.530

Table 3.2

Formulas used to determine Values of Φ, MΦ, σΦ and Weighted Values of same from mm grain size data

Vibrocore Number	Sample and	Number Depth	Grain Size	D - % Coarser	Phi Diameter	Geometric Mean φ16+φ50+φ84)/3 (M_{Φ})	Inclusive Graphic Deviation $(\varphi_{84}-\varphi_{16})/4+(\varphi_{95}-\varphi_{05})/6$ (σ_{Φ})	Depth represented by sample D (ft)	Wtd, GM GM*D $D*M_{\Phi}$	Wtd. IGD IGD*D $D*\sigma_{\Phi}$
NJV - 455	S -1	0.0 - 5.0.	0.12	95	=-LOG(D4,2)					
			0.13	84	=-LOG(D5,2)					
			0.15	75	=-LOG(D6,2)					
			0.17	50	=-LOG(D7,2)	=AVERAGE(F9,F7,F5)	=(F5-F9)/4+(F4-F10)/6	5	=PRODUCT(G7,I7)	=PRODUCT(H7,I7)
			0.2	25	=-LOG(D8,2)					
			0.23	16	=-LOG(D9,2)					
			0.26	5	=-LOG(D10,2)					
NJV - 455	S - 2	5.5 - 10.0	0.13	95	=-LOG(D12,2)					
			0.16	84	=-LOG(D13,2)					
			0.17	75	=-LOG(D14,2)					
			0.21	50	=-LOG(D15,2)	=AVERAGE(F17,F15,F13)	=(F13-F17)/4+(F12-F18)/6	5	=PRODUCT(G15,I15)	=PRODUCT(H15,I15)
			0.26	25	=-LOG(D16,2)					
			0.3	16	=-LOG(D17,2)					
			0.38	5	=-LOG(D18,2)					
NJV - 455	S - 3	10.0 - 15.0	0.13	95	=-LOG(D20,2)					
			0.15	84	=-LOG(D21,2)					
			0.17	75	=-LOG(D22,2)					
			0.24	50	=-LOG(D23,2)	=AVERAGE(F25,F23,F21)	=(F21-F25)/4+(F20-F26)/6	5	=PRODUCT(G23,I23)	=PRODUCT(H23,I23)
			0.35	25	=-LOG(D24,2)					
			0.37	16	=-LOG(D25,2)					
			0.72	5	=-LOG(D26,2)					
NJV - 455	S - 4	15.0 - 19.1	0.14	95	=-LOG(D28,2)					
			0.19	84	=-LOG(D29,2)					
			0.22	75	=-LOG(D30,2)					
			0.27	50	=-LOG(D31,2)	=AVERAGE(F33,F31,F29)	=(F29-F33)/4+(F28-F34)/6	5	=PRODUCT(G31,I31)	=PRODUCT(H31,I31)
			0.33	25	=-LOG(D32,2)					
			0.37	16	=-LOG(D33,2)					
			0.42	5	=-LOG(D34,2)					
				SUM		=SUM(G4:G35)	=SUM(H4:H35)			
				Average		=AVERAGE(G4:G35)	=AVERAGE(H4:H35)			
							SUM	=SUM(I7:I31)	=SUM(J6:J34)	=SUM(K6:K34)
							WTD. AVG.		=(J38/I38)	=(K38/I38)

GEOTECHNICAL APPENDIX

SECTION 4

Investigation Reports (DVD)

Electronic versions of the investigation reports and data utilized in the calculations are included for additional reference in the DVD attached to this report. The following is a list of the reports and information contained on the DVD:

Previous Investigation Reports

- Geotechnical Data Report Vibrational Coring Cape May Fillet Area Study Cape May County, New Jersey. May 16, 2006
- Geotechnical Data Report Vibrational Coring Cape May, New Jersey. October 12, 2007
- Geotechnical Data Report Vibrational Coring Townsends & Hereford Inlets Cape May County, New Jersey. October 12, 2007
- Geotechnical Data Report Vibrational Coring Wildwood to Hereford Inlet Cape May County, New Jersey. June 30, 2006
- Geotechnical Data Report Wildwood Beach Investigation Cape May County, New Jersey. April 17, 2007
- Geotechnical Investigation Vibrocoring along the New Jersey Coast – Townsend/Hereford Inlet Study Area. December 1998
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Geotechnical Data

- USACE – Grain Size Analysis, October 1978, January 1979, March, 1995
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GEOTECHNICAL APPENDIX

SECTION 5

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Section 5

Project Design

The following section contains the infrastructure damages from Hereford Inlet to Cape May Inlet feasibility study by cell (1-7) for each storm event (5, 10, 20, 50, 100, 200, 500 year).

Infrastructure damages include damages to gas lines, water lines electric utilities, light poles, boardwalk etc, from erosion, wave damage and inundation.

The selected plan layout and cross sections are contained at the end of the section.

CELL	1	500 yr	200 yr	100 yr	50 yr	20 yr	10 yr	5 yr			
	Utility Pole (EA)	67	67	29	1	0	0	0	Assume all electric is 120V/240V		
	Transformer (EA)	9	0	0	0	0	0	0			
	Street Light (EA)	66	66	31	1	0	0	0	PIPE TRENCH		5
	Electric (LF)	1101	410	136	39	0	0	0	Pipe Size		4
	Lateral Elec (LF)	540	315	30	0	0	0	0	Excavation		
	Telephone (LF)	1101	410	136	39	0	0	0		A=	7.111111
	Lateral Tel (LF)	540	315	30	0	0	0	0		V=	0.263374
	Cable TV (LF)	1101	410	136	39	0	0	0			
	Lateral Cable TV (LF)	540	315	30	0	0	0	0	Bedding		4
2	Gas (LF)	225	0	0	0	0	0	0		V=	0.016461
	Excavation (CY)	59.3	0	0	0	0	0	0			
	Bedding 4" (CY)	3.7	0	0	0	0	0	0	Backfill	V=	0.243681
	Backfill (CY)	54.8	0	0	0	0	0	0			
1.5	Lateral Gas (LF)	540	0	0	0	0	0	0			
	Excavation (CY)	142.2	0	0	0	0	0	0	Excavation Each Manhole		
	Bedding 4" (CY)	8.9	0	0	0	0	0	0	6" Bedding Each Manhole		
	Backfill (CY)	131.6	0	0	0	0	0	0	Backfill Each Manhole		
	Man Hole (EA)	12	12	0	0	0	0	0	Concrete Each Manhole		
	Excavation (CY)	1154.39	1154.39	0	0	0	0	0			
	Bedding 6" (CY)	8.91	8.91	0	0	0	0	0			
	Backfill (CY)	1056.11	1056.11	0	0	0	0	0	Outfall pipe lengths shown in red		
	Concrete (CY)	44.68	44.68	0	0	0	0	0	See attached outfall pipe support		
8" to 15"	Sanitary Sewer (LF)	1860	755	0	0	0	0	0	taken from the Lower Cape May		
	Excavation (CY)	1112.3	451.5	0	0	0	0	0	Assume excavation and backfill f		
	Bedding 4" (CY)	63.1	25.6	0	0	0	0	0			
	Backfill (CY)	964.6	391.5	0	0	0	0	0			
16" to 21"	Sanitary Sewer (LF)	0	0	0	0	0	0	0			
	Excavation (CY)	0	0	0	0	0	0	0			
	Bedding 6" (CY)	0	0	0	0	0	0	0			
	Backfill (CY)	0	0	0	0	0	0	0			
24" to 30"	Sanitary Sewer (LF)	0	0	0	0	0	0	0			
	Excavation (CY)	0	0	0	0	0	0	0			
	Bedding 6" (CY)	0	0	0	0	0	0	0			
	Backfill (CY)	0	0	0	0	0	0	0			
33" to 42"	Sanitary Sewer (LF)	0	0	0	0	0	0	0			
	Excavation (CY)	0	0	0	0	0	0	0			
	Bedding 6" (CY)	0	0	0	0	0	0	0			
	Backfill (CY)	0	0	0	0	0	0	0			
6	Lateral San Sew (LF)	540	315	0	0	0	0	0			
	Excavation (CY)	1020	595	0	0	0	0	0			
	Bedding 6" (CY)	30	17.5	0	0	0	0	0			
	Backfill (CY)	797.6	465.3	0	0	0	0	0			
	Man Hole SD (EA)	13	13	11	0	0	0	0			
	Excavation (CY)	1250.58	1250.58	1058.19	0	0	0	0			
	Bedding 6" (CY)	9.66	9.66	8.17	0	0	0	0			
	Backfill (CY)	1144.12	1144.12	968.1	0	0	0	0			

	Concrete (CY)	48.41	48.41	40.96	0	0	0	0				
	Catch Basin (EA)	0	0	0	0	0	0	0				
	Excavation (CY)	0	0	0	0	0	0	0				
	Bedding 6" (CY)	0	0	0	0	0	0	0				
	Backfill (CY)	0	0	0	0	0	0	0				
	Concrete (CY)	0	0	0	0	0	0	0				
6" to 15"	Storm Drain (LF)	0	0	0	0	0	0	0				
	Excavation (CY)	0	0	0	0	0	0	0				
	Bedding 4" (CY)	0	0	0	0	0	0	0				
	Backfill (CY)	0	0	0	0	0	0	0				
16" to 21"	Storm Drain (LF)	210	125	0	0	0	0	0				
	Excavation (CY)	179.4	106.8	0	0	0	0	0				
	Bedding 6" (CY)	8.4	5	0	0	0	0	0				
	Backfill (CY)	152.2	90.6	0	0	0	0	0				
24" to 30"	Storm Drain (LF)	1064	1064	1064	0	0	0	0				
	Excavation (CY)	1330	1330	1330	0	0	0	0				
	Bedding 6" (CY)	49.3	49.3	0	0	0	0	0				
	Backfill (CY)	1087.3	1087.3	1087.3	0	0	0	0				
33" to 48"	Storm Drain (LF)	2479	2271	2137	603	547	485	373				
	Excavation (CY)	4682.6	4289.7	4036.6	1139	1033.2	916.1	704.6				
	Bedding 6" (CY)	137.7	126.2	0	0	0	0	0				
	Backfill (CY)	3661.5	3354.3	3156.3	890.6	807.9	716.3	550.9				
6	Water (LF)	5058	3250	0	0	0	0	0				
	Excavation (CY)	3024.7	1943.5	0	0	0	0	0				
	Bedding 6" (CY)	171.7	110.3	0	0	0	0	0				
	Backfill (CY)	2623	1685.4	0	0	0	0	0				
1	Lateral Water (LF)	540	315	0	0	0	0	0				
	Excavation (CY)	1020	595	0	0	0	0	0				
	Bedding 4" (CY)	30	17.5	0	0	0	0	0				
	Backfill (CY)	797.6	465.3	0	0	0	0	0				
	Pavement (SF)	152459	108477	79133	0	0	0	0				
	6 ft Sidewalk (SF)	63437	42295	0	0	0	0	0				
	Curb (LF)	12800	7617	3050	0	0	0	0				
	Number of Buildings (N	36	21	0	0	0	0	0				
	Number of FH (F)	4	0	0	0	0	0	0				
	Typical Lateral (Ft)	15	15	15	15	15	15	15				
	Typical Road Width (W)	30	30	30	30	30	30	30				
	Number of pipe support	356	356	356	67	61	54	41				

CELL	2	500 yr	200 yr	100 yr	50 yr	20 yr	10 yr	5 yr
	Utility Pole (EA)	114	71	39	16			
	Transformer (EA)	11	3	0	0	0	0	0
	Street Light (EA)	114	71	39	16			
	Electric (LF)	4002	3433	3328	3128	0	0	0
	Lateral Elec (LF)	210	150	150	0	0	0	0
	Telephone (LF)	874	305	200	16	0	0	0
	Lateral Tel (LF)	210	150	150	0	0	0	0
	Cable TV (LF)	874	305	200	0	0	0	0
	Lateral Cable TV (LF)	210	150	150	0	0	0	0
2	Gas (LF)	0	0	0	0	0	0	0
	Excavation (CY)	0	0	0	0	0	0	0
	Bedding 4" (CY)	0	0	0	0	0	0	0
	Backfill (CY)	0	0	0	0	0	0	0
1.5	Lateral Gas (LF)	0	0	0	0	0	0	0
	Excavation (CY)	0	0	0	0	0	0	0
	Bedding 4" (CY)	0	0	0	0	0	0	0
	Backfill (CY)	0	0	0	0	0	0	0
	Man Hole (EA)	11	10	0	0	0	0	0
	Excavation (CY)	1058.19	961.99	0	0	0	0	0
	Bedding 6" (CY)	8.17	7.43	0	0	0	0	0
	Backfill (CY)	968.1	880.09	0	0	0	0	0
	Concrete (CY)	40.96	37.24	0	0	0	0	0
6" to 15"	Sanitary Sewer (LF)	750	0	0	0	0	0	0
	Excavation (CY)	448.5	0	0	0	0	0	0
	Bedding 4" (CY)	25.5	0	0	0	0	0	0
	Backfill (CY)	388.9	0	0	0	0	0	0
16" to 21"	Sanitary Sewer (LF)	0	0	0	0	0	0	0
	Excavation (CY)	0	0	0	0	0	0	0
	Bedding 6" (CY)	0	0	0	0	0	0	0
	Backfill (CY)	0	0	0	0	0	0	0
24" to 30"	Sanitary Sewer (LF)	0	0	0	0	0	0	0
	Excavation (CY)	0	0	0	0	0	0	0
	Bedding 6" (CY)	0	0	0	0	0	0	0
	Backfill (CY)	0	0	0	0	0	0	0
33" to 42"	Sanitary Sewer (LF)	0	0	0	0	0	0	0
	Excavation (CY)	0	0	0	0	0	0	0
	Bedding 6" (CY)	0	0	0	0	0	0	0
	Backfill (CY)	0	0	0	0	0	0	0
6	Lateral San Sew (LF)	210	150	0	0	0	0	0
	Excavation (CY)	396.7	283.3	0	0	0	0	0
	Bedding 6" (CY)	11.7	8.3	0	0	0	0	0
	Backfill (CY)	310.2	221.5	0	0	0	0	0
	Man Hole SD (EA)	15	10	10	0	0	0	0
	Excavation (CY)	1442.98	961.99	961.99	0	0	0	0

	Bedding 6" (CY)	11.14	7.43	7.43	0	0	0	0				
	Backfill (CY)	1320.14	880.09	880.09	0	0	0	0				
	Concrete (CY)	55.85	37.24	37.24	0	0	0	0				
	Catch Basin (EA)	0	0	0	0	0	0	0				
	Excavation (CY)	0	0	0	0	0	0	0				
	Bedding 6" (CY)	0	0	0	0	0	0	0				
	Backfill (CY)	0	0	0	0	0	0	0				
	Concrete (CY)	0	0	0	0	0	0	0				
6" to 15"	Storm Drain (LF)	924	420	0	0	0	0	0				
	Excavation (CY)	552.5	251.2	0	0	0	0	0				
	Bedding 4" (CY)	0	0	0	0	0	0	0				
	Backfill (CY)	479.2	217.8	0	0	0	0	0				
16" to 21"	Storm Drain (LF)	770	0	0	0	0	0	0				
	Excavation (CY)	657.7	0	0	0	0	0	0				
	Bedding 6" (CY)	0	0	0	0	0	0	0				
	Backfill (CY)	558.2	0	0	0	0	0	0				
24" to 30"	Storm Drain (LF)	0	1821	1701	0	0	0	0				
	Excavation (CY)	0	2276.3	2126.3	0	0	0	0				
	Bedding 6" (CY)	0	0	0	0	0	0	0				
	Backfill (CY)	0	1860.9	1738.2	0	0	0	0				
33" to 42"	Storm Drain (LF)	0	0	0	0	0	0	0				
	Excavation (CY)	0	0	0	0	0	0	0				
	Bedding 6" (CY)	0	0	0	0	0	0	0				
	Backfill (CY)	0	0	0	0	0	0	0				
6"	Water (LF)	1314	896	0	0	0	0	0				
	Excavation (CY)	785.8	535.8	0	0	0	0	0				
	Bedding 6" (CY)	44.6	30.4	0	0	0	0	0				
	Backfill (CY)	681.4	464.7	0	0	0	0	0				
1	Lateral Water (LF)	210	150	0	0	0	0	0				
	Excavation (CY)	396.7	283.3	0	0	0	0	0				
	Bedding 4" (CY)	11.7	8.3	0	0	0	0	0				
	Backfill (CY)	310.2	221.5	0	0	0	0	0				
	Pavement (SF)	51312	28392	6504	0	0	0	0				
	6 ft Sidewalk (SF)	12888	8320	6600	0	0	0	0				
	Curb (LF)	2200	968	0	0	0	0	0				
	Number of Buildings (N	14	10	10	0	0	0	0				
	Number of FH (F)	2	0	0	0	0	0	0				
	Typical Lateral (Ft)	15	15	15	15	15	15	15				
	Typical Road Width (W)	30	30	30	30	30	30	30				
	Number of pipe support	626	249	189	0	0	0	0				

CELL	3	500 yr	200 yr	100 yr	50 yr	20 yr	10 yr	5 yr
	Utility Pole (EA)	62	9	6	5	0	0	0
	Transformer (EA)	14	6	0	0	0	0	0
	Street Light (EA)	62	9	6	5	0	0	0
	Electric (LF)	3983	1288	1087	876	0	0	0
	Lateral Elec (LF)	465	75	30	0	0	0	0
	Telephone (LF)	3983	1288	1087	876	0	0	0
	Lateral Tel (LF)	465	75	30	0	0	0	0
	Cable TV (LF)	3983	1288	1087	876	0	0	0
	Lateral Cable TV (LF)	465	75	30	0	0	0	0
2	Gas (LF)	586	233	0	0	0	0	0
	Excavation (CY)	154.3	61.4	0	0	0	0	0
	Bedding 4" (CY)	9.6	3.8	0	0	0	0	0
	Backfill (CY)	142.8	56.8	0	0	0	0	0
1	Lateral Gas (LF)	465	0	0	0	0	0	0
	Excavation (CY)	122.5	0	0	0	0	0	0
	Bedding 4" (CY)	7.7	0	0	0	0	0	0
	Backfill (CY)	113.3	0	0	0	0	0	0
	Man Hole (EA)	30	0	0	0	0	0	0
	Excavation (CY)	2885.96	0	0	0	0	0	0
	Bedding 6" (CY)	22.28	0	0	0	0	0	0
	Backfill (CY)	2640.28	0	0	0	0	0	0
	Concrete (CY)	111.71	0	0	0	0	0	0
6" to 15"	Sanitary Sewer (LF)	5185	0	0	0	0	0	0
	Excavation (CY)	3100.6	0	0	0	0	0	0
	Bedding 4" (CY)	176	0	0	0	0	0	0
	Backfill (CY)	2688.9	0	0	0	0	0	0
16" to 21"	Sanitary Sewer (LF)	1280	0	0	0	0	0	0
	Excavation (CY)	1093.3	0	0	0	0	0	0
	Bedding 6" (CY)	51.4	0	0	0	0	0	0
	Backfill (CY)	927.9	0	0	0	0	0	0
24" to 30"	Sanitary Sewer (LF)	0	0	0	0	0	0	0
	Excavation (CY)	0	0	0	0	0	0	0
	Bedding 6" (CY)	0	0	0	0	0	0	0
	Backfill (CY)	0	0	0	0	0	0	0
33" to 42"	Sanitary Sewer (LF)	0	0	0	0	0	0	0
	Excavation (CY)	0	0	0	0	0	0	0
	Bedding 6" (CY)	0	0	0	0	0	0	0
	Backfill (CY)	0	0	0	0	0	0	0
6	Lateral San Sew (LF)	465	0	0	0	0	0	0
	Excavation (CY)	878.3	0	0	0	0	0	0
	Bedding 6" (CY)	25.8	0	0	0	0	0	0
	Backfill (CY)	686.8	0	0	0	0	0	0
	Man Hole SD (EA)	30	7	5	1	0	0	
	Excavation (CY)	2885.96	673.39	480.99	96.2	0	0	
	Bedding 6" (CY)	22.28	5.2	3.71	0.74	0	0	
	Backfill (CY)	2640.28	616.07	440.05	88.01	0	0	
	Concrete (CY)	111.71	26.07	18.62	3.72	0	0	

	Catch Basin (EA)	11	0	0	0	0	0					
	Excavation (CY)	1058.19	0	0	0	0	0					
	Bedding 6" (CY)	8.17	0	0	0	0	0					
	Backfill (CY)	968.1	0	0	0	0	0					
	Concrete (CY)	40.96	0	0	0	0	0					
6" to 15"	Storm Drain (LF)	2372	1790	1570	1340	1066	885	705				
	Excavation (CY)	1418.4	1070.4	938.9	801.3	637.5	529.2	421.6				
	Bedding 4" (CY)	80.5	0	0	0	0	0	0				
	Backfill (CY)	1230.1	928.3	814.2	694.9	552.8	459	365.6				
16" to 21"	Storm Drain (LF)	7553	4626	3826	2986	2037	1541	1046				
	Excavation (CY)	6451.5	3951.4	3268	2550.5	1739.9	1316.3	893.5				
	Bedding 6" (CY)	303.1	0	0	0	0	0	0				
	Backfill (CY)	5475.6	3353.7	2773.7	2164.7	1476.7	1117.2	758.3				
24" to 30"	Storm Drain (LF)	7947	4025	3423	2790	2037	1540	1042				
	Excavation (CY)	9933.8	5031.3	4278.8	3487.5	2546.3	1925	1302.5				
	Bedding 6" (CY)	367.9	0	0	0	0	0	0				
	Backfill (CY)	8121	4113.1	3498	2851.1	2081.6	1573.7	1064.8				
33" to 42"	Storm Drain (LF)	0	0	0	0	0	0					
	Excavation (CY)	0	0	0	0	0	0	0				
	Bedding 6" (CY)	0	0	0	0	0	0	0				
	Backfill (CY)	0	0	0	0	0	0	0				
6"	Water (LF)	5880	0	0	0	0	0	0				
	Excavation (CY)	3516.2	0	0	0	0	0	0				
	Bedding 6" (CY)	199.6	0	0	0	0	0	0				
	Backfill (CY)	3049.3	0	0	0	0	0	0				
1"	Lateral Water (LF)	465	0	0	0	0	0	0				
	Excavation (CY)	878.3	0	0	0	0	0	0				
	Bedding 4" (CY)	25.8	0	0	0	0	0	0				
	Backfill (CY)	686.8	0	0	0	0	0	0				
	Pavement (SF)	364619	340185	0	0	0	0	0				
	6 ft Sidewalk (SF)	56160	0	0	0	0	0	0				
	Curb (LF)	4160	0	0	0	0	0	0				
	Number of Buildings (N	31	0	0	0	0	0	0				
	Number of FH (F)	9	0	0	0	0	0	0				
	Typical Lateral (Ft)	15	15	15	15	15	15	15				
	Typical Road Width (W	30	30	30	30	30	30	30				
	Number of pipe support	1160	1160	980	791	571	441	310				

CELL	4	500 yr	200 yr	100 yr	50 yr	20 yr	10 yr	5 yr
	Utility Pole (EA)	81	22	2	0	0	0	0
	Transformer (EA)	20	7	1	0	0	0	0
	Street Light (EA)	81	22	2	0	0	0	0
	Electric (LF)	6674	2298	460	0	0	0	0
	Lateral Elec (LF)	615	225	30	0	0	0	0
	Telephone (LF)	6674	2298	460	0	0	0	0
	Lateral Tel (LF)	615	225	30	0	0	0	0
	Cable TV (LF)	6674	2298	460	0	0	0	0
	Lateral Cable TV (LF)	615	225	30	0	0	0	0
2	Gas (LF)	1039	0	0	0	0	0	0
	Excavation (CY)	273.6	0	0	0	0	0	0
	Bedding 4" (CY)	17.1	0	0	0	0	0	0
	Backfill (CY)	253.2	0	0	0	0	0	0
1.5	Lateral Gas (LF)	615	0	0	0	0	0	0
	Excavation (CY)	162	0	0	0	0	0	0
	Bedding 4" (CY)	10.1	0	0	0	0	0	0
	Backfill (CY)	149.9	0	0	0	0	0	0
	Man Hole (EA)	38	9	1	0	0	0	0
	Excavation (CY)	3655.55	865.79	96.2	0	0	0	0
	Bedding 6" (CY)	28.23	6.69	0.74	0	0	0	0
	Backfill (CY)	3344.35	792.08	88.01	0	0	0	0
	Concrete (CY)	141.5	33.51	3.72	0	0	0	0
6" to 15"	Sanitary Sewer (LF)	3200	730	150	0	0	0	0
	Excavation (CY)	1913.6	436.5	89.7	0	0	0	0
	Bedding 4" (CY)	108.6	24.8	5.1	0	0	0	0
	Backfill (CY)	1659.5	378.6	77.8	0	0	0	0
16" to 21"	Sanitary Sewer (LF)	0	0	0	0	0	0	0
	Excavation (CY)	0	0	0	0	0	0	0
	Bedding 6" (CY)	0	0	0	0	0	0	0
	Backfill (CY)	0	0	0	0	0	0	0
24" to 30"	Sanitary Sewer (LF)	0	0	0	0	0	0	0
	Excavation (CY)	0	0	0	0	0	0	0
	Bedding 6" (CY)	0	0	0	0	0	0	0
	Backfill (CY)	0	0	0	0	0	0	0
33" to 42"	Sanitary Sewer (LF)	0	0	0	0	0	0	0
	Excavation (CY)	0	0	0	0	0	0	0
	Bedding 6" (CY)	0	0	0	0	0	0	0
	Backfill (CY)	0	0	0	0	0	0	0
6	Lateral San Sew (LF)	615	225	0	0	0	0	0
	Excavation (CY)	1161.7	425	0	0	0	0	0
	Bedding 6" (CY)	34.2	12.5	0	0	0	0	0
	Backfill (CY)	908.4	332.3	0	0	0	0	0
	Man Hole SD (EA)	0	0	0	0	0	0	
	Excavation (CY)	0	0	0	0	0	0	
	Bedding 6" (CY)	0	0	0	0	0	0	
	Backfill (CY)	0	0	0	0	0	0	
	Concrete (CY)	0	0	0	0	0	0	

	Catch Basin (EA)	0	0	0	0	0	0					
	Excavation (CY)	0	0	0	0	0	0					
	Bedding 6" (CY)	0	0	0	0	0	0					
	Backfill (CY)	0	0	0	0	0	0					
	Concrete (CY)	0	0	0	0	0	0					
6" to 15"	Storm Drain (LF)	0	0	0	0	0	0	0				
	Excavation (CY)	0	0	0	0	0	0	0				
	Bedding 4" (CY)	0	0	0	0	0	0	0				
	Backfill (CY)	0	0	0	0	0	0	0				
16" to 21"	Storm Drain (LF)	5760	4616	3967	3317	2435	1757	1148				
	Excavation (CY)	4920	3942.8	3388.5	2833.3	2079.9	1500.8	980.6				
	Bedding 6" (CY)	231.1	185.2	159.2	0	0	0	0				
	Backfill (CY)	4175.8	3346.4	2875.9	2404.7	1765.3	1273.8	832.3				
24" to 30"	Storm Drain (LF)	2456	2025	1715	1375	934	636	475				
	Excavation (CY)	3070	2531.3	2143.8	1718.8	1167.5	795	593.8				
	Bedding 6" (CY)	113.7	93.8	79.4	0	0	0	0				
	Backfill (CY)	2509.8	2069.3	1752.6	1405.1	954.5	649.9	485.4				
33" to 42"	Storm Drain (LF)	0	0	0	0	0	0					
	Excavation (CY)	0	0	0	0	0	0	0				
	Bedding 6" (CY)	0	0	0	0	0	0	0				
	Backfill (CY)	0	0	0	0	0	0	0				
6"	Water (LF)	1489	0	0	0	0	0	0				
	Excavation (CY)	890.4	0	0	0	0	0	0				
	Bedding 6" (CY)	50.6	0	0	0	0	0	0				
	Backfill (CY)	772.2	0	0	0	0	0	0				
1	Lateral Water (LF)	615	0	0	0	0	0	0				
	Excavation (CY)	1161.7	0	0	0	0	0	0				
	Bedding 4" (CY)	34.2	0	0	0	0	0	0				
	Backfill (CY)	908.4	0	0	0	0	0	0				
	Pavement (SF)	346669	108783	0	0	0	0	0				
	6 ft Sidewalk (SF)	227970	135024	0	0	0	0	0				
	Curb (LF)	21258	7200	0	0	0	0	0				
	Number of Buildings (N	41	15	0	0	0	0	0				
	Number of FH (F)	5	0	0	0	0	0	0				
	Typical Lateral (Ft)	15	15	15	15	15	15	15				
	Typical Road Width (W)	30	30	30	30	30	30	30				
	Number of pipe support	521	521	521	521	374	266	180				

CELL	5	500 yr	200 yr	100 yr	50 yr	20 yr	10 yr	5 yr
	Utility Pole (EA)	119	102	68	0	0	0	0
	Transformer (EA)	85	68	51	0	0	0	0
	Street Light (EA)	119	102	68	0	0	0	0
	Electric (LF)	16286	13702	10319	0	0	0	0
	Lateral Elec (LF)	1815	1260	960	0	0	0	0
	Telephone (LF)	16286	13702	10319	0	0	0	0
	Lateral Tel (LF)	1815	1260	960	0	0	0	0
	Cable TV (LF)	16286	13702	10319	0	0	0	0
	Lateral Cable TV (LF)	1815	1260	960	0	0	0	0
2	Gas (LF)	864	658	152	0	0	0	0
	Excavation (CY)	227.6	173.3	40	0	0	0	0
	Bedding 4" (CY)	14.2	10.8	2.5	0	0	0	0
	Backfill (CY)	210.5	160.3	37	0	0	0	0
1.5	Lateral Gas (LF)	1815	1260	960	0	0	0	0
	Excavation (CY)	478	331.9	252.8	0	0	0	0
	Bedding 4" (CY)	29.9	20.7	15.8	0	0	0	0
	Backfill (CY)	442.3	307	233.9	0	0	0	0
	Man Hole (EA)	46	42	27	0	0	0	0
	Excavation (CY)	4425.14	4040.35	2597.37	0	0	0	0
	Bedding 6" (CY)	34.17	31.2	20.06	0	0	0	0
	Backfill (CY)	4048.43	3696.39	2376.25	0	0	0	0
	Concrete (CY)	171.29	156.39	100.54	0	0	0	0
6" to 15"	Sanitary Sewer (LF)	9385	7585	4710	0	0	0	0
	Excavation (CY)	5612.2	4535.8	2816.6	0	0	0	0
	Bedding 4" (CY)	318.6	257.5	159.9	0	0	0	0
	Backfill (CY)	4867	3933.5	2442.6	0	0	0	0
16" to 21"	Sanitary Sewer (LF)	0	0	0	0	0	0	0
	Excavation (CY)	0	0	0	0	0	0	0
	Bedding 6" (CY)	0	0	0	0	0	0	0
	Backfill (CY)	0	0	0	0	0	0	0
24" to 30"	Sanitary Sewer (LF)	0	0	0	0	0	0	0
	Excavation (CY)	0	0	0	0	0	0	0
	Bedding 6" (CY)	0	0	0	0	0	0	0
	Backfill (CY)	0	0	0	0	0	0	0
33" to 42"	Sanitary Sewer (LF)	0	0	0	0	0	0	0
	Excavation (CY)	0	0	0	0	0	0	0
	Bedding 6" (CY)	0	0	0	0	0	0	0
	Backfill (CY)	0	0	0	0	0	0	0
6	Lateral San Sew (LF)	1815	1260	960	0	0	0	0
	Excavation (CY)	3428.3	2380	1813.3	0	0	0	0
	Bedding 6" (CY)	100.8	70	53.3	0	0	0	0
	Backfill (CY)	2680.7	1861	1417.9	0	0	0	0
	Man Hole SD (EA)	1	1	0	0	0	0	0
	Excavation (CY)	96.2	96.2	0	0	0	0	0
	Bedding 6" (CY)	0.74	0.74	0	0	0	0	0
	Backfill (CY)	88.01	88.01	0	0	0	0	0

	Concrete (CY)	3.72	3.72	0	0	0	0	0
	Catch Basin (EA)	5	2	0	0	0	0	0
	Excavation (CY)	480.99	192.4	0	0	0	0	0
	Bedding 6" (CY)	3.71	1.49	0	0	0	0	0
	Backfill (CY)	440.05	176.02	0	0	0	0	0
	Concrete (CY)	18.62	7.45	0	0	0	0	0
6" to 15"	Storm Drain (LF)	0	0	0	0	0	0	0
	Excavation (CY)	0	0	0	0	0	0	0
	Bedding 4" (CY)	0	0	0	0	0	0	0
	Backfill (CY)	0	0	0	0	0	0	0
16" to 21"	Storm Drain (LF)	2029	1729	1337	531	341	221	101
	Excavation (CY)	1733.1	1476.9	1142	453.6	291.3	188.8	86.3
	Bedding 6" (CY)	81.4	69.4	53.6	0	0	0	0
	Backfill (CY)	1470.9	1253.5	969.3	385	247.2	160.2	73.2
24" to 30"	Storm Drain (LF)	882	807	686	283	188	128	68
	Excavation (CY)	1102.5	1008.8	857.5	353.8	235	160	85
	Bedding 6" (CY)	40.8	37.4	31.8	0	0	0	0
	Backfill (CY)	901.3	824.7	701	289.2	192.1	130.8	69.5
33" to 42"	Storm Drain (LF)	0	0	0	0	0	0	0
	Excavation (CY)	0	0	0	0	0	0	0
	Bedding 6" (CY)	0	0	0	0	0	0	0
	Backfill (CY)	0	0	0	0	0	0	0
12"	Water (LF)	2178	0	0	0	0	0	0
	Excavation (CY)	1302.4	0	0	0	0	0	0
	Bedding 6" (CY)	73.9	0	0	0	0	0	0
	Backfill (CY)	1129.5	0	0	0	0	0	0
8"	Water (LF)	2326	0	0	0	0	0	0
	Excavation (CY)	1390.9	0	0	0	0	0	0
	Bedding 6" (CY)	79	0	0	0	0	0	0
	Backfill (CY)	1206.2	0	0	0	0	0	0
6"	Water (LF)	6500	4575	2500	0	0	0	0
	Excavation (CY)	3887	2735.8	1495	0	0	0	0
	Bedding 6" (CY)	220.7	155.3	84.9	0	0	0	0
	Backfill (CY)	3370.8	2372.6	1296.5	0	0	0	0
1"	Lateral Water (LF)	1815	1260	960	0	0	0	0
	Excavation (CY)	3428.3	2380	1813.3	0	0	0	0
	Bedding 4" (CY)	100.8	70	53.3	0	0	0	0
	Backfill (CY)	2680.7	1861	1417.9	0	0	0	0
	Pavement (SF)	377975	322300	198750	0	0	0	0
	6 ft Sidewalk (SF)	150060	127560	79500	0	0	0	0
	Curb (LF)	3680	3200	2880	0	0	0	0
	Number of Buildings (N	121	84	64	0	0	0	0
	Number of FH (F)	18	15	10	0	0	0	0
	Typical Lateral (Ft)	15	15	15	15	15	15	15
	Typical Road Width (W)	30	30	30	30	30	30	30
	Number of pipe support	31	31	31	31	21	14	8

3 lines							
= assumed depth of trench							W
15	21	30	42	48		DIA	MAX
						6" to 15"	8
16.14583	23.0625	33.75	51	63	SF	16" to 21"	10
0.597994	0.854167	1.25	1.888889	2.333333	CY/LF	24" to 30"	12
						33" to 42"	15
6	6	6	6	6		48" & grea	18
0.033951	0.040123	0.046296	0.055556	0.064815	CY/LF		
0.518592	0.724958	1.021898	1.476994	1.803096	CY/LF		
96.1988	cy						
0.7428	cy						
88.0093	cy						
3.7236	cy						

are pipes suported on timber piles and cribbing.
: drawing,
Meadows Beachfill
or these portions involves beach sand only.

CELL	6	500 yr	200 yr	100 yr	50 yr	20 yr	10 yr	5 yr
	Utility Pole (EA)	52	50	46	46	45	37	35
	Transformer (EA)	0	0	0	0	0	0	0
	Street Light (EA)	44	44	44	44	44	43	43
	Electric (LF)	0	0	0	0	0	0	0
	Lateral Elec (LF)	1275	1230	1215	1185	1170	1155	1110
	Telephone (LF)	0	0	0	0	0	0	0
	Lateral Tel (LF)	1275	1230	1215	1185	1170	1155	1110
	Cable TV (LF)	0	0	0	0	0	0	0
	Lateral Cable TV (LF)	1275	1230	1215	1185	1170	17325	16650
2	Gas (LF)	577.6	530.9	520.6	515.4	510.3	505	500
	Excavation (CY)	152.1	139.8	137.1	135.7	134.4	133	131.7
	Bedding 4" (CY)	9.5	8.7	8.6	8.5	8.4	8.3	8.2
	Backfill (CY)	140.8	129.4	126.9	125.6	124.4	123.1	121.8
1.5	Lateral Gas (LF)	1275	1230	1215	1185	1170	48786	47670
	Excavation (CY)	335.8	324	320	312.1	308.1	12849	12555.1
	Bedding 4" (CY)	21	20.2	20	19.5	19.3	803.1	784.7
	Backfill (CY)	310.7	299.7	296.1	288.8	285.1	11888.2	11616.3
	Man Hole (EA)	27	26	26	26	25	25	20
	Excavation (CY)	2597.37	2501.17	2501.17	2501.17	2404.97	2404.97	1923.98
	Bedding 6" (CY)	20.06	19.31	19.31	19.31	18.57	18.57	14.86
	Backfill (CY)	2376.25	2288.24	2288.24	2288.24	2200.23	2200.23	1760.19
	Concrete (CY)	100.54	96.81	96.81	96.81	93.09	93.09	74.47
6" to 15"	Sanitary Sewer (LF)	2746.9	2375.6	2173.3	2072.1	1976.1	1633.5	1145.1
	Excavation (CY)	1642.6	1420.6	1299.6	1239.1	1181.7	976.8	684.8
	Bedding 4" (CY)	93.3	80.7	73.8	70.3	67.1	55.5	38.9
	Backfill (CY)	1424.5	1232	1127.1	1074.6	1024.8	847.1	593.8
16" to 21"	Sanitary Sewer (LF)	4324.7	4324.7	4324.7	4324.7	4324.7	4324.7	4324.7
	Excavation (CY)	3694	3694	3694	3694	3694	3694	3694
	Bedding 6" (CY)	173.5	173.5	173.5	173.5	173.5	173.5	173.5
	Backfill (CY)	3135.2	3135.2	3135.2	3135.2	3135.2	3135.2	3135.2
24" to 30"	Sanitary Sewer (LF)	0	0	0	0	0	0	0
	Excavation (CY)	0	0	0	0	0	0	0
	Bedding 6" (CY)	0	0	0	0	0	0	0
	Backfill (CY)	0	0	0	0	0	0	0
33" to 42"	Sanitary Sewer (LF)	0	0	0	0	0	0	0

	Excavation (CY)	0	0	0	0	0	0	0
	Bedding 6" (CY)	0	0	0	0	0	0	0
	Backfill (CY)	0	0	0	0	0	0	0
6	Lateral San Sew (LF)	1275	1230	1215	1185	1170	1155	1110
	Excavation (CY)	2408.3	2323.3	2295	2238.3	2210	2181.7	2096.7
	Bedding 6" (CY)	70.8	68.3	67.5	65.8	65	64.2	61.7
	Backfill (CY)	1883.2	1816.7	1794.5	1750.2	1728.1	1705.9	1639.5
	Man Hole SD (EA)	37	36	35	35	35	34	34
	Excavation (CY)	3559.36	3463.16	3366.96	3366.96	3366.96	3270.76	3270.76
	Bedding 6" (CY)	27.48	26.74	26	26	26	25.26	25.26
	Backfill (CY)	3256.34	3168.33	3080.33	3080.33	3080.33	2992.32	2992.32
	Concrete (CY)	137.77	134.05	130.33	130.33	130.33	126.6	126.6
	Catch Basin (EA)	15	11	7	7	7	7	7
	Excavation (CY)	1442.98	1058.19	673.39	673.39	673.39	673.39	673.39
	Bedding 6" (CY)	11.14	8.17	5.2	5.2	5.2	5.2	5.2
	Backfill (CY)	1320.14	968.1	616.07	616.07	616.07	616.07	616.07
	Concrete (CY)	55.85	40.96	26.07	26.07	26.07	26.07	26.07
6" to 15"	Storm Drain (LF)	1681.3	1651.5	1630.2	1619.6	1609	1598.3	1587.7
	Excavation (CY)	1005.4	987.6	974.8	968.5	962.2	955.8	949.4
	Bedding 4" (CY)	57.1	56.1	55.3	55	54.6	54.3	53.9
	Backfill (CY)	871.9	856.5	845.4	839.9	834.4	828.9	823.4
16" to 21"	Storm Drain (LF)	9568.2	9497.3	9447.3	9422.3	9397.2	9372.2	9347.2
	Excavation (CY)	8172.8	8112.3	8069.6	8048.2	8026.8	8005.4	7984.1
	Bedding 6" (CY)	383.9	381.1	379.1	378.1	377	376	375
	Backfill (CY)	6936.5	6885.1	6848.9	6830.8	6812.6	6794.5	6776.3
24" to 30"	Storm Drain (LF)	5921.3	5851.1	5801.1	5776.1	5751.1	5726.1	5701.1
	Excavation (CY)	7401.6	7313.9	7251.4	7220.1	7188.9	7157.6	7126.4
	Bedding 6" (CY)	274.1	270.9	268.6	267.4	266.3	265.1	263.9
	Backfill (CY)	6051	5979.2	5928.1	5902.6	5877	5851.5	5825.9
33" to 42"	Storm Drain (LF)	0	0	0	0	0	0	0
	Excavation (CY)	0	0	0	0	0	0	0
	Bedding 6" (CY)	0	0	0	0	0	0	0
	Backfill (CY)	0	0	0	0	0	0	0
8	Water (LF)	5534.9	4416.7	4067.4	3666.8	3334.2	3252.4	3178
	Excavation (CY)	3309.8	2641.2	2432.3	2192.7	1993.8	1944.9	1900.4
	Bedding 6" (CY)	187.9	149.9	138.1	124.5	113.2	110.4	107.9
	Backfill (CY)	2870.4	2290.5	2109.3	1901.6	1729.1	1686.7	1648.1

1	Lateral Water (LF)	1275	1230	1215	1185	1170	1155	1110
	Excavation (CY)	2408.3	2323.3	2295	2238.3	2210	2181.7	2096.7
	Bedding 4" (CY)	70.8	68.3	67.5	65.8	65	64.2	61.7
	Backfill (CY)	1883.2	1816.7	1794.5	1750.2	1728.1	1705.9	1639.5
	Pavement (SF)	132480	96000	86400	81600	76800	72000	67200
	4 ft Sidewalk (SF)	7360	6400	5760	5440	5120	4800	4480
	Curb (LF)	3680	3200	2880	2720	2560	2400	2240
	Number of Buildings (N)	85	82	81	79	78	77	74
	Number of FH (F)	9	9	9	9	9	9	9
	Typical Lateral (Ft)	15	15	15	15	15	15	15
	Typical Road Width (W)	36	30	30	30	30	30	30

Hereford Inlet to Cape May Inlet			
Without Project Boardwalk Damages			
Cell 2	Damages occur at 100 year event		
	Item	**Quantity**	**Comments**
	2x6 decking	103700 sf	new decking is ipe lumber
	3x6 joists	130677 ft	new joists are southern yellow pine, grade No.1, 2.5pcf CCA preservative
	2x6 bridging	15250 ft	new bridging is southern yellow pine, grade No.1, 0.25pcf CCA preservative
	deck screws 3" long	399,000	screws are weather guard coated steel
	1/2" x 6" L. SS bolts	5,085	include nuts&washers
	twist straps	5085	straps are stainless steel
	light poles	20	poles are tubular aluminum 20' long
	electric cable @220 v	1800 ft	
	public telephone	20	
	telephone cable	1000 ft	
	1.5" dia. SS pipe	8105 ft	actual pipe length needed to fabricate railing, not railing distance
	for handrail (see sketch)		
	3/8" dia. X 6" L SS bolts	1010	for attaching handrail posts to outside joist
	(for handrail)		
	concrete walkway	600 cy	for tramcars-4000 psi concrete
	WWF	48800 sf	W2 x W2 galvanized
	plywood 1" thick	54900 sf	base for concrete walkway
	shotcrete	61000 sf	under concrete walkway
	8x10	12200 ft	support beam for concrete walkway- new lumber is southern yellow pine, grade No.1, 2.5pcf CCA preservative
Cell 3	Damages occur at 100 year event		
	Item	**Quantity**	**Materials**
	2x6 decking	239360 sf	new decking is ipe lumber
	3x6 joists	301312 ft	new joists are southern yellow pine, grade No.1, 2.5pcf CCA preservative
	2x6 bridging	35200 ft	new bridging is southern yellow pine, grade No.1, 0.25pcf CCA preservative
	deck screws 3" long	920,830	screws are weather guard coated steel
	1/2" x 6" L. SS bolts	23,470	include nuts&washers
	twist straps	23470	straps are stainless steel
	light poles	47	poles are tubular aluminum 20' long
	electric cable @220 v	4000 ft	
	public telephone	26	
	telephone cable	1300 ft	
	1.5" dia. SS pipe	18775 ft	actual pipe length needed to fabricate railing, not railing distance
	for handrail (see sketch)		
	3/8" dia. X 6" L SS bolts	2350	for attaching handrail posts to outside joist
	(for handrail)		
	concrete walkway	1390 cy	for tramcars-4000 psi concrete
	WWF	112640 sf	W2 x W2 galvanized
	plywood 1" thick	126720 sf	base for concrete walkway

	shotcrete		140800 sf		under concrete walkway									
	8x10		28160 ft		support beam for concrete walkway- new lumber is southern yellow pine, grade No.1, 2.5pcf CCA preservative									
Based on total water level and wave setup in vicinity of boardwalk decking. Damage is assumed to occur if waves are impacting the deck fom underneath or on top.														

Wildwood/ North Wildwood
Pier damages

Surfside Pier Cell 2 (located at approx.Station 65+00 - Refer to attached retaining wall cross section

Retaining Wall (located at seaward end of pier)
Damages occur at 10 year event

Description	Quantity
Shoreguard Series 300 Vinyl Sheetpile, 12' L	1143 lf
6x6 wale .40 CCA	1143 lf
5/8" tie rod 12' L A307, HD Galvanized	190
4x12 1.0 CCA	1143 lf
4x4x4'L posts 1.0 CCA	95
3" Ogee Washer	380
Concrete sidewalk 3000 psi	116,600 sf
2x2 welded wire mesh	116,600 sf
Backfill clean well draining sandy fill	17274 cy

Utility Damages (lineal feet)

	Event	5	10	20	50	100*	200*	500*
distance to each amusement/ building (lf)		2763	3177	3598	4113	4113	4113	4113
**2" dia. pvc gas		2763	3177	3598	4113	4113	4113	4113
**4" pvc san. sewer		2763	3177	3598	4113	4113	4113	4113
**4" pvc water		2763	3177	3598	4113	4113	4113	4113
# of pipe hangar supports (for all pvc pipe)								
2" dia. pvc gas		277	318	360	412	412	412	412
4" pvc san. sewer		277	318	360	412	412	412	412
4" pvc water		277	318	360	412	412	412	412

* following 50 year event erosion lines are landward of pier and include boardwalk

** Utility lines are not located underground in trenches
but are suspended from pier deck substructure with clamps and pipe hangars (see attached).
~Assume installation of utility lines will require 2 carpenters and 2 electricians.
~Assume utility lines can be installed at the rate of 100' per day.
~Use 20% of the cost of the electrical cables to account for mounting hardware costs.

Wildwood/ North Wildwood
Pier damages

Mariner's Landing Cell 3 (located at approx.Station 88+00-refer to attached retaining wall cross section)

Retaining Wall (located at seaward end of pier)
Damages occur at 10 year event

Description	Quantity
Shoreguard Series 300 Vinyl Sheetpile, 12' L	350 lf
6x6 wale .40 CCA	350 lf
5/8" tie rod 12' L A307, HD Galvanized	58
4x12 1.0 CCA	350 lf
4x4x4'L posts 1.0 CCA	29
3" Ogee Washer	116
Concrete sidewalk 3000 psi	13260 sf
2x2 welded wire mesh	13260 sf
Backfill clean well draining sandy fill	1719 cy

Utility Damages (lineal feet)

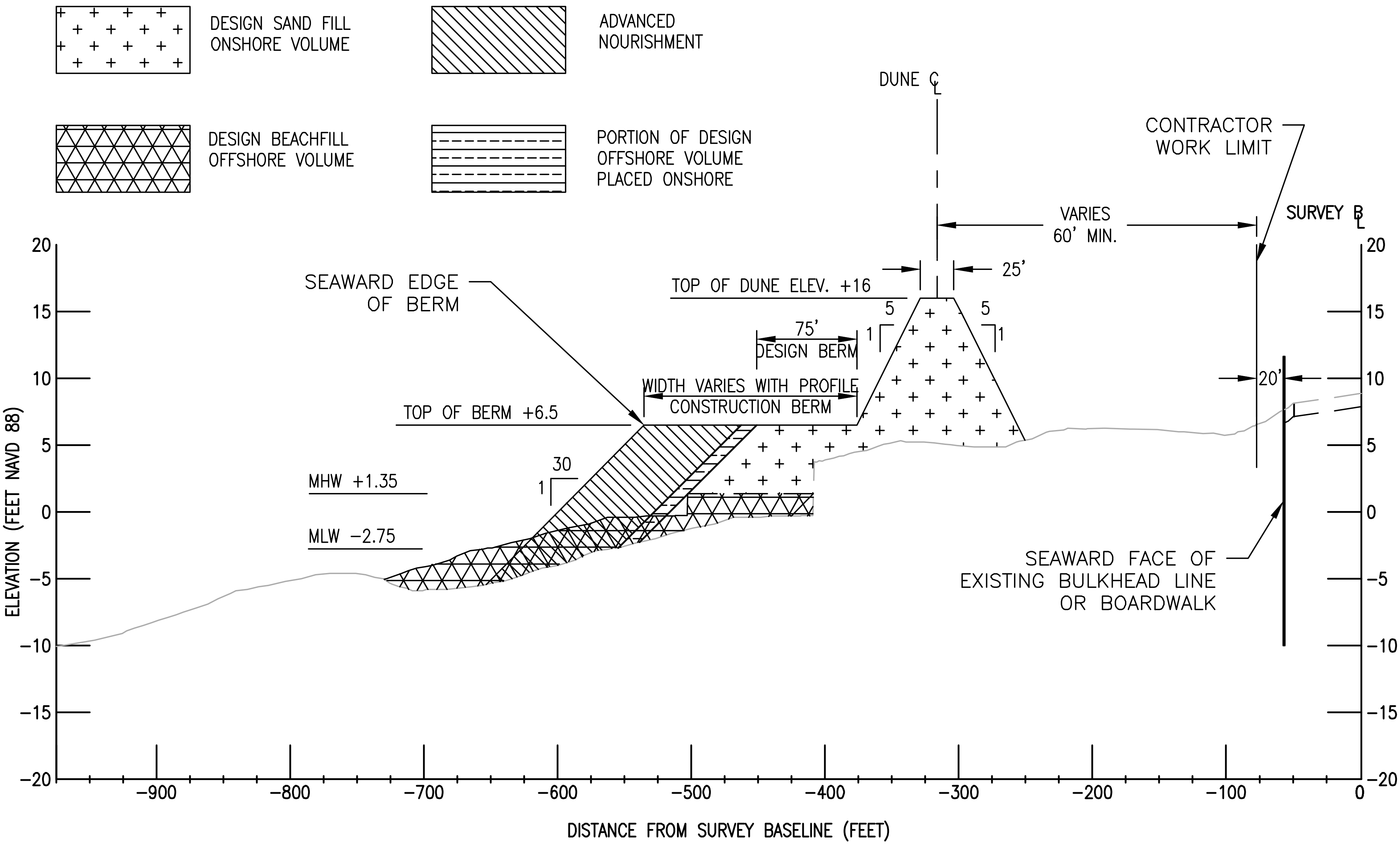
Event	**5**	**10**	**20**	**50**	**100***	**200***	**500***
distance to each amusement/ building (lf)	0	105	1397	2942	3948	5129	6609
2" dia. pvc gas	0	105	1397	2942	3948	5129	6609
4" pvc san. sewer	0	105	1397	2942	3948	5129	6609
4" pvc water	0	105	1397	2942	3948	5129	6609

Event	**5**	**10**	**20**	**50**	**100***	**200***	**500***
# of pipe hangar supports (for all pvc pipe)							
2" dia. pvc gas	0	10	140	295	395	513	661
4" pvc san. sewer	0	10	140	295	395	513	661
4" pvc water	0	10	140	295	395	513	661

* following 50 year event erosion lines are landward of pier and include boardwalk

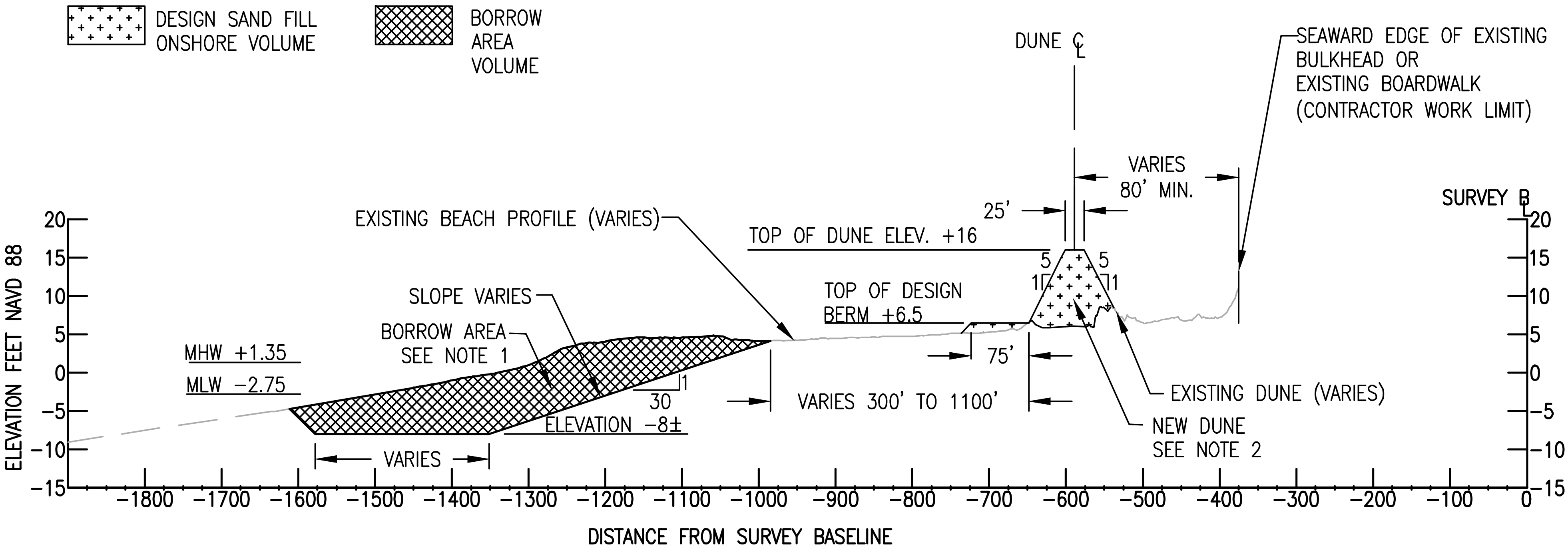
** Utility lines are not located underground in trenches
but are suspended from pier deck substructure with clamps and pipe hangars (see attached).
~Assume installation of utility lines will require 2 carpenters and 2 electricians.
~Assume utility lines can be installed at the rate of 100' per day.

~Use 20% of the cost of the electrical cables to account for mounting hardware costs.



HEREFORD INLET TO CAPE MAY INLET FEASIBILITY STUDY

TYPICAL BEACHFILL DESIGN AND CONSTRUCTION CROSS SECTION
STATION 0+00 TO 85+00
ELEVATION +6.5 BERM +16' DUNE ELEVATION
SCALE AS SHOWN IN FEET



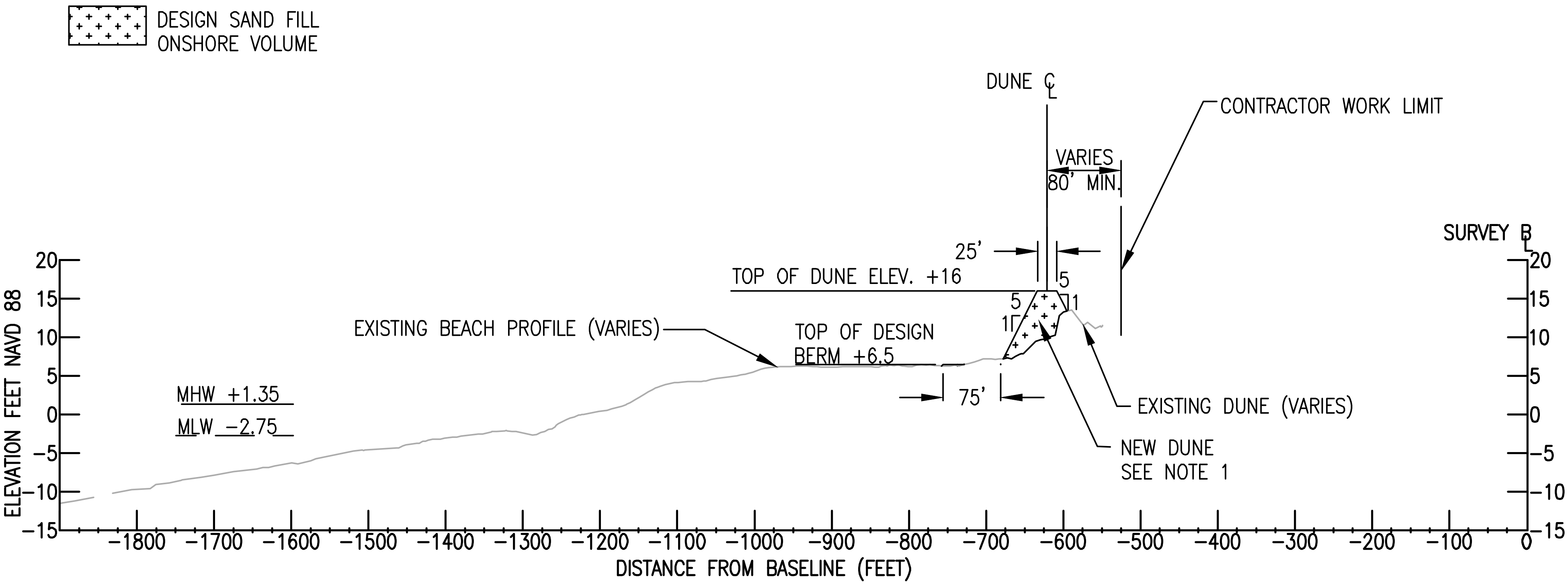
NOTES:

1. EXACT LIMITS OF BORROW AREA MAY VARY FROM THAT SHOWN HERE, BASED ON EXISTING BEACH CONDITIONS AND DREDGING EQUIPMENT SELECTED AT TIME OF CONSTRUCTION.
2. CENTERLINE OF NEW DUNE IS LOCATED ALONG THE SEAWARD EDGE OF THE EXISTING DUNES IN ORDER TO MINIMIZE SAND VOLUME REQUIRED
3. NEW DUNE VEGETATION IS NOT SHOWN.

HEREFORD INLET TO CAPE MAY INLET FEASIBILITY STUDY

TYPICAL DESIGN CROSS SECTION STATION 85+00 TO 240+00
BEACH BORROW AREA WITH ELEVATION +16 DUNE

SCALE AS SHOWN IN FEET



NOTES:

1. CENTERLINE OF NEW DUNE IS LOCATED ALONG THE SEAWARD EDGE OF THE EXISTING DUNES IN ORDER TO MINIMIZE SAND VOLUME REQUIRED.
2. NEW DUNE VEGETATION IS NOT SHOWN.

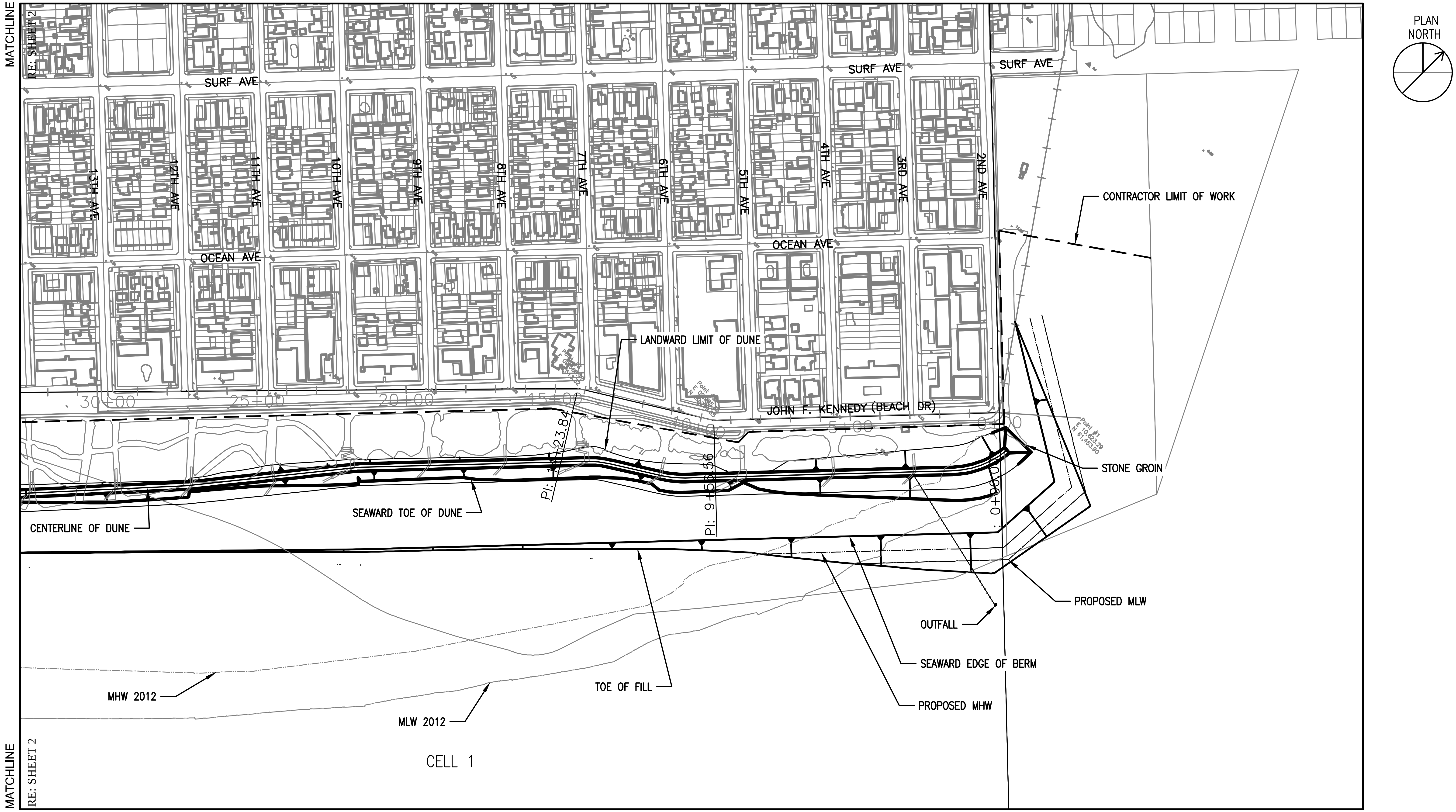
HEREFORD INLET TO CAPE MAY INLET FEASIBILITY STUDY

TYPICAL DESIGN CROSS SECTION

STATION 240+00 TO 314+03

ELEVATION +16 DUNE

SCALE AS SHOWN IN FEET



BORROW AREA

NOTES:
1. ALL EXISTING DUNE CROSSOVERS AND BEACH ACCESS POINTS WILL BE EXTENDED OVER THE PROPOSED DUNE. SPREAD SHEET AVAILABLE OF LOCATIONS AND CROSSING TYPE.
2. SAND WILL BE PLACED ON THE SEAWARD SIDE OF ESTABLISHED DUNES. IN ORDER TO REACH PROPOSED HEIGHT.

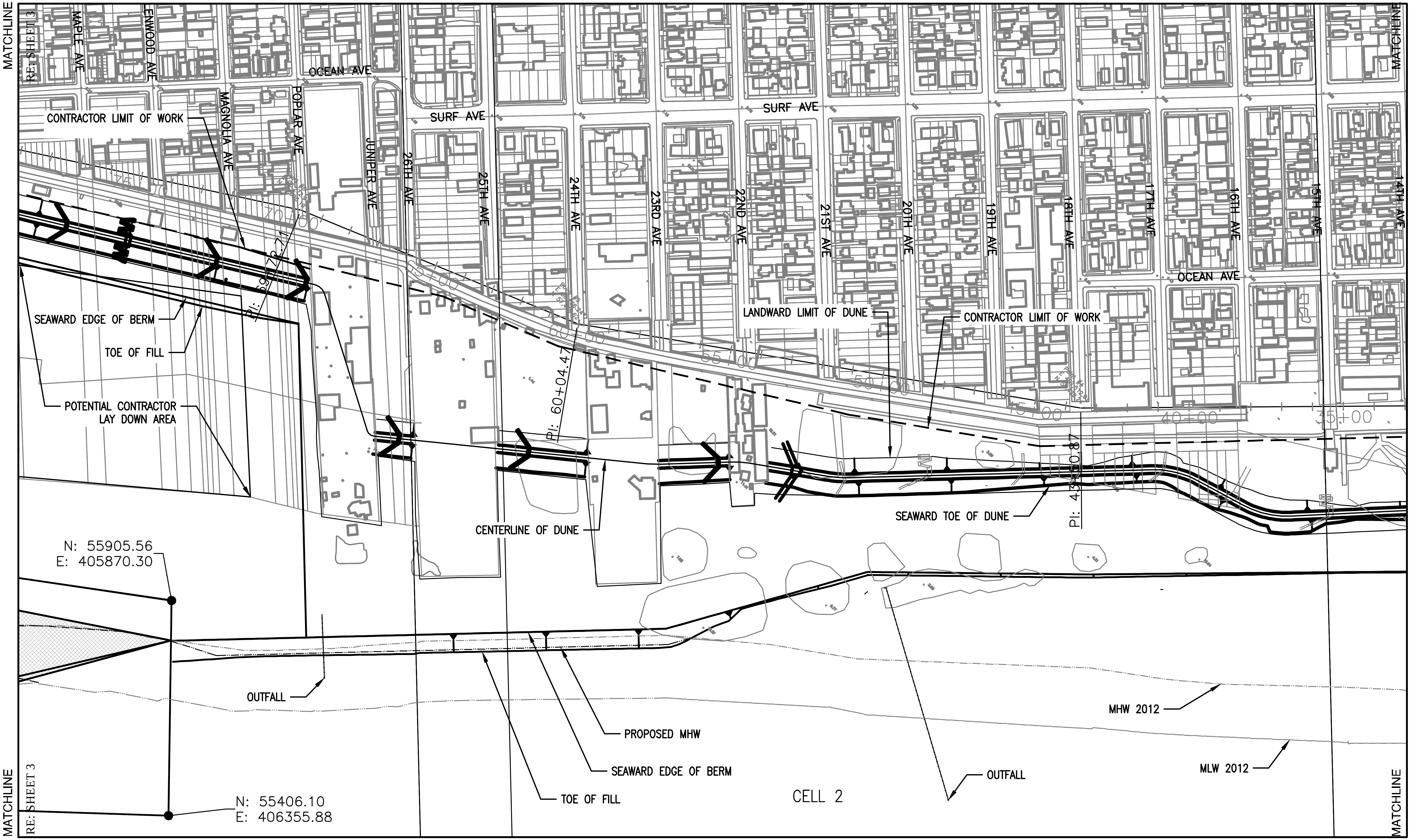
GRAPHIC SCALE
300 0 150 300 600 900
(IN FEET)
1 inch = 300 ft.

HEREFORD INLET TO CAPE MAY INLET FEASIBILITY STUDY

PLAN – BEACH FILL AND BORROW AREA

PHILADELPHIA DISTRICT, CORPS OF ENGINEERS

SHEET 1 OF 6



BORROW AREA

NOTES:

1. ALL EXISTING DUNE CROSSOVERS AND BEACH ACCESS POINTS WILL BE EXTENDED OVER THE PROPOSED DUNE. SPREAD SHEET AVAILABLE OF LOCATIONS AND CROSSING TYPE.
2. SAND WILL BE PLACED ON THE SEAWARD SIDE OF ESTABLISHED DUNES. IN ORDER TO REACH PROPOSED HEIGHT.

GRAPHIC SCALE

300 0 150 300 600 900

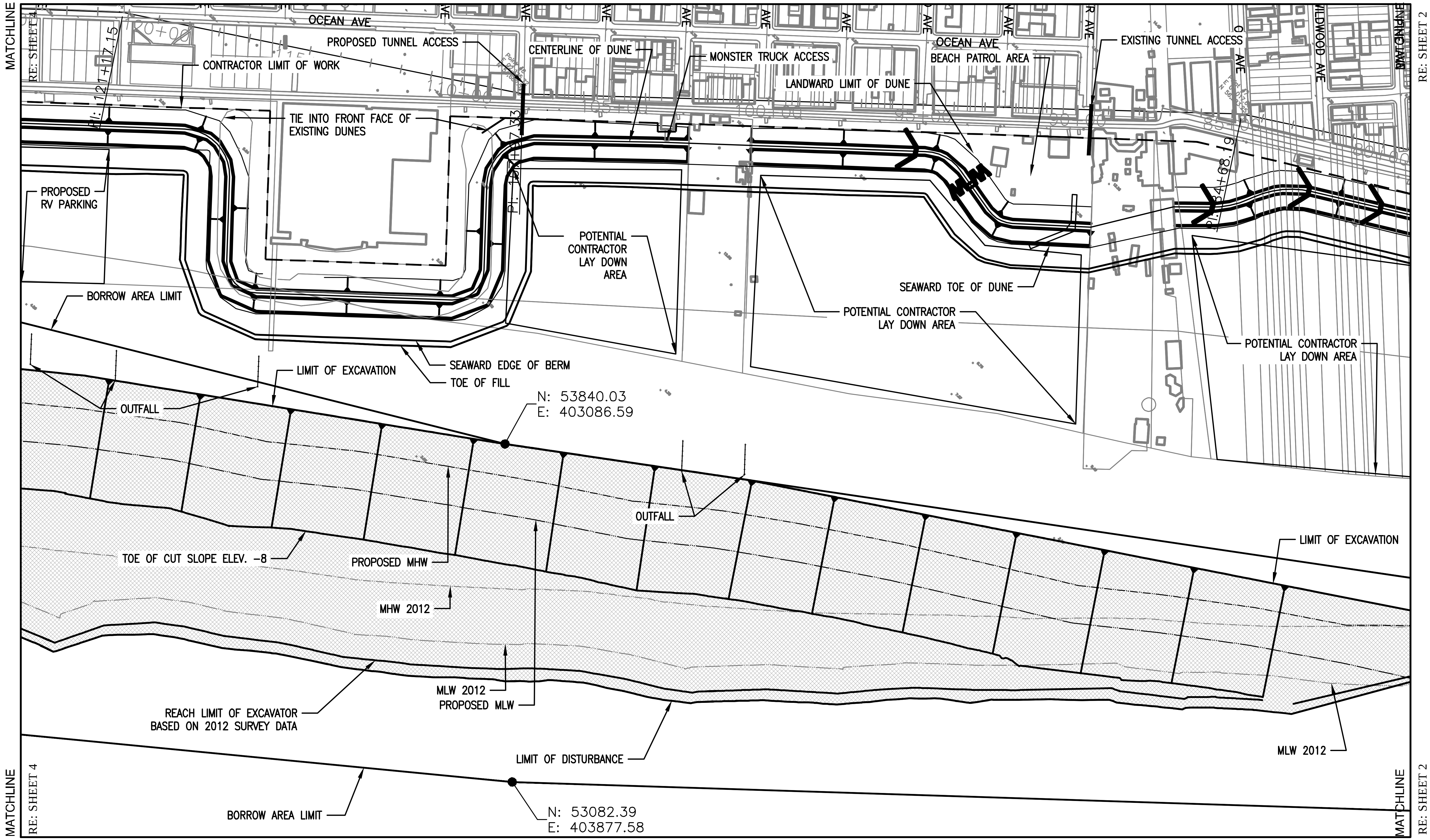
(IN FEET)
1 inch = 300 ft.

HEREFORD INLET TO CAPE MAY INLET FEASIBILITY STUDY

PLAN – BEACH FILL AND BORROW AREA

PHILADELPHIA DISTRICT, CORPS OF ENGINEERS

SHEET 2 OF 6



BORROW AREA

NOTES:
1. ALL EXISTING DUNE CROSSOVERS AND BEACH ACCESS POINTS WILL BE EXTENDED OVER THE PROPOSED DUNE. SPREAD SHEET AVAILABLE OF LOCATIONS AND CROSSING TYPE.
2. SAND WILL BE PLACED ON THE SEAWARD SIDE OF ESTABLISHED DUNES. IN ORDER TO REACH PROPOSED HEIGHT.

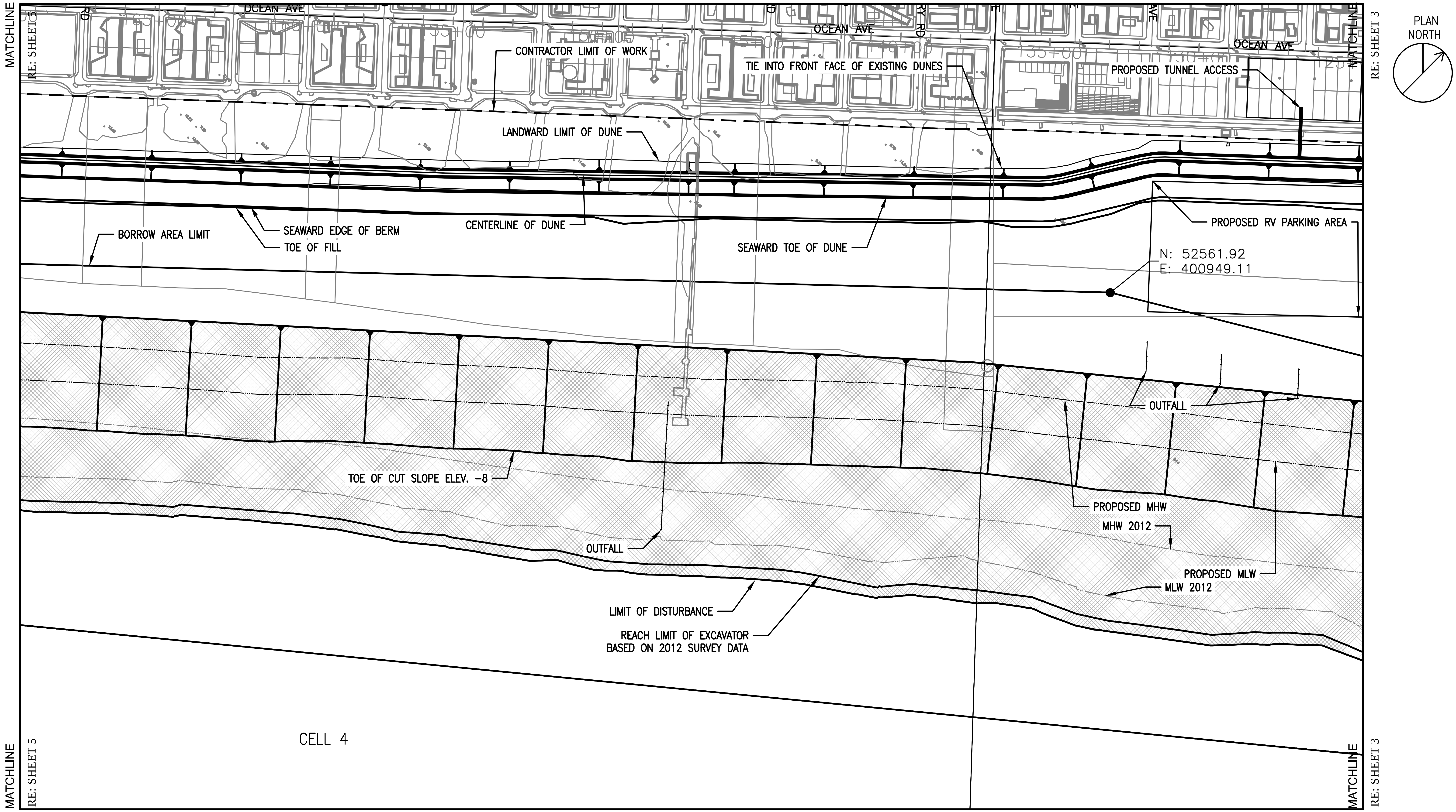
GRAPHIC SCALE
300 0 150 300 600 900
(IN FEET)
1 inch = 300 ft.

HEREFORD INLET TO CAPE MAY INLET FEASIBILITY STUDY

PLAN – BEACH FILL AND BORROW AREA

PHILADELPHIA DISTRICT, CORPS OF ENGINEERS

SHEET 3 OF 6



BORROW AREA

NOTES:

1. ALL EXISTING DUNE CROSSOVERS AND BEACH ACCESS POINTS WILL BE EXTENDED OVER THE PROPOSED DUNE. SPREAD SHEET AVAILABLE OF LOCATIONS AND CROSSING TYPE.
2. SAND WILL BE PLACED ON THE SEAWARD SIDE OF ESTABLISHED DUNES. IN ORDER TO REACH PROPOSED HEIGHT.

GRAPHIC SCALE

300 0 150 300 600 900

(IN FEET)

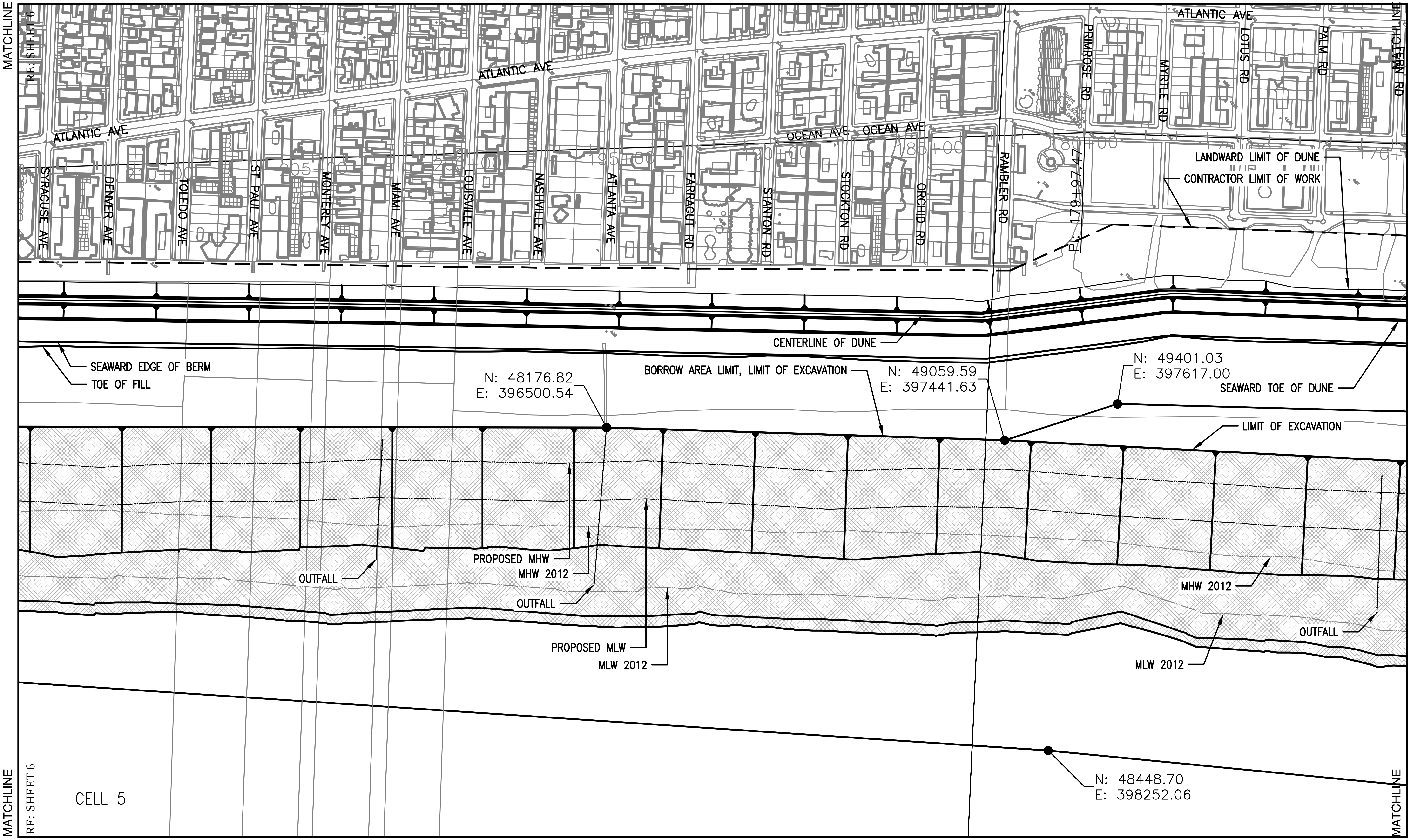
1 inch = 300 ft.

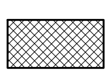
HEREFORD INLET TO CAPE MAY INLET FEASIBILITY STUDY

PLAN – BEACH FILL AND BORROW AREA

PHILADELPHIA DISTRICT, CORPS OF ENGINEERS

SHEET 4 OF 6



 BORROW AREA

NOTES:

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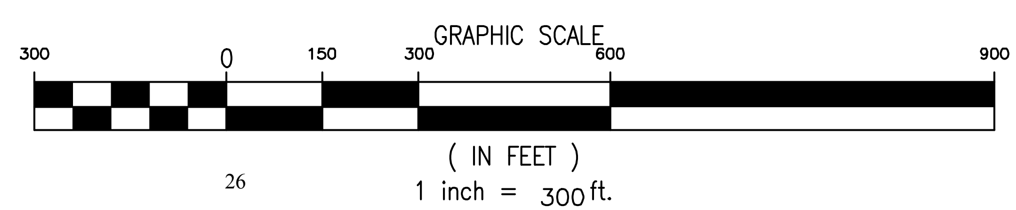
2. SAND WILL BE PLACED ON THE SEAWARD SIDE OF ESTABLISHED DUNES. IN ORDER TO REACH PROPOSED HEIGHT.

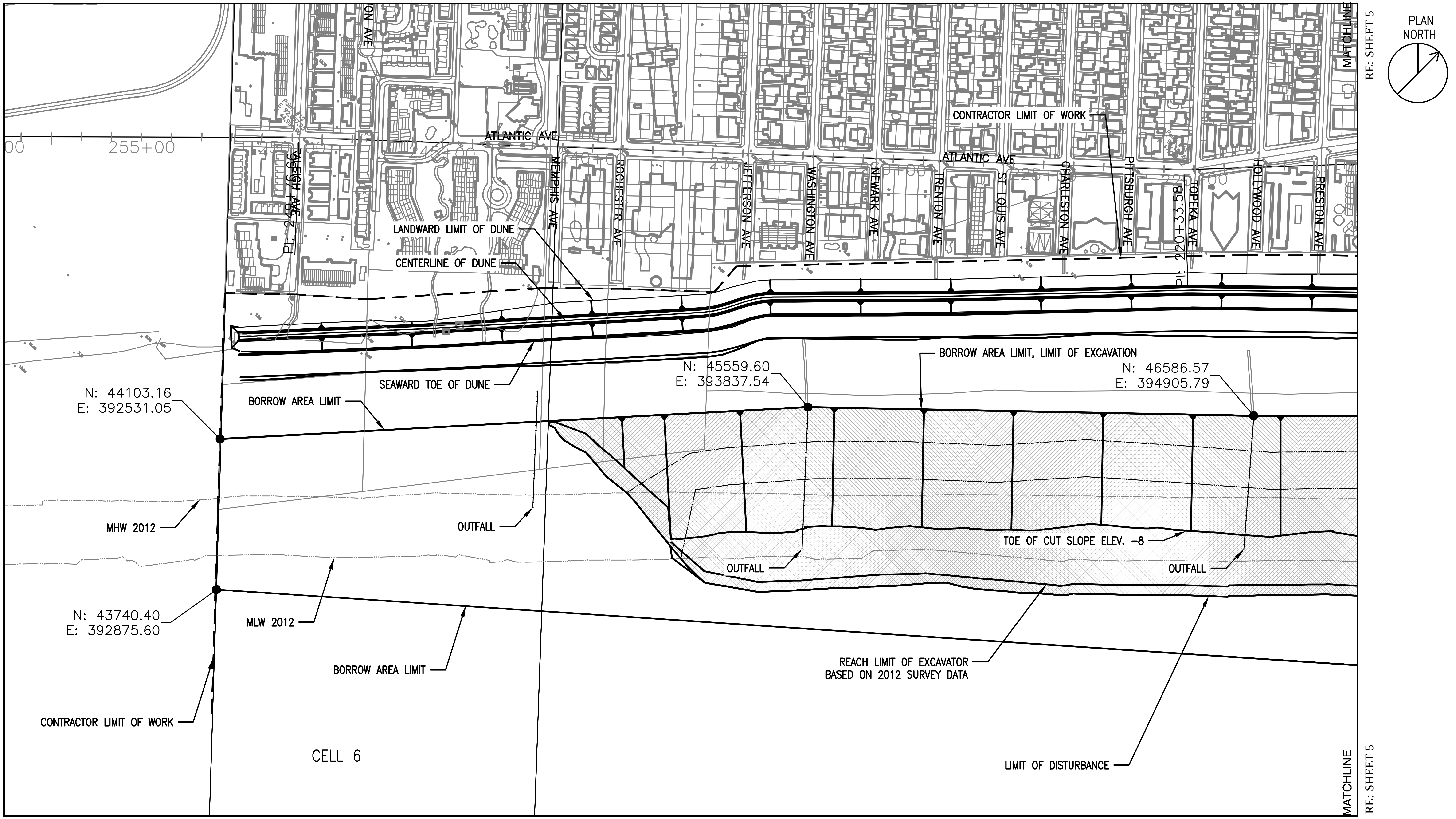


HEREFORD INLET TO CAPE MAY INLET FEASIBILITY STUDY

PLAN – BEACH FILL AND BORROW AREA

PHILADELPHIA DISTRICT, CORPS OF ENGINEERS

SHEET 5 OF 6



BORROW AREA

NOTES:
1. ALL EXISTING DUNE CROSSOVERS AND BEACH ACCESS POINTS WILL BE EXTENDED OVER THE PROPOSED DUNE. SPREAD SHEET AVAILABLE OF LOCATIONS AND CROSSING TYPE.
2. SAND WILL BE PLACED ON THE SEAWARD SIDE OF ESTABLISHED DUNES. IN ORDER TO REACH PROPOSED HEIGHT.

GRAPHIC SCALE
300 0 150 300 600 900
(IN FEET)
1 inch = 300 ft.

HEREFORD INLET TO CAPE MAY INLET FEASIBILITY STUDY

PLAN – BEACH FILL AND BORROW AREA

PHILADELPHIA DISTRICT, CORPS OF ENGINEERS

SHEET 6 OF 6

Section 6

Construction Procedures and Water Control Plan

The specific method of construction to be used by the contractor is not specified beforehand in the contract specification. The contractor will be instructed to back-pass the sand hydraulically from the borrow area to the placement area, but may chose other method to mobilize the material from the borrow area. However, based on the information provide in the feasibility study, the contractor will likely use a mobile crane with an educator or centrifugal pump to mobilize the sand, and series of booster pumps to transport the sand to the placement site. this pipeline will run north south along the beach, likely above the mean high water line on the beach/berm.

The contractor will likely use excavators, bulldozers, and frontend loaders to maneuver the sand around the placement site to get the dune to the design specifications. The discharge pipe will be smaller than traditional 30" pipe used in other projects, and likely be 8" hgh density polyethylene pipe (HDPE) that will be maneuvered by front end loaders with grapple arms. Miscellaneous equipment to be stored on the beach will include a light tower, fuel tank with containment, welding machine, a temporary shelter for construction personal and a site trailer. Upland staging areas will be provided for construction field offices, temporary storage during construction.

Water quality monitoring is described in the environmental section of the main report.

Section 7

Initial Reservoir Filling and Surveillance Plan

N/A

Section 8

Storm Emergency Plan

An emergency plan, New Jersey Hurricane Evacuation Study, was created in 2007 for the Federal Emergency Management Association, the Department of Homeland Security, and the Philadelphia District of the Army Corp of Engineers. A copy of the report can be found at the website below. http://www.state.nj.us/njoem/plan/pdf/maps/hurrevacution_study.pdf

Section 9

Construction Materials

The beachfill material is from an onshore beach borrow source and is fully compatible with the existing beach sand. The dredged material is clean beach sand and chemical contamination is not an issue with this type of material . Vehicular crossovers and pedestrian crossovers will be constructed with pressure treated pine lumber and I-5 type gravel will likely be used as a base material for their construction. This base material will be trucked in from an outside source.

Section 10

Reservoir Clearing

N/A

Section 11

Operation and Maintenance

Operation, Maintenance, repair, replacement and Rehabilitation (OMRR&R) of the completed initial beachfill project is a non Federal Responsibility. The non-Federal sponsor will be furnished with an OMRR&R manual to assist them I carrying out their obligations under ER 1110-2-2902. Some items considered as the non Federal sponsor’s responsibility include dune grass and sand fence, dune crossovers, and some of the project monitoring. Periodic nourishment of the project is expected to occur every 4 years subsequent to the completion of the intial construction and as part of continuing construction, will be a Federal non-Federal cost share responsibility.

Section 12

Access Roads

Most of the work in conjunction with this project will be done in the nearshore. The required equipment will be transported to the project site via local roads in accordance with state and local regulations including a traffic control plan. Exact contractor acess to the beach will be coordinated in the Real Estate plan with the location of Temporary Work Are Easements and contractor lay down areas, and further refined in the plans and specifications.

Section 13

Corrosion Mitigation

N/A

Section 14

Project Security

Initial Construction and periodic Nourishment of the project wil lnecessatatea temporary restrictive closure of a 1,000’-2,000’section of beach as filling operation move along the beach. Sand ramps over the dredge pipe on the beach will be provided at public acess points during construction.

For security and public safety, temporary fencing along with signage will be required around work areas. Contractor personnel will be required to insure security and public safety. Navigation will not impacted by the submerged pipeline and the coast guard will issue a standard notification to mariners. The District addresses project security and public acess in more detail during the Plans and Specs phase.

Section 15

Cost Engineering Appendix

**** TOTAL PROJECT COST SUMMARY ****

Printed:5/20/2014

PROJECT: New Jersey Shore Protection, Hereford Inlet to Cape May Inlet
PROJECT NO: P2 109882
LOCATION: Wildwood, North Wildwood and Wildwood Crest, NJ

DISTRICT: Philadelphia PREPARED: 5/20/2014
POC: ACTING CHIEF, COST ENGINEERING, HARRY P. STEINER

This Estimate reflects the scope and schedule in report; Draft Final Feasibility Study

Civil Works Work Breakdown Structure		ESTIMATED COST				PROJECT FIRST COST (Constant Dollar Basis)						TOTAL PROJECT COST (FULLY FUNDED)			
						Program Year (Budget EC): 2014									
						Effective Price Level Date: 1 OCT 13									
WBS NUMBER	Civil Works Feature & Sub-Feature Description	COST ($K)	CNTG ($K)	CNTG (%)	TOTAL ($K)	ESC (%)	COST ($K)	CNTG ($K)	**TOTAL ($K)**	Spent Thru: **10/1/2013** ($K)	**FIRST COST** ($K)	ESC (%)	COST ($K)	CNTG ($K)	FULL ($K)
A	*B*	*C*	*D*	*E*	*F*	*G*	*H*	*I*	*J*				*M*	*N*	*O*
02	RELOCATIONS	$0	$0	-	$0	-	$0	$0	$0	$0	$0	-	$0	$0	$0
04	DAMS	$0	$0	-	$0	-	$0	$0	$0	$0	$0	-	$0	$0	$0
05	LOCKS	$0	$0	-	$0	-	$0	$0	$0	$0	$0	-	$0	$0	$0
06	FISH & WILDLIFE FACILITIES	$0	$0	-	$0	-	$0	$0	$0	$0	$0	-	$0	$0	$0
17	BEACH REPLENISHMENT - INITIAL	$13,674	$3,432	25.1%	$17,106	0.0%	$13,674	$3,432	$17,106	$0	$17,106	8.1%	$14,787	$3,711	$18,498
	CONSTRUCTION ESTIMATE TOTALS:	$13,674	$3,432		$17,106	0.0%	$13,674	$3,432	$17,106	$0	$17,106	8.1%	$14,787	$3,711	$18,498
01	LANDS AND DAMAGES	$1,019	$254	25%	$1,273	0.0%	$1,019	$254	$1,273	$0	$1,273	7.1%	$1,091	$272	$1,364
30	PLANNING, ENGINEERING & DESIGN	$1,617	$243	15%	$1,860	0.0%	$1,617	$243	$1,860	$4,200	$6,060	11.1%	$1,798	$270	$6,267
31	CONSTRUCTION MANAGEMENT	$1,188	$178	15%	$1,366	0.0%	$1,188	$178	$1,366	$0	$1,366	15.1%	$1,368	$205	$1,573
	PROJECT COST TOTALS:	$17,498	$4,107	23%	$21,605		$17,498	$4,107	**$21,605**	$4,200	**$25,805**	8.8%	$19,043	$4,459	$27,702

______________ ACTING CHIEF, COST ENGINEERING, HARRY P. STEINER

______________ PROJECT MANAGER, BRIAN P. BOGLE

______________ CHIEF, REAL ESTATE, CRAIG R. HOLMESLEY

______________ CHIEF, PLANNING, PETER R. BLUM

______________ CHIEF, ENGINEERING, PETER M. TRANCHIK

______________ CHIEF, OP, xxx

______________ CHIEF, CONSTRUCTION, CHRISTINE D. CLAPP

______________ CHIEF, CONTRACTING, KISHAYRA J. LAMBERT

______________ CHIEF, DP-CW, FRANK R. MASTER

______________ CHIEF, DPM, NATHAN C. BARCOMB

ESTIMATED FEDERAL COST:	**65.0%**	**$18,006**
ESTIMATED NON-FEDERAL COST:	**35.0%**	**$9,696**
INITIAL CG ESTIMATED TOTAL PROJECT COST:		**$27,702**
ESTIMATED FEDERAL COST:	**46.5%**	**$93,126**
ESTIMATED NON-FEDERAL COST:	**53.5%**	**$107,207**
OUT-YEAR (50-YR) FULLY FUNDED COST:		**$200,333**

**** TOTAL PROJECT COST SUMMARY **** Printed:5/20/2014

**** CONTRACT COST SUMMARY ****

PROJECT: New Jersey Shore Protection, Hereford Inlet to Cape May Inlet DISTRICT: Philadelphia PREPARED: 5/20/2014
LOCATION: Wildwood, North Wildwood and Wildwood Crest, NJ POC: ACTING CHIEF, COST ENGINEERING, HARRY P. STEINER
This Estimate reflects the scope and schedule in report; Draft Final Feasibility Study

Civil Works Work Breakdown Structure		ESTIMATED COST				PROJECT FIRST COST (Constant Dollar Basis)				TOTAL PROJECT COST (FULLY FUNDED)				
		Estimate Prepared: 5/20/2014 Effective Price Level: 3/1/2014				Program Year (Budget EC): 2014 Effective Price Level Date: 1 OCT 13								
WBS NUMBER	Civil Works Feature & Sub-Feature Description	COST ($K)	CNTG ($K)	RISK BASED CNTG (%)	TOTAL ($K)	ESC (%)	COST ($K)	CNTG ($K)	TOTAL ($K)	Mid-Point Date	ESC (%)	COST ($K)	CNTG ($K)	FULL ($K)
A	*B*	*C*	*D*	*E*	*F*	*G*	*H*	*I*	*J*	*P*	*L*	*M*	*N*	*O*
	PHASE 1													
02	RELOCATIONS	$0	$0	0%	$0	0.0%	$0	$0	$0	0	0.0%	$0	$0	$0
04	DAMS	$0	$0	0%	$0	0.0%	$0	$0	$0	0	0.0%	$0	$0	$0
05	LOCKS	$0	$0	0%	$0	0.0%	$0	$0	$0	0	0.0%	$0	$0	$0
06	FISH & WILDLIFE FACILITIES	$0	$0	0%	$0	0.0%	$0	$0	$0	0	0.0%	$0	$0	$0
17	BEACH REPLENISHMENT - INITIAL	$13,674	$3,432	25.1%	$17,106	0.0%	$13,674 $0	$3,432	$17,106	2018Q2	8.1%	$14,787	$3,711	$18,498
	CONSTRUCTION ESTIMATE TOTALS:	$13,674	$3,432	25%	**$17,106**		$13,674	$3,432	$17,106			$14,787	$3,711	$18,498
01	LANDS AND DAMAGES	$1,019	$254	25%	$1,273	0.0%	$1,019	$254	$1,273	2017Q4	7.1%	$1,091	$272	$1,364
30	PLANNING, ENGINEERING & DESIGN													
2.3%	Project Management	$310	$47	15%	$357	0.0%	$310	$47	$357	2017Q2	10.8%	$343	$52	$395
3.1%	Planning & Environmental Compliance	$420	$63	15%	$483	0.0%	$420	$63	$483	2017Q2	10.8%	$465	$70	$535
3.6%	Engineering & Design	$487	$73	15%	$560	0.0%	$487	$73	$560	2017Q2	10.8%	$540	$81	$621
0.4%	Reviews, ATRs, IEPRs, VE	$51	$8	15%	$59	0.0%	$51	$8	$59	2017Q2	10.8%	$57	$8	$65
0.5%	Life Cycle Updates (cost, schedule, risks)	$68	$10	15%	$79	0.0%	$68	$10	$79	2017Q2	10.8%	$76	$11	$87
1.1%	Contracting & Reprographics	$154	$23	15%	$177	0.0%	$154	$23	$177	2017Q2	10.8%	$171	$26	$196
0.9%	Engineering During Construction	$127	$19	15%	$146	0.0%	$127	$19	$146	2018Q2	15.1%	$146	$22	$168
0.0%	Planning During Construction	$0	$0	15%	$0	0.0%	$0	$0	$0	0	0.0%	$0	$0	$0
0.0%	Project Operations	$0	$0	15%	$0	0.0%	$0	$0	$0	0	0.0%	$0	$0	$0
31	CONSTRUCTION MANAGEMENT													
8.7%	Construction Management	$1,188	$178	15%	$1,366	0.0%	$1,188	$178	$1,366	2018Q2	15.1%	$1,368	$205	$1,573
0.0%	Project Operation:	$0	$0	15%	$0	0.0%	$0	$0	$0	0	0.0%	$0	$0	$0
0.0%	Project Management	$0	$0	15%	$0	0.0%	$0	$0	$0	0	0.0%	$0	$0	$0
	CONTRACT COST TOTALS:	$17,498	$4,107		**$21,605**		$17,498	$4,107	$21,605			$19,043	$4,459	**$23,502**

WBS Civil Works WBS Feature Description	ESTIMATED COST Risk Based COST ($K)	CNTG ($K)	CNTG (%)	TOTAL ($K)	PROJECT FIRST COST Program Price Level Date: 2014 ESC (%)	COST ($K)	CNTG ($K)	TOTAL ($K)	50-YR COSTS (FULLY FUNDED) INFLATED (%)	COST ($K)	CNTG ($K)	TOTAL ($K)
2 RELOCATIONS												
3 RESERVOIRS												
4 DAMS												
5 LOCKS												
6 FISH & WILDLIFE FACILITIES												
7 POWER PLAN												
8 ROADS RAILROADS & BRIDGES												
9 CHANNELS & CANALS												
10 BREAKWATER & SEAWALLS												
11 LEVEES & FLOODWALLS												
12 NAVIGATION PORTS & HARBORS												
13 PUMPING PLANT												
14 RECREATION FACILITIES												
15 FLOODWAY CONTROL & DIVERSION STRUCTURE												
16 BANK STABILIZATION												
17 BEACH REPLENISHMENT	55,173	16,556	30.01%	71,730	0.00%	55,173	16,556	71,730	88.37%	103,932	31,517	135,448
18 CULTURAL RESOURCE PRESERVATION												
19 BUILDINGS GROUNDS & UTILITIES												
20 PERMANENT OPERATING EQUIPMENT												
30 PLANNING ENGINEERING and DESIGN	2,703	405	15.00%	3,108	0.00%	2,703	405	3,108	375.15%	12,842	1,926	14,768
31 CONSTRUCTION MANAGEMENT	6,601	990	15.00%	7,591	0.00%	6,601	990	7,591	374.73%	31,336	4,700	36,036
TOTALS	64,477	17,952	27.84%	82,428	0.00%	64,477	17,952	82,428	129.71%	148,109	38,143	186,252
99 OMRR&R	5,779	1,721	29.78%	7,500	0.00%	5,779	1,721	7,500	87.13%	10,814	3,266	14,081

		Federal	Non-Federal
Estimated Federal Cost:	50.00%	93,126	
Estimated Non-Federal Cost:	50.00%		93,126
Operation Maintenance Repair Replacement and Rehabilitation (OMRR&R):			14,081
Estimated Total 50-yr Project Nourishment Cost:		93,126	107,207

		Estimated Price Level		Jan-Mar / 2014 2Q 2014	2014.00	Programmed Level Jan-Mar / 2014		2014.00 2Q 2014		Annual Beach Replenishment Costs					
FEATURE	**YEAR**	**COST ($K)**	**CNTG ($K)**	**CNTG (%)**	**TOTAL ($K)**	**ESC (%)**	**COST ($K)**	**CNTG ($K)**	**TOTAL ($K)**	**MID-PT (DATE)**	**MID-PT Jul -Sep**	**INFLATED (%)**	**COST ($K)**	**CNTG ($K)**	**TOTAL ($K)**
17 Beach Replenishment	1	172	43	25.10%	215	0.00%	172	43	215	2019.50	2019	10.99%	190	48	238
	2	120	30	25.10%	150	0.00%	120	30	150	2020.50	2020	13.21%	136	34	170
	3	73	18	25.10%	91	0.00%	73	18	91	2021.50	2021	15.47%	84	21	105
PN	4	4,231	1,062	25.10%	5,293	0.00%	4,231	1,062	5,293	2022.50	2022	17.78%	4,983	1,251	6,234
	5	117	29	25.10%	146	0.00%	117	29	146	2023.50	2023	20.14%	140	35	175
	6	73	18	25.10%	91	0.00%	73	18	91	2024.50	2024	22.54%	89	22	111
	7	73	18	25.10%	91	0.00%	73	18	91	2025.50	2025	24.99%	91	23	113
PN	8	4,231	1,062	25.10%	5,293	0.00%	4,231	1,062	5,293	2026.50	2026	27.49%	5,394	1,354	6,748
	9	117	29	25.10%	146	0.00%	117	29	146	2027.50	2027	30.04%	152	38	190
	10	73	18	25.10%	91	0.00%	73	18	91	2028.50	2028	32.64%	96	24	120
	11	69	21	31.00%	91	0.00%	69	21	91	2029.50	2029	35.30%	93	29	122
PN	12	4,223	1,309	31.00%	5,532	0.00%	4,223	1,309	5,532	2030.50	2030	38.00%	5,828	1,807	7,634
	13	111	35	31.00%	146	0.00%	111	35	146	2031.50	2031	40.76%	157	49	206
	14	69	21	31.00%	91	0.00%	69	21	91	2032.50	2032	43.58%	99	31	130
	15	69	21	31.00%	91	0.00%	69	21	91	2033.50	2033	46.45%	101	31	133
PN	16	4,223	1,309	31.00%	5,532	0.00%	4,223	1,309	5,532	2034.50	2034	49.38%	6,308	1,955	8,264
	17	111	35	31.00%	146	0.00%	111	35	146	2035.50	2035	52.36%	170	53	222
	18	69	21	31.00%	91	0.00%	69	21	91	2036.50	2036	55.41%	107	33	141
	19	69	21	31.00%	91	0.00%	69	21	91	2037.50	2037	58.52%	110	34	143
PN	20	4,223	1,309	31.00%	5,532	0.00%	4,223	1,309	5,532	2038.50	2038	61.69%	6,828	2,117	8,945
	21	111	35	31.00%	146	0.00%	111	35	146	2039.50	2039	64.92%	184	57	241
	22	69	21	31.00%	91	0.00%	69	21	91	2040.50	2040	68.22%	116	36	152
	23	69	21	31.00%	91	0.00%	69	21	91	2041.50	2041	71.59%	119	37	155
MR	24	5,394	1,672	31.00%	7,066	0.00%	5,394	1,672	7,066	2042.50	2042	75.02%	9,440	2,926	12,367
	25	111	35	31.00%	146	0.00%	111	35	146	2043.50	2043	78.52%	199	62	261
	26	69	21	31.00%	91	0.00%	69	21	91	2044.50	2044	82.09%	126	39	165
	27	69	21	31.00%	91	0.00%	69	21	91	2045.50	2045	85.73%	128	40	168
PN	28	4,223	1,309	31.00%	5,532	0.00%	4,223	1,309	5,532	2046.50	2046	89.45%	8,000	2,480	10,480
	29	111	35	31.00%	146	0.00%	111	35	146	2047.50	2047	93.23%	215	67	282
	30	69	21	31.00%	91	0.00%	69	21	91	2048.50	2048	97.10%	136	42	178
	31	69	21	31.00%	91	0.00%	69	21	91	2049.50	2049	101.04%	139	43	182
PN	32	4,223	1,309	31.00%	5,532	0.00%	4,223	1,309	5,532	2050.50	2050	105.06%	8,660	2,684	11,344
	33	111	35	31.00%	146	0.00%	111	35	146	2051.50	2051	109.16%	233	72	305
	34	69	21	31.00%	91	0.00%	69	21	91	2052.50	2052	113.35%	147	46	193
	35	69	21	31.00%	91	0.00%	69	21	91	2053.50	2053	117.61%	150	47	197
PN	36	4,223	1,309	31.00%	5,532	0.00%	4,223	1,309	5,532	2054.50	2054	121.97%	9,373	2,906	12,279
	37	111	35	31.00%	146	0.00%	111	35	146	2055.50	2055	126.41%	252	78	331
	38	69	21	31.00%	91	0.00%	69	21	91	2056.50	2056	130.93%	160	49	209
	39	69	21	31.00%	91	0.00%	69	21	91	2057.50	2057	135.55%	163	50	213
PN	40	4,223	1,309	31.00%	5,532	0.00%	4,223	1,309	5,532	2058.50	2058	140.26%	10,146	3,145	13,291
	41	111	35	31.00%	146	0.00%	111	35	146	2059.50	2059	145.07%	273	85	358
	42	69	21	31.00%	91	0.00%	69	21	91	2060.50	2060	149.97%	173	54	226
	43	69	21	31.00%	91	0.00%	69	21	91	2061.50	2061	154.97%	176	55	231
PN	44	4,223	1,309	31.00%	5,532	0.00%	4,223	1,309	5,532	2062.50	2062	160.07%	10,982	3,405	14,387
	45	111	35	31.00%	146	0.00%	111	35	146	2063.50	2063	165.27%	296	92	387
	46	69	21	31.00%	91	0.00%	69	21	91	2064.50	2064	170.58%	187	58	245
	47	69	21	31.00%	91	0.00%	69	21	91	2065.50	2065	175.99%	191	59	250
PN	48	4,223	1,309	31.00%	5,532	0.00%	4,223	1,309	5,532	2066.50	2066	181.51%	11,888	3,685	15,573
	49	111	35	31.00%	146	0.00%	111	35	146	2067.50	2067	187.14%	320	99	419
	50	69	21	31.00%	91	0.00%	69	21	91	2068.50	2068	192.88%	202	63	265
17 Beach Replenishment		55,173	16,556	30.01%	71,730	0.00%	55,173	16,556	71,730			88.37%	103,932	31,517	135,448
17 Beach Replenishment		**55,173**	**16,556**	**30.01%**	**71,730**	**0.00%**	**55,173**	**16,556**	**71,730**			**88.37%**	**103,932**	**31,517**	**135,448**
Feature Being USED	1 (1=YES, 0=NO)														

						2014.00			2014.00							
			Estimated Price Level				Programmed Level						Annual OMRR&R Costs			
					Jan-Mar - 2014				Jan-Mar - 2014							
	FEATURE	**YEAR**	**COST**	**CNTG**	**CNTG**	**TOTAL**	**ESC**	**COST**	**CNTG**	**TOTAL**	**MID-PT**	**MID-PT**	**INFLATED**	**COST**	**CNTG**	**TOTAL**
			($K)	**($K)**	**(%)**	**($K)**	**(%)**	**($K)**	**($K)**	**($K)**	**(DATE)**	**Jul -Sep**	**(%)**	**($K)**	**($K)**	**($K)**
17_	OMRR&R	1	120	30	25.10%	150	0.00%	120	30	150	2019.50	2019	10.99%	133	33	166
OMRR&R		2	120	30	25.10%	150	0.00%	120	30	150	2020.50	2020	13.21%	136	34	170
		3	120	30	25.10%	150	0.00%	120	30	150	2021.50	2021	15.47%	138	35	173
	PN	4	120	30	25.10%	150	0.00%	120	30	150	2022.50	2022	17.78%	141	35	177
		5	120	30	25.10%	150	0.00%	120	30	150	2023.50	2023	20.14%	144	36	180
		6	120	30	25.10%	150	0.00%	120	30	150	2024.50	2024	22.54%	147	37	184
		7	120	30	25.10%	150	0.00%	120	30	150	2025.50	2025	24.99%	150	38	187
	PN	8	120	30	25.10%	150	0.00%	120	30	150	2026.50	2026	27.49%	153	38	191
		9	120	30	25.10%	150	0.00%	120	30	150	2027.50	2027	30.04%	156	39	195
		10	120	30	25.10%	150	0.00%	120	30	150	2028.50	2028	32.64%	159	40	199
		11	115	35	31.00%	150	0.00%	115	35	150	2029.50	2029	35.30%	155	48	203
	PN	12	115	35	31.00%	150	0.00%	115	35	150	2030.50	2030	38.00%	158	49	207
		13	115	35	31.00%	150	0.00%	115	35	150	2031.50	2031	40.76%	161	50	211
		14	115	35	31.00%	150	0.00%	115	35	150	2032.50	2032	43.58%	164	51	215
		15	115	35	31.00%	150	0.00%	115	35	150	2033.50	2033	46.45%	168	52	220
	PN	16	115	35	31.00%	150	0.00%	115	35	150	2034.50	2034	49.38%	171	53	224
		17	115	35	31.00%	150	0.00%	115	35	150	2035.50	2035	52.36%	174	54	229
		18	115	35	31.00%	150	0.00%	115	35	150	2036.50	2036	55.41%	178	55	233
		19	115	35	31.00%	150	0.00%	115	35	150	2037.50	2037	58.52%	182	56	238
	PN	20	115	35	31.00%	150	0.00%	115	35	150	2038.50	2038	61.69%	185	57	243
		21	115	35	31.00%	150	0.00%	115	35	150	2039.50	2039	64.92%	189	59	247
		22	115	35	31.00%	150	0.00%	115	35	150	2040.50	2040	68.22%	193	60	252
		23	115	35	31.00%	150	0.00%	115	35	150	2041.50	2041	71.59%	196	61	257
	MR	24	115	35	31.00%	150	0.00%	115	35	150	2042.50	2042	75.02%	200	62	263
		25	115	35	31.00%	150	0.00%	115	35	150	2043.50	2043	78.52%	204	63	268
		26	115	35	31.00%	150	0.00%	115	35	150	2044.50	2044	82.09%	208	65	273
		27	115	35	31.00%	150	0.00%	115	35	150	2045.50	2045	85.73%	213	66	279
	PN	28	115	35	31.00%	150	0.00%	115	35	150	2046.50	2046	89.45%	217	67	284
		29	115	35	31.00%	150	0.00%	115	35	150	2047.50	2047	93.23%	221	69	290
		30	115	35	31.00%	150	0.00%	115	35	150	2048.50	2048	97.10%	226	70	296
		31	115	35	31.00%	150	0.00%	115	35	150	2049.50	2049	101.04%	230	71	302
	PN	32	115	35	31.00%	150	0.00%	115	35	150	2050.50	2050	105.06%	235	73	308
		33	115	35	31.00%	150	0.00%	115	35	150	2051.50	2051	109.16%	239	74	314
		34	115	35	31.00%	150	0.00%	115	35	150	2052.50	2052	113.35%	244	76	320
		35	115	35	31.00%	150	0.00%	115	35	150	2053.50	2053	117.61%	249	77	326
	PN	36	115	35	31.00%	150	0.00%	115	35	150	2054.50	2054	121.97%	254	79	333
		37	115	35	31.00%	150	0.00%	115	35	150	2055.50	2055	126.41%	259	80	340
		38	115	35	31.00%	150	0.00%	115	35	150	2056.50	2056	130.93%	264	82	346
		39	115	35	31.00%	150	0.00%	115	35	150	2057.50	2057	135.55%	270	84	353
	PN	40	115	35	31.00%	150	0.00%	115	35	150	2058.50	2058	140.26%	275	85	360
		41	115	35	31.00%	150	0.00%	115	35	150	2059.50	2059	145.07%	281	87	368
		42	115	35	31.00%	150	0.00%	115	35	150	2060.50	2060	149.97%	286	89	375
		43	115	35	31.00%	150	0.00%	115	35	150	2061.50	2061	154.97%	292	91	382
	PN	44	115	35	31.00%	150	0.00%	115	35	150	2062.50	2062	160.07%	298	92	390
		45	115	35	31.00%	150	0.00%	115	35	150	2063.50	2063	165.27%	304	94	398
		46	115	35	31.00%	150	0.00%	115	35	150	2064.50	2064	170.58%	310	96	406
		47	115	35	31.00%	150	0.00%	115	35	150	2065.50	2065	175.99%	316	98	414
	PN	48	115	35	31.00%	150	0.00%	115	35	150	2066.50	2066	181.51%	322	100	422
		49	115	35	31.00%	150	0.00%	115	35	150	2067.50	2067	187.14%	329	102	431
		50	115	35	31.00%	150	0.00%	115	35	150	2068.50	2068	192.88%	335	104	439
17_	OMRR&R		5,779	1,721	29.78%	7,500	0.00%	5,779	1,721	7,500			87.13%	10,814	3,266	14,081
17_	**OMRR&R**		**5,779**	**1,721**	**29.78%**	**7,500**	**0.00%**	**5,779**	**1,721**	**7,500**			**87.13%**	**10,814**	**3,266**	**14,081**
	Feature Being USED	1 (1=YES, 0=NO)														

					2014.00			2014.00							
		Estimated Price Level **Jan-Mar / 2014**				Programmed Level Jan-Mar / 2014				Annual PE&D Costs					
				2Q 2014				2Q 2014							
FEATURE	**YEAR**	**COST ($K)**	**CNTG ($K)**	**CNTG (%)**	**TOTAL ($K)**	**ESC (%)**	**COST ($K)**	**CNTG ($K)**	**TOTAL ($K)**	**MID-PT (DATE)**	**MID-PT Jul -Sep**	**INFLATED (%)**	**COST ($K)**	**CNTG ($K)**	**TOTAL ($K)**
30 Planning Engineering &	1		0	15.00%	0	0.00%	0	0	0	2019.50	2019	21.22%	0	0	0
Design (PE&D)	2		0	15.00%	0	0.00%	0	0	0	2020.50	2020	26.04%	0	0	0
	3		0	15.00%	0	0.00%	0	0	0	2021.50	2021	31.08%	0	0	0
PN	4	222	33	15.00%	255	0.00%	222	33	255	2022.50	2022	36.35%	302	45	348
	5		0	15.00%	0	0.00%	0	0	0	2023.50	2023	41.91%	0	0	0
	6		0	15.00%	0	0.00%	0	0	0	2024.50	2024	47.81%	0	0	0
	7		0	15.00%	0	0.00%	0	0	0	2025.50	2025	54.08%	0	0	0
PN	8	222	33	15.00%	255	0.00%	222	33	255	2026.50	2026	60.74%	356	53	410
	9		0	15.00%	0	0.00%	0	0	0	2027.50	2027	67.80%	0	0	0
	10		0	15.00%	0	0.00%	0	0	0	2028.50	2028	75.30%	0	0	0
	11		0	15.00%	0	0.00%	0	0	0	2029.50	2029	83.30%	0	0	0
PN	12	222	33	15.00%	255	0.00%	222	33	255	2030.50	2030	91.84%	425	64	489
	13		0	15.00%	0	0.00%	0	0	0	2031.50	2031	100.95%	0	0	0
	14		0	15.00%	0	0.00%	0	0	0	2032.50	2032	110.67%	0	0	0
	15		0	15.00%	0	0.00%	0	0	0	2033.50	2033	121.01%	0	0	0
PN	16	222	33	15.00%	255	0.00%	222	33	255	2034.50	2034	132.09%	514	77	592
	17		0	15.00%	0	0.00%	0	0	0	2035.50	2035	143.87%	0	0	0
	18		0	15.00%	0	0.00%	0	0	0	2036.50	2036	156.25%	0	0	0
	19		0	15.00%	0	0.00%	0	0	0	2037.50	2037	169.25%	0	0	0
PN	20	222	33	15.00%	255	0.00%	222	33	255	2038.50	2038	182.91%	627	94	721
	21		0	15.00%	0	0.00%	0	0	0	2039.50	2039	197.27%	0	0	0
	22		0	15.00%	0	0.00%	0	0	0	2040.50	2040	212.36%	0	0	0
	23		0	15.00%	0	0.00%	0	0	0	2041.50	2041	228.21%	0	0	0
MR	24	265	40	15.00%	304	0.00%	265	40	304	2042.50	2042	244.86%	913	137	1,050
	25		0	15.00%	0	0.00%	0	0	0	2043.50	2043	262.36%	0	0	0
	26		0	15.00%	0	0.00%	0	0	0	2044.50	2044	280.75%	0	0	0
	27		0	15.00%	0	0.00%	0	0	0	2045.50	2045	300.07%	0	0	0
PN	28	222	33	15.00%	255	0.00%	222	33	255	2046.50	2046	320.38%	932	140	1,071
	29		0	15.00%	0	0.00%	0	0	0	2047.50	2047	341.71%	0	0	0
	30		0	15.00%	0	0.00%	0	0	0	2048.50	2048	364.12%	0	0	0
	31		0	15.00%	0	0.00%	0	0	0	2049.50	2049	387.68%	0	0	0
PN	32	222	33	15.00%	255	0.00%	222	33	255	2050.50	2050	412.43%	1,136	170	1,306
	33		0	15.00%	0	0.00%	0	0	0	2051.50	2051	438.43%	0	0	0
	34		0	15.00%	0	0.00%	0	0	0	2052.50	2052	465.75%	0	0	0
	35		0	15.00%	0	0.00%	0	0	0	2053.50	2053	494.46%	0	0	0
PN	36	222	33	15.00%	255	0.00%	222	33	255	2054.50	2054	524.63%	1,384	208	1,592
	37		0	15.00%	0	0.00%	0	0	0	2055.50	2055	556.33%	0	0	0
	38		0	15.00%	0	0.00%	0	0	0	2056.50	2056	589.64%	0	0	0
	39		0	15.00%	0	0.00%	0	0	0	2057.50	2057	624.63%	0	0	0
PN	40	222	33	15.00%	255	0.00%	222	33	255	2058.50	2058	661.41%	1,688	253	1,941
	41		0	15.00%	0	0.00%	0	0	0	2059.50	2059	700.05%	0	0	0
	42		0	15.00%	0	0.00%	0	0	0	2060.50	2060	740.65%	0	0	0
	43		0	15.00%	0	0.00%	0	0	0	2061.50	2061	783.31%	0	0	0
PN	44	222	33	15.00%	255	0.00%	222	33	255	2062.50	2062	828.13%	2,057	309	2,366
	45		0	15.00%	0	0.00%	0	0	0	2063.50	2063	875.23%	0	0	0
	46		0	15.00%	0	0.00%	0	0	0	2064.50	2064	924.72%	0	0	0
	47		0	15.00%	0	0.00%	0	0	0	2065.50	2065	976.72%	0	0	0
PN	48	222	33	15.00%	255	0.00%	222	33	255	2066.50	2066	1031.36%	2,507	376	2,884
	49		0	15.00%	0	0.00%	0	0	0	2067.50	2067	1088.78%	0	0	0
	50		0	15.00%	0	0.00%	0	0	0	2068.50	2068	1149.10%	0	0	0
30 Planning Engineering &		2,703	405	15.00%	3,108	0.00%	2,703	405	3,108			375.15%	12,842	1,926	14,768
30 Planning Engineering &		**2,703**	**405**	**15.00%**	**3,108**	**0.00%**	**2,703**	**405**	**3,108**			**375.15%**	**12,842**	**1,926**	**14,768**
Feature Being USED	1 (1=YES, 0=NO)														

					2014.00			2014.00							
		Estimated Price Level		Jan-Mar / 2014		Programmed Level Jan-Mar / 2014				Annual Construction Management Costs					
				2Q 2014				2Q 2014							
FEATURE	**YEAR**	**COST ($K)**	**CNTG ($K)**	**CNTG (%)**	**TOTAL ($K)**	**ESC (%)**	**COST ($K)**	**CNTG ($K)**	**TOTAL ($K)**	**MID-PT (DATE)**	**MID-PT Jul -Sep**	**INFLATED (%)**	**COST ($K)**	**CNTG ($K)**	**TOTAL ($K)**
31 Construction Management (S&A)	1		0	15.00%	0	0.00%	0	0	0	2019.50	2019	21.22%	0	0	0
	2		0	15.00%	0	0.00%	0	0	0	2020.50	2020	26.04%	0	0	0
	3		0	15.00%	0	0.00%	0	0	0	2021.50	2021	31.08%	0	0	0
PN	4	540	81	15.00%	620	0.00%	540	81	620	2022.50	2022	36.35%	736	110	846
	5		0	15.00%	0	0.00%	0	0	0	2023.50	2023	41.91%	0	0	0
	6		0	15.00%	0	0.00%	0	0	0	2024.50	2024	47.81%	0	0	0
	7		0	15.00%	0	0.00%	0	0	0	2025.50	2025	54.08%	0	0	0
PN	8	540	81	15.00%	620	0.00%	540	81	620	2026.50	2026	60.74%	867	130	997
	9		0	15.00%	0	0.00%	0	0	0	2027.50	2027	67.80%	0	0	0
	10		0	15.00%	0	0.00%	0	0	0	2028.50	2028	75.30%	0	0	0
	11		0	15.00%	0	0.00%	0	0	0	2029.50	2029	83.30%	0	0	0
PN	12	540	81	15.00%	620	0.00%	540	81	620	2030.50	2030	91.84%	1,035	155	1,190
	13		0	15.00%	0	0.00%	0	0	0	2031.50	2031	100.95%	0	0	0
	14		0	15.00%	0	0.00%	0	0	0	2032.50	2032	110.67%	0	0	0
	15		0	15.00%	0	0.00%	0	0	0	2033.50	2033	121.01%	0	0	0
PN	16	540	81	15.00%	620	0.00%	540	81	620	2034.50	2034	132.09%	1,252	188	1,440
	17		0	15.00%	0	0.00%	0	0	0	2035.50	2035	143.87%	0	0	0
	18		0	15.00%	0	0.00%	0	0	0	2036.50	2036	156.25%	0	0	0
	19		0	15.00%	0	0.00%	0	0	0	2037.50	2037	169.25%	0	0	0
PN	20	540	81	15.00%	620	0.00%	540	81	620	2038.50	2038	182.91%	1,526	229	1,755
	21		0	15.00%	0	0.00%	0	0	0	2039.50	2039	197.27%	0	0	0
	22		0	15.00%	0	0.00%	0	0	0	2040.50	2040	212.36%	0	0	0
	23		0	15.00%	0	0.00%	0	0	0	2041.50	2041	228.21%	0	0	0
MR	24	666	100	15.00%	766	0.00%	666	100	766	2042.50	2042	244.86%	2,296	344	2,640
	25		0	15.00%	0	0.00%	0	0	0	2043.50	2043	262.36%	0	0	0
	26		0	15.00%	0	0.00%	0	0	0	2044.50	2044	280.75%	0	0	0
	27		0	15.00%	0	0.00%	0	0	0	2045.50	2045	300.07%	0	0	0
PN	28	540	81	15.00%	620	0.00%	540	81	620	2046.50	2046	320.38%	2,268	340	2,608
	29		0	15.00%	0	0.00%	0	0	0	2047.50	2047	341.71%	0	0	0
	30		0	15.00%	0	0.00%	0	0	0	2048.50	2048	364.12%	0	0	0
	31		0	15.00%	0	0.00%	0	0	0	2049.50	2049	387.68%	0	0	0
PN	32	540	81	15.00%	620	0.00%	540	81	620	2050.50	2050	412.43%	2,765	415	3,180
	33		0	15.00%	0	0.00%	0	0	0	2051.50	2051	438.43%	0	0	0
	34		0	15.00%	0	0.00%	0	0	0	2052.50	2052	465.75%	0	0	0
	35		0	15.00%	0	0.00%	0	0	0	2053.50	2053	494.46%	0	0	0
PN	36	540	81	15.00%	620	0.00%	540	81	620	2054.50	2054	524.63%	3,370	506	3,876
	37		0	15.00%	0	0.00%	0	0	0	2055.50	2055	556.33%	0	0	0
	38		0	15.00%	0	0.00%	0	0	0	2056.50	2056	589.64%	0	0	0
	39		0	15.00%	0	0.00%	0	0	0	2057.50	2057	624.63%	0	0	0
PN	40	540	81	15.00%	620	0.00%	540	81	620	2058.50	2058	661.41%	4,108	616	4,724
	41		0	15.00%	0	0.00%	0	0	0	2059.50	2059	700.05%	0	0	0
	42		0	15.00%	0	0.00%	0	0	0	2060.50	2060	740.65%	0	0	0
	43		0	15.00%	0	0.00%	0	0	0	2061.50	2061	783.31%	0	0	0
PN	44	540	81	15.00%	620	0.00%	540	81	620	2062.50	2062	828.13%	5,008	751	5,759
	45		0	15.00%	0	0.00%	0	0	0	2063.50	2063	875.23%	0	0	0
	46		0	15.00%	0	0.00%	0	0	0	2064.50	2064	924.72%	0	0	0
	47		0	15.00%	0	0.00%	0	0	0	2065.50	2065	976.72%	0	0	0
PN	48	540	81	15.00%	620	0.00%	540	81	620	2066.50	2066	1031.36%	6,104	916	7,020
	49		0	15.00%	0	0.00%	0	0	0	2067.50	2067	1088.78%	0	0	0
	50		0	15.00%	0	0.00%	0	0	0	2068.50	2068	1149.10%	0	0	0
31 Construction Management		6,601	990	15.00%	7,591	0.00%	6,601	990	7,591			374.73%	31,336	4,700	36,036
31 Construction Management		**6,601**	**990**	**15.00%**	**7,591**	**0.00%**	**6,601**	**990**	**7,591**			**374.73%**	**31,336**	**4,700**	**36,036**
Feature Being USED	1	(1=YES, 0=NO)													

SECTION 15 - COST ESTIMATE

LIST OF TABLES USING MOBILE HYDRAULIC SAND BACK-PASSING FOR BEACH FILL

INITIAL PROJECT CHARGES USING HOPPER DREDGING FOR BEACH FILL

LIST OF TABLES USING HOPPER DREDGING FOR BEACH FILL

SECTION 15 - COST ESTIMATE

1. Introduction: Two separate beach fill methods were considered for this project: mobile hydraulic sand back-passing and hopper dredging. Mobile hydraulic sand back passing will be discussed first followed by hopper dredging

INITIAL PROJECT CHARGES USING MOBILE HYDRAULIC SAND BACK-PASSING FOR BEACH FILL

2. General: This section presents detailed cost estimates for initial construction, nourishment, maintenance, monitoring and major replacement resulting in total and annualized project costs for alternative storm damage reduction plans for mobile hydraulic sand back-passing. The fifteen alternative plans developed for mobile hydraulic sand back-passing include:

Plan	Description
A	115' wide berm with +12' NAVD dune using 4 Yr. Cycle
B	95' wide berm with +14' NAVD dune using 4 Yr. Cycle
C	75' wide berm with +16' NAVD dune using 4 Yr. Cycle
D	140' wide berm with +12' NAVD dune using 4 Yr. Cycle
E	120' wide berm with +14' NAVD dune using 4 Yr. Cycle
F	100' wide berm with +16' NAVD dune using 4 Yr. Cycle
G	165' wide berm with +12' NAVD dune using 4 Yr. Cycle
H	145' wide berm with +14' NAVD dune using 4 Yr. Cycle
I	125' wide berm with +16' NAVD dune using 4 Yr. Cycle
J	80' wide berm with +18' NAVD dune using 4 Yr. Cycle
K	105' wide berm with +18' NAVD dune using 4 Yr. Cycle
L	85' wide berm with +20' NAVD dune using 4 Yr. Cycle
M	110' wide berm with +20' NAVD dune using 4 Yr. Cycle
N	160' wide berm with +20' NAVD dune using 4 Yr. Cycle
O	No action

The top of the berm is at an elevation of +6.5' NAVD and extends from 2nd Avenue in North Wildwood to Juniper Avenue in Wildwood. The dune for each alternative has 1 on 5 side slopes and a top width of 25'. The dune extends the same distance as the berm. The initial construction for each of the above plans includes design and advanced nourishment beach fill. Also included are provisions for periodic nourishment, beach profile and environmental monitoring, and major replacement to restore the design beach profile damaged by significant storm events beyond that designed for in the nourishment cycle quantity. The plan layout of the NED plan with typical improved beach sections is shown in the section of the Feasibility Study, Main Report describing the NED Plan.

3. Basis of Cost: Cost estimates presented herein for the Cycle 3 analysis are based on June 2007 price levels. Initial beach fill costs are based on beach surveys taken in October 2003. The unit prices were developed in accordance with the construction procedures outlined herein. All initial construction, nourishment costs, and major replacement costs presented in this appendix are NED costs.

4. Initial fill costs are based on the assumption that mobile hydraulic sand back-passing was used for placement of the beach fill. Approximately 975,000 C.Y. of beach fill from onshore borrow area WW/WC was placed in Cells 1 and 2. The average pumping distance for the initial beach fill uses an average pipeline length of 14,667 L.F. A 150 hp electric submersible agitator dredge pump would be suspended from a crawler crane with a minimum 100' boom. A 300 kW generator would be mounted on the back of the crane to power the dredge pump. Sand up to 50% solids by weight, to be transferred through 10-inch gum tube-lined dredge hose to 12-inch HDPE pipe on the beach. Diesel engine, skid mounted booster pumps would be placed every 5,000 feet to transfer the sand slurry to the outlet location where the beach fill would take place. Horsepower for each booster is 400 hp. Instrumentation including magnetic flow meters would provide flow rates and production numbers. A 350 C.Y. per hour production rate was used for cost estimating purposes and is based on the Sand Bypass Plant, Indian River Inlet, Delaware Coast Protection job constructed by NAP in 1989 and operated by Delaware Department of Natural Resources and Environmental Control (DENREC).

5. Periodic nourishment fill costs are based on the assumption that mobile hydraulic sand back-passing was used for placement of the beach fill. Approximately 366,000 C.Y. of beach fill material from onshore borrow area WW/WC was placed in Cells 1 and 2. The average pumping distance for the nourishment cycle uses an average pipeline length of 14,667 L.F. The placement of this material will follow the constructability outlines in paragraph 4.

6. Mobilization and demobilization costs are based on the assumption that beach filling equipment located within 250 miles from the project site will perform the work. Mobilization and demobilization costs also include subcontractor mob and demob. Construction access would be by local streets. The locations of the borrow areas are displayed in the section of the Feasibility Study, Main Report describing the NED Plan.

7. Real estate costs for the fifteen alternatives included in the Cycle 3 screening were not included since they are expected to be minimal as most of the land is a public beach owned by the sponsor. Real estate costs as shown in Table 1 are included as NED costs and reflect acquisition of easements on private beach and include surveys, appraisal, and administrative costs between the limits of beach filling. For more information refer to the Real Estate Appendix.

8. Environmental monitoring costs for the fifteen alternatives included in the Cycle 3 screening were not included since they are dependent on the EA document and that document was not finalized at the time of the Cycle 3 screening.

9. Construction Management costs for the fifteen alternatives included in the Cycle 3 screening were included as a percent of the construction cost and is based on ER415-1-16, Table E-1. A 15 percent contingency has been included in S&A costs.

10. Contingency allowances used for the fifteen alternatives included in the Cycle 3 screening were 15 percent for the beach fill work and 12 percent for the mobilization and demobilization work and is based on EM1110-2-1301, Appendix C.

11. <u>Alternatives Considered</u>: Alternative plans were developed in two phases for the plan selection process. In the first phase the alternative plans were compared during the Cycle 1 and Cycle 2 screening process. For more information on these plans, refer to the section of the

Feasibility Study, Main Report describing the NED Plan. Based on an analysis of these annual costs with their associated benefits, the beach restoration only plan was selected for the second phase for final plan optimization and selection.

12. The costs for the fifteen alternatives as described in paragraph 2 for this second phase of plan selection are shown in Tables 2A thru 2N.

13. Renourishment Interval Optimization: For more information on the renourishment interval optimization that selected the 4-year cycle, refer to the section of the Feasibility Study, Main Report describing the NED plan.

14. Total First Cost for Selected Plan: The estimated project first cost is for the selected plan - a dune and berm constructed using 1,007,250 CY of sand obtained from onshore borrow area WW/WC located on the beach in Wildwood and Wildwood Crest. A +16’ NAVD high dune with a top width of 25’ on a 75’ wide berm that is 6.5’ NAVD high would be constructed from North Wildwood to the northern border of Wildwood and is based on a selected nourishment cycle of 4 years. In Wildwood and Wildwood Crest, the project will consist of placing 520,000 CY of beach fill to construct a +16’ NAVD high dune and raising the elevation of the existing berm to 6.5’ NAVD. Side slopes for the dunes will be 1 on 5. The average pumping distance for the initial beach fill uses an average pipeline length of 15,600 L.F. In Wildwood and Wildwood Crest, the average pipeline length is 1,000 L.F. Also included is the placement of 64 acres of dune grass, 28,000 L.F of sand fence, extending 44 existing pedestrian crossovers, 7 new pedestrian crossovers, extending 7 existing handicap crossovers, 6 new handicap crossovers, extending 8 existing vehicle crossovers and 5 new vehicle crossovers. It was assumed that beach filling work would be performed by an earth moving contractor and the work for installing the dune appurtenances performed by a subcontractor. NED real estate acquisition costs and pertinent contingency, engineering and design and construction management costs are also included. Details of the initial construction cost estimate are shown in Table 1.

ANNUAL CHARGES FOR THE SELECTED PLAN

15. General: The estimate of annual charges for the selected plan is based on an economic project life of 50 years, an interest rate of 3.50% and a March 2014 price level. The annual charges include annualized first cost and interest during construction, the annualized periodic nourishment costs, post construction monitoring costs, and OMRR&R costs. It is noted that interest during construction was developed for the first cost of the project constructed over a nine-month period. For the selected plan, the total annualized cost is $2,688,000.

16. Periodic Nourishment: The periodic nourishment volume to be placed at 4 year cycles, subsequent to commencement of construction and throughout the 50 year economic life is 391,250 C.Y. from onshore borrow area WW/WC. Mobile hydraulic back-passing was used for placement of the beach fill. This volume includes overfill and tolerance. The placement of this material will follow the constructability outlines in paragraph 4. For more details on the development of the periodic nourishment quantity refer to the section of the Feasibility Study, Main Report describing the NED Plan. The borrow area for periodic nourishment are also shown in the section of the Feasibility Study, Main Report describing the NED Plan. Periodic nourishment costs for the selected cycle are shown in Tables 3 and 4.

17. Major Replacement Costs: Major replacement costs are included as an additional cost for significant storm events beyond that designed for in the selected nourishment cycle to restore the design profile. The major replacement losses are computed as the losses that would occur from the 50% risk event over the project life. For more detail on the development of the major replacement quantity, refer to the section of the Feasibility Study, Main Report describing the NED Plan. Major replacement costs are shown in Table 5.

18. OMRR&R Costs: OMRR&R costs for the selected plan were estimated to be $150,000 annually and cover maintenance of the beaches, dune grass, sand fencing, dune crossovers and some project monitoring.

19. Monitoring Costs: Post construction monitoring costs include coastal and environmental monitoring over the 50-year project life. Average annualized monitoring costs are $140,000.

CONTINGENCIES, PRECONSTRUCTION ENGINEERING & DESIGN, AND CONSTRUCTION MANAGEMENT FOR THE SELECTED PLAN

20. Contingencies: The estimated cost for each major subdivision or feature of the recommended project includes an item for "contingencies". The item for "contingencies" is an allowance against some adverse or unanticipated condition not susceptible to exact evaluation from the data at hand but which must be expressed or represented in the cost estimate. The contingency allowances used in the development of the cost estimate for the selected project were estimated as an appropriate percentage using Crystal Ball software for preparing risk analysis. 25.1 percent was applied to beach placement work for years 0 to 10 and 31 percent was applied to beach placement work for years 11 to 50 to account for concerns about pumping distances and borrow area selection, and to account for larger required beach fill quantities at the time of construction due to future preconstruction erosion, concerns about availability of pumping equipment, variances in the travel distance for the pump plant, and for increases in labor and fuel prices.

21. Preconstruction Engineering & Design (P, E & D): Preconstruction Engineering and Design costs include local cooperative agreements, environmental and regulatory activities, general design memorandum, preparation of plans and specifications, engineering during construction, A/E liability actions, cost engineering, construction and supply contract award activities, project management, and the development of the PCA. P, E & D costs were estimated as lump sums of $1,859,894 for the initial beach fill construction, $254,877 for the nourishment cycle, and $304,402 for the major replacement and are based on similar Corps of Engineers projects of the same magnitude and include 0.5% of construction costs to cover NAD labor requirements. A contingency factor of 15% is included in the P, E & D costs.

22. Construction Management (S&A): Construction Management costs include contract administration, review of shop drawings, inspection and quality assurance, project office operation, contractor initiated claims and litigations, and government initiated claims and litigations. S&A related costs were estimated as lump sums of $1,366,020 for the initial beach fill construction, $620,485 for the nourishment cycle, and $765,577 for the major replacement and were based on similar Corps of Engineers projects of the same magnitude. A contingency factor of 15% was included in all S&A costs.

CONSTRUCTION AND PROJECT SCHEDULE FOR THE SELECTED PLAN

23. General: The construction and project schedules of the selected plan are shown in Tables 6 and 7 of this Engineering Technical Appendix. The schedule is based on the timeliness of the report's approval and allocation of funds by Congress, the foregoing construction procedures, and the ability of local interests to implement the necessary items of local cooperation.

Table 1 - Total First Cost - Selected Plan
Plan C (75' Berm w/ 16' NAVD Dune using 4 Yr. Cycle)

Price Level: Mar 14
Construction duration: 9-months

ACCOUNT NUMBER	DESCRIPTION OF ITEM	QUANTITY	UOM	UNIT PRICE	ESTIMATED AMOUNT	CONTINGENCY	TOTAL COST
						@ 25.0%	
01.	Lands and Damages	1	Job	LS	$1,018,972	$254,539	$1,273,511
17.	Beach Replenishment					**@ 25.1%**	
17.01	Mobilization, Demob. And Preparatory Work	1	Job	LS	$1,026,656	$257,691	$1,284,346
17.70	Beach Fill	1	Job	LS	$9,883,656	$2,480,798	$12,364,454
17.99	Associated General Items	1	Job	LS	$2,763,564	$693,655	$3,457,219
	Total Beach Replenishment				$13,673,876	$3,432,143	$17,106,019
						@ 15.0%	
30.	Planning, Engineering and Design (P,E & D)	1	Job	LS	$1,617,299	$242,595	$1,859,894
31.	Construction Management (S & A)	1	Job	LS	$1,187,843	$178,177	$1,366,020
	Total Project First Cost				$17,497,990	$4,107,453	$21,605,444
	(Rounded)				$17,498,000	$4,107,000	$21,605,000

Notes:
Beachfill quantity includes 4 yr. nourishment cycle.

Table 2A - Total First Cost Price Level: Jun 07

Plan A (115' Berm w/ 12' NAVD Dune using 4 Yr. Cycle)

ACCOUNT NUMBER	DESCRIPTION OF ITEM	QUANTITY	UOM	UNIT PRICE	ESTIMATED AMOUNT	CONTINGENCY	TOTAL COST
01.	Lands and Damages	1	Job	LS	$0	$0	$0
17.	Beach Replenishment						
	Mobilization, Demob. And Preparatory Work	1	Job	LS	$426,853	$51,222	$478,075
	Beach Fill						
	Site Work - Cells 1 and 2						
	Excavation/Pumping Sand	879,000	CY	$6.66	$5,852,382	$877,857	$6,730,239
	Survey Crew @ Borrow Area D	5.71	Mo.	$144,980	$827,836	$124,175	$952,011
	Survey Crew @ Berm w/ Dune	5.71	Mo.	$87,471	$499,459	$74,919	$574,378
	Grading @ Berm w/ Dune	5.71	Mo.	$128,172	$731,862	$109,779	$841,641
	Site Security	5.72	Mo.	$6,572	$37,560	$5,634	$43,194
	Night Lighting	5.71	Mo.	$47,044	$268,668	$40,300	$308,969
	Total Beach Replacement				$8,644,620	$1,283,887	$9,928,508
30.	Planning, Engineering and Design (P,E & D)	1	Job	LS	$1,000,000	$150,000	$1,150,000
31.	Construction Management (S & A @ 8.60%)	1	Job	LS	$743,437	$111,516	$854,953
	Total Project First Cost				$10,388,058	$1,545,403	$11,933,461
	(Rounded)				$10,388,000	$1,545,000	$11,933,000

Notes:

Beach fill quantity includes 4 yr. nourishment cycle.

Table 2B - Total First Cost Price Level: Jun 07

Plan B (95' Berm w/ 14' NAVD Dune using 4 Yr. Cycle)

ACCOUNT NUMBER	DESCRIPTION OF ITEM	QUANTITY	UOM	UNIT PRICE	ESTIMATED AMOUNT	CONTINGENCY	TOTAL COST
01.	Lands and Damages	1	Job	LS	$0	$0	$0
17.	Beach Replenishment						
	Mobilization, Demob. And Preparatory Work	1	Job	LS	$444,860	$53,383	$498,243
	Beach Fill						
	Site Work - Cells 1 and 2						
	Excavation/Pumping Sand	922,000	CY	$6.66	$6,145,038	$921,756	$7,066,793
	Survey Crew @ Borrow Area D	5.99	Mo.	$144,980	$869,025	$130,354	$999,378
	Survey Crew @ Berm w/ Dune	5.99	Mo.	$87,471	$524,301	$78,645	$602,946
	Grading @ Berm w/ Dune	5.99	Mo.	$128,172	$767,750	$115,163	$882,913
	Site Security	5.99	Mo.	$6,832	$40,944	$6,142	$47,086
	Night Lighting	5.99	Mo.	$47,044	$281,982	$42,297	$324,279
	Total Beach Replacement				$9,073,900	$1,347,739	$10,421,639
30.	Planning, Engineering and Design (P,E & D)	1	Job	LS	$1,000,000	$150,000	$1,150,000
31.	Construction Management (S & A @ 8.23%)	1	Job	LS	$746,782	$112,017	$858,799
	Total Project First Cost				$10,820,682	$1,609,756	$12,430,438
	(Rounded)				$10,821,000	$1,610,000	$12,430,000

Notes:

Beach fill quantity includes 4 yr. nourishment cycle.

Table 2C - Total First Cost Price Level: Jun 07

Plan C (75' Berm w/ 16' NAVD Dune using 4 Yr. Cycle)

ACCOUNT NUMBER	DESCRIPTION OF ITEM	QUANTITY	UOM	UNIT PRICE	ESTIMATED AMOUNT	CONTINGENCY	TOTAL COST
01.	Lands and Damages	1	Job	LS	$0	$0	$0
17.	Beach Replenishment						
	Mobilization, Demob. And Preparatory Work	1	Job	LS	$467,073	$56,049	$523,122
	Beach Fill						
	Site Work - Cells 1 and 2						
	Excavation/Pumping Sand	975,000	CY	$6.66	$6,498,083	$974,712	$7,472,795
	Survey Crew @ Borrow Area D	6.34	Mo.	$144,980	$919,608	$137,941	$1,057,549
	Survey Crew @ Berm w/ Dune	6.34	Mo.	$87,471	$554,829	$83,224	$638,053
	Grading @ Berm w/ Dune	6.34	Mo.	$128,172	$812,610	$121,892	$934,502
	Site Security	6.34	Mo.	$6,572	$41,686	$6,253	$47,939
	Night Lighting	6.34	Mo.	$47,044	$298,259	$44,739	$342,998
	Total Beach Replacement				$9,592,148	$1,424,810	$11,016,958
30.	Planning, Engineering and Design (P,E & D)	1	Job	LS	$1,000,000	$150,000	$1,150,000
31.	Construction Management (S & A @ 8.23%)	1	Job	LS	$789,434	$118,415	$907,849
	Total Project First Cost				$11,381,582	$1,693,225	$13,074,807
	(Rounded)				$11,382,000	$1,693,000	$13,075,000

Notes:
Beach fill quantity includes 4 yr. nourishment cycle.

Table 2D - Total First Cost Price Level: Jun 07

Plan D (140' Berm w/ 12' NAVD Dune using 4 Yr. Cycle)

ACCOUNT NUMBER	DESCRIPTION OF ITEM	QUANTITY	UOM	UNIT PRICE	ESTIMATED AMOUNT	CONTINGENCY	TOTAL COST
01.	Lands and Damages	1	Job	LS	$0	$0	$0
17.	Beach Replenishment						
	Mobilization, Demob. And Preparatory Work	1	Job	LS	$502,344	$60,281	$562,625
	Beach Fill						
	Site Work - Cells 1 and 2						
	Excavation/Pumping Sand	1,081,400	CY	$6.54	$7,077,655	$1,061,648	$8,139,303
	Survey Crew @ Borrow Area D	7.03	Mo.	$142,344	$1,000,806	$150,121	$1,150,927
	Survey Crew @ Berm w/ Dune	7.03	Mo.	$85,881	$603,829	$90,574	$694,404
	Grading @ Berm w/ Dune	7.03	Mo.	$125,841	$884,662	$132,699	$1,017,362
	Site Security	7.03	Mo.	$6,452	$45,363	$6,805	$52,168
	Night Lighting	7.03	Mo.	$46,189	$324,755	$48,713	$373,468
	Total Beach Replacement				$10,439,415	$1,550,842	$11,990,257
30.	Planning, Engineering and Design (P,E & D)	1	Job	LS	$1,000,000	$150,000	$1,150,000
31.	Construction Management (S & A @ 8.23%)	1	Job	LS	$859,164	$128,875	$988,038
	Total Project First Cost				$12,298,579	$1,829,717	$14,128,295
	(Rounded)				$12,299,000	$1,830,000	$14,128,000

Notes:

Beach fill quantity includes 4 yr. nourishment cycle.

Table 2E - Total First Cost Price Level: Jun 07

Plan E (120' Berm w/ 14' NAVD Dune using 4 Yr. Cycle)

ACCOUNT NUMBER	DESCRIPTION OF ITEM	QUANTITY	UOM	UNIT PRICE	ESTIMATED AMOUNT	CONTINGENCY	TOTAL COST
01.	Lands and Damages	1	Job	LS	$0	$0	$0
17.	Beach Replenishment						
	Mobilization, Demob. And Preparatory Work	1	Job	LS	$1,040,079	$124,809	$1,164,888
	Beach Fill						
	Site Work - Cells 1 and 2						
	Excavation/Pumping Sand	1,124,400	CY	$6.44	$7,246,646	$1,086,997	$8,333,642
	Survey Crew @ Borrow Area D	5.42	Mo.	$188,056	$1,019,959	$152,994	$1,172,953
	Survey Crew @ Berm w/ Dune	5.42	Mo.	$113,438	$614,834	$92,225	$707,059
	Grading @ Berm w/ Dune	5.42	Mo.	$167,122	$905,801	$135,870	$1,041,671
	Site Security	5.42	Mo.	$6,452	$34,976	$5,246	$40,223
	Night Lighting	5.42	Mo.	$46,189	$250,344	$37,552	$287,896
	Total Beach Replacement				$11,112,640	$1,635,694	$12,748,333
30.	Planning, Engineering and Design (P,E & D)	1	Job	LS	$1,000,000	$150,000	$1,150,000
31.	Construction Management (S & A @ 8.23%)	1	Job	LS	$914,570	$137,186	$1,051,756
	Total Project First Cost				$13,027,210	$1,922,879	$14,950,089
	(Rounded)				$13,027,000	$1,923,000	$14,950,000

Notes:

Beach fill quantity includes 4 yr. nourishment cycle.

Table 2F - Total First Cost Price Level: Jun 07

Plan F (100' Berm w/ 16' NAVD Dune using 4 Yr. Cycle)

ACCOUNT NUMBER	DESCRIPTION OF ITEM	QUANTITY	UOM	UNIT PRICE	ESTIMATED AMOUNT	CONTINGENCY	TOTAL COST
01.	Lands and Damages	1	Job	LS	$0	$0	$0
17.	Beach Replenishment						
	Mobilization, Demob. And Preparatory Work	1	Job	LS	$1,083,665	$130,040	$1,213,705
	Beach Fill						
	Site Work - Cells 1 and 2						
	Excavation/Pumping Sand	1,177,400	CY	$6.44	$7,585,988	$1,137,898	$8,723,886
	Survey Crew @ Borrow Area D	5.68	Mo.	$188,056	$1,068,722	$160,308	$1,229,031
	Survey Crew @ Berm w/ Dune	5.68	Mo.	$113,438	$644,328	$96,649	$740,977
	Grading @ Berm w/ Dune	5.68	Mo.	$167,122	$949,253	$142,388	$1,091,641
	Site Security	5.68	Mo.	$6,452	$36,662	$5,499	$42,161
	Night Lighting	5.68	Mo.	$46,189	$262,400	$39,360	$301,760
	Total Beach Replacement				$11,631,018	$1,712,143	$13,343,160
30.	Planning, Engineering and Design (P,E & D)	1	Job	LS	$1,000,000	$150,000	$1,150,000
31.	Construction Management (S & A @ 8.23%)	1	Job	LS	$957,233	$143,585	$1,100,818
	Total Project First Cost				$13,588,250	$2,005,728	$15,593,978
	(Rounded)				$13,588,000	$2,006,000	$15,594,000

Notes:

Beach fill quantity includes 4 yr. nourishment cycle.

Table 2G - Total First Cost Price Level: Jun 07

Plan G (165' Berm w/ 12' NAVD Dune using 4 Yr. Cycle)

ACCOUNT NUMBER	DESCRIPTION OF ITEM	QUANTITY	UOM	UNIT PRICE	ESTIMATED AMOUNT	CONTINGENCY	TOTAL COST
01.	Lands and Damages	1	Job	LS	$0	$0	$0
17.	Beach Replenishment						
	Mobilization, Demob. And Preparatory Work	1	Job	LS	$871,659	$104,599	$976,258
	Beach Fill						
	Site Work - Cells 1 and 2						
	Excavation/Pumping Sand	1,306,400	CY	$6.44	$8,419,617	$1,262,943	$9,682,560
	Survey Crew @ Borrow Area D	6.30	Mo.	$188,056	$1,184,866	$177,730	$1,362,595
	Survey Crew @ Berm w/ Dune	6.30	Mo.	$113,438	$714,659	$107,199	$821,858
	Grading @ Berm w/ Dune	6.30	Mo.	$167,122	$1,052,869	$157,930	$1,210,799
	Site Security	6.30	Mo.	$6,452	$40,666	$6,100	$46,766
	Night Lighting	6.30	Mo.	$46,189	$290,991	$43,649	$334,639
	Total Beach Replacement				$12,575,326	$1,860,149	$14,435,476
30.	Planning, Engineering and Design (P,E & D)	1	Job	LS	$1,304,348	$195,652	$1,500,000
31.	Construction Management (S & A @ 7.97%)	1	Job	LS	$1,002,254	$150,338	$1,152,592
	Total Project First Cost				$14,881,928	$2,206,139	$17,088,067
	(Rounded)				$14,882,000	$2,206,000	$17,088,000

Notes:

Beach fill quantity includes 4 yr. nourishment cycle.

Table 2H - Total First Cost Price Level: Jun 07

Plan H (145' Berm w/ 14' NAVD Dune using 4 Yr. Cycle)

ACCOUNT NUMBER	DESCRIPTION OF ITEM	QUANTITY	UOM	UNIT PRICE	ESTIMATED AMOUNT	CONTINGENCY	TOTAL COST
01.	Lands and Damages	1	Job	LS	$0	$0	$0
17.	Beach Replenishment						
	Mobilization, Demob. And Preparatory Work	1	Job	LS	$1,225,165	$147,020	$1,372,185
	Beach Fill						
	Site Work - Cells 1 and 2						
	Excavation/Pumping Sand	1,349,400	CY	$6.44	$8,694,872	$1,304,231	$9,999,103
	Survey Crew @ Borrow Area D	6.51	Mo.	$188,056	$1,224,245	$183,637	$1,407,881
	Survey Crew @ Berm w/ Dune	6.51	Mo.	$113,438	$738,481	$110,772	$849,254
	Grading @ Berm w/ Dune	6.51	Mo.	$167,122	$1,087,964	$163,195	$1,251,159
	Site Security	6.51	Mo.	$6,452	$42,012	$6,302	$48,314
	Night Lighting	6.51	Mo.	$46,189	$300,690	$45,104	$345,794
	Total Beach Replacement				$13,313,430	$1,960,260	$15,273,690
30.	Planning, Engineering and Design (P,E & D)	1	Job	LS	$1,000,000	$150,000	$1,150,000
31.	Construction Management (S & A @ 7.97%)	1	Job	LS	$1,061,080	$159,162	$1,220,242
	Total Project First Cost				$15,374,511	$2,269,422	$17,643,932
	(Rounded)				$15,375,000	$2,269,000	$17,644,000

Notes:

Beach fill quantity includes 4 yr. nourishment cycle.

Table 2I - Total First Cost
Plan I (125' Berm w/ 16' NAVD Dune using 4 Yr. Cycle)

Price Level: Jun 07

ACCOUNT NUMBER	DESCRIPTION OF ITEM	QUANTITY	UOM	UNIT PRICE	ESTIMATED AMOUNT	CONTINGENCY	TOTAL COST
01.	Lands and Damages	1	Job	LS	$0	$0	$0
17.	Beach Replenishment						
	Mobilization, Demob. And Preparatory Work	1	Job	LS	$1,268,783	$152,254	$1,421,037
	Beach Fill						
	Site Work - Cells 1 and 2						
	Excavation/Pumping Sand	1,402,400	CY	$6.44	$9,038,328	$1,355,749	$10,394,077
	Survey Crew @ Borrow Area D	6.76	Mo.	$188,056	$1,272,180	$190,827	$1,463,007
	Survey Crew @ Berm w/ Dune	6.76	Mo.	$113,438	$766,841	$115,026	$881,867
	Grading @ Berm w/ Dune	6.76	Mo.	$167,122	$1,129,745	$169,462	$1,299,206
	Site Security	6.76	Mo.	$6,452	$43,616	$6,542	$50,158
	Night Lighting	6.76	Mo.	$46,189	$312,353	$46,853	$359,206
	Total Beach Replacement				$13,831,845	$2,036,713	$15,868,558
30.	Planning, Engineering and Design (P,E & D)	1	Job	LS	$1,000,000	$150,000	$1,150,000
31.	Construction Management (S & A @ 7.97%)	1	Job	LS	$1,102,398	$165,360	$1,267,758
	Total Project First Cost				$15,934,243	$2,352,073	$18,286,316
	(Rounded)				$15,934,000	$2,352,000	$18,286,000

Notes:
Beach fill quantity includes 4 yr. nourishment cycle.

Table 2J - Total First Cost Price Level: Jun 07

Plan J (80' Berm w/ 18' NAVD Dune using 4 Yr. Cycle)

ACCOUNT NUMBER	DESCRIPTION OF ITEM	QUANTITY	UOM	UNIT PRICE	ESTIMATED AMOUNT	CONTINGENCY	TOTAL COST
01.	Lands and Damages	1	Job	LS	$0	$0	$0
17.	Beach Replenishment						
	Mobilization, Demob. And Preparatory Work	1	Job	LS	$1,136,316	$136,358	$1,272,674
	Beach Fill						
	Site Work - Cells 1 and 2						
	Excavation/Pumping Sand	1,241,400	CY	$6.44	$7,998,278	$1,199,742	$9,198,020
	Survey Crew @ Borrow Area D	5.99	Mo.	$188,056	$1,126,455	$168,968	$1,295,424
	Survey Crew @ Berm w/ Dune	5.99	Mo.	$113,438	$679,494	$101,924	$781,418
	Grading @ Berm w/ Dune	5.99	Mo.	$167,122	$1,001,061	$150,159	$1,151,220
	Site Security	5.99	Mo.	$6,452	$38,659	$5,799	$44,458
	Night Lighting	5.99	Mo.	$46,189	$276,741	$41,511	$318,253
	Total Beach Replacement				$12,257,004	$1,804,461	$14,061,466
30.	Planning, Engineering and Design (P,E & D)	1	Job	LS	$1,000,000	$150,000	$1,150,000
31.	Construction Management (S & A @ 8.23%)	1	Job	LS	$1,008,751	$151,313	$1,160,064
	Total Project First Cost				$14,265,756	$2,105,774	$16,371,530
	(Rounded)				$14,266,000	$2,106,000	$16,372,000

Notes:

Beach fill quantity includes 4 yr. nourishment cycle.

Table 2K - Total First Cost Price Level: Jun 07

Plan K (105' Berm w/ 18' NAVD Dune using 4 Yr. Cycle)

ACCOUNT NUMBER	DESCRIPTION OF ITEM	QUANTITY	UOM	UNIT PRICE	ESTIMATED AMOUNT	CONTINGENCY	TOTAL COST
01.	Lands and Damages	1	Job	LS	$0	$0	$0
17.	Beach Replenishment						
	Mobilization, Demob. And Preparatory Work	1	Job	LS	$1,321,434	$158,572	$1,480,006
	Beach Fill						
	Site Work - Cells 1 and 2						
	Excavation/Pumping Sand	1,466,400	CY	$6.48	$9,508,284	$1,426,243	$10,934,527
	Survey Crew @ Borrow Area D	5.69	Mo.	$235,029	$1,337,315	$200,597	$1,537,912
	Survey Crew @ Berm w/ Dune	5.69	Mo.	$141,773	$806,688	$121,003	$927,692
	Grading @ Berm w/ Dune	5.69	Mo.	$208,887	$1,188,567	$178,285	$1,366,852
	Site Security	5.69	Mo.	$6,452	$36,737	$5,511	$42,248
	Night Lighting	5.69	Mo.	$68,679	$390,784	$58,618	$449,401
	Total Beach Replacement				$14,589,809	$2,148,828	$16,738,638
30.	Planning, Engineering and Design (P,E & D)	1	Job	LS	$1,000,000	$150,000	$1,150,000
31.	Construction Management (S & A @ 7.97%)	1	Job	LS	$1,162,808	$174,421	$1,337,229
	Total Project First Cost				$16,752,617	$2,473,250	$19,225,867
	(Rounded)				$16,753,000	$2,473,000	$19,226,000

Notes:

Beach fill quantity includes 4 yr. nourishment cycle.

Table 2L - Total First Cost

Price Level: Jun 07

Plan L (85' Berm w/ 20' NAVD Dune using 4 Yr. Cycle)

ACCOUNT NUMBER	DESCRIPTION OF ITEM	QUANTITY	UOM	UNIT PRICE	ESTIMATED AMOUNT	CONTINGENCY	TOTAL COST
01.	Lands and Damages	1	Job	LS	$0	$0	$0
17.	Beach Replenishment						
	Mobilization, Demob. And Preparatory Work	1	Job	LS	$1,380,667	$165,680	$1,546,347
	Beach Fill						
	Site Work - Cells 1 and 2						
	Excavation/Pumping Sand	1,538,400	CY	$6.48	$9,974,755	$1,496,213	$11,470,968
	Survey Crew @ Borrow Area D	5.97	Mo.	$235,029	$1,403,123	$210,468	$1,613,592
	Survey Crew @ Berm w/ Dune	5.97	Mo.	$141,773	$846,385	$126,958	$973,343
	Grading @ Berm w/ Dune	5.97	Mo.	$208,887	$1,247,055	$187,058	$1,434,114
	Site Security	5.97	Mo.	$6,452	$38,530	$5,780	$44,310
	Night Lighting	5.97	Mo.	$68,679	$410,014	$61,502	$471,516
	Total Beach Replacement				$15,300,529	$2,253,659	$17,554,188
30.	Planning, Engineering and Design (P,E & D)	1	Job	LS	$1,000,000	$150,000	$1,150,000
31.	Construction Management (S & A @ 7.97%)	1	Job	LS	$1,219,452	$182,918	$1,402,370
	Total Project First Cost				$17,519,981	$2,586,577	$20,106,558
	(Rounded)				$17,520,000	$2,587,000	$20,107,000

Notes:

Beach fill quantity includes 4 yr. nourishment cycle.

Table 2M - Total First Cost Price Level: Jun 07

Plan M (110' Berm w/ 20' NAVD Dune using 4 Yr. Cycle)

ACCOUNT NUMBER	DESCRIPTION OF ITEM	QUANTITY	UOM	UNIT PRICE	ESTIMATED AMOUNT	CONTINGENCY	TOTAL COST
01.	Lands and Damages	1	Job	LS	$0	$0	$0
17.	Beach Replenishment						
	Mobilization, Demob. And Preparatory Work	1	Job	LS	$1,431,318	$171,758	$1,603,076
	Beach Fill						
	Site Work - Cells 1 and 2						
	Excavation/Pumping Sand	1,906,600	CY	$6.54	$12,477,553	$1,871,633	$14,349,186
	Survey Crew @ Borrow Area D	6.20	Mo.	$284,688	$1,765,066	$264,760	$2,029,825
	Survey Crew @ Berm w/ Dune	6.20	Mo.	$171,762	$1,064,924	$159,739	$1,224,663
	Grading @ Berm w/ Dune	6.20	Mo.	$251,683	$1,560,435	$234,065	$1,794,500
	Site Security	6.20	Mo.	$6,452	$40,022	$6,003	$46,025
	Night Lighting	6.20	Mo.	$92,378	$572,836	$85,925	$658,761
	Total Beach Replacement				$18,912,153	$2,793,883	$21,706,037
30.	Planning, Engineering and Design (P,E & D)	1	Job	LS	$1,000,000	$150,000	$1,150,000
31.	Construction Management (S & A @ 7.76%)	1	Job	LS	$1,467,583	$220,137	$1,687,721
	Total Project First Cost				$21,379,736	$3,164,021	$24,543,757
	(Rounded)				$21,380,000	$3,164,000	$24,544,000

Notes:
Beach fill quantity includes 4 yr. nourishment cycle.

Table 2N - Total First Cost Price Level: Jun 07

Plan N (160' Berm w/ 20' NAVD Dune using 4 Yr. Cycle)

ACCOUNT NUMBER	DESCRIPTION OF ITEM	QUANTITY	UOM	UNIT PRICE	ESTIMATED AMOUNT	CONTINGENCY	TOTAL COST
01.	Lands and Damages	1	Job	LS	$0	$0	$0
17.	Beach Replenishment						
	Mobilization, Demob. And Preparatory Work	1	Job	LS	$1,900,174	$228,021	$2,128,195
	Beach Fill						
	Site Work - Cells 1 and 2						
	Excavation/Pumping Sand	2,674,600	CY	$6.51	$17,411,646	$2,611,747	$20,023,393
	Survey Crew @ Borrow Area D	6.50	Mo.	$377,373	$2,452,925	$367,939	$2,820,863
	Survey Crew @ Berm w/ Dune	6.50	Mo.	$227,654	$1,479,751	$221,963	$1,701,714
	Grading @ Berm w/ Dune	6.50	Mo.	$334,728	$2,175,732	$326,360	$2,502,092
	Site Security	6.50	Mo.	$6,452	$41,938	$6,291	$48,229
	Night Lighting	6.50	Mo.	$114,868	$746,734	$112,010	$858,744
	Total Beach Replacement				$26,208,899	$3,874,330	$30,083,229
30.	Planning, Engineering and Design (P,E & D)	1	Job	LS	$1,000,000	$150,000	$1,150,000
31.	Construction Management (S & A @ 7.34%)	1	Job	LS	$1,923,733	$288,560	$2,212,293
	Total Project First Cost				$29,132,633	$4,312,890	$33,445,522
	(Rounded)				$29,133,000	$4,313,000	$33,446,000

Notes:
Beach fill quantity includes 4 yr. nourishment cycle.

Table 3 - Periodic Nourishment Cost (Years 4 and 8)

Price Level: Mar 14
Construction duration: 4-months

ACCOUNT NUMBER	DESCRIPTION OF ITEM	QUANTITY	UOM	UNIT PRICE	ESTIMATED AMOUNT	CONTINGENCY	TOTAL COST
17.	Beach Replenishment					**@ 25.1%**	
17.01	Mobilization, Demob. And Preparatory Work	1	Job	LS	$895,921	$224,876	$1,120,797
17.70	Beach Fill	1	Job	LS	$2,904,614	$729,058	$3,633,672
17.99	Associated General Items	1	Job	LS	$257,874	$64,726	$322,600
	Total Beach Replenishment				$4,058,408	$1,018,661	$5,077,069
						@ 15%	
30.	Planning, Engineering and Design (P,E & D)	1	Job	LS	$221,632	$33,245	$254,877
31.	Construction Management (S & A)	1	Job	LS	$539,552	$80,933	$620,485
	Total Project First Cost				$4,819,593	$1,132,838	$5,952,431
	(Rounded)				$4,820,000	$1,133,000	$5,952,000

Table 4 - Periodic Nourishment Cost (Years 12, 16, 20, 28, 32, 36, 40, 44 and 48)

Price Level: Mar 14
Construction duration: 4-months

ACCOUNT NUMBER	DESCRIPTION OF ITEM	QUANTITY	UOM	UNIT PRICE	ESTIMATED AMOUNT	CONTINGENCY	TOTAL COST
17.	Beach Replenishment					**@ 31%**	
17.01	Mobilization, Demob. And Preparatory Work	1	Job	LS	$895,921	$277,735	$1,173,656
17.70	Beach Fill	1	Job	LS	$2,904,614	$900,430	$3,805,045
17.99	Associated General Items	1	Job	LS	$257,874	$79,941	$337,814
	Total Beach Replenishment				$4,058,408	$1,258,107	$5,316,515
						@ 15%	
30.	Planning, Engineering and Design (P,E & D)	1	Job	LS	$221,632	$33,245	$254,877
31.	Construction Management (S & A)	1	Job	LS	$539,552	$80,933	$620,485
	Total Project First Cost				$4,819,593	$1,372,284	$6,191,877
	(Rounded)				$4,820,000	$1,372,000	$6,192,000

Table 5 - Major Replacement Cost (Yr. 24)

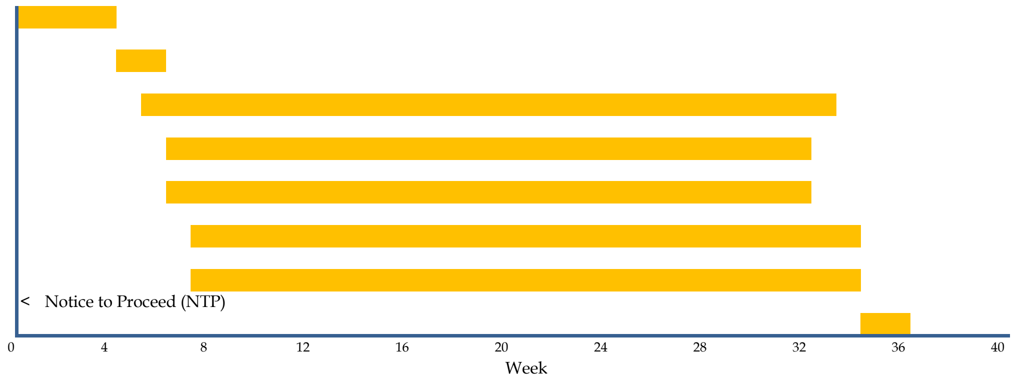
Price Level: Mar 14
Construction duration: 5-months

ACCOUNT NUMBER	DESCRIPTION OF ITEM	QUANTITY	UOM	UNIT PRICE	ESTIMATED AMOUNT	CONTINGENCY	TOTAL COST
17.	Beach Replenishment					**@ 31%**	
17.01	Mobilization, Demob. And Preparatory Work	1	Job	LS	$935,261	$289,931	$1,225,192
17.70	Beach Fill	1	Job	LS	$3,961,931	$1,228,199	$5,190,130
17.99	Associated General Items	1	Job	LS	$332,175	$102,974	$435,149
	Total Beach Replenishment				$5,229,368	$1,621,104	$6,850,472
						@ 15%	
30.	Planning, Engineering and Design (P,E & D)	1	Job	LS	$264,697	$39,705	$304,402
31.	Construction Management (S & A)	1	Job	LS	$665,719	$99,858	$765,577
	Total Project First Cost				$6,159,784	$1,760,666	$7,920,450
	(Rounded)				$6,160,000	$1,761,000	$7,920,000

Notes:
Beachfill quantity includes 4 yr. nourishment cycle.

Table 6 - Herford to Cape May Feasibility Study Beach Fill Initial Construction Schedule

#	TASK	WEEKS
1	Work Plans & Submittals	4
2	Mobilization	2
3	Surveys	28
4	Beach Fill	26
5	Structure Monitoring	26
6	Dune Crossover Work	27
7	Dune Planting and Seeding	27
8	Demobilization	2



Notes:

Herford to Cape May Feasibility Study initial construction duration = 36 weeks, use 9 months.

Activity ID	Type of Funds (Override)	Activity Name	Pr...
109882 Hereford / Cape May Inlet, NJ			
109882.CW Standard Civil Works Project			
109882.CW.00500 Project Management Plan (PMP)			
PMP0200		Approve PMP	P...
PMP0011		Coastal Project Mangement/NF	
PMP0020		Start PMP/Project-P1	
PMP0010	96 3121 132	Travel	
PMP0009	96 8862	Vehicle Travel	
109882.CW.21000 Recon/Sec 905(b) Studies			
109882.CW.21000.21T00 Recon Prog & Proj Mgmt			
A1000		Pre-P2 Work Items/ prior costs	
109882.CW.21000.21P00 Engineering & Design /Cost Estimates			
REC1210		Feasibility Cost Estimate	
109882.CW.21000.21S00 Recon Report/Sec 905(b)			
REC1490		NJDEP Ltr of Intent - Feas	
REC1620		Recon Report Approval P-4	
REC1460		Recon Study P-3	
109882.CW.21000.21M00 All Other Studies			
109882.CW.21000.21V00 FCSA			
REC1720		Signed/Executed FCSA P-5	R...
109882.CW.21000.21W00 Damages Assessed A/E			
109882.CW.22000 Feasibility Studies			
109882.CW.22000.22T00 Feas Prog & Projt Mgmt			
FEA1767	96 8862 132	(2LFC3C) Coastal Project Mangement/NF	
FEA1790	96 3121 132	(C74K85) Public Involvement	
FEA1769	96 8862	(HOJGD8) Project Management & Admin/no...	
FEA1768	96 3121 132	(JCCO5J) Coastal Project Management/ Fed	
FEA1751	96 3121 132	Admin	
FEA1760	96 3121 132	Fiscal Year 2006 Federal Feasibility Tasks	
FEA1750	96 8862 132	Fiscal Year 2006 Non-Federal Feasibility Tasks	
109882.CW.22000.22P00 Eng & Design/Cost Est			
FEA1800	96 8862	(B9JGD8) Hydrology & Hydraulics w/project a...	
FEA1799	96 3121 132	(D40JGD) H&H W/O project analysis/Fed	
FEA1791	96 3121 132	(DCB963) Engineering Management	
FEA1840	96 3121 132	(L42LFC) Eng & Design Structure Analysis/Fed	FE...
FEA1820	96 3121 132	(LFDJGD) Geotech Analysis/ Fed	FE...
A1020	96 8862	A.C. Survey Office-AE SVCS CONTRACT	
FEA1792	96 8862	A.C. Survey Office-NF LABOR	
A1030	96 8862	A.C. Survey Office-NF-AE SERVICES CONT...	
FEA1793	96 3121 132	AC Survey office/FED	
A1040	96 3121 132	AC Survey office/FED AE SERVICES CONT...	
A1010		AC Survey office/FED-AE SVCS CONTRACT	
A1050	96 3121 132	Aerial photo contract	
A1060	96 3121 132	beach vibracore contract	
FEA1821		borrow area/ suitability analysis/geo history	FE...
FEA1841	96 3121 132	Cost Engineering	FE...
FEA1797	96 3121 132	Flood Plains	
A1070	96 8862	Hereford- A&E services labor (109882)	
FEA1845		infrastructure replacement costs	FE...
FEA1801		without project hydraulic analysis	FE...
109882.CW.22000.22C00 Socio/Economics			
FEA1941	96 8862	(36B963) Economics/ NF	
FEA1940	96 3121 132	(GD8KB3) Economic Analysis/Fed	
FEA1942		Average Annual Damages/Average Annual Be...	FE...
109882.CW.22000.22H00 Real Estate			
FEA2000		RE Supplement/Plan	FE...
FEA2040		real estate/public access plans	FE...
FEA2020		recieve preliminary feasibility plans	FE...
109882.CW.22000.22E00 Environmental			
FEA2130	96 8862	(B85985) Environmental Conditions/ NF	
FEA2131	96 3121 132	(CCJGD7) Env. HTRW & OEW Analysis/Fed	FE...
109882.CW.22000.22F00 F&W Coordination			
FEA2260		F&W COORDINATION	
FEA2259		F&W Coordination - Milestone	FE...
109882.CW.22000.22L00 HTRW Studies			
FEA2270		HTRW Prelim Assmt	FE...
109882.CW.22000.22D00 Cultural Resources			
FEA2308	96 3121 132	(60GD85) Cultural Resources/ NF	
FEA2309	96 3121 132	(LFC72L) Cultural -106 Coordination /Fed	
FEA2323		Cultural Mitigation Plan	FE...
FEA2310		Mapping/Surveys/GIS	
109882.CW.22000.22A00 Public Involvement			
FEA2390		Public Comments/Involvement	
109882.CW.22000.22R00 Plan Formulation			
FEA2420		Feasibility Scoping Meeting - P6	FE...
FEA2415		Plan Formulation	FE...
FEA2472		Project Impacts, Project Cost Estimates, OM...	FE...
FEA2470		Selected Plan	FE...
109882.CW.22000.22S00 Feasibility Report			
FEA2680		ASA(CW) Trans to Cong	FE...
FEA2480		concurrence/comments on recommended plan	FE...
FEA2474		Draft Feas Rpt/NEPA	FE...
FEA2510		Feas Public Review Period	FE...
FEA2475		Feas Review Conference (FRC) or Alternative ...	FE...
FEA2473		Feasibility Formulation Meeting - P7	FE...
FEA2580		Final Rpt Doc P-9	FE...
FEA2560		HQ Policy Compl Review	FE...
FEA2590		Issue Feas Div Commander's Notice P-10	FE...
FEA2670		OMB Letter to ASA(CW)	FE...
FEA2591		Prepare Chiefs Report	FE...
FEA2581		Prepare Division Public Notice	FE...
FEA2665		Prepare OMB letter	FE...
FEA2685		Prepare ROD	FE...
FEA2500		Program Guidance Memorandum (PGM)	FE...
FEA2690		ROD	FE...
FEA2660		Submit Feas Chief 's Report	FE...
109882.CW.22000.22M00 All Other Feas. Act			
109882.CW.22000.22X00 Damages Assess A/E			
109882.CW.22000.22Q00 Management Docs			

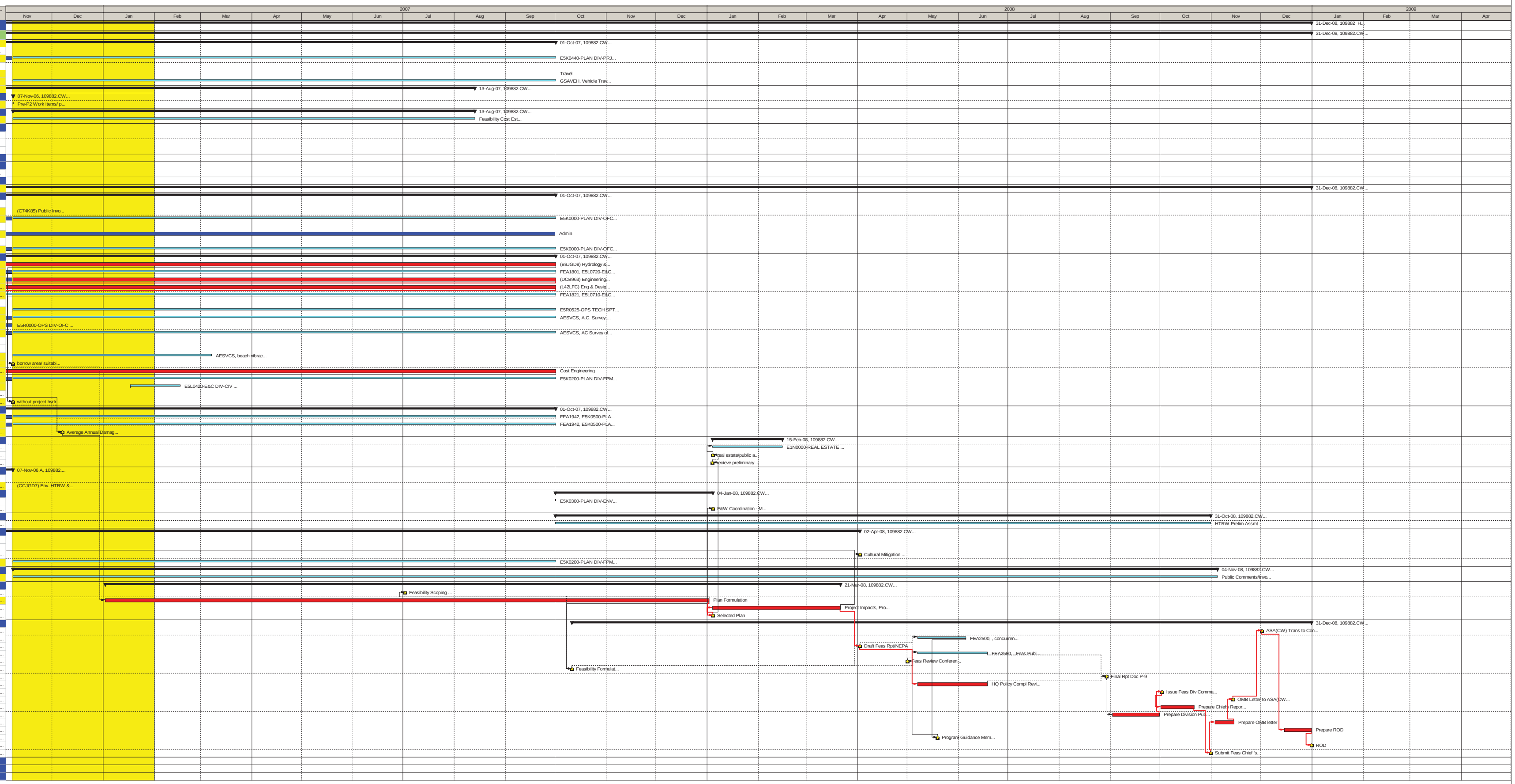


Table 7 - Project Schedule

Actual Work | Remaining Work | Critical Remaining Work | Milestone | Summary

HERF2CAPEMAY_FEAS3v4-2

Hereford Inlet to Cape May Inlet, NJ
New Jersey Shore Protection Study
Draft Final Feasibility Report

Estimated by	Cost Engineering Section
Designed by	Peter Gori, EC-EG; Alyssa Dunlap, EC-EC
Prepared by	William Welk
Preparation Date	5/12/2014
Effective Date of Pricing	3/1/2014
Estimated Construction Time	270 Days

Description	Page

Description	Page

Designed by		Design Document	Draft Final Feasibility Report
Peter Gori, EC-EG; Alyssa Dunlap, EC-EC		Document Date	5/12/2014
Estimated by		District	Philadelphia District
Cost Engineering Section		Contact	William Welk
Prepared by		Budget Year	2014
William Welk		UOM System	Original

Direct Costs

LaborCost
EQCost
MatlCost
SubBidCost
Lump Sum

Timeline/Currency

Preparation Date	5/12/2014
Escalation Date	3/1/2014
Eff. Pricing Date	3/1/2014
Estimated Duration	270 Day(s)
Currency	US dollars
Exchange Rate	1.000000

Costbook CB12EB-b: MII English Cost Book 2012-b

Labor Region 1: Labor Region 1 -2012

Labor Rates

LaborCost1
LaborCost2
LaborCost3
LaborCost4

Equipment EP11R01: MII Equipment 2011 Region 01

01 NORTHEAST

Sales Tax	7.00
Working Hours per Year	1,360
Labor Adjustment Factor	1.12
Cost of Money	2.50
Cost of Money Discount	25.00
Tire Recap Cost Factor	1.50
Tire Recap Wear Factor	1.80
Tire Repair Factor	0.15
Equipment Cost Factor	1.00
Standby Depreciation Factor	0.50

Fuel

Electricity	0.190
Gas	3.600
Diesel Off-Road	3.860
Diesel On-Road	4.350

Shipping Rates

Over 0 CWT	18.08
Over 240 CWT	16.61
Over 300 CWT	14.46
Over 400 CWT	12.44
Over 500 CWT	6.96
Over 700 CWT	6.96
Over 800 CWT	10.55

Date	Author	Note
5/19/2008	Bill Welk	1. Prepared by the U.S. Army Corps of Engineers, Philadelphia District, Wanamaker Building, 100 Penn Square East, Philadelphia, PA 19107-3391.
7/10/2012		2. SUMMARY OF WORK: Work includes, but is not limited to beach fill in North Wildwood, Wildwood, and Wildwood Crest, NJ. The major work items for the selected plan: Plan C - 75' wide berm with +16' NAVD high dune including 4-year nourishment cycle using mobile hydraulic sand backpassing, are as follows:
7/10/2012		Approximately 1,007,250 C.Y. of beach fill from onshore borrow Area WW/WC will be placed in North Wildwood (Cells 1 and 2). The average pumping distance for the initial construction beach fill uses an average pipeline length of 15,600 L.F. A 150 hp electric submersible agitator dredge pump would be suspended from a crawler crane with a minimum 100' boom. A 300 kW generator would be mounted on the back of the crane to power the dradge pump. Sand up to 50% solids by weight, to be transferred through 10-inch gum tube-lined hose to 12-inch HDPE pipe on the beach. Diesel engine, skid mounted booster pumps would be placed every 5,000 feet to transfer the sand slurry to the outlet location where the beach fill would take place. Horsepower for each booster is 400 hp. Instrumentation including magnetic flow meters would provide flow rates and production numbers.
7/10/2012		In Wildwood and Wildwood Crest, the project will consist of placing 520,000 CY of beach fill to construct a +16' NAVD high dune and raising the existing berm elevation to 6.5' NAVD. Side slopes for the dunes will be 1 on 5. In Wildwood and Wildwood crest, the average pipeline length is 1,000LF. Aso included is the placement of 64 acres of dune grass, 28,000 LF of sand fence, extending 44 existing pedestrian crossovers, 7 new pedestrian crossovers, extending 7 exinting handicap crossovers, 6 new handicap crossovers, extending 8 existing vehicle crossovers and 5 new vehicle crossovers.
7/10/2012		3. Construction schedule:
7/10/2012		- Report completion (Program Year) - September 2014
5/14/2013		- Estimated start of construction - October 2017
5/14/2013		- Mid-point of construction - February 2018 based on 9-month construction duration.
5/14/2013		4. Used Cape May County, NJ labor rates, General Decision Number NJ140050, Mod. No. 0 dated 01/03/14.
5/14/2013		5. Real estate costs (project feature 01) provided thriugh PL-PC and furnished by CENAB-RE.
5/14/2013		6. P,E&D costs (project feature 30) and S&A costs (project feature 31) provided by PL-PC.
5/14/2013		7. Price level: March 2014
5/14/2013		8. Contingencies are based on Crystal Ball software for preparing risk analysis and are:
5/14/2013		- Initial construction work - 25.1%; Nourishment (Years 4 and 8) - 25.1%; Nourishment (all other years) and Major Replacement (year 24) - 31%
5/14/2013		- Real estate costs - 24.9%
5/14/2013		- S&A and P,E&D - 15%
5/14/2013		9. Critical assumptions:
5/14/2013		- Beach fill work will be permitted only from September to April due to the tourist season.
5/14/2013		- A 350 C.Y. per hour production rate was used for cost estimating purposes and is based on the Sand Bypass Plant, Indian River Inlet, Delaware Coast Protection job constructed by NAP in 1989 and operated by Delaware Department of Natural Resources and Environmental Control (DENREC).

Date	Author	Note
5/14/2013		- There will be no severe weather events during construction.
2/12/2014		- Beach fill work will take place Monday to Friday, 24-hours per day.
3/12/2014		- Job will be open bid.
3/12/2014		10. Used R.S. Means, MII Cost Book, price quotes and historic data for material costs as noted.

Direct Cost Markups	Category	Method
Productivity	Productivity	Productivity
Overtime	Overtime	Overtime

	Days/Week	Hours/Shift	Shifts/Day	1st Shift	2nd Shift	3rd Shift
Standard	5.00	8.00	2.00	8.00	7.50	0.00
Actual	5.00	8.00	2.00	12.00	12.00	0.00

Day	OT Factor	Working	OT Percent	FCCM Percent
Monday	1.50	Yes	19.79	(66.67)
Tuesday	1.50	Yes		
Wednesday	1.50	Yes		
Thursday	1.50	Yes		
Friday	1.50	Yes		
Saturday	1.50	No		
Sunday	2.00	No		

Sales Tax	TaxAdj	Running % on Selected Costs
LaborCost		

Contractor Markups	Category	Method
JOOH (Small Tools)	JOOH	% of Labor
JOOH	JOOH	JOOH (Calculated)
JOOH %	JOOH	Running %
HOOH	HOOH	Running %
Profit %	Profit	Running %
Profit WG	Profit	Profit Weighted Guidelines

Guideline	Value	Weight	Percentage
Risk	0.080	20	1.60
Difficulty	0.080	15	1.20
Size	0.030	15	0.45
Period	0.070	15	1.05
Invest (Contractor's)	0.070	5	0.35
Assist (Assistance by)	0.090	5	0.45
SubContracting	0.090	25	2.25
Total		100	7.35

Bond	Bond	Bond Table

Class A, Tiered, 24 months, 1.00% Surcharge

Contract Price	Bond Rate
500,000	11.88
2,000,000	7.39
2,500,000	5.81
2,500,000	5.41

100,000,000,000	*4.88*	
Bond %	Bond	Running %

Owner Markups	**Category**	**Method**
Contingency	Contingency	Running %
Escalation	Escalation	Running %
SIOH	SIOH	Running %

Description	Quantity	UOM	ContractCost	Escalation	Contingency	SIOH	ProjectCost
Project Cost Summary			**17,497,990**	**0**	**0**	**0**	**17,497,990**
HEREFORD TO CAPE MAY INLETS FEASIBILITY STUDY - SELECTED PLAN COST ESTIMATE	**1.0**	**LS**	**17,497,990**	**0**	**0**	**0**	**17,497,990**
01. LANDS AND DAMAGES	**1.0**	**LS**	**1,018,972**	**0**	**0**	**0**	**1,018,972**
01. Lands and Damages	**1.0**	**LS**	**1,018,972**	**0**	**0**	**0**	**1,018,972**
17.IC INITAL CONSTRUCTION BEACH REPLENISHMENT - Move Beach Fill w/ Mobile Hydraulic Backpass System	**1.0**	**LS**	**13,673,876**	**0**	**0**	**0**	**13,673,876**
01. Mobilization, Demobilization and Preparatory Work	**1.0**	**LS**	**1,026,656**	**0**	**0**	**0**	**1,026,656**
70. Beach Fill	**1.0**	**LS**	**9,883,656**	**0**	**0**	**0**	**9,883,656**
99. Associated General Items	**1.0**	**LS**	**2,763,564**	**0**	**0**	**0**	**2,763,564**
30. PLANNING ENGINEERING & DESIGN	**1.0**	**LS**	**1,617,299**	**0**	**0**	**0**	**1,617,299**
01. Planning Engineering & Design	**1.0**	**LS**	**1,617,299**	**0**	**0**	**0**	**1,617,299**
31. CONSTRUCTION MANAGEMENT	**1.0**	**LS**	**1,187,843**	**0**	**0**	**0**	**1,187,843**
01. Construction Management	**1.0**	**LS**	**1,187,843**	**0**	**0**	**0**	**1,187,843**

Description	Quantity	UOM	DirectCost	SubCMU	CostToPrime	PrimeCMU	ContractCost
Project Indirect Summary			14,816,763	115,234	14,931,997	2,565,993	17,497,990
HEREFORD TO CAPE MAY INLETS FEASIBILITY STUDY - SELECTED PLAN COST ESTIMATE	1.0	LS	14,816,763	115,234	14,931,997	2,565,993	17,497,990
01. LANDS AND DAMAGES	1.0	LS	1,018,972	0	1,018,972	0	1,018,972
01. Lands and Damages	1.0	LS	1,018,972	0	1,018,972	0	1,018,972
17.IC INITAL CONSTRUCTION BEACH REPLENISHMENT - Move Beach Fill w/ Mobile Hydraulic Backpass System	1.0	LS	10,992,649	115,234	11,107,883	2,565,993	13,673,876
01. Mobilization, Demobilization and Preparatory Work	1.0	LS	780,423	20,739	801,162	225,494	1,026,656
70. Beach Fill	1.0	LS	7,641,891	70,926	7,712,817	2,170,839	9,883,656
99. Associated General Items	1.0	LS	2,570,335	23,569	2,593,904	169,661	2,763,564
30. PLANNING ENGINEERING & DESIGN	1.0	LS	1,617,299	0	1,617,299	0	1,617,299
01. Planning Engineering & Design	1.0	LS	1,617,299	0	1,617,299	0	1,617,299
31. CONSTRUCTION MANAGEMENT	1.0	LS	1,187,843	0	1,187,843	0	1,187,843
01. Construction Management	1.0	LS	1,187,843	0	1,187,843	0	1,187,843

Description	DirectCost	JOOH	HOOH	Profit	Bond	Escalation	CostToPrime	ContractorOwnCost
Contractor Indirect Summary								
AA Prime Contractor (Initial Construction) - Land Based Equipment	**8,602,507**	**811,672**	**753,134**	**747,298**	**109,146**	**0**	**8,602,507**	**11,538,019**
SU Survey Sub	**245,437**	**24,544**	**21,598**	**24,784**	**0**	**0**	**316,363**	**316,363**
EL Electrical Sub	**70,320**	**7,032**	**6,188**	**7,519**	**0**	**0**	**91,059**	**91,059**
SE Security Sub	**83,271**	**8,327**	**7,328**	**7,914**	**0**	**0**	**106,840**	**106,840**
AB Prime Contractor - No markups.	**5,815,228**	**0**	**0**	**0**	**0**	**0**	**5,815,228**	**5,815,228**

Description	ManHours	LaborCost	EQHours	CrewHours	CrewCost
Crews Backup					
	1.00	*69.98*	*0.00*		*69.98*
GOV COEMEQMD1 1 eqoprmed	10,528.0	736,749	0.0	10,528.0	736,749
MIL B-EQOPRMED Equip. Operators, Medium	*1.0*	*70*			
	1.00	*80.90*	*0.00*		*80.90*
RSM ELEC ELEC	263.2	21,290	0.0	263.2	21,290
MIL B-ELECTRN Electricians	*1.0*	*81*			
	3.00	*215.24*	*0.00*		*215.24*
RSM Q19 Q19	40.0	2,870	0.0	13.3	2,870
MIL B-STM/PIPE Steam/Pipefitters	*1.0*	*59*			
MIL B-STM/PIPE Steam/Pipefitters	*1.0*	*76*			
MIL B-ELECTRN Electricians	*1.0*	*81*			
	1.00	*56.30*	*0.00*		*56.30*
USR CLABA1 1 laborer	11,658.0	656,345	0.0	11,658.0	656,345
MIL X-LABORER Outside Laborers, (Semi-Skilled)	*1.0*	*56*			
	1.00	*57.30*	*0.00*		*57.30*
USR CLABA2 1 laborer foreman	1,793.0	102,739	0.0	1,793.0	102,739
MIL X-LABORER Outside Laborers, (Semi-Skilled)	*1.0*	*57*			
	1.00	*66.44*	*0.00*		*66.44*
USR COELB1 1 eqoprlt	16,224.0	1,077,923	0.0	16,224.0	1,077,923
MIL X-EQOPRLT Outside Equip. Operators, Light	*1.0*	*66*			
	1.00	*71.65*	*0.00*		*71.65*
USR COEMEQHY1 1 eqoprhvy	5,389.0	386,122	0.0	5,389.0	386,122
MIL X-EQOPRHVY Outside Equip. Operators, Heavy	*1.0*	*72*			
	1.00	*75.56*	*0.00*		*75.56*
USR MPLUPLUM1 1 plumber	320.0	24,179	0.0	320.0	24,179
MIL X-PLUMBER Outside Plumbers	*1.0*	*76*			
	0.00	*0.00*	*0.00*		*0.00*
USR N/A No Crew	0.0	0	0.0	5,112.0	0
	1.00	*25.56*	*0.00*		*25.56*
USR USURSURV1 Surveyor, Chief	1,600.0	40,896	0.0	1,600.0	40,896
FOP FC-SURYC Surveyors, Chief	*1.0*	*26*			
	1.00	*24.02*	*0.00*		*24.02*
USR USURSURVS Surveyors	2,400.0	57,648	0.0	2,400.0	57,648
FOP FC-SURYR Surveyors	*1.0*	*24*			

Description	ManHours	LaborCost	EQHours	CrewHours	CrewCost
	0.00	*0.00*	*1.00*		*53.28*
USR ZCRA14T Crane, hydraulic, truck mtd, 14 ton	0.0	0	589.0	589.0	31,380
GEN C80Z2240 CRANE, HYDRAULIC, TRUCK MOUNTED, 14 TON (12.7 MT), 80' (24.4 M) BOOM, 6X4			*1.0*		
	0.00	*0.00*	*1.00*		*144.21*
USR ZDOZHVY2 Dozer, 310 HP, w/blade	0.0	0	9,640.0	9,640.0	1,390,190
MAP T15CA016 TRACTOR, CRAWLER (DOZER), 310 HP, POWERSHIFT, W/15.3 CY SEMI-U BLADE (ADD ATTACHMENTS)			*1.0*		
	0.00	*0.00*	*1.00*		*3.21*
USR ZGENPORT5.6KW Generator set, portable, 5.6 Kw, 120/240V	0.0	0	286.0	286.0	919
MAP G10WC002 GENERATOR SET, PORTABLE, 5.6 KW, 120/240V, 60 HZ			*1.0*		
	0.00	*0.00*	*1.00*		*66.81*
USR ZGENSKIDMTD300KW Generator set, skid mtd. 300 KW	0.0	0	4,800.0	4,800.0	320,707
EP G10XX012 GENERATOR SET, SKID MTD, 300 KW			*1.0*		
	0.00	*0.00*	*1.00*		*32.28*
USR ZMARBOAT18-FTW/OCABIN Boat, 18' River Runner, w/o Cabin Vee Hull, Cap 1,350 lbs, Outboard, 18'x7.9'x0.5'	0.0	0	800.0	800.0	25,825
MAP M10SM005 MARINE EQUIPMENT, BOATS & LAUNCHES, 18' RIVER RUNNER, VEE HULL, NO CABIN, CAP 1,350 LBS, OUTBOARD, 18' X 7.9' X 0.5'			*1.0*		
	0.00	*0.00*	*1.00*		*1.58*
USR ZMISSMTOOL1 Small Tools	0.0	0	21,378.0	21,378.0	33,777
NON XMIXX020 SMALL TOOLS			*1.0*		
	0.00	*0.00*	*1.00*		*16.23*
USR ZTRKTRPKP1 Truck, pickup, 1 ton	0.0	0	6,290.0	6,290.0	102,096
EP T50XX012 TRUCK, HIGHWAY, CREW, 1 TON PICKUP, 4X4			*1.0*		
	0.00	*0.00*	*1.00*		*15.58*
USR ZWELGAS3KWTRMTD Welder, Engine Driven, Gas, 300 Amp, 3 KW, Trailer Mtd.	0.0	0	210.0	210.0	3,272
EP W35XX023 WELDER, ENGINE DRIVEN, GAS, DC-CC, 300 AMP, 3 KW, TRAILER MTD			*1.0*		

Description	SUIExperience	SUIRate	FICA	FUIRate	PayrollTax	State	ContractorCla	WCIBaseRate	WCIExperience	WCIRate
Contractors Labor Payroll Markup Report										
1 AA Prime Contractor (Initial Construction) - Land Based Equipment	256.60	6.88	7.65	0.60	15.13	NJ	Excavation -- rock/earth NOC	5.32	98.01	17.48
1.2 SU Survey Sub	256.60	6.88	7.65	0.60	15.13	NJ	Excavation -- rock/earth NOC	5.32	328.66	17.48
1.3 EL Electrical Sub	256.60	6.88	7.65	0.60	15.13	NJ	Electrical Wiring -- inside	3.32	526.51	17.48
1.4 SE Security Sub	256.60	6.88	7.65	0.60	15.13	NJ	Clerical Help	0.24	7,284.06	17.48
2 AB Prime Contractor - No markups.	256.60	6.88	7.65	0.60	15.13	NJ	Excavation -- rock/earth NOC	5.32	98.01	17.48

Description	BaseWage	Overtime	Payroll	WCI	TaxableFringe	NonTaxFringe	Travel	Total	ManHours
Labor Backup									
	37.71				*0.00*	*7.92*	*0.00*	*57.93*	
FOP FA-AGENS General Superintendents (P.M.)	144,429	0	21,857	25,251	0	30,334	0	221,870	3,830.0
	37.71				*0.00*	*7.92*	*0.00*	*57.93*	
FOP FA-PROJM Project Managers	1,057	0	160	185	0	222	0	1,623	28.0
	25.74				*0.00*	*6.62*	*0.00*	*40.76*	
FOP FB-ACONT Contract Administrators	8,160	0	1,235	1,427	0	2,099	0	12,919	317.0
	14.72				*0.00*	*5.42*	*0.00*	*24.94*	
FOP FB-CLTYP Clerks, Typists, Bookkeepers & Receptionist	589	0	89	103	0	217	0	998	40.0
	35.18				*0.00*	*7.65*	*0.00*	*54.30*	
FOP FC-ENGCI Engineers, Civil	149,867	0	22,680	26,201	0	32,589	0	231,337	4,260.0
	25.36				*0.00*	*6.58*	*0.00*	*40.21*	
FOP FC-FLDRT Field Draftsmen	5,807	0	879	1,015	0	1,507	0	9,208	229.0
	19.61				*0.00*	*5.95*	*0.00*	*31.96*	
FOP FC-SURYC Surveyors, Chief	31,376	0	4,748	5,486	0	9,520	0	51,130	1,600.0
	18.22				*0.00*	*5.80*	*0.00*	*29.96*	
FOP FC-SURYR Surveyors	43,728	0	6,617	7,646	0	13,920	0	71,911	2,400.0
	35.18				*0.00*	*7.65*	*0.00*	*54.30*	
FOP FD-SAENG Safety Engineers	145,504	0	22,020	25,439	0	31,640	0	224,603	4,136.0
	18.63				*0.00*	*5.84*	*0.00*	*30.55*	
FOP FD-SECWT Security, Watchmen/Guards	48,289	0	7,308	8,442	0	15,137	0	79,176	2,592.0
	41.49				*0.00*	*23.23*	*0.00*	*78.25*	
MIL B-CARPNTER Carpenters	8,630	0	1,306	1,509	0	4,832	0	16,277	208.0
	46.51				*0.00*	*34.39*	*0.00*	*96.07*	
MIL B-ELECTRN Electricians	17,697	0	2,678	3,093	0	13,086	0	36,554	380.5
	41.48				*0.00*	*28.50*	*0.00*	*83.51*	
MIL B-EQOPRMED Equip. Operators, Medium	436,701	78,805	131,679	76,349	0	300,048	0	1,023,583	10,528.0
	36.23				*0.00*	*28.50*	*0.00*	*76.55*	
MIL B-EQOPROIL Equip. Operators, Oilers	4,058	0	614	709	0	3,192	0	8,573	112.0
	43.02				*0.00*	*32.54*	*0.00*	*89.59*	
MIL B-STM/PIPE Steam/Pipefitters	574	0	87	100	0	434	0	1,195	13.3

Description	BaseWage	Overtime	Payroll	WCI	TaxableFringe	NonTaxFringe	Travel	Total	ManHours
	26.24				*0.00*	*32.54*	*0.00*	*67.34*	
MIL B-STM/PIPE Steam/Pipefitters	350	0	53	61	0	434	0	898	13.3
	31.05				*0.00*	*21.49*	*0.00*	*62.67*	
MIL B-TRKDVRHV Truck Drivers, Heavy	8,135	0	1,231	1,422	0	5,630	0	16,419	262.0
	43.07				*0.00*	*28.58*	*0.00*	*95.23*	
MIL X-EQOPRHVY Outside Equip. Operators, Heavy	232,104	40,913	45,589	40,579	0	154,018	0	513,202	5,389.0
	37.94				*0.00*	*28.50*	*0.00*	*89.06*	
MIL X-EQOPRLT Outside Equip. Operators, Light	615,539	121,815	137,547	107,615	0	462,384	0	1,444,899	16,224.0
	33.90				*0.00*	*22.40*	*0.00*	*77.77*	
MIL X-LABORER Outside Laborers, (Semi-Skilled)	395,206	32,202	149,037	69,094	0	261,139	0	906,679	11,658.0
	34.90				*0.00*	*22.40*	*0.00*	*68.68*	
MIL X-LABORER Outside Laborers, (Semi-Skilled)	62,576	0	9,470	10,940	0	40,163	0	123,149	1,793.0
	43.02				*0.00*	*32.54*	*0.00*	*89.59*	
MIL X-PLUMBER Outside Plumbers	13,766	0	2,083	2,407	0	10,413	0	28,669	320.0

Description	Depr/Rntl	FCCM	Fuel	FOG	TireWear	TireRepair	EQRepair	Total	EQHours
Equipment Backup									
	4.77	*0.16*	*51.21*	*5.83*	*0.00*	*0.00*	*4.52*	*66.50*	
EP G10XX012 GENERATOR SET, SKID MTD, 300 KW	22,910	763	245,830	27,973	0	0	21,706	319,182	4,800.0
	2.61	*0.26*	*9.07*	*1.21*	*0.23*	*0.04*	*2.81*	*16.23*	
EP T50XX012 TRUCK, HIGHWAY, CREW, 1 TON PICKUP, 4X4	16,431	1,666	57,063	7,593	1,445	243	17,655	102,096	6,290.0
	1.11	*0.12*	*11.66*	*1.33*	*0.03*	*0.00*	*1.32*	*15.58*	
EP W35XX023 WELDER, ENGINE DRIVEN, GAS, DC-CC, 300 AMP, 3 KW, TRAILER MTD	234	26	2,449	279	5	1	278	3,272	210.0
	8.90	*1.37*	*29.32*	*5.02*	*1.11*	*0.19*	*7.38*	*53.28*	
GEN C80Z2240 CRANE, HYDRAULIC, TRUCK MOUNTED, 14 TON (12.7 MT), 80' (24.4 M) BOOM, 6X4	5,243	805	17,268	2,955	653	110	4,347	31,380	589.0
	7.20	*0.97*	*8.51*	*1.13*	*0.14*	*0.02*	*11.37*	*29.34*	
GEN D30Z2840 DRILL, EARTH/AUGER, HYDRAULIC AUGER, 14" (356 MM) DIA, 30' (9.1 M) DEPTH, 3,500 FT-LBS (483.9 KGF-M), W/TRAILER (ADD COST FOR DRILL STEEL AND CUTTING EDGE WEAR)	231	31	272	36	4	1	364	939	32.0
	5.41	*0.74*	*6.21*	*3.07*	*0.70*	*0.12*	*6.85*	*23.09*	
GEN L50Z4640 LOADER/BACKHOE, WHEEL, 0.80 CY (0.6 M3) FRONT END BUCKET, 9.8' (3.0 M) DEPTH OF HOE, 24" (0.61 M) DIPPER, 4X4	130	18	149	74	17	3	164	554	24.0
	1.25	*0.10*	*0.00*	*0.00*	*0.00*	*0.00*	*1.15*	*2.51*	
GEN T40Z7090 TRUCK OPTION, DUMP BODY, REAR, 12 CY (9.2 M3) (ADD 45,000 LB (20,412 KG) GVW TRUCK)	58	5	0	0	0	0	53	115	46.0
	2.85	*0.36*	*24.19*	*3.45*	*0.42*	*0.07*	*2.86*	*34.21*	
GEN T50Z7400 TRUCK, HIGHWAY, 25,000 LB (11,340 KG) GVW, 4X2, 2 AXLE (ADD ACCESSORIES)	68	9	581	83	10	2	69	821	24.0
	30.63	*1.69*	*33.35*	*4.75*	*15.91*	*2.67*	*34.05*	*123.06*	
MAP C75GV019 CRANES, HYDRAULIC, SELF-PROPELLED, ROUGH TERRAIN, 50 TON, 110' BOOM, 4X4	149,974	8,253	163,284	23,262	77,915	13,090	166,707	602,484	4,896.0
	0.32	*0.03*	*2.34*	*0.27*	*0.00*	*0.00*	*0.26*	*3.21*	
MAP G10WC002 GENERATOR SET, PORTABLE, 5.6 KW, 120/240V, 60 HZ	92	8	668	76	0	0	75	919	286.0
	1.91	*0.32*	*24.43*	*3.71*	*0.00*	*0.00*	*1.91*	*32.28*	

Description	Depr/Rntl	FCCM	Fuel	FOG	TireWear	TireRepair	EQRepair	Total	EQHours
MAP M10SM005 MARINE EQUIPMENT, BOATS & LAUNCHES, 18' RIVER RUNNER, VEE HULL, NO CABIN, CAP 1,350 LBS, OUTBOARD, 18' X 7.9' X 0.5'	1,528	259	19,541	2,968	0	0	1,530	25,825	800.0
	32.23	*1.77*	*40.68*	*5.42*	*0.00*	*0.00*	*60.61*	*140.72*	
MAP T15CA016 TRACTOR, CRAWLER (DOZER), 310 HP, POWERSHIFT, W/15.3 CY SEMI-U BLADE (ADD ATTACHMENTS)	310,722	17,040	392,198	52,272	0	0	584,293	1,356,525	9,640.0
	5.34	*0.59*	*0.00*	*0.50*	*1.58*	*0.27*	*3.54*	*11.82*	
MAP T45XX019 TRUCK TRAILER, LOWBOY, 75 TON, 3 AXLE (ADD TOWING TRUCK)	1,195	133	0	112	354	59	794	2,648	224.0
	6.87	*1.02*	*32.36*	*4.31*	*0.94*	*0.16*	*6.87*	*52.52*	
MAP T50XX029 TRUCK, HIGHWAY, 50,000 LBS GVW, 3 AXLE, 6X4 (CHASSIS ONLY-ADD OPTIONS)	1,854	275	8,738	1,163	253	43	1,854	14,180	270.0
	0.50	*0.22*	*0.16*	*0.07*	*0.00*	*0.00*	*0.63*	*1.58*	
NON XMIXX020 SMALL TOOLS	10,892	3,384	3,485	1,525	0	0	13,723	34,417	21,783.0

INITIAL PROJECT CHARGES USING HOPPER DREDGING FOR BEACH FILL

24. General: This section presents detailed cost estimates for initial construction, nourishment, maintenance, monitoring and major rehabilitation resulting in total and annualized project costs for alternative storm damage reduction plans using hopper dredging for beach fill. The fifteen alternative plans developed using hopper dredging for beach fill include:

Plan	Description
A	115' wide berm with +12' NAVD dune using 4 Yr. Cycle
B	95' wide berm with +14' NAVD dune using 4 Yr. Cycle
C	75' wide berm with +16' NAVD dune using 4 Yr. Cycle
D	140' wide berm with +12' NAVD dune using 4 Yr. Cycle
E	120' wide berm with +14' NAVD dune using 4 Yr. Cycle
F	100' wide berm with +16' NAVD dune using 4 Yr. Cycle
G	165' wide berm with +12' NAVD dune using 4 Yr. Cycle
H	145' wide berm with +14' NAVD dune using 4 Yr. Cycle
I	125' wide berm with +16' NAVD dune using 4 Yr. Cycle
J	80' wide berm with +18' NAVD dune using 4 Yr. Cycle
K	105' wide berm with +18' NAVD dune using 4 Yr. Cycle
L	85' wide berm with +20' NAVD dune using 4 Yr. Cycle
M	110' wide berm with +20' NAVD dune using 4 Yr. Cycle
N	160' wide berm with +20' NAVD dune using 4 Yr. Cycle
O	No action

The top of the berm is at an elevation of +6.5' NAVD and extends from 2nd Avenue in North Wildwood to Juniper Avenue in Wildwood. The dune for each alternative has 1 on 5 side slopes and a top width of 25'. The dune extends the same distance as the berm. The initial construction for each of the above plans includes design and advanced nourishment beach fill. Also included are provisions for periodic nourishment, beach profile and environmental monitoring, and major replacement to restore the design beach profile damaged by significant storm events beyond that designed for in the nourishment cycle quantity. The plan layout of the NED plan with typical improved beach sections is shown in the section of the Feasibility Study, Main Report describing the NED Plan.

25. Basis of Cost: Cost estimates presented herein are based on June 2007 price levels. Initial beach fill costs are based on beach surveys taken in October 2003. The unit prices were developed in accordance with the construction procedures outlined herein. All initial construction and nourishment costs and major rehabilitation costs presented in this appendix are NED costs.

26. Initial beach fill costs are based on the assumption that a generic medium-size hopper dredge was used for placement of the beach fill. Approximately 944,500 C.Y. of beach fill material from offshore borrow area H was placed in Cells 1 and 2. The average haul distance for the initial beach fill is 0.47 miles. A mooring barge was located approximately 3,400 feet offshore of North Wildwood beach based on the benthics to allow access for a loaded dredge. The average pumping distance for these cells uses an average pipeline length of 6,640 L.F. for the initial beach fill.

27. Periodic nourishment beach fill costs are based on the assumption that a generic medium-size hopper dredge was used for placement of the beach fill. Approximately 341,600 C.Y.of beach fill material from offshore borrow area OS-2 was placed in Cells 1 and 2. The average haul distance for the nourishment cycle is 7.1 miles. A mooring barge was located approximately 5,100 feet off shore of Wildwood beach based on the benthics to allow access for a loaded dredge. The average pumping distance for the nourishment cycle uses an average pipeline length of 5,120 L.F.

28. Mobilization and demobilization costs are based on the assumption that beach filling equipment located within 250 miles from the project site will perform the work. Construction access would be by local streets. The locations of the borrow areas are displayed in the section of the Feasibility Study, Main Report describing the NED Plan.

29. Real estate costs for the fifteen alternatives included in the Cycle 3 screening were not included since they are expected to be minimal as most of the land is a public beach owned by the sponsor.

30. Environmental monitoring costs for the fifteen alternatives included in the Cycle 3 screening were not included since they are dependent on the EA document and that document was not finalized at the time of the Cycle 3 screening.

31. Construction Management costs for the fifteen alternatives included in the Cycle 3 screening were included as a percent of the construction cost and is based on ER415-1-16, Table E-1. A 15 percent contingency has been included in S&A costs.

32. Alternatives Considered: Alternative plans were developed in two phases for the plan selection process. In the first phase the alternative plans were compared during the Cycle 1 and Cycle 2 screening process. For more information on these plans, refer to the section of the Feasibility Study, Main Report describing the NED Plan. Based on an analysis of these annual costs with their associated benefits, the beach restoration only plan was selected for the second phase for final plan optimization and selection.

33. The costs for the fifteen alternatives as described in paragraph 24 for this second phase of plan selection are shown in Tables 5A thru 5N.

34. Renourishment Interval Optimization: For more information on the renourishment interval optimization that selected the 4-year cycle, refer to the section of the Feasibility Study, Main Report describing the NED plan.

35. Total First Cost for Selected Plan: The estimated project first cost is for the selected plan - dune and berm constructed using 944,500 CY of hydraulically placed design and advanced nourishment beach fill from offshore borrow area H. A +16’ NAVD high dune with a top width of 25’ on a 75’ wide berm that is 6.5’ NAVD high would be constructed from North Wildwood to the northern border of Wildwood and is based on a selected nourishment cycle of 4 years. It was assumed that beach filling work would be performed by a dredging contractor. Pertinent contingency, engineering and design and construction management costs are also included. For more information on the selected plan using hopper dredging for beach fill as an option, refer to the section of the Feasibility Study, Main Report describing the NED plan.

Table 5A - Total First Cost
Plan A (115' Berm w/ 12' NAVD Dune using 4 Yr. Cycle)

Price Level: Jun 07

ACCOUNT NUMBER	DESCRIPTION OF ITEM	QUANTITY	UOM	UNIT PRICE	ESTIMATED AMOUNT	CONTINGENCY	TOTAL COST
01.	Lands and Damages	1	Job	LS	$0	$0	$0
17.	Beach Replenishment						
	Mobilization, Demob. And Preparatory Work	1	Job	LS	$688,723	$82,647	$771,370
	Hopper Dredging						
	Site Work, Excavation and Disposal						
	Cell 1	801,900	CY	$6.50	$5,212,350	$781,853	$5,994,203
	Cell 2	46,600	CY	$6.63	$308,958	$46,344	$355,302
	Total Beach Replacement				$6,210,031	$910,843	$7,120,874
30.	Planning, Engineering and Design (P,E & D)	1	Job	LS	$1,000,000	$150,000	$1,150,000
31.	Construction Management (S & A @ 9.89%)	1	Job	LS	$614,172	$90,082	$704,254
	Total Project First Cost				$7,824,203	$1,150,925	$8,975,128
	(Rounded)				$7,824,000	$1,151,000	$8,975,000

Notes:
Dredging quantity includes 4 yr. nourishment cycle.

Table 5B - Total First Cost Price Level: Jun 07

Plan B (95' Berm w/ 14' NAVD Dune using 4 Yr. Cycle)

ACCOUNT NUMBER	DESCRIPTION OF ITEM	QUANTITY	UOM	UNIT PRICE	ESTIMATED AMOUNT	CONTINGENCY	TOTAL COST
01.	Lands and Damages	1	Job	LS	$0	$0	$0
17.	Beach Replenishment						
	Mobilization, Demob. And Preparatory Work	1	Job	LS	$688,723	$82,647	$771,370
	Hopper Dredging						
	Site Work, Excavation and Disposal						
	Cell 1	824,900	CY	$6.49	$5,353,601	$803,040	$6,156,641
	Cell 2	66,600	CY	$6.61	$440,226	$66,034	$506,260
	Total Beach Replacement				$6,482,550	$951,721	$7,434,271
30.	Planning, Engineering and Design (P,E & D)	1	Job	LS	$1,000,000	$150,000	$1,150,000
31.	Construction Management (S & A @ 9.89%)	1	Job	LS	$641,124	$94,125	$735,249
	Total Project First Cost				$8,123,674	$1,195,846	$9,319,520
	(Rounded)				$8,124,000	$1,196,000	$9,320,000

Notes:
Dredging quantity includes 4 yr. nourishment cycle.

Table 5C - Total First Cost

Plan C (75' Berm w/ 16' NAVD Dune using 4 Yr. Cycle)

Price Level: Jun 07

ACCOUNT NUMBER	DESCRIPTION OF ITEM	QUANTITY	UOM	UNIT PRICE	ESTIMATED AMOUNT	CONTINGENCY	TOTAL COST
01.	Lands and Damages	1	Job	LS	$0	$0	$0
17.	Beach Replenishment						
	Mobilization, Demob. And Preparatory Work	1	Job	LS	$688,723	$82,647	$771,370
	Hopper Dredging						
	Site Work, Excavation and Disposal						
	Cell 1	853,900	CY	$6.47	$5,524,733	$828,710	$6,353,443
	Cell 2	90,600	CY	$6.60	$597,960	$89,694	$687,654
	Total Beach Replacement				$6,811,416	$1,001,051	$7,812,467
30.	Planning, Engineering and Design (P,E & D)	1	Job	LS	$1,000,000	$150,000	$1,150,000
31.	Construction Management (S & A @ 9.89%)	1	Job	LS	$673,649	$99,004	$772,653
	Total Project First Cost				$8,485,065	$1,250,055	$9,735,120
	(Rounded)				$8,485,000	$1,250,000	$9,735,000

Notes:
Dredging quantity includes 4 yr. nourishment cycle.

Table 5D - Total First Cost

Price Level: Jun 07

Plan D (140' Berm w/ 12' NAVD Dune using 4 Yr. Cycle)

ACCOUNT NUMBER	DESCRIPTION OF ITEM	QUANTITY	UOM	UNIT PRICE	ESTIMATED AMOUNT	CONTINGENCY	TOTAL COST
01.	Lands and Damages	1	Job	LS	$0	$0	$0
17.	Beach Replenishment						
	Mobilization, Demob. And Preparatory Work	1	Job	LS	$688,723	$82,647	$771,370
	Hopper Dredging						
	Site Work, Excavation and Disposal						
	Cell 1	981,500	CY	$6.46	$6,340,490	$951,074	$7,291,564
	Cell 2	60,200	CY	$6.58	$396,116	$59,417	$455,533
	Total Beach Replacement				$7,425,329	$1,093,138	$8,518,467
30.	Planning, Engineering and Design (P,E & D)	1	Job	LS	$1,000,000	$150,000	$1,150,000
31.	Construction Management (S & A @ 9.89%)	1	Job	LS	$734,365	$108,111	$842,476
	Total Project First Cost				$9,159,694	$1,351,249	$10,510,943
	(Rounded)				$9,160,000	$1,351,000	$10,511,000

Notes:

Dredging quantity includes 4 yr. nourishment cycle.

Table 5E - Total First Cost Price Level: Jun 07

Plan E (120' Berm w/ 14' NAVD Dune using 4 Yr. Cycle)

ACCOUNT NUMBER	DESCRIPTION OF ITEM	QUANTITY	UOM	UNIT PRICE	ESTIMATED AMOUNT	CONTINGENCY	TOTAL COST
01.	Lands and Damages	1	Job	LS	$0	$0	$0
17.	Beach Replenishment						
	Mobilization, Demob. And Preparatory Work	1	Job	LS	$688,723	$82,647	$771,370
	Hopper Dredging						
	Site Work, Excavation and Disposal						
	Cell 1	1,004,500	CY	$6.45	$6,479,025	$971,854	$7,450,879
	Cell 2	80,200	CY	$6.57	$526,914	$79,037	$605,951
	Total Beach Replacement				$7,694,662	$1,133,538	$8,828,200
30.	Planning, Engineering and Design (P,E & D)	1	Job	LS	$1,000,000	$150,000	$1,150,000
31.	Construction Management (S & A @ 9.89%)	1	Job	LS	$761,002	$112,107	$873,109
	Total Project First Cost				$9,455,664	$1,395,644	$10,851,309
	(Rounded)				$9,456,000	$1,396,000	$10,851,000

Notes:
Dredging quantity includes 4 yr. nourishment cycle.

Table 5F - Total First Cost

Price Level: Jun 07

Plan F (100' Berm w/ 16' NAVD Dune using 4 Yr. Cycle)

ACCOUNT NUMBER	DESCRIPTION OF ITEM	QUANTITY	UOM	UNIT PRICE	ESTIMATED AMOUNT	CONTINGENCY	TOTAL COST
01.	Lands and Damages	1	Job	LS	$0	$0	$0
17.	Beach Replenishment						
	Mobilization, Demob. And Preparatory Work	1	Job	LS	$688,723	$82,647	$771,370
	Hopper Dredging						
	Site Work, Excavation and Disposal						
	Cell 1	1,033,500	CY	$6.43	$6,645,405	$996,811	$7,642,216
	Cell 2	104,200	CY	$6.56	$683,552	$102,533	$786,085
	Total Beach Replacement				$8,017,680	$1,181,990	$9,199,670
30.	Planning, Engineering and Design (P,E & D)	1	Job	LS	$1,000,000	$150,000	$1,150,000
31.	Construction Management (S & A @ 9.89%)	1	Job	LS	$792,949	$116,899	$909,847
	Total Project First Cost				$9,810,629	$1,448,889	$11,259,518
	(Rounded)				$9,811,000	$1,449,000	$11,260,000

Notes:
Dredging quantity includes 4 yr. nourishment cycle.

Table 5G - Total First Cost Price Level: Jun 07

Plan G (165' Berm w/ 12' NAVD Dune using 4 Yr. Cycle)

ACCOUNT NUMBER	DESCRIPTION OF ITEM	QUANTITY	UOM	UNIT PRICE	ESTIMATED AMOUNT	CONTINGENCY	TOTAL COST
01.	Lands and Damages	1	Job	LS	$0	$0	$0
17.	Beach Replenishment						
	Mobilization, Demob. And Preparatory Work	1	Job	LS	$688,723	$82,647	$771,370
	Hopper Dredging						
	Site Work, Excavation and Disposal						
	Cell 1	1,178,200	CY	$6.42	$7,564,044	$1,134,607	$8,698,651
	Cell 2	78,000	CY	$6.54	$510,120	$76,518	$586,638
	Total Beach Replacement				$8,762,887	$1,293,771	$10,056,658
30.	Planning, Engineering and Design (P,E & D)	1	Job	LS	$1,000,000	$150,000	$1,150,000
31.	Construction Management (S & A @ 9.46%)	1	Job	LS	$828,969	$122,391	$951,360
	Total Project First Cost				$10,591,856	$1,566,162	$12,158,018
	(Rounded)				$10,592,000	$1,566,000	$12,158,000

Notes:
Dredging quantity includes 4 yr. nourishment cycle.

Table 5H - Total First Cost Price Level: Jun 07

Plan H (145' Berm w/ 14' NAVD Dune using 4 Yr. Cycle)

ACCOUNT NUMBER	DESCRIPTION OF ITEM	QUANTITY	UOM	UNIT PRICE	ESTIMATED AMOUNT	CONTINGENCY	TOTAL COST
01.	Lands and Damages	1	Job	LS	$0	$0	$0
17.	Beach Replenishment						
	Mobilization, Demob. And Preparatory Work	1	Job	LS	$688,723	$82,647	$771,370
	Hopper Dredging						
	Site Work, Excavation and Disposal						
	Cell 1	1,201,200	CY	$6.40	$7,687,680	$1,153,152	$8,840,832
	Cell 2	98,000	CY	$6.53	$639,940	$95,991	$735,931
	Total Beach Replacement				$9,016,343	$1,331,790	$10,348,133
30.	Planning, Engineering and Design (P,E & D)	1	Job	LS	$1,000,000	$150,000	$1,150,000
31.	Construction Management (S & A @ 9.46%)	1	Job	LS	$852,946	$125,987	$978,933
	Total Project First Cost				$10,869,289	$1,607,777	$12,477,066
	(Rounded)				$10,869,000	$1,608,000	$12,477,000

Notes:
Dredging quantity includes 4 yr. nourishment cycle.

Table 5I - Total First Cost Price Level: Jun 07

Plan I (125' Berm w/ 16' NAVD Dune using 4 Yr. Cycle)

ACCOUNT NUMBER	DESCRIPTION OF ITEM	QUANTITY	UOM	UNIT PRICE	ESTIMATED AMOUNT	CONTINGENCY	TOTAL COST
01.	Lands and Damages	1	Job	LS	$0	$0	$0
17.	Beach Replenishment						
	Mobilization, Demob. And Preparatory Work	1	Job	LS	$688,723	$82,647	$771,370
	Hopper Dredging						
	Site Work, Excavation and Disposal						
	Cell 1	1,230,200	CY	$6.39	$7,860,978	$1,179,147	$9,040,125
	Cell 2	122,000	CY	$6.52	$795,440	$119,316	$914,756
	Total Beach Replacement				$9,345,141	$1,381,109	$10,726,250
30.	Planning, Engineering and Design (P,E & D)	1	Job	LS	$1,000,000	$150,000	$1,150,000
31.	Construction Management (S & A @ 9.46%)	1	Job	LS	$884,050	$130,653	$1,014,703
	Total Project First Cost				$11,229,191	$1,661,762	$12,890,954
	(Rounded)				$11,229,000	$1,662,000	$12,891,000

Notes:

Dredging quantity includes 4 yr. nourishment cycle.

Table 5J - Total First Cost

Plan J (80' Berm w/ 18' NAVD Dune using 4 Yr. Cycle)

Price Level: Jun 07

ACCOUNT NUMBER	DESCRIPTION OF ITEM	QUANTITY	UOM	UNIT PRICE	ESTIMATED AMOUNT	CONTINGENCY	TOTAL COST
01.	Lands and Damages	1	Job	LS	$0	$0	$0
17.	Beach Replenishment						
	Mobilization, Demob. And Preparatory Work	1	Job	LS	$688,723	$82,647	$771,370
	Hopper Dredging						
	Site Work, Excavation and Disposal						
	Cell 1	1,068,500	CY	$6.47	$6,913,195	$1,036,979	$7,950,174
	Cell 2	133,200	CY	$6.55	$872,460	$130,869	$1,003,329
	Total Beach Replacement				$8,474,378	$1,250,495	$9,724,873
30.	Planning, Engineering and Design (P,E & D)	1	Job	LS	$1,000,000	$150,000	$1,150,000
31.	Construction Management (S & A @ 9.89%)	1	Job	LS	$838,116	$123,674	$961,790
	Total Project First Cost				$10,312,494	$1,524,169	$11,836,663
	(Rounded)				$10,312,000	$1,524,000	$11,837,000

Notes:

Dredging quantity includes 4 yr. nourishment cycle.

Table 5K - Total First Cost Price Level: Jun 07

Plan K (105' Berm w/ 18' NAVD Dune using 4 Yr. Cycle)

ACCOUNT NUMBER	DESCRIPTION OF ITEM	QUANTITY	UOM	UNIT PRICE	ESTIMATED AMOUNT	CONTINGENCY	TOTAL COST
01.	Lands and Damages	1	Job	LS	$0	$0	$0
17.	Beach Replenishment						
	Mobilization, Demob. And Preparatory Work	1	Job	LS	$688,723	$82,647	$771,370
	Hopper Dredging						
	Site Work, Excavation and Disposal						
	Cell 1	1,265,200	CY	$6.37	$8,059,324	$1,208,899	$9,268,223
	Cell 2	151,000	CY	$6.50	$981,500	$147,225	$1,128,725
	Total Beach Replacement				$9,729,547	$1,438,770	$11,168,317
30.	Planning, Engineering and Design (P,E & D)	1	Job	LS	$1,000,000	$150,000	$1,150,000
31.	Construction Management (S & A @ 9.46%)	1	Job	LS	$920,415	$136,108	$1,056,523
	Total Project First Cost				$11,649,962	$1,724,878	$13,374,840
	(Rounded)				$11,650,000	$1,725,000	$13,375,000

Notes:
Dredging quantity includes 4 yr. nourishment cycle.

Table 5L - Total First Cost Price Level: Jun 07

Plan L (85' Berm w/ 20' NAVD Dune using 4 Yr. Cycle)

ACCOUNT NUMBER	DESCRIPTION OF ITEM	QUANTITY	UOM	UNIT PRICE	ESTIMATED AMOUNT	CONTINGENCY	TOTAL COST
01.	Lands and Damages	1	Job	LS	$0	$0	$0
17.	Beach Replenishment						
	Mobilization, Demob. And Preparatory Work	1	Job	LS	$688,723	$82,647	$771,370
	Hopper Dredging						
	Site Work, Excavation and Disposal						
	Cell 1	1,304,200	CY	$6.35	$8,281,670	$1,242,251	$9,523,921
	Cell 2	184,000	CY	$6.49	$1,194,160	$179,124	$1,373,284
	Total Beach Replacement				$10,164,553	$1,504,021	$11,668,574
30.	Planning, Engineering and Design (P,E & D)	1	Job	LS	$1,000,000	$150,000	$1,150,000
31.	Construction Management (S & A @ 9.46%)	1	Job	LS	$961,567	$142,280	$1,103,847
	Total Project First Cost				$12,126,120	$1,796,302	$13,922,421
	(Rounded)				$12,126,000	$1,796,000	$13,922,000

Notes:
Dredging quantity includes 4 yr. nourishment cycle.

Table 5M - Total First Cost
Plan M (110' Berm w/ 20' NAVD Dune using 4 Yr. Cycle)

Price Level: Jun 07

ACCOUNT NUMBER	DESCRIPTION OF ITEM	QUANTITY	UOM	UNIT PRICE	ESTIMATED AMOUNT	CONTINGENCY	TOTAL COST
01.	Lands and Damages	1	Job	LS	$0	$0	$0
17.	Beach Replenishment						
	Mobilization, Demob. And Preparatory Work	1	Job	LS	$688,723	$82,647	$771,370
	Hopper Dredging						
	Site Work, Excavation and Disposal						
	Cell 1	1,624,900	CY	$6.32	$10,269,368	$1,540,405	$11,809,773
	Cell 2	209,400	CY	$6.45	$1,350,630	$202,595	$1,553,225
	Total Beach Replacement				$12,308,721	$1,825,646	$14,134,367
30.	Planning, Engineering and Design (P,E & D)	1	Job	LS	$1,000,000	$150,000	$1,150,000
31.	Construction Management (S & A @ 9.46%)	1	Job	LS	$1,164,405	$172,706	$1,337,111
	Total Project First Cost				$14,473,126	$2,148,353	$16,621,479
	(Rounded)				$14,473,000	$2,148,000	$16,621,000

Notes:
Dredging quantity includes 4 yr. nourishment cycle.

Table 5N - Total First Cost

Price Level: Jun 07

Plan N (160' Berm w/ 20' NAVD Dune using 4 Yr. Cycle)

ACCOUNT NUMBER	DESCRIPTION OF ITEM	QUANTITY	UOM	UNIT PRICE	ESTIMATED AMOUNT	CONTINGENCY	TOTAL COST
01.	Lands and Damages	1	Job	LS	$0	$0	$0
17.	Beach Replenishment						
	Mobilization, Demob. And Preparatory Work	1	Job	LS	$1,377,446	$165,294	$1,542,740
	Hopper Dredging						
	Site Work, Excavation and Disposal						
	Cell 1	2,306,900	CY	$6.27	$14,464,263	$2,169,639	$16,633,902
	Cell 2	248,900	CY	$6.40	$1,592,960	$238,944	$1,831,904
	Total Beach Replacement				$17,434,669	$2,573,877	$20,008,546
30.	Planning, Engineering and Design (P,E & D)	1	Job	LS	$1,000,000	$150,000	$1,150,000
31.	Construction Management (S & A @ 8.92%)	1	Job	LS	$1,555,172	$229,590	$1,784,762
	Total Project First Cost				$19,989,841	$2,953,467	$22,943,308
	(Rounded)				$19,990,000	$2,953,000	$22,943,000

Notes:
Dredging quantity includes 4 yr. nourishment cycle.

Section 16

Dredging Technology

1. Coastal and Hydraulics Laboratory Letter Report
2. A Guide to the Planning and Design of Jet Pump Remedial Sand Bypassing Systems
3. World Wide Sand By-passing Systems: Data Report

Coastal and Hydraulics Laboratory Letter Report

Mobile Hydraulic Backpassing System for the Hereford to Cape May Feasibility Study

March 26, 2008

James E. Clausner, PE
Timothy L. Welp

Coastal and Hydraulics Laboratory Letter Report - Mobile Hydraulic Backpassing System for the Hereford to Cape May Feasibility Study

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Note – this page is considered optional, and if needed, completed after receipt of comments.

Background:

Beach erosion is a problem at North Wildwood, NJ, which is immediately south of Hereford Inlet (Figure 1). As part of the Hereford to Cape May Feasibility Study (simply referred to as the feasibility study from now on) the US Army Corps of Engineers Philadelphia District (NAP) is investigating various options to economically backpass sand from the wide beach to the south, Wildwood, NJ, to North Wildwood (Figure 2). In early February 2008, Mr. Brian Bogle, the project manager for the NAP's Hereford Inlet to Cape May Feasibility study, contacted Messrs. James Clausner (Associate Technical Director for Navigation) and Timothy Welp (Hydraulic Engineer, Coastal Engineering Branch) of the U.S. Army Engineer Research Development Center's (ERDC) Coastal and Hydraulic Laboratory (CHL). Based on recommendations Mr. Jeffrey Gebert (NAP), Mr. Bogle requested CHL assistance in evaluating mobile hydraulic based backpassing systems as possible solutions to the beach erosion problem at North Wildwood. Due to Mr. Welp's prior commitments, Mr. Clausner agreed to conduct the study and have Mr. Welp review the final report. The remainder of this section describes volumes to be backpassed, conventional backpassing options, why mobile submersible pumps may be more desirable for this application, desired submersible pump operating features for this project, and other pertinent engineering details related to this project.

Backpass Volumes. The NAP estimates between 100K and 200K cy/yr need to be backpassed from Wildwood to North Wildwood on an annual basis. However, because of the existing deficit on the North Wildwood beach, a one time "beach nourishment level" effort of 500,000 to 1,000,000 cy is needed prior to the annual backpassing.

Conventional Backpass Options. A proven option for bypassing/backpassing modest volumes is to use conventional earthmoving equipment to collect the sand and then truck it from one location to another. This has been done at Avalon, NJ, twice in the past five years and is being considered at Wildwood. However, the cost of this option, estimated at about $20/cy by NAP for the Wildwood application, is considered to be too expensive. This option also significantly interferes with beach use.

Submersible Pump Options. The high cost and interference with beach use resulting from the conventional earthmoving option for backpassing led to NAP's interest in evaluating mobile hydraulic pumps to entrain the sand at Wildwood in conjunction with booster pumps to transfer the sand to North Wildwood with the desire to reduce costs (ideally to less than $10 per cubic yard). In a recent paper, Chase (2006), described a system based on a submersible pump (Figure 3) deployed from a crane that appears to meet many of the NAP's requirements. The goal Chase's system was to mine a specific volume, 100 to 200K cy from over as limited an area as possible. Chase's (2006) system used a crane for deployment, creating pits in the nearshore zone 15 to 20 ft deep and based costs on backpassing 200K cy a distance of up to 24,000 ft over a period of four months. He also provided data for shorter distances and 100K cy. Chase also included as an option a sheet pile that could be used to increase production by allowing the crane to be positioned further seaward. This sheet pile wall, if used, would have to be removed and moved periodically to mine the relatively thin layer of sand proposed for this project.

NAP Desired Submersible Pump Back Passing Methods. The NAP desires a more mobile system than that described in Chase (2006) with minimal infrastructure support and impact to beach users. Also, rather than dig a series of relatively deep pits, NAP would prefer to remove sand in a thin layer over a wide area with excavation pits no deeper than 10 ft below the zero elevation on the cross section provided by NAP (Figure 4), thought to be typical of the Wildwood Beach. Initially, NAP expressed a desire to remove the material between the blue line (original shoreline) and the red line (minimum cut width) or the black line (maximum cut width) between +5 and -10 ft elevation. Note the datum on this figure in NAVD88. Also, NAP (after discussions with Messrs. Clausner and Welp) requested the contractors consider options to deploy a submersible pump or eductor (jet pump) that could be less expensive than using a crane. Other less expensive deployment options include an A-frame or tripod to deploy the submersible pump. Another option considered by NAP, as described in an e-mail from Mr. Gebert, was to use mechanical equipment, scrapers and dozers to stockpile sand and then use hydraulic pumps to transfer the sand to North Wildwood.

Other Pertinent Details.

Sand Grain Size. The sand to be backpassed is 0.15 mm.

Backpass Distance. Required distance for backpassing is between 5,000 and 15,000 ft, thus booster pumps will be required.

Tide Range. Tide range at the site is 4 ft.

Schedule. Ideally the system would transfer sand outside of the summer tourist season when the beach is most heavily used, thus the material should be transferred from Sep 15th to May15th, a total of 8 months.

With the above background information, most of which was provided during a conference call on Feb 14, 08, Mr. Clausner began the study in early March 2008. Over the course of the study, NAP supplied additional information in response to Mr. Clausner's requests and those from the various companies. An important piece of information was that the datum on the cross section provided was NAVD88. Dr. Don Stauble (CHL), who has done considerable work in the Cape May, provided an estimate of MLLW at Wildwood, which based on NOAA tidal station data, is estimated to be 3 ft below 0.0 NAVD (Stauble 2008).

Study Approach:

Based on the limited time available, less than one month, Mr. Clausner decided to contact a number of reputable companies that sell and, in some cases, perform dredging, with submersible pumps or eductors. Mr. Clausner developed a short Request for Information (RFI) that he supplied to five companies: Javeler Construction Company (Toyo Pumps), Hagler Pumps, Heger Pumps (trade name Drag Flow), DOP (a Dutch company), and Standard Gravel (Genflo Jet Pumps). Appendix A contains the names, phone numbers, e-mail addresses and web sites of the companies contacted. Appendix B contains the full RFI. Following the initial contact, Mr. Clausner had subsequent e-mails and phone conversations with several of the company representatives. Note, we did not investigate any environmental concerns that might be associated with this type of beach mining operation.

Results from Contractor Input and CHL Calculations

Between the conference call on February 14, 2008 and late February, 2008, Messrs Clausner and Bogle developed a mutually agreeable SOW and Mr. Bogle provided funds. During the first week in March, Mr. Clausner, with the assistance of Mr. Welp, developed the list of contractors and POCs, and Mr. Clausner developed the RFI. On March 5th, Mr. Clausner provided the RFI to the above listed contractors. The results of the initial contacts follow.

1. Heger Pumps, which sells Drag Flow Pumps, POC is Siegfried Heger. Mr. Heger responded to the initial RFI, and had a phone conversation with Mr. Clausner. The phone conversation with Mr. Heger showed him to be quite knowledgeable (he also provided information to Mr. Chase in the preparation of his 2006 paper). Mr. Clausner was under the impression that Mr. Heger would provide a written response. However, that did not occur.

2. Hagler Pumps, POC's were Mr. Robert Hagler and Ms. Laurie Nalley. Mr. Hagler and Ms. Nalley did not respond to the RFI.

3. Standard Gravel, POC's were Mr. Spencer Green and Mr. John Green. After the initial RFI, Mr. Spencer Green e-mailed Mr. Clausner that he and his father were leaving shortly on an extended trip would not be able to review the RFI until 14 March at the earliest. Mr. Green did not provide any response.

4. Damen Dredging Equipment, a Dutch Company, which sells DOP Submersible Dredge Pumps, POC was Mr. David Tenwolde. Mr. Tenwolde provided considerable information on appropriate DOP submersible pumps, booster pumps and related equipment appropriate for this application. The complete set of information is provided in Appendix C. Perhaps of most interest were details of a somewhat similar application on the east coast of Italy between Venice and Ravenna conducted in 2003, which used two DOP 2320 submersible pumps and a diesel driven booster and associated instrumentation. In this application, the pumping distances ranged from 3,000 to 18,000 ft, and material was transported through a 300 mm (12-inch) pipe. The submersible pump was deployed from a CAT 320 excavator removing sand from a sheet pile lined pit to prevent the excavator from falling into the pit. Average production rate was 400 cu m/hr (520 cy/hr) over the entire operation.
    a. While the information on this Italian application showed the DOP equipment is very likely quite suitable for the Wildwood Backpass project, specific information on proposed operating scenarios, and total costs for the Wildwood Backpass project were not provided. A rough cost for the full set of equipment specified was provided by Mr. Tenwolde, which included a D2320 pump, diesel driven hydraulic power unit and instrumentation ($410,000), production meter ($66,000), and diesel powered booster pump station ($257,000), which totals to $733,000 based on a $1.56 per Euro conversion rate.
    b. Requesting additional information on the Italian application and attempt to get more detailed costs for Wildwood application was considered. However, our

expectation of success was low, so it was decided to focus our efforts in the information provided by Javeler Construction Company described next.

5. Javeler Construction Company (Toyo Pumps), our initial POC was Mr. Leslie Cross, however, following the initial e-mail, Mr. Richard Binning responded to the RFP. Toyo is a Japanese submersible pump, sold in the US via the Toyo Pumps office in Vancouver, BC, Canada (associated with Javeler Construction Company). Javeler Construction company has its main office in New Iberia, LA, and while the office in Vancouver, BC that helps to market Toyo Pumps, it is primarily a specialized construction company. Mr. Binning in the Vancouver, BC office provided considerable information that specifically addressed the RFI. The remainder of the report is based on information provided by Mr. Binning and developed by Mr. Clausner and modifications to the operating details developed through a series of e-mails between Messrs Clausner, Bogle and Gebert. Provided initially is the information developed by Mr. Clausner, followed by input from Mr. Binning.

CHL Calculations of Project Duration. To assist both the contractors and NAP, we calculated the time required to backpass 100K cy, 200K cy, 500K cy, and 1 M cy at average production rates of 400 cy/hr and 600 cy/hr. The calculation rates of 400 cy/hr and 600 cy/hr calculations were based on initial information from Mr. Binning as possible production rates for a 12 inch and 14 inch Toyo pump, respectively. Not knowing the specific operating schedule, calculations were made for 5, 6, and 7 day/week operations and daily work times of 8, 9, 10, 12, 16 and 24 hr. A subsequent set of calculations were made at a 300 cy/hr production rate. This was based on a March 18th e-mail from Mr. Binning where he indicated a production rate of 300 cy/hr was assumed as a long-term average production rate to include crane movement. Using this same logic, i.e., a 25% reduction in production in the 600 cy/hr rate for the larger Toyo pump to account for crane movement, a set of calculations using a production rate of 450 cy/hr was made. Tables 1and 2 provide the project durations based on the 300 and 450 cy/hr production rates, respectively. An Excel spreadsheet with the full set of production rate calculations is being provided to NAP along with this report.

Because the vast majority of the Javeler information is based on a 300 cy/hr production rate, most of the following discussions are based on Table 1. Also, the discussions focus on the assumptions made by Mr. Binning: a 5-day work week, 8 dredging hours per day, 22 working days per month (4.33 weeks/month) a 15,000 ft pumping distance. Of prime importance is the assumption is that the long term average production rate of 300 cy/hr is based on being able to mine sand from an elevation of +5 NAVD to -10 NAVD, or a vertical height of 15 ft. For the annual backpass rates of 100,000 and 200,000 cy/hr, the actual project durations, defined as the months required for pumping only, not including mob and demob, based on the 5 day, 8 hr/day are 1.9 months (100K cy) and 3.8 months (200 K cy). Based on the same pumping rate and schedule, backpassing 500K cy is probably marginal, 9.5 months, which is longer than the 8 month non-summer season). Backpassing 1.0M cy at the 300 cy/hr, 8 hr/day, 5 day/week would take almost 19 months, and is assumed to be unacceptable.

From Table 1, it can be seen that increasing the number of hours per day of pumping or the number of days/week pumping occurs, shortens the duration required, for example pumping 9 hours per day reduces the pumping time to 3.4 months (200K cy) and 1.7 months (100K cy).

To backpass 500K or 1M cy in less than 8 months would require higher production rates. Examining Table 2 shows 500K cy could be backpassed in 6.3 months at the 8-hr/day, 5 day/wk work schedule and 1.0 M cy could be backpassed in 7 months on a 12-hr/day, 6 day per week schedule. These larger backpass volume requirements might be more logically treated as a more conventional beach nourishment job, i.e., a 24-hr/day, 7-day/wk schedule. In that case, the pumping duration would reduce to 1.5 months for the 500K cy and 3.0 months for the 1.0M cy. The tables also provide information on pumping duration based on other schedules, e.g., 12-hr/day and 16-hr/day which might be better received by the residents.

CHL Calculations of Volumes Available for Different Beach Mining Swath Widths. To assist the contractors and NAP, we calculated volumes of sand available per unit width of beach and used that information to calculate the volumes available for a given swath width. A swath is defined as the width of the beach that can be mined without moving the submersible pump deployment device. For example a crane with a 100 ft boom, would have a swath width of approximately 200 ft.

The vertical datum on the cross section (Figure 4) is NAVD88. At this location in New Jersey, the Mean Lower Low Water (MLLW) datum is approximately 3.0 ft below NAVD88. NAP indicated they would like to limit the sand mining depth to about -3.0 ft MLLW (-6.0 ft NAVD88) to minimize offshore beach impacts. Based on this assumption, slopes of the various profiles were measured, including the max cut profile (Black line), minimum cut profile (Red line), and original profile (Blue Line), both on the “dry beach” assumed to be between +5 ft NAVD88 (+8 ft MLLW) and 0.0 ft NAVD88 (+3 ft MLLW); and on the “wet beach” 0.0 ft NAVD88 (+3 ft MLLW) and -6.0 ft NAVD88 (-3.0 ft MLLW). The somewhat arbitrary division between the “dry beach” and “wet beach” was based on the change in the slope at the 0.0 ft elevation NAVD88. The average distances between the lines were also measured and converted to feet, with the results shown in Table 3.

The dry beach widths listed in the top of Table 3 are easy to visualize and were used in volume computations described below. The wet beach slopes may help contractors decide operating methods.

Using this information, the cross sectional areas of the dry and wet beaches were computed as shown in Table 4. For example, the cross sectional area of dry beach for the max cut is 1,500 ft^2. Per foot of beach width, that corresponds to a volume of 55.6 cy. To assist in computing how many cubic yards could be removed from a “single” positioning of the submersible pump, the volume available in 50 ft increments were computed from 50 to 250 ft for both the “maxim cut” and “minimum cut” for the dry beach and wet beach. For example, if a large crane is used to mine a 250 ft wide swath of beach (i.e., a crane with a boom length of 125 ft), the volume available is 22,000 cy from single locations. For the dry beach I assumed a rectangular area (plan view), while for the wet beach, I assumed a half circle in area to account for the swing radius of the crane.

Table 5 shows the length of beach required to provide a given volume, from 100K cy to 1M cy based on both the maximum and minimum cuts. Noteworthy is the length of beach required for mining to get 1M cy; over 10,000 ft for the max cut and almost 17,000 ft for the minimum cut. These required lengths may cause the District to consider increasing the maximum mining depth of –3.0 ft MLLW.

Table 6 provides he number of times the system would have to be moved to remove the volumes required at the minimum and maximum cut for a range of swath widths. This clearly shows the value of having a system that can remove sediment from a wide swath width. For example using the minimum cut at a 50 ft swath width to move the larger beach nourishment volume of 1M cy would require 370 moves, while a 250 ft swath width would required only 74 moves. For the smaller, annual backpassing requirements, the narrow swath widths are not as much of a burden. For example to transfer 100K cy at a 50 ft swath width for the maximum cut requires the system to be repositioned 22 times, while at the 250 ft swath width and the maximum cut only requires 4 moves. Thus, for the renourishment, a system with a larger swath removal width would likely be required to keep costs low, while for the smaller annual backpassing, a less costly system based on a smaller swath width may be suitable.

Response to CHL REP from Richard Binning (Javeler Construction Company). The information below has been extracted from the response provided by Mr. Richard Binning on March 11th, 2009. The complete original response can be found in Appendix D. Mr. Binning stated in his introduction that

> " Based on your description of the North Wildwood application, we can meet your target price of $ 10 per cubic yard. There are a lot of details to go over, but conceptually, this is achievable. Javeler has the capability to do the work as a contractor or to provide equipment and technical support. "

Mr. Binning provided information for two options, the first option, on which his more detailed cost estimates were based, was for a Javeler 12 inch Electric Submersible Mobile Dredging System. The second option was based on a Javeler 14 inch Submersible Mobile Dredging System.

Option 1 – 12 Inch System. The 12-inch electric system is capable of transferring 3,000 – 4,000 gpm of slurry at a maximum distance of 15,000 ft. Sand production of up to 400 cubic yards per hour of material is possible with this system. The prime mover in this system is a Toyo 150 hp electric submersible agitator dredge pump which would be suspended from a crane with 120 – 160 feet of boom. Power would be from a 300kW generator to be mounted on the back of the crane, fuel usage is estimated at18 gal/hr. The unit is capable of transferring sand at up to 50 % solids by weight. Output from the pump is through a 10-inch gum lined dredge hose to 12 inch HPDE pipe on the beach. Diesel engine powered, skid mounted boosters would be placed every 5,000 feet to transfer the sand slurry to the south inlet location. Horsepower requirement for each booster is 400 hp.

Responding to a subsequent e-mail from Mr. Clausner, Mr. Binning provided the following assumptions on which the $10/ per cubic yard estimate was based.

1. The Javeler proposal at $10/yd was based on dredging from +5 to -10 NAVD88.
2. Javeler assumed 100,000 to 200,000 cubic yards of sand to be moved.
3. The cost estimate was based on:
    a. Single shift (8 dredging hours per day); 5 days/wk; 22 days per month.
    b. $ 4/gallon diesel fuel.
    c. Included beach grading on discharge side.
    d. 15,000 feet pumping distance with maximum 15' vertical lift.
4. The cost estimate was based on utilizing the most standard, readily available submersible dredging system that Javeler has, the 150 hp Toyo electric submersible dredge pump through 12 inch HDPE pipe with boosters every 5,000 ft.
5. Production was based on an average production of 300 cy/hr, although at times, the system will be pumping in excess of 400 cy/hr. Time to relocating the crane is included in the average production rate.

Option 2 - 14 inch System. The 14 inch hydraulic system is capable of transferring 5,000 – 6,000 gpm of slurry at a maximum distance of 15,000 ft. The maximum sand production rate of the system is 600 cy/hr. The system consists of a Toyo 12-inch hydraulic driven submersible agitator dredge pump suspended from a crane with 120 – 160 feet of boom, or mount directly to a large excavator and use the hydraulics from the rig to run the pump. An external hydraulic power unit (350 hp), if required, would be mounted on the back of the crane. The system will pump a sand slurry up to 50 % solids by weight through a 12 inch gum tube lined dredge hose to 14 inch HPDE pipe on the beach. Diesel engine powered, skid mounted booster pumps with marine gear drives, will be placed every 5,000 feet to transfer the sand slurry to the south inlet location. The horsepower requirement for each booster is 500 hp.

Manpower required for the backpassing system is a function of pumping distance. Javeler likes to have one additional man per booster station (every 5,000 ft). The base system with 5,000 ft of line requires 3 people to operate it (Javeler is non union).

Mr. Binning noted that the deeper the "cut" the more efficient the submersible agitator dredging system is. Also allowing a cut to -10 ft NAVD ensures an adequate water supply to the submersible and uninterrupted dredging operations. Excavation in the intertidal zone should ensure quick sand replacement. The simplest, most effective system is to hang the submersible from a crane.

BullDozer Production and Cost Information. This is not an area of our expertise, however, between the internet and other sources we compiled the following information that may prove useful in developing backpass options that use land based equipment. An internet search found Figure 5, maximum production rates for a range of Caterpillar bulldozers for a range of haul distances. This information came from the Food and Agriculture Organization of the United Nations (1998). The chapter provided factors that can be used to compute production under a range of materials, slopes and conditions. Not having any background in this area, we choose not to attempt to modify these production rates. However, it provides a starting point. For example a D8 dozer's maximum production rate with loose material is about 400 cu m/hr (520 cy/hr) for a 50 meter (164 ft) average dozing distance. This is approximately equal to the production rate of the submersible pumps and thus appears to be a reasonable option to "feed" the submersible pump in a backpassing situation.

We also developed some limited cost data that may be useful in evaluating options that make greater use of land based equipment. Mr. Brian Peterson of St. Paul District Fountain City Area Office (personal communication), estimated a D-7 dozer uses 100 gallons of diesel fuel in a 10-hr day of moving sand. Assuming fuel consumption is roughly comparable to hp, and a D-7 bulldozer hp is 240, while a D8 bulldozer hp is 310, thus fuel consumption of a D-8 bulldozer is 130 gallons in a 10 hr day or 13 gallons per hour. Mr. William Welk (personal communication), cost estimator at NAP, provided a cost estimate for a D-8 Bulldozer. The estimate, based on an 8-hr day at fuel at about $3.10 per gallon in bulk, was $2,250 per day. Details of the cost estimate are attached (PDF file).

Hopefully, the above information could be used by a competent cost estimator or dredging contractor in producing cost estimates for bulldozer assistance in a hydraulic based mobile backpass system.

Chase (2006) Review Comments. Mr. Clausner reviewed the Chase (2006) paper in light of this study and has the following comments. The backpass system proposed by Chase is somewhat different than the system desired in this study as noted earlier. The backpass fuel costs, assumed to be for crane, submersible pump and booster pumps are based on a fuel cost of $2.00 per gallon. Increasing the fuel cost to $4.00 per gallon raises the estimated backpass costs by about 9%, increasing the cost for bypassing 200,000 cy from $7.05/cy to $7.65. Note, there was no attempt to update any of the other costs presented in the Chase paper. We assume that other fuel costs are associated with the nourished area grading, but were not broken out separately and thus the overall increase associated with the increasing cost of fuel could not be calculated. For future efforts on this study, persuading Mr. Chase to provide the details he used to develop his paper would be worthwhile.

Conclusions

Information provided by Javeler Construction Company concludes it is possible to backpass 200K cy of beach sand a distance of 15,000 ft over a period of about two to four months using a crawler crane to deploy a submersible pump at a cost of about $10/cy. This is based on the following assumptions: use of a rented crane with a 120 to 160 ft boom, mining pits down to – 10 ft NAVD88, and $4/gallon diesel fuel. Mr. Welp's experience with Javeler Construction Company has been positive, and Mr. Clausner's limited experience with Javeler has also been positive. The bottom line is that the information from Chase 2006, reinforced by the more recent and Wildwood project specific information from Javeler Construction Company leads us to believe that a mobile based hydraulic backpass system has good potential for meeting NAP's goal of approximately $10/ cy.

However, it must be noted that with the limited information we developed, we cannot conclude that the $10/cy unit price is possible with "surgical" or a limited depth beach (-3 ft MLLW) mining plan. Also, we were not able to get feedback from our contractors on backpassing costs that included the use of land based equipment, i.e., bulldozers and scrapers, to feed material to a hydraulic based system. Our intuition is that this option would be more expensive than the crane based system, but we cannot say for sure or how much more expensive. We were able to gather some data that may assist others in conducting that analysis.

Another question we were unable to answer is potential cost savings associated with a less expensive deployment method. We think the lower cost of the simpler deployment system may be more than offset by the smaller capture area with the simpler deployment system and large number of times the system would have to be moved. In fact, after additional discussions, we believe a crawler crane is probably by far the best method to effectively deploy the submersible pump.

In addition to examining the annual backpassing requirements, NAP requested unit cost estimates for a one time backpassing of 500K to 1 M cy. The duration to backpass 500K cy, 7.7 months working 10 hr days five days a week and 1 M cy, 8 months working 16-hr days, 6 days/wk, make use of the smaller system marginal at best. The larger system proposed by Javeler Construction Company appears capable of bypassing these larger amounts in a reasonable amount of time, i.e., easily less than one eight month beach off season, e.g., 500K cy in 6.4 months pumping 8-hr/day, 5 days/week, and 1 M cy in 7.1 months pumping 12-hr/days and 6days/wk. Working 24/7 makes these durations much shorter, 1.5 months for 500K cy and 3.1 months for 1M cy.

A major issue for the large one time beach nourishment volumes is the long length of beach required to provide sufficient volume if the shallower sand mining limit (-3 ft MLLW) is used. For example, to mine 500 K cy over the minimum cut width would require about 8,400 ft of beach and to mine 1 M cy over the minimum cut width would require almost 17,000 ft of beach. Even at the maximum cut width, the beach length required is over 5,000 ft for 500K cy and 10,000 ft for 1M cy.

The shallower mining depths will increase costs due to the frequency the system would have to be moved. Assuming no infilling occurs and a 100 ft wide swath width is being removed, the system would have to be moved 19 times if the minimum beach width cut is used. For a 500K or 1M cy backpassing, the number of movements required with a 100 ft wide swath range from 56 (500K cy, max cut width), to 185 (1M cy, min cut width). While repositioning a crane and using a flexible HDPE pipeline is not a major issue, these figures imply that without significant infilling of the craters during operation, these minimum swath widths would likely be impractical for the beach nourishment volume and significantly increase costs for the smaller annual volumes, particularly for a non-crane deployment option.

The 100 to 200K cy annual backpassing duration is not excessive, so "aesthetics" (i.e., a large crane on the beach) may be less of a problem." For a 500K to 1,000 K cy backpass operation, where the crane/dozers, etc., will be on the beach longer, this may become more of an issue.

However, using a larger, more robust system on a longer work schedule, perhaps even a 24/7 basis would reduce beach impacts by lessening the time required to complete the project.

Recommendations

First get good data on crater infilling rates. While sediment transport models can provide some estimates, the rate of filling of the craters at Indian River Inlet will provide the best information. If the rates of filling are sufficiently high, then allowing deeper mining depths may be justified. Also, as recommended by Chase (2006), a bucket could be mounted on the crane and the crater refilled to some degree to reduce safety issues. This would, obviously, add to costs and durations.

Perhaps the best recommendation would be to provide a modest sum to an experienced contactor, e.g., Javeler, and have them do several more detailed cost estimates with various options; submersible pumps deployed with a crane, other deployment options, e.g., using land based equipment for a large portion of the sub aerial sand removal.

Another issue to consider when developing options would be to examine purchasing the majority of the HDPE pipeline as opposed to renting it. The last 5,000 to 8,000 ft could remain buried on the North Wildwood beach once a sufficient beach width is established.

Consider a small demonstration project, by identifying an application with a current need for backpassing some modest amount of sedments. Javeler Construction Company has indicated they could mobilize a 12 inch system to pump up to 5,000 feet within 2 weeks. They estimate the dredging costs would not exceed $10/yard, based on a minimum volume of 40,000 yards. Prior to this demonstration, the contractor should visit the Indian River Inlet Bypass plant to get information on present operating methods and lessons learned, e.g., the influence of waves and tide on crater infilling and production rates and advantages and disadvantage of working alongshore vs cross shore. This information would also be valuable for the contractor performing the more detailed cost estimates described above.

Finally, the recent increases in the price of diesel fuel will likely raise prices across the board. We believe the purely land based backpassing option, i.e., using scrapers, dozers, and dump trucks, would be more impacted than the hydraulic based backpassing options.

References

Chase, S. (2006). “Sand back-passing with land based equipment, a cost-effective approach for beach restoration,” Shore and Beach Vol 74, No 2, Spring 2006), pp 19-25.

Food and Agriculture Organization of the United Nations (1998). “Watershed Management Field Manual, FAO Conservation Guide 13/5, Chapter 6, Road Construction Techniques, page 6. http://www.fao.org/docrep/006/t0099e/T0099e06.htm

Peterson, B. (2008). Personal communication, telephone call with Mr. Tim Welp, March 24, 2008.

Stauble, D. (2008). Personal communication, e-mail dated March 11, 2008.

Welk, W. (2008). Personal communication, e-mail dated March 24, 2008.

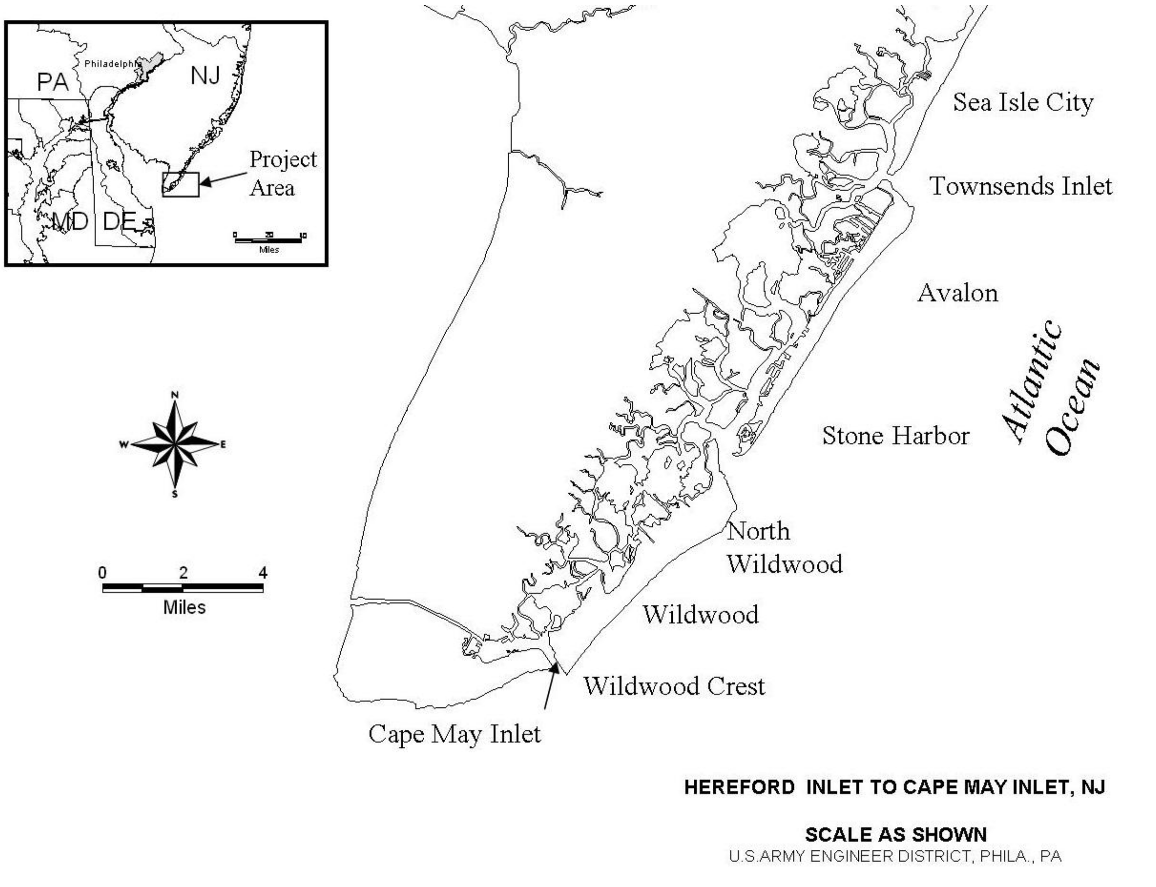

Figure 1. Location map.

Figure 2. Aerial photo of project area.

Figure 3. Crane deploying submersible pump from jetty

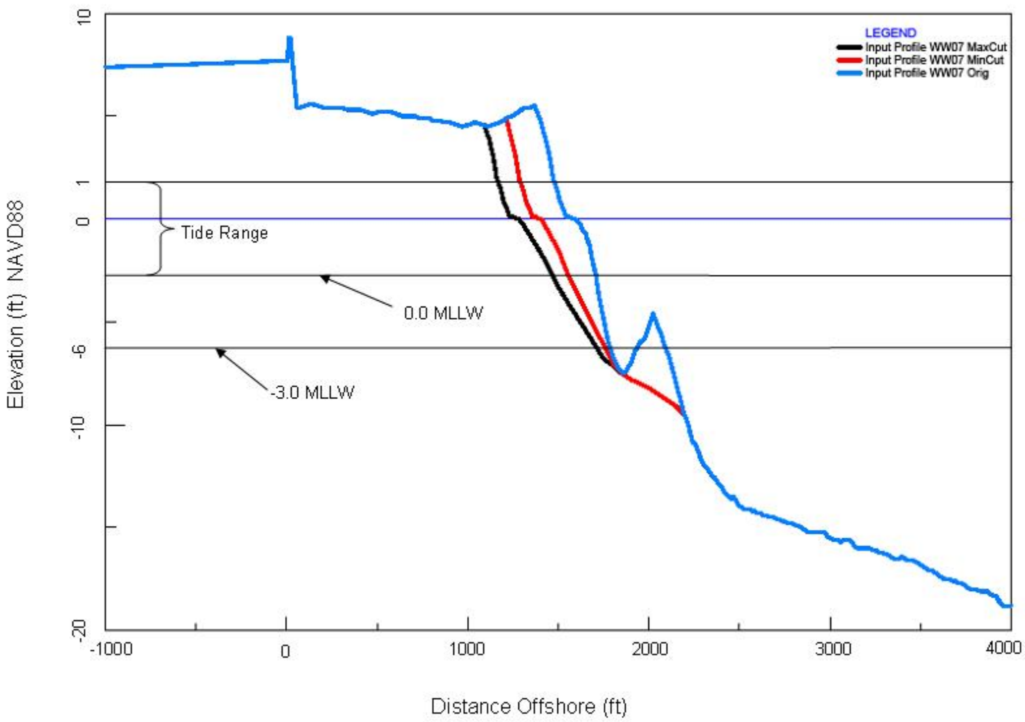

Figure 4. Beach profile at Wildwood showing proposed borrow location. Note this figure has an error, it shows the tide range as 6 ft. This will be corrected on Wednesday AM.

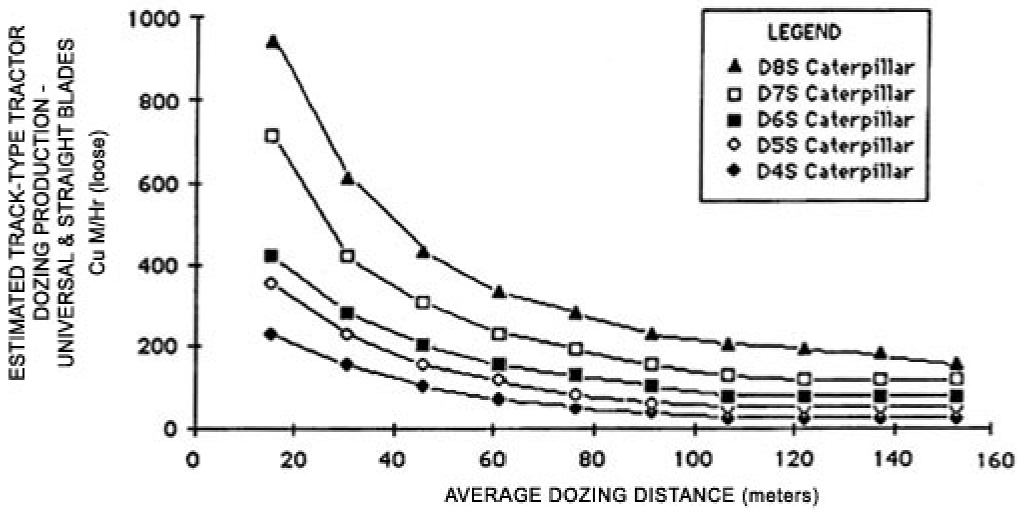

Figure 5. Maximum production rates for different bulldozers equipped with straight blades in relation to haul distance (from Caterpillar Handbook, 1984).

Table 1. Hereford Inlet - Wildwood Beach Backpassing Rates and Durations
Based on 300 cy/hr long term average production

Scenario Number	Volume Backpassed (cy)	Average production Rate cy/hr	Working hours per day	Avg Daily Production (cy)	Working days per week	Avg Weekly Production (cy)	Job Duration (months)
1	1,000,000	300	8	2,400	5	12,000	18.9
2	1,000,000	300	9	2,700	5	13,500	16.8
3	1,000,000	300	10	3,000	5	15,000	15.2
4	1,000,000	300	12	3,600	5	18,000	12.6
5	1,000,000	300	16	4,800	5	24,000	9.5
6	1,000,000	300	8	2,400	6	14,400	15.8
7	1,000,000	300	9	2,700	6	16,200	14.0
8	1,000,000	300	10	3,000	6	18,000	12.6
9	1,000,000	300	12	3,600	6	21,600	10.5
10	1,000,000	300	16	4,800	6	28,800	7.9
11	1,000,000	300	24	7,200	7	50,400	4.5
12	500,000	300	8	2,400	5	12,000	9.5
13	500,000	300	9	2,700	5	13,500	8.4
14	500,000	300	10	3,000	5	15,000	7.6
15	500,000	300	12	3,600	5	18,000	6.3
16	500,000	300	16	4,800	5	24,000	4.7
17	500,000	300	8	2,400	6	14,400	7.9
18	500,000	300	9	2,700	6	16,200	7.0
19	500,000	300	10	3,000	6	18,000	6.3
20	500,000	300	12	3,600	6	21,600	5.3
21	500,000	300	16	4,800	6	28,800	3.9
22	500,000	300	24	7,200	7	50,400	2.3
23	200,000	300	8	2,400	5	12,000	3.8
24	200,000	300	9	2,700	5	13,500	3.4
25	200,000	300	10	3,000	5	15,000	3.0
26	200,000	300	12	3,600	5	18,000	2.5
27	200,000	300	16	4,800	5	24,000	1.9
28	200,000	300	8	2,400	6	14,400	3.2
29	200,000	300	9	2,700	6	16,200	2.8
30	200,000	300	10	3,000	6	18,000	2.5
31	200,000	300	12	3,600	6	21,600	2.1
32	200,000	300	16	4,800	6	28,800	1.6
33	200,000	300	24	7,200	7	50,400	0.9

Scenario Number	Volume Backpassed (cy)	Average production Rate cy/hr	Working hours per day	Avg Daily Production (cy)	Working days per week	Avg Weekly Production (cy)	Job Duration (months)
34	100,000	300	8	2,400	5	12,000	1.9
35	100,000	300	9	2,700	5	13,500	1.7
36	100,000	300	10	3,000	5	15,000	1.5
37	100,000	300	12	3,600	5	18,000	1.3
38	100,000	300	16	4,800	5	24,000	1.0
39	100,000	300	8	2,400	6	14,400	1.6
40	100,000	300	9	2,700	6	16,200	1.4
41	100,000	300	10	3,000	6	18,000	1.3
42	100,000	300	12	3,600	6	21,600	1.1
43	100,000	300	16	4,800	6	28,800	0.8
44	100,000	300	24	7,200	7	50,400	0.5

Table 2. Hereford Inlet - Wildwood Beach Backpassing Rates and Durations							
Based on 450 cy/hr long term average production							
Scenario Number	Volume Backpassed (cy)	Average production Rate cy/hr	Working hours per day	Avg Daily Production (cy)	Working days per week	Avg Weekly Production (cy)	Job Duration (months)
1	1,000,000	450	8	3,600	5	18,000	12.6
2	1,000,000	450	9	4,050	5	20,250	11.2
3	1,000,000	450	10	4,500	5	22,500	10.1
4	1,000,000	450	12	5,400	5	27,000	8.4
5	1,000,000	450	16	7,200	5	36,000	6.3
6	1,000,000	450	8	3,600	6	21,600	10.5
7	1,000,000	450	9	4,050	6	24,300	9.4
8	1,000,000	450	10	4,500	6	27,000	8.4
9	1,000,000	450	12	5,400	6	32,400	7.0
10	1,000,000	450	16	7,200	6	43,200	5.3
11	1,000,000	450	24	10,800	7	75,600	3.0
12	500,000	450	8	3,600	5	18,000	6.3
13	500,000	450	9	4,050	5	20,250	5.6
14	500,000	450	10	4,500	5	22,500	5.1

15	500,000	450	12	5,400	5	27,000	4.2
16	500,000	450	16	7,200	5	36,000	3.2
Scenario Number	Volume Backpassed (cy)	Average production Rate cy/hr	Working hours per day	Avg Daily Production (cy)	Working days per week	Avg Weekly Production (cy)	Job Duration (months)
17	500,000	450	8	3,600	6	21,600	5.3
18	500,000	450	9	4,050	6	24,300	4.7
19	500,000	450	10	4,500	6	27,000	4.2
20	500,000	450	12	5,400	6	32,400	3.5
21	500,000	450	16	7,200	6	43,200	2.6
22	500,000	450	24	10,800	7	75,600	1.5
23	200,000	450	8	3,600	5	18,000	2.5
24	200,000	450	9	4,050	5	20,250	2.2
25	200,000	450	10	4,500	5	22,500	2.0
26	200,000	450	12	5,400	5	27,000	1.7
27	200,000	450	16	7,200	5	36,000	1.3
28	200,000	450	8	3,600	6	21,600	2.1
29	200,000	450	9	4,050	6	24,300	1.9
30	200,000	450	10	4,500	6	27,000	1.7
31	200,000	450	12	5,400	6	32,400	1.4
32	200,000	450	16	7,200	6	43,200	1.1
33	200,000	450	24	10,800	7	75,600	0.6
34	100,000	450	8	3,600	5	18,000	1.3
35	100,000	450	9	4,050	5	20,250	1.1
36	100,000	450	10	4,500	5	22,500	1.0
37	100,000	450	12	5,400	5	27,000	0.8
38	100,000	450	16	7,200	5	36,000	0.6
39	100,000	450	8	3,600	6	21,600	1.1
40	100,000	450	9	4,050	6	24,300	0.9
41	100,000	450	10	4,500	6	27,000	0.8
42	100,000	450	12	5,400	6	32,400	0.7
43	100,000	450	16	7,200	6	43,200	0.5
44	100,000	450	24	10,800	7	75,600	0.3

Table 3. Beach widths used to compute backpass volumes				
Beach Segment	Elevation	Line Color – Average Slope	Max Cut Width	Min Cut Width
Dry Beach	+5.0 ft NAVD 88 (+8.0 ft MLLW) to 0.0 ft NAVD 88 (+3.0 ft MLLW)	All colors 1V:25H	300 ft	185 ft
Wet Beach	0.0 NAVD88 (+3 ft MLLW) to -6.0 ft NAVD 88 (-3.0 ft MLLW)			
		Blue Line - original 1V:26 H		
		Black Line – max cut 1V:69H		
		Red line – min cut 1V:54H		

Table 4. Hereford Inlet - Wildwood Beach Sand Volumes Available for Different Mining Depths										
Cross Sectional Areas										
"Dry Beach - +5 NAVD88 (+8 ft MLLW) to 0.0 NAVD88 (+3 ft MLLW)										
		Width (ft)	Height (ft)	Area (sq ft)	Volume (cy per ft/beach)	Volume per 50 ft (cy)	Volume per 100 ft (cy)	Volume per 150 ft (cy)	Volume per 200 ft (cy)	Volume per 250 ft width (cy)
Max Cut - Blue to Black Line		300	5	1,500	56	2,800	5,600	8,300	11,000	14,000
Min Cut – Red to Black Line		185	5	925	34	1,700	3,400	5,100	6,900	8,600
"Wet Beach - 0.0 NAVD88 (+3 ft MLLW) to - 6 NAVD88 (-3 ft MLLW)										
Assume a trapezoidal shape	Top Width (ft)	Bottom Width (ft)	Height (ft)							
Max Cut - Blue to Black Line	300	83	6	1140	42.6	1,700	3,300	5,000	6,700	8,400
Min Cut – Red to Black Line	185	41.4	6	679	25.2	990	2000	3,000	3,900	4,900
			Total Volume Available for wet and dry beach							
					Max Cut	4,400	8,900	13,000	18,000	22,000
					Min Cut	2,700	5,400	8,100	11,000	14,000

Table 5. Total Volume Available/ft of beach width						
			Length of beach (ft) needed to remove			
		Volume (CY per ft/beach)	100,000 cy	200,000 cy	500,000 cy	1,000,000 cy
	Max Cut - Blue to Black Line	98	1,000	2,000	5,100	10,000
	Min Cut - Red to Black Line	59	1700	3,400	8,400	17,000

Table 6. Number of deployment system movements required to move a given volume of sand

			100,000 cy	200,000 cy	500,000 cy	1,000,000 cy
Movements for a 50 ft swath		Max	22	45	112	225
Movements for a 50 ft swath		Min	37	74	185	370
Movements for a 100 ft swath		Max	11	22	56	112
Movements for a 100 ft swath		Min	19	37	93	185
Movements for a 150 ft swath		Max	7	15	37	75
Movements for a 150 ft swath		Min	12	25	62	123
Movements for a 200 ft swath		Max	6	11	28	56
Movements for a 200 ft swath		Min	9	19	46	93
Movements for a 250 ft swath		Max	4	9	22	45
Movements for a 250 ft swath		Min	7	15	37	74

Appendix A
Contractor Contact Information

1. Heger Pumps. POC - Mr. Siegfried Heger, (562)-989-5432, Sheger@hegerpumps.com
http://www.dragflow.com/Default.htm

2. Hagler Pumps. POC – Mr. Robert Hagler and Ms. Laurie Nalley, 803-278-2728,
bobhagler@haglersystems.com , laurieNalley@haglersystems.com
http://www.haglersystems.com/about.htm

3. Javeler Construction Company. POC – Mr. Richard Binning, 604-929-9543,
rbinning@telus.net
http://www.toyopumps.com/pumps/submersibles/submersiblelist.html

4. Standard Gravel Company, POC – Mr. Spencer Green, (985) 839-3442,
sgreen@genflopumps.com http://www.genflopumps.com/

5. Damen Dredging Equipment, POC – Mr. David Tenwolde, +31(0)33 247 40 40
dt@damendredging.com http://damendredging.com/html/en/dop.htm

Appendix B
Request for Information Provided to Contractors

Request for Conceptual Mobile Hydraulic Backpassing System for the Hereford to Cape May Feasibility Study

Background: The beach immediate south of Hereford Inlet, North Wildwood, NJ, is experiencing erosion (Figure 1). As part of the solution to this erosion problem, the US Army Corps of Engineers Philadelphia District (NAP) is investigating the various options to that would economically backpass sand from the wide beach to the south, Wildwood, NJ, to North Wildwood (Figure 2).

The NAP estimates between 100,000 and 200,000 cy/yr need to be bypassed on an annual basis. However, because of the existing deficit on the North Wildwood beach, a one time "beach nourishment" effort of 500,000 to 1,000,000 cy is needed initially.

Need/Requirements: A proven option for bypassing/backpassing modest volumes is to use conventional earthmoving equipment to collect the sand and then trucking the sand from one location to another. This has been done at Avalon, NJ, twice in the past five years and is being considered at Wildwood. However, the cost of this option, estimated a $20/cy by NAP for the Wildwood application, is considered to be too expensive, it also significantly interferes with beach use. NAP is interested in looking at mobile hydraulic pumps to entrain the sand at Wildwood in conjunction with booster pumps to transfer the sand to North Wildwood with the desire to reduce costs, ideally to less than $10 per cubic yard. The attached paper by Stuart Chase (Shore and Beach Vol 74, No 2, Spring 2006), describes a system based on a submersible pump deployed from a crane that meets many of the NAP's requirements. However, the sheet pile wall specified as part of the system, would add to the cost and create a feature that would have to be removed and moved periodically.

The NAP desires a more mobile system with minimal infrastructure support and impact to beach users. Also, rather than digging series of relatively deep pits, NAP would prefer to remove sand in a thin layer over a wide area with excavation pits no deeper than 10 ft below mlw. A typical cross section of the Wildwood beach is shown in Figure 4. NAP desires to remove the material between the blue and black lines between +5 and -10 (the outermost lines in case the colors are not visible), a horizontal distance of between 250 and 350 ft. Also, to reduce costs they would like to avoid using a crane to deploy the submersible pump (or eductor), instead using less expensive system such as an A-frame or tripod to deploy the submersible pump. Another option is to use mechanical equipment, scrapers and dozers to stockpile sand and then use hydraulic pumps to transfer the sand to North Wildwood.

The sand to be backpassed is 0.15 mm. A challenge will be the distance to bypass the sand, between 5,000 and 15,000 ft, thus booster pumps will be required. Tide range at the site is 4 ft.

Ideally the system would transfer sand outside of the summer tourist season when the beach is most heavily used, thus the material should be transferred from 15 Sep to 15 May.

Additional Needs for Portable, Mobile Hydraulic Backpass Systems. As noted, this information is for a feasibility study, and thus the information provided is not a commitment to you for actual work. The actual work for the Wildwood to North Wildwood backpassing is probably several years away. However, the need to backpass sediments from wider beaches to erosion "hot spots" is a problem at many locations around the US. While typical beach renourishment intervals are 5 to 10 years, in many locations erosion "hot spots" develop quickly (within months to a couple of years) that require from a few 10K cy to 100 K cy with distances on the order of a few thousand feet or more. In New Jersey alone, NAP estimates there are currently several projects, e.g., Avalon, Atlantic City, Brigantine, and Ocean City, that could make use of a low-cost hydraulic based system now. Obviously, for the smaller volume projects, a very mobile system that could be easily moved to another site would be advantageous. **Hopefully the potential to quickly turn a good conceptual system into a functioning system will motivate you to devote some serious effort to this request.**

Deliverables: This is a request for you to describe a conceptual system/systems that can meet the requirements stated above. Please include the following.

1. A description of the system components including specific pump(s), hp, discharge line sizes, power sources, etc.
2. A description how the system would be operated.
3. Manpower requirements and skills for the operators (ideally the system or most if it could be operated by city workers)
4. Estimated costs and durations (please state assumed schedule, i.e., 5 days/per week, 8 hours/day, etc.) to backpass annual volumes of 100K cy, 200k cy, 500K cy, and 1,000,00 cy. The cost estimate should be based on an assumed cost for diesel fuel of $4.00/gallon. The cost estimate should have sufficient information so the cost estimate can be updated based on changes in the cost of diesel fuel.
5. Please note any uncertainties or additional information that would be needed for improving the accuracy of your proposal.
6. Typical "beach nourishment" grading of the sand placed in North Wildwood will be required, i.e., a specific berm width, height, and foreshore slope.
7. We realize that some systems may not be able to fully meet every requirement. However, please feel free to submit a system that meets most the requirements and note the limitations.
8. As noted above, if time permits, you may want to provide information for a second, smaller, and more portable system.
9. We hope to have cost and performance data on the land based backpassing at Avalon, NJ, in a day or two.

Deadline: Final input is desired by COB on 13 March, and no later than COB 14 March. Ideally this will be in the form of MS Word document.

Follow-on Questions. I expect in most cases, you will have additional questions. I, James Clausner (US Army Engineer Research and Development Center, Vicksburg, MS), and NAP staff will be available for follow-up phone calls on Thursday and Friday, March 6 and 7, and Monday, March 10th, to answer additional questions. Please contact me to set up a time.

POC:

James E. Clausner, PE
Associate Technical Director for Navigation,
Coastal and Hydraulics Laboratory,
CEERD-HV-T
3909 Halls Ferry Rd
Vicksburg, MS 39180, 601-634-2009
james.e.clausner@usace.army.mil

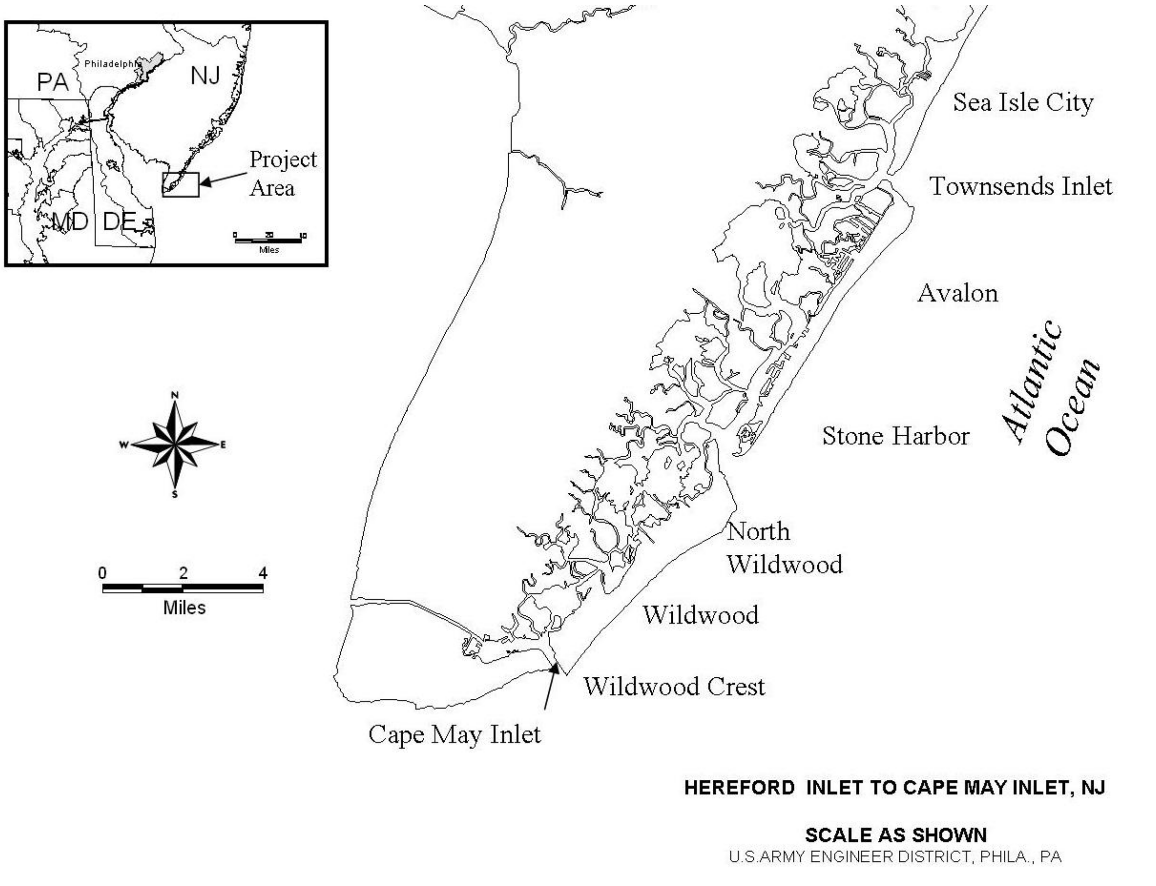

Figure 1. Location map.

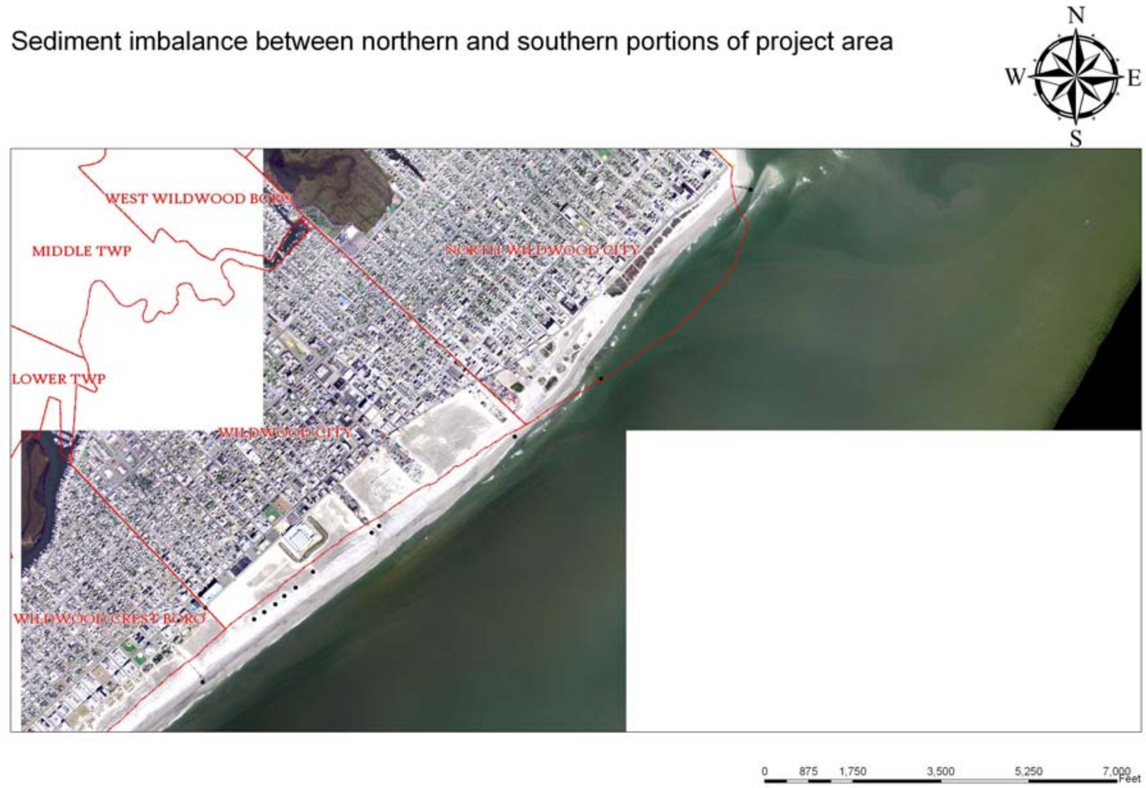

Figure 2. Aerial photo of project area.

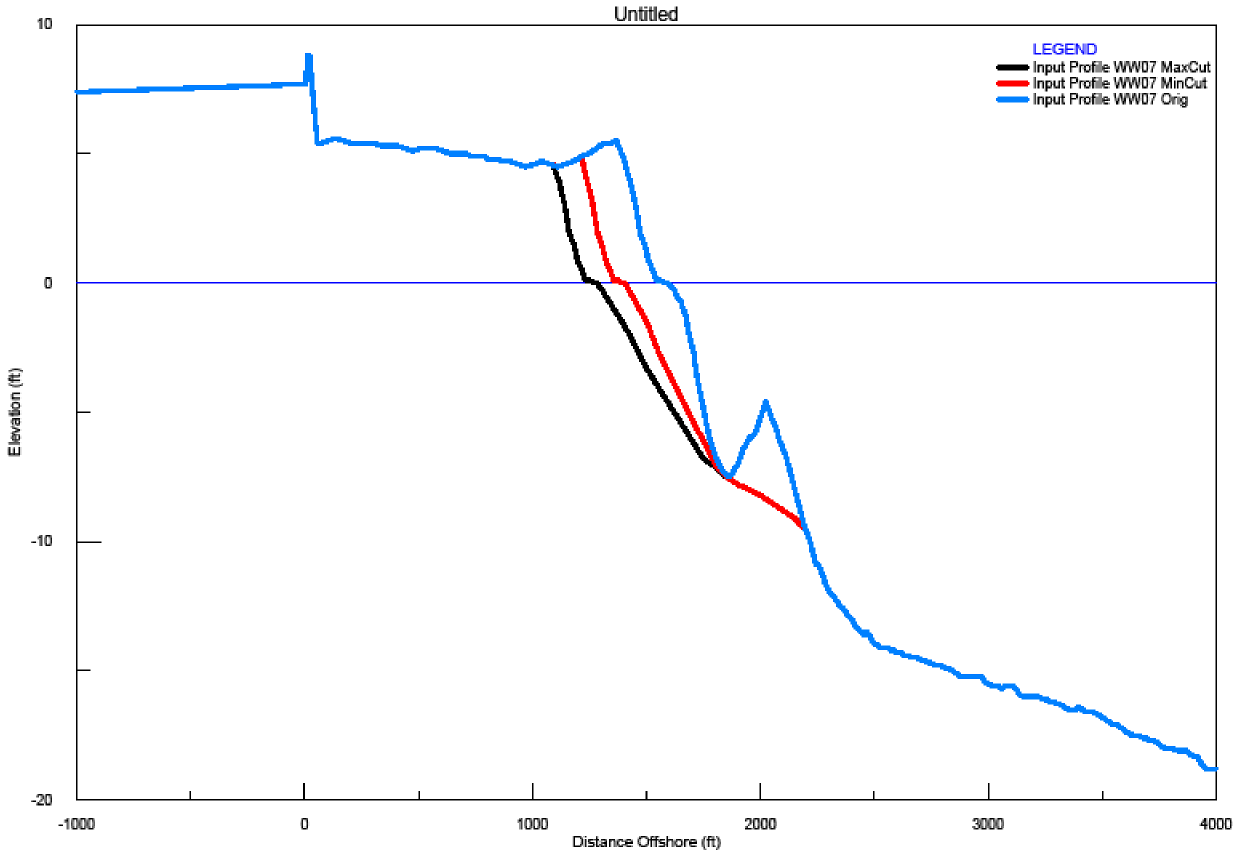

Figure 4. Beach profile at Wildwood showing proposed borrow location

Appendix C
Information Provided by Damen Dredging Equipment on DOP Submersible Pumps

Project description Sand Backpass Italy

Project location : East coast of Italy between Venice and Ravenna
Year : 2003/2004
Equipment : 2 * DOP 2320 + Diesel driven Booster BS 250 + Instrumentation
Pumping distance : 3000 – 18000 ft
Pipe Diameter : 300 mm ID (12 inch)

General:

As the DOP pumps are designed for pumping high densities and long distances, it is the ideal tool for this kind of operations. A relatively low investments and it's multipurpose use (excavator, pump and power pack -> the power pack can be a Pileco/Ice Europe Powerpack suitable for vibratory hammers, can all be used separated from each other) make it a versatile tool for USACE and their contractors.

More detailed technical information can be downloaded from our website WWW.damendredging.com

Loading:

At this location truck's where loaded with an hydraulic excavator for logistic purposes.
The first DOP 2320 pump was mounted on a CAT 320 excavator and a fixed position was created using Sheet piles in order to prevent sliding of the excavator in the Dredged pit. The DOP pump was fitted with a sand production head with jetwater nozzles. A separate pump provided Jetwater. A separate, cabin controlled Diesel-Hydraulic Power pack provides the power for the DOP pump drive.

The operator has a cross needle indicator and operational instrumentation in his cabin for an optimal and safe production process. Client fitted instrumentation can be more professional with steel supports etc.

The pump is connected to a flexible discharge hose with floats and connected to a bud-welded HDPE pipe with an internal diameter of 300 mm.

Production measurement & registration (PMR unit) :

The HDPE pipe feeds into a production measurement and control unit that feeds a signal of Velocity and Density into the operators cabin and finally to the Cross Needle yield indicator unit. All data is stored on a PC and registers Velocity, Density and Production (dry material). This data can be used for payment purposes but also for analyses of the project.

PMR unit with protective cover.

Internals of PMR unit

Boosters:

A second DOP 2320 unit is located on the beach and used as booster and/or back-up unit for the excavator mounted DOP. Pressure and Vacuum signals are fed to the operators cabin in order to follow and control the long distance pumping process. A second (diesel driven) Booster is located further down the beach and is also remote operated from the operators cabin. The diesel driven booster is equipped with a mechanical seal and therefore no additional water supply systems are required, and make the operation very flexible and not labor intensive during repositioning of the equipment.

Booster BS 250 & PMR unit during test in Holland

Result:

Production estimates for Hereford Inlet:

The **estimated production** for the **maximum distance** results in an average production of **400 m3/hr** without downtime and pipe/equipment shifts taken into account. This requires a careful and dedicated excavator operator who has basic knowledge about pumping liquefied slurries. Qualified general mechanics and operators can be involved for general assistance and maintenance.

These production estimates are based on 300 mm ID pipes and pump revolutions of approx. 1000 RPM (which is higher then the usual 850 RPM), Influence of wear will be lower due to low velocities, the higher pump revolutions will increase wear however this is partly compensated by the fine grains of the material to be pumped. The velocities are close to the critical velocity for this kind of material.
If the contractor desires to pump at higher velocities, an extra booster will be required.

Appendix D
Information Provided by Javeler Construction Company

Jim Clausner
US Army Engineer Research Center
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Vicksburg, MS 39180
Tel: 601.634.2009
james.e.clausner@us.army.mil

March 12, 2008

Subject: Javeler Mobile Sand Backpassing Dredging System

Dear Jim:

With regards to your March 5 email about mobile sand backpassing systems; specifically, the beach erosion problem at North Wildwood, NJ, please find enclosed our response. Javeler is very interested in this application. As one of the most experienced mobile sand dredging contractors/equipment providers in the country, we look forward to providing assistance.

Based on your description of the North Wildwood application, we can meet your target price of $ 10 per cubic yard. There are a lot of details to go over, but conceptually, this is achievable.

Javeler has the capability to do the work as a contractor or to provide equipment and technical support.

The Toyo submersible dredge pumps have a reputation as the most rugged solids handling submersible pumps available. On the first job we used the Toyo in 1983, we pumped over 500,000 cubic yards of sand with zero unscheduled down time. We simply mounted the Toyo submersible from a dragline. The simplicity of this dredging system made it very easy to operate and the reliability of the equipment eliminated down time.

Various size submersible dredge pump systems are available – both hydraulic and electric drive. I have provided both an electric 12" and a hydraulic 14" option below. The electric 12" pump system option is more readily available. Additional capital costs are required to set up for the 14" hydraulic system.

<u>Option 1 - Javeler 12 inch Electric Submersible Mobile Dredging System</u>

The 12 inch electric system is capable of transferring 3,000 – 4,00 gpm of slurry at a maximum distance of 15,000'. Sand production up to 400 cubic yards per hour of material.

Suspend a Toyo 150 hp electric submersible agitator dredge pump from a crane with 120 – 160 feet of boom. Generator (300 kw), to be mounted on the back of the crane. Sand up to 50 % solids by weight, to be transferred through 10 inch gum tube lined dredge hose to 12 inch HPDE pipe on the beach.

Diesel engine, skid mounted boosters will be placed every 5,000 feet to transfer the sand slurry to the south inlet location. Horsepower requirement for each booster is 400 hp.

Javeler Electric Submersible System Components

- Toyo 150 hp submersible agitator pump and will produce 3,000 – 4,000 gpm at 64 – 74 feet of head. The pump weighs 8,000 lbs and will pass a 4.7 inch rock. The motor is 460 volt/3 phase/60hz and has 150' of cable. Nema starter included.
- One hundred twenty feet of 10 inch heavy duty, 3/8 inch gum tube lined, dredge hose. The 150 psi rated hose is in 40 foot lengths, and has 150 # flange connections.
- 300 kw sound attenuated diesel engine driven generator (fuel usage estimated at 18 gal/hr).
- GIW (or equal) horizontal booster pumps; diesel engine 400 hp; marine gear drives; skid mounted. Gland water from portable seal water tanks.
- 12 inch SDR 17 HDPE pipe welded together; flange connections every 400' for access.
- magnetic flow meter to provide flow rates and production numbers

Option 2 - Javeler 14 inch Submersible Mobile Dredging System

The 14 inch hydraulic system is capable of transferring 5,000 – 6,00 gpm of slurry at a maximum distance of 15,000'. Sand production up to 600 cubic yards per hour of material.

Suspend a Toyo 12 inch hydraulic driven submersible agitator dredge pump from a crane with 120 – 160 feet of boom, or mount directly to a large excavator and use the hydraulics from the rig to run the pump. Hydraulic power unit (350 hp), if required, to be mounted on the back of the crane. Sand up to 50 % solids by weight, to be transferred through 12 inch gum tube lined dredge hose to 14 inch HPDE pipe on the beach.
Diesel engine, skid mounted boosters with marine gear drives, will be placed every 5,000 feet to transfer the sand slurry to the south inlet location. Horsepower requirement for each booster is 500 hp.

Javeler Hydraulic Pump System Components

- Toyo TO 160B submersible agitator pump, has variable speed capability up to 850 rpm and will produce 5,000 – 6,000 gpm at 100 – 115 feet of head. The pump weighs 8,000 lbs and will pass a 4.7 inch rock. The TO 160B has a Rexroth 500 hydraulic motor and 200 feet of hydraulic lines.

- One hundred twenty feet of 12 inch heavy duty, 3/8 inch gum tube lined, dredge hose. The 150 psi rated hose is in 40 foot lengths, and has 150 # flange connections.

- Hydraulic power unit with Cat Tier 3 engine; Chevron Clarity biodegradable hydraulic oil, electronic monitoring warns of engine trouble, Rexroth piston motor.

- GIW (or equal) horizontal booster pumps; diesel engine 500 hp; marine gear drives; skid mounted. Gland water from portable seal water tanks.

- 14 inch SDR 11 HDPE pipe welded together; flange connections every 400' for access.

- magnetic flow meter to provide flow rates and production numbers

Manpower required for the backpassing system is a function of pumping distance. We like to have one additional man per booster station (every 5,000 feet). Base system with 5,000 feet of line requires 3 people to operate it (Javeler is non union).

We look forward to working with the Corps of Engineers on this and other sand backpassing projects. You can reach me at 604-929-9543 or Les Cross at our Louisiana office at 337-364-5841.

Regards
Javeler Construction Co., Inc.

Richard Binning

INSTRUCTION REPORT HL-81-1

A GUIDE TO THE PLANNING AND HYDRAULIC DESIGN OF JET PUMP REMEDIAL SAND BYPASSING SYSTEMS

by

Thomas W. Richardson, Ernest C. McNair, Jr.

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September 1981

Final Report

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Hydraulic machinery
Jet pumps
Sand

20. ABSTRACT *(Continue on reverse side if necessary and identify by block number)*

This report is intended as an aid to engineers studying alternative solutions to a coastal sand bypassing problem. Using this report, engineers already familiar with coastal processes and centrifugal pumping systems will be able to: (a) determine the feasibility of a jet pump remedial bypassing system at a given site, (b) develop initial layouts for such a system, and (c) perform the system basic hydraulic design. A set of considerations and

(Continued)

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20. ABSTRACT (Continued).

guidelines relative to feasibility and intial layout are discussed in varying degrees of detail. Two step-by-step hydraulic design procedures are presented, one being an iterative type adaptable to computer solution, the other a graphical approach. Both procedures are keyed to a simple system with one jet pump and one booster pump. Additional considerations are given for systems using multiple jet and/or booster pumps. A companion report to be issued at a later date will describe techniques and equipment for building, operating, and monitoring a jet pump bypassing system.

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PREFACE

This report is the result of research performed under the Improvement of Operations and Maintenance Techniques (IOMT) research program which is sponsored by the Office, Chief of Engineers (OCE), and conducted at the U. S. Army Engineer Waterways Experiment Station (WES). This report contains guidance for the planning of a jet pump remedial sand bypassing system and also contains specific instructions for preparing the basic hydraulic design for such a system.

A companion report will be issued at a later date describing techniques and equipment for building, operating, and monitoring a jet pump bypassing system. This companion report will also include example designs illustrating the procedures and recommendations from both reports. Both reports are based on testing conducted by WES investigators in both laboratory and field installations.

The IOMT work unit was entitled "Eductor Systems for Sandtrap Bypassing" and was performed during the period 1973-1979. The study was performed under the direction of Messrs. H. B. Simmons, Chief of the Hydraulics Laboratory, F. A. Herrmann, Jr., Assistant Chief of the Hydraulics Laboratory, and R. A. Sager, Chief of the Estuaries Division. The work was performed by Messrs. W. B. Fenwick, T. W. Richardson, P. L. Chandler, J. C. Roberge, S. R. Bredthauer, and E. W. Flowers under the supervision of Mr. E. C. McNair, Jr., Chief of the Research Projects Group. This report was prepared by Messrs. Richardson and McNair. This report was reviewed in draft form by several CE Division offices, by the U. S. Army Coastal Engineering Research Center, by the Engineering and Operations Divisions of OCE, and by Dr. D. R. Basco of E_2O Consultants, Inc., as a consultant to WES.

Commanders and Directors of Wes during the conduct of this work unit and the preparation and publication of this report were COL G. H. Hilt, CE, COL John L. Cannon, CE, COL Nelson P. Conover, CE, and COL Tilford C. Creel, CE. Technical Director was Mr. F. R. Brown.

CONTENTS

CONVERSION FACTORS, U. S. CUSTOMARY TO METRIC (SI) UNITS OF MEASUREMENT

U. S. customary units of measurement used in this report can be converted to metric (SI) units as follows:

Multiply	By	To Obtain
cubic yards	0.7645549	cubic metres
Fahrenheit degrees	5/9	Celsuis degrees or Kelvins*
feet	0.3048	metres
feet of water	2988.98	kilograms per square centimetre
feet per second per second	0.3048	metres per second per second
gallons per minute	0.06308	litres per second
inches	25.4	millimetres
square feet	0.0929	square metres

* To obtain Celsius (C) temperature readings from Fahrenheit (F) readings, use the following formula: $C = (5/9)(F - 32)$. To obtain Kelvin (K) readings, use: $K = (5/9)(F - 32) + 273.15$.

A GUIDE TO THE PLANNING AND HYDRAULIC DESIGN OF JET PUMP REMEDIAL SAND BYPASSING SYSTEMS

PART I: INTRODUCTION

1. Sand bypassing is a term used to denote the transfer of cohesionless sediments past man-made or natural barriers that trap, divert, or otherwise interfere with the natural process of coastal sediment transport. This bypassing can be accomplished by natural forces, as is the case in most uncontrolled and unimproved tidal inlets, or bypassing can make use of pumps or other means for excavating and transporting littoral materials.

2. This report provides specific guidance in the design of remedial bypassing systems that employ jet pumps for initial solids handling. The term "remedial" refers to bypassing for the purpose of alleviating an existing problem, as opposed to preventing a possible future problem. However, many of the techniques and approaches used can be applied to either situation. Although jet pumps have been used as suction boosters on hydraulic dredges for many years, their use in sand bypassing was developed as new technology in a research program sponsored by the Office, Chief of Engineers (OCE). Work under this program was performed by the Hydraulics Laboratory of the U. S. Army Engineer Waterways Experiment Station (WES).

3. This report also provides some general guidance in the coastal engineering approach to sand bypassing problems. The approach presented in this report is oriented toward the requirements of jet pump systems, but useful information will result regardless of the type of bypass system finally selected.

4. PART I of this report is devoted to evaluating a site and defining the parameters on which the bypassing system can be designed. Characteristics of jet pump bypassing systems and potential jet pump system configurations are also presented. PART II of this report deals with preparing a preliminary system layout. Methods are presented for

preliminary selection of portions of a jet pump system based on site requirements and characteristics. Specific design procedures and calculations are presented in PART III of this report. A subsequent report will contain instructions and suggestions for building, operating, and monitoring a jet pump bypassing system, as well as example designs.

The Bypassing Problem

5. Any activity in the coastal zone that impounds or diverts littoral sediments implies the need for sand bypassing. The earliest planning stages of such an activity should include consideration of ways to accomplish sand bypassing. When consideration for bypassing was not made prior to construction, the need for such a capability must be determined and remedial action taken if necessary.

6. There are several indicators of the need for a sand bypassing system of some type. Navigational problems caused by channel shoals and downdrift beach erosion coupled with updrift beach accretion (Figures 1 and 2) are by far the most common indicators that natural processes are being altered and that mechanical bypassing of some type may be needed. In many of these circumstances, complaints of local citizens and navigation interests will be heard.

7. Confirmation that a situation does exist which may be alleviated with a bypassing system can be made by personnel versed in coastal engineering who perform the following steps:

a. Site visits and inspections. First-hand, visual inspections of a site will usually provide evidence of updrift accretion/downdrift erosion, indicative of littoral interruption. Visual inspection may even show evidence of channel shoal or offshore bar formation by unusual breaking wave patterns in or near channel areas.

b. Review of site history. A review of photographs, charts, and maps will provide an excellent indication of the behavior of the site. General beach recession or aggradation, both updrift and downdrift, can often be diagnosed. Such studies are not only helpful in diagnosing problems, but may later prove invaluable in establishing magnitudes. For instance, comparisons of high-water marks in aerial photographs of a jetty accretion fillet may help identify the

Figure 1. Aerial view of Mexico Beach, Florida, showing updrift accretion, downdrift erosion, and channel shoaling

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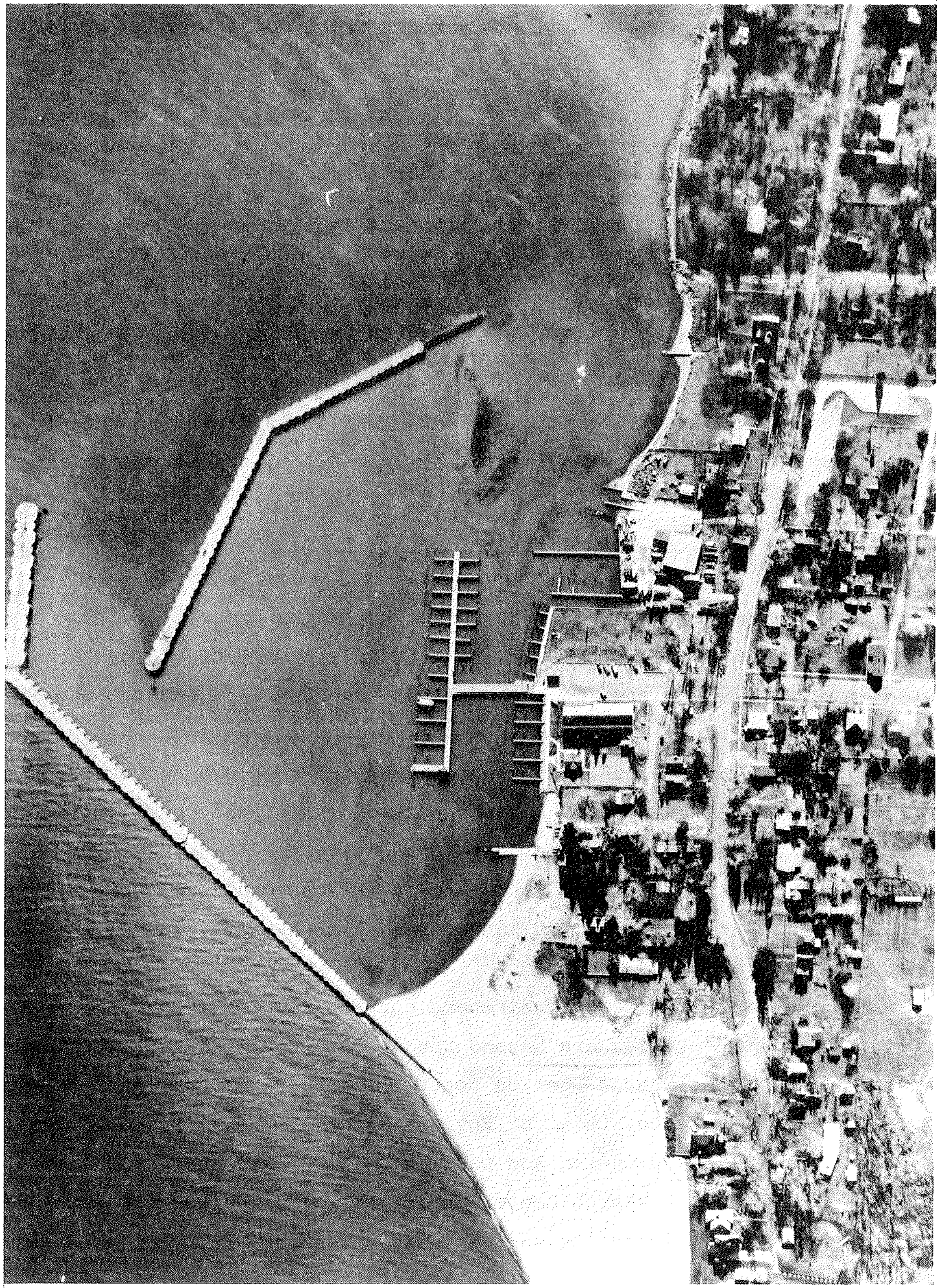

Figure 2. Aerial view of Port Sanilac, Michigan, showing accretion fillet and downdrift erosion

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rate of accretion of littoral drift for a particular period of history.

c. Hearings and interviews - Dialogue with local citizens who have observed the site over an extended period is helpful in establishing behavioral patterns. Such discussions may define events that occur during and immediately following severe storms. Extent of wave runup or overtopping, extent of beach damage, rate of beach rebuilding following storms, areas of concentrated wave attack, etc., are examples of site characteristics that might be identified by this method. Reports of wind speeds, wave heights, and water levels should be used judiciously since these parameters are extremely difficult to quantify by casual observation.

These steps should provide sufficient information to decide whether a problem exists that might require further site study and investigation.

Site Study and Problem Formulation

8. Design and employment of a sand bypassing system require a specialized coastal processes study of the site. Results of such a study provide the basis on which to select the most appropriate sand bypassing approach. If a field data collection program is needed, the cyclic nature of many coastal phenomena requires that the collection period be at least one year. Periods exceeding one year usually give more complete results. Because of this relatively long observation period, the coastal processes study should be implemented as soon as possible following the decision to investigate a mechanical bypassing solution.

9. Coastal processes studies are complex and the methods for carrying out such studies are beyond the scope of this report. Engineer Manual 1110-2-3300, "Beach Erosion Control and Shore Protection Studies," published by the Office, Chief of Engineers, in 1966 and the Shore Protection Manual (SPM) prepared and published in 1977 by the U. S. Army Coastal Engineering Research Center (CERC) may be consulted for guidance in designing and implementing such a study. However, to provide additional explanation, some of the important items of a coastal processes study for a jet pump bypass system are listed in approximate order of

importance, together with explanatory remarks:

a. Littoral transport. Transport vectors (rates and directions) as a function of time should be determined. The smaller the time increment for which transport vectors can be quantified, the better. Especially important are vectors for storm events, when large transport rates may be expected. In addition, an attempt should be made to establish the confidence limits of these vectors, taking into account such factors as data accuracy and effects of any simplifying assumptions used in processing the data.

b. Movement paths and deposition patterns. Of equal importance to identifying transport vectors is the determination of paths along which the transport is moving and the patterns in which it is depositing. This is especially important in the vicinity of structures, for two reasons:

(1) Structures have complicated and often unpredictable effects on littoral transport. The only reliable way of determining movement paths and deposition patterns near structures is to collect and analyze field data on them. Model test results may be used to supplement such data, but should not be considered a substitute.

(2) A jet pump bypassing system is usually used near structures to take advantage of the channelization of sand movement and the concentrated deposition that often occurs there. Also, the structures can provide protection and a foundation for the land-based portion of such a system.

c. Waves. Waves have direct effects on a jet pump sand bypassing system mainly in the restrictions they place on jet pump deployment and in their effect on pumphouse location and characteristics. The wave climate has many indirect effects, however, such as being a prime cause of littoral transport and causing alterations in water levels due to setup. A frequency distribution of significant wave heights at the site, the representative wave periods, and possibly yearly directional roses of significant height and period usually provide information for the direct effects which waves have on a bypassing system. For determining indirect effects, however, a much more detailed description of the wave climate at the site may be required.

d. Sediment characteristics. A description of the sediment to be bypassed is essential to design of the bypass system. Characteristics that must be determined include but are not limited to:

(1) Grain size distribution.

(2) In situ porosity.

(3) Specific gravity of sediment solids.

(4) Presence of cohesive material or cementing agents.

(5) Presence of large objects such as cobbles, shells, or debris.

(6) Subsurface profiles in areas where jet pump system may operate, preferably obtained from core samples.

e. Water-level fluctuations. The magnitude and frequency of water-level fluctuations due to tides, wave setup, surges, seiches, and other causes should be determined. Of special importance is identifying what combinations of these fluctuations might be expected over different time periods.

f. Morphology. Detailed surveys should be made to determine nearshore bathymetry at and adjacent to the bypassing site. Yearly morphologic cycles as well as longer term trends should be identified using these surveys, previous ones, and other data. An attempt should be made to predict future morphological trends.

g. Currents. The only direct effect which currents might have on a jet pump bypassing system would be on jet pump deployment. Maximum expected currents, their location, and direction should be identified. Indirect effects of currents include the potential to transport sediment to jet pump locations or even past these locations in suspension if strong enough.

10. Other information that is needed for preliminary design of the jet pump system but would not necessarily result from a coastal processes study includes:

a. Above-water layout of bypassing site, to include plan views and cross sections of structures, topographic features, rights of way, locations of utilities, etc.

b. Physical description of areas to which bypassed material will be pumped, and identification of possible routes for pipelines.

c. Design characteristics of structures in the vicinity of the bypassing system (design parameters and criteria, armor unit sizes, etc.).

Consideration of Bypassing Alternatives

11. A number of bypassing methods and approaches should be considered for any given bypassing problem. Very rarely will a problem

be so well defined and limited in scope that it can be alleviated only by one type of system. The designer then has the task of selecting the system which most nearly satisfies the bypassing requirements of that site. The following is a brief discussion of some aspects of the spectrum of bypassing systems.

Classification

12. Many ways of classifying sand bypassing systems are possible. However, aside from capacity, the single characteristic of any system that most affects its suitability for a particular project is the degree of mobility which it possesses. Mobility in this sense is defined as the ease with which the system can reach various areas of the project site. Accordingly, the following classifications are suggested:

a. Fixed systems, in which the entire physical plant is fixed as to location. Examples could be dredge pump systems operating from a house or platform or jet pump systems using fixed jet pumps. Such systems require a high degree of predictability of littoral transport vectors, movement paths, and deposition patterns.

b. Mobile systems, in which the entire physical plant can be relocated readily to reach various areas of the bypassing site or other sites. Examples could be floating dredges or jet pump systems mounted on trailers. Such systems may be more vulnerable to the physical environment than other types. Dredges, for instance, may be affected by wave action.

c. Semimobile systems, in which mobility is restricted to a single, well-defined area of the project site, the scope of which can be a determining factor in system design. Examples could be dredge pump systems mounted on tracks or rails, or jet pump systems using mobile jet pumps.

13. An important aspect of the classification system described in paragraph 12 is that particular equipment may fit more than one category, depending on site conditions and how it is used. For instance, a land-based clamshell crane might be used in one location only, making it essentially fixed. If a suitable roadway exists on a jetty, the clamshell might be moved back and forth along the jetty's length, in which case it could be termed a semimobile system. Driven onto a barge, the clamshell crane could become the major part of a mobile system. While this situation may appear confusing at first, in fact a

mobility-type classification is useful to the designer. Since it is based not just on system characteristics but on the interrelationship between these characteristics and project conditions and requirements, it deals directly with the problem at hand: choosing the best system for a particular situation.

Equipment

14. The list of equipment that can be used to form a bypass system is extensive. Anything from a hand shovel to a hopper dredge could conceivably be employed. The following list, not at all complete, gives items of equipment that exist at present and that have been or could easily be used in a bypass system:

a. Floating dredges.
   (1) Trailing suction hopper.
   (2) Cutter suction.
   (3) Plain suction.
   (4) Bucket ladder.
   (5) Clamshell.
   (6) Dipper.
   (7) Backhoe.

b. Land-based mechanical equipment.
   (1) Dragline.
   (2) Clamshell.
   (3) Backhoe.

c. Hydraulic equipment.
   (1) Dredge pump.
   (2) Jet pump.
   (3) Other types of solids-handling pumps.

Structures

15. The role of structures as part of a total sand bypassing system should never be underestimated. Structures can perform the following important functions, among others:

a. Direct and channelize movement of littoral drift.

b. Cause deposition of littoral drift at predetermined locations.

c. Provide access to areas of project site seaward of shoreline.

d. Provide foundation for part or all of bypass system.

e. Shelter bypass system from wave action.

Any bypass system design should try to make maximum use of the benefits that existing structures can offer. In addition, serious consideration should be given to structural changes or additions that might help in operation or design of a bypass system. Such additions might include the creation of deposition areas for littoral drift by means of breakwaters or weir sections in jetties.

16. The engineer responsible for the solution of a bypassing problem will undoubtedly study a number of possible methods in detail and will develop several potential solutions that will be studied even further. If a jet pump bypassing system is identified as a possible alternative, the remainder of this report will serve as a guide in developing designs for such a system.

Site Conditions Affecting Feasibility of Shore-Based Jet Pump Bypassing System

17. Certain site characteristics and bypassing requirements could make a jet pump bypassing system viable at a given site. Assuming that such a system would be deployed from shore (as opposed to a floating base), these characteristics and requirements are:

a. Need for continuous bypassing. Such a requirement definitely indicates that a jet pump system should be considered. Jet pump systems operate at a relatively low pumping rate compared with a large hydraulic dredge. Bypassing performed by a jet pump system can be made to proceed at a rate of the same order of magnitude as the average littoral drift rate.

b. Littoral transport near shore or structures. Littoral transport moving close to shore or to structures at the site can usually be handled by a shore-based jet pump system. At most sites, at least one location can be identified where this occurs. More specific criteria will be given later.

c. Moderate peak transport rates. Although not an absolute requirement, the less littoral transport rates at a site vary with time over a yearly cycle, the better suited the site is for jet pump bypassing. Those sites with significant variation can be dealt with using concepts that will be explained later.

d. Littoral drift impoundment area. An existing sheltered impoundment area, such as that created by a detached breakwater or a weir section in a jetty, is of great benefit in making a jet pump system viable. An exposed impoundment area, such as a jetty accretion fillet, may be helpful depending on other factors in the system design.

18. Many other, more site-specific factors will have to be considered in determining the viability of jet pump bypassing at a particular site. In addition, not all of the above factors are necessary in order for a site to be suitable for a jet pump system. However, a site that possesses all of these factors can be considered a prime possibility for jet pump bypassing unless one or more conditions exist that preclude use of a jet pump system.

19. Some site conditions will probably preclude the use of a jet pump bypassing system. These are:

a. Presence of cohesive or cemented materials. Cohesive clays and cemented sands cannot be effectively dislodged by the jet pump using presently available cutting techniques. Even relatively thin layers of such material may cause problems.

b. Transport and/or accretion over a broad area. If littoral transport and/or accretion of such transport occurs over broad or poorly defined areas at the site, it may be difficult to design a reasonable jet pump bypassing system. In many ways, this situation would be the converse of the factors described in paragraphs 17 b and d.

c. Absence of suitable location for clear water intake. This will be discussed fully in later parts of this report, but a relatively sheltered, accretion-free location must be available from which clear water can be drawn to drive the jet pump.

Other considerations may impact upon the feasibility of a bypass system, such as property ownership, aesthetics, and local attitudes. Such items must be dealt with but are beyond the scope of this report.

Description and Configurations of Jet Pump Bypassing Systems

20. The center-drive* jet pump, the primary component of the jet pump bypassing system, is different from other pumps in that it contains no moving parts and is powered by a jet of water. The jet pump operates completely submerged, resting on the bottom with its suction tube buried in the material to be pumped. The basic principle behind the operation of the jet pump is the exchange of momentum within the pump. Clear water, normally supplied by a centrifugal pump, enters the jet pump through a nozzle as a turbulent jet (Figure 3). In the mixing chamber, turbulent

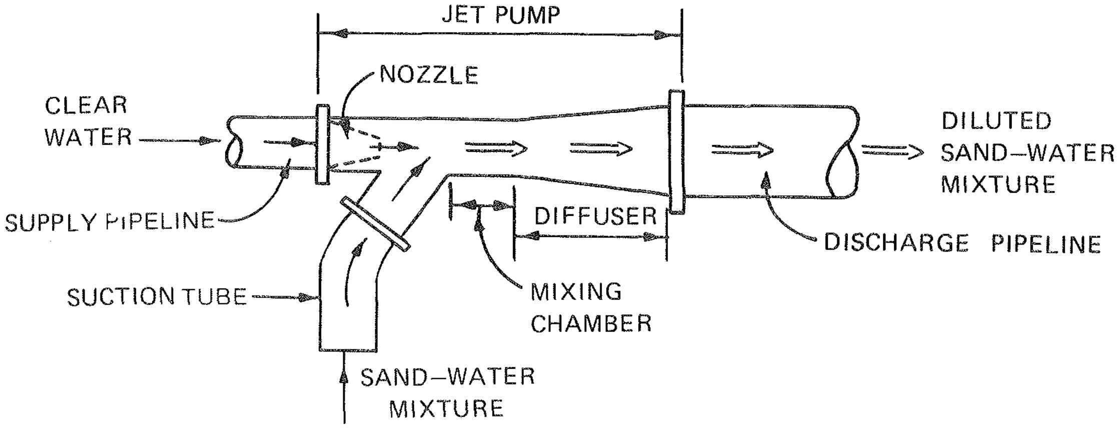

Figure 3. Jet pump principles of operation

mixing occurs between the water jet and a sand-water mixture drawn into the suction tube. This mixing causes a transfer of momentum from the jet to the sand-water mixture. At the same time, the sand-water mixture is diluted by the jet water. The diluted mixture then passes into the diffuser section of the jet pump, causing more sand-water mixture to be drawn into the suction tube in a continuous process. In the diffuser, a gradual expansion of the jet pump walls results in some flow energy changing from velocity to pressure. After exiting the diffuser, the diluted mixture moves through a discharge pipeline, usually to a

---

* The term "center-drive" refers to a jet pump with a nozzle located on the center line of the main pump body. "Side-drive" or "peripheral-drive" jet pumps, on the other hand, have one or more nozzles located on the periphery of the main pump body.

conventional dredge pump acting as a discharge booster.

21. The performance of the center-drive jet pump in a given medium can be defined in terms of three dimensionless parameters. These parameters are: (a) head ratio, N ; (b) flow ratio, M ; and (c) area ratio, R . The head ratio, N , is defined as

$$N = \frac{HDIS - HSUC}{HSUP - HDIS} \tag{1}$$

where

HDIS = total energy head in the discharge pipeline at the jet pump

HSUC = total energy head in the jet pump suction tube at the jet pump

HSUP = total energy head in the supply pipeline at the jet pump

The flow ratio, M , is defined as

$$M = \frac{QSUC}{QSUP} \tag{2}$$

where

QSUC = volumetric flow rate into the jet pump suction

QSUP = volumetric flow rate through the jet pump nozzle

The area ratio, R , is defined as

$$R = \frac{ANOZ}{AMIX} \tag{3}$$

where

ANOZ = area of the opening at the tip of the jet pump nozzle

AMIX = inside area of the mixing chamber of the jet pump

Locations of these parameters on a center-drive jet pump are shown in Figure 4.

22. Gosline and O'Brien (1934), Mueller (1964), Reddy and Kar (1968), and others have worked on defining the relationships between M , N , and R for various jet pump configurations pumping water. Several investigators, such as Fish (1970), Zandi and Govatos (1970), and Silvester and Vongvisessomjai (1970), have worked on comprehensive theories for jet pumps pumping solids. However, experimentation with a

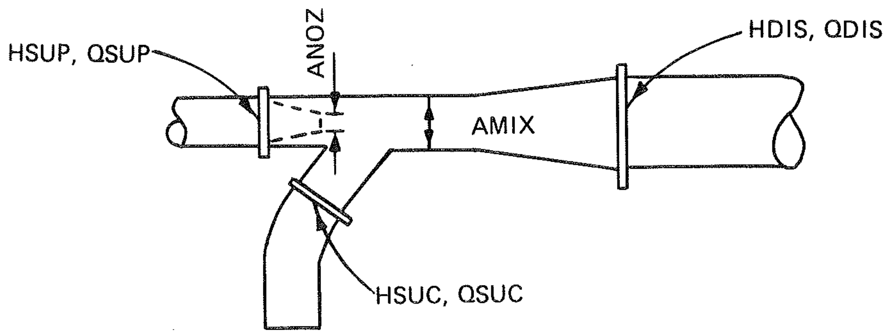

Figure 4. Location of jet pump parameters

particular jet pump is usually required for best results. Experimental data from the WES research program were used to define the relationships for a specific jet pump type pumping both clear water and medium sand. Relationships for pumping sand are presented in a later portion of this report.

23. Figures 5-7 illustrate the basic components of a simple jet pump bypassing system. More complex configurations are possible and

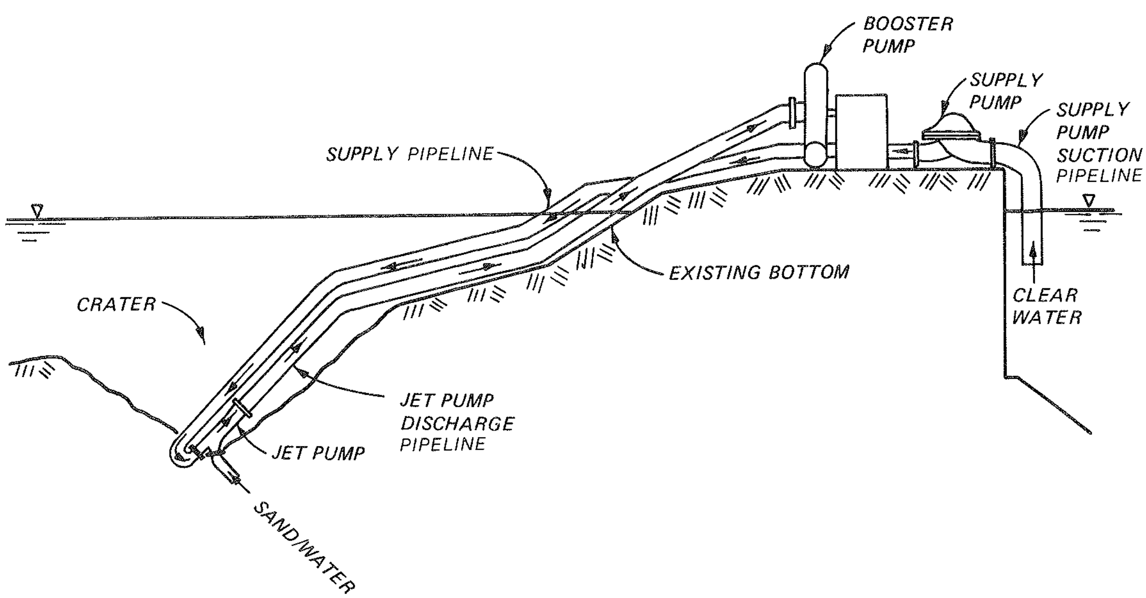

Figure 5. Elevation view of simple jet pump system

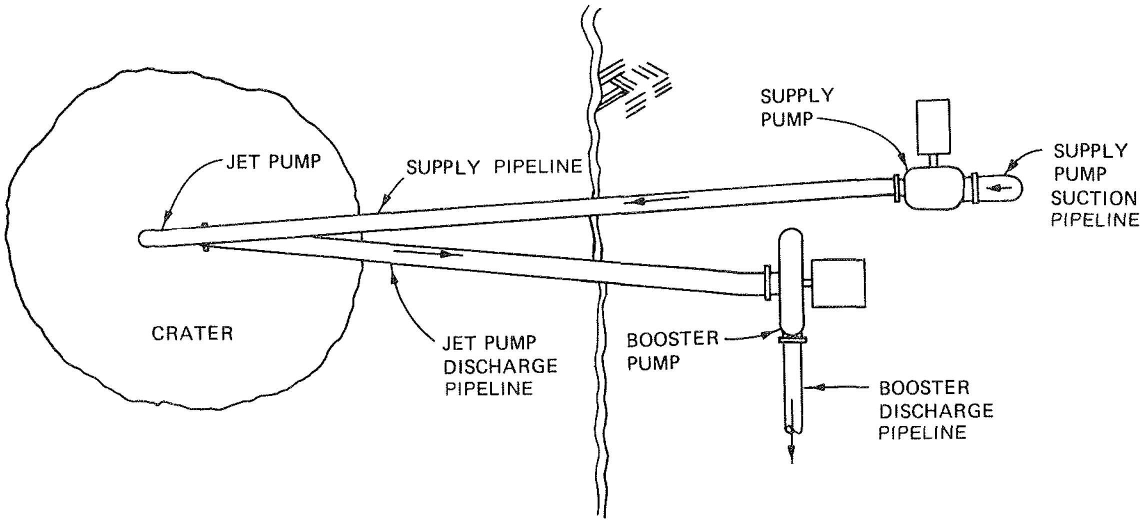

Figure 6. Plan view of simple jet pump system

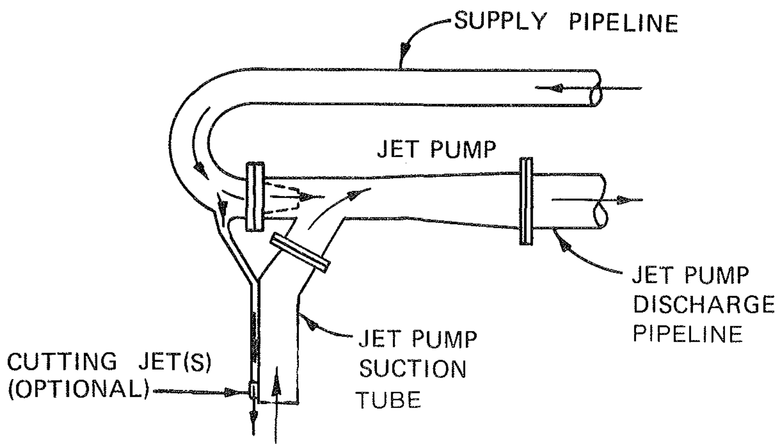

Figure 7. Elevation view of jet pump

are frequently required, but the operating principles remain the same. Figures 5-7 are especially important because the terminology shown for various system components will be used throughout this report.

24. The component parts of the simple jet pump system shown in Figures 5-7 and their purposes are as follows:

a. Supply pump. Provides clear water to drive the jet pump. Also supplies water for jet pump cutting jets, if such are used, and may supply flushing water for the booster pump. Supply pump suction pipeline must be located in an area relatively free of shoaling or large amounts of suspended sediment or small debris. The supply pump is usually an ordinary centrifugal pump.

b. Supply pipeline. Carries clear water from supply pump to jet pump. May be made of rigid pipe, flexible hose, or a combination of both. Also carries water for cutting jets.

c. Jet pump. Dredges sand/water mixture and provides head to move it through jet pump discharge pipeline to booster pump. Jet pump suction tube is used to ensure burial of jet pump suction opening and consequent intake of high solids content sand/water mixture. Cutting jet(s) aid in burial of suction tube and in excavation of consolidated material.

d. Crater. One of the most important parts of any jet pump system. The crater is formed on the sea or lake bottom and is maintained by virtue of the jet pump dredging below the level of the surrounding bottom. If the jet pump supply and discharge pipelines are flexible, a jet pump resting on an undisturbed bottom will excavate a crater by simply following the bottom of the crater downward as it removes sand. This process is illustrated in Figure 8. A jet pump buried below the undisturbed bottom, on the other hand, with rigid supply and discharge pipelines will excavate a crater above itself by removing sand from underneath. Figure 9 illustrates the formation of such a crater. The crater acts as a trap for littoral drift that would otherwise pass by. Without a crater or an array of craters, the jet pump has no chance of intercepting moving littoral drift. Crater size and shape are functions of many factors, such as depth of the jet pump below surrounding bottom, characteristics of in situ sediment, and rate of dredging by jet pump relative to rates of littoral material influx and slumping of crater sides.

e. Jet pump discharge pipeline. Conveys jet pump discharge mixture from jet pump to booster pump. May be of same construction as supply pipeline.

f. Booster pump. Provides energy to move jet pump discharge mixture to selected discharge area. Several booster pumps may be required along length of booster discharge pipeline, depending on distance mixture is to be pumped. The booster pump is usually an ordinary dredge pump.

g. Booster discharge pipeline. Carries discharge mixture from booster pump to discharge area. Usually of rigid construction.

25. One characteristic of a jet pump bypassing system is that many variations on the simple system shown in Figures 5-7 are possible in order to adapt the system to specific requirements. Some of these variations are:

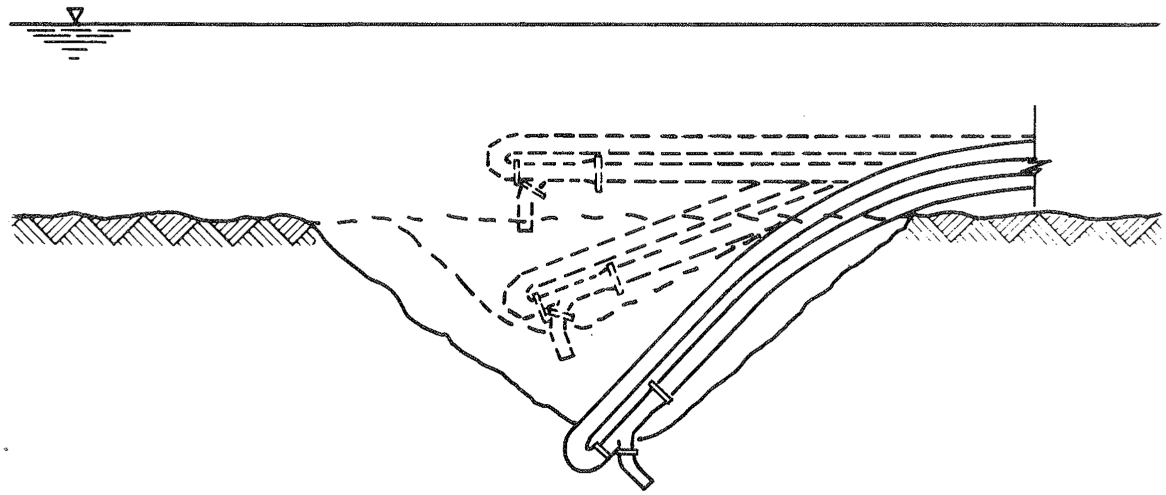

Figure 8. Excavation of crater by jet pump resting on bottom

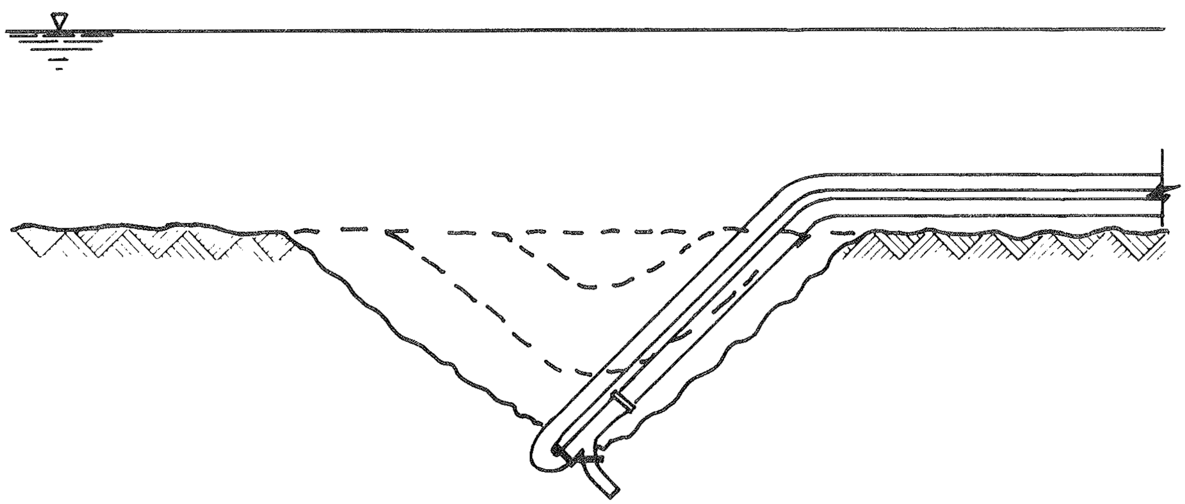

Figure 9. Excavation of crater by jet pump buried below bottom

a. Mobility of on-shore components.

(1) Permanent. Onshore system components, such as the supply and booster pumps, are located on fixed foundations onshore or on a structure such as a jetty. Usually, they are protected from the elements by an enclosure.

(2) Portable. Onshore components are mounted on a movable platform, such as a truck or trailer, and operate from parking locations onshore or on a structure. These components can then be used at several locations within a site or at different sites.

b. Mobility of jet pump(s).

(1) Fixed. Jet pump is installed permanently at a certain elevation, below the existing bottom. In such a configuration, the jet pump is virtually immune to wave action or the effects of currents. Multiple fixed jet pumps can be installed to create craters which cover a certain area.

(2) Mobile. Jet pump is equipped with a variable buoyancy float and flexible hoses, allowing it to be raised from its crater, moved a certain distance, and sunk to create another crater. This configuration of jet pump is best employed in areas sheltered from severe wave action.

c. Flexibility of jet pump pipelines.

(1) Rigid. Jet pump supply and discharge pipelines are constructed of steel pipe or other rigid material. Usually, this type of piping is used in conjunction with fixed jet pumps and can, in fact, be the means by which the jet pump is fixed in place.

(2) Flexible. Flexible hose, such as rubber dredging hose, is used for jet pump pipelines. This hose is normally used with a mobile jet pump to allow easy relocation of the jet pump. In such a use, the hose would be equipped with floats of fixed buoyancy to prevent it from being buried in the existing bottom or in the jet pump crater.

(3) Combination. Lengths of rigid pipe are connected by lengths of flexible hose to form the supply and discharge pipelines. Such a configuration provides a certain degree of flexibility at less cost than an all-flexible system. With floats attached, this type of pipeline can be used with mobile jet pumps in areas subject to very mild wave action. Without floats, such piping could be used with fixed jet pumps, provided that the jet pumps are supported by some means independent of the pipelines.

d. Number of jet pumps operating simultaneously.

(1) Single. System in which only one jet pump operates at a time. Such a system is the simplest to design and operate. A number of jet pumps may be installed at a site and operated individually, although the complications of piping and valving arrangements will increase rapidly as the number of jet pumps increases. In general, however, a single-type installation is preferable to a multiple one.

(2) Multiple. System in which two or more jet pumps operate at a time. This type of system should be considered only where the requirements of the bypassing project cannot be met by a single-type system, or where an excessive number of independent jet pumps are required for a single-type system. If a multiple system is necessary, the number of jet pumps operated simultaneously should be kept to a minimum.

Except as noted, any of the system variations discussed above can be used in conjunction with any of the others.

26. It should be noted that all discussion in this report deals with shore-based bypassing systems. However, there is no reason why a jet pump system designed using the techniques presented in this guide could not be placed on a floating platform. Such deployment would place the system philosophically in the category of dredges; therefore, no discussion of that deployment technique is presented in this report.

PART II: INITIAL SYSTEM LAYOUT

27. The aim of this part of the report is to provide guidance for the designer in arriving at general system configurations for which more detailed designs can be made. Although subsequent discussions will be in terms of a single system layout, in reality the designer should consider several alternative layouts simultaneously. All the alternatives should be treated equally until economics or other factors influence the choice of a final design.

28. Two points regarding this part of the report should be understood at the outset:

a. A series of topics are discussed that relate to the initial system layout. These topics are not presented in sequential order; in fact, no such order can be applied to them. Most of the topics interact with one or more of the others, the degree of interaction sometimes depending upon other factors as well. The designer must develop a grasp of the concepts being presented rather than trying to follow the presentation in a step-by-step fashion.

b. The success of the initial system layout in meeting the actual bypassing requirements of the site will depend primarily upon the quality of the coastal processes study. THE IMPORTANCE OF THE COASTAL PROCESSES STUDY CANNOT BE OVEREMPHASIZED. It is the foundation upon which the bypassing system is designed. Designing a jet pump bypassing system without detailed information on items such as littoral transport vectors is not advised.

29. The principal purpose of the initial system layout is to provide an approximation of a bypassing system that can be refined and altered using the design procedures presented in PART III. A secondary purpose of the initial system layout is to identify problem areas at the site that may be independent of coastal processes or system pumping performance.

30. Certain guidelines are presented relating to the initial layout. These guidelines pertain to selecting the mode of operation, location, operating time, capacity, sizing of various elements, and certain other system features. It should be remembered that the guidelines are only aids and that significant modifications may be needed after

more detailed calculations are performed. In general, however, use of the guidelines should result in reasonable selections of components for the system.

System Purpose

31. There are two basic purposes for which any bypassing system can be designed:

- a. Reduction of navigation shoaling caused by littoral drift.
- b. Alleviation of undesirable beach changes caused by interruption of littoral drift.

The purpose that the system is to serve should be specifically identified so that requirements pertaining to that purpose can be satisfied. For example, a bypassing system whose purpose is to reduce channel maintenance will be designed, installed, and operated so as to bypass material causing shoals in the navigation channel. On the other hand, a system whose purpose is to provide periodic nourishment for a downdrift beach may be installed and operated quite differently.

32. The designer should be especially wary of attempting to design a "dual-purpose" bypass system; i.e., one that tries to reduce navigation shoaling and alleviate beach changes at the same time. Although the problems of shoaling and beach changes are often interrelated at a particular site, attempting to solve both simultaneously with one bypass system can be difficult for the following reasons:

- a. The interrelationship between the two problems is often far more complex than it appears.
- b. The optimum approach to solving one of the two problems with a bypass system can be very different from the optimum approach to solving the other problem.

The end result of such a compromise design will often be a bypass system that solves neither problem very well. A better approach is to design the system to help solve one problem only. Then, at the end of the design process, review the projected effects of the system on the other problem. Many times it will be found that a system designed for one problem will have significant beneficial effects on the other as well.

The review may also suggest some modifications to the system design that would aid in solving the other problem without affecting performance on the primary problem.

Mode of Operation

33. A jet pump bypassing system has two possible modes of operation:

a. Removal of littoral materials from a deposition area.

b. Interception of moving littoral drift.

In making the initial system layout, one of these two modes should be selected as the primary mode of operation and the system designed accordingly. Generally speaking, mode a is preferable to mode b, all other factors at a site being equal. A system designed for mode a will probably be of smaller capacity and consequently cost less. The deposition area will provide a trap for littoral drift moving at high rates, allowing the accumulated drift to be bypassed later during times of lower drift rates. This fact in turn implies a more regular operating schedule for the system. A system designed for interception, however, must be operated when drift is moving, whether day or night. The dependence of system configuration on mode of operation is illustrated in Figure 10, where the choice of interception as the primary mode indicated the use of fixed jet pumps located in the path of active transport movement. Sand moving along this path will (hopefully) fall into the jet pump craters and be bypassed by the system. An existing sheltered impoundment basin at the site, on the other hand, might be well suited to mobile jet pumps digging craters at different locations to maintain the basin as a trap for littoral drift. Figure 11 shows a system of this type. The possible negative effects of interception should be considered at this stage, also. If the system is effective in intercepting drift at a certain point, it may cause erosion immediately downdrift of that point. Serious stability problems with adjacent structures could be caused by such erosion.

34. The preceding two examples should not be taken as firm guidance. For instance, there is no reason why mobile jet pumps cannot

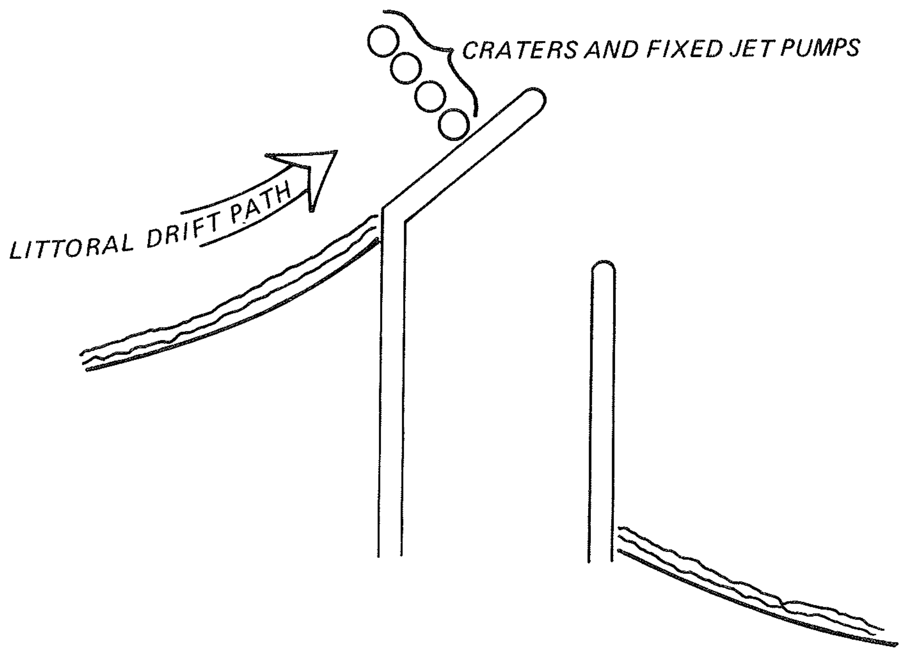

Figure 10. Fixed jet pumps in interception mode

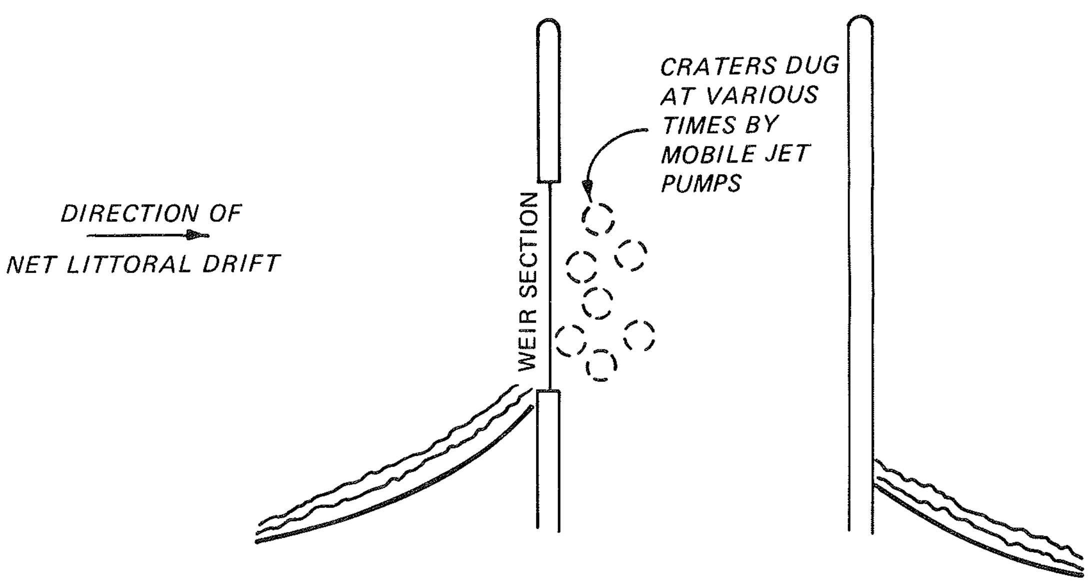

Figure 11. Maintenance of impoundment basin by mobile jet pump(s)

operate in an interception mode or why a field of fixed jet pumps cannot be placed in an impoundment basin. The final choice of mode/system combination must be dictated by the requirements and restrictions of each individual site.

Interaction with Structures

35. A number of general areas are possible for placement of the jet pumps, depending upon the purpose of the bypass system, results of the coastal processes study, and arrangement of structures at the site. The first two items have already been discussed. Some types of structures that may be found at a site and that pertain directly to bypassing are:

- a. Jetty.
- b. Offshore breakwater.
- c. Shore-connected breakwater.
- d. Weir section.

36. Figures 12-15 show some possible configurations of these structures at a site and possible locations for system jet pumps. THESE FIGURES ARE BY NO MEANS DEFINITIVE. Initial selection of jet pump locations should be based on consideration of a number of factors, including the following:

- a. Littoral transport vectors.
- b. Transport movement paths and deposition patterns.
- c. Mode of operation.
- d. Proximity to shore-based equipment.

However, Figures 12-15 indicate some locations that might prove feasible and that present themselves as a direct result of structural configurations. Hatched areas in the figures indicate regions within which jet pumps might be located.

37. An implied assumption in Figures 12-15 is the existence of a strongly predominant net drift direction. At many potential bypassing sites, however, the littoral drift may be approximately equal from both directions. In such cases it may be necessary to utilize jet pumps on

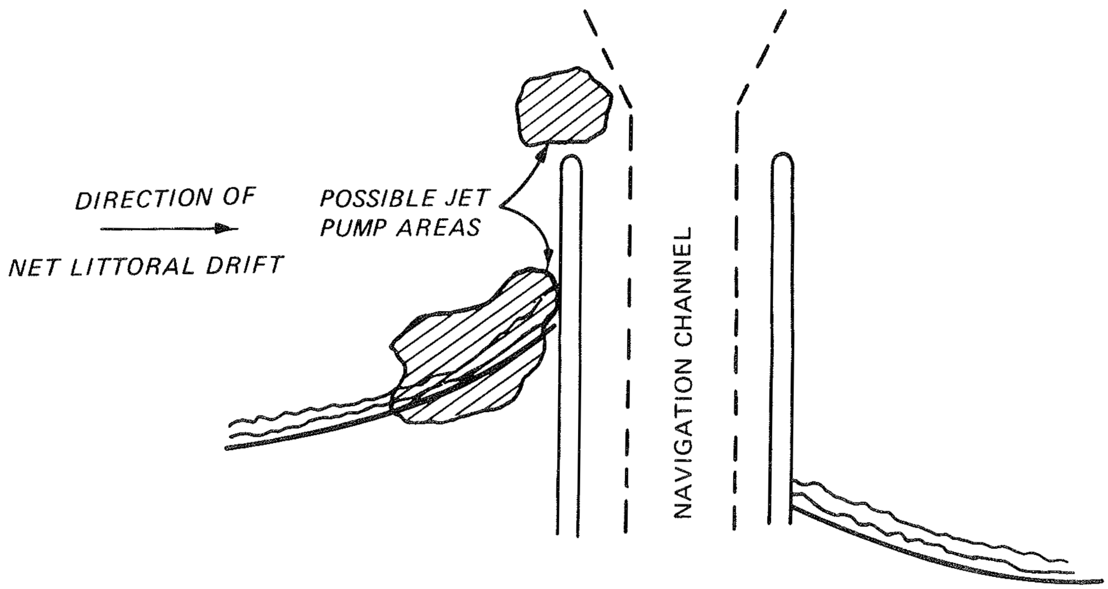

Figure 12. Jettied navigation channel

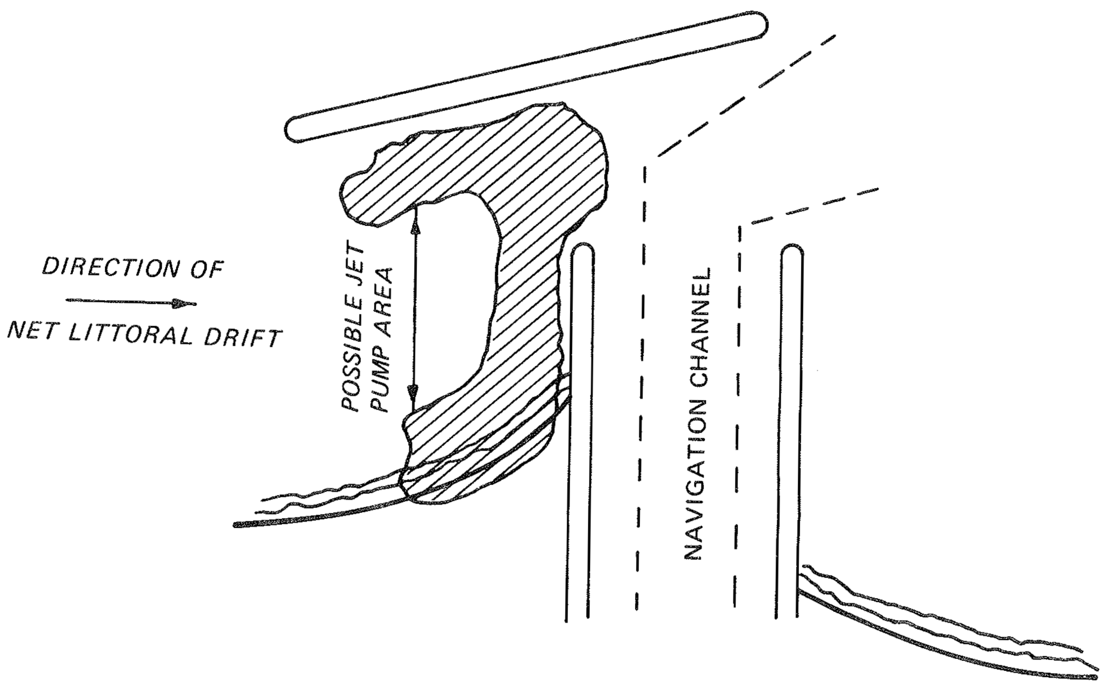

Figure 13. Offshore breakwater and jetties

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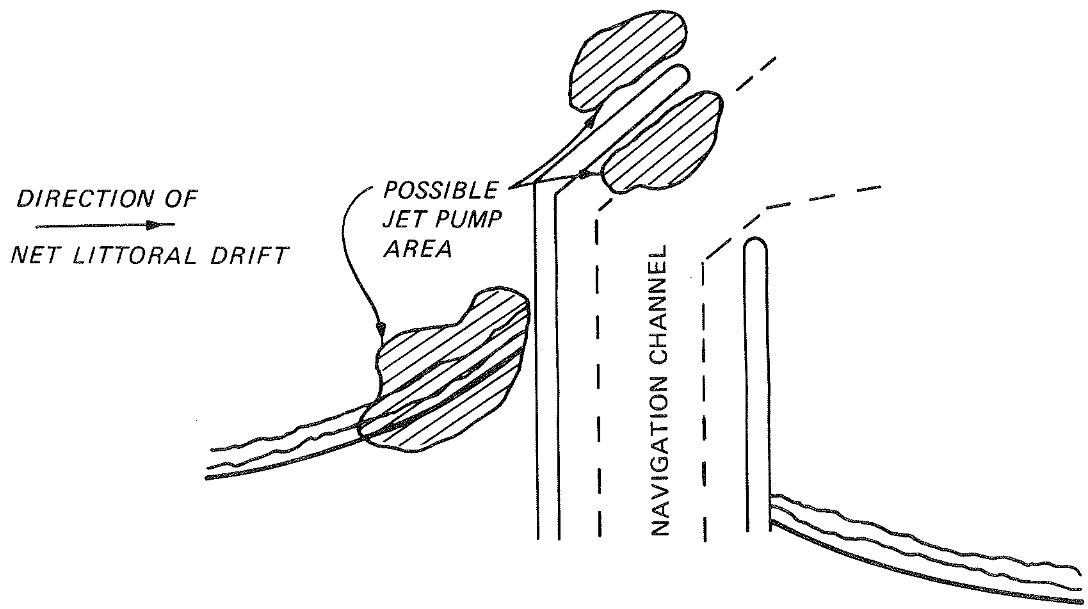

Figure 14. Shore-connected breakwater and jetties

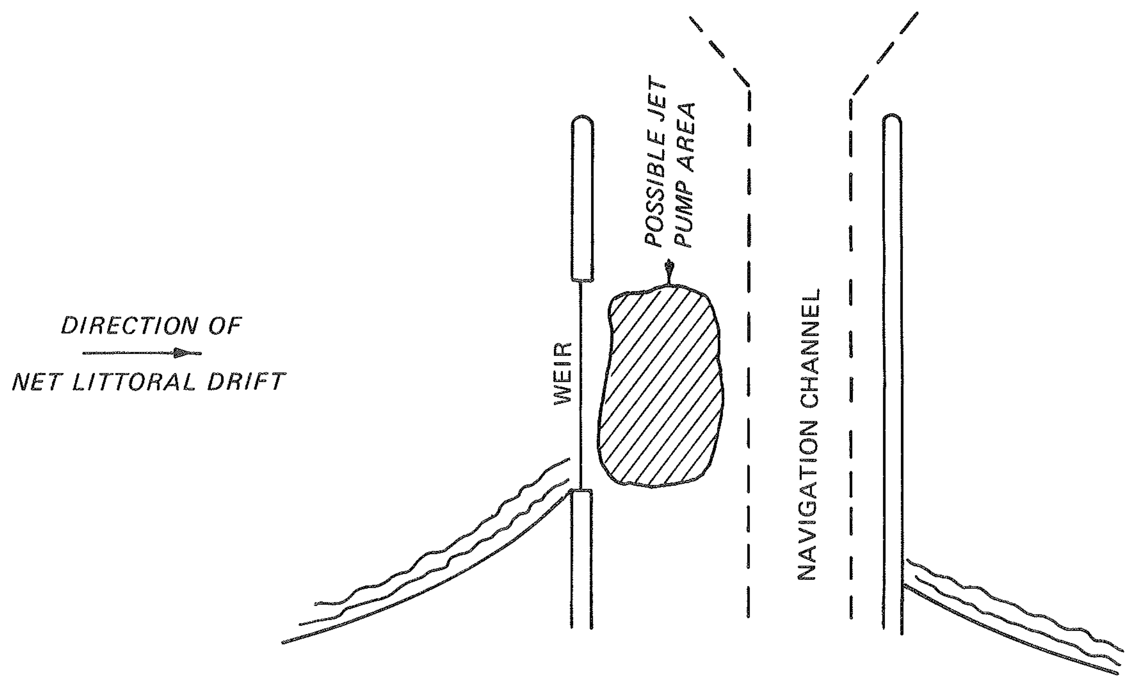

Figure 15. Jetties with weir section

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both sides of the site to bypass littoral drift approaching from either direction.

Location of Shore-Based Equipment

38. Tentative location(s) for shore-based equipment should be selected for the initial layout. However, the choice of location for the shore-based portion of a jet pump system is interactive with location of the jet pump(s). Therefore, considerations of this section must be meshed with those of the following section on locating the jet pumps to assure a sound approach. On one hand, shore-based equipment must be as close as possible to the jet pumps. On the other hand, jet pumps cannot be used in areas without a suitable nearby location for the shore-based equipment. The following factors must be considered in evaluating potential sites for shore-based equipment. The factors pertaining to shore-based equipment are listed in approximate order of importance, although this will vary somewhat from site to site:

a. Proximity to jet pump location(s). Distance along potential pipeline routes from the shore based equipment to the farthest jet pump should be less than about 600 to 700 ft*. This requirement is not rigid and will be discussed later in more detail.

b. Supply pump location. The supply pump must be located as close as possible, both vertically and horizontally, to a suitable location from which it can draw clear water through the supply pump suction pipeline. THIS LOCATION MUST ALWAYS BE FREE OF SIGNIFICANT SHOALING. If the location is subject to shoaling under existing conditions, then measures must be taken to change the shoaling pattern preferably by passive means such as structural alterations or additions. For the initial layout, try to place the supply pump such that the following relation is satisfied:

$$\text{ELSUP} + 0.03(\text{LSUPSA}) \leq 15.0 \qquad (4)$$

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* A table of factors for converting U. S. customary units of measurement to metric (SI) units is presented on page 4.

where

ELSUP = elevation of supply pump center line above mean low water, ft

LSUPSA = approximate length of supply pump suction pipeline, ft

If this relation cannot be satisfied, choose the location that comes closest and proceed with the system layout. It may be necessary later to make some significant changes in the system configuration, depending upon values calculated in PART III of this report.

c. Access. Preferably, the area where the shore-based equipment is located should be accessible by land vehicle. This is especially important during system construction and is a major convenience for system operation. Access by other means such as by boat is possible but much less desirable.

d. Exposure. The shore-based equipment should be located in an area as sheltered from wave action as possible. The more exposed the location, the more expensive the nonfunctioning portions of the system will be (pump houses, pipe anchors, etc.).

e. Fuel or power. Consideration should be given to the ease of fuel delivery or power hookup when choosing a site for shore-based equipment, although this is usually not a controlling factor.

Location of Individual Jet Pumps

39. The main intent of locating individual jet pumps in the initial design layout is to establish approximate values for the following items:

a. Total number of jet pumps in system.

b. Length(s) of jet pump supply and discharge pipeline(s).

c. Size, location, and number of craters.

Items a and c are interdependent to a certain extent. Obviously, the number of craters in the system will be related to the number of jet pumps. However, the number of jet pumps will also be a function of the size of the craters; for instance, when fixed jet pumps are being used to cover a certain area with craters. In such case, the larger the craters, the fewer jet pumps are needed. For the initial layout, craters can be assumed to be conical in shape, with relative dimensions as shown in Figure 16.

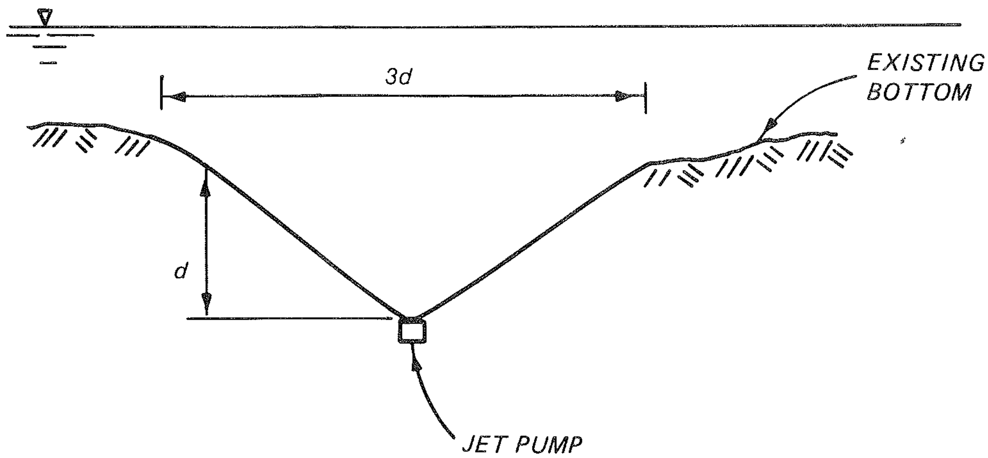

Figure 16. Suggested design crater dimensions

40. The depth, d , of the jet pump below the existing bottom is limited by several factors. A practical maximum is approximately 25 ft, although theoretically no real limit exists. The presence of hard or cohesive strata will limit d to the depth at which they begin. If such strata occur within 5 or 6 ft of the existing bottom, the applicability of jet pumps at that location is doubtful. Also, the proximity of a particular jet pump to rubble-mound structures such as jetties or breakwaters imposes an indirect limit on d at that location. In such a case, placement of the jet pump too close to the structure may result in localized undermining of the structure foundation. Conversely, if the jet pump is placed too far away, a significant portion of the littoral transport may move past the system next to the structure face. A rule of thumb to use for initial jet pump location adjacent to a structure is shown in Figure 17. This rule is based on an isolated jet pump adjacent to a structure. A group of such jet pumps might pose a greater threat to structural stability. The possible stability effects of such a group should be investigated thoroughly on a case-by-case basis.

41. For mobile jet pumps, the term "location" implies determining the area in which each jet pump will operate. The following guidelines

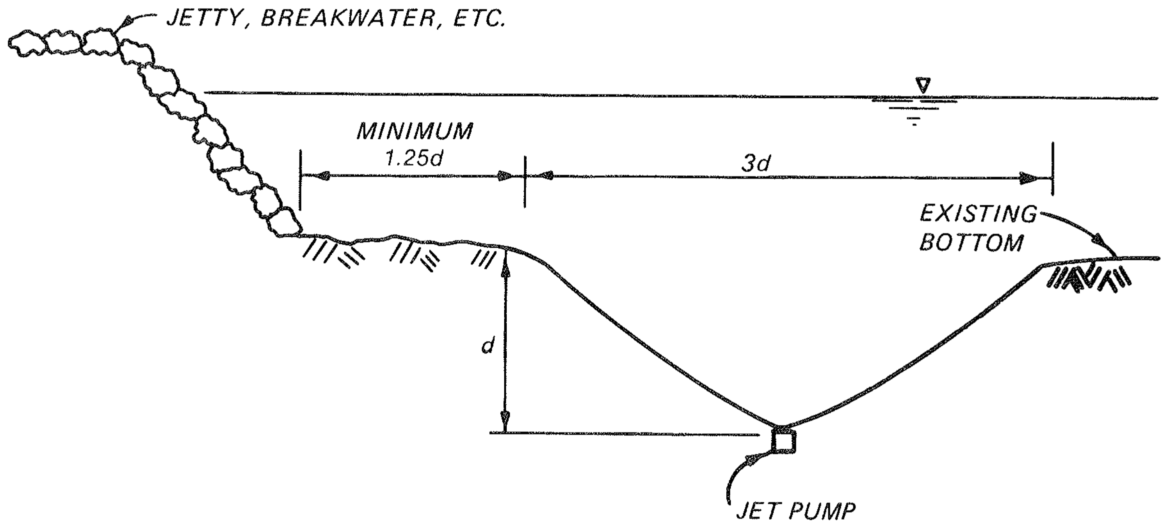

Figure 17. Suggested design dimensions for jet pump location adjacent to structure

should be kept in mind in mobile pump layout:

a. Mobile jet pumps are moved much more easily along an arc than back and forth along a radius (Figure 18). The latter type of movement usually involves lengthening or shortening the supply and discharge pipelines.

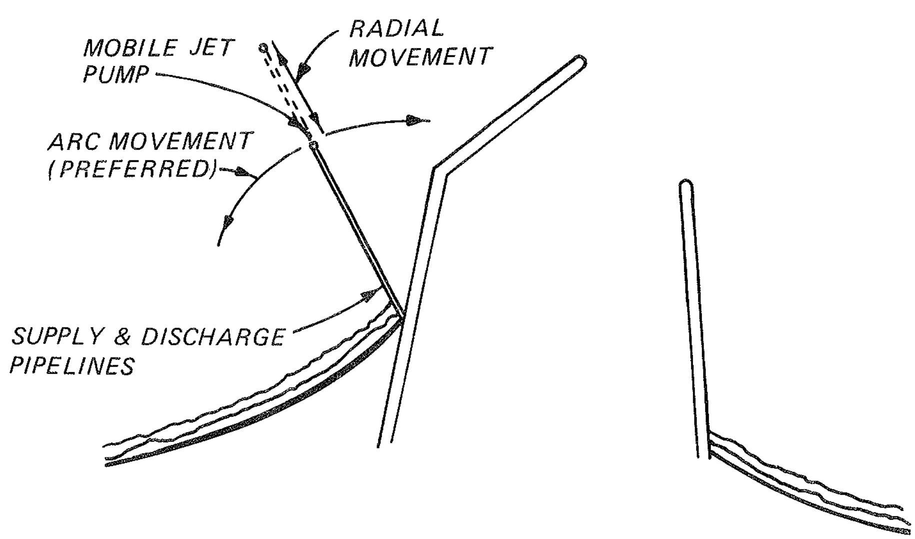

Figure 18. Movement of mobile jet pump

b. Supply and discharge pipelines in the water should be arranged so as to be approximately perpendicular to approaching waves wherever possible (Figure 19).

c. The maximum length of supply and discharge pipelines in the water should be less than 400 ft. This is a practical limitation based on the difficulty of handling long floating pipelines. Ideally, the shorter these pipelines are, the better.

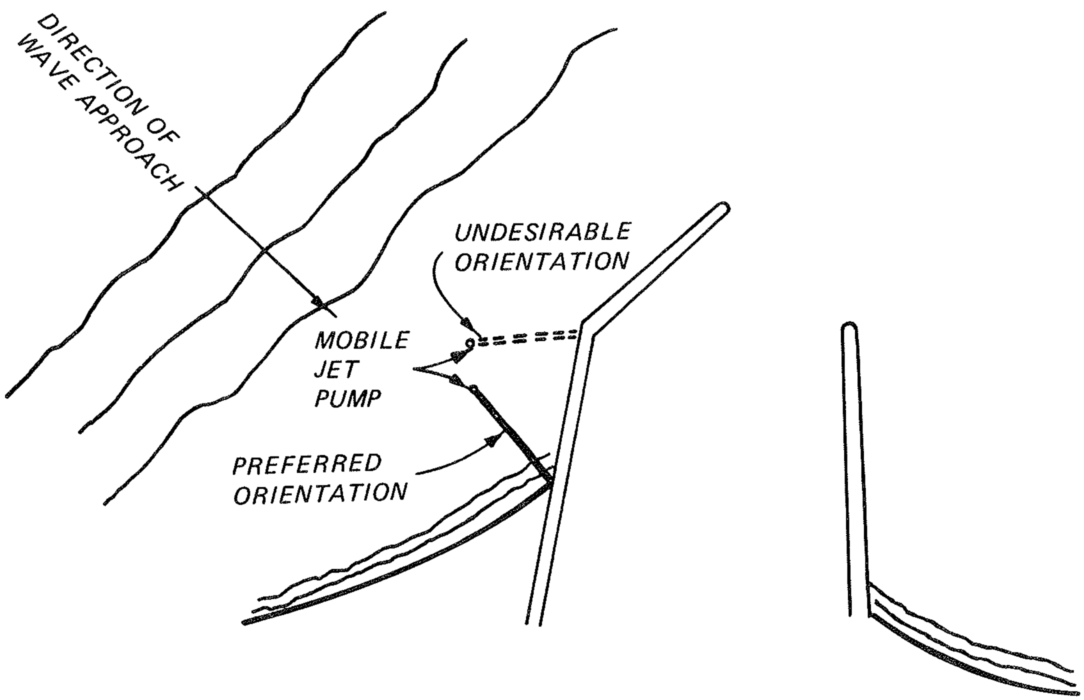

Figure 19. Orientation of mobile jet pump

Site to Which Bypassed Material Is Pumped

42. The initial system layout should include an approximate location or locations to which the material picked up by the system will be pumped. The purpose of selecting such a location at this time is to allow determination of the required booster discharge pipeline length, as well as the approximate number of booster pumps that will be needed. Selection of the discharge location(s) will be determined largely by the information gathered in the coastal processes study, the purpose of the bypassing system, and special requirements and restrictions of the

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site. The discharge point(s) should be no farther away from the bypass site than necessary but should not be so close that the discharged material is returned to the bypass site by local littoral processes. In addition, the discharge point(s) should not be located in areas of little or no littoral movement where the bypassed material might stagnate.

43. A rough estimate of the required number of booster pumps can be made by assuming an equal initial spacing between the pumps along the booster discharge pipeline. This initial spacing may have to be adjusted later based on PART III calculations. Figure 20 shows a suggested range of values to use for the initial estimate of booster spacing as a function of the median size, or d_{50}, of the sediment to be bypassed. The designer should choose a value from this figure that falls in the shaded area for the applicable sediment size, then use this value in laying out the booster system. For example, suppose the sediment to be bypassed has a d_{50} of 0.20 mm. Based upon the physical layout of the site, the designer might then choose an initial booster spacing of, say, 3000 ft. If the total discharge line length is 9000 ft, the inital system layout will then include three booster pumps. The first would be located as close as possible to the jet pump(s) (usually at the same location as the supply pump). The second would be at the 3000-ft point on the discharge line, and the third at the 6000-ft point (Figure 21).

Effective Operating Time

44. The concept of an "effective" as opposed to a total time of system operation will now be introduced. Definition of effective time of operation is necessary for computation of the required system capacity. This step requires that a schedule of operation to accomplish the bypassing be selected. The schedule of operation depends on many factors including availability of manpower, local restrictions on operation of the system, availability of material to be bypassed, and other factors that may be peculiar to the site such as ice during parts of the year.

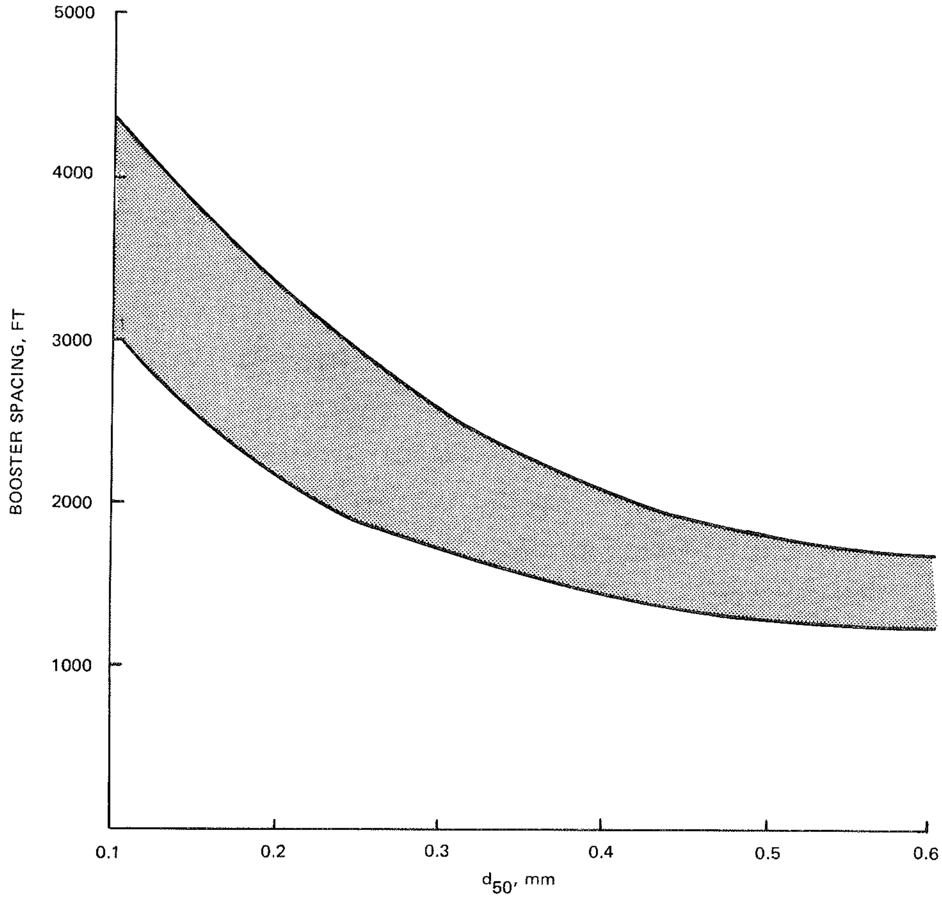

Figure 20. Initial estimate of booster station spacing as a function of sediment size

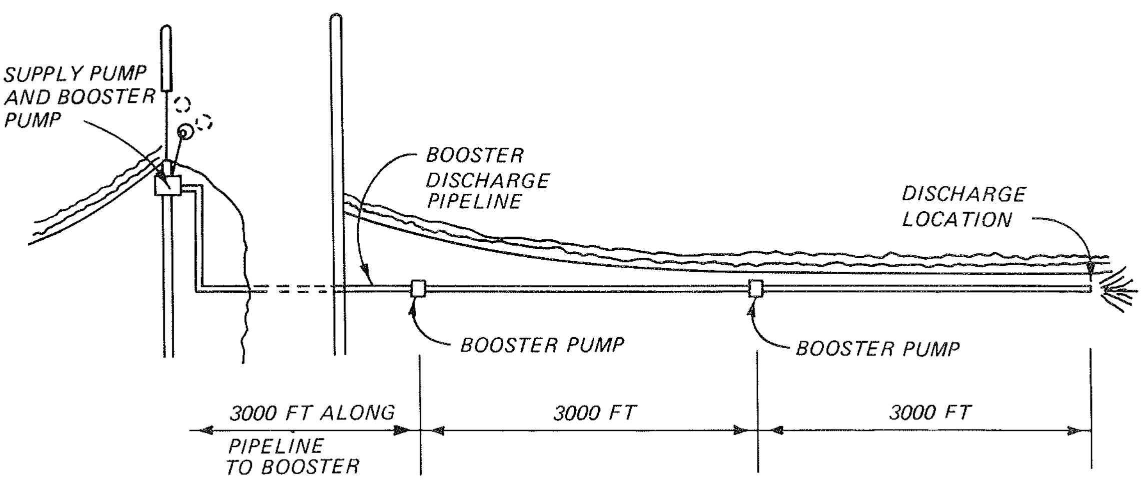

Figure 21. Example arrangement of multiple boosters on 9000-ft-long discharge line for 0.20-mm sand

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The basic schedules that should be considered are:

> Daily: Operation for a regular period each day on a 5- or 6-day-per-week basis.
>
> Intermittent: Operation only when conditions are such that bypassing is needed. This means that the system may be idle for days at a time and may be operated 24 hours a day during other periods.

Either of the above two schedules can be employed year-round, or only on a seasonal basis, or as a mixture of the two. For instance, it might be decided to operate a system on a daily basis during times of the year when drift rates are high, and on an intermittent basis at other times.

45. The total operating time is the number of hours per year when personnel are present and the system could be in operation. The effective operating time is defined as the number of hours per year that the bypassing system is actually pumping sand. The effective time of operation can be approximated by estimating the number of working days per year, multiplying by the operating hours per day to get the total operating hours per year, and correcting this value for work or operational interruptions. A system of correction factors is presented below to assist in determining an effective operating time.

46. The correction factor for interruptions due to system repair and replacement is applicable to both daily and intermittent operation but should be less for the latter. Intermittent operation should allow a higher level of preventive maintenance to be performed, reducing the frequency of repairs. No standard correction factor is available to apply to jet pump bypassing systems, but a reduction in total operating time of 10 to 15 percent was observed during the WES research program and appears to be a reasonable long-term average for daily operation. The system should perform with less downtime during early life, but may have more during later years.

47. Other work interruptions take many forms, but the most prevalent are jet pump suction blockages, temporary lack of littoral materials, and need to relocate mobile jet pumps. The reduction factor for suction blockages should be greater for sites with a high number of

shell fragments or cobbles or where sea grasses or debris are present. Mobile jet pumps also call for a higher correction factor than do fixed pumps. Factors for pump blockages should range from 5 percent for fixed pumps in relatively clean sandy shoals to 10 percent for mobile pump assemblies in clean sandy shoals to 20 percent for pumps in areas that have a high shell or cobble content or other debris. In general, fixed pumps are more susceptible to blockage by moving causes (seaweed, debris, etc.) while mobile pumps are affected by in situ causes (cobbles, for instance) and to a lesser degree by moving causes. A high incidence of blockages from in situ and moving causes may make frequent relocation of a mobile pump necessary.

48. Temporary lack of littoral materials is a situation that primarily affects fixed jet pump assemblies. In theory, a properly planned bypassing system will have littoral materials available for pumping at one location or another in the bypassing area throughout the period constituting the total operating time. However, there may be occasions when transport and deposition patterns and rates change enough to "starve" the system of littoral material. There is no set way of estimating this effect, but a factor of 5 percent might be applied to areas with known transport and deposition anomalies.

49. Relocation of mobile pump assemblies for reasons exclusive of suction blockages is a variable dependent on pumping rate, depth to which craters are dug, and rate of littoral material influx. When the pumping rate and the littoral influx rate to the crater are similar, the pump may require only occasional repositioning. When the influx rate is low and crater depth shallow, however, frequent movement of the pump may be necessary. Obviously, this is a highly variable situation, but a factor of 10 percent might be applied to mobile assemblies under average conditions as a first guess. A high anticipated incidence of pump movement might increase this factor to 15 or 20 percent.

50. The Effective Operating Time, EOT , in hours per year for the bypassing system can be determined from the relationship:

$$EOT = (NOD \times HD)\left[1.00 - (RR + PB + ALM + RMP)\right] \qquad (5)$$

where

NOD = number of operating days per year

HD = number of working hours in an operating day

RR = correction factor for system repair and replacement ÷ 100

PB = correction factor for pump blockages ÷ 100

ALM = correction factor for absence of littoral materials ÷ 100

RMP = correction factor for relocation of mobile pump assemblies ÷ 100

51. Proper application of the factors presented here should result in a realistic estimate of the effective operating time for the bypassing system. This analysis shows that the effective operating time could be as little as 50 percent or less of the total operating time. If the effective operating time seems to be relatively low, it should be remembered that many dredging systems have similar characteristics. Hopper dredges may spend a large amount of time in transit to and from the discharge site, while cutter suction dredges have to cease operation to periodically move swing wire anchors or to allow vessels to pass in a navigation channel. If the dredging site is subject to significant wave action, floating dredge downtime may be further increased.

System Capacity

52. The short term relationship between the rate of littoral influx, the production capacity of the bypassing system, and the available storage volume at a site is given by the expression

$$Q_L(\Delta t) - STORE_{\Delta t} = EXC(EOT_{\Delta t}) \qquad (6)$$

where

Q_L = average rate of net littoral influx to storage area(s) during interval Δt , cu yd/hr

Δt = a time interval of the bypassing season, hr; should be as short as possible consistent with available data

$STORE_{\Delta t}$ = storage volume available during interval Δt , cu yd

EXC = required capacity of bypassing system, cu yd/hr

$EOT_{\Delta t}$ = system effective operating time over interval Δt, hr

53. $STORE_{\Delta t}$ is not a fixed quantity, but is dependent upon factors such as the condition of the storage area at the start of Δt. Storage of littoral material in one form or another takes place at all littoral barriers. Often, it is this very storage or a portion of it that makes bypassing necessary. Evaluation of storage for bypassing purposes is complicated by the fact that only a part of the total storage at a site may be available for transfer by a bypassing system. Material stored in locations not reachable by the system cannot be included in the system analysis. Also, because of the variation of littoral rates during the year, the system may temporarily remove all the material within the effective storage area. This temporary removal may result in deepening of the adjacent bottom and localized slumping into pumping areas. This readjustment of the adjacent area must occur without endangering nearby structures.

54. For purposes of the initial layout, areas that are potentially within reach of a bypassing system should be identified as storage areas. As a rule of thumb, all areas below mean high water and located such that the total length of the jet pump discharge line will not exceed 600 to 700 ft can be considered to be potentially within reach of a jet pump bypassing system. The figure of 600 to 700 ft is given here for initial estimating purposes, not as an absolute limit. The range of mobile jet pumps will be determined more by the practical limitations of handling long reaches of floating pipelines and may therefore be less unless special measures are taken. For fixed jet pumps, where floating components are not a problem, the range of the system is a function of hydraulic and power considerations only. In most cases, however, the designer will find rapidly increasing power requirements if the jet pump discharge lines become too long. Areas such as jetty fillets, offshore bars, shoals, and prepared impoundment areas should be considered for use as potential storage areas. Past surveys of these areas together with estimates from the coastal processes study of transport vectors should be analyzed to determine: (a) that littoral transport does in

fact enter each area, and (b) what deposition patterns occur and how these patterns change with time. Only after this sort of analysis has been performed can a reasonable value of storage capacity (the amount of material that the area is capable of retaining at any one time) be determined.

55. Q_L as used in Equation 6 is an average net influx from all directions. In other words, it is the average rate at which littoral material moves into the storage area and remains there during Δt. The maximum value for Q_L is the average gross transport rate from all directions into (but not out of) the storage area during Δt. Use of this value would be based on the assumption that the storage area "traps" all littoral material which moves into it. In some instances, total trapping may not be the case, and a portion of the gross influx will continue on through the storage area. This might occur during periods of high wave activity or strong currents. The portion that moves through would not be available for bypassing; consequently, less storage and/or a smaller system production capacity would be indicated. Operation of the bypass system in the storage area will affect the value chosen for Q_L. Usually, the trapping capability of the storage area will be increased due to the craters formed by the system jet pumps. The amount of this increase will depend on variables such as the number, size, location, and condition of the craters.

56. An earlier section of this report (paragraph 50) outlined how to determine EOT, the Effective Operating Time per year. The same techniques can be applied to determining $EOT_{\Delta t}$ for a particular time interval.

57. Once $STORE_{\Delta t}$, Q_L, and $EOT_{\Delta t}$ have been determined, Equation 6 can be used in a rearranged form to determine various values of EXC, the system production capacity:

$$EXC = \frac{Q_L(\Delta t) - STORE_{\Delta t}}{EOT_{\Delta t}} \quad (7)$$

Equation 7 can be incorporated into many possible schemes for arriving at a final design value of EXC . The following approach is given only as an example of such a scheme:

a. For a Δt corresponding to a particular time interval of the year, a value or possible range of values for Q_L is determined from the coastal processes study, keeping in mind the discussion in paragraph 53.

b. $EOT_{\Delta t}$ is calculated, using the considerations described in paragraphs 44-51. Several values may be possible.

c. The storage capacity of the storage area(s) is determined (see paragraphs 53 and 54). If the storage capacity varies during the year, this effect should also be taken into account, since this calculation is for a particular time of the year.

d. Reasonable estimates are made of what range of initial conditions might exist in the storage area(s) at the beginning of the Δt interval. The bounds of this range are that the storage area is either completely full or completely empty. However, the actual condition or range of conditions will probably lie in between these bounds.

e. A range of $STORE_{\Delta t}$ values is calculated, based upon the range of initial conditions determined in paragraph *d*. For any particular initial condition,

$$STORE_{\Delta t} = STCAP - STIN \quad (8)$$

where

STCAP = storage capacity of storage area, cu yd

STIN = initial condition of storage area; i.e., volume of material already there at beginning of interval Δt , cu yd

f. Reasonable combinations of Q_L , $EOT_{\Delta t}$, and $STORE_{\Delta t}$ are determined. Although a range of values for each variable may have been identified, it does not necessarily follow that each value within the range of a particular variable can occur in combination with all values of the other two variables. Also, some combinations of the three variables may be more likely to occur than others.

g. Equation 7 is solved using the combinations determined in paragraph *f*. The values of EXC thus calculated are recorded.

h. Steps *a* through *g* are repeated for other Δt intervals occurring at other times of the year. The result is a number of possible values for EXC . From these, a design value or set of values is chosen to use in sizing the system components. The criteria for choosing a value or values for EXC will have to be determined by the designer. One criterion, and possibly a wasteful one in terms of system construction costs, would be simply to use the largest value of EXC . Another method might involve constructing a frequency distribution graph of EXC , plotting the relative frequencies of occurrence of classes of EXC values based on data from the above calculations. Then, from this graph, some value or values of EXC would be chosen corresponding to predetermined frequency criteria. Inherent in such a method is the understanding that on occasion, the storage capacity of the system will be exceeded.

58. Again, it is emphasized that the preceding approach is given only as an example. The important aspect of this section of the report is to understand the basic concepts of effective operating time and storage, and how their interaction with littoral drift rates should determine the design capacity of the system.

Considerations for Interception Type of System

59. Occasionally, the designer may find that little or no storage capacity is available at the site, and that the bypass system must function by intercepting sand which is in continuous motion. In this situation, of course, the concept of storage does not apply in determining a design value of EXC . The relationship expressed in Equation 6 then becomes simply:

$$Q_L(\Delta t) = EXC(EOT_{\Delta t}) \tag{9}$$

60. Q_L as used in Equation 9 takes on a somewhat different character than for Equation 6. None of the littoral material remains in the vicinity of the system; it is either all caught by the jet pump(s) or else moves past the system and is gone, presumably forever. Therefore, Q_L is now simply the rate at which littoral material approaches the system during Δt . It is imperative that Q_L values be determined for Δt intervals which are as small as possible. The ideal

situation would be to have estimates of hourly rates of Q_L . The major concept here, however, is that the bypass system capacity is a direct function of the littoral drift rate for an interception type of system.

Distribution of System Capacity

61. The number of jet pumps to be operated simultaneously should be decided in the initial system layout. Factors such as the system mode of operation, littoral transport vectors, and required system capacity must be considered. The size of jet pumps employed in the system will have an influence as well. WES experience thus far with jet pump bypassing systems has indicated that two sizes of the Pekor center-drive jet pump* have pumping capacities which match the requirements of many bypassing situations. The manufacturer's designation of these sizes, the 4 × 4 × 6 and the 6 × 6 × 8, describes the nominal inside diameters of the suction, mixing chamber, and discharge, respectively. For each pump, the approximate range of pumping capacity suggested for use in this report is shown below.

Jet Pump	Pumping Capacity, cu yd/hr
4 × 4 × 6	Up to 100
6 × 6 × 8	Up to 200

62. The tabulated values given are intended only as guidelines and not guarantees. The actual pumping capacity depends on a number of site-specific conditions. Simply because a pump is capable of moving, say, 100 cu yd/hr, does not mean that it will be able to pump at that rate in a given situation. Final determination of jet pump size based on site conditions will be made in PART III of this report. However, the tabulated values will allow the designer at this stage of design to identify available options in the number of simultaneously operating jet pumps. For example, if a design value for EXC of 140 cu yd/hr has been selected, the designer might choose to operate one 6 × 6 × 8 jet pump at

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* Manufactured by Pekor Iron Works, Columbus, Georgia.

a time with a capacity of 140 cu yd/hr. Or, he might elect to operate two 4 × 4 × 6 jet pumps simultaneously, each pumping 70 cu yd/hr. Depending on circumstances at the site, he might even investigate using three or more 4 × 4 × 6 jet pumps at a time, each with the appropriate pumping rate. KEEP IN MIND, HOWEVER, THAT THE BETTER SYSTEM WILL USUALLY HAVE THE LEAST NUMBER OF JET PUMPS OPERATING AT ONE TIME. IF THE JOB CAN BE DONE WITH FEWER JET PUMPS, DON'T USE MORE.

63. Once the number of simultaneously operating jet pumps has been determined, the required capacity of a single jet pump, EXC1 , should be estimated. This is actually a complex task if done rigorously, involving a series of iterative calculations for a pipeline network. One of the characteristics of a multiple jet pump system is that all other variables being equal, the capacity of each jet pump relative to the others will vary inversely with jet pump discharge pipeline length. For example, in Figure 22, jet pump #1 would have a greater capacity

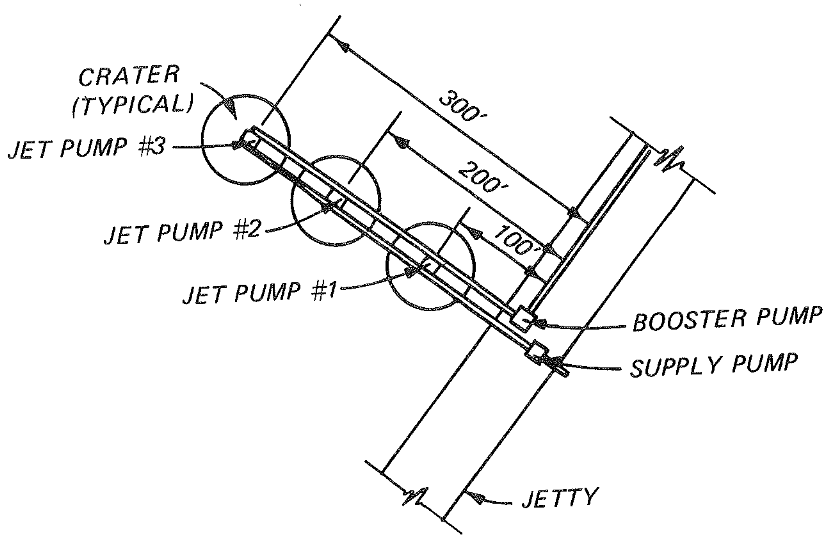

Figure 22. Example jet pump discharge pipeline lengths, multiple jet pump system

than #2, which in turn would pump more than #3. EXC1 can be approximated from the following simple relation:

$$EXC1 = \frac{EXC}{NUM} \tag{10}$$

where NUM is the number of jet pumps operated simultaneously. However, the designer must remember that in a multiple system with different jet pump discharge pipeline lengths, it is extremely difficult to obtain exactly the same output from each jet pump. Also, it may be necessary, due to local transport vectors, for one jet pump to have a capacity in excess of EXC1 as calculated above. The point of all this is that a value for EXC1 should be chosen not calculated for a multiple jet pump situation, taking local conditions and requirements into account. In PART III of this report, the designer will be shown how this chosen value is used to arrive at a design for the complete multiple system.

Backflushing

64. Backflushing, an operational technique peculiar to jet pumps, must be discussed in the initial layout phase of system design, since it influences the layout and selection of discharge pipelines in systems with more than one jet pump. For a jet pump operating with a certain supply flow rate QSUP , if the flow resistance in the jet pump discharge pipeline is increased, the suction flow rate QSUC will decrease. If the discharge pipeline flow resistance is increased enough, QSUC will become zero. If an "infinite" discharge flow resistance is created by closing a valve in the jet pump discharge pipeline, the jet pump supply water will flow out the suction of the jet pump (Figure 23). This property of a jet pump can be useful in clearing the suction of blockages due to shells, debris, etc. Operationally, such a technique is called "backflushing," and the valve in the jet pump discharge pipeline is called the "backflush valve."

65. The influence of backflushing requirements on discharge pipeline layout is illustrated by Figures 24 and 25. Each figure shows a system with two jet pumps operated simultaneously. In Figure 24, the jet pumps share a common discharge pipeline, while in Figure 25 they have separate pipelines that join just before the booster pump. The common discharge pipeline in Figure 24 has one backflush valve, while each pipeline in Figure 25 has its own.

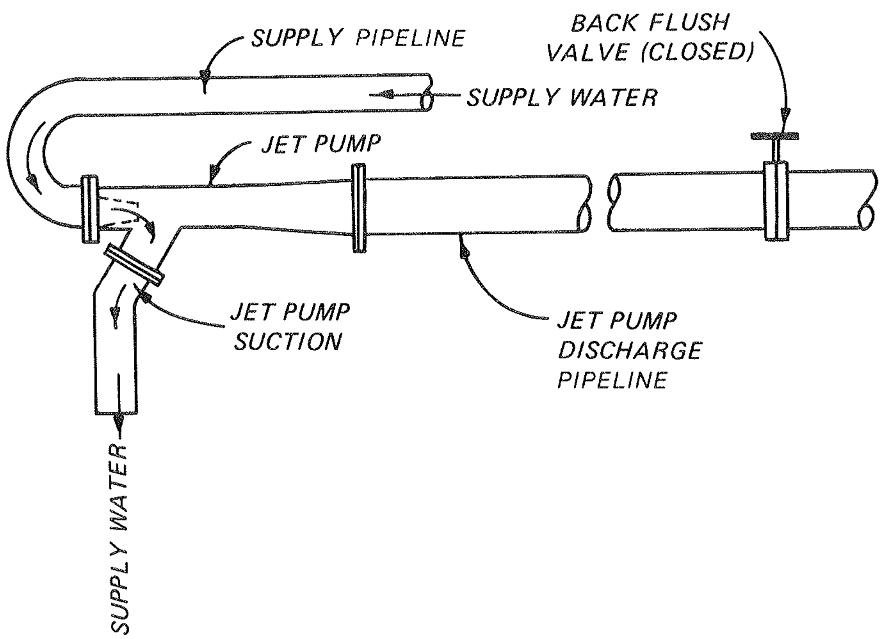

Figure 23. Jet pump backflushing

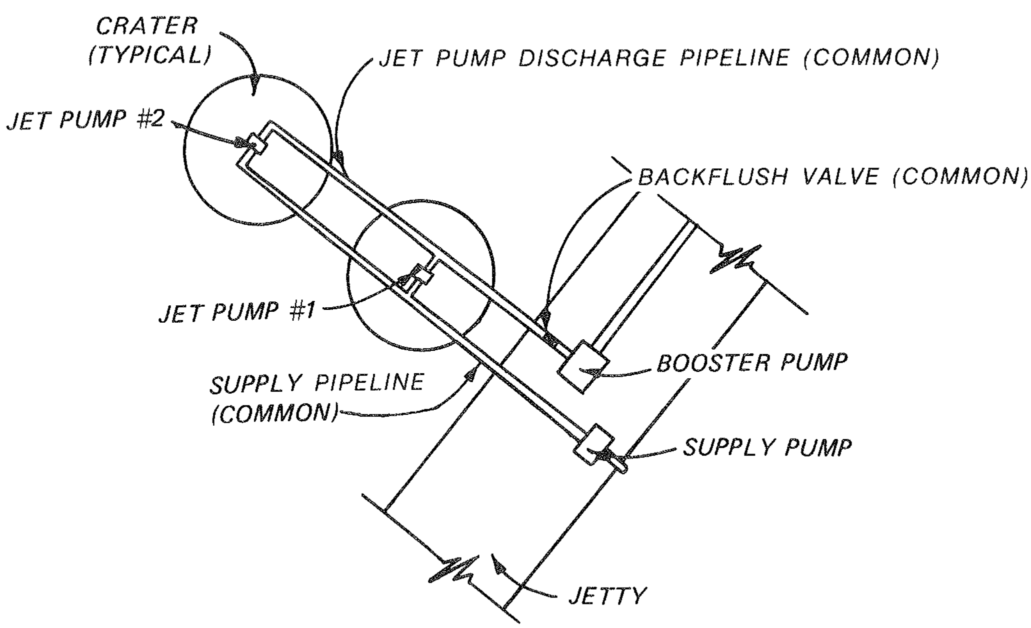

Figure 24. Two jet pumps with common discharge pipelines

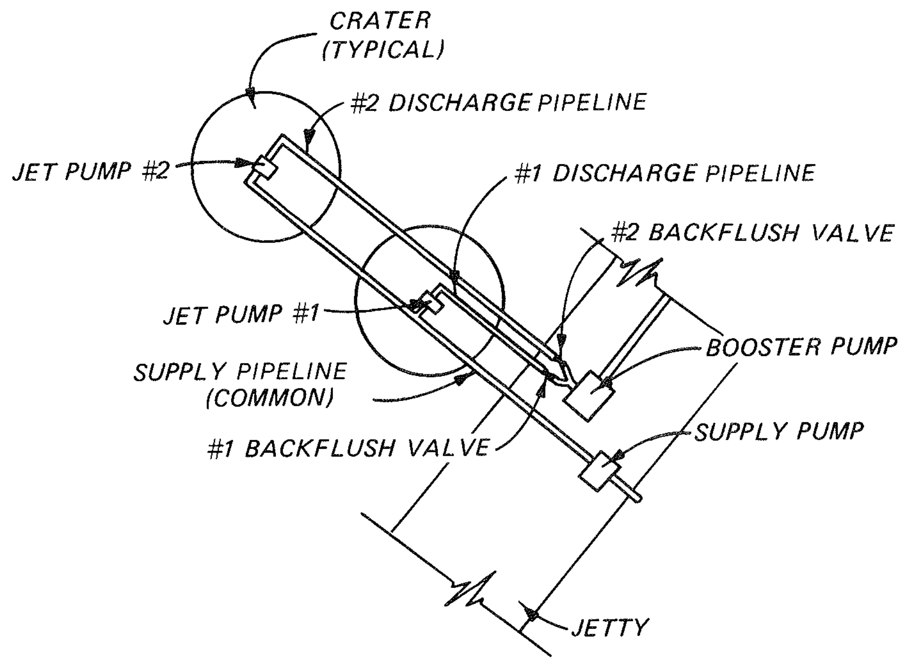

Figure 25. Two jet pumps with separate discharge pipelines

66. In the Figure 24 system, closure of the backflush valve will cause both jet pumps to backflush, even if only one has a suction blockage. There may also be a tendency for more flow to be directed outward through the unblocked suction since the hydraulic resistance there is less. In the Figure 25 system, each jet pump can be backflushed independently. However, more discharge piping and backflush valves are required and system installation may be more difficult.

67. A backflushing capability is necessary for all mobile jet pumps, since they often develop suction blockages when excavating new craters. Fixed jet pumps, on the other hand, are subject to blockage mainly by objects falling into their craters. Backflushing will remove these objects from the jet pump suction, but may not remove them from the crater. If they remain in the crater, they will eventually reenter the jet pump suction. It is usually necessary, therefore, to provide some type of coarse screen around the suction of a fixed jet pump to prevent large objects from entering. Then, the question of how to provide a backflushing capability will be answered by the degree of

discharge pipeline complexity the designer is willing to introduce.

System Pipe Sizes and Materials

68. The initial system layout must include a "first guess" at the size and material for the following pipelines:

a. Jet pump supply.

b. Supply pump suction.

c. Jet pump discharge.

d. Booster pump discharge.

Actually, several alternates should be developed at this stage for each of the above pipelines. These alternates should be carried through the design procedure in PART III as parallel calculations. Then, the relative effects of each will be apparent, and the most suitable combinations of size and material can be chosen.

69. The following general guidelines may be followed in choosing initial pipe sizes for single jet pump systems. These guidelines apply only to pipe with inside dimensions corresponding to Schedule 40 specifications. For other types of pipe, the nominal sizes given below may not be acceptable. The pipe sizes given are for initial estimating purposes only and may be changed in the final design, based on the results of PART III calculations:

a. Jet pump supply pipeline.

(1) 4 × 4 × 6 jet pump - 6 in. for line lengths up to 500 ft; 8 in. for greater lengths.

(2) 6 × 6 × 8 jet pump - 8 in. for line lengths up to 500 ft; 10 in. for greater lengths.

b. Supply pump suction pipeline. At least one pipe size larger than the jet pump supply pipeline.

c. Jet pump discharge pipeline.

(1) The best that can be done at this stage in the design process is to choose a jet pump discharge pipeline size which falls in the middle of a range of design possibilities. The approach used consists of choosing a pipe size based on the calculated value of EXC1 , then adjusting the jet pump discharge pipeline length so that the combination of EXC1 ,

pipe size, and pipeline length fall within a predetermined range of values. The designer may later have to revise his initial layout following more detailed calculations in PART III.

(2) The designer should enter Figure 26 using the value of EXC1 from paragraph 63 and move vertically to the first line encountered corresponding to a particular pipe size, which then becomes the "first guess" at the jet pump discharge pipeline size. Then, the approximate center of the range of pipeline lengths possible for EXC1 and that pipe size can be found from the vertical axis. If the value from Figure 26 is greater than 700 ft, the designer may lengthen the pipeline to the new value, if necessary. If the Figure 26 value is substantially less than the value used up to now, the designer must shorten the jet pump discharge pipeline to the new value. If the discharge pipeline cannot be shortened to the new value, the designer may move upward on Figure 26 to the line corresponding to the next larger pipe size. This pipe

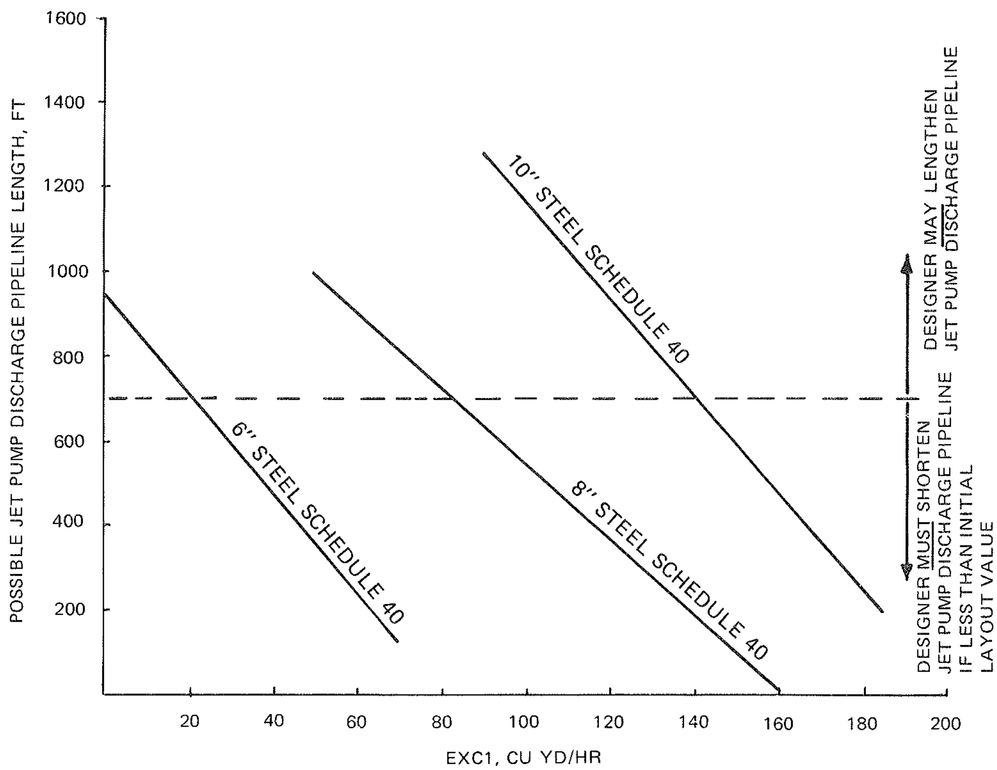

Figure 26. Possible jet pump discharge pipeline lengths versus EXC1 for various pipe sizes

size then becomes the "first guess" in subsequent calculations. The designer should resist the urge to use longer or larger pipelines than necessary, since the system will be more expensive to build and operate. Also, the designer should not use 6-in. pipe with a 6 × 6 × 8 jet pump or 10-in. pipe with a 4 × 4 × 6. If the discharge pipeline length is changed, the designer should review all aspects of the initial layout to see whether other changes are necessary. In particular, if the area of influence of the jet pump(s) is altered, a new system capacity will have to be calculated.

d. Booster pump discharge pipeline. Same size as jet pump discharge pipeline, or possibly one pipe size larger if flow in the jet pump discharge pipeline is near maximum for that pipe size.

For multiple jet pump systems that use common supply and discharge pipelines to serve all simultaneously operating jet pumps, pipe sizes for items a and c should be chosen such that their inside areas are roughly the appropriate multiple of the areas for the sizes shown above for single systems. For instance, if a 6-in. supply pipeline would be chosen for a certain single jet pump system, then an 8-in. pipeline is indicated for a two-jet pump system with the same pipeline lengths (the inside area of an 8-in. pipe is slightly less than twice that of a 6-in. pipe). The instructions given above for items b and d apply to multiple jet pump systems as well.

70. An initial selection of pipe materials should be made at this stage, so that the complete hydraulic characteristics of the pipelines will be known for PART III. Although different materials will have different hydraulic characteristics, no adjustments for pipe material (such as changing pipeline lengths) should be made to the initial system layout at this stage. Some options that may be considered and that have performed satisfactorily in WES field tests are:

a. Jet pump supply pipeline.

(1) Steel - For fixed jet pumps.

(2) Flexible rubber hose - For mobile jet pumps.

(3) High density polyethylene (HDPE) - For fixed jet pumps where the pipe will not have to support any external load, such as the weight of the jet pump.

b. Supply pump suction pipeline.

(1) Steel.

(2) HDPE.

c. Jet pump discharge pipeline. Same as jet pump supply pipeline.

d. Booster pump discharge pipeline.

(1) Steel.

(2) HDPE.

Other Considerations

71. Two other items must be considered by the designer before proceeding with the system hydraulic design: (a) booster pump flushing water and (b) jet pump cutting jets. Flushing water is clear water (i.e. no solids) which is continually provided to the booster pump stuffing box to prevent solid particles from entering. The flushing water must be at a pressure greater than the discharge pressure of the booster pump. The system supply pump may be able to provide flushing water to a nearby booster pump if the supply pump discharge pressure is sufficiently greater than that of the booster pump. If this arrangement is used, the supply pump will have to be sized to provide this additional flow.

72. As described previously, cutting jets are often used around the jet pump suction to aid in suction burial and in excavating consolidated material. Mobile jet pumps should always be provided with cutting jets, due to the range of conditions they often encounter. For fixed jet pumps, cutting jets may or may not be needed depending on the size and characteristics of the material that enters the jet pump crater. Fine sand that tends to pack quickly may require some cutting action to loosen it. Coarse, well-graded sand, on the other hand, may flow easily under the influence of suction alone. The point is that for fixed jet pumps, the question of whether to provide cutting jets should be answered by sediment information gathered in the coastal processes study. The water for cutting jets has to be provided by the supply pump (Figure 7), imposing an additional flow requirement on it.

System Schematic Drawings

73. The final aspect of initial system layout is the preparation of system schematic drawings in both plan and elevation. Schematic drawings should show the following information as a minimum:

a. Major system components (jet, supply, and booster pumps and structures).
   - (1) Location.
   - (2) Elevation.

b. Pipelines.
   - (1) Routing.
   - (2) Length.
   - (3) Size.
   - (4) Material.

c. Valves and pipe fittings (bends, reducers, etc.).
   - (1) Location.
   - (2) Size.
   - (3) Type.

d. Craters.
   - (1) Location.
   - (2) Dimensions.

e. Jet pump excavation rate(s).

74. A set of example schematic drawings for a mobile jet pump system with two jet pumps, one of which operates at a time, is shown in Figures 27-31.

Equivalent Lengths

75. A convenient method to account for hydraulic energy losses caused by bends, valves, or other fittings in a pipeline is to replace these fittings in the calculations with lengths of straight pipe which give the same energy losses as the fittings. These "equivalent lengths" of straight pipe are added to the actual length of the pipeline. Then, a calculated loss factor is applied to the total equivalent pipeline

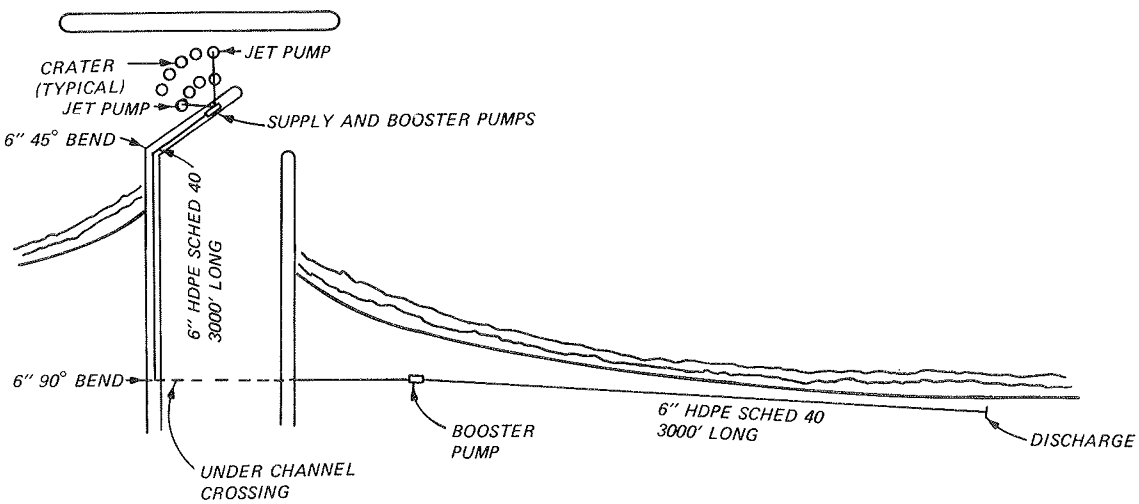

Figure 27. Overall system schematic

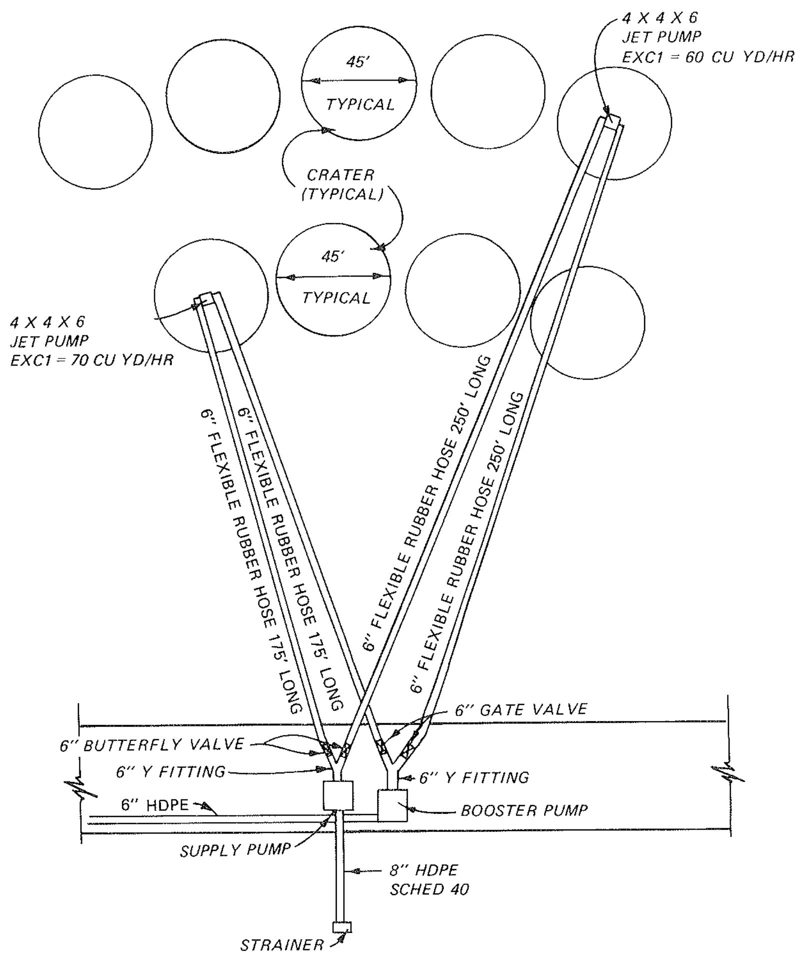

Figure 28. Detailed schematic - jet, supply, and booster pumps (plan view)

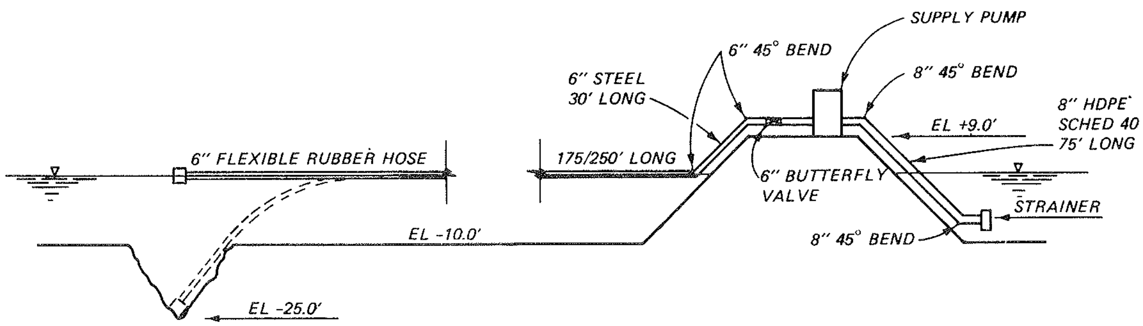

Figure 29. Jet pump supply system (elevation view)

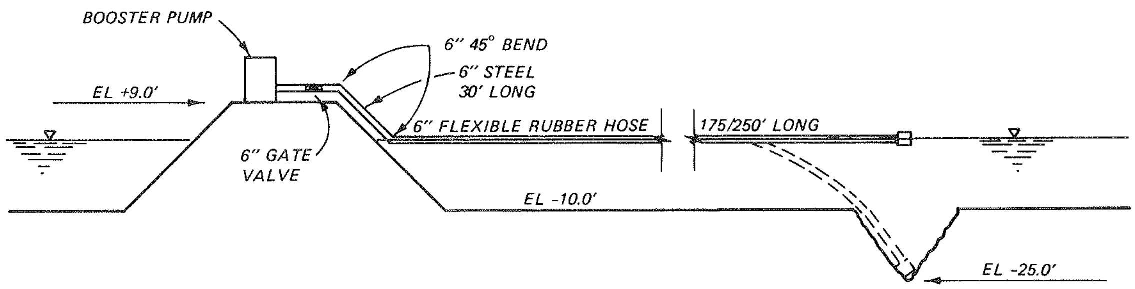

Figure 30. Jet pump discharge system (elevation view)

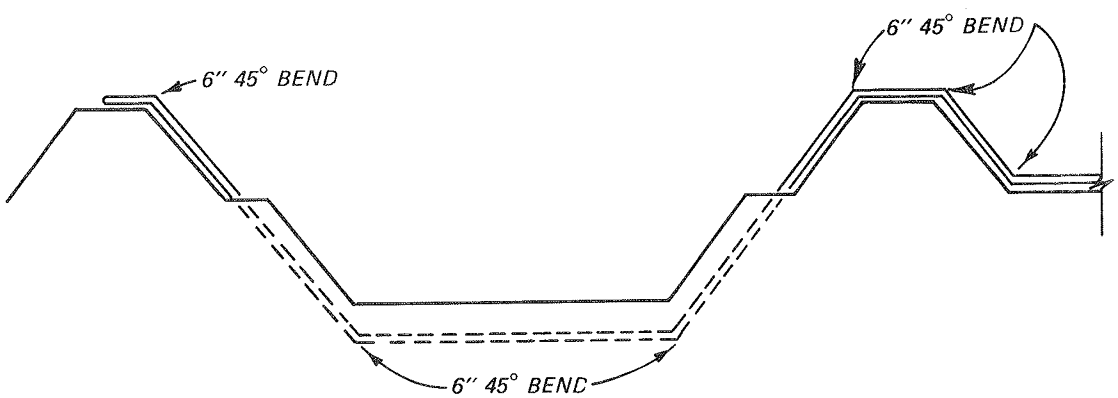

Figure 31. Booster discharge pipeline underchannel crossing (elevation view)

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length (actual plus fitting equivalents), giving the total system energy loss in one computation.

76. Tables giving equivalent lengths of steel or cast iron pipe for different types and sizes of fittings are in many hydraulic handbooks (see Hydraulic Institute Standards, 1965, p E(I)-7). For fittings and pipe materials not included in such tables, Appendix B suggests some approaches to be used. The tabulation below shows an example of equivalent length calculations for the supply pump suction pipeline shown in Figures 28 and 29.

Type of Fitting	Number of Fittings	Equivalent Length per Fitting, ft	Total ft
Strainer	1	35.6	35.6
45-deg bend	2	7.7	15.4
Straight pipe	--	--	75.0
		Total equivalent length	126.0

PART III: HYDRAULIC DESIGN

77. Two separate procedures are described in this section that can be followed in designing jet pump systems for pumping sand. These procedures are presented in step-by-step fashion. In some steps, it is necessary that judgment be applied, and explanatory text is included to guide the designer. The knowledge or information that is necessary to perform the calculations or to make the decisions required by each step is specified where possible. A simplified flow chart is included so that the logical structure of the hydraulic design process can be followed.

78. The first procedure consists of a series of well-defined calculation steps. Included in the procedure is an iteration loop to determine values of the jet pump operating parameters. This procedure can be partially adapted to a programmable desk calculator or a computer. Such adaptations are not included in this report.

79. The second procedure utilizes graphs of jet pump performance together with discharge head curves generated by the designer to arrive at values of the operating parameters. The major advantage of this procedure is that it allows the designer to consider the entire range of jet pump possibilities at once, without having to recalculate design values. Also, jet pump cavitation limits are built into the graphs and values of efficiencies are shown.

80. The basic jet pump system for pumping sand consists of a centrifugal pump to provide supply water, a jet pump, a booster pump, and the interconnecting pipelines with valves and fittings. Either of the design procedures that follow will result in the following information:

- a. Values of operating parameters for:
  - (1) Jet pump.
  - (2) Supply pump.
  - (3) Booster pump.
- b. Flow rates and velocities at all points in system.
- c. Values of energy losses in all pipelines.
- d. Percent solids pumped at design operating point.
- e. Methods for selecting:

(1) Supply pump.

(2) Booster pump.

81. The given procedures are for the basic single jet pump system with one booster pump. Procedures for systems with multiple jet and/or booster pumps are similar except for some additional hydraulic considerations. These will be discussed at the end of PART III.

82. The design calculations make use of the initial layout and schematics of the bypassing system generated in PART II. If the design calculations show that a system element is inadequate or produces inefficient operation, a more reasonable selection should be inserted into the layout together with other associated corrections. By this method, the designer is assured of considering the bypassing system as an entity rather than as a group of components.

83. In both design procedures described in this part of the report, the total system design develops as a function of the jet pump design operating point. A major consideration in choosing the jet pump design operating point is efficiency of the jet pump at that point. Since the other demands on the jet pump at that point (excavation rate, pumping distance, etc.) resulted directly from project considerations discussed in PART II, such an approach will usually produce a well-designed system. However, the designer must remember that overall system efficiency, i.e., accomplishing the required bypassing with the least amount of total energy, is what really matters. Jet pump efficiency is a major factor in overall system efficiency, but the supply and booster pump operating characteristics play a role as well. Therefore, the designer should consider several alternative designs and compare the projected energy consumptions for each.

Iteration Design Procedure

84. The following design calculations are presented as a number of discrete steps. At the beginning of each step the information necessary to perform the calculations is given. The necessary relationships and equations are either presented in the step or are specifically

referenced. Some steps are check points and may show that reselection of system components and recalculation of new system values are necessary. These steps are specifically identified. A detailed flow chart showing relationships between steps as well as decision points in the design procedure is presented in Plate 1.

Step 1: Objective - Determine the minimum discharge pipeline velocity, often called critical velocity, necessary to maintain solids in suspension for all pipelines with solids flow. Also, determine the representative settling velocity of the sediment particles to be bypassed. This calculation should be done for each size of pipe used in the sediment-carrying portion of the system.

Information required - (a) Pipe inside diameter, D , ft; (b) median diameter of sediment to be bypassed, d_{50} , mm; and (c) specific gravity of the sediment solids, SGSOL (for quartz sand, SGSOL = 2.65).

Method - The first sediment parameter that will be determined is the settling velocity, W , of the d_{50} sediment particle. W can be determined from the curves in Figure 32, which shows plots of both empirical data and equations from several investigators for quartz particles settling in water at 68°F. For $d_{50} < 0.6$ mm , the center of the plots may be used. For $d_{50} > 0.6$ mm , the plots diverge rapidly, indicating that variables such as the particle shape become more important. In this range of particle sizes, it is suggested that plot 9 be used, since it is based on data using naturally occurring particle shapes. It should be noted that d_{50} is expressed in millimetres and W in millimetres per second in Figure 32. The value of W from Figure 32 should be multiplied by 0.00328 to convert it to feet per second for use in subsequent calculations.

The following empirical relationship from Durand (1953) is suggested as a means of determining the minimum discharge pipeline velocity, VCRIT :

$$VCRIT = F_L \sqrt{[2gD(SGSOL - 1)]} \tag{11}$$

F_L , a proportionality coefficient, can be determined from Figure 33. C_V in Figure 33 is the expected volumetric concentration of solids in the discharge pipeline. It is suggested at this stage in the calculations to use the curve marked $C_V = 15\%$ as a conservative value.

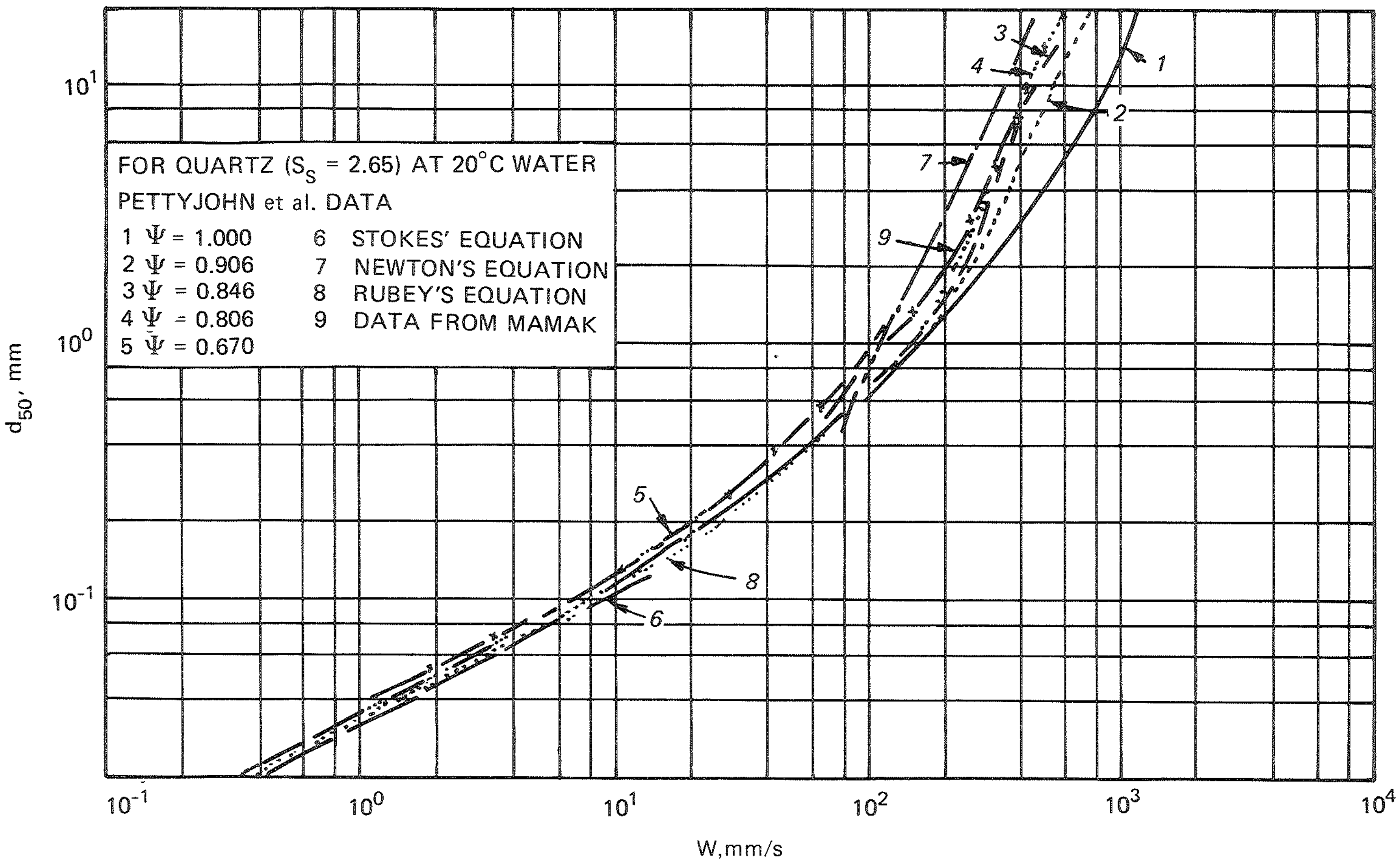


Figure 32. Settling velocity of quartz particles (from Hydraulics of Sediment Transport by Graf (1971). ©1971 by McGraw-Hill Inc. Used with permission of McGraw-Hill Book Company)

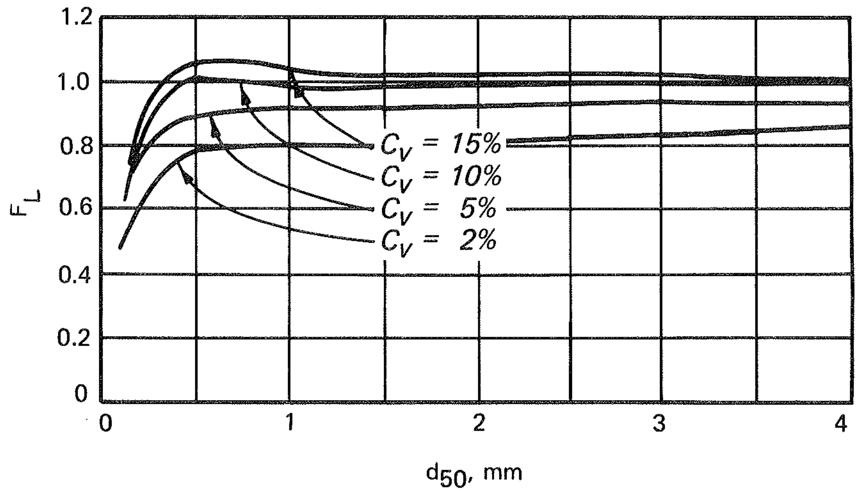


Figure 33. F_L versus d_{50} for Durand relationship (from Hydraulics of Sediment Transport by Graf (1971). ©1971 by McGraw-Hill Inc. Used with permission of McGraw-Hill Book Company)

Step 2: Objective - Determine the minimum required volumetric flow rate, $QSUP_{min}$, in the system discharge pipelines to assure conveyance of solids in suspension.

Information required - (a) VCRIT from Step 1, fps; (b) discharge pipe maximum inside area, ADIS , ft^2. ADIS should be the inside area of the largest pipe in the sediment-carrying portion of the system.

Rationale - The flow rate calculated in this step is the minimum clear water flow rate that should be supplied to the jet pump nozzle. Such a minimum flow rate will help ensure that sediment in the discharge pipelines can still be carried in suspension even if the jet pump intake plugs completely.

Method - Perform the following calculation to obtain $QSUP_{min}$ in gallons per minute (gpm):

$$QSUP_{min} = (448.831)(ADIS)(VCRIT) \qquad (12)$$

Step 3: Objective - Determine the jet pump suction flow necessary to produce the required project site bypassing rate.

Information required - (a) Individual jet pump required bypassing rate, EXC1 , cu yd/hr; (b) in situ porosity, n , of sediment to be bypassed; (c) specific gravity, SGSOL , of solids.

Rationale - The mixture entering the jet pump suction is called suction flow, and the volumetric rate of entry is termed QSUC . This mixture is composed partly of fluid and partly of solid sand particles. By determining the specific gravity of the in situ material at the site and making certain assumptions as to the specific gravity of the entering mixture, the value of QSUC can be determined.

Method - The required jet pump suction flow, QSUC , in gallons per minute, can be calculated from

$$QSUC = EXC1\ (3.37) \left(\frac{SGIN - SGWAT}{SGSUC - SGWAT}\right) \qquad (13)$$

where

SGIN = in situ material specific gravity = SGSOL(1 - n) + n(SGWAT)

SGWAT = specific gravity of ambient water (1.00 for fresh water, 1.025 for seawater)

SGSUC = assumed average specific gravity of mixture entering jet pump suction

In WES field tests, SGSUC varied between about 1.40 and 1.85 depending on pumping conditions. It is suggested that a value of SGSUC = 1.70 be used for preliminary estimation.

Step 4: Objective - Select a jet pump size to use in subsequent calculations.

Information required - Value of QSUC , gpm, from Step 3.

Rationale - Two sizes of the Pekor center-drive jet pump, the 4 × 4 × 6 and the 6 × 6 × 8, have pumping capacities that match the requirements of many bypassing situations. The same dimensionless performance characteristics are assumed for each, but flow rates in the larger pump are higher. It is therefore necessary to know which pump is being considered for steps later in this procedure.

Method - Compare the value of QSUC with those given below:

QSUC Range, gpm	Jet Pump
200 to 500	4 × 4 × 6
500 to 700	Transition
700 to 1500	6 × 6 × 8

Select the jet pump corresponding to the correct range of QSUC. For values of QSUC in the transition range either jet pump may prove the most feasible, depending on other system requirements. It is suggested that both types be considered simultaneously.

If QSUC from Step 3 is less than 200 gpm, the designer should consider at this point whether such a small capacity is correct, and if so, why. Since QSUC is a direct function of EXC1 (Equation 13), and the value of EXC1 for a given EXC depends only on NUM, the number of jet pumps operated simultaneously (Equation 10), it may be that QSUC is too small because NUM is too large. Or, EXC may be too small. The designer should return to the initial layout to check these possibilities. If such is not the case, the designer should reevaluate the entire situation to make sure the problem is of sufficient magnitude to warrant a sand bypassing system.

If QSUC is larger than 1500 gpm, the designer should also recheck his determination of EXC, NUM, and EXC1. If the value chosen for EXC still appears reasonable, the only option left is to increase NUM, the number of jet pumps operated simultaneously. As a general guideline, if adding more than one additional jet pump to NUM is required to make QSUC less than 1500 gpm, the designer should at this point reconsider the entire system layout. If the layout still looks sound, then there is a good possibility that the bypassing problem is beyond the feasible range of a jet pump solution. THE DESIGNER MUST RESIST THE TEMPTATION OF FORCING A JET PUMP SYSTEM TO FIT THE PROBLEM. For many situations, a jet pump system is not practical. Results of this step may be indicating just that.

Step 5: Objective - Determine the flow ratio M, the head ratio N, and the area ratio R, of the jet pump.

Information required - (a) $QSUP_{min}$, gpm, from Step 2, (b) QSUC, gpm, from Step 3, (c) jet pump dimensionless performance curves, Plate 2.

Rationale - As discussed earlier (paragraphs 20-22), the behavior of jet pumps of a given design when pumping a given medium can be described by three dimensionless ratios. These are:

$$\text{Head Ratio } N = \frac{HDIS - HSUC}{HSUP - HDIS} \quad (1 \text{ bis})$$

$$\text{Flow Ratio } M = \frac{QSUC}{QSUP} \quad (2 \text{ bis})$$

$$\text{Area Ratio } R = \frac{ANOZ}{AMIX} \quad (3 \text{ bis})$$

As part of the WES research program "Eductor Systems for Sandtrap Bypassing," the relationships between these ratios were defined for a Pekor center-drive jet pump under the condition of pumping medium sized sand (d_{50} = 0.5 mm). These relationships are shown graphically in Plate 2. It is evident from the plots shown in Plate 2 that if two of the ratios are selected, the third is uniquely defined. Since the effects of different grain sizes on M vs N relationships have not been measured, it is suggested that the plots in Plate 2 be used only for naturally occurring beach sands (d_{50} = 0.1 to 1.0 mm). More detailed performance data for a particular beach sand or data for coarser or finer sediments should be obtained by pump testing.

The operating efficiency, E , of the jet pump can be defined at any point from the relationship

$$E = M \times N \tag{14}$$

Peak operating efficiency is a goal of any design procedure and is a consideration in the accomplishment of this step.

Method - The optimum* flow ratio, M_{op} , can be found from the previous calculations of drive water flow rate requirements and jet pump suction flow rate requirements. Therefore

$$M_{op} = \frac{QSUC}{QSUP_{min}} \tag{15}$$

Enter Plate 2 at M_{op} and trace vertically to the jet pump dimensionless performance curve which gives the largest possible value of head ratio, N_{max} . Note the area ratio, R , associated with this curve. Determine the operating efficiency, E , of the jet pump at that point from the relation

$$E = M_{op} \times N_{max} \tag{16}$$

If E is approximately 0.20 or greater, the jet pump operating ratios have been selected. If E is between 0.14 and 0.20, the designer must choose whether to use

---

* This flow ratio is "optimum" in the sense that it derives directly from bypassing requirements and the minimum flow rate in the discharge line. "Optimum" in this context does not necessarily imply a degree of efficiency.

the M and N values just calculated, or try for a higher E value.

If E is less than 0.14, compare the value of M_{op} with values of M in the following tabulation corresponding to the proper R . If M_{op} is greater than the value from the tabulation, the value of $QSUP_{min}$ should be increased and the operating ratios recalculated. If M_{op} is less than the tabulated value, QSUC should be increased for recalculating the operating ratios. Once new ratios are calculated, use them to recalculate E . Continue this process until an acceptable value of E is found.

R	M
0.096	1.063
0.138	0.745
0.175	0.614
0.202	0.537
0.246	0.463
0.311	0.205

Step 6: Objective - Calculate the discharge flow from the jet pump, QDIS .

Information required - Jet pump suction flow, QSUC , and supply flow, QSUP , gpm, used in the final determination of M in Step 5.

Rationale - Flow rates in various segments of the discharge system must be calculated in determining the system design operating point. The input to the calculations of this step should be the rate of flow of the mixture entering the jet pump through the suction tube, QSUC , and the dimensionless flow ratio, M . These values must reflect any adjustments made in Step 5.

Method - The volumetric discharge flow rate from the jet pump, QDIS , can be calculated from the relationship

$$QDIS = QSUC + QSUP \quad (17)$$

Step 7: Objective - Calculate the expected maximum volumetric concentration of solids, CVMAX , in the jet pump discharge pipeline.

Information required - (a) Jet pump flow ratio, M , from Step 5; (b) specific gravity of sediment solids, SGSOL;

(c) in situ specific gravity, SGIN , of sediment to be bypassed (from Step 3); (d) jet pump discharge, QDIS , gpm, from Step 6; (e) porosity, n , of in situ sediment to be bypassed; (f) specific gravity of ambient water, SGWAT , from Step 3.

Rationale - Head loss calculations should be based on the maximum slurry concentration conditions that are reasonably expected to occur. The WES research program indicated certain relations that could be used for different jet pump configurations to estimate what the maximum sustained specific gravity of the mixture entering the jet pump might be. These relations are used in the following calculations.

Method - CVMAX can be determined from the relationship

$$\mathrm{CVMAX} = \left(\frac{\mathrm{M}}{1 + \mathrm{M}}\right)\left(\frac{\mathrm{SGSUCM} - \mathrm{SGWAT}}{\mathrm{SGSOL} - \mathrm{SGWAT}}\right) \quad (18)$$

where SGSUCM is the assumed maximum sustained specific gravity of the slurry entering the jet pump suction. Experiments at WES developed the following relationships between SGSUCM and SGIN :

For fixed jet pumps with no cutting assists:

$$\mathrm{SGSUCM} = \mathrm{SGIN} \quad (19)$$

For fixed jet pumps with cutting assists:

$$\mathrm{SGSUCM} = 0.85(\mathrm{SGIN}) + 0.15 \quad (20)$$

For floating jet pumps, which almost always require cutting assists:

$$\mathrm{SGSUCM} = 0.80(\mathrm{SGIN}) + 0.20 \quad (21)$$

Note - The solids concentration determined in this step is to be used for subsequent calculation of energy losses and booster pump requirements only. No attempt should be made to return to previous steps with this value since the bypassing system will not consistently attain solids concentrations as high as those calculated in this step.

The expected maximum specific gravity of slurry in the jet pump discharge pipeline, SGDISJ , can be computed from:

$$SGDISJ = CVMAX(SGSOL) + (1 - CVMAX)SGWAT \tag{22}$$

The maximum excavation rate of in situ material by one jet pump, EXCMAX , cu yd/hr, can be calculated by the expression

$$EXCMAX = \frac{0.297(CVMAX)}{1 - n}(QDIS) \tag{23}$$

EXCMAX as given by this expression should exceed EXC1 as used in Step 3. If it does not, a mistake was made somewhere in Steps 3 through 7. Calculations in these steps should be checked.

Step 8: Objective - Calculate the required jet pump discharge head, HDIS .

Information required - (a) Description of jet pump discharge pipeline including inside diameter and length (D and LDISJ), both in feet, with adjustments for valves, fittings, and bends; (b) hydraulic characteristics of jet pump discharge pipe; (c) total flow rate delivered by jet pump, QDIS , gpm, from Step 6; (d) settling velocity, W , of d_{50} particle diameter, fps (see Step 1); (e) maximum concentration of solids, CVMAX , in jet pump discharge pipeline; (f) specific gravity of sediment solids, SGSOL ; (g) elevation of booster pump center line, ZBOO , ft, relative to water surface datum; (h) maximum specific gravity of slurry in jet pump discharge pipeline, SGDISJ , from Step 7; (i) expected maximum water depth over jet pump while operating, DEPMAX , ft; (j) design pressure or vacuum head at the booster pump suction flange, PHSUCB , in feet of water. Suggestions for selecting a value of PHSUCB are given in this step.

Rationale - The jet pump discharge head represents the total energy output of the jet pump. It is composed of the energy required to overcome friction losses in the discharge pipeline, of velocity head energy, of energy required to raise the mixture to the level of the booster pump, and of the pressure or vacuum at the booster pump. Friction losses in pipelines carrying fluids and solids are calculated in two steps. First, losses attributable to the fluid flow alone are calculated. Then, these losses are adjusted to account for the presence of the solids. Equivalent lengths of pipeline should be used in the calculations.

Fluid energy losses: the Darcy-Weisbach formula is often used to calculate the rate of head loss per unit length of pipeline for fluid flow, $(\Delta h/\Delta L)_w$:

$$\left(\frac{\Delta h}{\Delta L}\right)_w = \frac{f}{D}\frac{V^2}{2g} \qquad (24)$$

where

f = dimensionless friction factor; a function of pipe relative roughness and Reynolds number

D = pipe inside diameter, ft

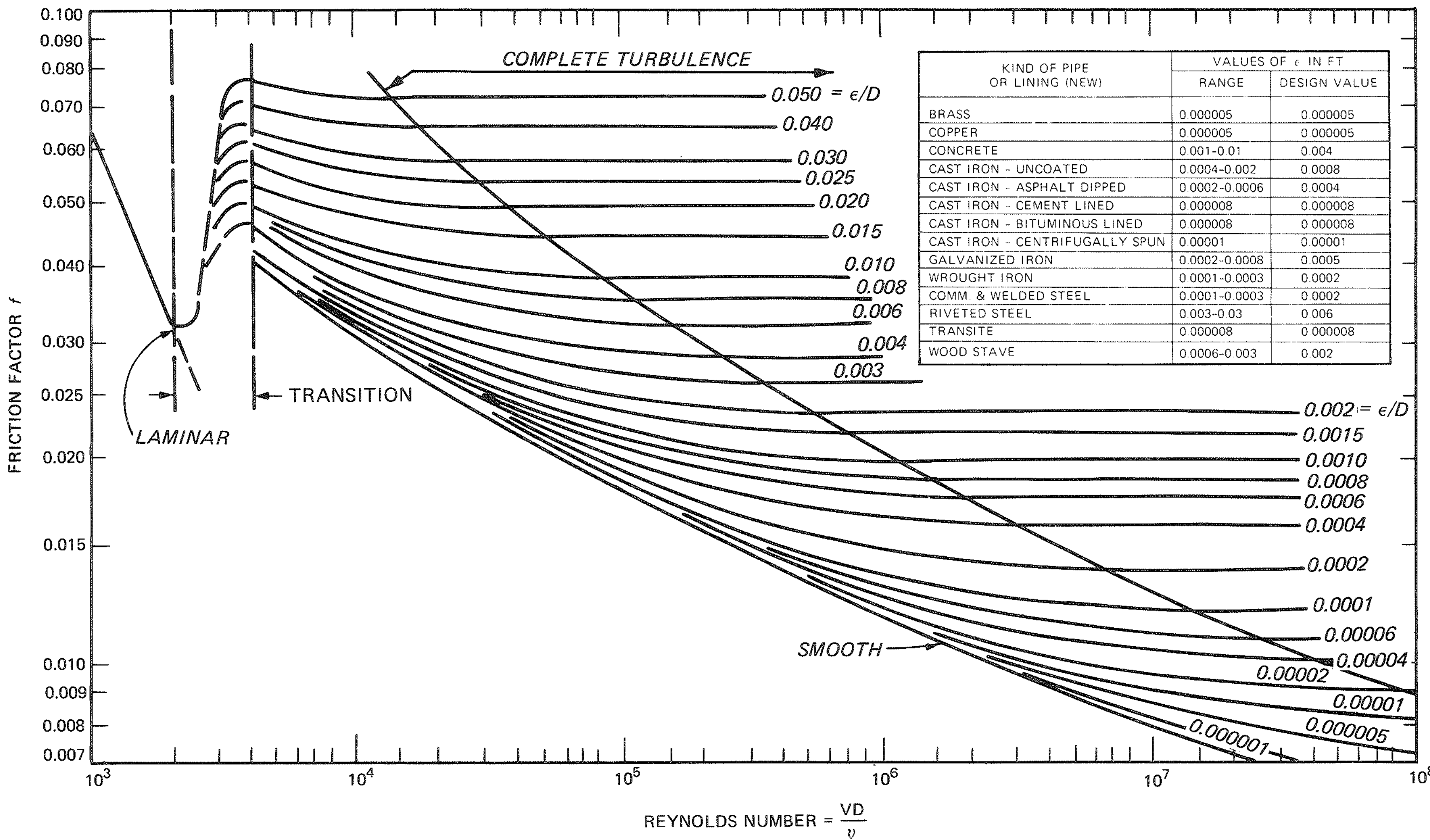
V = flow velocity in the pipe, fps

g = acceleration due to gravity, ft/sec^2

The friction factor, f , can be found from a Moody diagram such as that shown in Figure 34 (Moody 1944). f can also be found directly by an iterative solution of the Colebrook-White equation (Colebrook 1939). This method is described in Appendix B. Other relationships such as the Hazen-Williams formula are available to define fluid head loss rates, and the appropriate one should be selected by the designer on the basis of what information is available on the hydraulic characteristics of the pipe being used.

A word of caution is necessary at this point against using so-called "rules of thumb" for estimating head loss rates or any other hydraulic parameter. While such methods may indeed give answers approximating those of more complex formulas, their range of validity is often not known. At the very least, their use can result in wasteful overpowering of a system. In the worst case, they may give results which underestimate head losses, causing the system to be inadequate.

Mixture energy losses: the flow regime existing in the booster discharge pipeline must be determined before adjustments are made to fluid-only head loss rates to account for the presence of solids. The piping system has been designed to carry the mixture in a nonsettling mode, so the only test necessary is whether the mixture is a heterogeneous or homogeneous slurry. A heterogeneous slurry has a vertical concentration gradient in the pipe; i.e., more solid material is carried at the bottom of the pipe than at the top. In a homogeneous slurry, velocities in the pipe are high enough that solids are distributed



KIND OF PIPE OR LINING (NEW)	VALUES OF ε IN FT	
	RANGE	DESIGN VALUE
BRASS	0.000005	0.000005
COPPER	0.000005	0.000005
CONCRETE	0.001-0.01	0.004
CAST IRON - UNCOATED	0.0004-0.002	0.0008
CAST IRON - ASPHALT DIPPED	0.0002-0.0006	0.0004
CAST IRON - CEMENT LINED	0.000008	0.000008
CAST IRON - BITUMINOUS LINED	0.000008	0.000008
CAST IRON - CENTRIFUGALLY SPUN	0.00001	0.00001
GALVANIZED IRON	0.0002-0.0008	0.0005
WROUGHT IRON	0.0001-0.0003	0.0002
COMM & WELDED STEEL	0.0001-0.0003	0.0002
RIVETED STEEL	0.003-0.03	0.006
TRANSITE	0.000008	0.000008
WOOD STAVE	0.0006-0.003	0.002

Figure 34. Moody diagram (after Moody 1944)

more or less evenly over the pipe cross section. Head loss rates are much greater in the homogeneous flow range. Therefore, it is more efficient to size the discharge piping system to produce heterogeneous flow. The transition velocity between heterogeneous and homogeneous flow, VHOM , fps, can be found from:

$$VHOM = \sqrt[3]{1800\ gWD} \tag{25}$$

where D is the pipe inside diameter, ft, and W is the settling velocity of the sediment particles, fps.

Now, the mixture head loss per unit length, $(\Delta h/\Delta L)_m$, can be determined. For the situation

$$V \geqq VHOM \text{ (homogeneous regime):}$$

$$\left(\frac{\Delta h}{\Delta L}\right)_m = \left(\frac{\Delta h}{\Delta L}\right)_w [C_v(SGSOL - 1) + 1] \tag{26}$$

where the subscripts m and w refer to mixture and water, respectively, and C_v is the volumetric solids concentration in the mixture.

For the situation

$$VCRIT < V < VHOM \text{ (heterogeneous regime):}$$

$$\left(\frac{\Delta h}{\Delta L}\right)_m = \left(\frac{\Delta h}{\Delta L}\right)_w \left\{\frac{C_v}{(V)^3} [1100(SGSOL - 1)WgD] + 1\right\} \tag{27}$$

Equations 25 and 27 are attributable to Newitt et al. (1955). Equation 26 is from Graf and Acaroglu (1967).

<u>Method</u> - To apply the above calculation procedure to the jet pump discharge pipeline, substitute VDIS for V . VDIS is the velocity in the jet pump discharge pipeline, fps:

$$VDIS = \frac{QDIS}{(448.831)(ADIS)} \tag{28}$$

where ADIS is equal to the jet pump discharge pipe cross-sectional area, ft^2.

Also, substitute the jet pump discharge pipeline inside diameter, ft, for D and CVMAX from Step 7 for C_v in order to determine the unit mixture head loss from either Equation 26 or 27. Then, the total head loss in the jet pump discharge pipeline, HMJ , in feet of water, is calculated by the expression:

$$HMJ = \frac{\Delta h}{\Delta L_m} \; LDISJ \tag{29}$$

where LDISJ is the total equivalent length of the jet pump discharge pipeline, ft.

Consideration should be given at this point to the choice of PHSUCB , the estimated design pressure or vacuum at the booster pump suction flange. Choosing PHSUCB to be a vacuum will result in a smaller required HDIS from the jet pump, which will significantly decrease the jet pump supply water requirements. However, such a choice will increase the possibility of cavitation and water hammer at the booster pump when the jet pump suction becomes plugged or if the system as installed has different hydraulic characteristics from the design. A vacuum may also cause air to be drawn into the system through pipe joints or other openings, creating a loss in booster pump efficiency and adding to cavitation and water hammer problems. A compromise might be to choose PHSUCB as a mild pressure, say +10 ft, for the initial design and then recheck the design later for the effects of different flow possibilities.

The required jet pump discharge head, HDIS , in feet of water, can now be calculated from the expression

$$HDIS = HMJ + \frac{VDIS^2}{2g} + DEPMAX(SGDISJ - SGWAT) + SGDISJ(ZBOO) + PHSUCB \tag{30}$$

Step 9: Objective - Calculate the jet pump suction head, HSUC .

Information required - (a) Jet pump suction flow, QSUC , gpm, as used in Step 6; (b) length of jet pump suction tube, LSUC , ft; (c) inside area of jet pump suction, ASUC , ft^2.

Rationale - The jet pump suction head represents the total energy available at the jet pump suction. This value is composed of the velocity head of the suction fluid minus the head losses of the sediment/water mixture as it enters and flows through the suction tube.

Method - The jet pump suction head, HSUC , in feet of water, can be calculated from the expression

$$HSUC = \frac{VSUC^2}{2g} - \left[2(LSUC) + 4\left(\frac{VSUC^2}{2g}\right)\right] \qquad (31)$$

where

$$VSUC = QSUC/(ASUC \times 448.831) \text{ , in fps}$$

The expression in brackets in Equation 31, developed from laboratory observations, is an estimate of the total mixture head loss in the suction tube. Detailed information on design of the suction tube will be given in a subsequent report. A value of LSUC = 2.0 ft is suggested for the present design purpose.

Step 10: Objective - Calculate the required jet pump supply head, HSUP .

Information required - (a) Jet pump head ratio, N (see Step 5); (b) required jet pump discharge head, HDIS (see Step 8), in feet of water; (c) jet pump suction head, HSUC (see Step 9), in feet of water.

Rationale - The dimensionless jet pump head ratio, N , is a function of the jet pump supply head, the jet pump discharge head, and the jet pump suction head (Equation 1). Therefore, definition of any three of these parameters uniquely defines the fourth.

Method - The required jet pump supply head, HSUP , in feet of water, can be calculated from the expression

$$HSUP = \frac{HDIS - HSUC}{N} + HDIS \qquad (32)$$

Step 11: Objective - Calculate QSUP from the standpoint of nozzle hydraulics, to determine whether the value of $QSUP_{min}$ calculated in Step 2 is realistic.

Information required - (a) Jet pump supply head, HSUP , in feet of water; (b) area ratio, R , from Step 5; (c) jet pump suction head, HSUC , from Step 9, in feet of water.

Rationale - This step is the "closing" step of an iterative calculation procedure, the objective of which is to determine the operating parameters of the jet pump. The jet pump operates by means of supply water entering through a nozzle. Therefore, its behavior on the supply side can be characterized by a form of the basic nozzle equation. When the results of calculations from the nozzle equation are approximately the same as those arrived at via Steps 1 through 10, then the values of operating parameters determined in Steps 1 through 10 can be considered valid.

Method - Calculate QSUP , gpm, by means of the following equation, which is conservatively based on the results of laboratory tests of jet pumps:

$$QSUP = B(ANOZ)\sqrt{(HSUP - HSUC)} \qquad (33)$$

ANOZ can be calculated from the following relation:

$$ANOZ = R \times AMIX \qquad (34)$$

The tabulation below gives values of AMIX for Pekor 4 × 4 × 6 and 6 × 6 × 8 jet pumps:

Jet Pump	AMIX, ft^2
4 × 4 × 6	0.0873
6 × 6 × 8	0.1963

B is a coefficient which varies with the value of R . The following tabulation gives values of B to use with corresponding R values.

R	B, $gpm/ft^{5/2}$
0.096	3533.4
0.138	3429.7
0.175	3633.9
0.202	3776.5
0.246	3682.5
0.311	4544.8

Compare this value of QSUP with the value of QSUP used in the final determination of M in Step 5. If they are not within approximately 5% of each other (and they probably will not be), the iterative calculation procedure must be worked again. Replace the Step 5 QSUP with the value of QSUP calculated in this step, return to Step 5, and work through to this point again. Several runs through Steps 5-11 may be necessary until the two values of QSUP agree. After the first iteration, the value of QSUP calculated in this step is no longer compared with the original Step 5 QSUP, but with the value of QSUP calculated in the previous iteration. Caution: if at any time the value of QSUP calculated in this step is less than the Step 5 QSUP, the iteration must be carried out differently. Either the excavation rate, EXC, or the jet pump discharge pipe diameter can be reduced. The first adjustment will have the effect of reducing QSUC; the second will cause a reduction in QSUP. In either case, the designer must return to the system initial layout.

Step 12: Objective - Check for possible cavitation in jet pump.

Information required - (a) jet pump supply head, HSUP (see Step 10), in feet of water; (b) jet pump suction head, HSUC (see Step 9), in feet of water; (c) jet pump suction velocity, VSUC (see Step 9), fps; (d) jet pump area ratio, R (see Step 5); (e) jet pump flow ratio, M (see Step 5); (f) minimum anticipated water depth over jet pump while operating, DEPMIN, ft; (g) atmospheric pressure, ATMOS, in feet of water; (h) vapor pressure of water, VAP, in feet of water; (i) jet pump supply flow rate, QSUP, gpm, as finally determined in the Step 2 through 11 iteration process; (j) area of opening at jet pump nozzle tip, ANOZ, from Step 11, in ft^2.

Rationale - The exact prediction of cavitation in the Pekor jet pump, especially when pumping solids, has not been experimentally determined at this time. For this reason, it is recommended that the designer check the jet pump operating point as determined in Steps 5-11 against cavitation criteria taken from Silvester and Mueller (1968) and Wakefield (1972). Although those criteria do not apply directly to the Pekor pump, they will serve to indicate whether the operating point is near a "danger zone" of possible cavitation.

Method - The criterion from Silvester and Mueller is expressed in the terminology of this report as:

$$\frac{\text{HSUP} + \text{DEPMIN}}{\text{ATMOS} - \text{VAP} + \text{HSUC} - \left(\frac{\text{VSUC}^2}{2g}\right) + \text{DEPMIN}} \le \left[\frac{0.95(1 - R)}{M \times R}\right]^2 \qquad (35)$$

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Cavitation is assumed not to occur if this inequality is satisfied.

The criterion as given in Wakefield's publication involves use of a graph. For purposes of this manual, the graph is eliminated and the following expression used:

$$\frac{\left(HSUC - \frac{VSUC^2}{2g} + DEPMIN + ATMOS\right)2g}{VNOZ^2} > 0.046 - 0.126X + 1.44X^2 + 4.44X^3 - 9.18X^4 \quad (36)$$

where

$X = VSUC/VNOZ$

$VNOZ$ = velocity of water jet at tip of nozzle, fps

Again, cavitation is assumed not to occur if the inequality is satisfied. VNOZ may be calculated from the following expression:

$$VNOZ = \frac{QSUP}{(448.831)(ANOZ)} \quad (37)$$

If cavitation is indicated by either of these criteria, the designer has two alternatives: (a) decrease QSUC or (b) select a larger size jet pump. From Equations 13 and 10, it is seen that QSUC is a direct function of EXC1 , and that EXC1 depends solely on EXC and NUM . Therefore, choosing alternative (a) means returning to the initial system layout to see what can be altered to reduce EXC1 and thereby decrease QSUC . Alternative (b) is feasible only if QSUC falls in the transition zone defined in the tabulation in Step 4, p 62. If alternative (b) is chosen, the designer should also return to the initial system layout to determine if changes in pipe sizes are necessary.

Step 13: Objective - Calculate all expected energy losses in the booster pump discharge pipeline.

Information required - (a) Description of booster discharge pipeline, including inside diameter and length, both in feet, with equivalent length adjustments for valves, fittings, and bends; (b) hydraulic characteristics of booster discharge pipe; (c) total flow rate delivered by jet pump to booster pump, QDIS (see Step 6), gpm, (d) settling velocity, W , of d_{50} sediment particle (see Step 1); (e) maximum concentration of solids, CVMAX , in jet pump discharge pipeline (see Step 7); (f) specific gravity of sediment solids, SGSOL .

Rationale - The methodology for calculating friction losses in pipelines conveying sand/water slurries was presented in Step 8 and will not be repeated here.

Method - The velocity in the booster pump discharge pipeline, VDISB , fps, should be calculated on the basis of the total discharge of the jet pump plus the quantity of flushing water introduced into the booster pump. Therefore

$$\text{VDISB} = \frac{\text{QDIS} + \text{QFL}}{\text{ADISB}(448.831)} \tag{38}$$

where

QFL = flushing water volumetric flow rate into booster, gpm

ADISB = inside area of booster pump discharge pipe, ft^2

Recommended values of QFL vary from pump to pump, but QFL may usually be assumed as 100 to 150 gpm. The expected maximum volumetric concentration of solids in the booster discharge pipeline, CVMAXB , may be calculated from:

$$\text{CVMAXB} = \text{CVMAX}\left(\frac{\text{QDIS}}{\text{QDIS} + \text{QFL}}\right) \tag{39}$$

Using the relations from Step 8, and by substituting VDISB for V , the booster discharge pipe inside diameter, ft, for D , and CVMAXB for C_V , the head loss per unit length of pipeline, $(\Delta h/\Delta L)_m$, can be calculated.

Total head loss in the booster discharge pipeline, HMB , in feet of water, is calculated by the expression

$$\text{HMB} = \left(\frac{\Delta h}{\Delta L}\right)_m \text{LDISB} \tag{40}$$

where LDISB is the total equivalent length, ft, of the booster discharge pipeline.

Step 14: Objective - Calculate the operating requirements to be placed on the booster pump. These requirements include the total dynamic head of the booster pump, TDHB , the flow rate of the booster pump, QDISB , and the specific gravity of the booster pump discharge, SGDISB .

Information required - (a) Total frictional head loss in booster discharge pipeline, HMB , in feet of water (see Step 13); (b) estimated design pressure or vacuum at the booster pump suction flange, PHSUCB , in feet of water (see Step 8); (c) elevation of the end of the booster discharge pipeline, ZDIS , relative to water-surface datum, ft; (d) elevation of booster pump center line relative to water surface, ZBOO , ft; (e) maximum concentration of solids in the booster discharge pipeline, CVMAXB (see Step 13); (f) jet pump discharge flow rate, QDIS , gpm (see Step 6); (g) flushing water flow into booster pump, QFL , gpm (see Step 13); (h) specific gravity of sediment solids, SGSOL ; (i) manufacturers literature describing booster pumps.

Method - The total dynamic head required of the booster pump, TDHB , is composed of the total head loss in the booster discharge pipeline, HMB , in feet of water; a design pressure or vacuum at the booster pump suction flange, HSUCB , in feet of water; and the difference in elevation between the center line of the pump and the discharge pipe end at the disposal site, ft. TDHB may be calculated in feet of water from:*

$$TDHB = HMB - PHSUCB + (ZDIS - ZBOO) \qquad (41)$$

Note - Kinetic (velocity) energy terms are not included in this calculation because their only contribution to TDHB is the negligible difference between velocity heads at the pump suction and pump discharge.

At this point, the designer should review the manufacturers' literature and make a preliminary selection of a class of booster pumps so that the reasonableness of the flushing water flow rate estimates, QFL , can be

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* Equation 41 must be modified for systems with multiple booster pumps. See paragraph 91 for details.

verified. If possible, a more accurate value of QFL should be selected for subsequent use. The volumetric discharge flow rate of the booster pump, QDISB, gpm, can be determined from the expression:

$$QDISB = QDIS + QFL \tag{42}$$

The maximum specific gravity of the booster pump discharge slurry, SGDISB, is calculated from the expression

$$SGDISB = CVMAXB(SGSOL) + (1 - CVMAXB) \tag{43}$$

Step 15: Objective - Calculate the required production flow rate of the clear water supply pump.

Information required - (a) Jet pump supply water flow rate, QSUP, gpm, as determined in Step 5; (b) flushing water flow rate required by booster pump, QFL, gpm (Step 13).

Rationale - The production flow rate of the clear water supply pump is determined principally by the jet pump supply water requirements. Added to this will be the total flow rate requirement for jet pump cutting jets, QJET, if such jets are used. Finally, it may be possible in some cases to provide flushing water for the booster pump from the clear water supply pump. In such cases, this flow rate must also be added to the required clear water supply pump flow rate. Before finalizing a design using such a flushing water system, however, it would be advisable to contact the booster pump supplier about his specific requirements.

Method - The value of QJET has not been determined, but as a general rule, the relationship

$$QJET = 0.2(QSUC) \tag{44}$$

will provide a realistic estimate.

The required total volumetric flow rate of the supply pump, QSUPT, is given by the expression

$$QSUPT = QSUP + QJET + QFL \tag{45}$$

Note - QJET and QFL should be included in this calculation only if they are to be provided by the supply pump. The ultimate feasibility of providing QFL with the supply pump will not be known until the supply pump discharge head is calculated and compared with the booster pump discharge head.

It should be noted that in most cases the quantity QFL will be included in the pump suction water, but will be removed and introduced into the booster pump very near the supply pump discharge flange. Therefore, for the purpose of calculation, QFL is included in supply pump suction flow but not in the flow between supply pump and jet pump.

Step 16: Objective - Calculate the required total dynamic head of the clear water supply pump.

Information required - (a) Inside areas of supply pump suction and jet pump supply pipes, ASUPS and ASUPD, ft^2; (b) inside diameters of supply pump suction and discharge pipes, ft; (c) equivalent lengths of supply pump suction and jet pump supply pipelines, LSUPS and LSUPD, ft; (d) description of hydraulic characteristics of supply pump suction and discharge pipes; (e) required total production flow rate of clear water supply pump, QSUPT, gpm (Step 15); (f) elevation of supply pump, ZSUP, above water-surface datum, ft; (g) jet pump supply head, HSUP (Step 10), in feet of water; (h) supply flow to the jet pump, QSUP, gpm (Step 11); (i) supply flow to jet cutting assists, QJET, gpm (Step 15); (j) expected flushing water requirement, QFL, gpm (Step 13).

Rationale - The total dynamic head, or total head as it may be called, of the clear water supply pump represents the energy imparted by the supply pump to the liquid. This energy can take the form of a change in elevation, velocity, or pressure of the liquid being pumped. The sum of these changes expressed in feet of water is, by definition, the total dynamic head of the pump. For a centrifugal pump supplying water to a submerged jet pump, the change in elevation of water passing through the centrifugal pump is zero. The change in velocity may be approximated by the velocity head in the centrifugal pump discharge pipe. The change in pressure is due to two factors: (a) the required supply head HSUP at the jet pump and (b) the total frictional losses in the centrifugal pump suction and discharge pipelines.

Method - Use the method described in Step 8 for calculating the rate of head loss per unit length of pipeline for fluid flow (the Darcy-Weisbach formula). This method should be applied separately to the centrifugal pump suction and discharge pipelines. Velocity in the suction pipeline, VSUPS , fps, can be calculated by:

$$VSUPS = \frac{QSUP + QJET + QFL}{(448.831)(ASUPS)} \qquad (46)$$

Velocity in the jet pump supply pipeline, VSUPD , fps, can be obtained from:

$$VSUPD = \frac{QSUP + QJET}{(448.831)(ASUPD)} \qquad (47)$$

As in Step 15, QJET and QFL should be included in these calculations only when they are to be provided by the supply pump. QFL is not included in VSUPD calculations since it is normally removed from the discharge pipeline shortly after leaving the supply pump.

Using the appropriate values of velocity, inside diameter, and hydraulic characteristics, head loss rates per unit length should be calculated for the suction pipeline $(\Delta h/\Delta L)_{WSS}$ and jet pump supply pipeline $(\Delta h/\Delta L)_{WSD}$. Then, the total head loss in the supply pump suction pipeline, HWSS , in feet of water, can be calculated by:

$$HWSS = \left(\frac{\Delta h}{\Delta L}\right)_{WSS} \times LSUPS \qquad (48)$$

An important adjustment to include in LSUPS is an allowance for entrance losses into the suction pipe.

The total head loss in the jet pump supply pipeline, HWSD , can be obtained in feet of water from:

$$HWSD = \left(\frac{\Delta h}{\Delta L}\right)_{WSD} \times LSUPD \qquad (49)$$

The required total dynamic head of the supply pump, TDHS , in feet of water, is then:

$$TDHS = HSUP + HWSS + HWSD \qquad (50)$$

Step 17: Objective - Calculate the available net positive suction head, NPSHA , at the supply pump suction flange.

Information required - (a) Head loss in supply pump suction pipeline, HWSS (Step 16), in feet of water; (b) atmospheric pressure, ATMOS , in feet of water; (c) water vapor pressure, VAP , in feet of water; (d) maximum expected elevation of supply pump suction center line above free water surface, ZSUPM , ft; (e) velocity in supply pump suction pipeline, VSUPS , fps (Step 16).

Rationale - The available net positive suction head represents the absolute pressure in the liquid at the supply pump suction flange above its vapor pressure. Different types of centrifugal pumps require different values of NPSHA at a given operating point in order to avoid cavitation within the pump itself. This required value is often denoted NPSHR . NPSHR varies for a given pump with the operating point.

Method - ZSUPM should take into account low tide, low lake levels, seiches, surges, and the change in water surface due to waves propagating past the suction pipe. In other words, ZSUPM should be the elevation, ft, of the supply pump suction center line above the lowest water level, transient or otherwise, that could reasonably occur during regular pumping operations. The net positive suction head available, NPSHA , of the supply pump can be calculated in feet of water from the expression:

$$\text{NPSHA} = \text{ATMOS} - \text{VAP} - \text{HWSS} - \text{ZSUPM} - \frac{\text{VSUPS}^2}{2g} \qquad (51)$$

Step 18: Objective - Select supply pump to drive jet pump system.

Information required - (a) The required total dynamic head of the supply pump, TDHS (see Step 16), in feet of water; (b) the required total flow of the supply pump, QSUPT , gpm (see Step 15); (c) the net positive suction head available at the supply pump suction flange, NPSHA (see Step 17), in feet of water; (d) manufacturers' literature for centrifugal water pumps.

Method - The required operating point of the supply pump has been defined through calculation of TDHS , QSUPT , and NPSHA . Various pump curves from several manufacturers should be examined to locate a pump that can produce the desired operating point for the least shaft horsepower input, and that has a required net positive suction head, NPSHR , at the operating point equal to or less than the value of NPSHA .

Selecting a centrifugal pump inherently involves selecting a particular impeller diameter for that pump. The electric motor or diesel engine used to drive the pump should have a continuous horsepower rating not less than the maximum horsepower that the pump with the selected size of impeller will draw at "runout," or the maximum flow rate shown on the pump curve for that size impeller. For pumps that will be primed by a vacuum method, consideration should be given to specifying mechanical stuffing box seals instead of packing to reduce air leakage into the pump during priming. A horizontally split-case type of pump should be specified wherever possible to facilitate access to the impeller for repair or replacement. Corrosion-resistant materials should be specified for use in saltwater environments.

Step 19: Objective - Select booster pump to deliver slurry to discharge site.

Information required - (a) Required total dynamic head of the booster pump, TDHB (see Step 14), in feet of water; (b) required total flow rate of the booster pump, QDISB , gpm (see Step 14); (c) specific gravity of booster pump mixture, SGDISB (see Step 14); (d) sediment d_{50} grain size, mm; (e) maximum concentration of solids in the booster discharge pipeline, CVMAXB (see Step 13); (f) manufacturers' literature on dredge pumps.

Rationale - The terminology "booster pump" has been used to describe the pump that has the responsibility of supplying the majority of energy needed to deliver the mixture of bypassed sand and water to the discharge site. In actuality, the pump selected will probably be a dredge pump, which is a centrifugal pump that is specifically designed with large internal clearances and specially hardened and strengthened parts to allow handling of solids. Selection of the dredge pump is essentially the same as selection of the clear water supply pump in Step 18. The difference is that the presence of solids in the dredge pump has the combined effects of reducing the efficiency of the pump while increasing the horsepower required to convey a certain flow rate at a certain discharge pressure. Dredge pump performance curves are usually given for clear water pumping. Therefore, the information available from these curves must be corrected to account for the effects of solids.

Method - The first step is to convert the total dynamic head of the booster pump, TDHB , which is in feet of water, into feet of mixture, TDHBM . This is done by using the relationship:

$$TDHBM = \frac{TDHB}{SGDISB} \qquad (52)$$

Next, the ratio of efficiency when pumping slurry, EMIX , to efficiency when pumping water, EW , is determined by entering Figure 35 with the sediment d_{50} grain size and determining the ratio of EMIX/EW that corresponds to CVMAXB . The total dynamic head is then corrected for this decrease of efficiency by the relationship:

$$TDHBME = \frac{TDHBM}{\frac{EMIX}{EW}} \qquad (53)$$

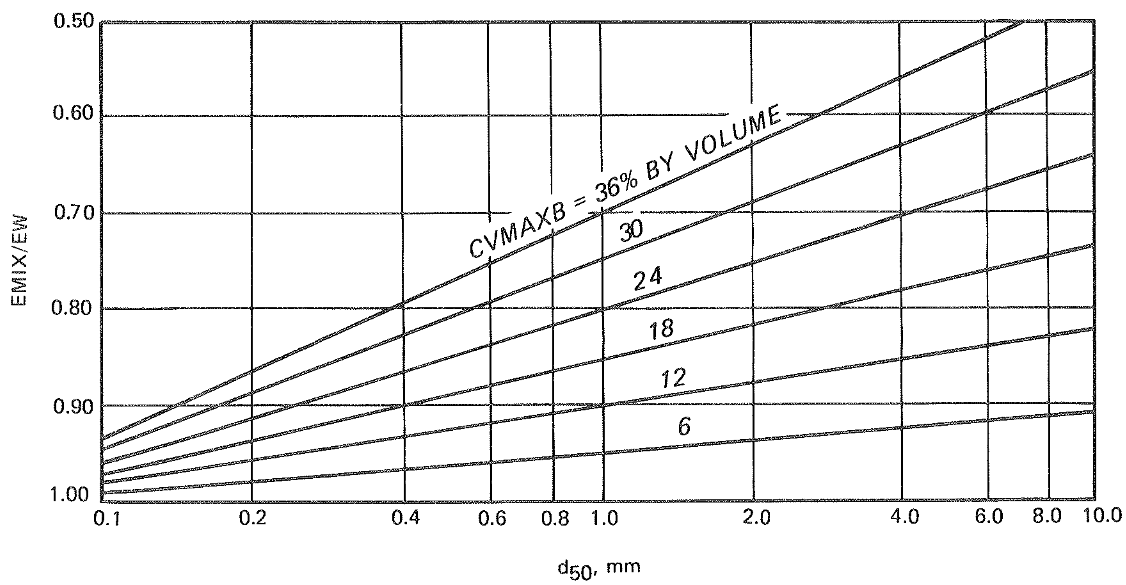

Figure 35. Ratio of efficiency mixture/efficiency water versus grain size (after Stepanoff 1969)

Select a pump by entering the manufacturer's pump performance curves for water with the required total dynamic head corrected for specific gravity and efficiency, TDHBME , and with the total flow required of the booster, QDISB .

The input shaft horsepower indicated at this point on the manufacturer's curves for water will represent the power required to pump water, BHPW . This must also be corrected to account for the heavier mixture being pumped. Assuming that the dredge pump will be operating near its point of peak efficiency, the required horsepower to pump the mixture, BHPM , is given by the relationship

$$BHPM = BHPW(SGDISB) \tag{54}$$

Graphical Design Procedure

85. This design procedure incorporates portions of the iteration design procedure. Such portions will not be repeated here but simply referred to. Therefore, familiarity with the iteration design procedure is a prerequisite to use of this procedure. Appendix C describes the derivation of the graphical design procedure and its relation to the iteration procedure.

Steps 1 through 3: Perform Steps 1 through 3 as described in the iteration design procedure.

Step 4: Objective - Generate a set of curves giving the required jet pump discharge head, HDIS , for a given value of jet pump suction flow, QSUC , and supply flow, QSUP .

Information required - (a) Minimum jet pump supply flow, $QSUP_{min}$ (see Step 2); (b) required jet pump suction flow, QSUC (see Step 3); (c) description of jet pump discharge pipeline, including length and adjustments for valves, fittings, and bends (several pipe sizes may be considered at this point); (d) hydraulic characteristics of jet pump discharge pipe; (e) settling velocity, W , of sediment d_{50} particle diameter (see Step 1); (f) specific gravity of solids, SGSOL ; (g) elevation of booster pump suction flange relative to water surface, ZBOO ; (h) estimated design pressure or vacuum at the booster pump suction flange, HSUCB (see discussion of this subject in Step 8 of the iteration design); (i) assumed maximum specific gravity of suction mixture, SGSUC , as discussed in Step 7 of the iteration design procedure.

Rationale - The required discharge head at the jet pump discharge is a function of the discharge flow rate, solids concentration, pipeline characteristics, and conditions at the booster pump suction. By assuming various values for some or all of these parameters, required discharge heads can be calculated for a variety of discharge conditions. Doing this in a systematic manner will result in a table of values that can be used to generate sets of required discharge head curves.

Method - If more than one diameter or material is being considered for the jet pump discharge pipeline, choose a size and type of pipe to begin calculations. Next, choose a starting value of jet pump supply flow, QSUP , equal to or somewhat less than $QSUP_{min}$. Also, choose a starting value of jet pump suction flow, QSUC , which is several hundred gallons per minute less than the QSUC value determined in Step 3.

Using these starting values, perform the following calculations:

$$QDIS = QSUP + QSUC \qquad (55)$$

$$VDIS = \frac{QDIS}{(448.831)(ADIS)} \qquad (28\ bis)$$

$$CVMAX = \left(\frac{QSUC}{QDIS} \; \frac{SGSUC - SGWAT}{SGSOL - SGWAT}\right) \qquad (56)$$

Apply these calculated values to the procedure outlined in Step 8 of the iteration design procedure for calculating the energy loss gradient in pipelines conveying sand/water slurries. The result will be a value of $(\Delta h/\Delta L)_m$. Continue with the remainder of Step 8 of the iteration design procedure to obtain a value of the required jet pump discharge head, HDIS . Record this value together with the corresponding values of QSUP and QSUC .

Keeping the same value of QSUP , increase the value of QSUC by an increment of 100 gpm or less and repeat the above procedure. Continue this process until QSUC is several hundred gallons per minute greater than the Step 3 value, recording each time the corresponding values of HDIS , QSUP , and QSUC .

Next, increase QSUP by an increment of 100 gpm or less and begin the calculation procedure again with the value of QSUC used initially. Execute this entire procedure several times until QSUP is several hundred gallons per minute greater than $QSUP_{min}$. The result will be a set of values of required HDIS corresponding to particular values of QSUC and QSUP for a certain size and type of discharge pipe. This information can be summarized in tabular form. Table 1 shows the hypothetical results of a set of such calculations for the same system layout but with two different sizes of discharge pipe.

The final task in this step is to use the tabulated information to generate a set of curves of QSUC versus HDIS . Figure 36 shows a set of such curves drawn using the information for the 6-in. discharge line in Table 1. Each curve represents a row from Table 1. To be compatible with the scale of the jet pump performance curves discussed in the next step, these curves should be drawn using axes with the following scales:

Horizontal (QSUC): 1 in. = 200 gpm

Vertical (HDIS): 1 in. = 20 feet of water

The curves should be drawn on or transferred to a transparent medium such as tracing paper so they can be used as overlays in the next step.

Table 1

Example: Required HDIS , ft

QSUP gpm	QSUC , gpm 300	400	500*	600	700	800
			6-in. Discharge Pipe			
500	35.6	39.3	43.1	47.1	51.4	62.1
600 ($QSUP_{min}$)	37.4	41.6	45.9	50.4	60.4	68.3
700	40.0	44.6	49.3	58.8	66.5	74.9
800	43.3	48.2	57.1	64.8	73.0	81.9
Heterogeneous flow ← | → Homogeneous flow						
			8-in. Discharge Pipe			
1100	29.0	31.0	33.0	34.9	36.8	38.7
1200 ($QSUP_{min}$)	29.9	32.0	34.0	36.0	38.0	40.1
1300	31.0	33.1	35.3	37.3	39.5	41.6
1400	32.3	34.5	36.6	38.8	41.0	43.3

* QSUC from Step 3.

Step 5: Objective - Correlate the curves of required HDIS from Step 4 with curves of actual jet pump performance.

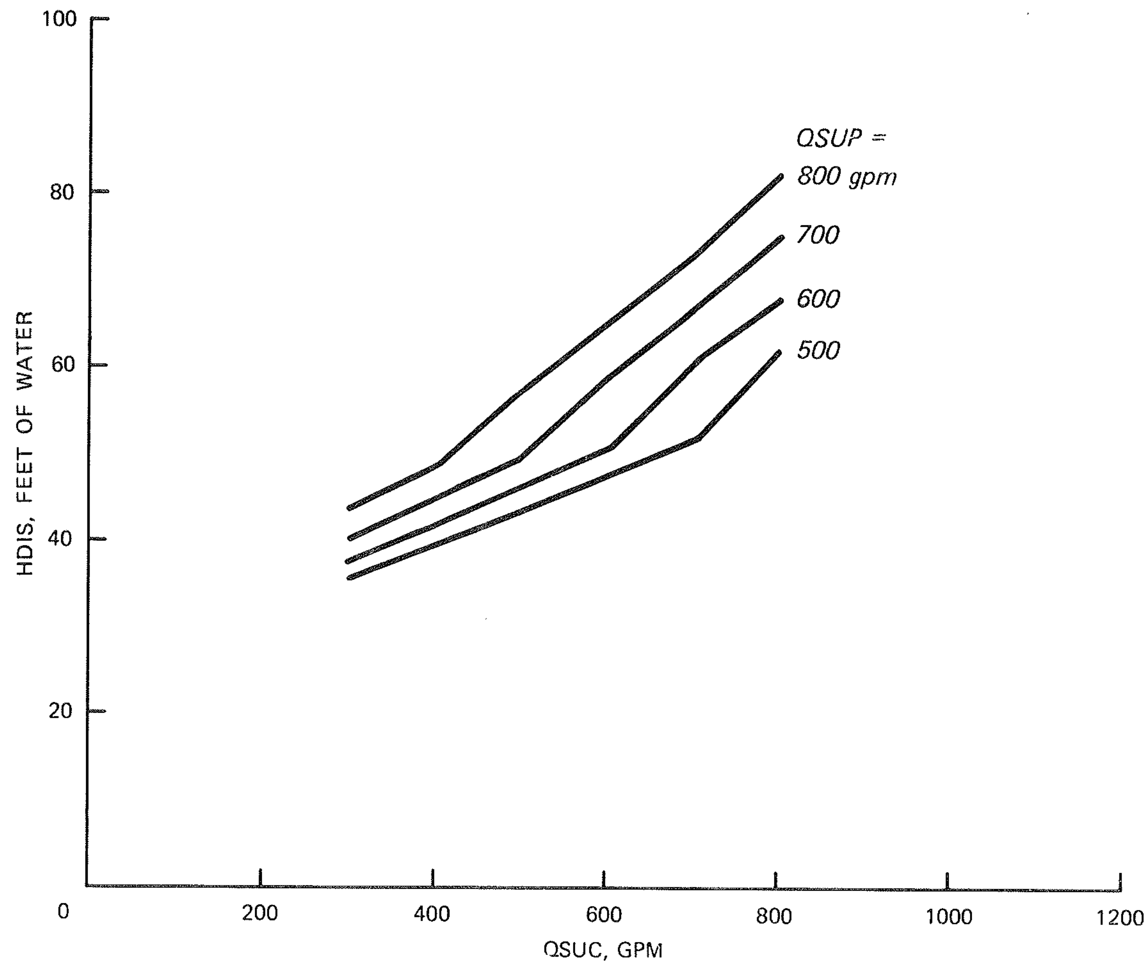

Figure 36. Example jet pump discharge pipeline curves for 6-in. pipe

<u>Information required</u> - Curves from Step 4 drawn on transparent paper.

<u>Rationale</u> - The information calculated in Step 4 is based solely on conditions in the jet pump discharge pipeline. The physics of operation of a jet pump are such that the values of three of its other operating parameters (HSUC , QDIS , and QSUC) also depend in part on conditions in the discharge pipeline. The purpose of this step, therefore, is to match what is required by the discharge piping system with what a particular jet pump is capable of producing under such conditions. The result of this step will be a number of possible operating points for the jet pump.

<u>Method</u> - Plates 3-14 give curves of actual jet pump performance pumping a sand/water slurry. Plates 3-8 apply to a 4 × 4 × 6 jet pump, while Plates 9-14 are for a 6 × 6 × 8 jet pump. Experience has shown that the characteristics of these two sizes of jet pumps will match the requirements of most sand bypassing situations.

Each plate shows curves of QSUC versus HDIS for particular values of QSUP . Each plate gives a complete set of curves for one nozzle size in a certain jet pump. THE CURVES SHOULD NOT BE EXTENDED UNDER ANY CIRCUMSTANCES. The limits as given are based on considerations which include cavitation, and operation outside of those limits entails the very real possibility of impaired system performance due to jet pump cavitation. Also, Plates 3-14 apply only when pumping a sand/water slurry. They are not valid for pumping water or materials other than sand. In addition to the value of QSUP for each curve, the recommended design value of HSUP is also given. Lines of equal efficiency values are also shown to aid the designer in selecting the most efficient jet pump configuration for a particular operating situation.

The technique in this step is simply to place the curves from Step 4 over Plates 3-14. The intersection of a curve from Step 4 that has a particular QSUP value with the curve on the plate that has the same QSUP value gives a potential operating point in terms of HDIS , HSUP , QSUC , and QSUP for the jet pump system. Figure 37 shows the example curves from Figure 36 superimposed on Plate 4. It is seen that two potential operating points have been identified by curve intersections. These operating points are as follows:

<u>a</u>. QSUP = 700 gpm
HSUP = 271 feet of water
QSUC = 350 gpm
HDIS = 42.5 feet of water
Efficiency = 14%

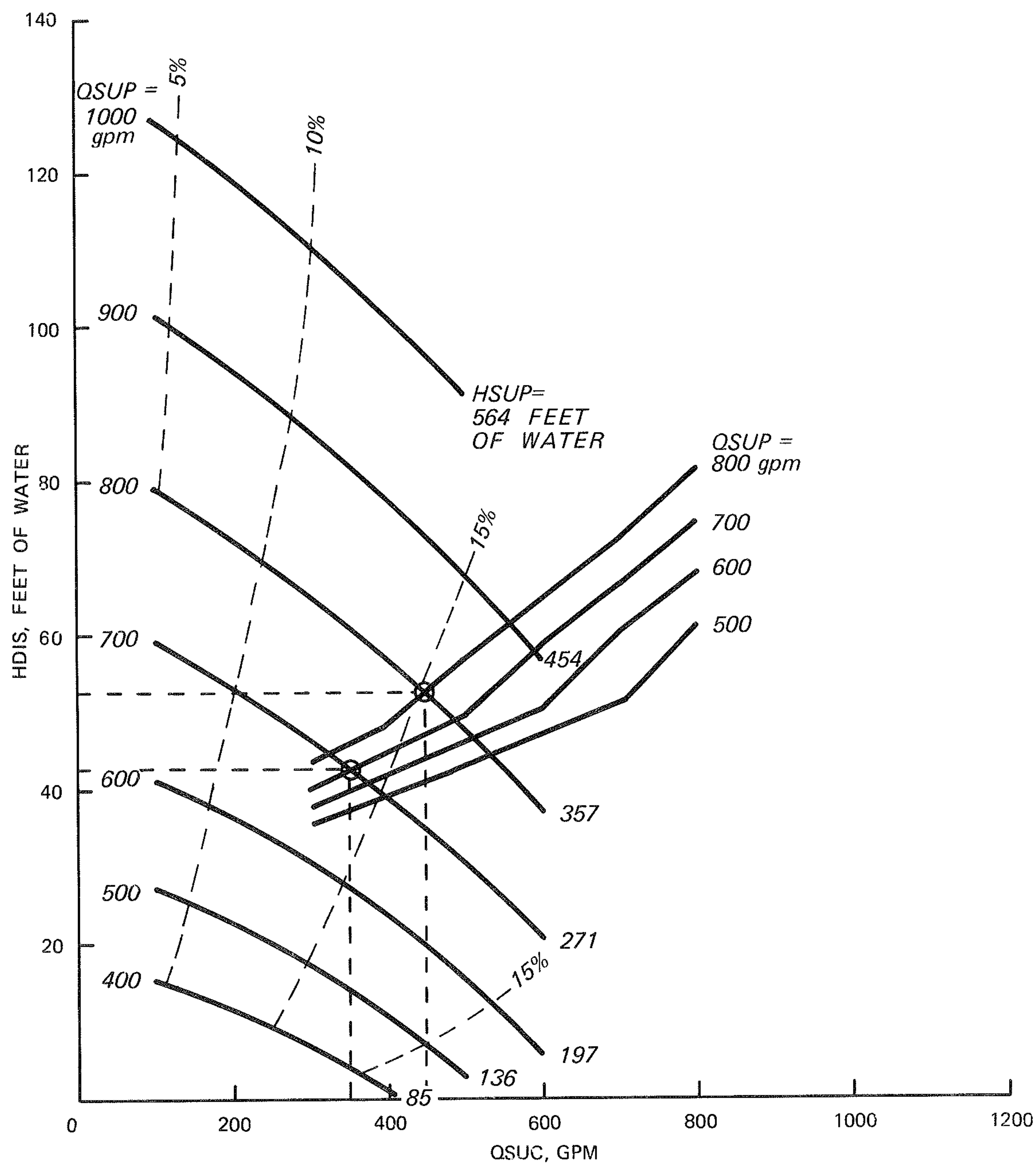

Figure 37. Example jet pump discharge pipeline curves for 6-in. pipe superimposed on Plate 4

b. QSUP = 800 gpm
HSUP = 357 feet of water
QSUC = 450 gpm
HDIS = 52.5 feet of water
Efficiency = 15%

Superimposing the Step 4 curves on other plates will generate more potential operating points. In some cases, few or none of the curves on a plate will coincide with the Step 4 curves. This situation indicates, of course, that such a nozzle size in that particular jet pump is unsuitable for the bypass system operating requirements. In other cases, the intersections may occur near the limits of the curves shown on a plate. While the operating points thus defined are valid, the designer should make sure that his predictions of system operating requirements are correct before using such operating points in designing an actual bypass system. Small variations from the predicted values may force the jet pump into cavitation.

The information for the operating points defined by this procedure can be summarized in tabular form. Table 2 shows the results of superimposing the example curves of Figure 36 on Plates 3-14.

Table 2

Example: Potential Operating Points, 6-in. Discharge Pipe

Jet Pump	Nozzle Size in.	QSUP gpm	HSUP ft	QSUC gpm	HDIS ft	Approximate Efficiency %
4 × 4 × 6	1.25	600	391	420	42.5	13
4 × 4 × 6	1.50	700	271	350	42.5	14
4 × 4 × 6	1.50	800	357	450	52.5	15
6 × 6 × 8			None			

The figures in Table 2 show that the example system design options with a 6-in. discharge pipe are limited in number and of poor efficiency. None of the possible operating points in Table 2 will give a QSUC value of 500 gpm, which as noted in Table 1, was the assumed design value from Step 3.

Figure 38 shows the curves of QSUC versus HDIS from Table 1 for an 8-in. discharge pipe for the same example system. Table 3 gives the results of superimposing these example curves on Plates 3-14.

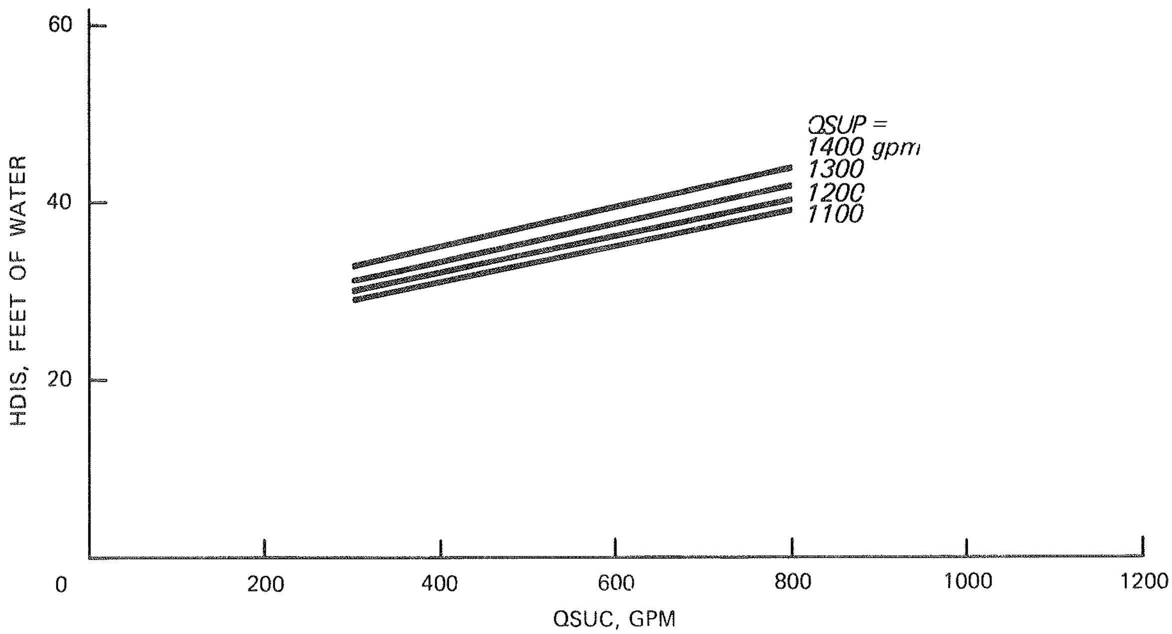

Figure 38. Example jet pump discharge pipeline curves for 8-in. pipe

Table 3

Example: Potential Operating Points, 8-in. Discharge Pipe

Jet Pump	Nozzle Size in.	QSUP gpm	HSUP ft	QSUC gpm	HDIS ft	Approximate Efficiency %
4 × 4 × 6	2.00	1100	180	580	34.5	25
4 × 4 × 6	2.25	1200	86	320	30.5	27
4 × 4 × 6	2.25	1300	101	375	33.0	24
4 × 4 × 6	2.25	1400	118	420	35.0	23
6 × 6 × 8	1.88	1200	299	660	37.0	12
6 × 6 × 8	1.88	1300	354	800	42.0	12
6 × 6 × 8	2.25	1400	207	500	36.5	12

Step 6: Objective - Select a jet pump design operating point.

Information required - (a) Potential operating points from Step 5, (b) assumed maximum specific gravity of suction mixture, SGSUCM , as discussed in Step 7 of the iteration design procedure.

Rationale - A number of potential jet pump operating points were determined in Step 5. This step requires a decision as to which of these points, if any, should be used in further calculations. If further calculations show the chosen point to be unfeasible, then another can be selected based on the Step 5 results.

Method - The first step is to look at the QSUC values of all the potential operating points. Those points with values of QSUC close to the value determined in Step 3 should be noted for further consideration.

The next step is to look at the remaining operating points to see whether some additional calculations with different values of QSUP would generate other operating points with QSUC values close to the Step 3 value. For instance, for the example being demonstrated, recall that QSUC from Step 3 was 500 gpm. Then, looking at Table 3, other possibilities for additional calculations are:

a. 4 × 4 × 6 jet pump -

   (1) 2.00-in. nozzle, QSUP = 1000 gpm.

   (2) 2.25-in. nozzle, QSUP = 1600 gpm.

b. 6 × 6 × 8 jet pump - Efficiencies shown are relatively low; additional calculations would probably not be worthwhile.

The designer should now return to Step 4 and perform the additional calculations indicated by this review. The resulting points together with the ones first noted constitute the group of potential design operating points.

If the potential design operating points identified in this step have uniformly low efficiencies, or if the points with acceptable efficiencies lie near the limits of the applicable jet pump performance curves, the designer may want to reconsider certain aspects of his system layout before proceeding any further. It is important at this point to carry on only with what appears to be a feasible design. Step 5, if performed properly, will show a range of possibilities for a particular design.

If the operating points within this range are inefficient or marginal, then the basic system layout may be at fault.

Once the design point has been selected, the design values of QSUP , HSUP , QSUC , and HDIS are automatically determined. Values of QDIS and CVMAX needed for subsequent calculations can be determined from Equations 55 and 56.

Remaining calculations - The remainder of the graphical design procedure can be accomplished by performing Steps 13 through 19 of the iteration design procedure. In effect, Steps 4, 5, and 6 of the graphical procedure have replaced Steps 5 through 12 of the iteration design procedure.

Additional Considerations for Multiple Jet or Booster Pumps

Multiple jet pumps

86. The steps in the two design procedures can be followed for systems in which more than one jet pump operates at a time, provided that certain hydraulic requirements are met. These requirements stem from two basic principles of pipeline flow at junctions:

a. At a pipe junction, the total head in all branches of the junction must be equal. Thus, from Figure 39,

$$H_1 + \frac{V_1^2}{2g} = H_2 + \frac{V_2^2}{2g} = H_3 + \frac{V_3^2}{2g} \quad (57)$$

b. Total flow away from the junction must equal total flow into the junction. Again, from Figure 39,

$$Q_1 + Q_2 = Q_3 \quad (58)$$

87. Figure 40 shows these two principles as applied to a system of two jet pumps without cutting assists operating simultaneously. The hydraulic requirements that would have to be met are as follows:

a. Supply pipeline

$$HSUP = HSUP1 = HSUPA$$

$$HSUP2 = HSUPA - HWSD_{1-2}$$

$$QSUP = QSUP1 + QSUP2$$

where

HSUP , HSUP1 , HSUPA , and HSUP2 are the total heads at the following respective locations:

93

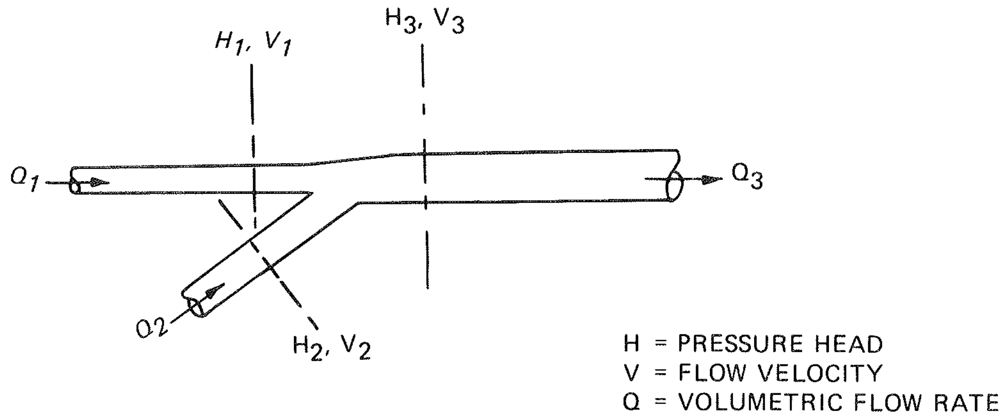

Figure 39. Junction of three pipes

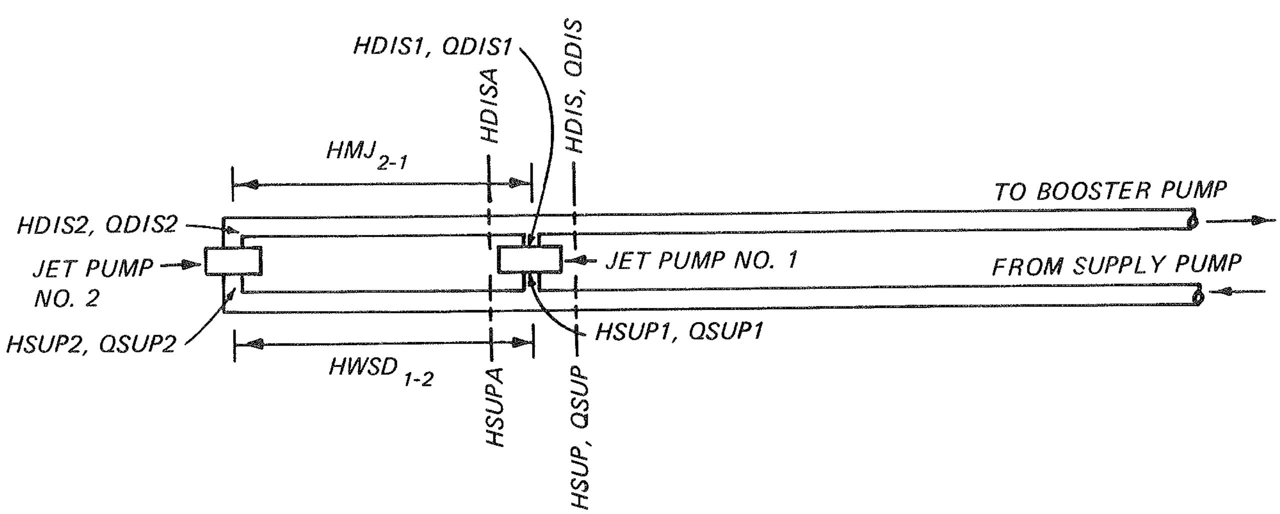

Figure 40. Dual jet pump system

94

(1) In the supply pipeline immediately before the junction of jet pump #1

(2) At jet pump #1 (on the supply side)

(3) In the supply pipeline immediately after the junction of jet pump #1

(4) At jet pump #2 (on the supply side).

$HWSD_{1-2}$ is the head loss in the supply pipeline between jet pumps #1 and #2

QSUP , QSUP1 , and QSUPA are the volumetric flow rates at the same locations as the respective total heads.

b. Discharge pipeline

$$HDIS = HDIS1 = HDISA$$

$$HDIS2 = HDISA + HMJ_{2-1}$$

$$QDIS = QDIS1 + QDIS2$$

where

HDIS , HDIS1 , HDISA , and HDIS2 are the total heads at the following respective locations:

(1) In the discharge pipeline immediately after the junction of jet pump #1.

(2) At jet pump #1 (on the discharge side).

(3) In the discharge pipeline immediately before the junction of jet pump #1.

(4) At jet pump #2 (on the discharge side).

HMJ_{2-1} is the head loss in the discharge pipeline between the junctions of jet pumps #2 and #1.

QDIS , QDIS1 , and QDIS2 are the volumetric flow rates at the same locations as the respective total heads.

88. The end result of these requirements is to increase the complexity of iterative calculations to determine values of the jet pump operating parameters. The number of iterations can be reduced if as

many system variables as possible are made into constants. Some ways of doing this are:

- a. Use the same size jet pump for all jet pumps.
- b. Use the same nozzle size in all jet pumps.
- c. Space jet pumps equal distances apart where possible.
- d. Choose pipe sizes to give similar flow velocities in all branches of a junction.

89. The number of iterations can be reduced further by choosing values of EXC1 for each jet pump that decrease with increasing distance to the booster pump. For instance, in Figure 40, jet pump #2 should have a smaller value of EXC1 than jet pump #1.

Multiple booster pumps

90. Multiple booster pumps located at intervals along the discharge pipeline require additional considerations not needed for a single-booster system. Monitoring, control, and sequencing of booster operation become very important to prevent cavitation or water hammer, either of which can ruin an expensive pump. Designing for a positive pressure head, PHSUCB , at each booster suction will aid in preventing such problems as well as helping keep air out of the discharge system. Booster pumps should be spaced at intervals along the discharge pipeline such that their operating points are roughly the same. Each pump must be provided clear flushing water at a pressure greater than the booster discharge pressure. After the initial hydraulic design, the entire bypass system should be analyzed for the effects of different possibilities, such as plugging a jet pump suction or stopping a booster pump unexpectedly.

91. Two adjustments must be made to the hydraulic design procedure to allow for multiple booster pumps. First, total flow in the discharge pipeline will increase at each booster due to the addition of flushing water. This must be accounted for in determining flow rates, velocities, and solids concentrations at each booster. Second, the total dynamic head required of a booster pump will depend on conditions somewhat different from those considered in Equation 41 of the iteration design procedure. For example, if the first and second boosters in a pipeline are

numbered 1 and 2, Equation 41 becomes:

$$TDHB_1 = HMB_{1-2} - PHSUCB_1 + (ZBOO_2 - ZBOO_1) + PHSUCB_2 \quad (59)$$

where HMB_{1-2} is the total frictional head loss in the discharge pipeline between boosters 1 and 2. Equation 59 should be used for all boosters in a multiple booster system except the final one, where Equation 41 applies.

PART IV: SUMMARY

92. This report is designed to be used by an engineer or group of engineers with a basic knowledge of coastal processes and a rudimentary knowledge of centrifugal pumping systems. Using this report, such a person or group will be able to perform the following tasks:

- a. Determine the general feasibility of a jet pump remedial sand bypassing system for a specific problem.
- b. Develop the initial layout(s) for a jet pump sand bypassing system.
- c. Perform the basic hydraulic design for such a system.

93. A set of factors relative to tasks a and b are discussed. Detailed instructions are given for aspects of the initial layout such as determining the system effective operating time and capacity. A format consisting of schematic drawings is suggested for presenting the results of the initial layout. Examples of such drawings are shown.

94. The designer is provided with two step-by-step procedures for performing task c. The first procedure, called the iteration procedure, consists of calculating in an iterative manner an operating point for the jet pump portion of the system. Then, the hydraulic characteristics of the total system are calculated linearly. The jet pump operating point reflects the results of the initial layout and is based upon the premises of having a minimum total flow in the system and of achieving a high jet pump efficiency. The second procedure utilizes a set of graphs to replace the iterative part of the first procedure. Use of these graphs gives a number of potential operating points for the jet pump portion of the system. One or more of these operating points is chosen by the designer to use in further calculations.

95. The specialized information resulting from use of this report, together with more routine design data, should allow the designer to estimate the approximate cost of a jet pump sand bypassing system relative to other solutions for a given problem. A subsequent report will provide information about the detailed design, construction, operation, and monitoring of a jet pump system, as well as design examples.

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APPENDIX A: NOTATION

The following notation is used in a general sense in this report:

C_v	Volumetric concentration of solids in a slurry
D	Inside diameter of a pipe
H	Pressure head
h_f	Energy loss due to a pipe fitting
L	Length of straight pipe
Q	Volumetric flow rate
V	Flow velocity

The following notation identifies specific quantities or parameters:

ADIS	Discharge pipe maximum inside area, ft^2
ADISB	Inside area of booster pump discharge pipe, ft^2
ALM	Correction factor for absence of littoral materials
AMIX	Inside area of mixing chamber of jet pump, ft^2
ANOZ	Area of opening at tip of jet pump nozzle, ft^2
ASUC	Inside area of jet pump suction, ft^2
ASUPD	Inside area of jet pump supply pipe, ft^2
ASUPS	Inside area of supply pump suction pipe, ft^2
ATMOS	Atmospheric pressure, feet of water
B	Coefficient in nozzle equation for QSUP , $gpm/ft^{5/2}$
BHPM	Horsepower required by dredge pump pumping slurry
BHPW	Horsepower required by dredge pump pumping water
CVMAX	Expected maximum volumetric concentration of solids in jet pump discharge pipeline
CVMAXB	Expected maximum volumetric concentration of solids in booster pump discharge pipeline
d_{50}	Median diameter of sediment to be bypassed, mm
DEPMAX	Maximum anticipated water depth over jet pump while operating, ft
DEPMIN	Minimum anticipated water depth over jet pump while operating, ft
E	Operating efficiency of jet pump
EMIX	Efficiency of dredge pump pumping slurry

EOT	Effective operating time of jet pump bypassing system over one-year operating period
$EOT_{\Delta t}$	Effective operating time of jet pump bypassing system over interval Δt operating period
EW	Efficiency of dredge pump pumping water
EXC	Required capacity of jet pump bypassing system, cu yd/hr
EXC1	Required capacity of one jet pump in bypassing system, cu yd/hr
EXCMAX	Maximum excavation rate of in situ material by one jet pump, cu yd/hr
f	Dimensionless friction factor used in Darcy-Weisbach formula
F_L	Dimensionless parameter in Durand relationship for critical velocity
g	Acceleration due to gravity, ft/sec^2
HD	Number of working hours in an operating day
HDIS	Total energy head in the discharge pipeline at the jet pump, feet of water
HMB	Total head loss in booster pump discharge pipeline, feet of water
HMJ	Total head loss in jet pump discharge pipeline, feet of water
HSUC	Total energy head in the jet pump suction tube at the jet pump, feet of water
HSUP	Total energy head in the supply pipeline at the jet pump, feet of water
HWSD	Total head loss in jet pump supply pipeline, feet of water
HWSS	Total head loss in supply pump suction pipeline, feet of water
K	Pipe fitting resistance coefficient
LDISB	Total equivalent length of booster discharge pipeline, ft
LDISJ	Total equivalent length of jet pump discharge pipeline, ft
LSUC	Length of jet pump suction tube, ft
LSUPD	Total equivalent length of jet pump supply pipeline, ft
LSUPS	Total equivalent length of supply pump suction pipeline, ft
LSUPSA	Approximate length of supply pump suction pipeline, ft
M	Jet pump flow ratio

M_{op}	Optimum jet pump flow ratio
n	In situ porosity of sediment to be bypassed
N	Jet pump head ratio
N_{max}	Largest possible value of N corresponding to a particular value of M
NOD	Number of operating days per year for jet pump bypassing system
NPSHA	Available net positive suction head at supply pump suction flange, feet of water
NPSHR	Required net positive suction head at supply pump suction flange, feet of water
NUM	The number of jet pumps operated simultaneously in a jet pump bypassing system
PB	Correction factor for pump blockages
PHSUCB	Design pressure or vacuum head at booster pump suction flange, feet of water
Q_L	Average rate of net littoral influx to storage area(s) during interval Δt operating period, cu yd/hr
QDIS	Jet pump volumetric discharge flow rate, gpm
QDISB	Booster pump volumetric discharge flow rate, gpm
QFL	Flushing water volumetric flow rate into booster pump, gpm
QJET	Volumetric flow rate through jet pump cutting jets, gpm
QSUC	Volumetric flow rate into jet pump suction, gpm
QSUP	Volumetric flow rate through jet pump nozzle, gpm
$QSUP_{min}$	Minimum required volumetric flow rate in discharge pipelines of jet pump bypassing system, gpm
QSUPT	Supply pump required total volumetric flow rate, gpm
R	Jet pump area ratio
Re	Reynolds number
RMP	Correction factor for relocation of mobile jet pumps
RR	Correction factor for system repair and replacement
SGDISB	Maximum specific gravity of booster pump discharge slurry
SGDISJ	Maximum specific gravity of jet pump discharge slurry
SGIN	In situ specific gravity of sediment to be bypassed
SGSOL	Specific gravity of sediment solids
SGSUC	Assumed average specific gravity of slurry entering jet pump suction

SGSUCM	Assumed maximum sustained specific gravity of slurry entering jet pump suction
SGWAT	Specific gravity of ambient water
STCAP	Storage capacity of storage area, cu yd
STIN	Initial condition of storage area, cu yd
$STORE_{\Delta t}$	Storage volume available during interval Δt , cu yd
TDHB	Total dynamic head required of booster pump, feet of water
TDHBM	Total dynamic head required of booster pump in terms of mixture being pumped, feet of mixture
TDHBME	Total dynamic head of booster pump corrected for decrease in pump efficiency due to presence of solids, feet of mixture
TDHS	Total dynamic head required of supply pump, feet of water
VAP	Vapor pressure of water, feet of water
VCRIT	Minimum velocity necessary to maintain solids in suspension in discharge pipelines, fps
VDIS	Velocity in jet pump discharge pipeline, fps
VDISB	Velocity in booster pump discharge pipeline, fps
VHOM	Velocity of transition between heterogeneous and homogeneous flow regimes, fps
VNOZ	Velocity of water jet at tip of jet pump nozzle, fps
VSUC	Velocity of mixture in jet pump suction tube, fps
VSUPD	Velocity in jet pump supply pipeline, fps
VSUPS	Velocity in supply pump suction pipeline, fps
W	Settling velocity of the d_{50} particle of sediment to be bypassed, fps
X	Ratio of VSUC to VNOZ as used in cavitation calculations
ZBOO	Elevation of booster pump center line relative to water surface datum, ft
ZDIS	Elevation of end of booster discharge pipeline relative to water surface datum, ft
ZSUP	Elevation of supply pump center line relative to water surface datum, ft
ZSUPM	Maximum expected elevation of supply pump suction center line above free water surface, ft
$(\Delta h/\Delta L)_m$	Head loss per unit length of pipeline for slurry flow
$(\Delta h/\Delta L)_w$	Head loss per unit length of pipeline for fluid flow

A4

$(\Delta h/\Delta L)_{WSD}$ Head loss per unit length of pipeline for jet pump supply pipeline

$(\Delta h/\Delta L)_{WSS}$ Head loss per unit length of pipeline for supply pump suction pipeline

Δt Time interval of the bypassing season, hr

ε Equivalent roughness of pipe wall (Nikuradse roughness), ft

ν Kinematic viscosity of fluid, ft^2/sec

A5

APPENDIX B: ADDITIONAL EQUATIONS

Colebrook-White Equation

1. For the range of conditions covered by this report, the friction factor, f , used in the Darcy-Weisbach formula for energy loss in pipe flow can be found via an iterative solution to the Colebrook-White equation:

$$\frac{1}{\sqrt{f}} = -2 \log\left(\frac{2.51}{Re\sqrt{f}} + \frac{\varepsilon}{3.71D}\right) \qquad (B1)$$

where

Re = VD/ν , the Reynolds number

V = average flow velocity in pipe, fps

D = inside diameter of pipe, ft

ν = kinematic viscosity of fluid, ft^2/sec

ε = equivalent roughness of pipe wall, sometimes called "Nikuradse roughness," ft

2. Values of ε for different pipe materials can be found in standard references on fluid flow in pipes. Values of ν can be found in hydraulic handbooks or similar references.

3. To solve Equation B1 by iteration, begin with an assumed value for f , say 0.007. Using this assumed value, solve the right-hand side of the equation. The result will be of the form:

$$\frac{1}{\sqrt{f}} = C \qquad (B2)$$

where C is the value of the right-hand side of Equation B1 using the assumed value of f and the appropriate pipe characteristics and flow conditions.

4. Next, solve Equation B2 for f . If this value of f and the assumed one are reasonably similar (say within 0.0005 of each other), f has been determined. If not, repeat the iteration process using f from Equation B2 as the assumed value. The iteration should converge

to an acceptable degree of accuracy in a few such calculations.

5. Obviously, this process is easily adapted to a computer, eliminating the need for a Moody diagram.

Equivalent Lengths

6. For pipe fittings not included in tables of equivalent lengths, the energy loss h_f is often given by an equation of the form:

$$h_f = K \frac{V^2}{2g} \qquad \text{(B3)}$$

where K is a resistance coefficient for a particular fitting type and size.

7. For straight pipe, an equation in common use for calculating energy losses is the Darcy-Weisbach equation in the form:

$$h_L = f \frac{L}{D} \frac{V^2}{2g} \qquad \text{(B4)}$$

where

h_L = energy loss in straight pipe
f = friction factor
L = length of straight pipe
D = pipe inside diameter

8. Equating Equations B3 and B4, an expression for an equivalent length can be obtained:

$$L = \frac{K \cdot D}{F} \qquad \text{(B5)}$$

where L now represents the length of straight pipe of inside diameter D equivalent to a fitting with resistance coefficient K and the same nominal pipe size.

9. For steel pipe and the range of flow velocities and pipe sizes commonly encountered in jet pump bypassing systems, Equation B5 can be approximated by:

$$L = \frac{K \cdot D}{0.014} \tag{B6}$$

10. Equation B6 will give an estimated equivalent length of steel pipe of inside diameter D for a fitting of the same nominal pipe size with resistance coefficient K.

11. For pipelines made of material other than steel, the method of equivalent lengths is more difficult to apply. In such cases, it is suggested that losses for fittings be calculated by Equation B3. These losses can then be added to the losses for straight pipe calculated by the Darcy-Weisbach equation to obtain the total pipeline energy loss.

APPENDIX C: DERIVATION OF GRAPHICAL DESIGN CURVES

1. The purpose of this appendix is to show, briefly, how the design curves of Plates 3 through 14 (main text) were derived from equations outlined in the iteration design procedure.

2. The dimensionless parameters describing the performance of a jet pump with a certain area ratio R are given by Equations 1 and 2:

$$N = \frac{HDIS - HSUC}{HSUP - HDIS} \qquad \text{(1 bis)}$$

$$M = \frac{QSUC}{QSUP} \qquad \text{(2 bis)}$$

3. The relationship between these parameters is assumed to be of the form:

$$N = a \cdot M + b \qquad \text{(C1)}$$

which is the equation for a straight line where a and b are constants. The values of these constants can be obtained from the M versus N relations shown in Plate 2.

4. Substituting Equations 1 and 2 into Equation C1,

$$\frac{HDIS - HSUC}{HSUP - HDIS} = a\left(\frac{QSUC}{QSUP}\right) + b \qquad \text{(C2)}$$

5. The relationship between QSUP , HSUP , and HSUC must satisfy Equation 33:

$$QSUP = B(ANOZ)\ \sqrt{(HSUP - HSUC)} \qquad \text{(33 bis)}$$

From Equation 34,

$$ANOZ = f(R\ ,\ AMIX) \qquad \text{(C3)}$$

For a given jet pump size, AMIX is constant. For a given R value, B is constant (see tabulation on p 73 of main text). Therefore, for a given jet pump size and R value,

$$QSUP = f(HSUP\ ,\ HSUC) \qquad (C4)$$

6. Equation 31 gives a suggested expression for calculating HSUC :

$$HSUC = \frac{VSUC^2}{2g} - \left[2LSUC + 4\left(\frac{VSUC^2}{2g}\right)\right] \qquad (31\ bis)$$

For a given jet pump size,

$$VSUC = f(QSUC) \qquad (C5)$$

7. Therefore, for a given jet pump size and assumed LSUC value,

$$HSUC = f(QSUC) \qquad (C6)$$

Equation C4 can now be rewritten:

$$QSUP = f(HSUP\ ,\ QSUC) \qquad (C7)$$

or

$$HSUP = f(QSUP\ ,\ QSUC) \qquad (C8)$$

8. Substituting Equations C6 and C8 into Equation C2,

$$\frac{HDIS - f(QSUC)}{f(QSUP\ ,\ QSUC) - HDIS} = a\left(\frac{QSUC}{QSUP}\right) + b \qquad (C9)$$

9. Equation C9 is the basis for the design curves of Plates 3-14. It contains three variables (HDIS , QSUC , and QSUP) and two known constants (a and b). By holding one variable constant, a unique relationship is defined between the other two variables. The

graphical design curves were generated by holding QSUP constant and solving for HDIS for different values of QSUC . Then, the value of QSUP was changed, and the process repeated. The only departure from the iteration design procedure was in assuming a certain LSUC value. However, this assumption can be shown to have a minimal effect on the calculated value of HDIS .

10. The values of HSUP shown on the graphical design curves were calculated using a rearranged form of Equation 33:

$$HSUP = \left[\frac{QSUP}{B(ANOZ)}\right]^2 + HSUC \qquad (C10)$$

11. For a given jet pump size and R value, if HSUC is held constant in Equation C10, then HSUP depends only on the value of QSUP . For each design curve shown in Plates 3-14, the value of HSUC at the curve midpoint was used as a constant in calculating HSUP from Equation C10. Therefore, the value of HSUP given for each curve is exactly correct at the curve midpoint, with an increasing error toward either end of the curve. The magnitude of this error at the curve ends, in most cases, is in the range of 5 percent. For any design operating point, therefore, the error involved in using an HSUP value from the graphical design curves instead of one rigorously calculated from the iteration procedure will be much less than the error envelope of the entire design process.

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Richardson, Thomas W.
A guide to the planning and hydraulic design of jet pump remedial sand bypassing systems : final report / by Thomas W. Richardson, Ernest C. McNair, Jr. (Hydraulics Laboratory, U.S. Army Engineer Waterways Experiment Station). -- Vicksburg, Miss. : The Station ; Springfield, Va. : available from NTIS, [1981].
111 p. in various pagings : ill. ; 27 cm. -- (Instruction report / U.S. Army Engineer Waterways Experiment Station ; HL-81-1)
Cover title.
"September 1981."
"Prepared for Office, Chief of Engineers, U.S. Army."
Bibliography: p. 99-100.

1. Hydraulic engineering. 2. Hydraulic machinery. 3. Jet pumps. 4. Sand. I. McNair, Ernest C., Jr. II. United States. Army. Corps of Engineers. Office of the Chief of Engineers. III. U.S. Army Engineer Waterways Experiment Station. Hydraulics Laboratory.

Richardson, Thomas W.
A guide to the planning and hydraulic design : ... 1981.
(Card 2)

IV. Title V. Series: Instruction report (U.S. Army Engineer Waterways Experiment Station) ; HL-81-1.
TA7.W34i no.HL-81-1

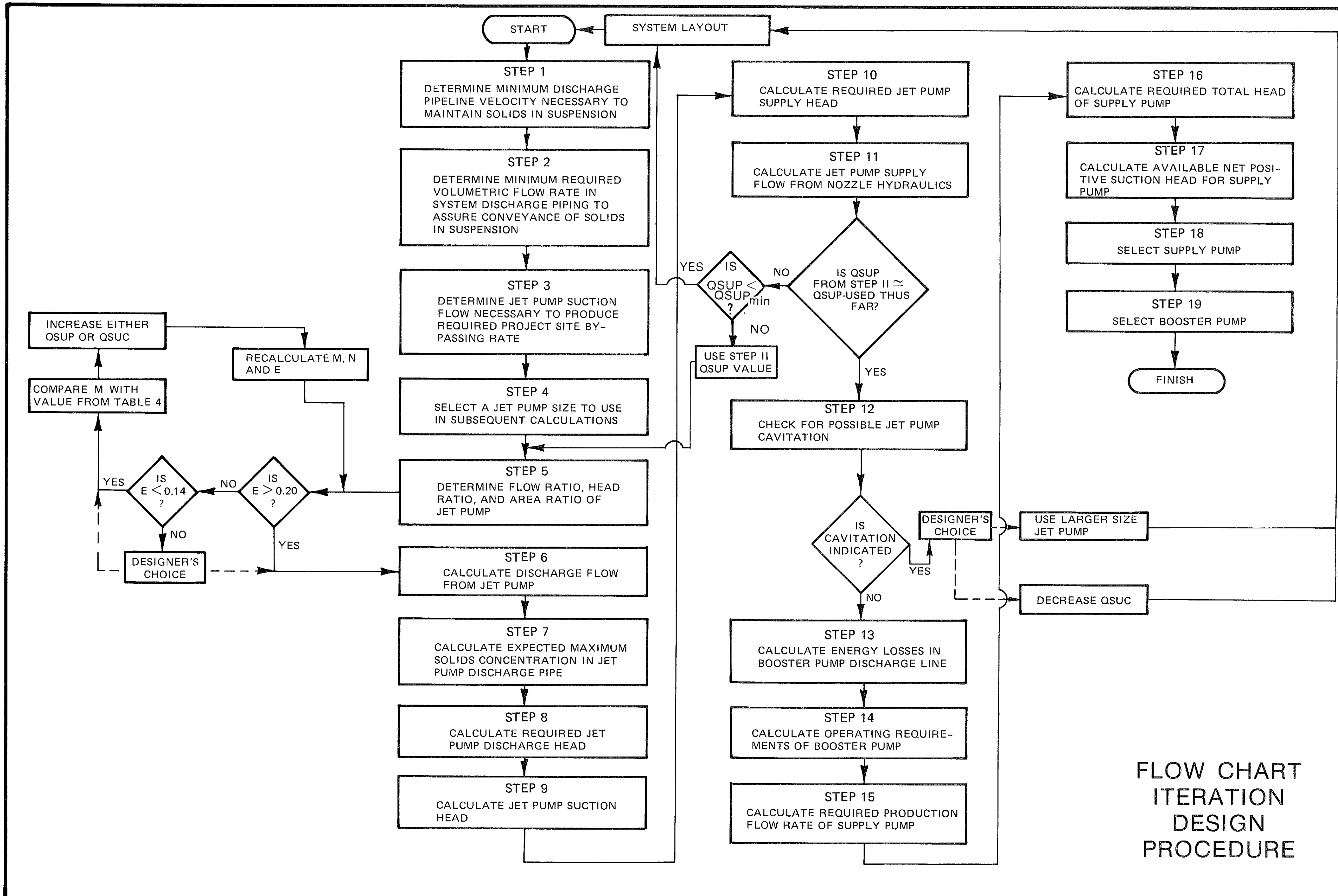
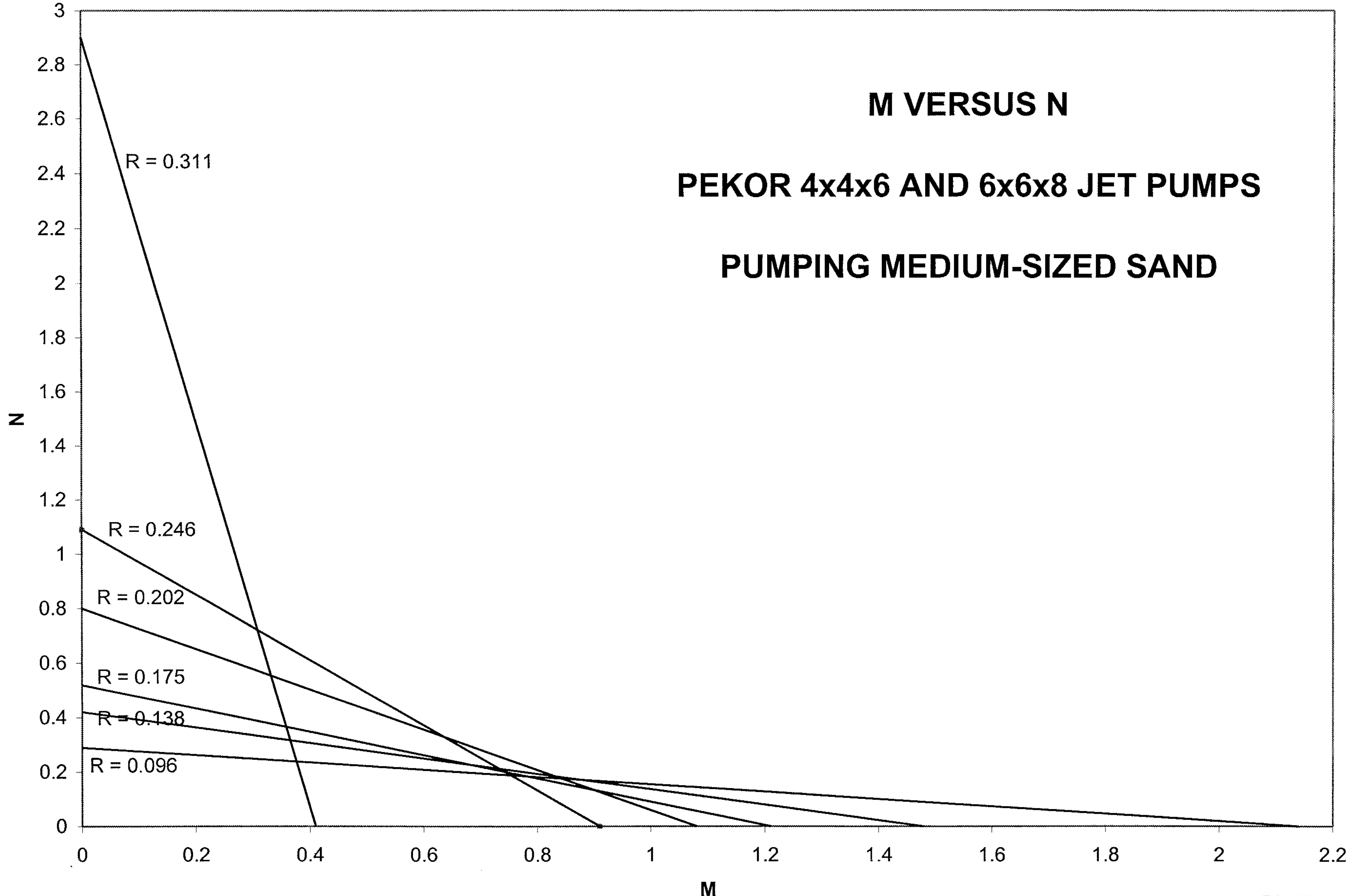

START
SYSTEM LAYOUT
STEP 1
DETERMINE MINIMUM DISCHARGE PIPELINE VELOCITY NECESSARY TO MAINTAIN SOLIDS IN SUSPENSION
STEP 2
DETERMINE MINIMUM REQUIRED VOLUMETRIC FLOW RATE IN SYSTEM DISCHARGE PIPING TO ASSURE CONVEYANCE OF SOLIDS IN SUSPENSION
STEP 3
DETERMINE JET PUMP SUCTION FLOW NECESSARY TO PRODUCE REQUIRED PROJECT SITE BY-PASSING RATE
STEP 4
SELECT A JET PUMP SIZE TO USE IN SUBSEQUENT CALCULATIONS
STEP 5
DETERMINE FLOW RATIO, HEAD RATIO, AND AREA RATIO OF JET PUMP
STEP 6
CALCULATE DISCHARGE FLOW FROM JET PUMP
STEP 7
CALCULATE EXPECTED MAXIMUM SOLIDS CONCENTRATION IN JET PUMP DISCHARGE PIPE
STEP 8
CALCULATE REQUIRED JET PUMP DISCHARGE HEAD
STEP 9
CALCULATE JET PUMP SUCTION HEAD
INCREASE EITHER QSUP OR QSUC
RECALCULATE M, N AND E
COMPARE M WITH VALUE FROM TABLE 4
IS E < 0.14 ?
IS E > 0.20 ?
YES
NO
NO
YES
DESIGNER'S CHOICE
STEP 10
CALCULATE REQUIRED JET PUMP SUPPLY HEAD
STEP 11
CALCULATE JET PUMP SUPPLY FLOW FROM NOZZLE HYDRAULICS
IS QSUP FROM STEP II ≃ QSUP-USED THUS FAR?
NO
YES
IS QSUP < QSUP min ?
YES
NO
USE STEP II QSUP VALUE
STEP 12
CHECK FOR POSSIBLE JET PUMP CAVITATION
IS CAVITATION INDICATED ?
YES
NO
DESIGNER'S CHOICE
USE LARGER SIZE JET PUMP
DECREASE QSUC
STEP 13
CALCULATE ENERGY LOSSES IN BOOSTER PUMP DISCHARGE LINE
STEP 14
CALCULATE OPERATING REQUIREMENTS OF BOOSTER PUMP
STEP 15
CALCULATE REQUIRED PRODUCTION FLOW RATE OF SUPPLY PUMP
STEP 16
CALCULATE REQUIRED TOTAL HEAD OF SUPPLY PUMP
STEP 17
CALCULATE AVAILABLE NET POSITIVE SUCTION HEAD FOR SUPPLY PUMP
STEP 18
SELECT SUPPLY PUMP
STEP 19
SELECT BOOSTER PUMP
FINISH
FLOW CHART ITERATION DESIGN PROCEDURE

PLATE 1


M VERSUS N
PEKOR 4x4x6 AND 6x6x8 JET PUMPS
PUMPING MEDIUM-SIZED SAND
R = 0.311
R = 0.246
R = 0.202
R = 0.175
R = 0.138
R = 0.096
N
M
3
2.8
2.6
2.4
2.2
2
1.8
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
0
0.2
0.4
0.6
0.8
1
1.2
1.4
1.6
1.8
2
2.2

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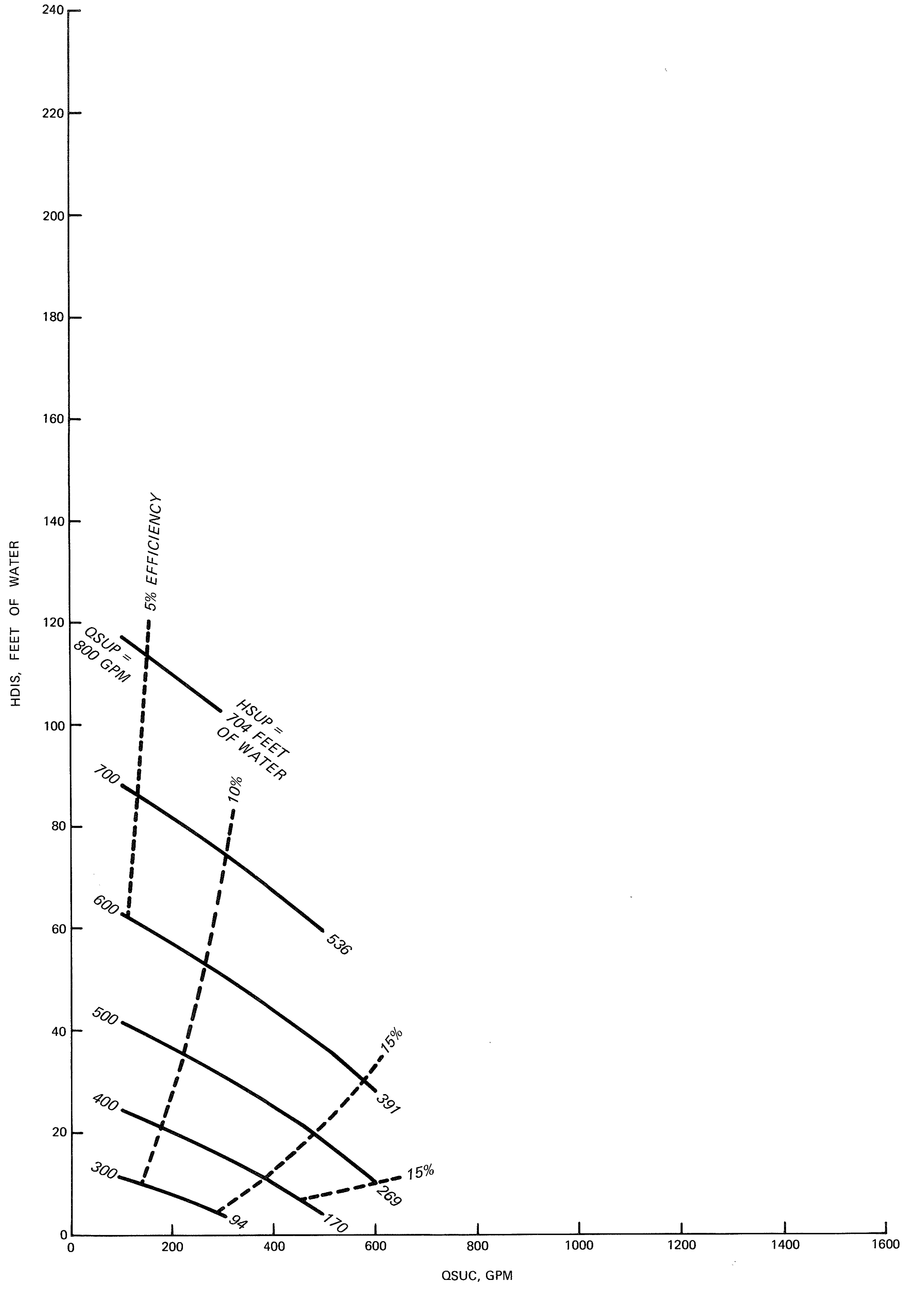
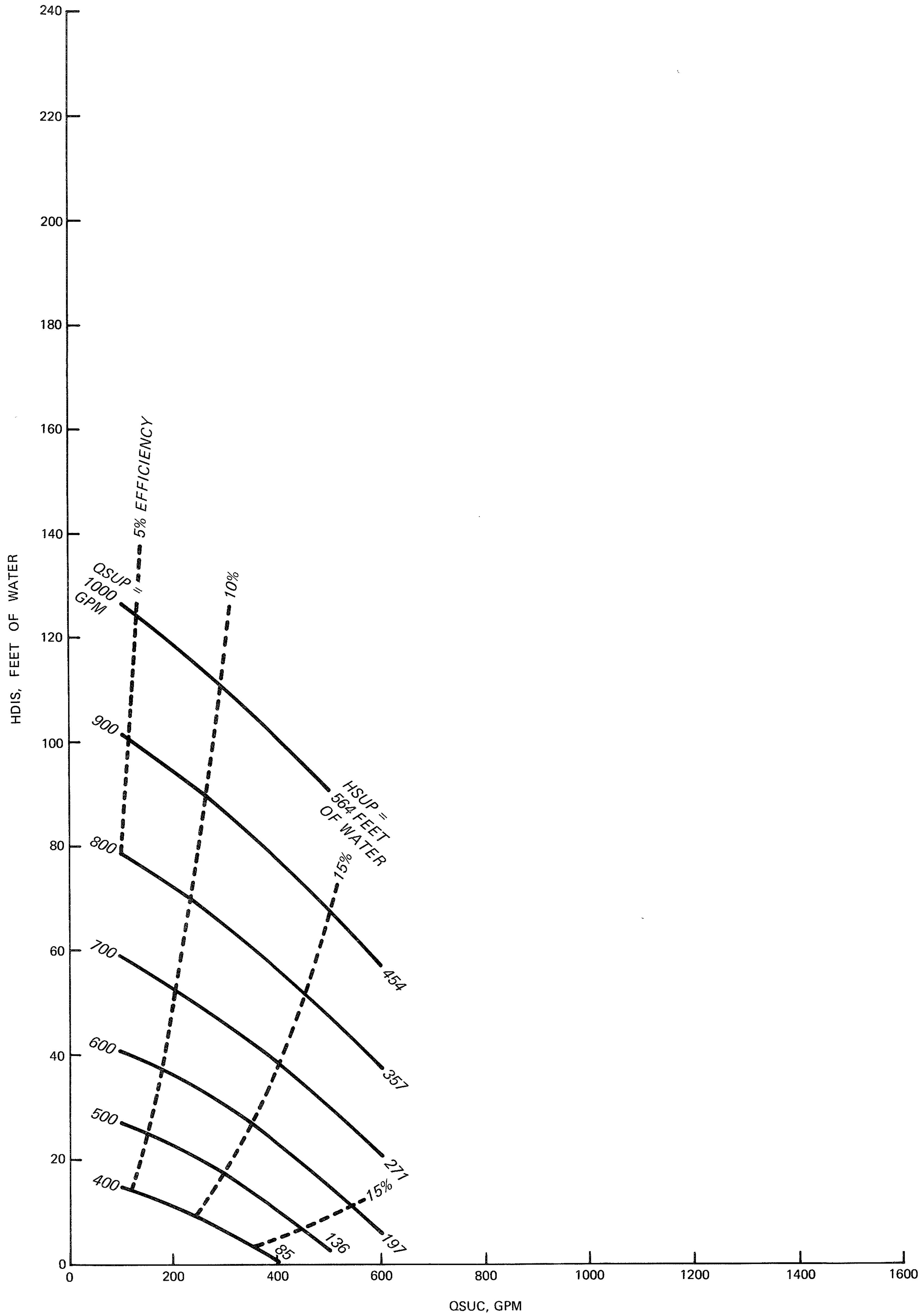


HDIS VERSUS QSUC

4×4×6 PEKOR JET PUMP PUMPING SAND

R = 0.096

PLATE 3



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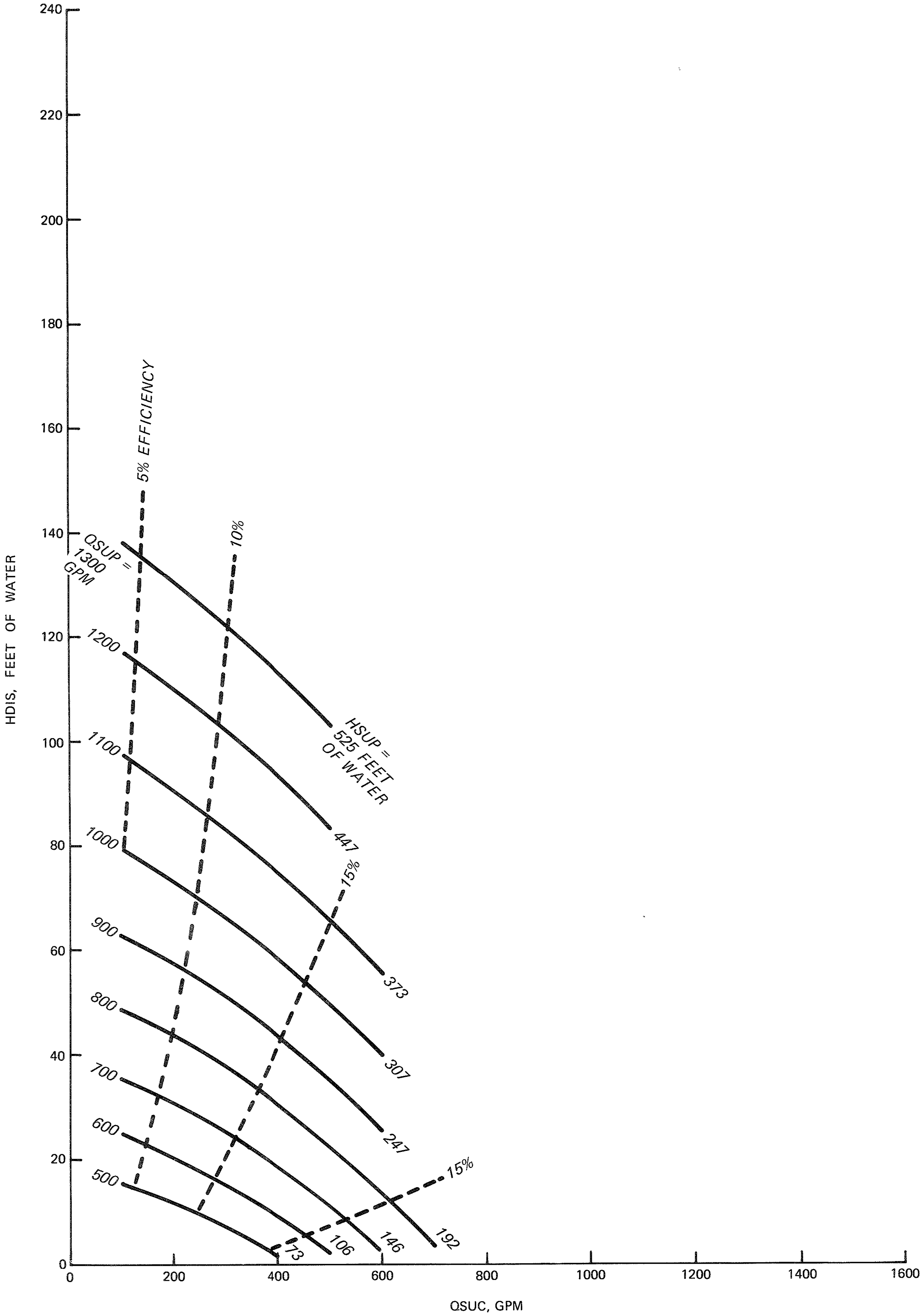
R = 0.138

PLATE 4



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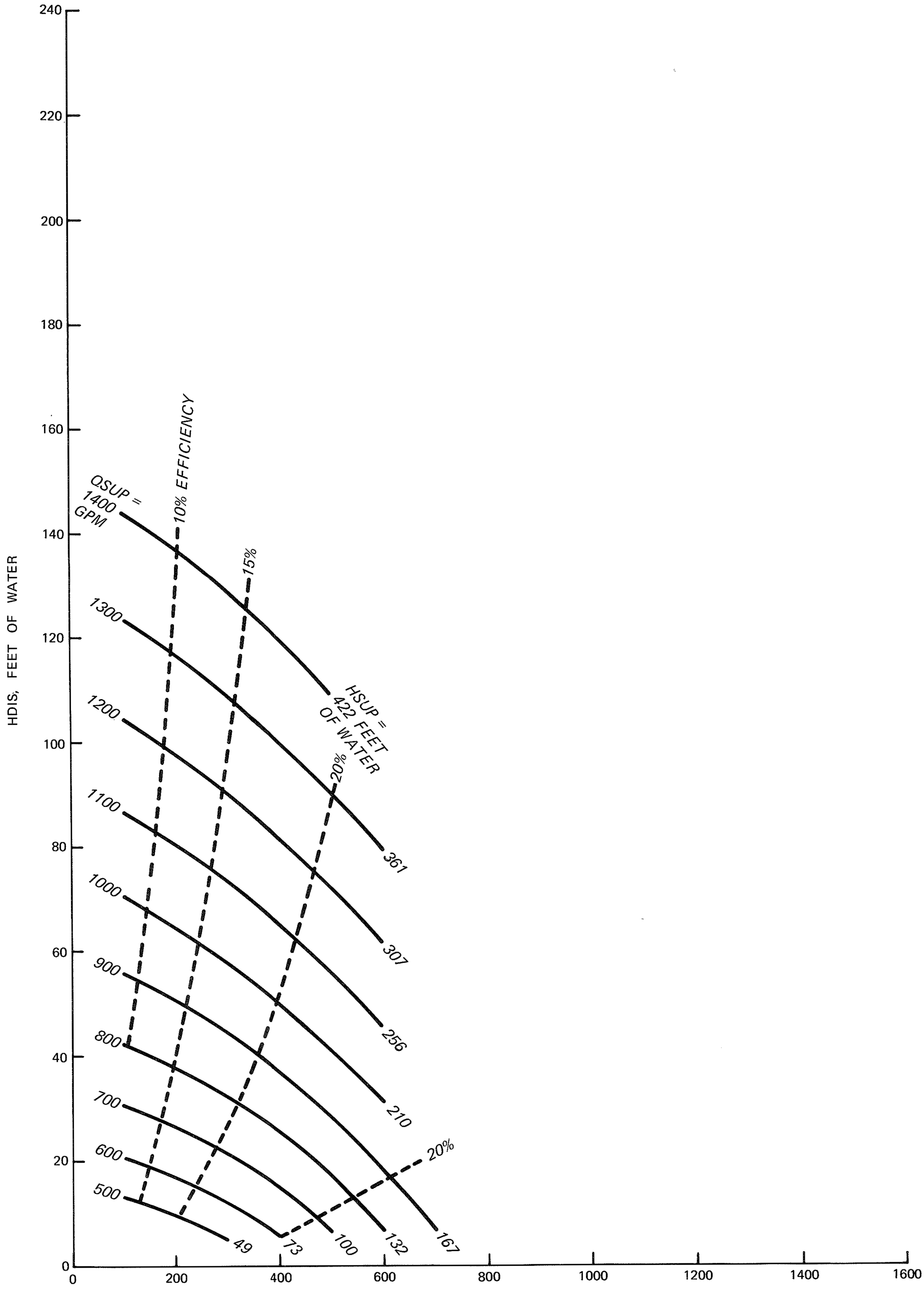
PUMPING SAND
R = 0.175



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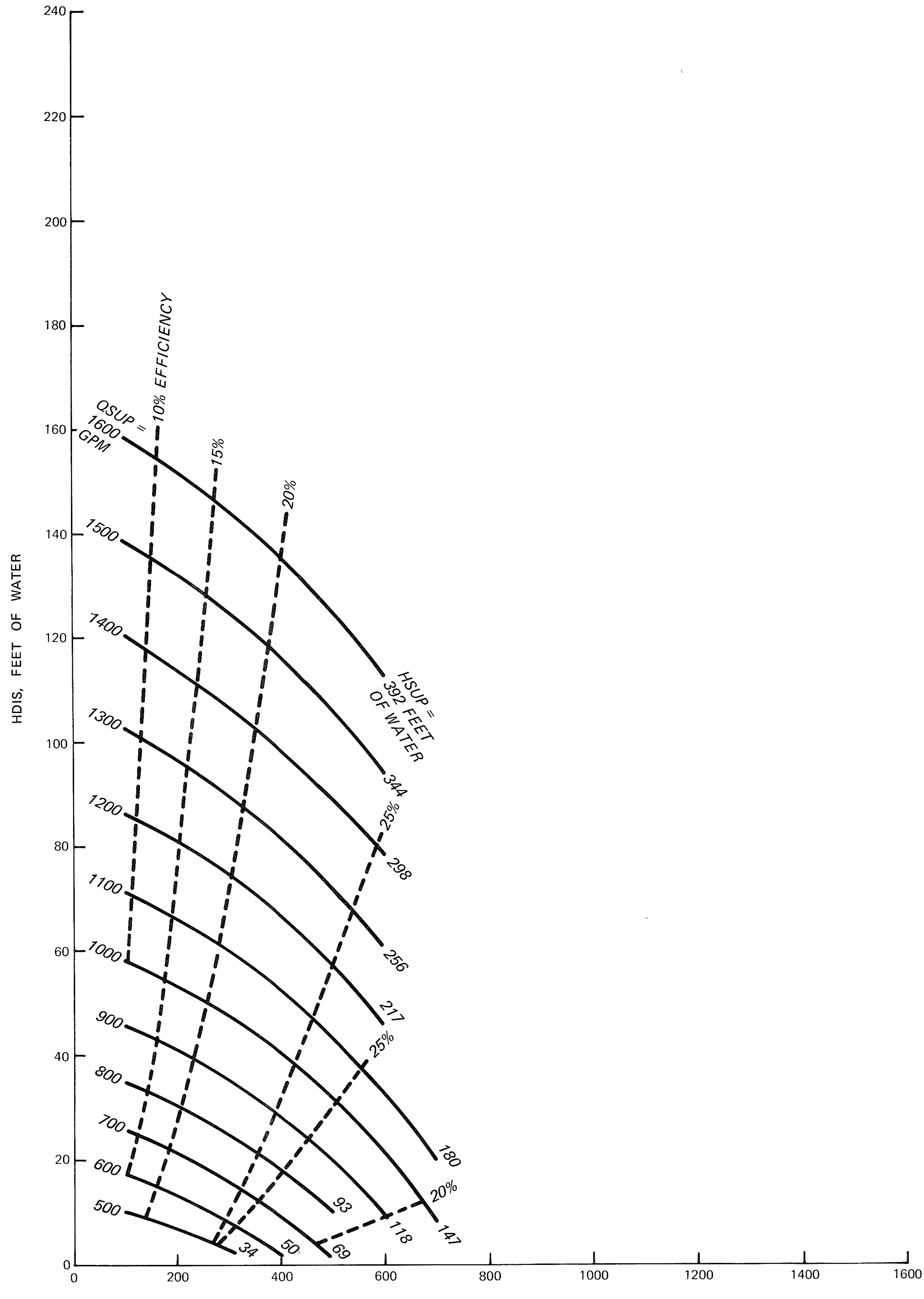

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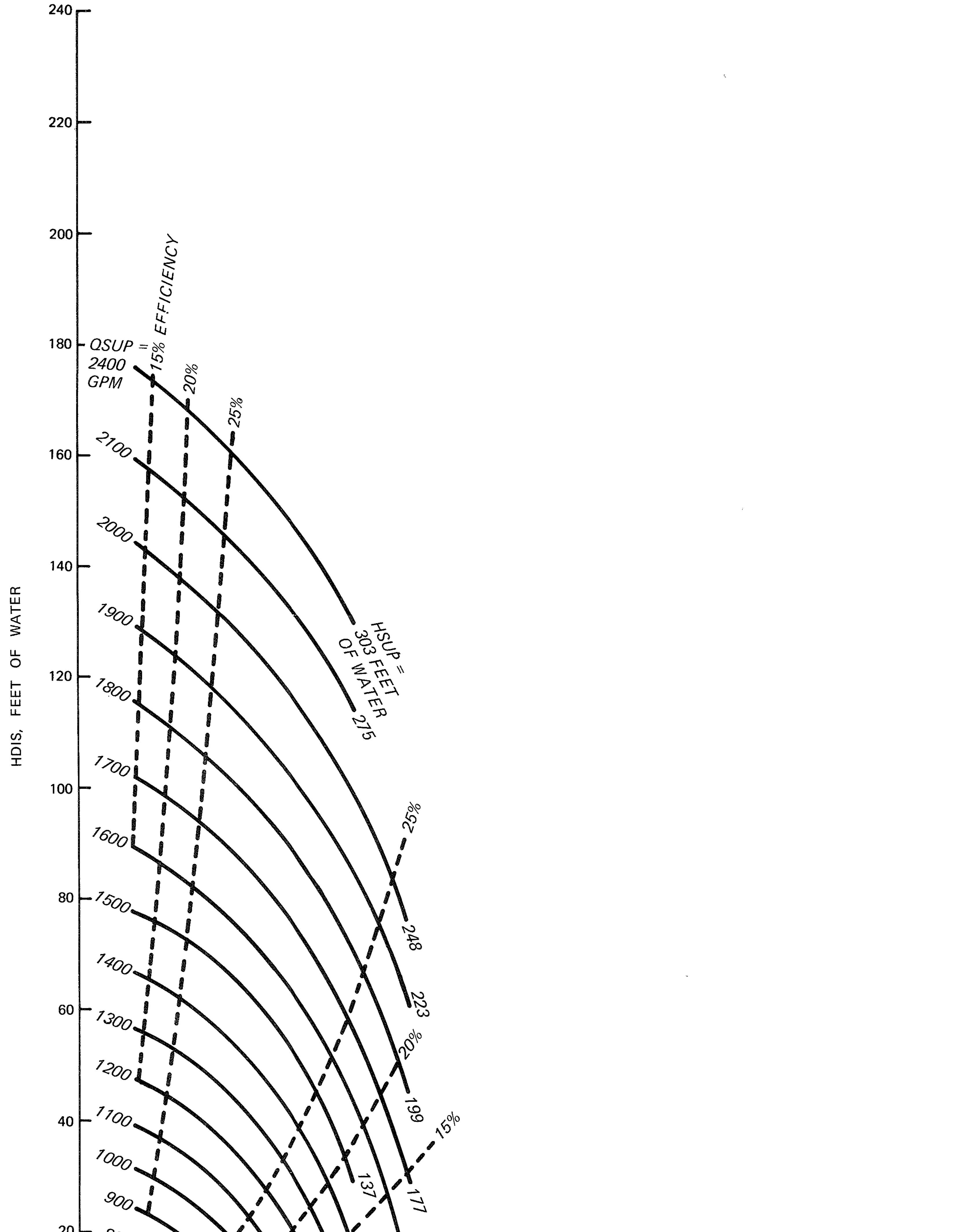

HDIS VERSUS QSUC

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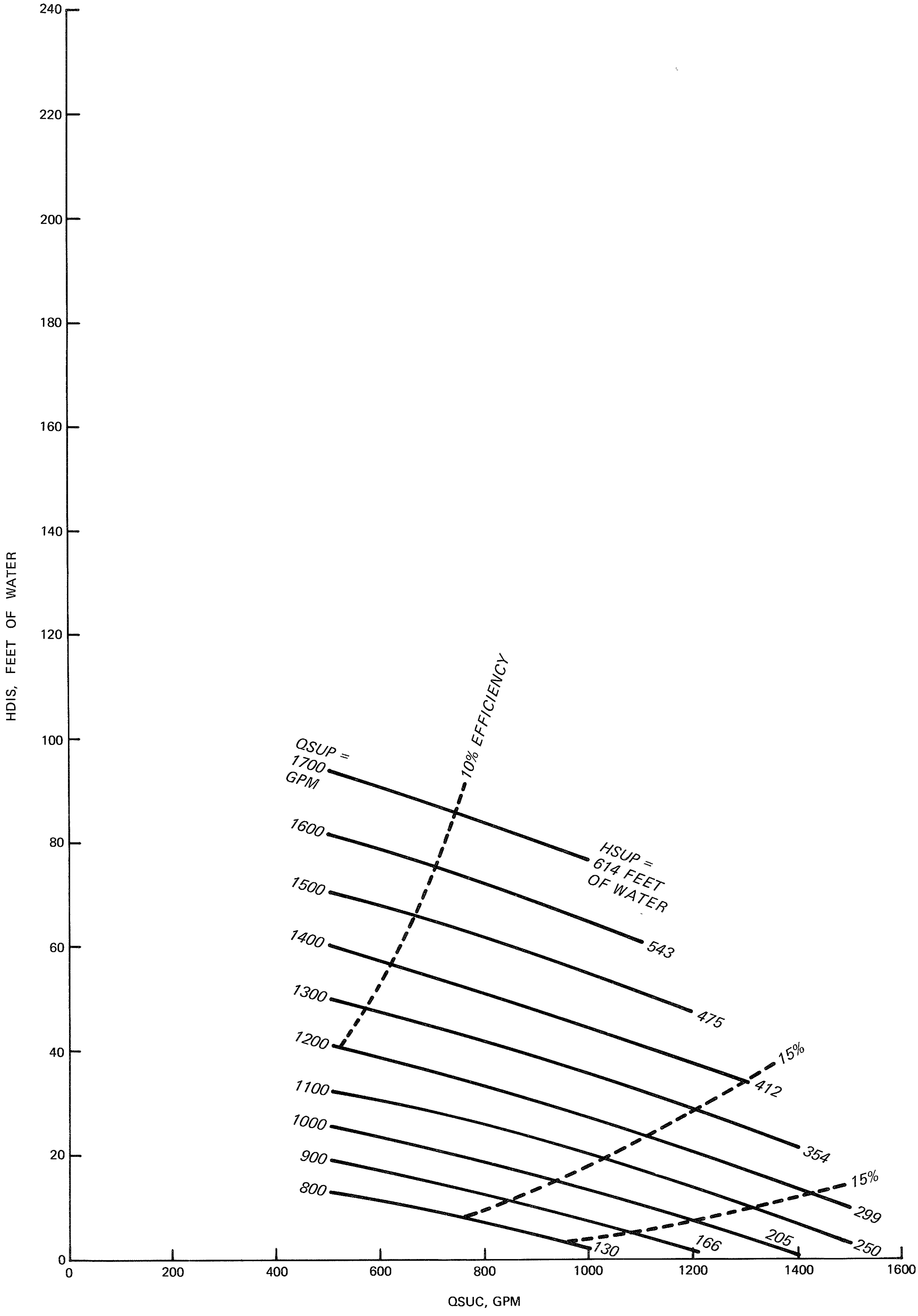
PUMPING SAND
R = 0.311

PLATE 8

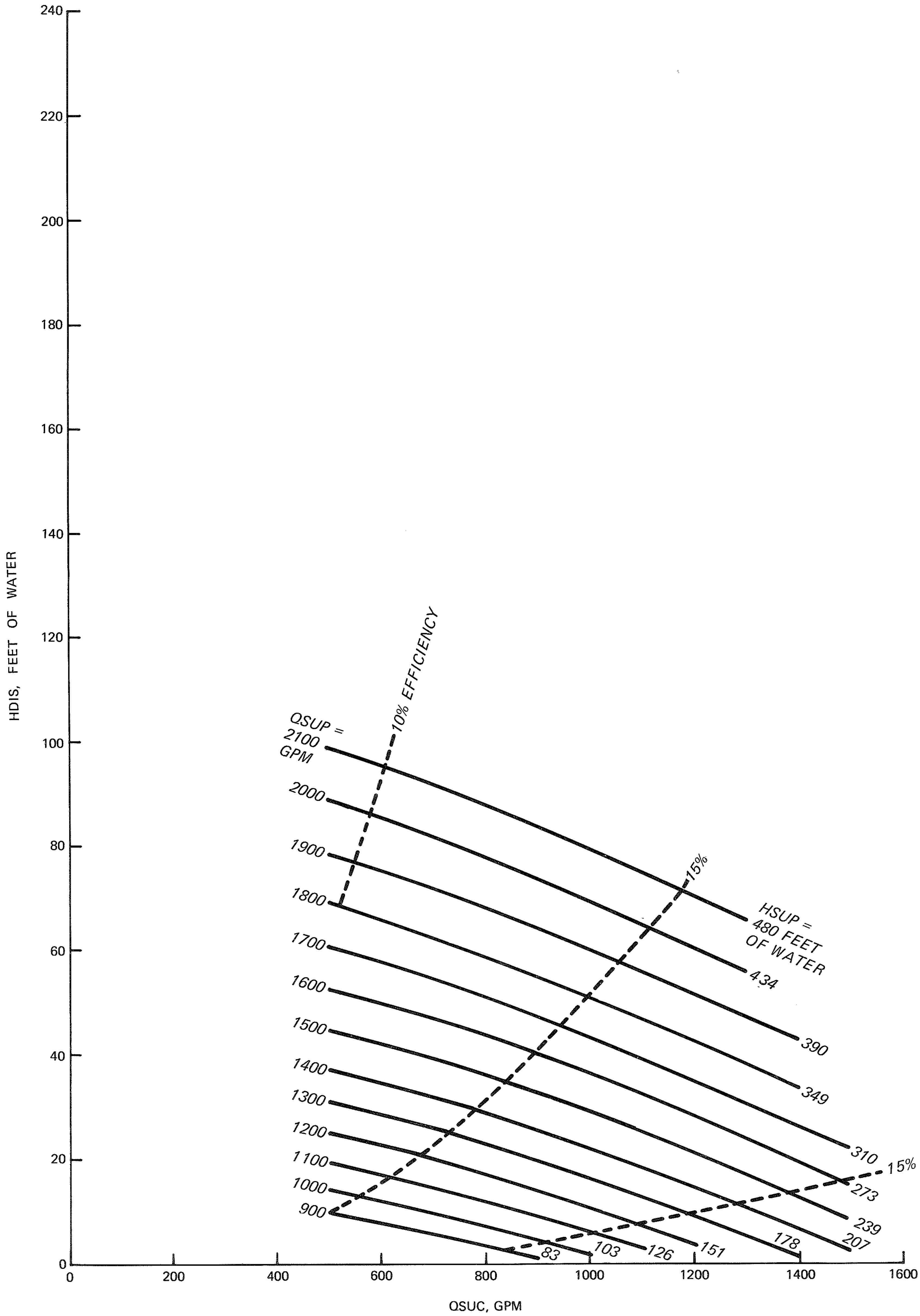


HDIS VERSUS QSUC

6×6×8 PEKOR JET PUMP PUMPING SAND

R = 0.096

PLATE 9



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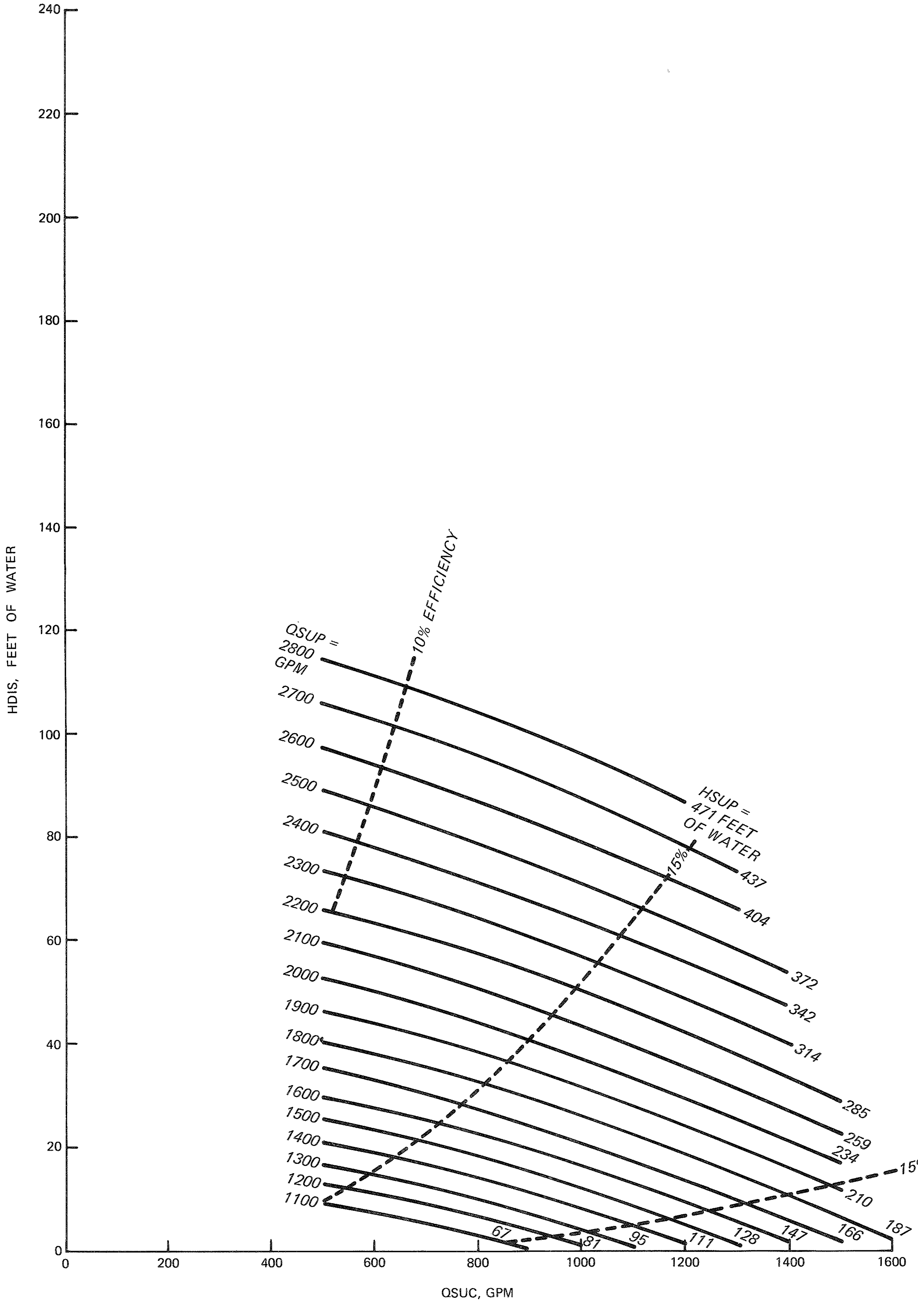
6×6×8 PEKOR JET PUMP PUMPING SAND

R = 0.138

PLATE 10

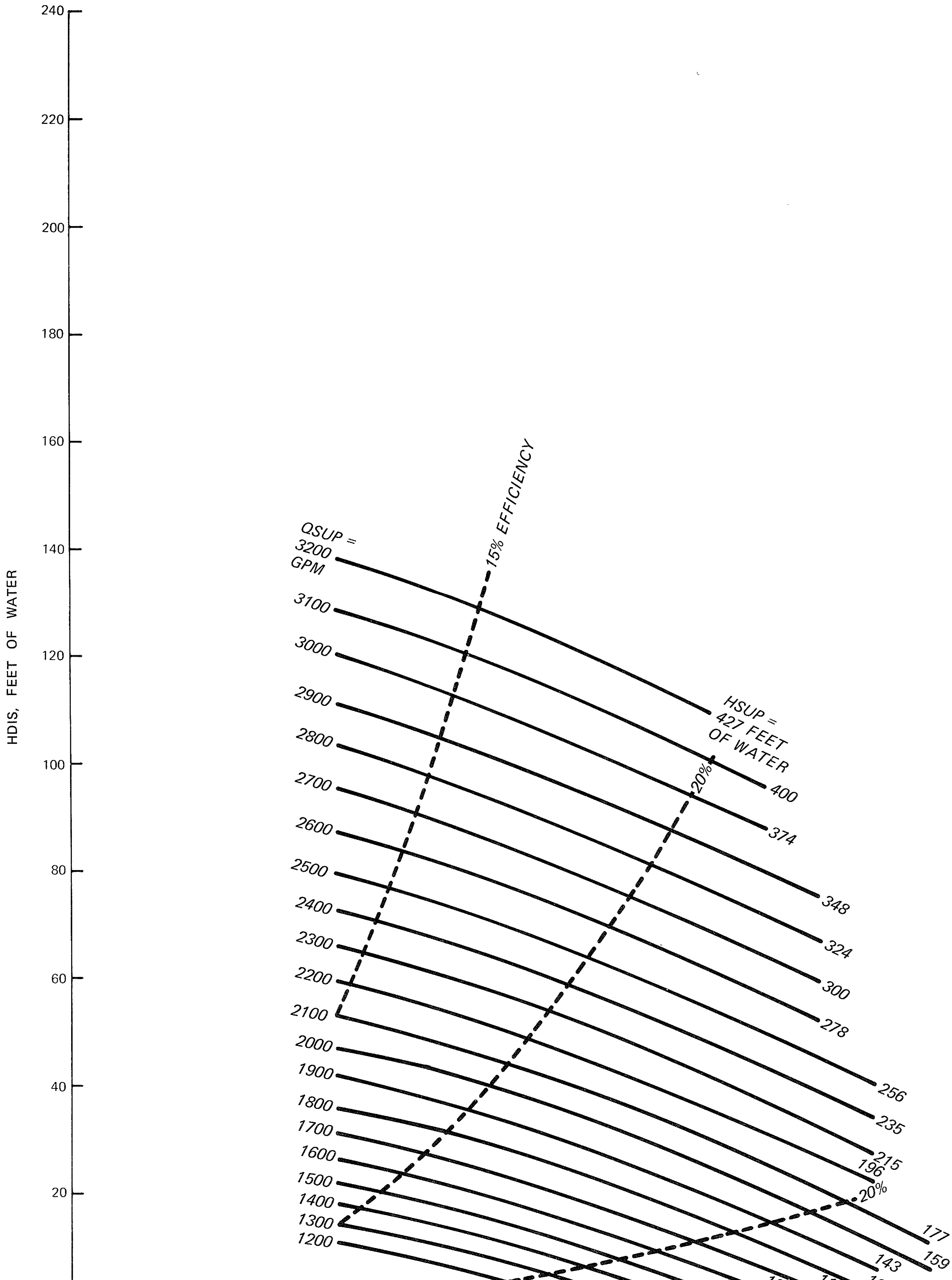


HDIS VERSUS QSUC

6×6×8 PEKOR JET PUMP PUMPING SAND

R = 0.175

PLATE 11



HDIS VERSUS QSUC

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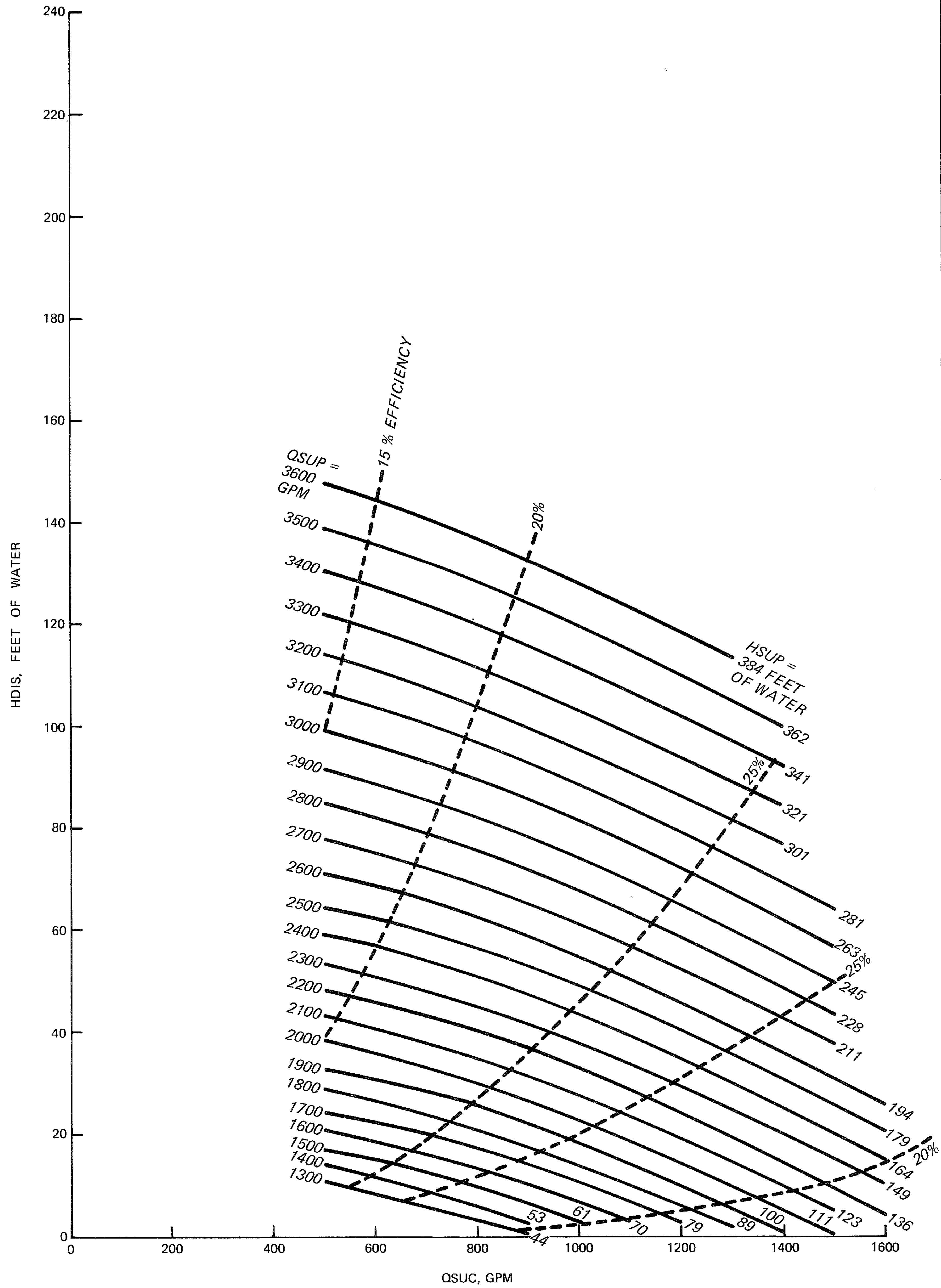
R = 0.202

PLATE 12



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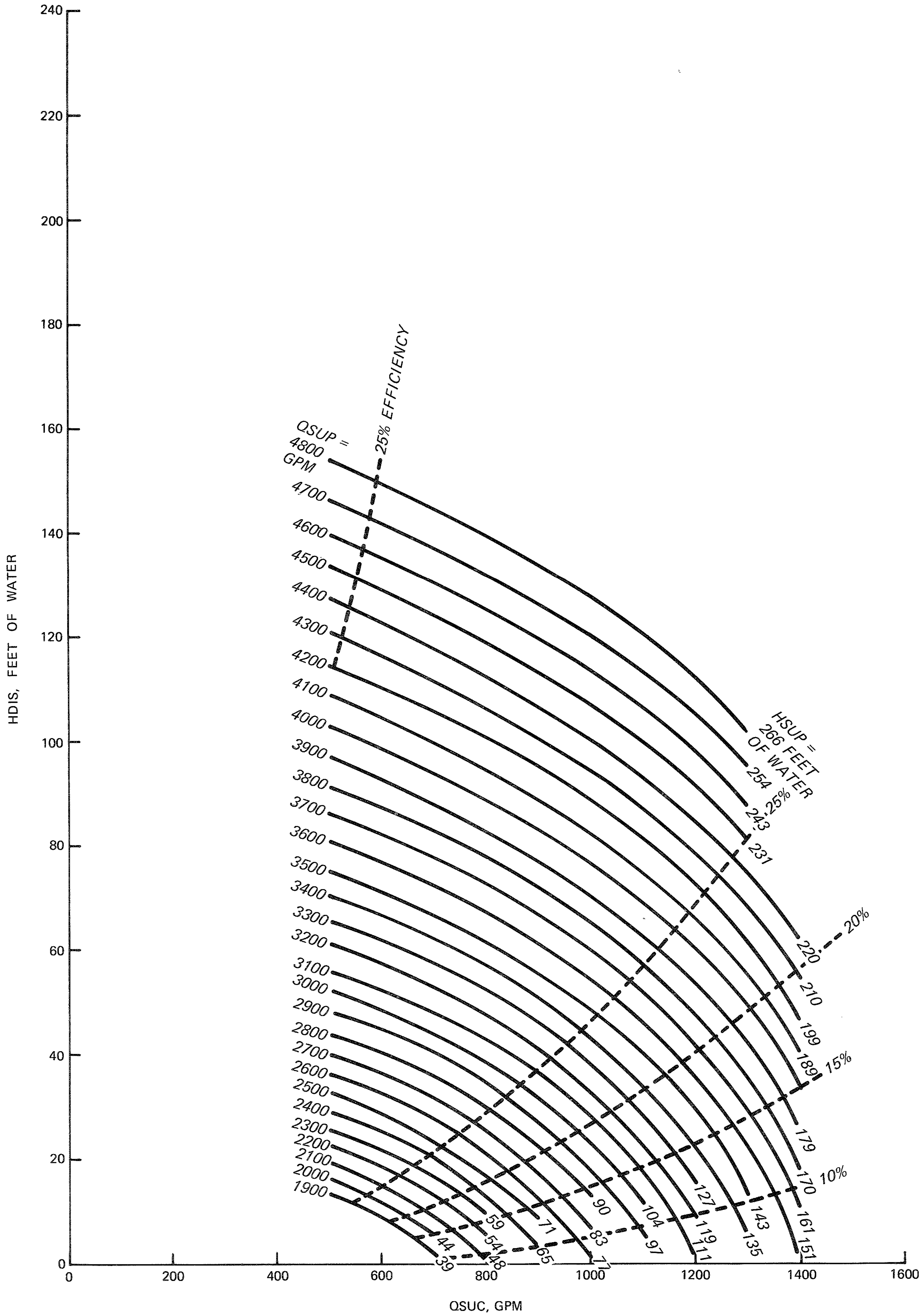

HDIS VERSUS QSUC

6×6×8 PEKOR JET PUMP PUMPING SAND

R = 0.311

World-wide sand bypassing systems: data report

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Queensland Government
Environmental Protection Agency

Hereford Inlet to Cape May Inlet

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World-wide Sand Bypassing Systems: Data Report
(Compiled 1997)

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Preface

This report has been prepared by Mr Paul Boswood, Coastal Services Branch, Environmental and Technical Services, Environmental Protection Agency, and Mr Russell Murray, formerly Project Director, Tweed River Entrance Sand Bypassing Project.

This report was prepared in 1996/97 as background information for the assessment of bypassing systems for the Tweed River Entrance Sand Bypassing Project. The information contained within this report has been obtained from a number of sources. The authors wish to thank all those who have provided assistance. In particular, the advice and feedback from project personnel within the Queensland Environmental Protection Agency, NSW Department of Land and Water Conservation, and Brown and Root as well as Queensland Transport, was greatly appreciated.

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i. List of symbols

AC = asbestos cement.
CD = Chart Datum.
cyl. = cylinder.
D_{50} = median sediment particle size.
dia. = diameter.
dir = wave direction.
HDPE SDR-9 = high density polyethylene standard density rating.
H_{max} = maximum wave height.
Hrs Op = hours operation.
Hs = significant wave height.
Hs(10%) = significant wave height exceeded 10% of the time.
Hs(50%) = significant wave height exceeded 50% of the time.
Hs,o = deep water significant wave height.
ID = inside diameter.
LWD = low water datum.
MDPE = medium density polyethylene.
MHHW = mean higher high water.
MLLW = mean lower low water.
MLW = mean low water.
MSL = mean sea level.
NW = north-west.
pa. = per annum.
PVC = polyvinyl chloride.
S = south.
SE = south-east.
std dev. = standard deviation.
SW = south-west.
T = wave period.
T_{ave} = average wave period.
T_p = spectral peak wave period.
typ. = typical or typically.
WNW = west of north-west.

ii. Dimensions and units

cy = cubic yard.
ft = feet.
gpm = gallons per minute.
hp = horse power.
hr = hour.
km = kilometre.
kV = kilovolt.
kW = kilowatt.
kWh = kilowatt hour.
lps = litres per second.
m = metre.
m^3 = cubic metre.
m^3 pa = cubic metres per annum.
m^3/yr = cubic metres per year.
mm = millimetre.
s = second.
yr = year.

1. Introduction

The Tweed River Entrance Sand Bypassing Project is a joint project undertaken by the State Governments of Queensland and New South Wales in conjunction with the Gold Coast City Council and Tweed Shire Council. The main aims are to establish and maintain a navigable entrance to the Tweed River and to enhance and maintain the amenity of the southern Gold Coast beaches.

The project involves two inter-related components, namely:

- initial dredging of the Tweed River bar and entrance area and nourishment of the southern Gold Coast beaches between Snapper Rocks and North Kirra (Stage 1).
- an artificial sand bypassing system, to operate in perpetuity (Stage 2).

To aid project delivery, world-wide experience operating various sand bypassing systems has been examined for their potential application to this project, and to expand knowledge on existing bypass technology and problems encountered.

This data report provides:

- a non-exhaustive reference list as of 1997;
- a short description of world-wide bypassing systems; and
- a set of data sheets providing a detailed brief description of selected bypassing systems.

It provides a reference source for the project team, consultants engaged for the project, potential contractors, regulating authorities, advisory bodies, the community and others with an interest in sand bypassing.

2. Terminology

This report summarises sand bypassing works undertaken around the world, with international references to these systems. Terminology used to describe key coastal works components will vary according to geographic location. This report uses the following terms for some of these key components:

Training wall: coastal structure aligned along the inlet sides and extending seawards to stabilise an inlet entrance and maintain a channel. Sometimes referred to as a jetty or breakwater.

Trestle: a structure extending seawards from the shore used for recreational rather than protective measures. Sometimes referred to as a jetty, pier, or wharf.

Breakwater: a coastal structure used to protect open coast regions from waves. Extensively used in harbours or mariners.

Weir Training Wall: a training wall with a depressed section of the wall usually near the beach to allow movement of sand into a controlled section of the channel. Usually associated with a sand trap to allow dredging in sheltered conditions.

Revetment: A protective layer usually of rock or concrete placed over a bank, scarp or in front of foreshore development to protect it from wave attack and currents.

3. Sand bypassing: general description

Natural sand bypassing is the process where the longshore sand transport (littoral drift) along an open coast travels across inlets in the direction of the net sediment transport. For inlets where the tidal prism of the inlet is small compared to the transport rate along the coast, a bar will form across the entrance of the inlet to convey sand to the other side. Such bars can be hazardous to navigation. Breakwaters or training walls may be erected along the entrance banks and seawards to stabilise movement of the inlet, to produce new inlets or harbours, and to improve navigation. While the result may be an improved entrance channel in the short term, the training walls trap the littoral drift such that the updrift beach accumulates against the training wall, whilst the downdrift beach erodes due to a lack of sand supply. In the long term, this process may continue until the sand can once again naturally bypasses around the entrance, creating another entrance bar.

To maintain a navigable entrance and neighbouring beach amenity, sand bypassing systems have been created to artificially bypass the littoral drift. A number of different systems have been developed and employed around the world. Most systems fall under one or a combination of the following generic types:
1. water based mobile systems including maintenance dredging either of the channel or sand trap;
2. land based mobile systems; and
3. fixed systems such as a trestle- or breakwater-mounted.

4. World-wide sand bypassing systems

A reference list has been prepared from a wide number of sources of information and is presented in section 8 below. Appendix A lists the world-wide sand bypassing systems found from a non-exhaustive search of the cited references. The locality of these systems are shown in figure A1.

No list of sand bypassing systems (including this one) can be regarded as fully complete because different definitions of bypassing are used in different jurisdictions and by different investigators. The list covers major systems in operation, other systems trialed or operated for a limited time, and some systems in development phase as of 1997.

5. Selected sand bypassing systems

Based on this list, the available references, and the knowledge of project staff, a selection of sand bypassing systems was chosen for a more detailed summary to cover a range of various types of systems in operation. The list of selected bypassing systems considered for a more detailed summary is given in table 1.

Table 1. List of selected bypassing systems.

Plant location	Country	Type of bypass system
Nerang River Entrance, Queensland	Australia	Trestle and jet pump system (fixed).
Boca Raton, Florida	USA	Weir training wall and trap with conventional dredging.
Channel Islands Harbour, California	USA	Detached breakwater and sand trap with biannual dredging and pumping down coast of Port Hueneme.
Dawesville, Western Australia	Australia	Crawler excavator (mobile) and crawler mounted pump system.
Indian River Inlet, Delaware	USA	Jet pump and crane (mobile system).
Oceanside Harbour, California	USA	Jet pumps and fluidisers (experimental fixed system).
South Lake Worth Inlet, Palm Beach County, Florida	USA	Fixed hydraulic suction dredge with a rotating boom (fixed).

A data sheet on each of these systems is given in appendices B to H. These data sheets provide a systematic description of key environmental and system parameters, a site description, and a specific reference list with some additional references not given in the bibliography. The measuring units provided in these appendices depends on the source of information and varies between metric and imperial. A description of unit abbreviations is provided in section 2.

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Appendix A

List of sand bypassing systems (as of 1997).

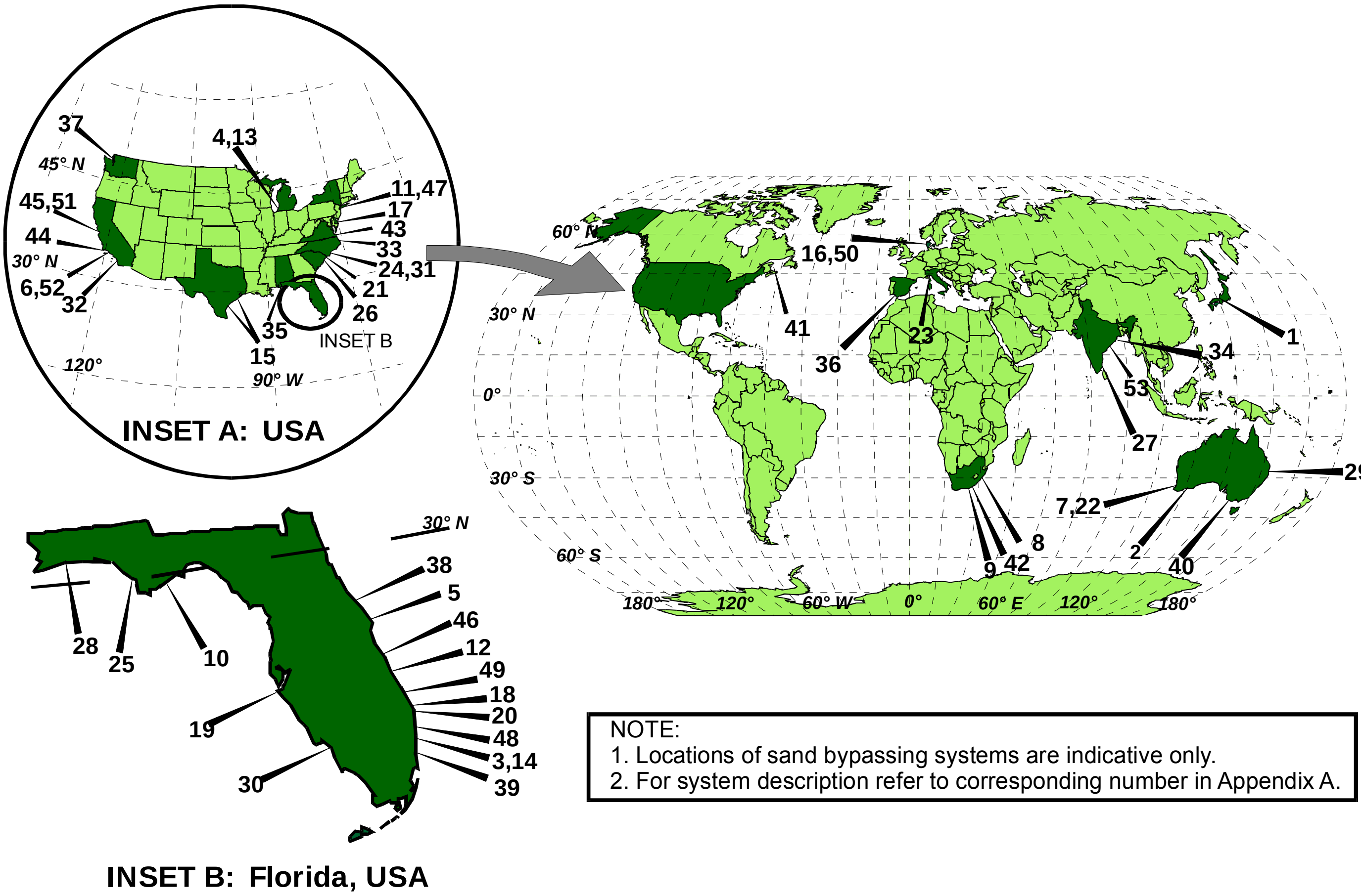
	Plant location	Country	Type of bypassing system [and reference]
1.	Amanohashidate coast	Japan	Investigation and trial only [54,258].
2.	Bandy Creek Harbour, Esperance, Western Australia	Australia	Natural bypassing around entrance with offshore breakwater to prevent sediment returning (constructed 1989) [24].
3.	Boca Raton, Florida	USA	Weir training wall and trap with conventional dredging [71,81].
4.	Bridgman, Michigan (Lake Michigan)	USA	Small quantities by hydraulic bypassing from accretion fillet with remainder of nourishment from mined sand from dunes (1971-1973) [120].
5.	Canaveral Harbour, Florida	USA	Conventional dredging from nearshore borrow area (recommended plan as of 1995) [108].
6.	Channel Islands Harbour, California	USA	Detached breakwater and sand trap with biannual dredging and pumping down coast of Port Hueneme [106,211,226].
7.	Dawesville, Western Australia	Australia	Crawler excavator (mobile) and crawler mounted pump system [22,50,111].
8.	Durban	South Africa	Maintenance dredging of entrance and trap updrift of breakwater (installed 1982). Considering fixed system of jet pumps as of 1996 [10,129,191].
9.	East London Port	South Africa	Maintenance dredging of trap [129].
10.	East Pass, Florida	USA	Weir training wall and trap with conventional dredging (1969-1985) [207].
11.	Fire Island, New York	USA	Maintenance dredging of bay shoals [41].
12.	Ft. Pierce, Florida	USA	Maintenance dredging of bay shoals [41].
13.	Great Lakes	USA	Mobile system consisting 200 mm jet pump with cutting assists, flotation buoy, and two propulsion jets connected by flexible hose to two land-based trailers supporting pumping and control equipment to travel between harbours (constructed in 1978) [189].
14.	Hillsboro Inlet, Florida	USA	Weir training wall and trap with 36 cm floating hydraulic dredge (mobile) [81,109].
15.	Houston, Corpus Christie, Texas	USA	Dredging of bay and ocean shoals with disposal offshore [41].
16.	Hvide Sande	Denmark	Maintenance dredging of entrance, as well as nourishment from offshore borrow site. Booster station in entrance for pumping during summer [115].

	Plant location	Country	Type of bypassing system [and reference]
17.	Indian River Inlet, Delaware	USA	Single jet pump and crane (mobile system) [1,6,56,61,65,67,69,96,131,181,182,183,234,240].
18.	Jupiter Inlet, Florida	USA	Conventional dredging of trap (constructed 1966) in Inlet [81].
19.	Lake LaVista Channel, Anna Maria Island, Florida.	USA	Demonstration of sand fluidisation system in 1986 [72].
20.	Lake Worth Inlet, Florida	USA	Electrically driven moveable suction head suspended from a boom (1960-1990); and maintenance dredging of entrance [150,191,203,211,227,261].
21.	Little River Inlet, South Carolina	USA	Weir in both training walls for bypassing. Weirs covered, to be opened when required [174].
22.	Mandurah Inlet, Western Australia	Australia	Crawler excavator (mobile) and crawler mounted pump system [22,50,111].
23.	Marina di Carrara	Italy	A 250mm suction pipe dredge mounted and swivels on a fixed circular concrete trestle off the updrift side of the harbour breakwater (installed 1972) [188,191].
24.	Masonboro Inlet, North Carolina	USA	Weir training wall and trap with conventional dredging (commenced 1966) [141,201,211].
25.	Mexico Beach, Florida	USA	Two fixed jet pumps operating from crater (constructed 1976). Replaced by floating dredge in 1978 [167].
26.	Murrells Inlet, South Carolina	USA	Weir training wall and trap with conventional dredging (mobile) [12,172].
27.	Nagapattinam (Bay of Bengal)	India	Pump on trestle pier with shutters [41].
28.	Navarre Beach, Florida	USA	Considering moveable dredge plant as of 1989 [23].
29.	Nerang, Queensland (Gold Coast Seaway)	Australia	Ten jet pumps along a trestle (fixed) (commenced 1986) [58,59,60,137,140,173,175,176,180,191,206, 216,256,257].
30.	New Pass, Florida	USA	Maintenance dredging of ocean shoal [41].
31.	New River Inlet, North Carolina	USA	Sidecasting dredge with split hull barge for deposition within 2m depth (experiment, 1976) [199,200].
32.	Oceanside Harbour, California	USA	Jet pumps and fluidisers (experimental fixed system, 1989 to 1996) [11,14,18,21,80,152,153,166,226,228, 246].
33.	Oregon Inlet, North Carolina	USA	Cutter-suction pipeline dredge operating in openings in proposed entrance walls (in consideration, 1985) [53,116,117].

	Plant location	Country	Type of bypassing system [and reference]
34.	Paradip, Orissa (Bay of Bengal)	India	Moveable plant on trestle with additional maintenance dredging [41].
35.	Perdido Pass, Alabama	USA	Weir training wall and trap with conventional dredging (construction commenced in 1968) [207].
36.	Playa de Castilla beach (Huelva Spain)	Spain	Trailing suction hopper dredge dredging shoals trapped by updrift dike, and pumping via 2 km long steel submerged pipeline to downdrift beaches [86].
37.	Point Roberts Marina, Strait of Georgia (northern Puget Sound), Washington	USA/ Canada border	Small-scale land based equipment bypassing beach sand by truck (mobile) [132,133].
38.	Ponce de Leon Inlet, Florida	USA	Weir training wall and trap with conventional dredging [201].
39.	Port Everglades, Florida	USA	Nourishment from offshore borrow site, and maintenance dredging [41].
40.	Portland, Victoria	Australia	Sand shifter system operated from breakwater or from barge [129].
41.	Prince Edward Island	Canada	Trailer-mounted jet pump and telescoping hydraulic crane (mobile, commenced 1982) [191].
42.	Richards Bay	South Africa	Maintenance dredging of trap [129].
43.	Rudee Inlet, Virginia Beach, Virginia	USA	Weir training wall and trap with conventional dredging (1968-1972). Two jet pumps on flexible hose (semi-mobile) installed in 1972 at trap, supplemented by maintenance dredging [188].
44.	Santa Barbara, California	USA	Maintenance dredging of harbour [211,226,248].
45.	Santa Cruz, California	USA	Annual maintenance dredging of entrance channel (commenced 1965 with floating pipeline dredge) [126,188].
46.	Sebastian Inlet, Florida	USA	Maintenance dredging of channel sand trap with periodic transfer to downdrift beaches (commenced in 1989) [229].
47.	Shinnecock Inlet, New York	USA	Design/construct of inlet including bypass system in process as of 1992 [156].
48.	South Lake Worth inlet, Palm Beach County, Florida	USA	Fixed hydraulic suction dredge with a rotating boom (fixed) [8,51,158,191,260].
49.	St. Lucie, Florida	USA	Weir training wall and trap with conventional dredging (proposed as of 1987) [41].
50.	Torsminde	Denmark	Maintenance dredging of entrance, as well as nourishment from offshore borrow site [115].

	Plant location	Country	Type of bypassing system [and reference]
51.	Twin Lakes Harbour, Santa Cruz, California	USA	Fixed plant (commenced 1972) [41].
52.	Ventura, California	USA	Detached breakwater (constructed 1972) and sand trap with annual dredging (bypassing and some backpassing) [226].
53.	Visakhapatnam (Bay of Bengal)	India	Detached breakwater trap and transfer by pipeline across entrance to harbour [41,79,185].

***Figure A1**: Locality of world-wide sand bypassing systems.*



Appendix B

Data sheet: Nerang River Sand Bypassing System, Queensland, Australia.

Location:	The Nerang River flows to the sea through a broad shallow tidal estuary called the Broadwater, meeting the Pacific Ocean between the southern end of South Stradbroke Island and the Southport Spit. The entrance is located at the northern end of the City of Gold Coast, south-east Queensland, Australia.
Problem:	The progressive movement of the entrance northwards at a rate of 20 - 40 m per year has involved accretion of the Southport Spit and erosion of the southern tip of South Stradbroke Island. Hazardous navigation through the changing entrance shoals, and the possible threat of breakthrough at the South Stradbroke Island township of Currigee in the future, lead the Queensland Government to train and stabilise the river mouth between September 1984 to May 1986. The construction included revetments and breakwaters, opening of a new entrance and closure of the old entrance, creation of Wavebreak Island and Broadwater channels, and installation of a fixed bypass system.
Wave climate:	Based on recorded wave data offshore from Southport in approx. 40 m depth for 1987 - 1994: modal Hs(50%) = ~1 m H_{max} = 9.98 m during Tropical Cyclone Roger The majority of the waves range in height of Hs = 0.25 - 3.0 m (99 %) with 65 % of the data occurring within Hs = 0.5 - 1.25 m. The wave period (spectral peak) ranges typically between 3 and 15 s (99 %) with 65 % of the data within Tp = 7 - 11 s. The wave climate is influenced by the predominant south-easterly swells with intense storms associated with low pressure systems and tropical cyclones approaching from the north.
Inlet characteristics:	Nerang River: catchment = 480 km^2; semidiurnal mean spring tide range = 1.3 m extending to a limit of 21 km upstream from the mouth.
Inlet usage:	Recreational boating, fishing, and commercial vessels (for recreational hire).
Sediment characteristics:	D_{50} = 0.27 mm for the intertidal sands on adjacent beaches (ranges from 0.2 to 0.3 mm along the profile).
Drift rate:	Net northerly transport = 500,000 m^3/yr (~654,000 cy/yr) (Beach Protection Authority, 1981). Gross transport = 655,000 m^3/yr (~857,000 cy/yr). Northerly transport = 575,000 m^3/yr (~752,000 cy/yr). Southerly transport = 80,000 m^3/yr (~105,000 cy/yr).
Beach erosion rate:	The bypass system was constructed in conjunction with the training of the entrance and so there was no erosion as a result of the entrance. Before training of the inlet, there was a progressive movement of the entrance northwards at a rate of 20 - 40 m per year.
Type of bypass:	Ten jet pumps along a trestle (fixed).

Bypass system components:	Clear water intake from Broadwater through a 4 ft (~1.2 m) dia. concrete pipe; low pressure pump station with two 150 kW (200 hp) turbines (total 780 lps, 10,300 gpm); 24 inch (600 mm) dia. AC pipeline 2,300 ft (~700 m) long to the control building; high pressure jet water supply pumps housed in control station consisting of two 560 kW (750 hp) Centrifugal pumps (total 770 lps, 10,200 gpm); 14 inch (450 mm) coal tar epoxy lined water supply pipeline; 6 inch (150 mm) feed pipelines to jet pumps; ten 3.5 inch (90 mm) Genflo sandbug jet pumps with rate of 135 cy/hr (~100 m^3/hr) spaced 30 m apart along a 490 m long trestle; an elevated 23 inch (600 mm) dia. slurry pipe flume (1,214 ft or approx. 370 m long), on a 2.5 % slope to gravity feed into a density adjusting slurry pit which is a conical 189 cy (145 m^3) hopper; discharge pump housed in control station consisting of a 710 kW (950 hp) Centrifugal pump (total 489 lps, 6,500 gpm). The jet pumps are lowered up to 11 m below mean sea level and create a trap of length 270 m. The trestle consists of a timber deck supported on steel piles. The jet pumps run on rails attached to the steel support piles to allow for installation and removal for maintenance work. The operations are controlled by an automatic programmable logic controller. A nuclear density meter and electromagnetic flow meter are installed in the discharge line for the control of the flow rate and slurry solids concentration by the automatic system, and for operation monitoring records. The system is powered by an 11 kV underground cable.
Outlet type:	406 mm (16 inch) dia. polyurethane lined steel pipe discharging at approximately the high water level, approx. 400 m north of the northern breakwater. Three outlet locations were considered in the design of the system, the further most discharge point being approx. 1,710 ft (~520 m) north of the northern training wall. The discharge pipe passes through steel sleeve tubes in the rock training walls for protection, and passes beneath the channel with pile supports.
Bypass rate:	Design Parameters: Average rate = 500,000 m^3/yr; peak annual rate = 750,000 m^3; nominal transport capacity = 300 m^3/hr; maximum 5 day transport = 100,000 m^3; maximum monthly transport = 200,000 m^3; maximum sand trap capacity = 40,000 m^3. The system was designed for the operation of 4 to 7 jet pumps with nominal capacities of 335 to 580 m^3/hr and an operating performance of 3.15 kWh/m^3. Operational experience has indicated the use of 3 to 5 jet pumps to be more effective.
Degree of bypassing: (e.g. all, 50%, etc.)	Designed for 100 % bypassing, however an unknown quantity of sand bypasses the trestle. No dredging of the entrance channel has been required.

Costs: Construction of bypass system and ancillary works (Jan 1985 - June 1986): $8,134,000 (AUD).

Operating expenses since commencement of bypassing: (July to June)

ITEM	89/90	90/91	91/92	92/93	93/94
Electricity	183,400	152,100	167,600	140,200	241,000
Salaries and Wages	90,700	93,400	95,000	102,800	95,700
Repair and Maintenance	111,900	100,100	184,000	318,800	266,200
TOTAL ($)	386,000	345,600	446,600	516,800	602,900

ITEM	94/95	95/96	96/97
Electricity	221,847	154,421	163920
Salaries and Wages	104,054	119,573	112,204
Repair and Maintenance	360,544	397,438	459,165
TOTAL ($)	686,445	671,432	735,289

Funding: State Government.

Contract type: Contract to design and construct. Operations and maintenance conducted by owner. A contract was let for the management of the structure as a fishing platform by the general public. In 1992, a painting contract was let for the complete painting requirements for the offshore structure.

Owner: State Government, Queensland Department of Transport.

Operator: State Government, Queensland Department of Transport.

Supervisor of operations: State Government, Queensland Department of Transport, Marine Services Section.

Staffing: Total of 3 people: an operator, assistant operator, and labourer working a normal daytime shift.

Operating cycle: The system runs automatically overnight, and sometimes weekends, to take advantage of cheaper electricity rates. The operator selects the appropriate jet pumps (depending on sand supply in each crater and the presence of debris) and commences pumping in the afternoon to run through the night. The system automatically performs an initial warm up and flushing of the lines, before the valves to the jet pumps are opened and bypassing commences.

Environmental constraints: No known constraints. Bypassing takes place at night and the discharge point is on an undeveloped part of an island, therefore having no direct effect on beach users.

Environmental management issues: A monitoring programme is undertaken to examine the performance and impacts of the entire project. This includes undertaking hydrographic surveys, aerial photography, sand bypassing records, visual observation of beach and surf zone conditions, wave recording, and the recording of water levels in the Nerang River and the Broadwater.

Commencement date of bypassing: May, 1986.

Performance:
(include any leakage to inlet, formation of entrance bar, etc.)

Summary of Sand Bypassing Statistics (July to June)

ITEM	89/90	90/91	91/92	92/93	93/94
m^3 Pumped	378,756	440,287	376,841	286,974	569,013
kWhrs	2,077,111	2,101,010	1,859,789	1,608,946	2,434,098
kWhr/m^3	5.48	4.77	4.93	5.61	4.28
Hrs Op	1839	1568	1433	1210	1642
m^3/hr	206	281	263	237	347
\$/m^3	1.02	0.78	1.18	1.95	1.06

Summary of Sand Bypassing Statistics (Continued)

ITEM	94/95	95/96	96/97
m^3 Pumped	570,293	408,917	563,831
kWhrs	2,250,130	1,566,335	2,146,236
kWhr/m^3	3.95	3.83	3.81
Hrs Op	1518	1117	1539
m^3/hr	376	366	366
\$/m^3	1.20	1.64	1.30

For the financial years (July to June) up to 1989/90, the system had delivered 138,236 m^3 (85/86), 544,002 m^3 (86/87), 464,435 m^3 (87/88), and 392,821 m^3 (88/89). For the 1997/98 financial year the system pumped a total of 587,869 m^3. The system operates with 3 to 5 jet pumps achieving capacities in the range of 330 to 540 m^3/hr depending on factors such as weather, blockages, density of sand and slurry, and sand supply to the traps. As of September 1998, a bypassing rate of approx. 420 m^3/hr has been able to be maintained owing to continual improvements to the efficiency of the system.

The system was originally designed to create a long continuous sand trap of 270m length under the trestle. However, in practice, individual steep slope craters (typically 1:1 to 1:1.5) have formed around each jet pump.

There has been an unknown quantity of sand bypassing the trestle and building a bar formation, but no maintenance dredging of the channel between or seaward of the walls has been required. There has been some build-up of sand requiring dredging at the Broadwater end of the entrance.

There has been some significant scouring of the channel from strong ebb currents which has exposed the discharge pipe. The pipe has subsequently been supported by piles. The ebb tidal bar is forming further offshore then prior to the works but is not a problem for navigation. Some occasional growth of the sand spit around the southern training wall and into the entrance occurs and there is a progressive sand build-up in the nearshore areas to the north of the entrance.

The jet pumps are subject to clogging from debris especially during and after storm events. This has resulted in the plant not being operational during storms as originally envisaged. Key components of the jet pumps have undergone severe wear and have been through a series of improvements to reduce the problem. Difficulties are also encountered in retrieving the jet pumps for maintenance works owing to the limited working area for the crane.

Present plant status:
(as of 1996)

Successful. Still in operation.

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***Figure B1**: Nerang River Entrance Sand Bypassing System, Locality plan (Munday, 1995).*

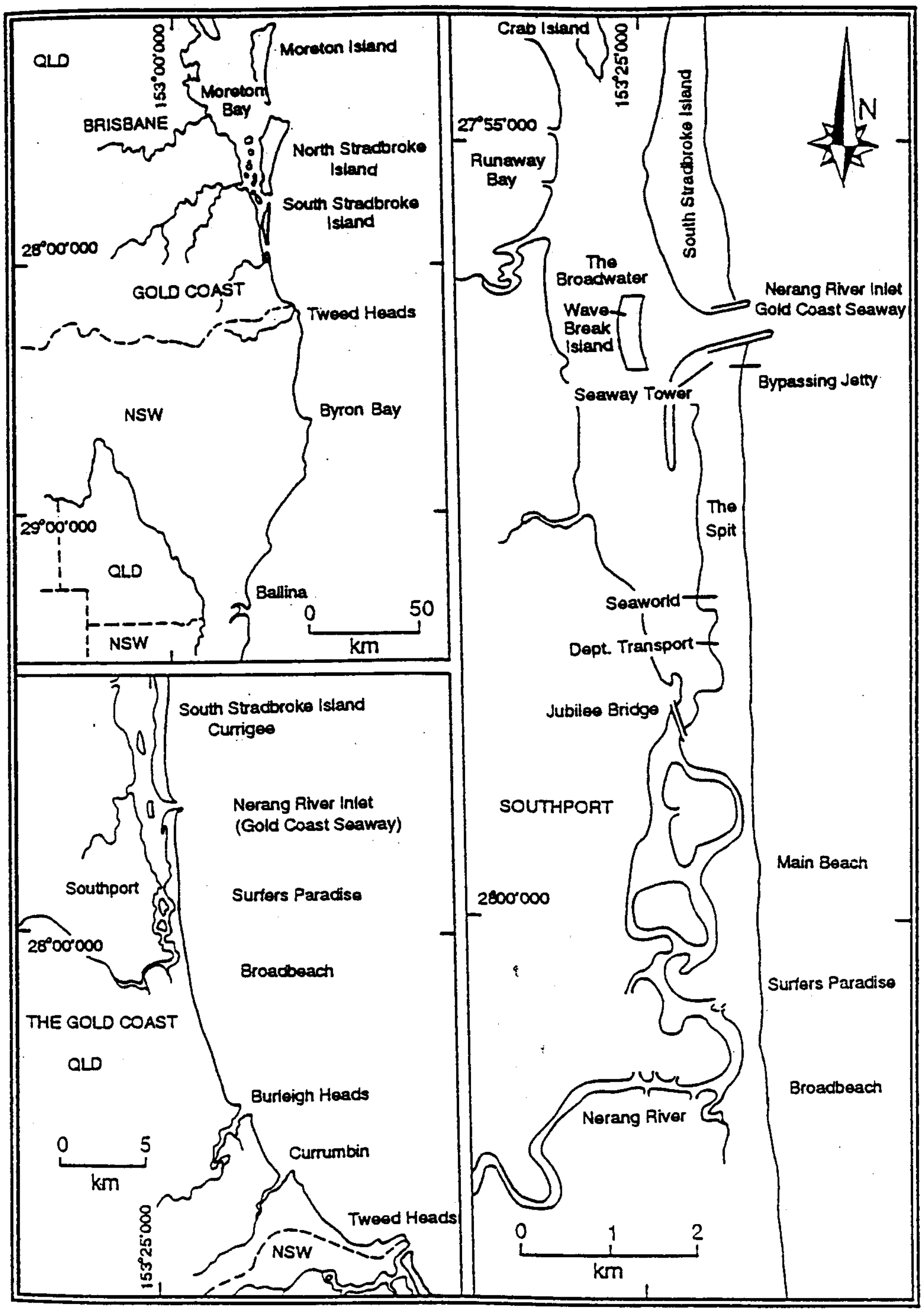


***Figure B2**: Nerang River Entrance Sand Bypassing System, System layout (Witt and Hill, 1987).*

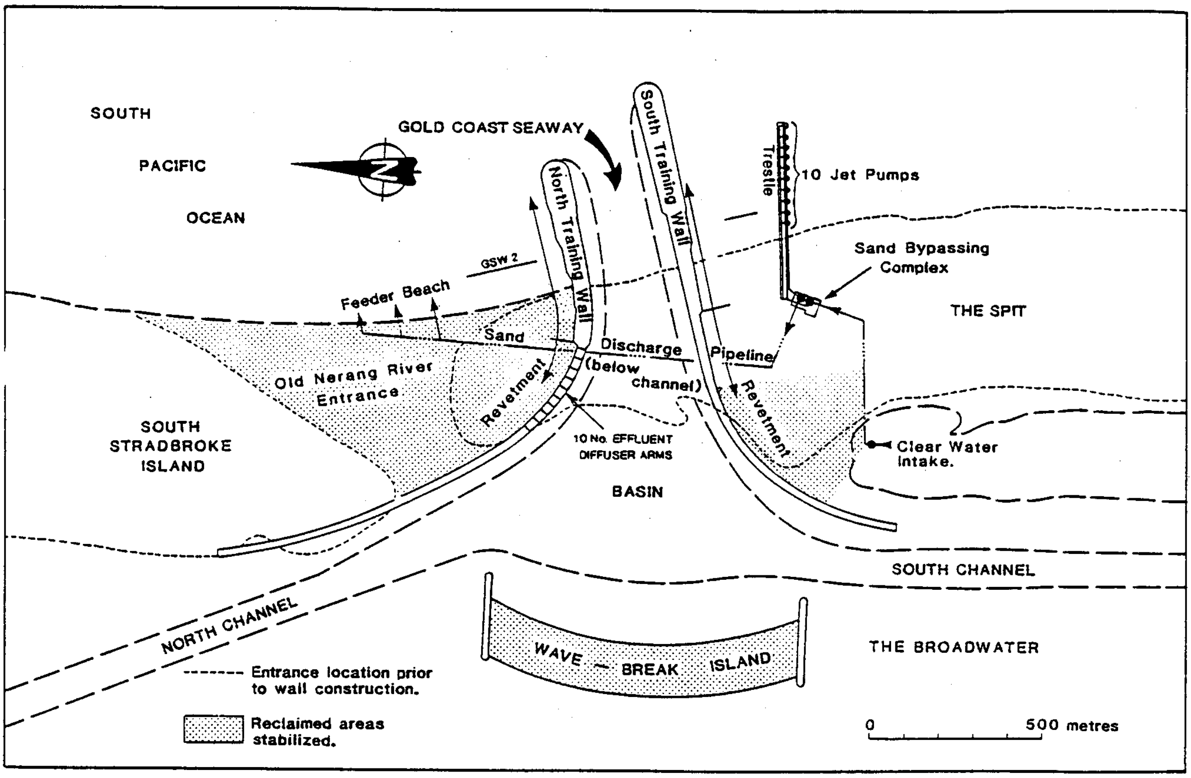
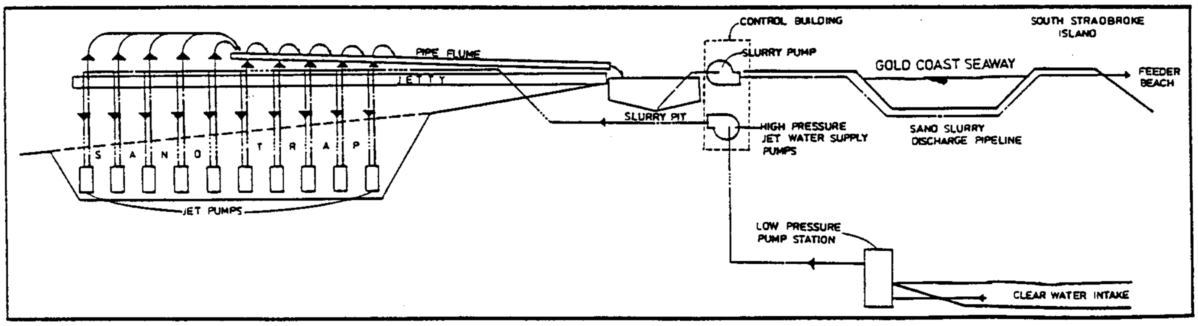


***Figure B3**: Schematic of Nerang River Entrance Sand Bypassing System (Witt and Hill, 1987).*



Appendix C

Data sheet: Boca Raton Inlet Sand Bypassing System, Florida, U.S.A.

Location:	Boca Raton Inlet is a natural entrance connecting Lake Boca Raton to the Atlantic Ocean. The inlet is situated within the City of Boca Raton in the south-eastern region of Palm Beach County, Florida, USA, between South Lake Worth Inlet (23 km to the south) and Hillsboro Inlet (9 km to the north).
Problem:	Erosion of the southern beaches and the creation of an ebb shoal at the entrance becoming a hazard to navigation.
Wave climate:	No published information available for this site, however refer to the Data Sheet for South Lake Worth Inlet (Appendix H) which is 23 km to the north of this site, for some general idea of conditions.
Inlet characteristics:	Tide range = approx. 2.5 ft (~0.75 m).
Inlet usage:	Small craft from southern Palm Beach and northern Broward counties.
Sediment characteristics:	Not known.
Drift rate:	Net southerly drift = 93,000 m^3/yr (~122,000 cy/yr). Transport is to the north for nine months and to the south for three months of the year during winter.
Beach erosion rate:	<u>1975 - 1979</u>: following extension of the training walls, the beach immediately south of the inlet receded by 187 ft (~57 m). <u>August 1985 - August 1995</u>: following the 1985 nourishment which widened the southern beach (3,400 ft or 1036 m length) on average 75 ft (~23 m), the same beach had receded approx. 138 ft (~42 m) by August 1995. (Coastal Planning & Engineering, 1996)
Type of bypass:	Weir training wall and channel trap with conventional dredging (mobile).
Bypass system components:	<u>1972</u>: 335 hp, 8 inch (~200 mm) hydraulic pipeline dredge and small tugboat. <u>1975</u>: northern training wall extended seawards 180 ft (~55 m). <u>1980</u>: construction of a 65 ft (~20 m) long weir section in the northern training wall at 180 ft (~55 m) in from the seaward end of the wall; added a second engine to the tug; modifications to the dredge and spoil pipelines to facilitate the dredging of the inshore portions of the ebb tidal shoal. <u>1985</u>: South Boca Raton Ebb Shoal Dredging/Feeder Beach Project placed 221,000 cy (~169,000 m^3) of sand from the ebb tidal shoal to a 3,400 ft (~1,036 m) length of beach south of the inlet. <u>1996</u>: A second replenishment project is planned. The Boca Raton Inlet Ebb Tidal Shoal Sand Transfer Project provides for the dredging of another 252,000 cy (~193,000 m^3) of sand from the ebb tidal shoal to be placed on a 3,960 ft (~1.2 km) length of beach south of the inlet.
Outlet type:	Pipe discharge from dredge directly on to southern beach via approx. 200 mm PVC pipe.
Bypass rate:	Average bypass rate = 32,000 m^3/yr (~41,850 cy/yr).
Degree of bypassing: (e.g. all, 50%, etc.)	34 % artificial bypassing; 47 % natural (Dombrowski and Mehta, 1990).
Costs:	<u>1972</u>: purchase cost = $140,000 (US) for dredge and tugboat (Coastal Planning & Engineering, 1996).

Funding:	1972: City of Boca Raton All inlet/beach maintenance projects and monitoring activities are funded jointly by the Florida Department of Environmental Protection (75 %) and the City of Boca Raton (25 %) (Coastal Planning & Engineering, 1996).
Contract type:	Operated by the City of Boca Raton.
Owner:	Prior 1972: private ownership. After 1972: City of Boca Raton.
Operator:	City of Boca Raton.
Supervisor of operations:	Experienced dredge master, employed by the City of Boca Raton.
Staffing:	3 people.
Operating cycle:	The dredge is not certified for ocean operations and so cannot proceed past the end of the walls. Works within the entrance proceed with, and are governed by, the sand, wave, and current conditions. Operates during winter and intermittently during summer.
Environmental constraints:	Not known.
Environmental management issues:	Narrow inlet with heavy usage by recreational vessels. Heavy beach usage.
Commencement date of bypassing:	Dredge and tug commenced in 1972.
Performance: (include any leakage to inlet, formation of entrance bar, etc.)	The plant only bypasses 34 % of the southerly drift with 47 % naturally bypassing around the ebb tidal shoal. A further 18 % is retained by the northern training wall, and 1 % is deposited on the flood shoal. Strong currents exist within the narrow inlet and a bar offshore from the entrance requires dredging by other equipment occasionally. The amount of artificial bypassing did not stop erosion of the southern beach, while the natural bypassing had made navigation of the ebb shoal hazardous. The beach nourishment project of 1985 using sand from the ebb shoal, provided on average 30 % (28,000 m^3/yr based on a 6 year return period for nourishment works) of the annual littoral drift to the southern beach, resulting in a total of 111 % (103,000 m^3/yr) of the net southerly drift being bypassed both artificially and naturally. (Dombrowski and Mehta, 1993)
Present plant status: (as of 1996)	Still in operation.
References:	Coastal Planning & Engineering, Inc., 1996. *The Boca Raton Inlet Ebb Tidal Shoal Sand Transfer Project and Ongoing Interior Sand Transfer Program*, Brochure, 25th International Conference on Coastal Engineering, Orlando, Florida, USA. Dombrowski, M.R., Mehta, A.J., 1993. Inlets and Management Practices: Southeast Coast of Florida, *Journal Coastal Research*, Special Issue No. 18, Fort Lauderdale, Florida, pp29-57.

***Figure C1**: Boca Raton Inlet Sand Bypassing System, Locality plan (Coastal Planning and Engineering, 1996)*

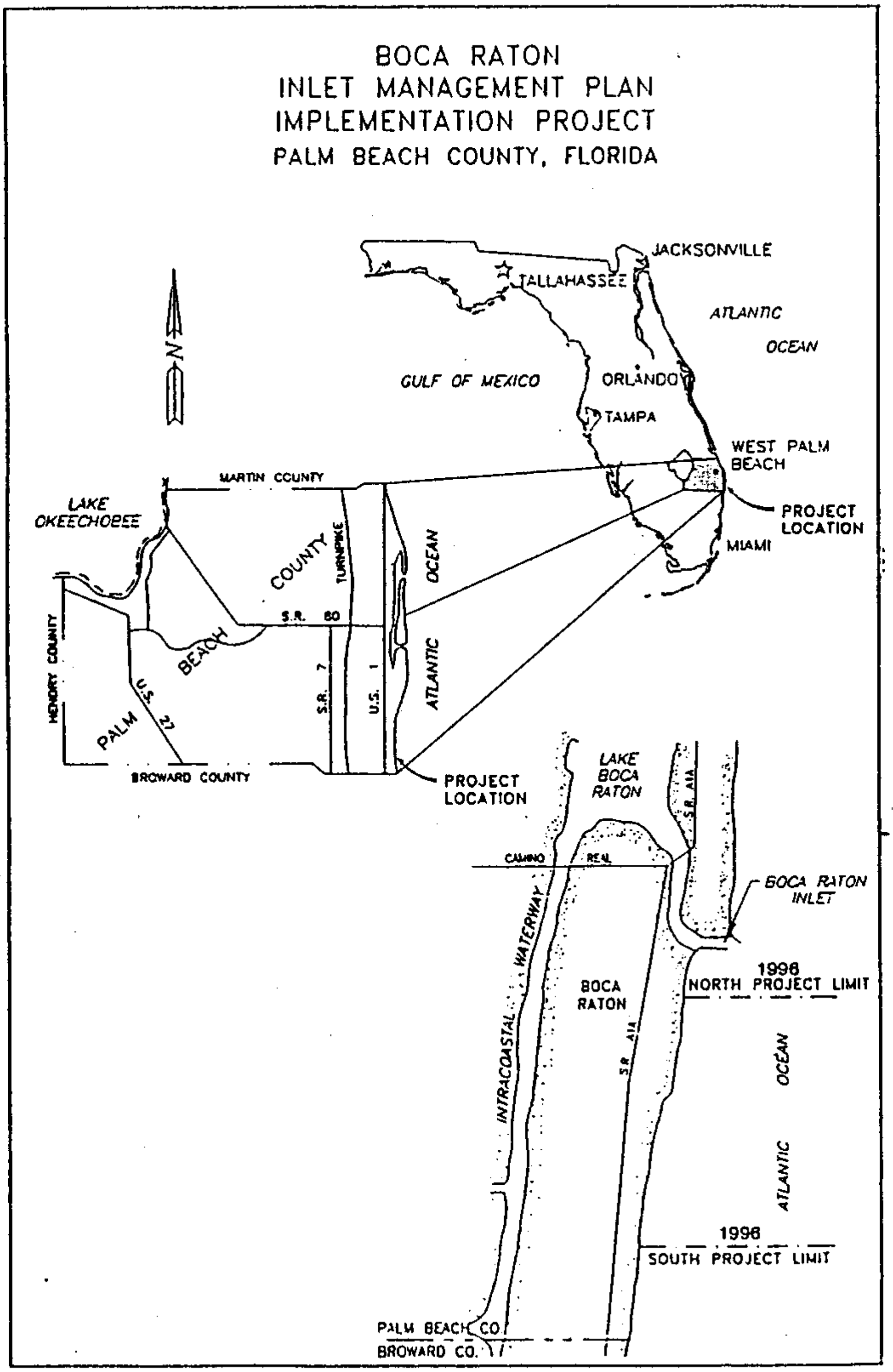
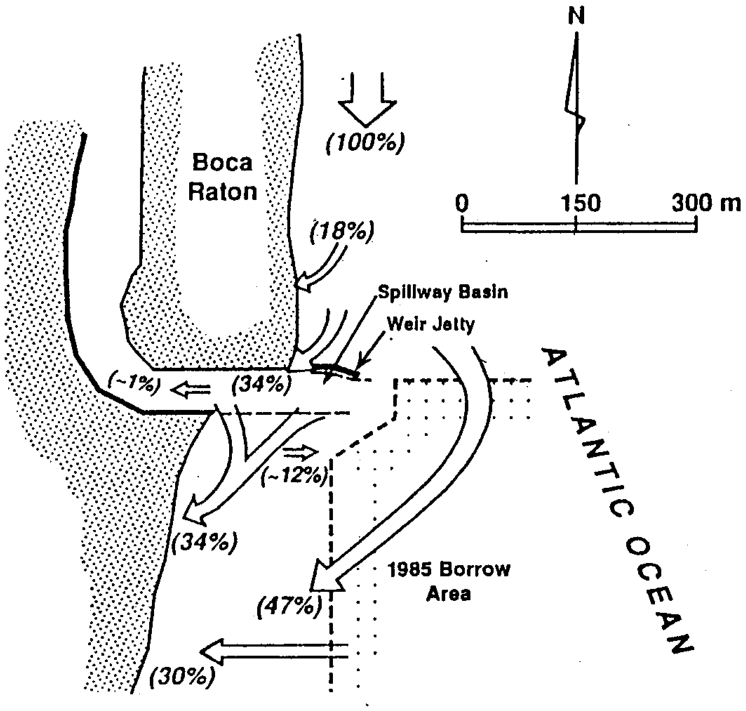


Figure C2*: Boca Raton Inlet Sand Bypassing System (Dombrowski and Mehta, 1993)*



Appendix D

Data sheet: Channel Islands Harbour Sand Bypassing System, California, U.S.A.

Location:	The artificial Channel Islands Harbour was constructed in 1960 and is situated 1.6 km to the north-west of Port Hueneme (pronounced "Why-nee-mee") in the City of Oxnard in Ventura County, California, USA. The harbour is approx. 60 miles (~96 km) Northwest of Los Angeles facing the Santa Barbara Channel. The area is the coastal edge of the Oxnard Plain, an abandoned flood plain of Santa Clara River which is bound by the Sulphur mountains to the south and the Santa Monica mountains to the north. The Ventura and Santa Clara Rivers are to the north.
Problem:	With the construction of the artificial Port Hueneme in 1938, the southerly drift was halted causing accretion behind the upcoast breakwater and severe erosion downcoast at Ormond Beach threatening Federal, industrial, and residential property. The sand which began to naturally bypass the harbour was lost from the littoral system to Hueneme submarine canyon. Channel Islands harbour was constructed to trap sand which was being diverted offshore into the Hueneme submarine canyon, and to supply sand by mechanical bypassing to Ormond Beach and other downdrift beaches.
Wave climate:	Both the sea and swell are predominantly from the west and north-west owing to restrictions caused by Point Conception and offshore islands. The breaking wave heights common to this shoreline range from 3 - 8 ft (~0.9 - 2.4 m). Some local short duration winter storms and limited amount of summer swell from the South Pacific, produce short periods of northward transport. Wave periods of 14 s or greater often occur in this region (Herron and Harris, 1966). The significant wave conditions used as a basis for design of the offshore breakwater using hindcast data from 1936 - 1938 were : dir = 280° (WNW); T = 6 - 13 s; Hs = 9.4 - 15.7 ft (~2.8 - 4.8 m) at the structure. dir = 215° (SW); T = 7 s; Hs = 10.3 ft (~3.1 m) at the structure. dir = 175° (S); T = 7 s; Hs = 8.1 ft (~2.5 m) at the structure. (Herron and Harris, 1966)
Inlet characteristics:	The man-made harbour has a width of approx. 500 ft (~150 m) and an entrance depth of 20 ft (~6 m) (MLLW).
Inlet usage:	Channel Islands: small-craft (serves up to 1,100 small craft). The harbour is an access point for the islands offshore (i.e. Anacapa, Santa Cruz, Santa Rosa, and San Miguel Islands). (Port Hueneme: deep water US Navy and commercial facility.)
Sediment characteristics:	The Oxnard Plain consists of alluvial deposits of sand, silt and clay.
Drift rate:	Net southerly drift = ~1,000,000 m^3/yr (Walker, 1991) or 1,200,000 cy/yr (Herron, and Harris, 1966) Sources: Santa Clara River = 800, 000 cy/yr (~612,000 m^3/yr); Ventura River = 100,000 cy/yr (~76,500 m^3/yr); littoral drift = 270,000 cy/yr (~206,000 m^3/yr) (Herron, and Harris, 1966)

Beach erosion rate: Between 1940 (completion of Port Hueneme) and 1961 (establishment of permanent bypass system) approx. 1,000 ft (~765 m) beach recession occurred in the vicinity of the City of Port Hueneme (south of the Port), tapering to no shoreline retreat approx. 7, 000 ft (~2.1 km) downcoast. During this period almost 4,000,000 cy (~3,058,000 m^3) of sand was placed on this stretch of beach between 1940 and 1954. Approximately 500 acres of industrial, residential and agricultural land was lost of a total volume of 21,000,000 cy (~16,100,000 m^3).
(Herron, and Harris, 1966)

Type of bypass: Updrift offshore breakwater sheltered trap with conventional hydraulic pipeline dredging using floating plant moored in and near the entrance, behind the breakwater.

Bypass system components: 1953 -1954: dredged 4,000,000 cy (~3,058,000 m^3) from the fillet upcoast of Port Hueneme Harbour northern breakwater, and pumped under the harbour to southern beach. Project cancelled after only 2,000,000 cy (~1,500,000 m^3) was bypassed owing to difficulties in dredging in the surf.
Dec 1958 - Oct 1960: construction of Channel Islands Harbour entrance training walls (finished Sep 1959), and the offshore breakwater (finished Oct 1960). The offshore breakwater is situated in 30 ft (~9 m) depth (MLLW) and is 2,300 ft (~700 m) long with the southern end in line with the southern training wall.
Feb 1960 - Jun 1961: initial dredging of Channel Islands Harbour (3,708,500 cy or ~2,835,400 m^3) and sand trap (2,627,000 cy or ~2,000,000 m^3) was bypassed to Ormond Beach by pipeline beneath both Channel Islands and Port Hueneme Harbours.
Jun 1963 - Sep 1963: first biennial dredging of the trap, bypassing 1,986,000 cy (~1,520,000 m^3).
Apr 1965 - Sep 1965: biennial dredging and bypassing of 3,527,000 cy (~2,697,000 m^3). The larger quantity was dredged to increase the capacity of the trap owing to overfilling and leakage into the entrance since the first dredging project.
Apr 1967 - Sep 1967: biennial dredging and bypassing of approx. 3,000,000 cy (~2,300,000 m^3). Again, the large quantity was to increase the trap capacity.

It was intended that future biennial bypassing would be reduced to between 2.0 and 2.5 million cy (~1,500,000 - 1,900,000 m^3).
(Herron, and Harris, 1966)

Walker (1991) reports that the annual bypassing rate has been about 1,000,000 m^3 (~1,300,000 cy) with the majority of the sand going to Ormond Beach and a minor amount going to the beach between the two harbours and backpassed to the updrift beach.

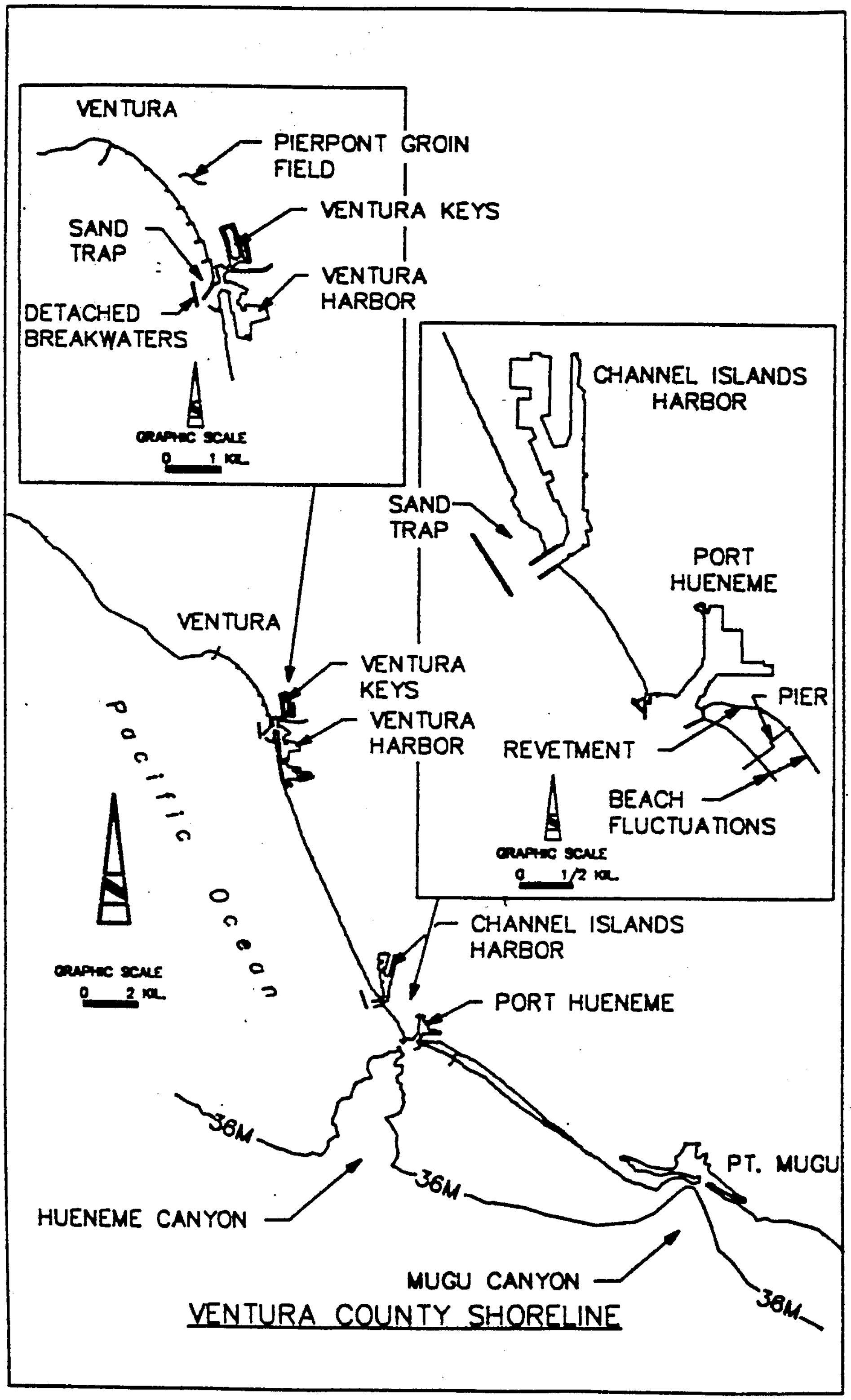
Outlet type: Pipeline underneath both the Channel Islands and Port Hueneme Harbours to discharge on Ormond Beach.

Bypass rate: Average bypass rate = 1,000,000 m^3/yr (~1,300,000 cy). Approximately 14,500,000 m^3 (~19,000,000 cy) was bypassed over the first 14 years of operation (Walker, 1991).

Degree of bypassing: (e.g. all, 50%, etc.) The majority of the sand reaching the Channel Islands Harbour has been bypassed. Walker (1991) suggests that a annual loss of 600,000 m^3 to the Mugu Canyon is occurring.

Costs:	<u>1953 -1954</u>: $1,837,865 (US) for total 4,000,000 cy. <u>Dec 1958 - Oct 1960</u>: $669,000 (US) for training wall construction; $3,351,000 (US) for offshore breakwater construction <u>Feb 1960 - Jun 1961</u>:$1,250,000 (US) for bypassing from sand trap. <u>Jun 1963 - Sep 1963</u>: $951,000 (US) for bypassing. <u>Apr 1965 - Sep 1965</u>: $1,092,000 (US) for bypassing. <u>Apr 1967 - Sep 1967</u>: $500,000 (US) for bypassing. The estimated average annual cost of sand bypassing only, including depreciation and maintenance, was $0.38 (US) /cy. (Herron, and Harris, 1966)
Funding:	US Army Corps of Engineers.
Contract type:	A contract is let for each biennial project.
Owner:	US Army Corps of Engineers.
Operator:	Contract dredger.
Supervisor of operations:	US Army Corps of Engineers.
Staffing:	Dredge crew.
Operating cycle:	Biennial during summer months.
Environmental constraints:	Not known.
Environmental management issues:	Entrance is heavily used for navigation. Beaches are heavily used and backed by beachfront houses and apartments.
Commencement date of bypassing:	February, 1960 with the initial dredging of the harbour and sand trap.
Performance: (include any leakage to inlet, formation of entrance bar, etc.)	The bypass system has performed well with all sand reaching the trap being bypassed (Herron and Harris, 1966).
Present plant status: (as of 1996)	Still in operation.
References:	Herron, W.J., Harris, R.L., 1966. Littoral Bypassing and Beach Restoration in the Vicinity of Port Hueneme California, *Proceedings of Tenth Conference on Coastal Engineering*, ASCE, pp651-675. Walker, J.R., 1991. Downdrift Effects of Navigation Structures on the California Coast, *Coastal Zone '91: Proceedings of the 7th Symposium on Coastal and Ocean Management*, Vol. 1, ASCE, Long Beach, California, USA, July 8-12, pp1889-1903.

***Figure D1**: Channel Islands Harbour Sand Bypassing System (Walker, 1991).*



Appendix E

Data sheet: Dawesville and Mandurah Inlets Sand Bypassing System, Western Australia.

Location:	The Dawesville and Mandurah inlets connect the Peel-Harvey inlet system to the Indian Ocean. Mandurah is approx. 65 km south of Perth, Western Australia. Dawesville is approximately 15 km south west of Mandurah.
Problem:	Severe algae pollution was caused by poor circulation and increased phosphate levels from agricultural land run-off exacerbated by the low ocean tide range and shoaling single entrance at Mandurah. The construction of the new Dawesville inlet was implemented to increase the flushing and salinity of the Peel-Harvey Inlet system.
Wave climate:	Predominantly south-westerly swell.
Inlet characteristics:	a. Dawesville: Inlet width = 200 m; depth = 4.5 - 6.5 m below mean sea level at seaward end; water exchange / tidal cycle = 16.5 x 10^6 m^3 (summer) and 17.1 x 10^6 m^3 (winter); diurnal tides. b. Mandurah: Inlet width = 90 m; depth limited by rock sill to 3 m below CD. Design navigation channel is 30 m wide by 2.5 m deep.
Inlet usage:	a. Dawesville: fishing industry and recreational boating. b. Mandurah: fishing industry and recreational boating.
Sediment characteristics:	Clean marine sand.
Drift rate:	a. Dawesville: net northerly rate = 85,000 m^3/yr. b. Mandurah: The littoral drift is understood to vary between 100,000 and 200,000 m^3/yr from west to east without significant reversals in direction. Most of the drift occurs in quantities of 10,000 to 30,000 m^3 during the winter storm events.
Beach erosion rate:	a. Dawesville: In 1992, 107,000 m^3 of sand excavated from the channel was placed north of the channel. Between 1992 and 1993 there was a net loss of 90,000 m^3. Since 1993 the volume of sand north of the channel has fluctuated between 100,000 m^3 and 150,000 m^3 less than in 1992. b. Mandurah:
Type of bypass:	Mobile land based system consisting of a crawler excavator feeding a crawler mounted screen and pump system called the "Slurrytrak" (system operates both Dawesville and Mandurah). .
Bypass system components:	1. Cat 245 Excavator with 3m^3 bucket digging on beach and feeding "Slurrytrak" inlet hopper. 2. "Slurrytrak" consists of inlet hopper with sieves, gravity feeding to a reciprocating tray feeder on to a inclined cleated conveyor with belt weighometer. Conveyor feeds to a linear motion scalping screen on top of agitation hopper which is fed with water (middle and lower). Centrifugal slurry pump fed from bottom of hopper pumps a slurry with approx. 45% sand content by weight through discharge pipe (MDPE and some flexible sections). System is self propelled with diesel motor. 3. Clear water supplied by separate pump via a 315 mm OD Class 12 MDPE pipe from inlet. At the Mandurah Inlet, a 75 m groyne was constructed in 1986 - 87 approx. 300 - 350 m west of the western entrance training wall to allow for the dredging of a large trap between the groyne and breakwater without affecting the public beach to the west of the groyne.

At the Dawesville Inlet, a spur groyne was constructed projecting updrift (approx. south) off the southern training wall to create a sand trap behind it.

Outlet type:	A 315 mm OD Class 12 MDPE discharge pipe to downdrift beaches for both inlets. a. Dawesville: channel crossing by 2 fixed pipes trenched in bottom; 0.5km to discharge. b. Mandurah: channel crossing by HDPE line weighted; 1km to discharge.
Design bypass rate:	a. Dawesville: up to 85,000 m^3 pa. b. Mandurah: up to 110,000 m^3 pa.
Degree of bypassing: (e.g. all, 50%, etc)	Desired to be 100%. At Mandurah a bar still exists seawards of the entrance and there is some channel infill during winter storm events. At Dawesville, the trap is not capturing 100% of the sand with accumulation offshore of the trap in depths of -5 m to -8 m CD (approx. 150,000 m^3).
Costs:	In general, bypass operation costs about \$3/m^3 and monitoring and management costs approx. \$1/m^3.

a. Dawesville:

Bypassing costs (July to June):

Year	Volume (m^3)	Cost ($)
1995/96	22,000	68,000
1996/97	39,000	103,000
1997/98	85,000	280,000
TOTAL	146,000	451,000
AVERAGE	49,000	150,000

b. Mandurah:

Bypassing costs (July to June):

Year	Volume (m^3)	Cost ($)
1995/96	55,000	179,000
1996/97	156,000	426,000
1997/98	86,000	262,000
TOTAL	296,000	868,000
AVERAGE	99,000	289,000

Funding:	West Australian State Government Department of Transport.
Contract type:	5 year design, construct and operate. Paid per cubic metre (weighed); plus payment per re-establishment; plus guarantee of minimum quantity for each establishment (15,000 m^3 from Dawesville; 20,000 m^3 from Mandurah).
Owner:	Contractor.
Operator:	Local contractor for 5 years.
Supervisor of operations:	Department of Transport.
Staffing:	2 full-time.
Operating cycle:	Up to approx. 48 weeks/year (including maintenance periods) with plant alternating between Dawesville and Mandurah. System is envisaged to operate at each location 2 to 3 times per annum with re-establishments directed by supervisor. System has actually operated 1 to 2 times per year at each site. Minimum quantity for each session as to be 15,000 m^3 (Dawesville) and 20,000 m^3 (Mandurah). Periods of higher sediment inflow at each site are generally not synchronous.

Environmental constraints:	Rock lobster fishing requirement demands a navigation depth of 2.5m LWD from 1 November; main sand infill occurs in winter.
Environmental management issues:	Not known.
Commencement date of bypassing:	December 1995.
Performance: (include any leakage to inlet, formation of entrance bar, etc)	a. Dawesville: trap is not collecting design quantity and is not filling to expected volume; it is believed that there is leakage. Channel has remained relatively stable. Between 1994 and 1996 accretion occurred offshore from the sand trap in depths of -5 m to -8 m CD (approx. 150,000 m^3), reducing sand accumulation in the trap. Offshore bathymetry has since stabilised. b. Mandurah: bar decreasing in volume. The target depth of 2.5 m CD has not been achieved continuously, but access has been provided to most vessels most of the time. Problems stem from insufficient trap capacity during winter storm events. Sand trap has been extended.
Present plant status: (as of 1999)	Still in operation.

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***Figure E1**: Dawesville and Mandurah Inlets Sand Bypassing System, Locality plan (Moloney et al, 1999).*

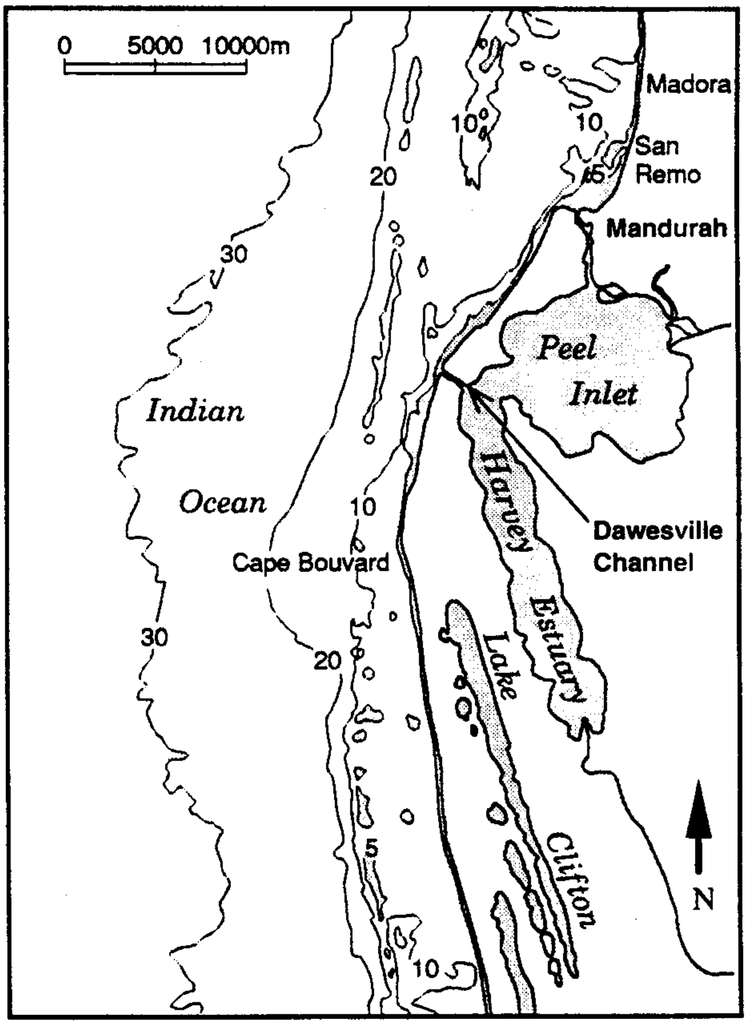


Figure E2: *Layout of Dawesville Sand Bypassing System (Moloney et al, 1999).*

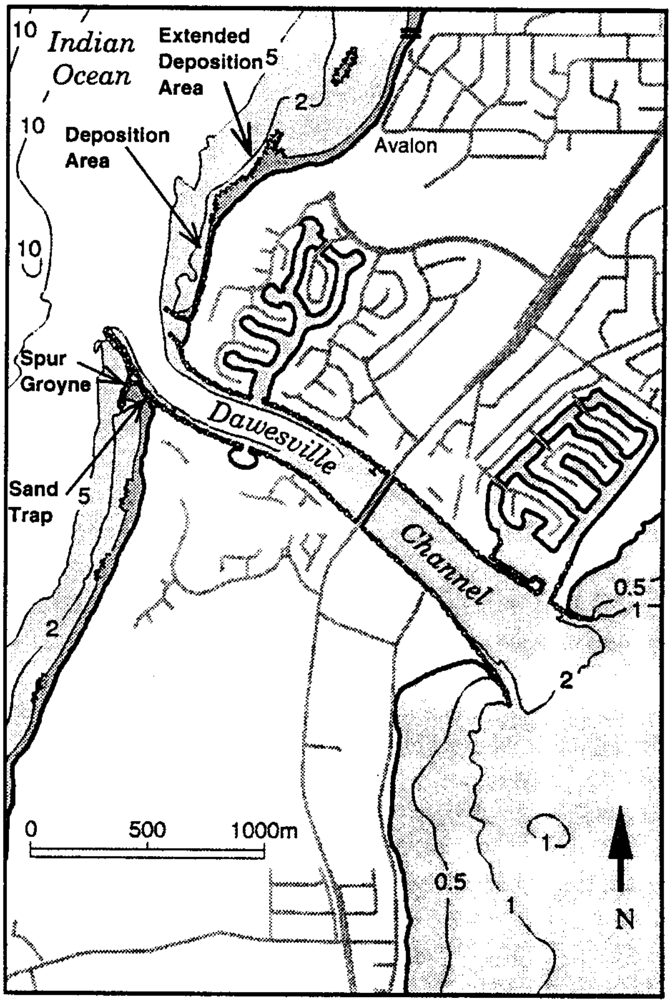


Figure E3: *Layout of Mandurah Sand Bypassing System (Moloney et al, 1999).*

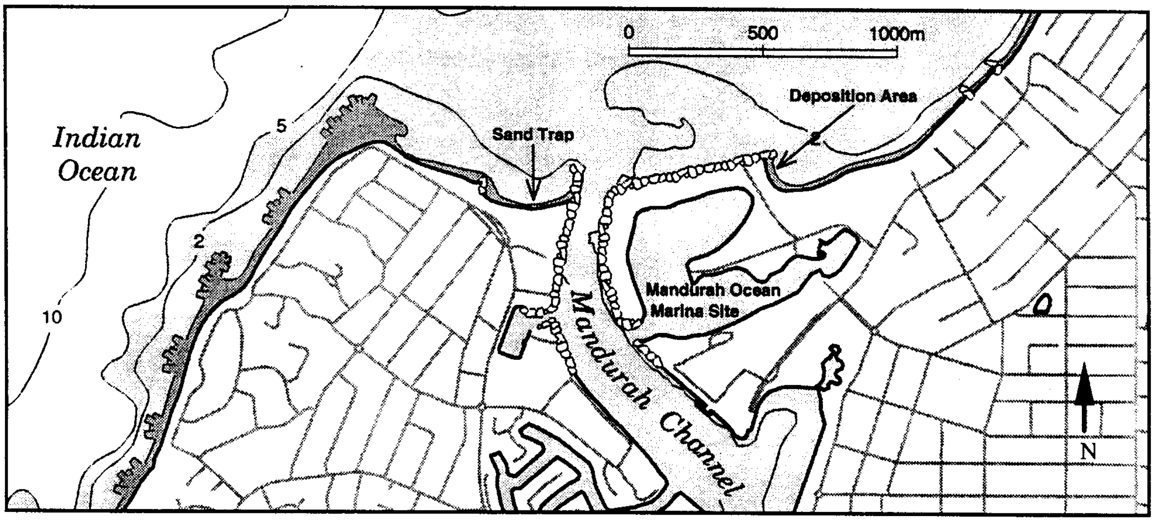
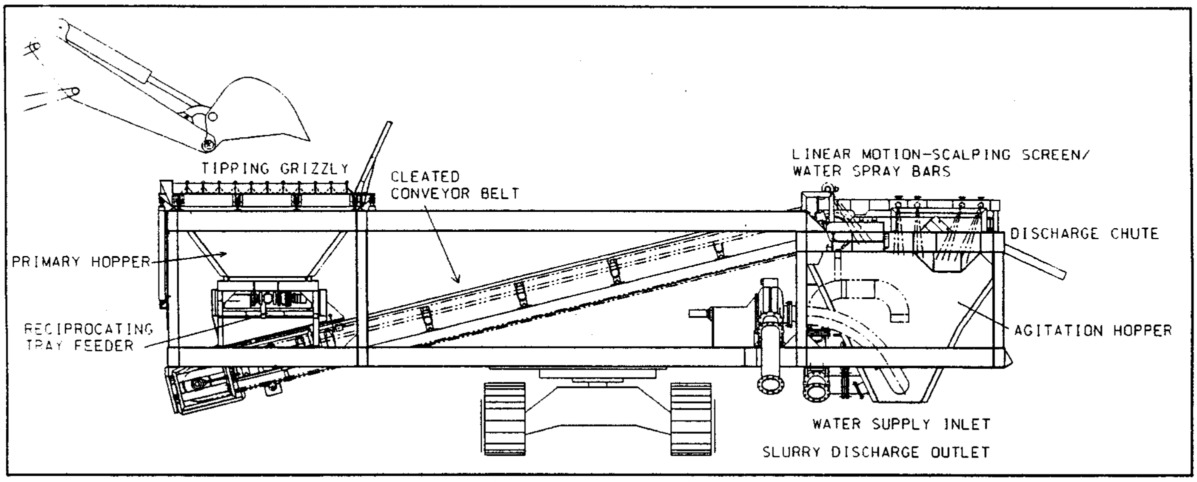


***Figure E4**: General arrangement of the Slurrytrak 300-65 HH used for sand bypassing at Dawesville and Mandurah Inlets (Moloney et al, 1999).*



Appendix F

Data sheet: Indian River Inlet Sand Bypassing System, Delaware, U.S.A.

Location:	Indian River Inlet, Delaware situated on the Atlantic coast approx. 10 miles (~16km) north of Ocean City, Maryland, USA, connects Indian River Bay and Rehoboth Bay to the Atlantic Ocean.
Problem:	Construction and training of the 500 ft (~150 m) wide inlet in 1938-1940 to stabilise the existing channel (which was prone to migrating within a 2 mile (~3.5 km) region, as well as closing occasionally) has resulted in the gradual erosion of the beach adjacent the northern training wall, threatening the Route 1 state highway which runs parallel to the coast line.
Wave climate:	Not known. Calculation of the annual longshore sediment transport rate was based on the use of Phase III WIS (Wave Information Study nearshore hindcast wave data) statistics utilising data from WIS Atlantic Coast Station 65 (Gebert et al, 1992).
Inlet characteristics:	Wall centre line to wall centre line spacing = 500 ft (~150 m); semidiurnal tide; mean tide range = ~4 ft (~1.2 m); spring tide range = ~5 ft (~1.5 m); design channel depth = 15 ft (~4.5 m) MLW; channel dredged to 14 ft (~4.2 m) MLW in 1938 (Anders et al, 1990); existing channel depth = typ. 40 - 90 ft (12 - 27 m) MLW . Channel currents in excess of 9 ft/s (~2.7 m/s) (Anders et al, 1990). *Indian River Bay and Rehoboth Bay*: mean tide ranges = 2.1 ft (~0.64 m) and 1.0 ft (~0.3 m) respectively; combined surface area = 29 square miles (~75 km^2); total tributary area = 250 square miles (~647 km^2). (Gebert et al, 1992)
Inlet usage:	Small commercial and recreational vessels (Gebert et al, 1992, p506).
Sediment characteristics:	Medium sand (Gebert et al, 1992). Typical grain size of the order of 0.4 mm (Anders et al, 1990).
Drift rate:	Net northerly drift of 110,000 cy/yr (~84,000 m^3/yr) based on WIS data, analysis of historic beach profile and hydrographic survey data, and beach erosion data (Clausner et al, 1992). From WIS study: 160,000 cy/yr (~122,000 m^3/yr); std dev. = 90,000 cy/yr (69,000 m^3/yr) (Gebert et al, 1992).
Beach erosion rate:	In the region 200 ft (~60 m) to 1800 ft (~550 m) north of the training wall the shore position has receded 150 - 194 ft (~45 - 59 m) from November 1984 to October 1989 (Gebert et al, 1992, table 1).
Type of bypass:	Single jet pump mounted 135 ton capacity rated crawler crane with 120 ft (~37 m) boom (mobile system) operating from southern beach.
Bypass system components:	Clear water 12 inch (~305 mm) HDPE SDR-9 (9.9 inch or ~250 mm ID) supply line from inlet (approx. 20 m from pump house); water supply pump (8 cyl. motor, 400 hp) in pump house on southern side; Genflo eductor with 2.5 inch (63 mm) nozzle and 6 inch (150 mm) mixing chamber with rate of 200 cy/hr (~153 m^3/hr) positioned in swash zone using Crawler crane; 12 inch (305 mm) HDPE SDR-13.5 (10.8 inch or ~274 mm ID) discharge line; discharge booster pump (12 cyl. motor, 600 hp but running typ. at 400 hp) in pump house; HDPE pipe across Route 1 bridge extending up to a maximum distance of 1,500 ft (457 m) north of the inlet. The jet pump creates an 18 ft (~5.5 m) deep and 48 ft (~14.6 m) diameter crater. The crane can create a trench of three crater diameters length

before requiring repositioning. Collection occurs over a stretch of the southern beach from 100 - 400 ft (30 - 120 m) south of the inlet.

Outlet type: 12 inch (305 mm) HDPE SDR-13.5 (10.8 inch or ~274 mm ID) pipe discharging directly onto the beach within 1,500 ft (457 m) north of the inlet.

Bypass rate: Design rate = 200 cy/hr (~153 m^3/hr); 100,000 - 110,000 cy/yr (76,000 - 84,000 m^3/yr). Following experience and system operating enhancements, approx. 330 cy/hr (~250 m^3/hr) can be achieved. The suggested maximum capacity is 552 cy/hr (~422 m^3/hr). Pumping concentration of approx. 40% by weight.

Suitable for sites where maximum bypass rate < 150,000 m^3/yr (Watson et al, 1993).

Degree of bypassing: (e.g. all, 50%, etc.) Proposed to bypass all the northwards transport. However, the system is limited by the quantity of sand reaching the collection area. Strong flow conditions maintains (and are in fact scouring) the inlet depth.

Costs: Final cost of plant construction: $1.7 million (US)
Estimated operating and maintenance: $290,000 (US) (includes annualised replacement costs). The actual operating costs for 1990 to 1996 are given in *Performance* below.

Funding: Shared between the State of Delaware and the Federal Government of USA. Federal Government contributes 40.755%.

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Contract type: State performs work for Federal Government.

Owner: State of Delaware.

Operator: State of Delaware, which has a state dredging program.

Supervisor of operations: State of Delaware; oversight by US Army Corps of Engineers.

Staffing: Total of 3 people: a primary operator, operator's assistant, and crane operator. The staff are supervised by an experienced dredge master (off site) who covers several projects.

Operating cycle: 5 day (7.5 hr day) week (37.5 hr week) with a 2 day weekend shutoff, operating 9 months per year.
1 hr (min) to 7 hr (max.) operation per day. The system operates only 40 % of available days owing to limitations of the amount of littoral material transported and trapped within reach of the system (Watson et al, 1993).

Environmental constraints: Social: the beach north and south of the inlet is a state park and used by tourists during the summer season. Bypassing is not allowed in summer between Memorial Day (late May) and Labour Day (early September). However, State park service have allowed bypassing during summer months within 100 - 200 ft (30 - 60 m) south of the training wall provided that the area is fenced off and marked with warning signs and buoys.

Cold weather conditions and location mean that week day beach usage during the operational window in winter is low; but anglers use the training wall. Surfers also surf adjacent to both breakwaters during the operating season.

Environmental management issues: The northern beach is a nesting spot for the piping plover, an endangered species of bird, during March through August. Guidelines follow that if a nest is sighted, the discharge operation will stay several hundred feet away, and walkovers will be built to allow young birds to cross the discharge pipe (Rambo et al, 1991).

Commencement date of bypassing: January, 1990.

Performance: (include any leakage to inlet, formation of entrance bar, etc.)

Summary of sand bypassing statistics (Watson et al, 1993)

	1990	1991	1992	Total (90-92)
m^3 bypassed	86,000	63,000	51,700	200,700
[cy]	[112,700]	[82,335]	[67,670]	[262,700]
No. Days Bypassing	71	55	60	186
No. Mths Bypassing	11	9	9	29
Avg Production (m^3/day)	1,225	1,150	850	1075
[cy/day]	[1,600]	[1,500]	[1,100]	[1,400]
Avg Days/Month Bypassing	6.45	6.11	6.7	6.41

Short term rate remains about 200 m^3/hr. The higher bypassed amount for 1990 was a result of the initial large volume of trapped sand, and bypassing during summer. As stated by Watson et al (1993), "apparently the system is only able to capture about 60 to 80% of the estimated net northerly drift, though the variable nature of littoral transport in this area makes this conclusion very preliminary".

The rates and operating costs from Feb 1990 to May 1996 for each calendar year (Jan - Dec) as detailed in the additional data for operating expenses were:

Year	cy Pumped	m^3 Pumped	cost/cy	cost/m^3
1990	112,700	86,000	$1.00	$1.30
1991	82,330	63,000	$1.70	$2.20
1992	67,670	51,700	$1.85	$2.40
1993	67,800	51,800	$2.50	$3.25
1994	84,570	64,660	$1.65	$2.15
1995	68,750	52,560	$2.30	$3.00
1996 (partial)	31,550	24,100	$3.00	$3.90

Present plant status: (as of 1996) Still in operation.

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Other data:

Indian River Inlet Sand Bypass Plant operating expenses from February, 1990 to May, 1996.

Sand Bypass Plant capital replacement schedule for 1996.

Sand Bypass Plant standard operating procedures.

***Figure F1**: Indian River Inlet Sand Bypassing System (Rambo et al, 1991).*

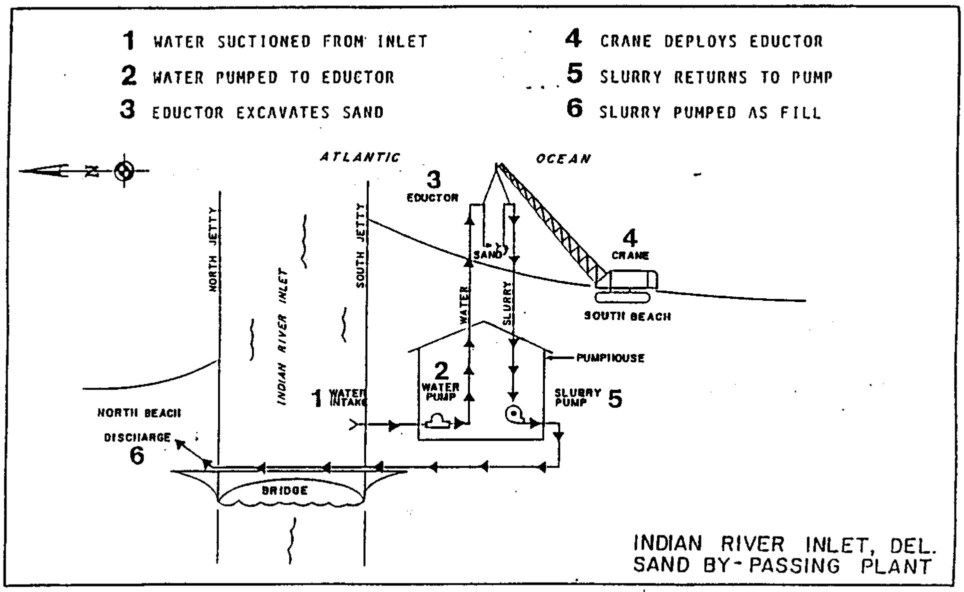
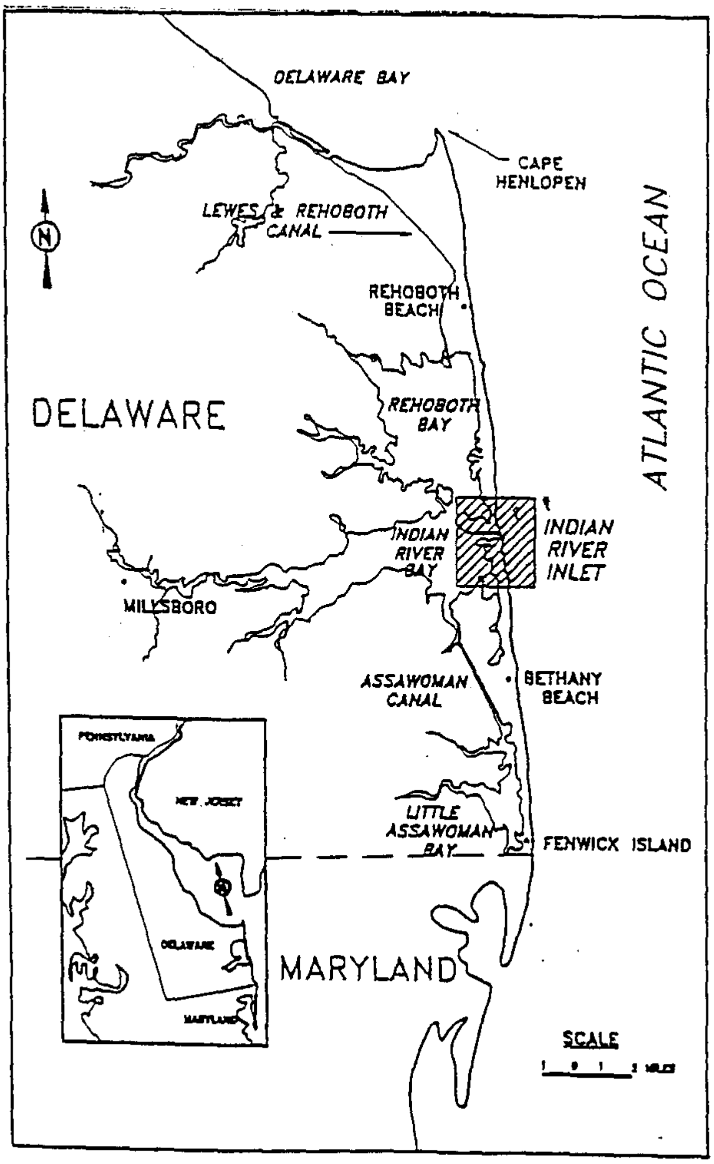


***Figure F2**: Indian River Inlet Sand Bypassing System, Locality plan (Rambo et al, 1991).*



Appendix G

Data sheet: Oceanside Harbour Sand Bypassing System, California, U.S.A.

Location:	Oceanside Harbour is situated on the west coast of California, USA, approx. 80 miles (~130 km) south-east of Los Angeles and 30 miles (~48 km) north of San Diego. The harbour is bordered by Santa Margarita River 6,600 ft (~2 km) to the north, and San Luis Rey River 2,400 ft (~730 m) to the south. The City of Oceanside is located to the south of the harbour, and the US Naval Base of Camp Pendelton is located immediately north of the harbour. The harbour services both the U.S. Navy Del Mar boat basin, constructed in 1942, and the City of Oceanside Small-Craft Harbour, constructed in 1963 (with sand dredged from the harbour used to nourish Oceanside Beach).
Problem:	The construction of the harbour complex has interrupted the littoral transport which has resulted in accretion along the northern breakwater, shoals developing in the entrance, and erosion to the beaches to the south (specifically Oceanside beach). The region is also affected by a large gross transport resulting in shoals entering the harbour from both the north and south.
Wave climate:	Camp Pendleton surf and weather station (depth = 32 ft or 9.75 m MLLW): highest measured Hs = 10.8 ft (~3.3 m) with T = 17.8 s Hs(50%) = 3.5 ft (~1.1 m); Hs(10%) = 5 ft (~1.5 m) based on 7 years of data. California coastal data collection program, near Oceanside Pier (depth = 32 ft or 9.75 m MLLW): highest measured Hs = 8.3 ft (~2.5 m) with T = 14 to 16 s Hs(50%) = ~2.0 ft (~0.6 m); Hs(10%) = ~4.0 ft (~1.2 m) based on 3 years of data. Typically, the Oceanside wave climate consists of: Northern hemisphere swell: Hs,o < 10 ft (~3 m); T = 12 - 18 s; Dir = 260° to 270° (November to April). Southern hemisphere swell: Hs,o < 4 ft (~1.2 m); T = 18- 21 s; Dir = S to SE (May to October). Local sea: Hs = 2 - 5 ft (~0.6 - 1.5 m); T_{ave} = 7 s; Dir = predominantly NW (all year). Eastern North Pacific tropical cyclones approaching from the south to south-west (May to November) seldom produce large waves that reach the site. Largest waves at Oceanside occurred in 1939 producing a significant breaking wave height = 24 ft (~7.3 m) (> 100 - 200 yr recurrence interval). (Moffatt & Nichol, Engineers, 1983)
Inlet characteristics:	Tide range: 5.6 ft (~1.7 m) from MHHW to MLLW, or 3.78 ft (~1.15 m) from MHW to MLW (Moffatt & Nichol, Engineers, 1983).
Inlet usage:	U.S. Navy and public small-craft.
Sediment characteristics:	North fillet: D_{50} = 0.21 mm Entrance channel: D_{50} = 0.18 mm
Drift rate:	Net southerly drift = 100,000 - 250,000 cy/yr (~75,000 - 190,000 m^3/yr) Gross transport rate = 1,200,000 cy/yr (~917,000 m^3/yr) (Weisman, 1996) Based on predicted longshore transport rates by three different studies, Dolan et al (1987) presented the following averages: Gross northerly transport = 546,000 cy/yr (~417,000 m^3/yr) Gross southerly transport = 740,000 cy/yr (~565,000 m^3/yr) Net southerly transport = 194,000 cy/yr (~150,000 m^3/yr)

Beach erosion rate:	Camp Pendleton to the north of the harbour continues to accrete, while Oceanside to the south is eroding.
Type of bypass:	Experimental system of jet pumps and fluidisers to be constructed in phases (fixed). Main system location in harbour entrance; secondary capture location at northern breakwater.
Bypass system components:	Phase I: single jet pump (Pekor 6x6x8 inch or 150x150x200 mm, capacity of 330 cy/hr (~250 m^3/hr)) and crane at north breakwater for bypassing sand from the north fillet; two jet pumps (Pekor 4x4x6 inch or 100x100x150 mm, capacity of 230 cy/hr (~175 m^3/hr)) in the entrance adjacent the south breakwater with deployment fluidisers attached to jet pump support beams; mobile hoist barge with pumps (supply pump of 750 hp and main booster pump of 1,050 hp) and controls moving between the north and south jetty riser structures; undersea pipelines to riser structures; cross harbour pipeline; shore booster station (pump of 1,050 hp) used during bypassing of north fillet; discharge line. The hoist barge was a contractor modification due to earthquake/stability concerns regarding jack-up (as designed). Phase II: Addition of 150 ft (~45 m) fluidiser oriented shoreward and parallel to the south breakwater at entrance to feed shoreward entrance jet pump, and 200 ft (~60 m) fluidiser oriented seaward and parallel to the south breakwater at entrance to feed seaward entrance jet pump. The fluidisers are supported on 25 - 30 ft (~7.6 - 9.1 m) spaced steel 12 inch (~305 mm) dia. piles driven in 20 - 22 ft (~6.1 - 6.7 m). The fluidisers are SDR 11 HDPE pipes with 1/8 inch (~3 mm) holes every 2 inches (~50 mm) aligned horizontally, with flanged connections at 50 ft (~15 m) lengths. A valve was introduced into the system to supply firstly to the fluidisers, and then the jets (the supply pump could not support the operation of both the fluidisers and jets at the same time). To improve jet recovery problems, the jets were attached to a 63 ft long (~19 m) strongback (I section) pivoted at a support pile. A fluidiser was attached to this to ease deployment/recovery problems. Phase II contract included operation and maintenance. Phase III (cancelled): Addition of two 200 ft (~60 m) fluidisers to feed sand from the tip of the southern breakwater to both entrance jet pumps; lengthen existing shoreward fluidiser another 145 ft (~44 m); increase entrance jet pumps to 6x6x8 inch (150x150x200 mm); add separate pump to power fluidisers. (Weisman et al, 1996, and Clausner et al, 1990).
Outlet type:	14 inch (~355 mm) HDPE discharge pipe extending 11,000 ft (~3.3 km) to the south along the beach with 3 discharge points along the length. .
Bypass rate:	Ultimately, the system was expected to bypass 250,000 cy/yr (~190,000 m^3/yr) at the entrance and 150,000 cy/yr (~115,000 m^3/yr) from the north fillet (Clausner et al, 1990). Design rate = 200 cy/hr (~153 m^3/hr) (Weisman et al, 1996)
Degree of bypassing: (e.g. all, 50%, etc.)	Only in experimental stages, full bypassing not achieved. It was not designed to achieve full bypassing.
Costs:	Estimated first construction cost of $5,000,000 (US) with a planned project life of 5 years. Actual costs = $15,000,000 (US) approx.
Funding:	Phase I: Federal Government of USA. Phase II: Federal Government of USA. Phase III: Federal with contributions from State and Local Governments.

Contract type:	Phase I: designed by consultant for the owner; fixed price construction contract. Phase II: contractor C & W Diver Services Inc. under contract with payments for maintenance of owners equipment and hire rate for pumping.
Owner:	U.S. Army Corps of Engineers (capital equipment, excluding barge owned by contractor).
Operator:	Contracted out. Phase II contractor C & W Diver Services Inc. under contract with payments for maintenance of owners equipment and hire rate for pumping.
Supervisor of operations:	US Army Corps of Engineers (LA District).
Staffing:	Total of 4 people: main operator to control the SCADA system (Supervisory Control and Data Acquisition); a mechanic overseeing component operations and manual operation of pumps in case of SCADA failure; a shore booster pump operator; and observer at the discharge point (Clausner et al, 1990).
Operating cycle:	Design Plan: 5 days a week, for up to 10 hours per day. Summer months (April - September): bypass from entrance jet pumps Winter months (October - March): bypass from northern fillet. (Clausner et al, 1990) Actual: bypassing only carried out for one year, with approx. 2 weeks only from northern fillet.
Environmental constraints:	No mining allowed of the north fillet on Camp Pendelton U.S. Marine Corps Base Property (rejected by the local base commander) and no mining of the fillet between the south breakwater and San Luis Rey River Groin (rejected by the City of Oceanside) (Weisman, 1996). North breakwater bypass system was placed on the breakwater beyond the intertidal zone without permanent structures as required by Marine Corps restrictions (Walker et al, 1987). Concerns regarding the nesting of the Lesser Tern restricted the operational window to the winter months.
Environmental management issues:	Required to carefully monitor the effects of the system on fauna, fish, plankton, grunion, and other marine species (Walker et al, 1987). Beach outlet required supervision during operation due to 'quick' sand and public usage. Outlet pipes were required to traverse rock walls seaward of beachfront condominiums and lifeguard station at pier, exposing them to wave action.
Commencement date of bypassing:	Phase I: June, 1989 (to August, 1990) Phase II: November, 1991 Phase III: Cancelled (insufficient funds)
Performance: (include any leakage to inlet, formation of entrance bar, etc.)	Phase I (June 1989 to August 1990 excluding January 1990 to April 1990): Total bypassed = 18,300 cy (~13,990 m^3); overall average = 63 cy/hr (~48 m^3/hr); total operational hours = 744; pumping sand hours = 305; minimum monthly pumping hours = 2; maximum monthly pumping hours = 55. Phase II (December 1991 to December 1992 inclusive): Total bypassed = 106,000 cy (~81,000 m^3); overall average = 95 cy/hr (~73 m^3/hr) (58% increase from Phase I); pumping sand hours = 1,128; total system downtime and maintenance hours = 607; minimum monthly pumping hours = 35; maximum monthly pumping hours = 126.

The major problems were associated with clogging and plugging of the fluidisers with sand, and the covering of the craters with kelp which reduced the amount of sand being pumped by the jets. The key problem with this project was that the shoals were forming from transport from both the south and the north, covering a large area to bypass.
(Weisman et al, 1996)

Other significant problems:
(a) difficult conditions for maintenance divers due to long period swell producing a surge in entrance;
(b) inability to access equipment except by using divers;
(c) system was in the entrance adjacent to the navigation channel, providing some constriction to navigation;
(d) funding was not guaranteed for multiple-year operations;
(e) funding was not available (budgets not confirmed) until 1 to 2 months after start of operational window;
(f) equipment was designed to operate at two sites;
(g) expensive booster station.

Present plant status: (as of 1996)

Entrance of harbour had been dredged for many years by conventional suction dredge. Owing to insufficient funding to continue with phase III, the system was closed in 1996 pending removal. At September 1996, documentation was being finalised to call for tenders to remove all of the system. The barge had been removed, and capital equipment on it sold.

Tenders closed 6 November 1996 for the approx. $3 million (US) removal of the bypass system including pipes on breakwaters and to jet pumps, cross channel discharge pipe, support piles, pipe rack, south and north riser structures for jack-up barge, fluidisers, jet pumps. Optional items for removal included the discharge pipe line from the beach south of San Luis Rey River Groin. Items to remain include the booster pump station, discharge pipe between the southern breakwater and San Luis Rey River Groin, and pipes under the southern breakwater spur.

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Figure G1: *Oceanside Harbour Sand Bypassing System locations (Patterson et al, 1991).*

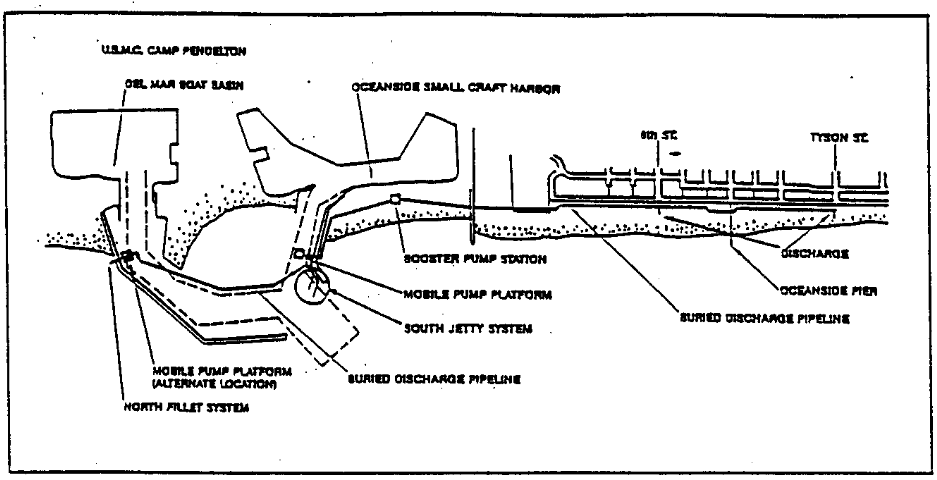


Figure G2: *Oceanside Harbour Sand Bypassing System, Locality plan (Weisman et al, 1996).*

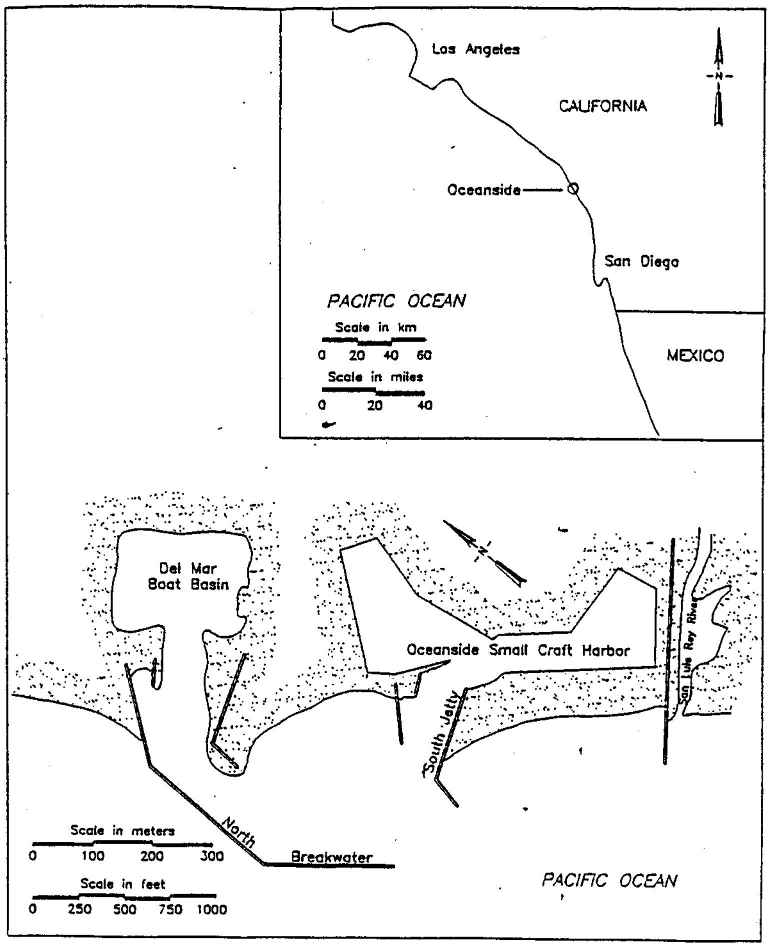
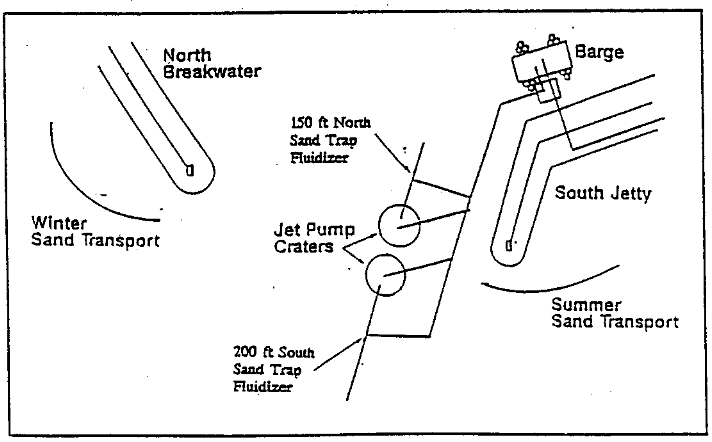


Figure G3: *Oceanside Harbour Sand Bypassing System, Fluidiser locations (not to scale) (Weisman et al, 1996).*



Appendix H

Data sheet: South Lake Worth Inlet Sand Bypassing System, Florida, U.S.A.

Location:	South Lake Worth (Boynton) Inlet is an artificial entrance located in Palm Beach County, Florida, USA, connecting Lake Worth to the Atlantic Ocean. The two adjacent inlets are Boca Raton Inlet 23 km to the south, and Lake Worth Inlet 25 km to the north.
Problem:	The inlet was constructed in 1927 to provide tidal circulation thereby improving the water quality of the Lake. The training walls halted the net southerly transport resulting in erosion of the adjoining southern beach, and also shoaling of the entrance channel from sand moving around the northern training wall. The erosion downdrift threatened upland structures and Highway A1A.
Wave climate:	Varies seasonally; influenced by the sheltering effects of the Bahamas. Strong north-east storms in winter produce the net southerly drift, while more persistent southerly waves generated by local winds occur during summer. Tropical storms and occasionally hurricanes also affect the area (Walker and Dunham, 1977).
Inlet characteristics:	Width varies from 90 m at the entrance to 40 m; depth = 3.0 m (MSL); spring tide range = 3.3 ft (~1.0 m); semidiurnal tides; flood channel flows = 5 ft/s (~1.5 m/s).
Inlet usage:	Small commercial and recreational craft.
Sediment characteristics:	60 % shell; 40 % medium to course sand with significant fractions of quartz and feldspars. Grain size bypassed is slightly in excess of 0.3 mm.
Drift rate:	Net southerly drift = 134,000 - 172,000 m^3/yr (Dombrowski and Mehta, 1990).
Beach erosion rate:	Mean recession rate to approx. 4000 m south of the inlet = 0.9 m/yr with the existing bypass system (Dombrowski and Mehta, 1990).
Type of bypass:	Fixed hydraulic suction dredge with a rotating boom.
Bypass system components:	Initial plant (installed 1937): 8 inch (~200 mm) suction line; 6 inch (~150 mm) diesel centrifugal pump (65 hp); 1200 ft (~365 m) of 6 inch (~150 mm) discharge line crossing the inlet via the highway bridge. An A-frame derrick on the roof of the pump house enabled the intake to be swung in a horizontal arc as well as raising and lowering. The bypass plant was situated on the northern training wall approx. 50 ft (~15 m) from the seaward end. Upgrade, 1948: 10 inch (~250 mm) intake mounted on a swinging boom of 30 ft (~9.1 m) radius with a flexible rubber sleeve at the centre of the turning radius; jet attached to side of intake for agitating sand; 8 inch (~200 mm) diesel centrifugal pump (600 rpm); 1200 ft (~365 m) of 8 inch (~200 mm) discharge line. The bypass plant can create a circular trench of 8 - 10 ft (~2.4 - 3.0 m) depth and 30 ft (~9.1 m) length with a sand fill capacity of ~800 m^3 (~1050 cy). (Caldwell, 1950; Dombrowski and Mehta, 1990). Upgrade, 1967 (present plant): 125 m curved extension to the northern breakwater (curved to the south); 20 m extension to southern breakwater; training wall constructed from the inlet to Lake Worth; plant relocated 36 m seaward of the 1937 position (or approx. 100 ft (~30 m) seaward of the MHW line on the north breakwater); 12 inch (~300 mm) suction intake line; diesel Caterpillar engine pump (400 hp) rated to pump 4,000 gpm with 20% solids in suspension; 10 inch (~250 mm) discharge line. (Yeend and Hatheway, 1988; Dombrowski and Mehta, 1990).

Outlet type:	Discharge pipe on to southern beach to deposit between 60 and 150 m south of the inlet. The pipeline crosses the inlet by the highway bridge.
Bypass rate:	Average bypass rate = 53,500 m^3/yr; pumping capacity = 110 m^3/hr (Dombrowski and Mehta, 1990).
Degree of bypassing: (e.g. all, 50%, etc.)	35 % artificial bypassing; 45 % natural (Dombrowski and Mehta, 1990).
Costs:	Initial plant (installed 1937): installation cost = $15,000 (US). Upgrade, 1948: installation costs = $15,000 - 20,000 (US, 1950 prices). (Caldwell, 1950) Upgrade, 1967 (present plant): not known The unit price for sand bypassing is $8 - 9 /$m^3$ (US) (Bruun, 1993).
Funding:	Initial plant (installed 1937): South Lake Worth Inlet District and a property owner. Upgrade, 1948: Palm Beach County. (Caldwell, 1950)
Contract type:	Not known.
Owner:	Publicly owned.
Operator:	Palm Beach County.
Supervisor of operations:	Not known.
Staffing:	2 people for maintenance and operation (Caldwell, 1950).
Operating cycle:	All year round, the operating period being governed by the rate of infill of the bypassing trap. Peak pumping periods occur during September to March (Yeend and Hatheway, 1988). In Caldwell (1950) the plant operated 2 to 3 hours during calm weather, while during periods of north-east weather, pumping for 18 hours still did not match the transport rate.
Environmental constraints:	Not known.
Environmental management issues:	Beaches on both sides of entrance are heavily used.
Commencement date of bypassing:	Original plant: 1937 (ceased operation 1942 - 1945 during World War 2).
Performance: (include any leakage to inlet, formation of entrance bar, etc.)	The plant only bypasses 35 % of the southerly drift with 45 % naturally bypassing via the inlet ebb tidal shoal and bypass bar which attaches to the beach approx. 600 - 900 m south of the inlet. A further 11 % is retained by the northern training wall, and 7 % is deposited on the flood and ebb shoals (2 % of the material entering the flood shoal is dredged and placed on the southern beach). The limitation of reach and capacity prevent a full 100 % bypassing. On only a fifth of occasions does the crater fill faster than dredged (Olsen, 1996). The original design had been for a system with a large boom mounted on rails to give greater trap capacity. The strong velocities produced by the narrow entrance have scoured the

channel to a hard bottom. A bar exists seaward of the entrance.

Present plant status: (as of 1996) Still in operation.

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***Figure H1**: South Lake Worth Inlet Sand Bypassing System, Locality plan (Olsen Associates, 1996).*

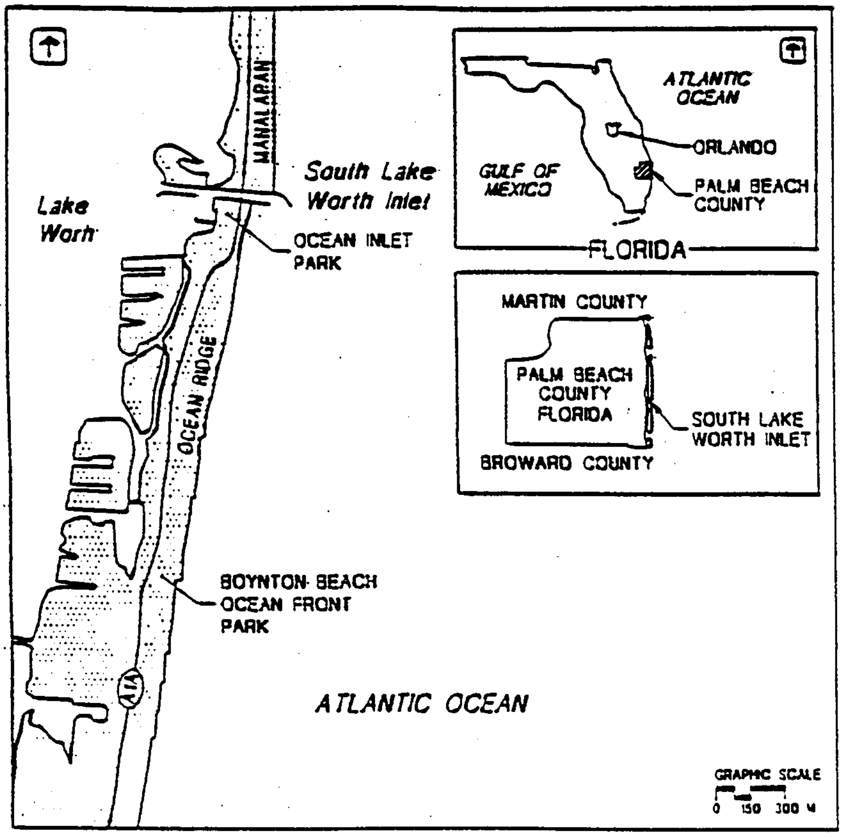

***Figure H2**: South Lake Worth Inlet Sand Bypassing System (Yeend and Hatheway, 1988)*