ANNEX E:

BEACH MORPHOLOGY AND PREDICTION OF SHORELINE CHANGES

Annex E: Beach Morphology and Prediction of Shoreline Changes

E.1 Diagrams of Shoreline Changes Based on Historical Maps

In Chapter 4, the amounts of advance or retreat of the shoreline have been presented in Figs. 4.1.1 to 4.1.3 of Volume 1. Many more diagrams of the shoreline advance or retreat have been prepared with the baselines of different survey years. They are presented in the following pages.

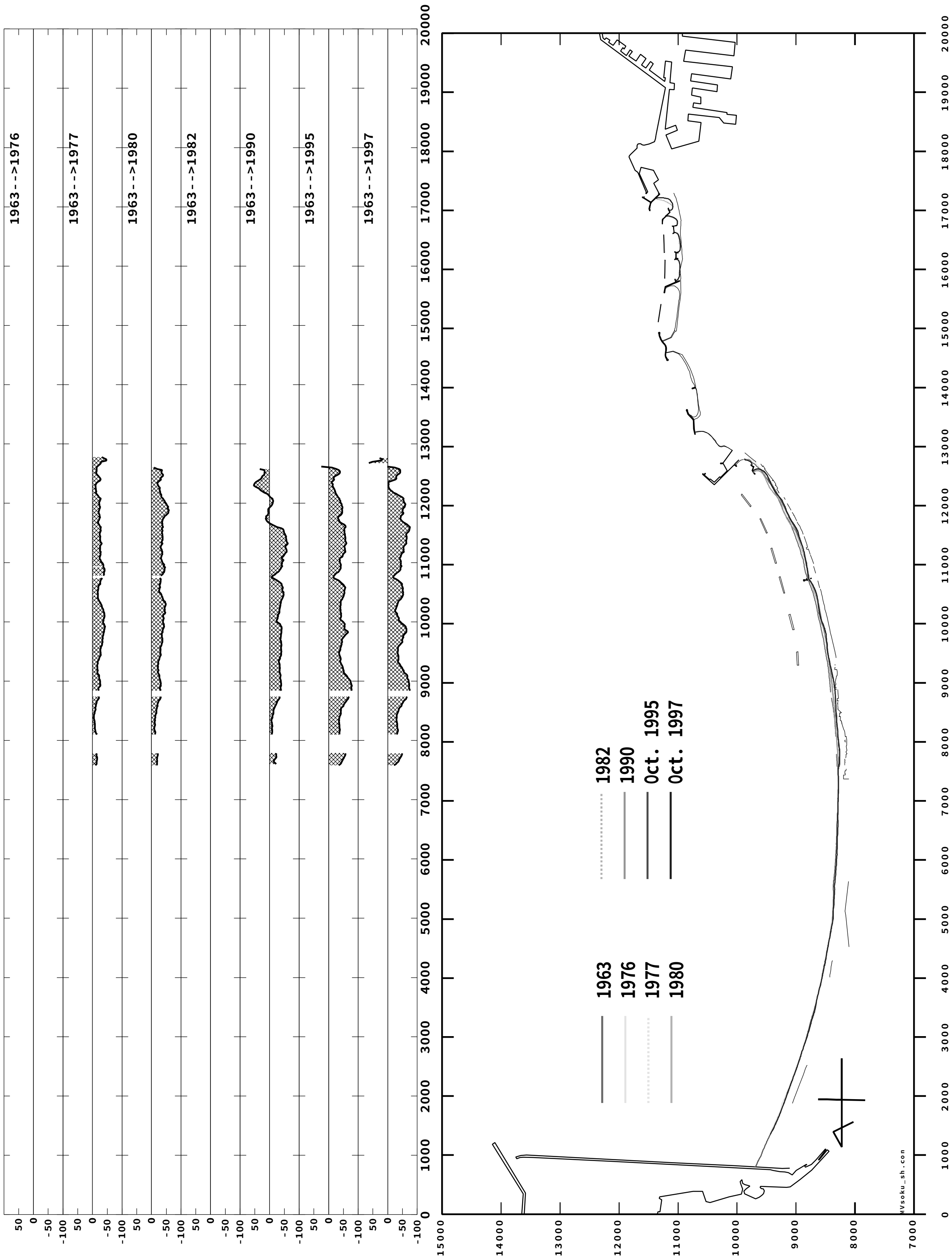

Fig. E.1.1: Advance or retreat of shoreline in Constanța Sector from Năvodari to Tomis with the baseline of 1963

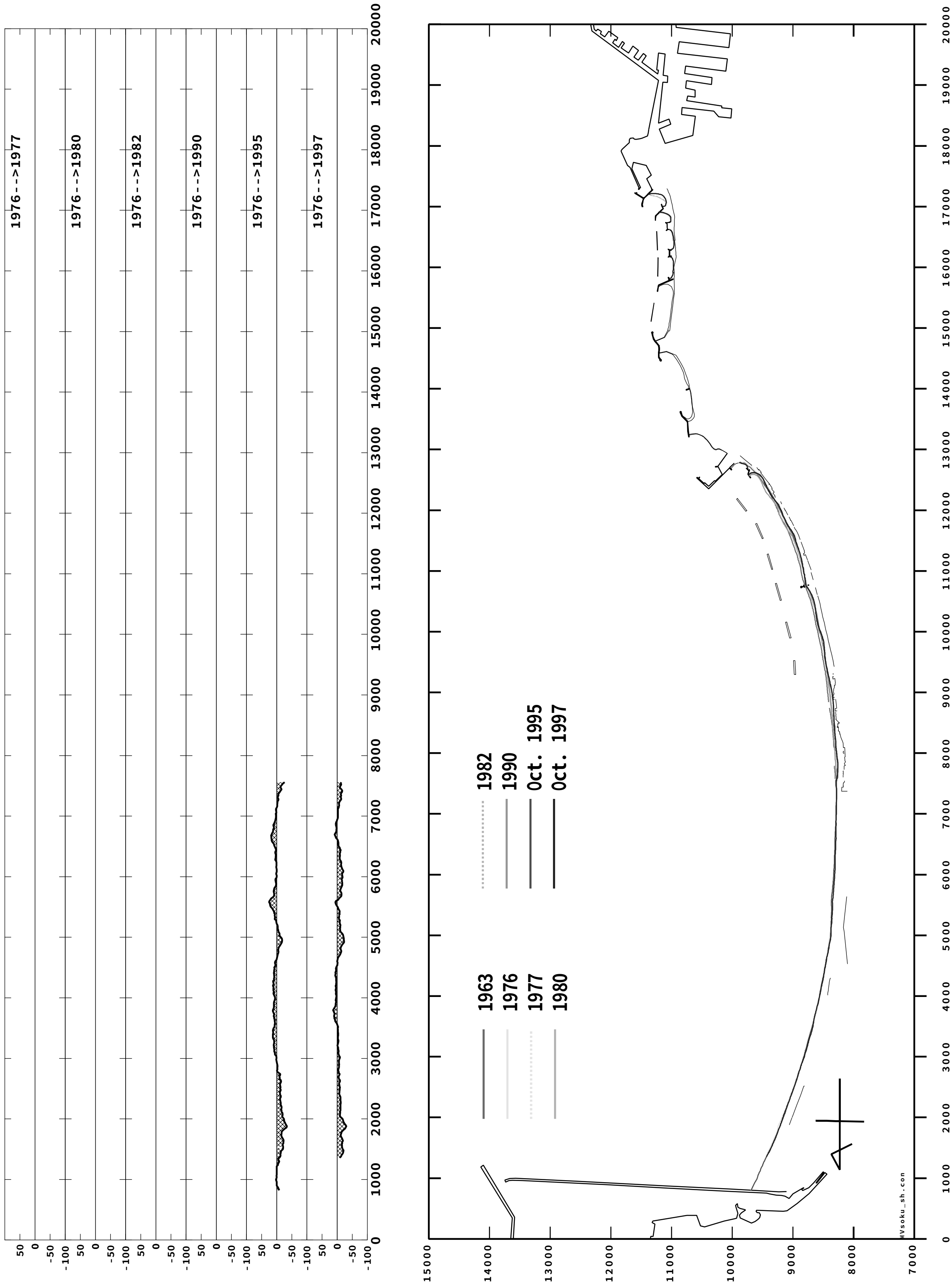

Fig. E.1.2: Advance or retreat of shoreline in Constanța Sector from Năvodari to Tomis with the baseline of 1976

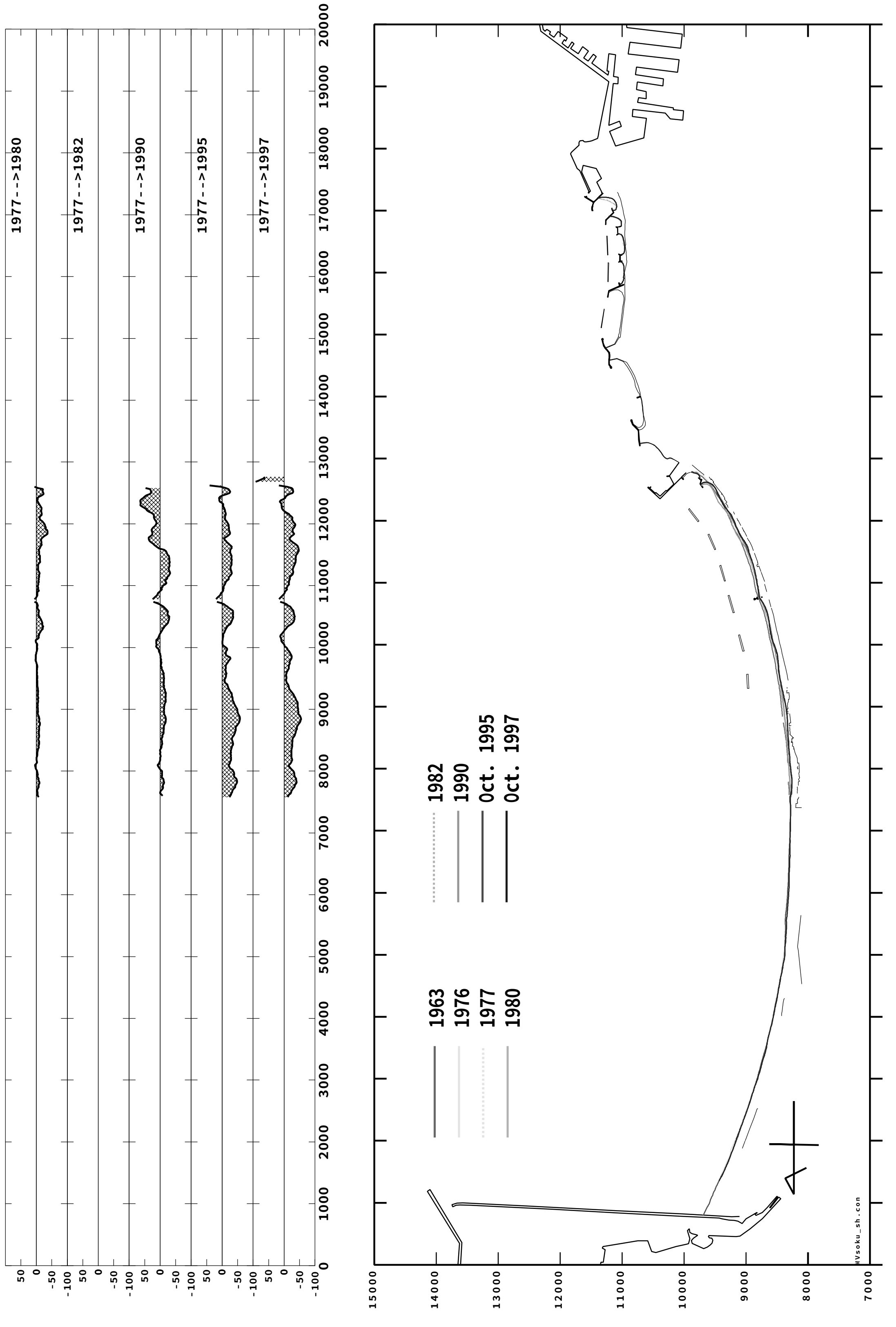

Fig. E.1.3: Advance or retreat of shoreline in Constanța Sector from Năvodari to Tomis with the baseline of 1977

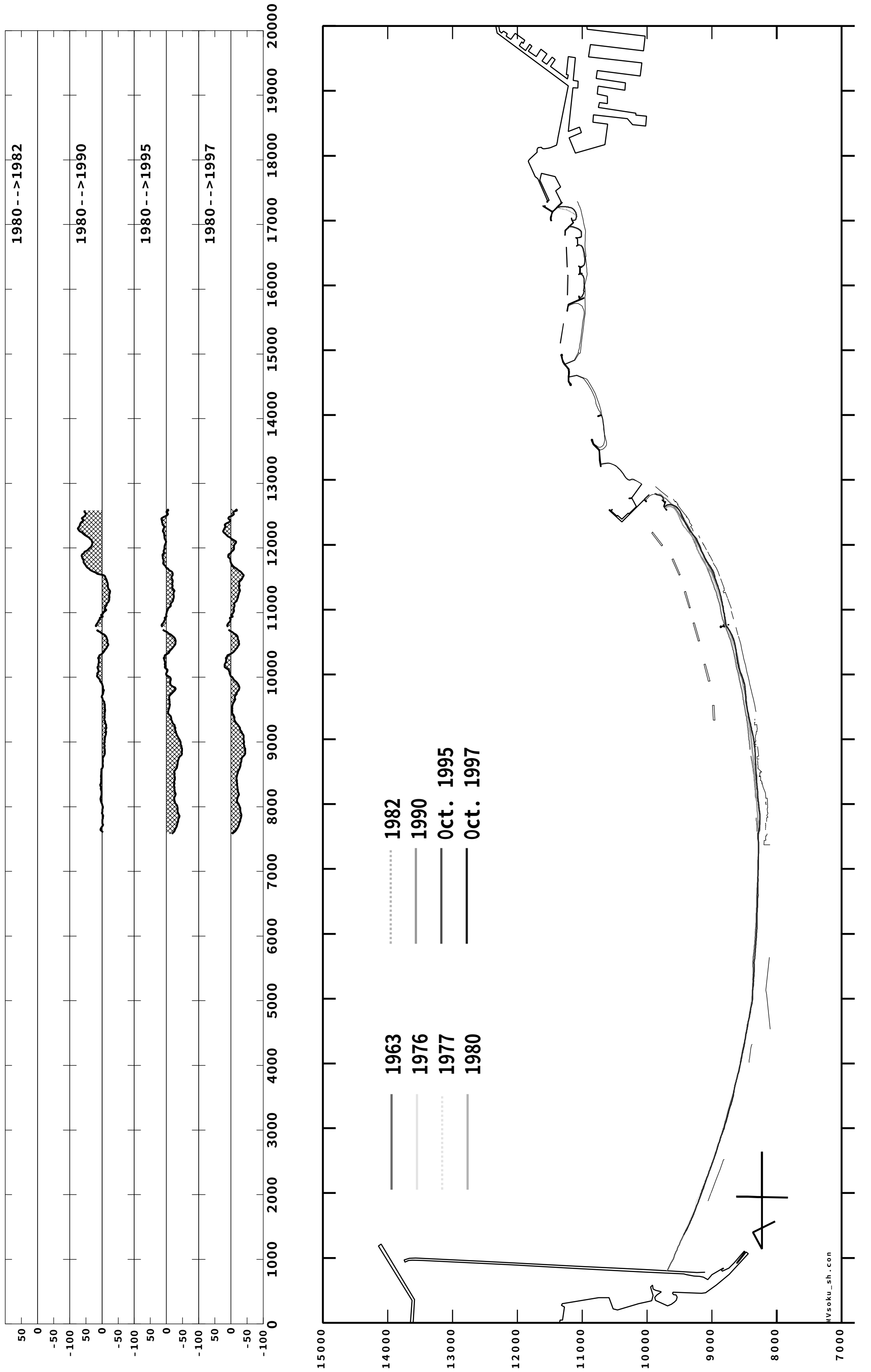

Fig. E.1.4: Advance or retreat of shoreline in Constanța Sector from Năvodari to Tomis with the baseline of 1980

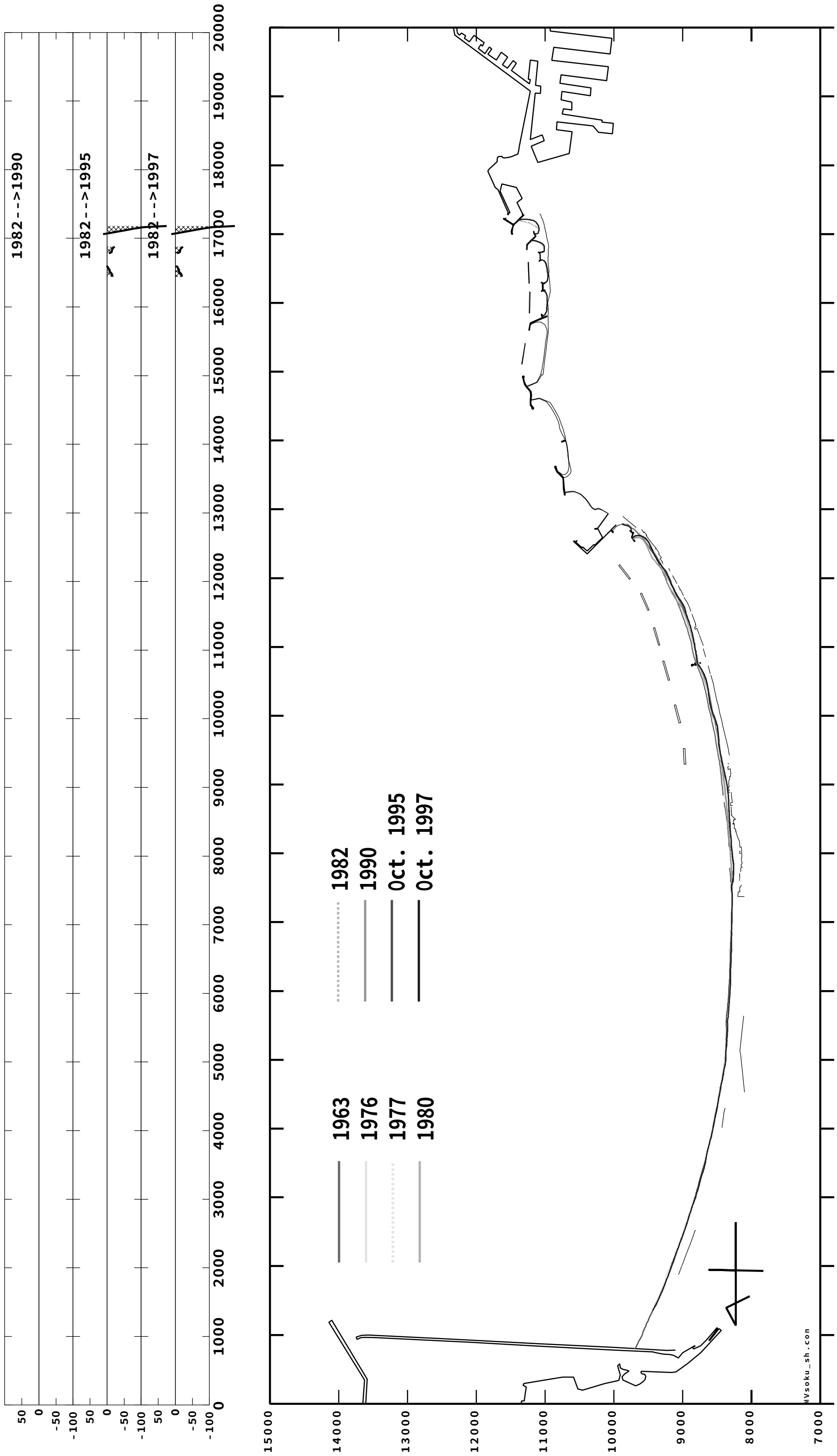

Fig. E.1.5: Advance or retreat of shoreline in Constanța Sector from Năvodari to Tomis with the baseline of 1982

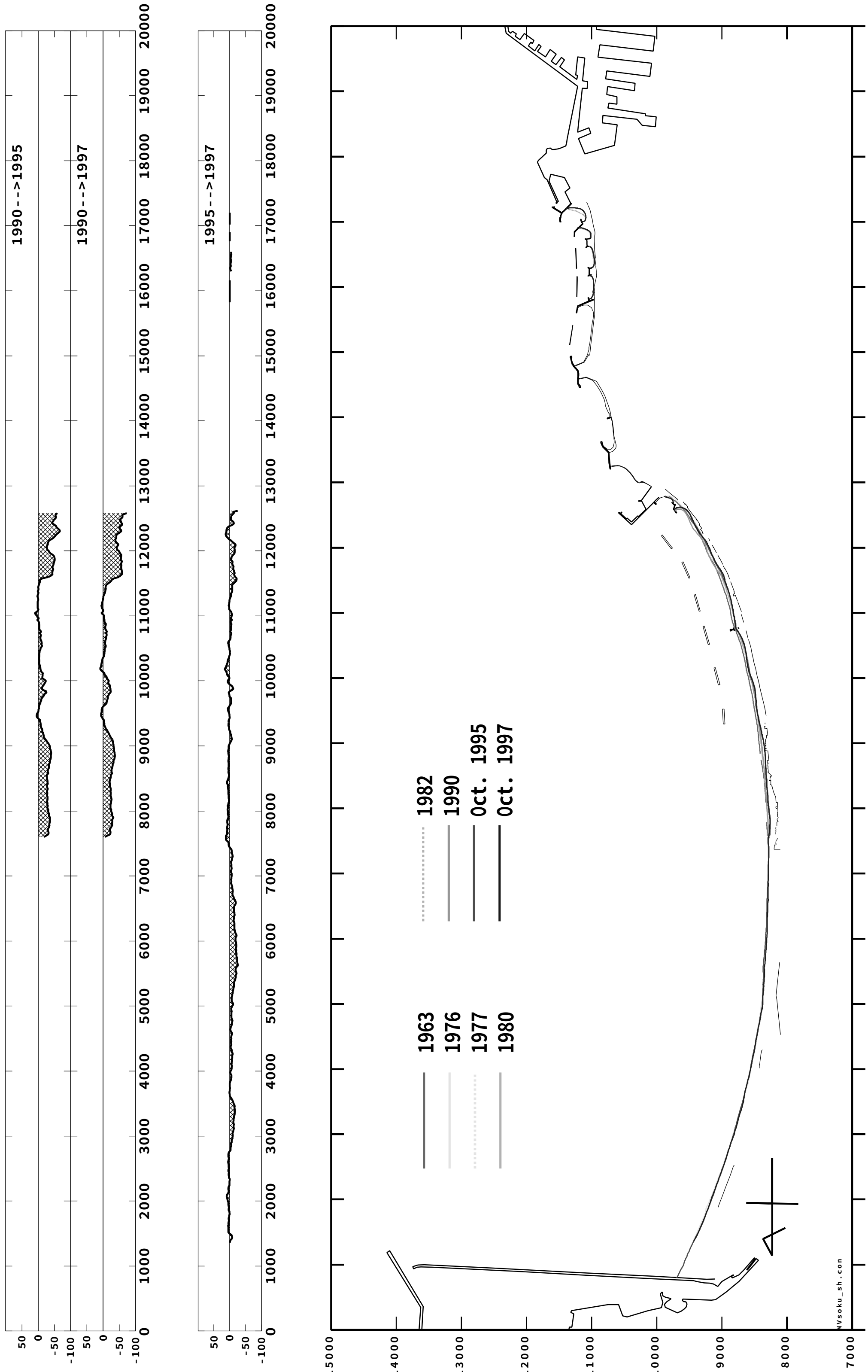

Fig. E.1.6: Advance or retreat of shoreline in Constanța Sector from Năvodari to Tomis with the baseline of 1990 and 1995

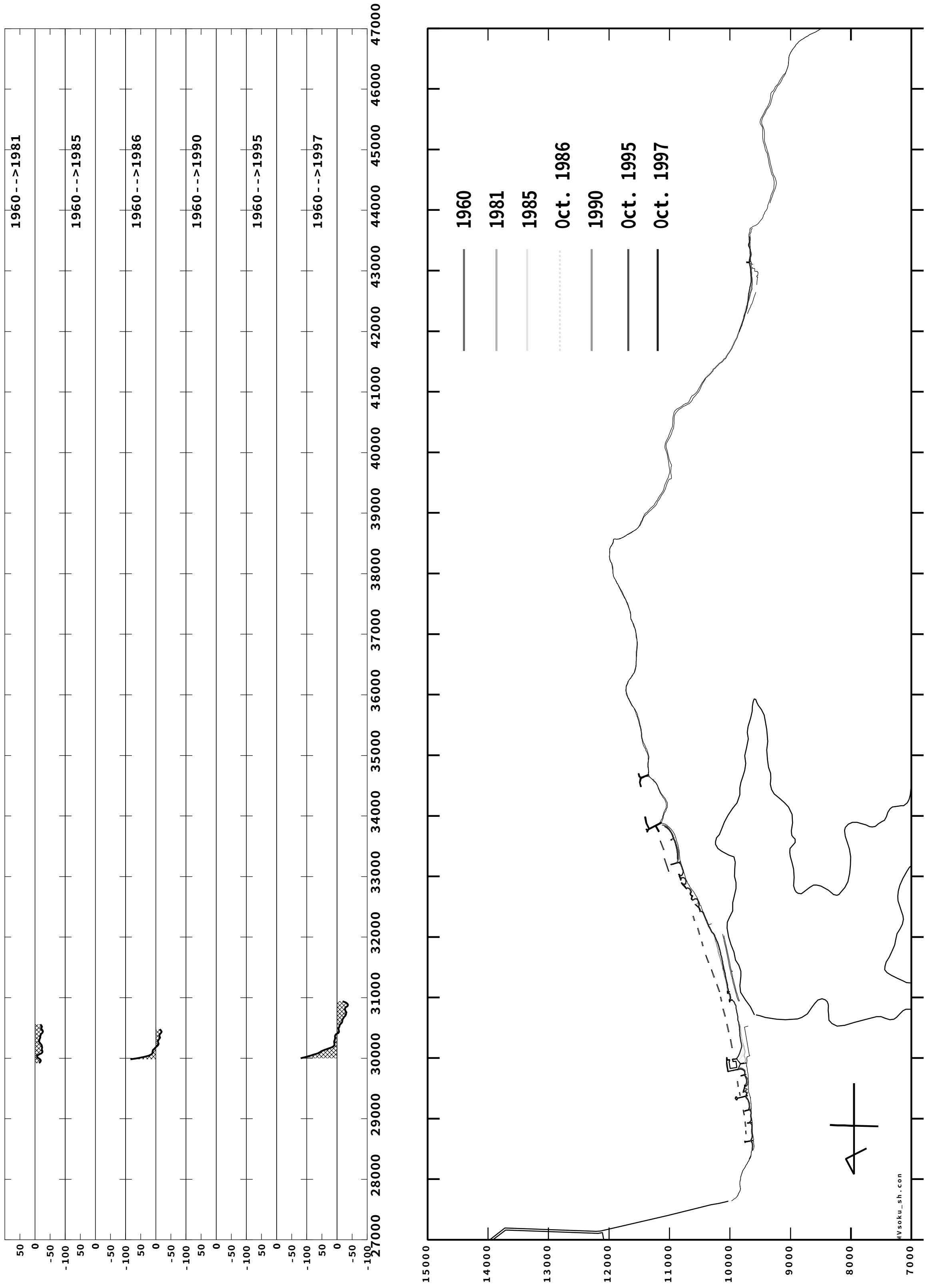

Fig. E.1.7: Advance or retreat of shoreline from Eforie Sector to Schitu with the baseline of 1960

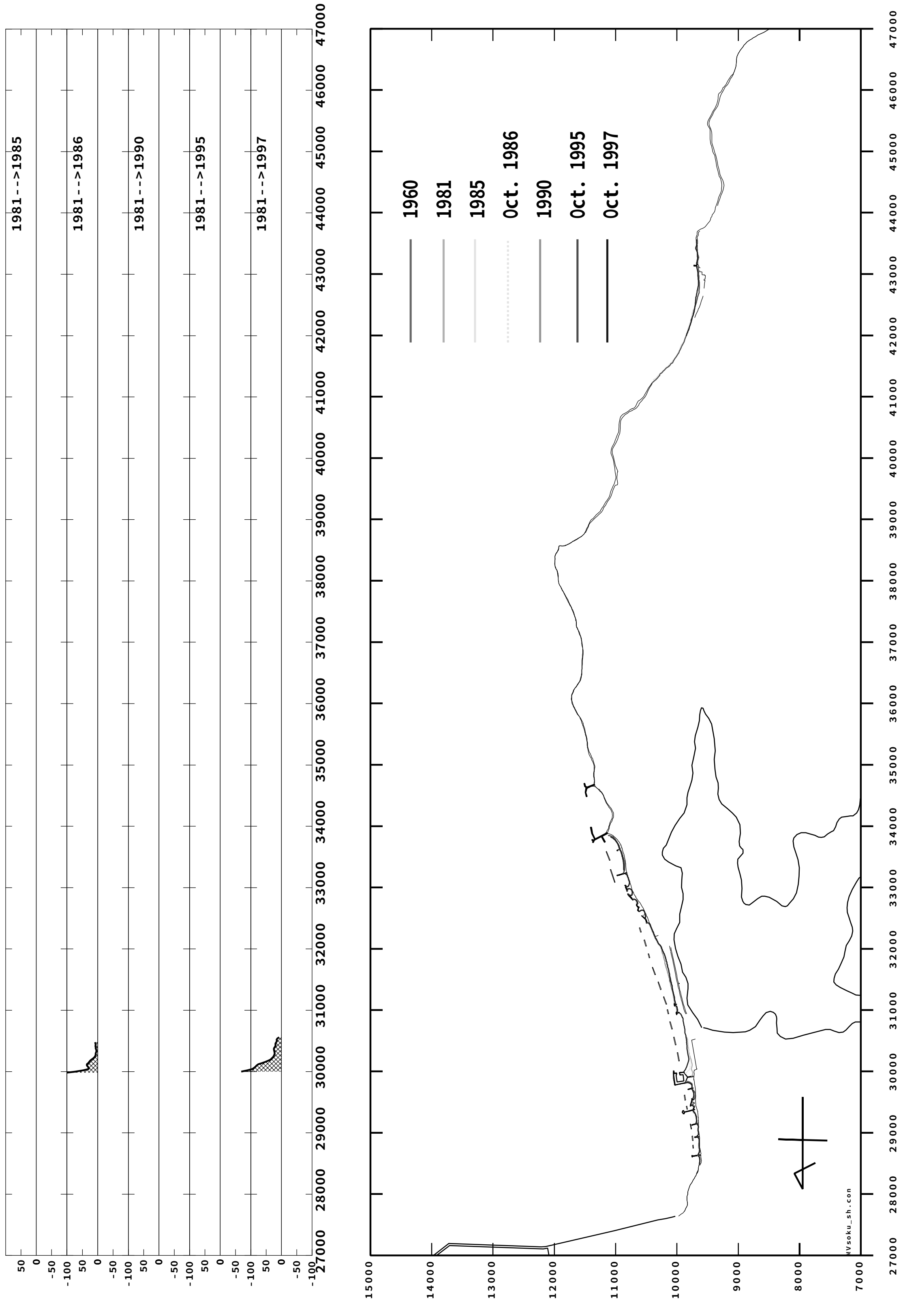

Fig. E.1.8: Advance or retreat of shoreline from Eforie Sector to Schitu with the baseline of 1981

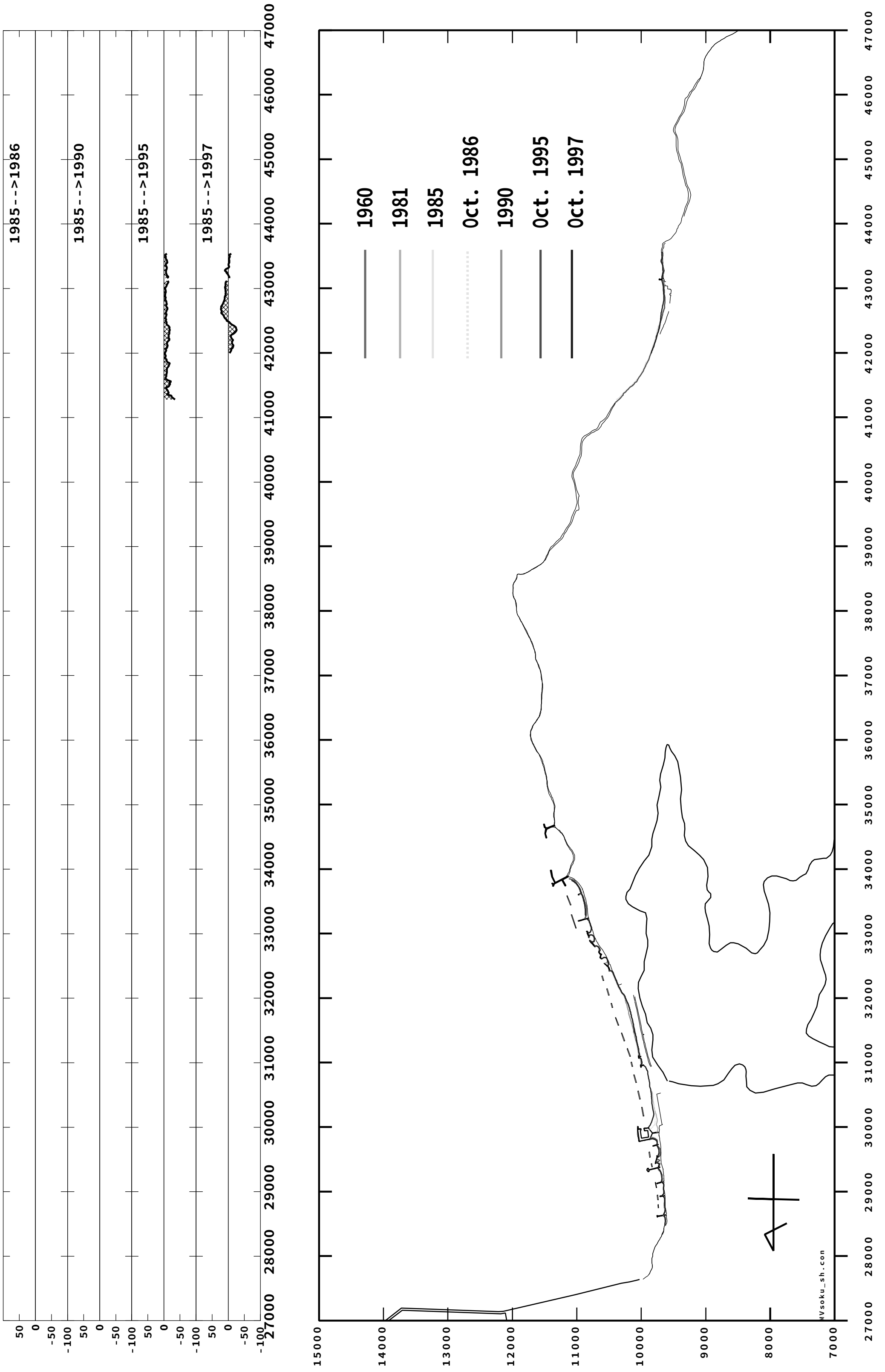

Fig. E.1.9: Advance or retreat of shoreline from Eforie Sector to Schitu with the baseline of 1985

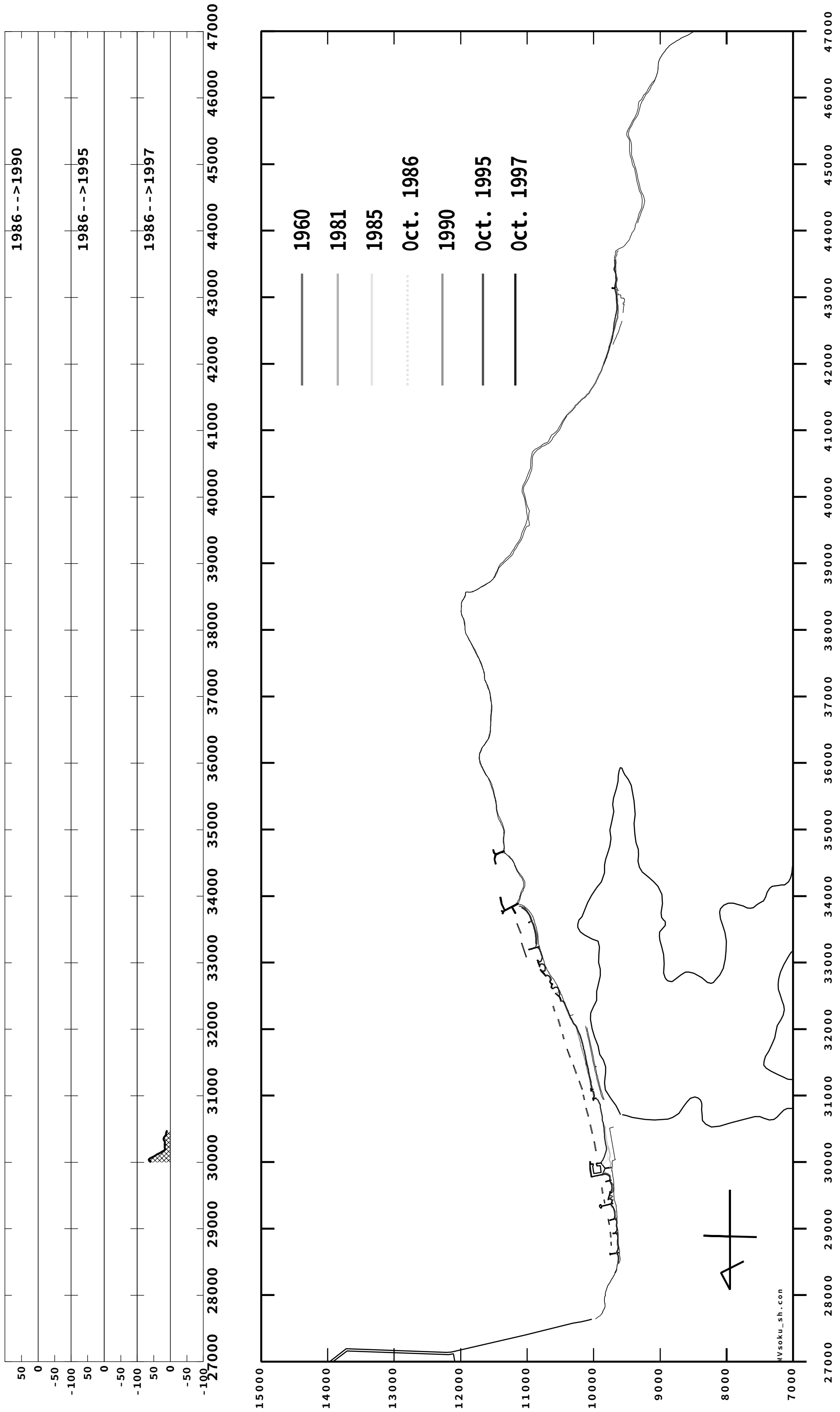

Fig. E.1.10: Advance or retreat of shoreline from Eforie Sector to Schitu with the baseline of 1986

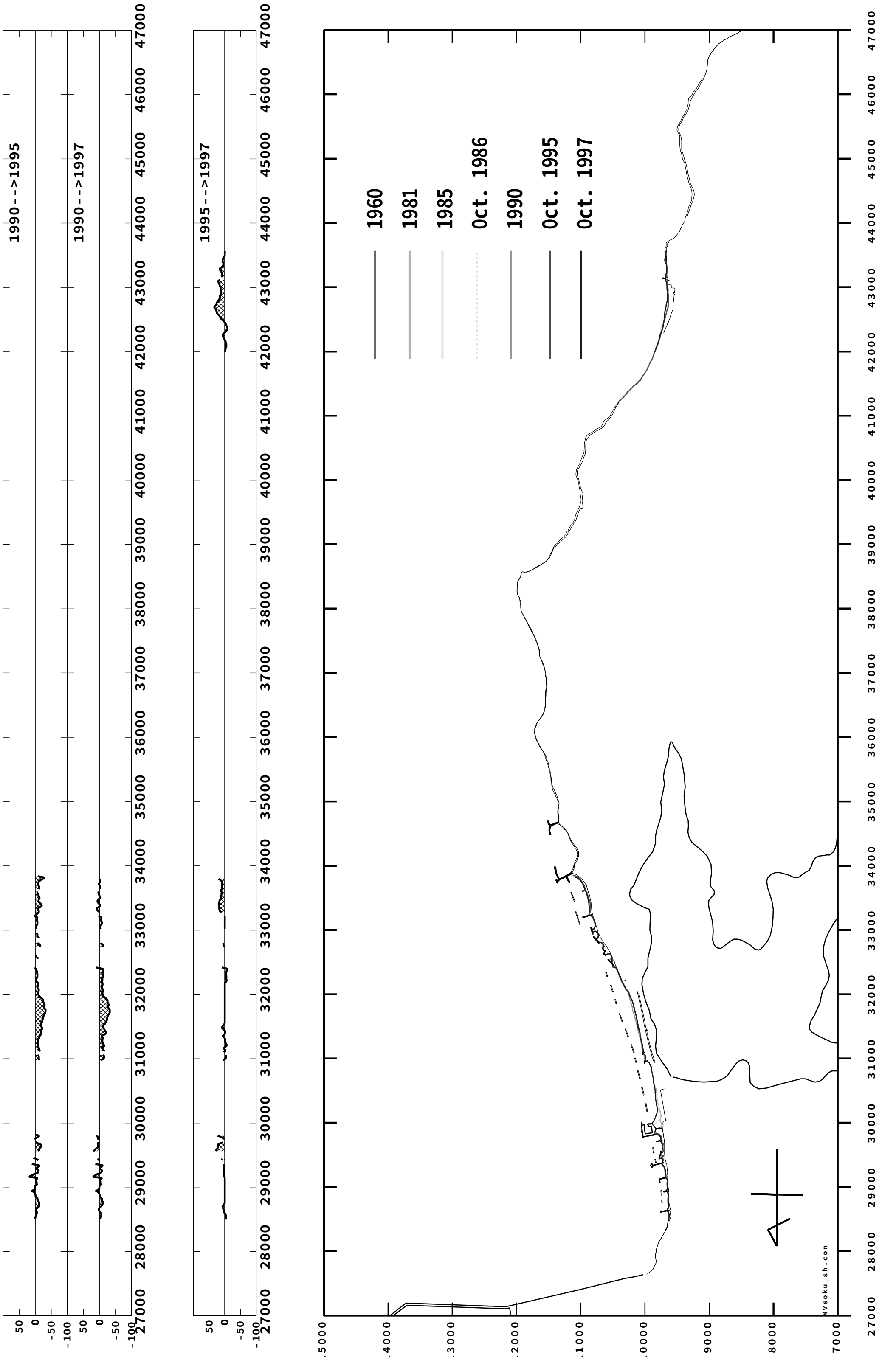

Fig. E.1.11: Advance or retreat of shoreline from Eforie Sector to Schitu with the baseline of 1990 and 1995

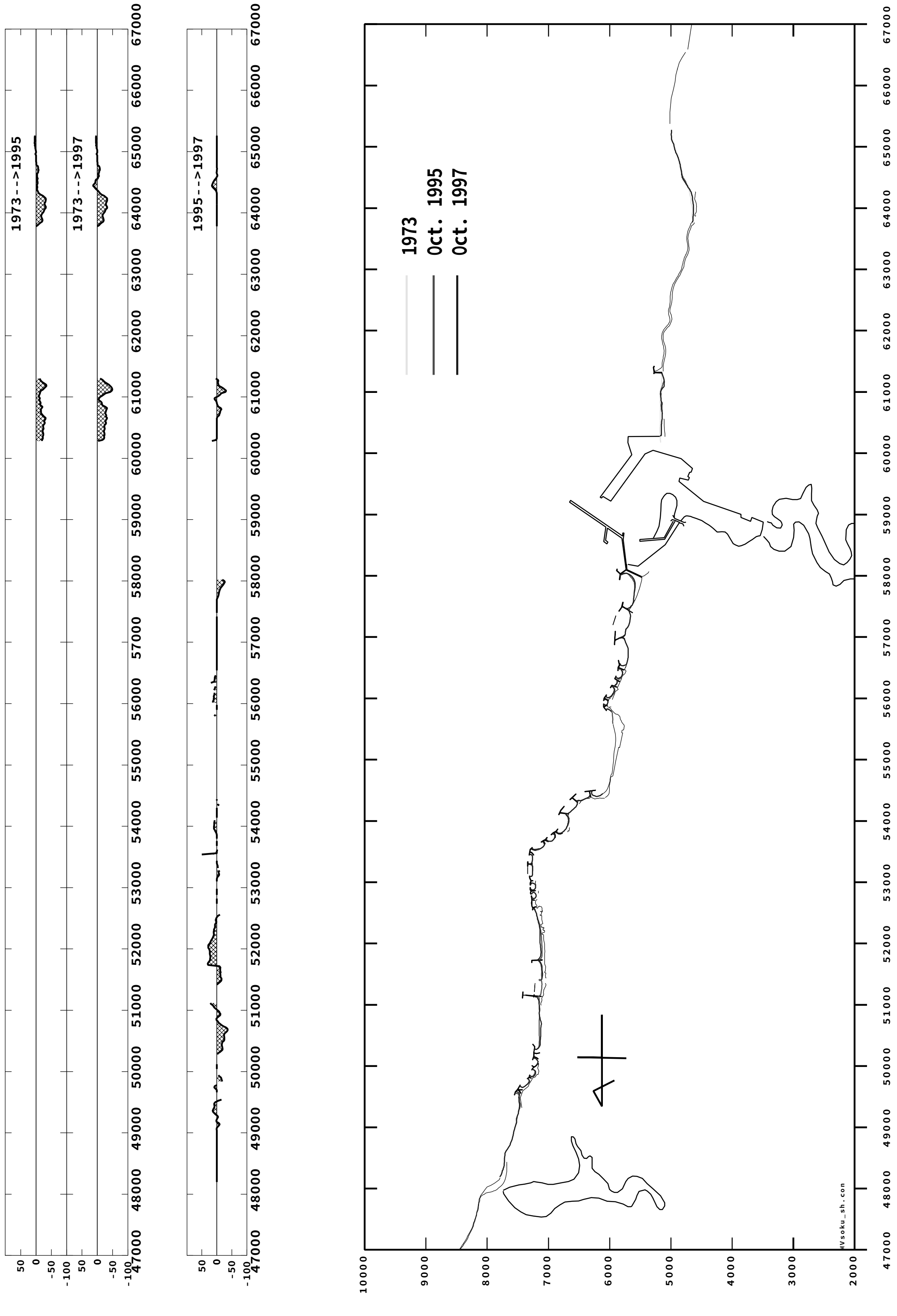

Fig. E.1.12: Advance or retreat of shoreline from Mangalia Sector to Limanu Sector with the baseline of 1973 and 1995

E.2 Results of Regression Analysis on Shoreline Locations

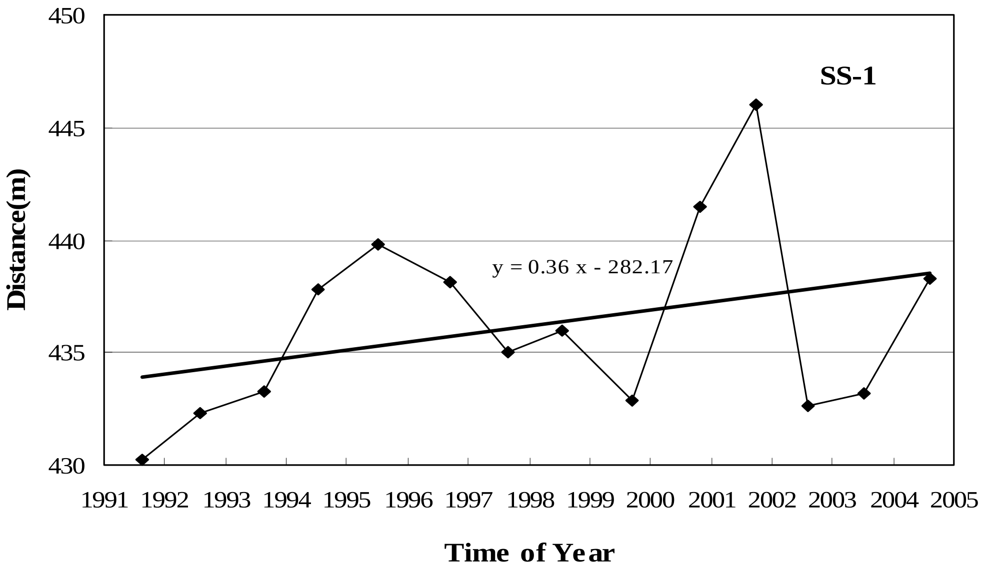

Fig. E.2.1: Regression line of shoreline distance versus time at SS-1

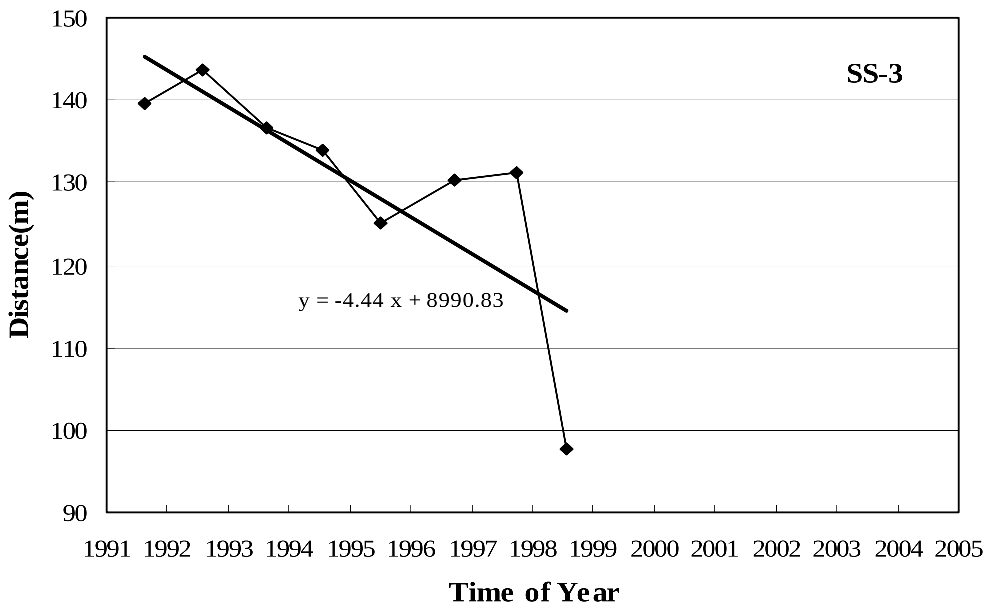

Fig. E.2.2: Regression line of shoreline distance versus time at SS-3

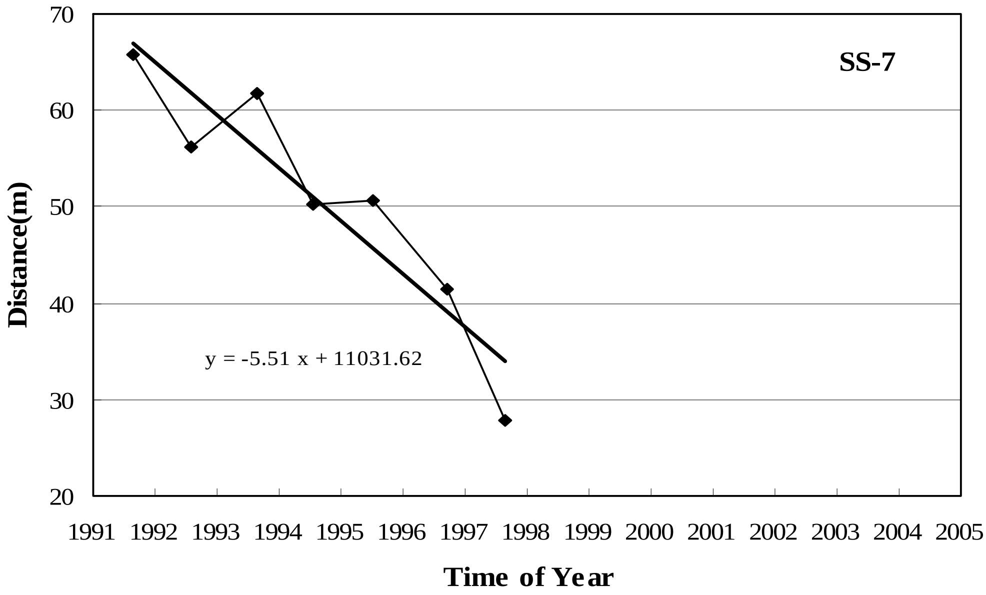

Fig. E.2.3: Regression line of shoreline distance versus time at SS-7

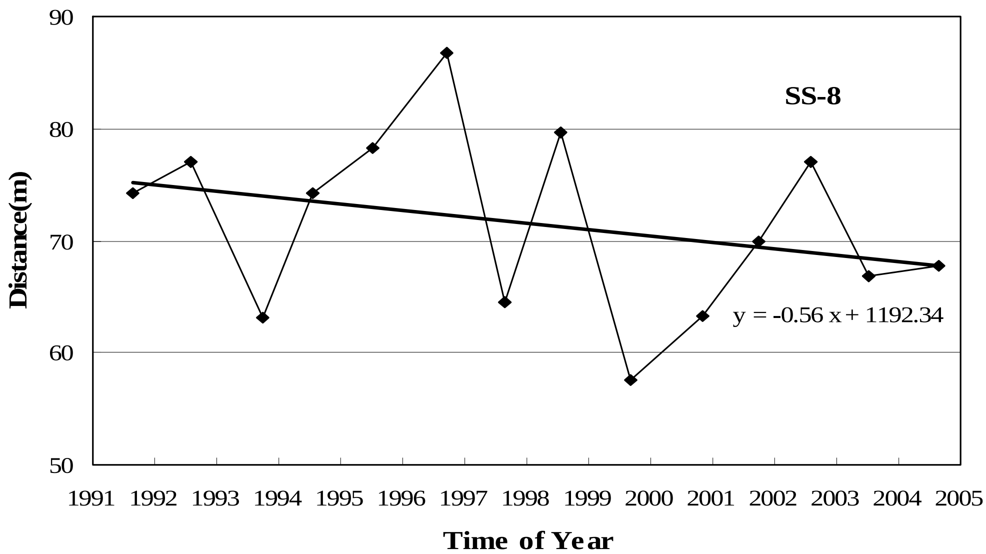

Fig. E.2.4: Regression line of shoreline distance versus time at SS-8

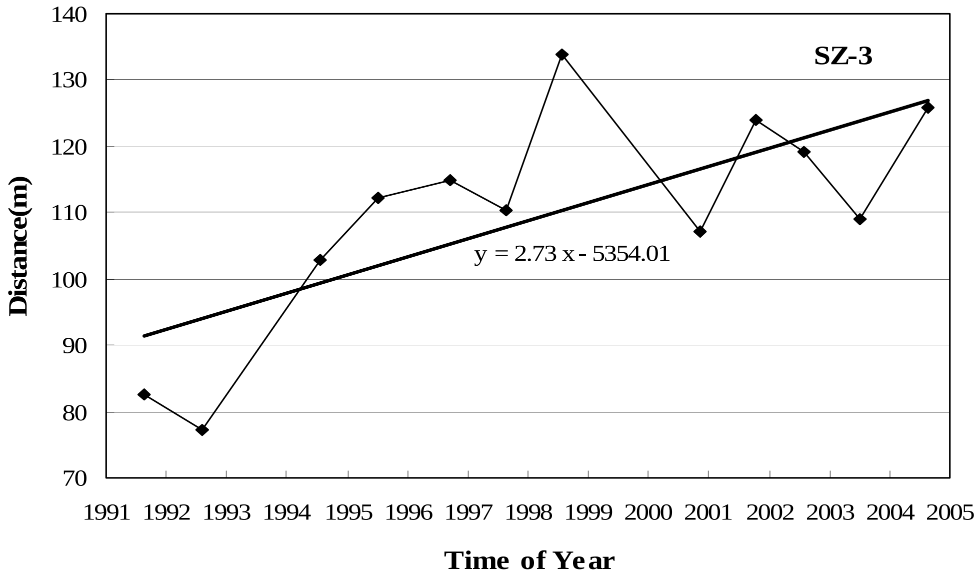

Fig. E.2.5: Regression line of shoreline distance versus time at SZ-3

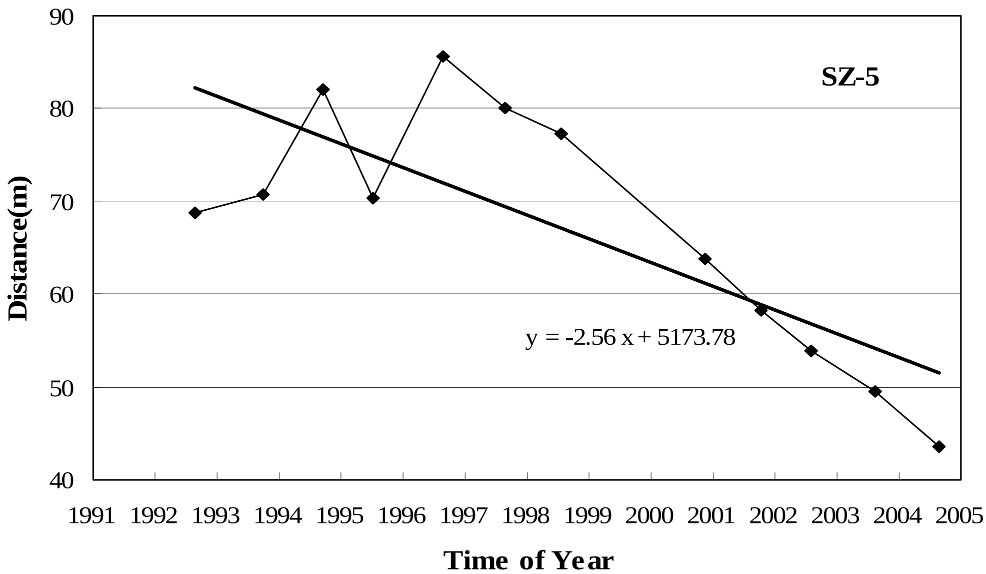

Fig. E.2.6 Regression line of shoreline distance versus time at SZ-5

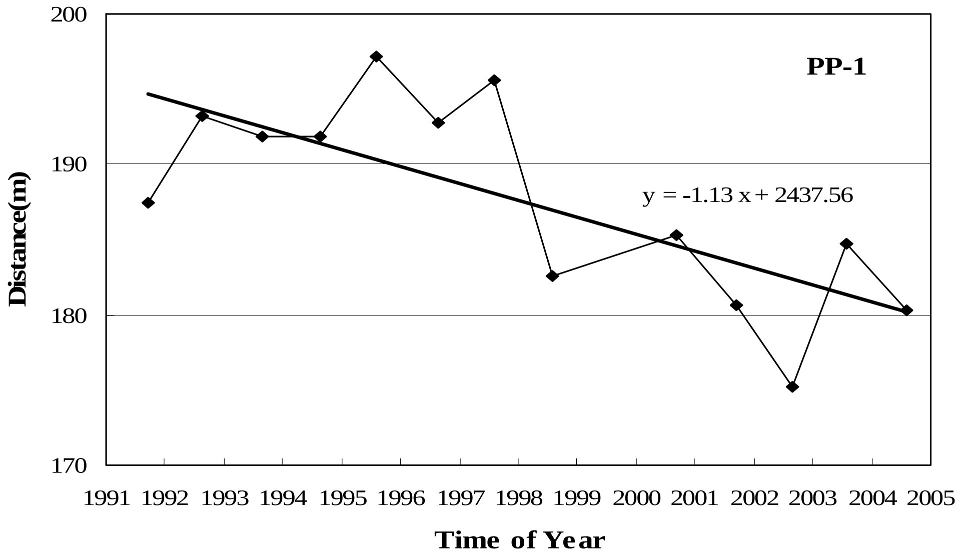

Fig. E.2.7: Regression line of shoreline distance versus time at PP-1

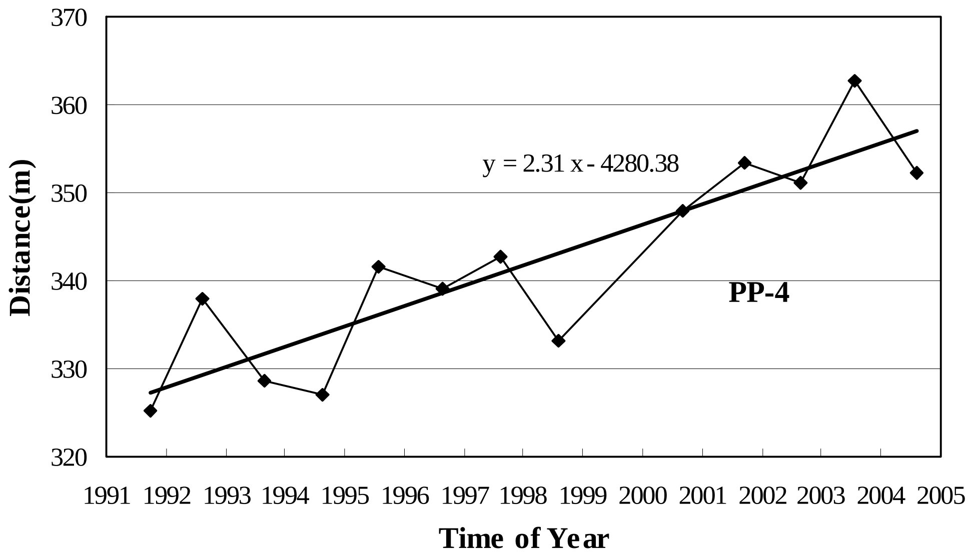

Fig. E.2.8: Regression line of shoreline distance versus time at PP-4

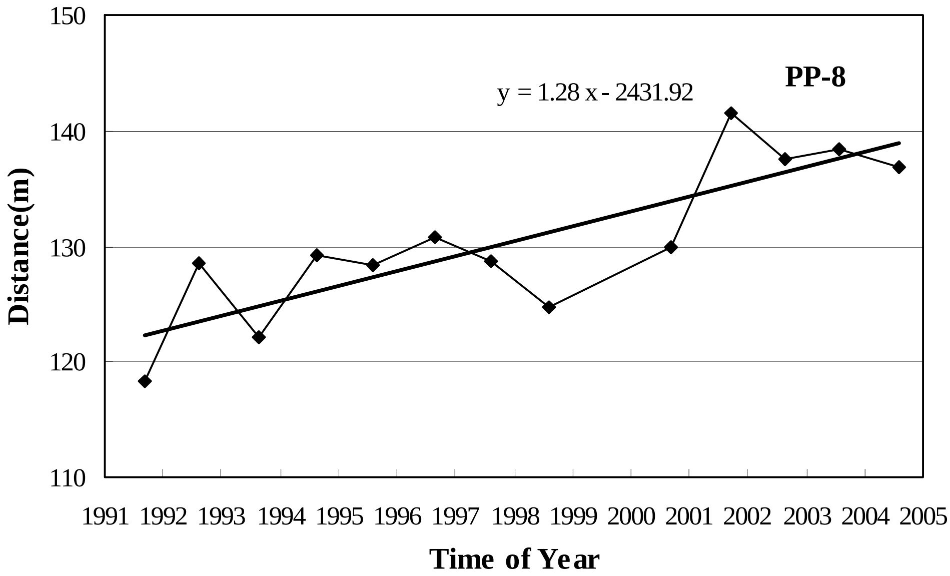

Fig. E.2.9: Regression line of shoreline distance versus time at PP-8

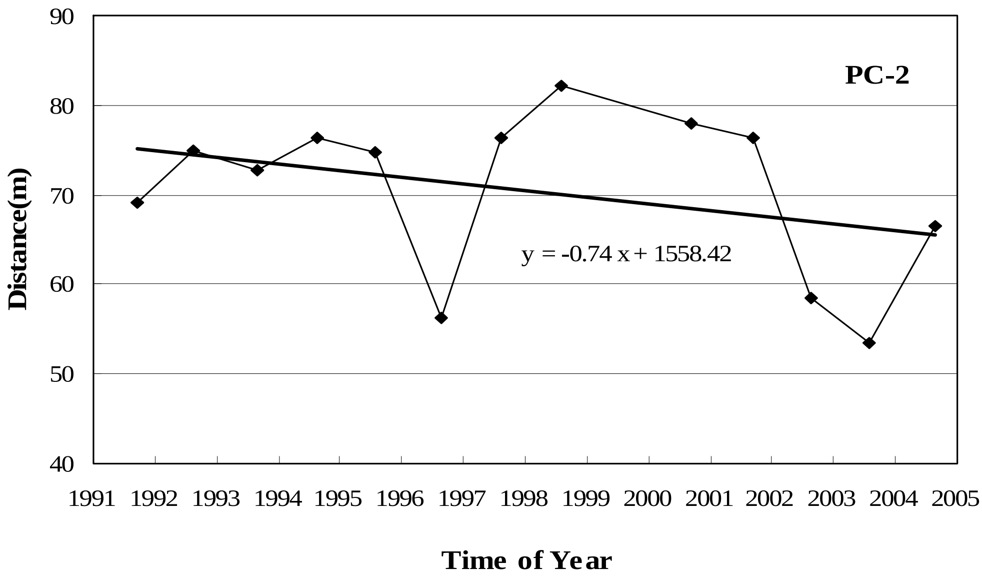

Fig. E.2.10: Regression line of shoreline distance versus time at PC-2

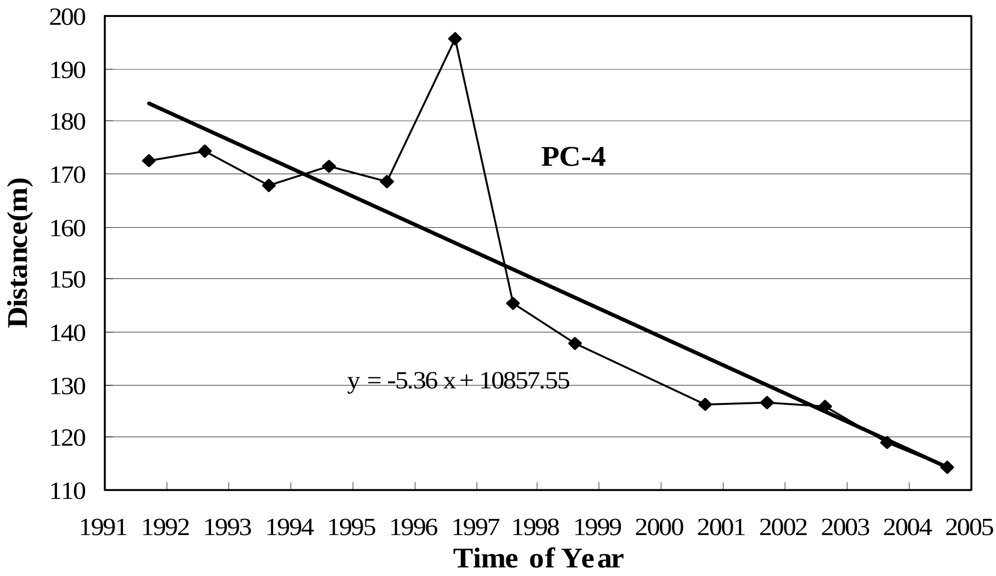

Fig. E.2.11: Regression line of shoreline distance versus time at PC-4

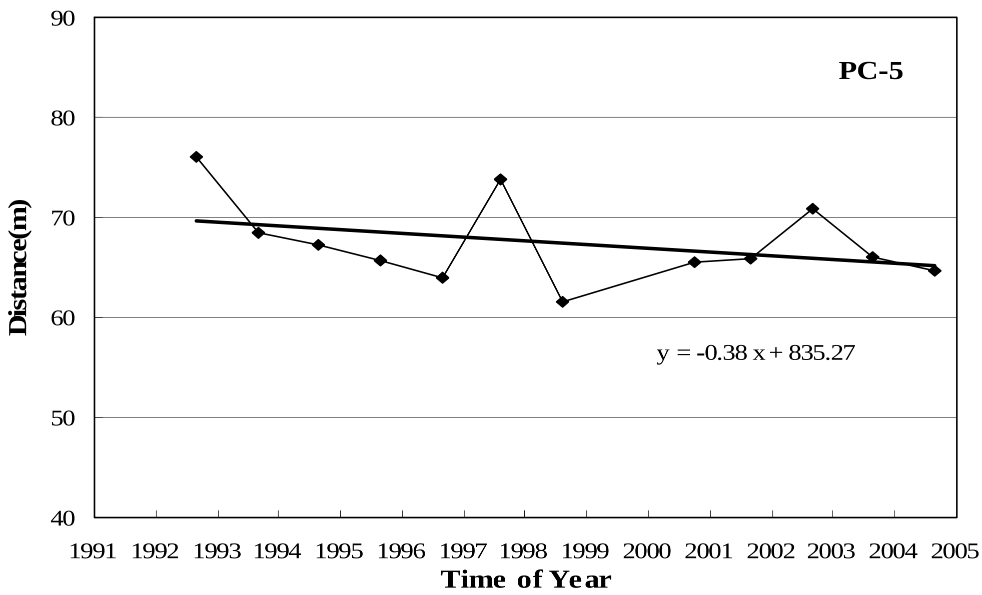

Fig. E.2.12: Regression line of shoreline distance versus time at PC-5

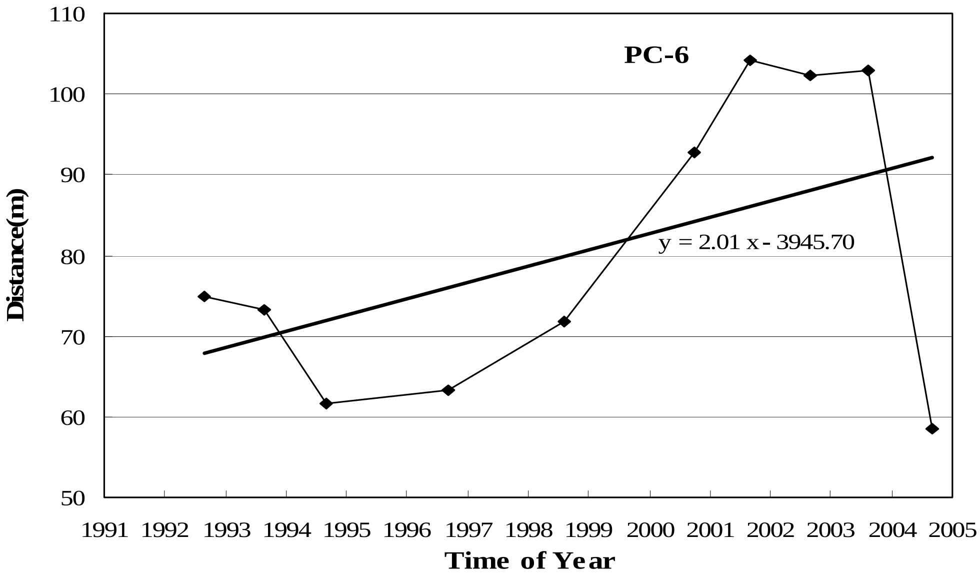

Fig. E.2.13: Regression line of shoreline distance versus time at PC-6

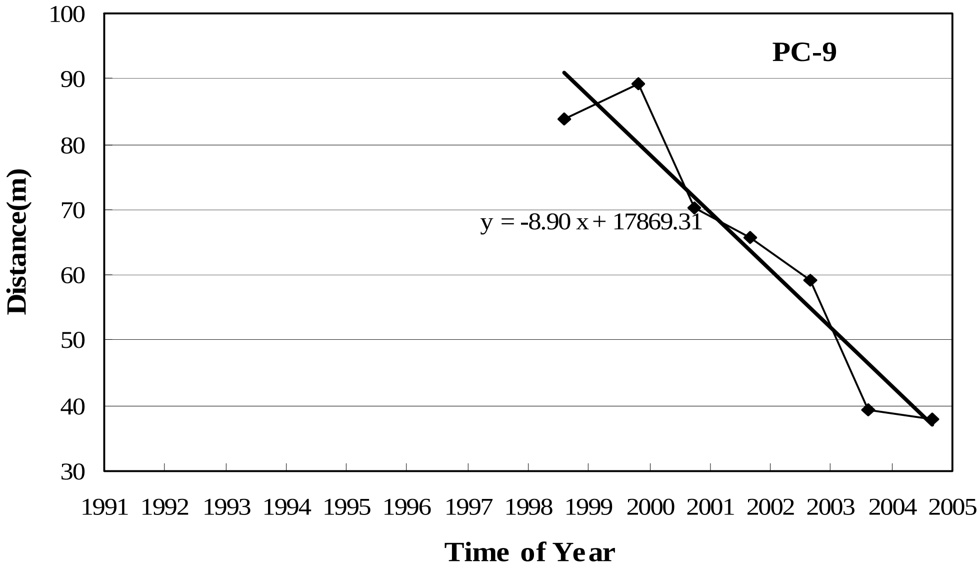

Fig. E.2.14: Regression line of shoreline distance versus time at PC-9

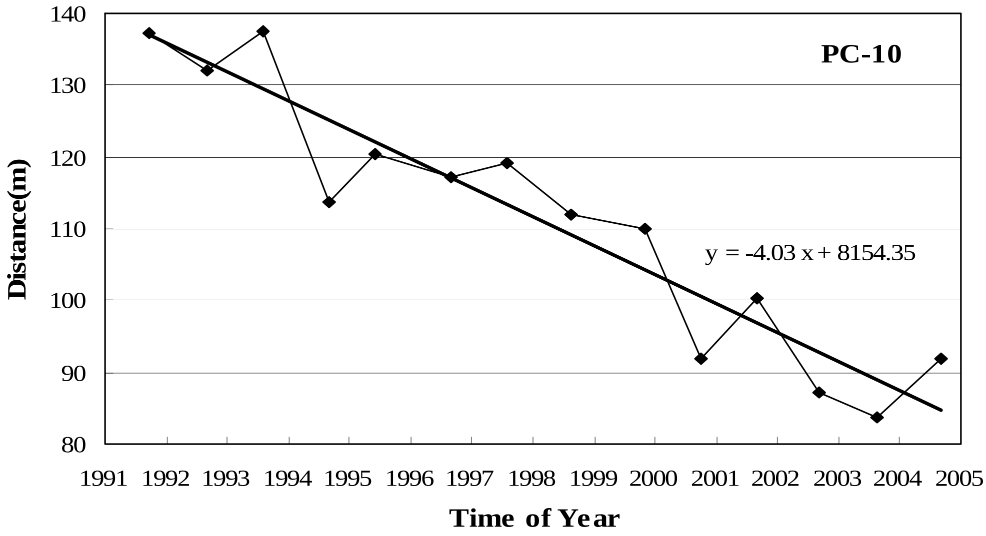

Fig. E.2.15: Regression line of shoreline distance versus time at PC-10

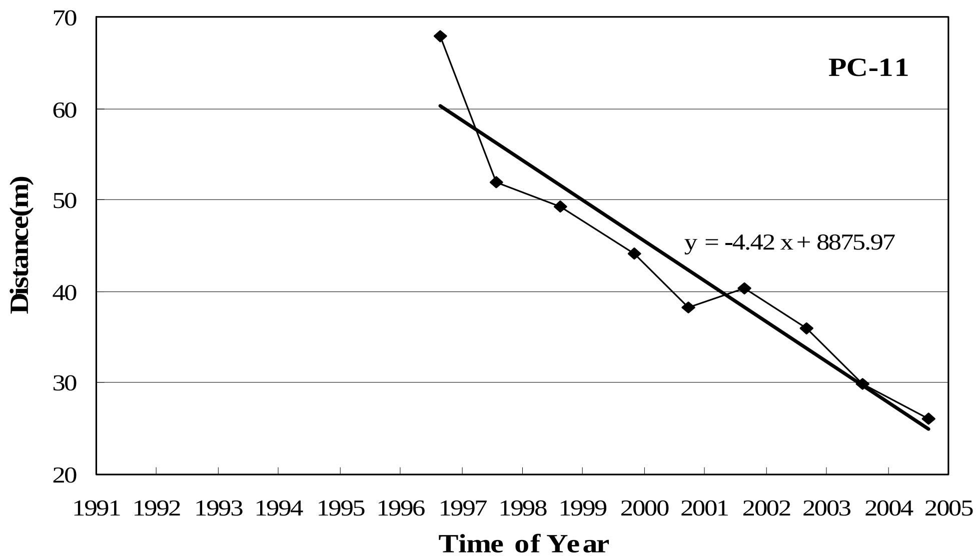

Fig. E.2.16: Regression line of shoreline distance versus time at PC-11

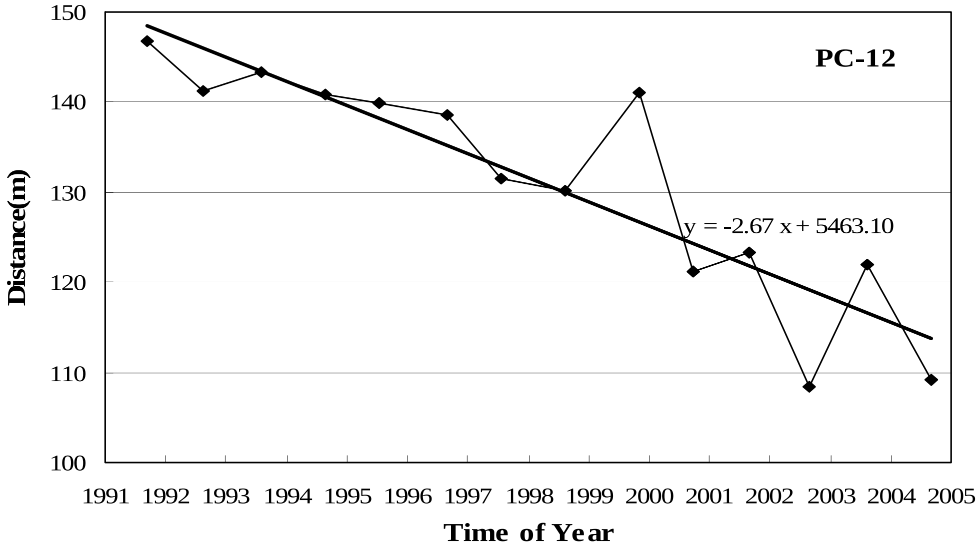

Fig. E.2.17: Regression line of shoreline distance versus time at PC-12

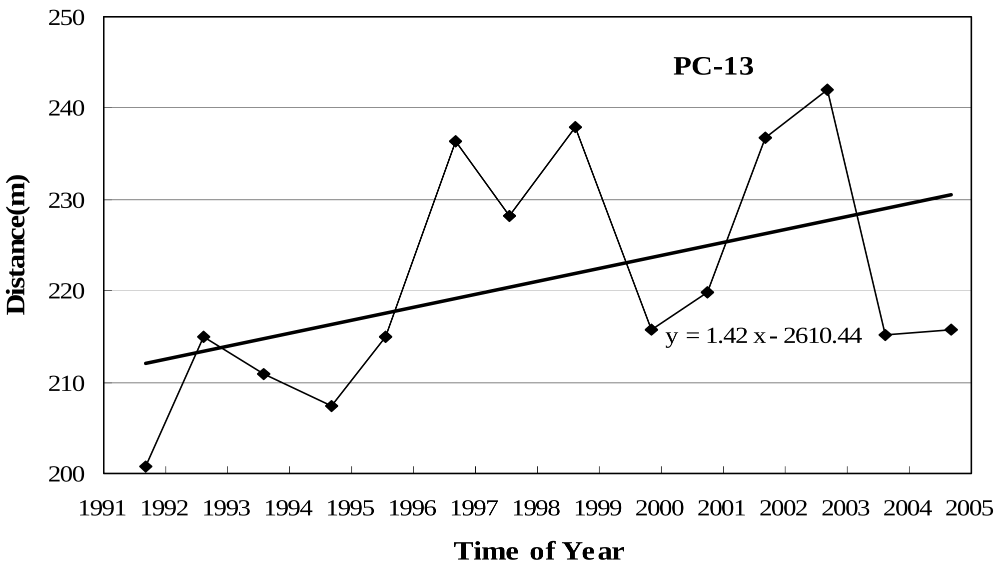

Fig. E.2.18: Regression line of shoreline distance versus time at PC-13

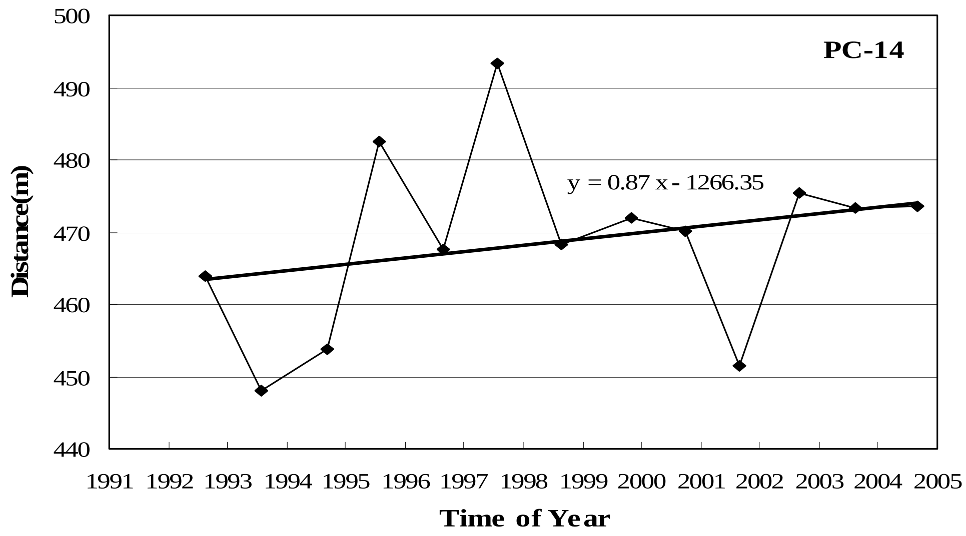

Fig. E.2.19: Regression line of shoreline distance versus time at PC-14

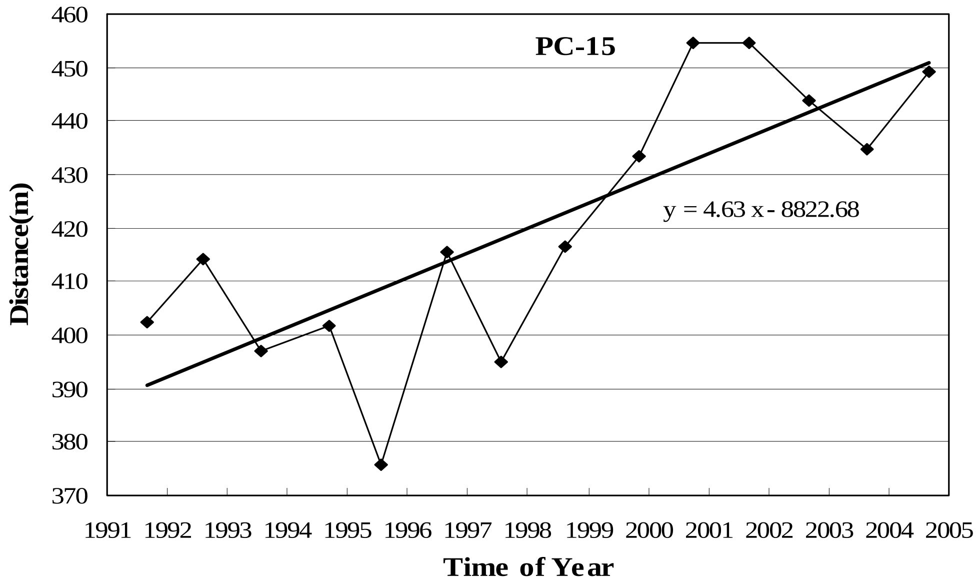

Fig. E.2.20: Regression line of shoreline distance versus time at PC-15

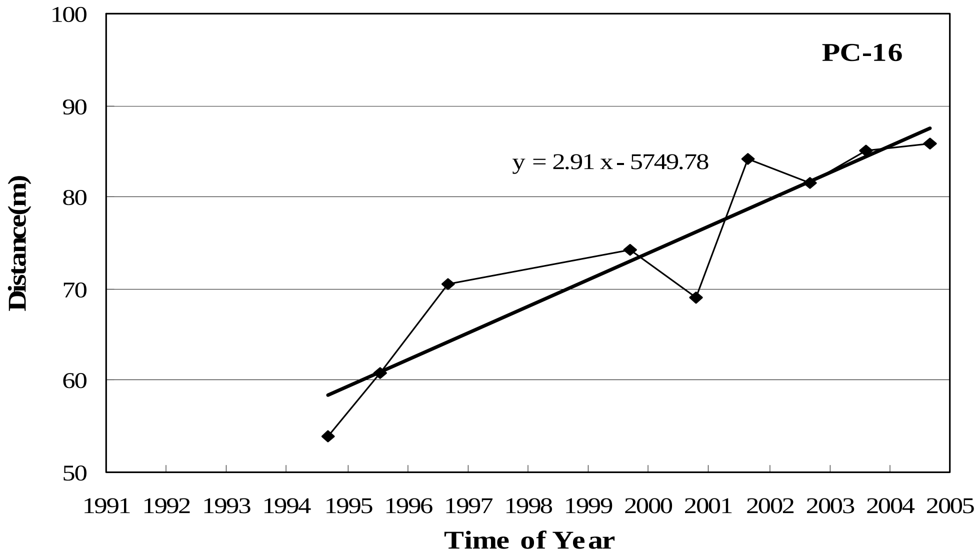

Fig. E.2.21: Regression line of shoreline distance versus time at PC-16

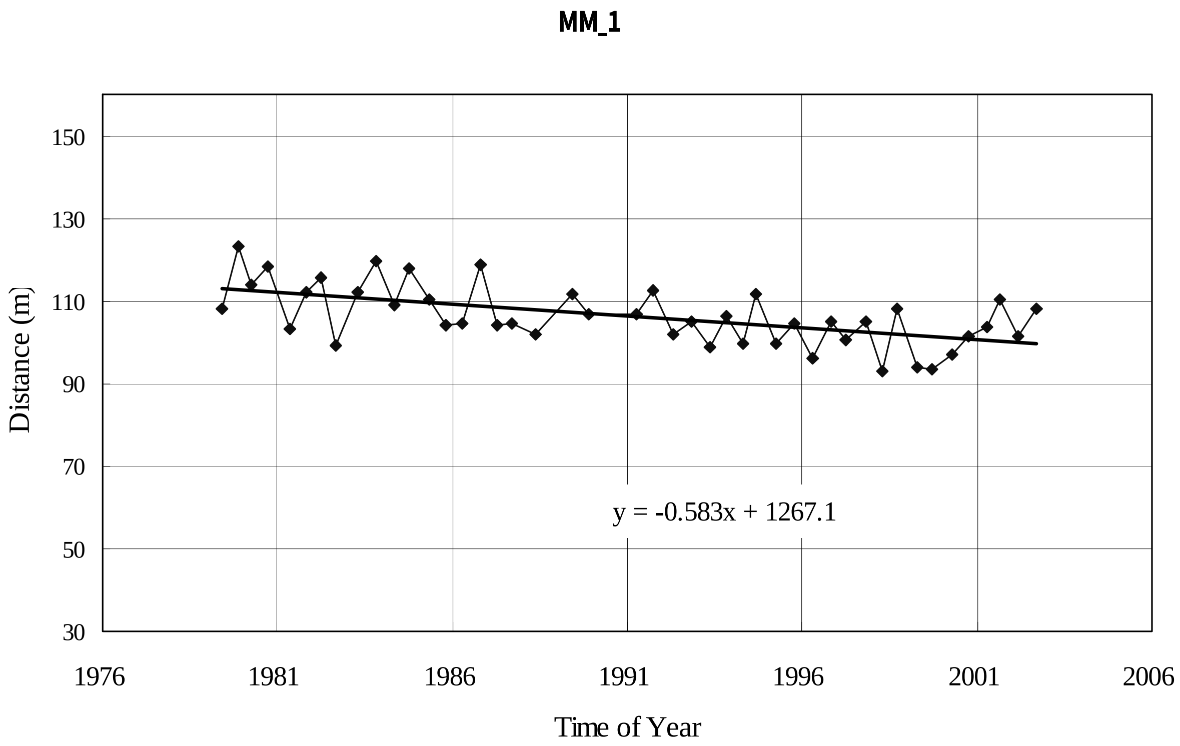

Fig. E.2.22: Regression line of shoreline distance versus time at MM-1

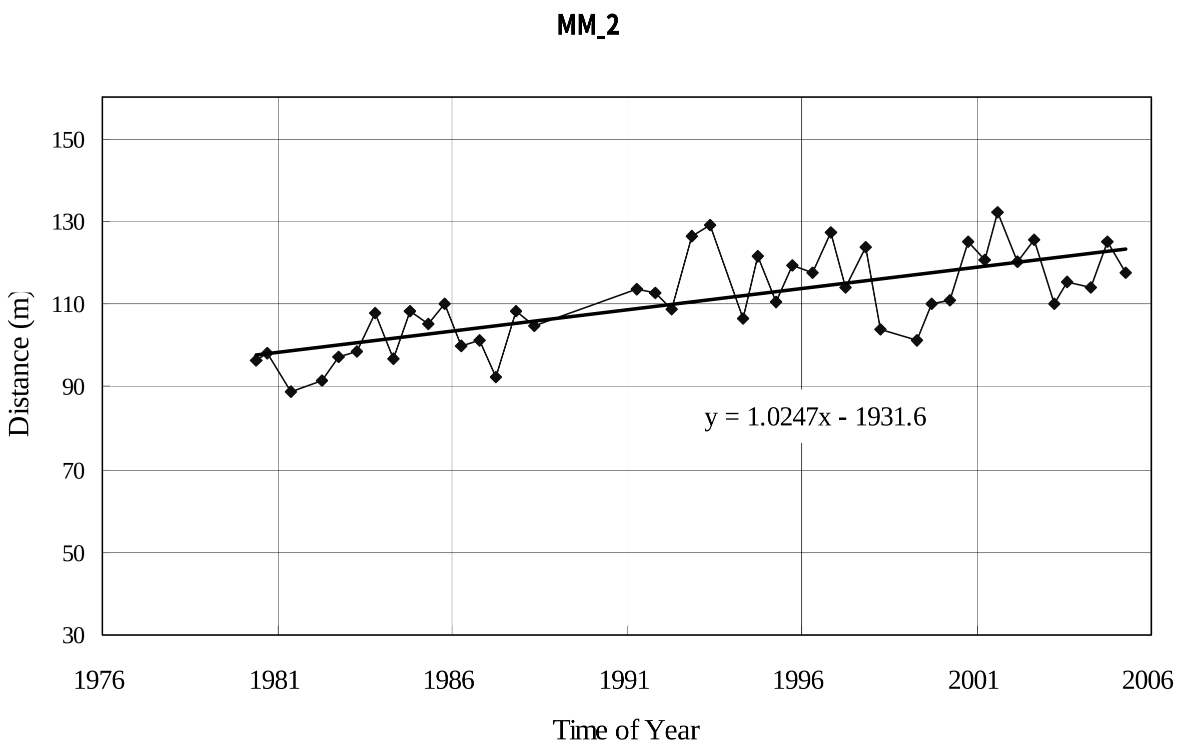

Fig. E.2.23: Regression line of shoreline distance versus time at MM-2

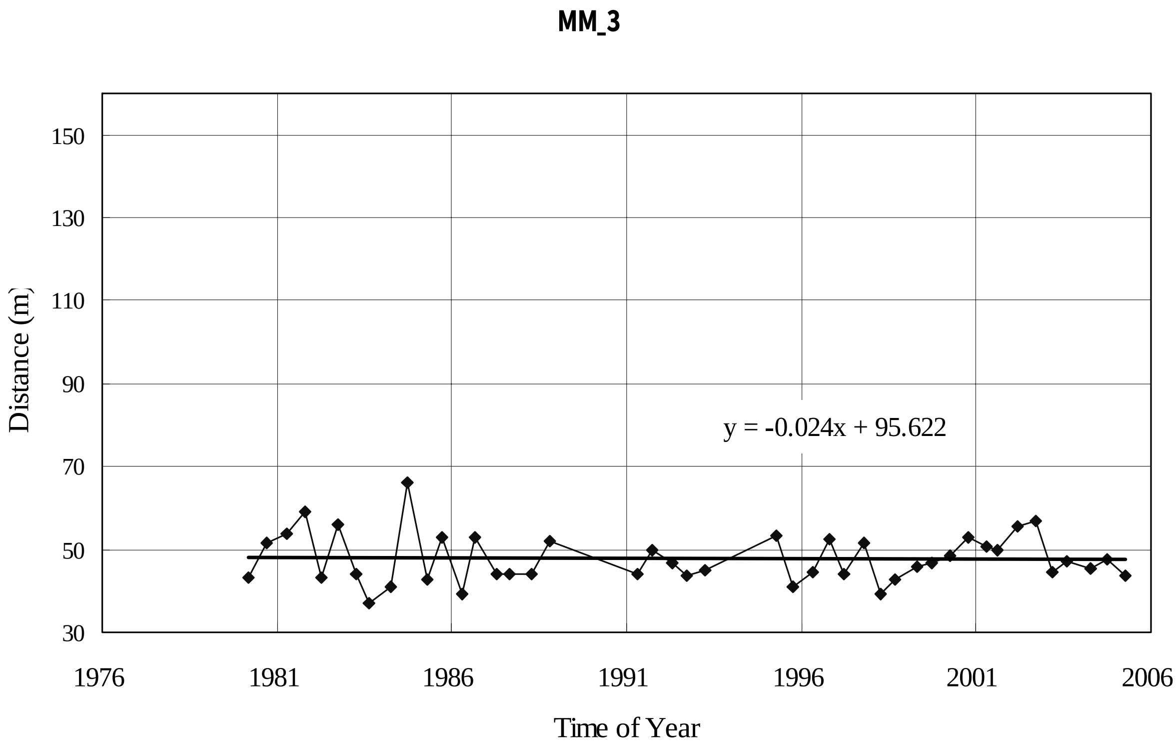

Fig. E.224: Regression line of shoreline distance versus time at MM-3

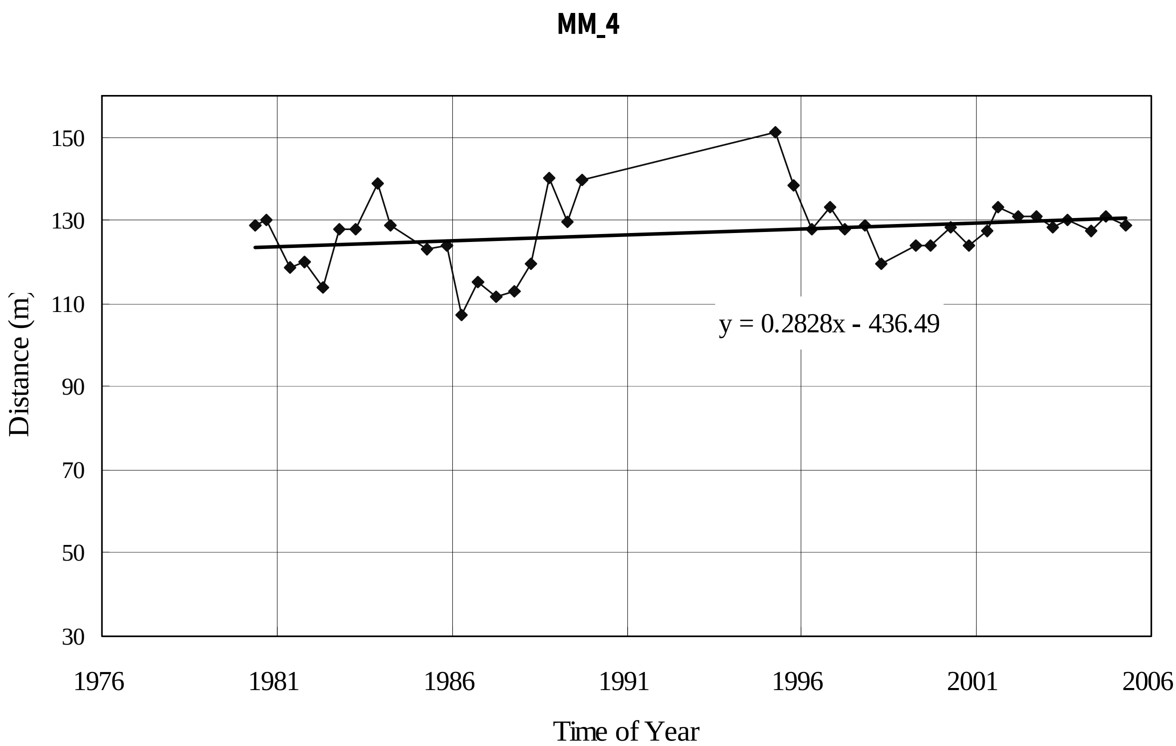

Fig. E.2.25: Regression line of shoreline distance versus time at MM-4

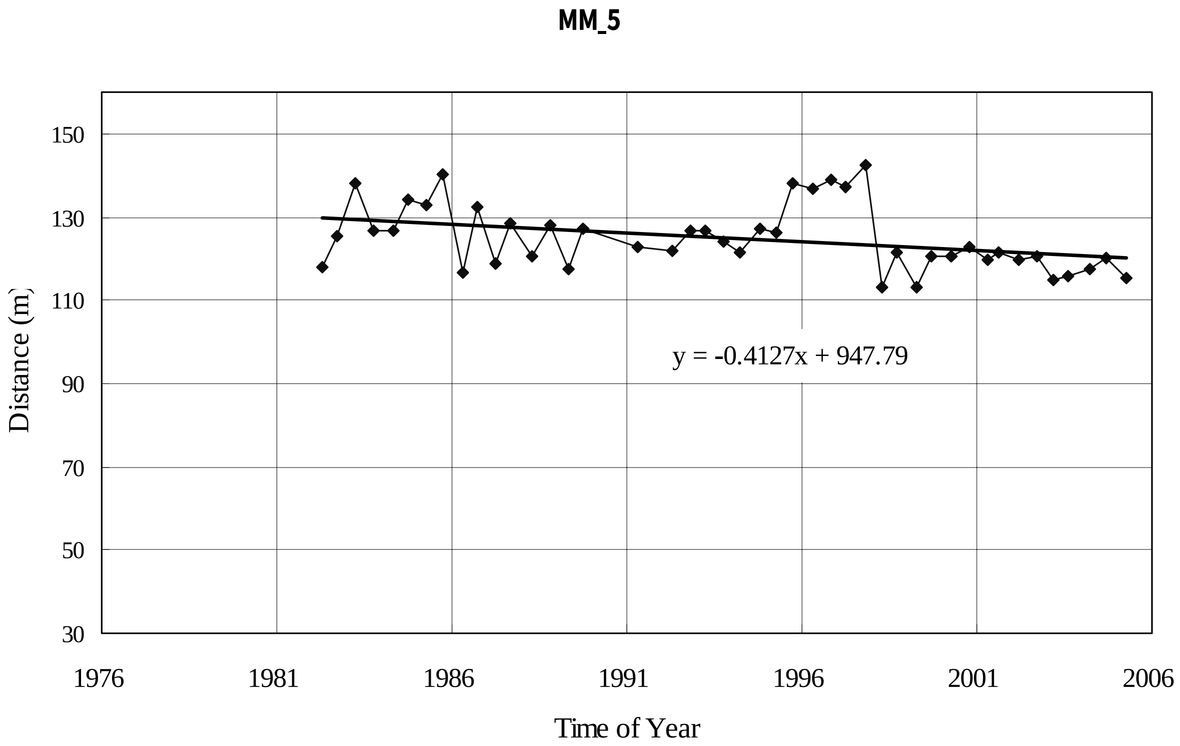

Fig. E.2.26: Regression line of shoreline distance versus time at MM-5

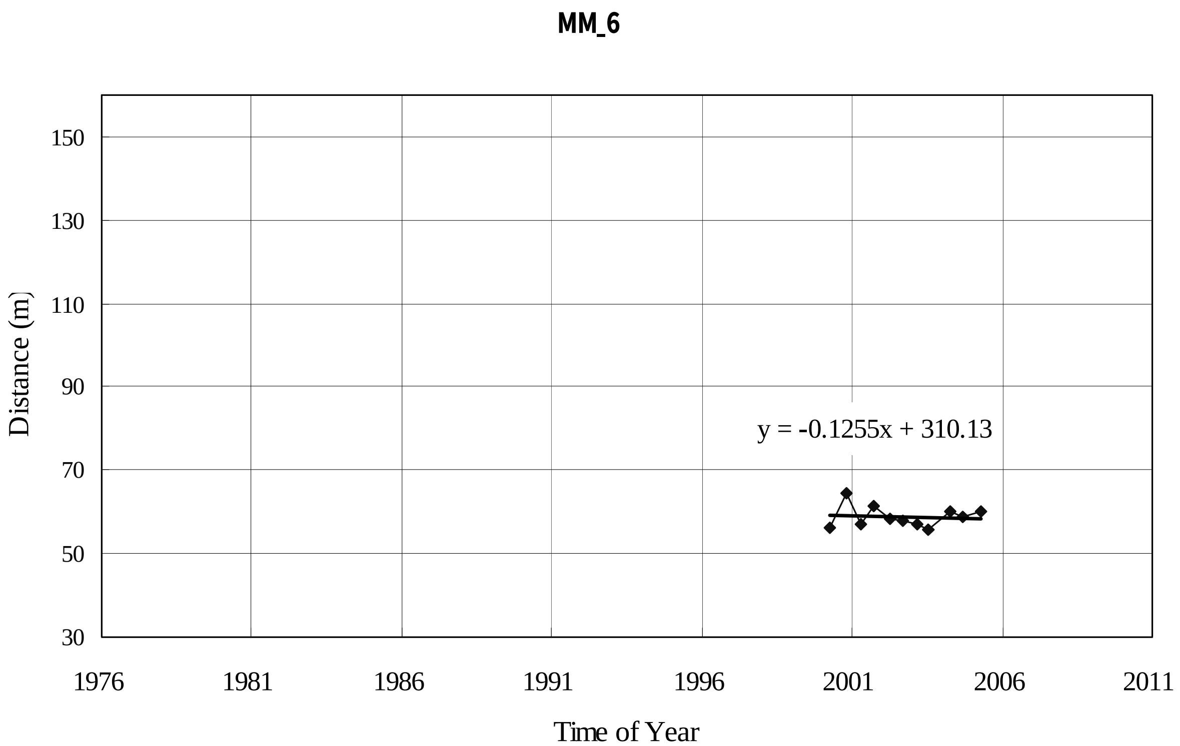

Fig. E.2.27: Regression line of shoreline distance versus time at MM-6

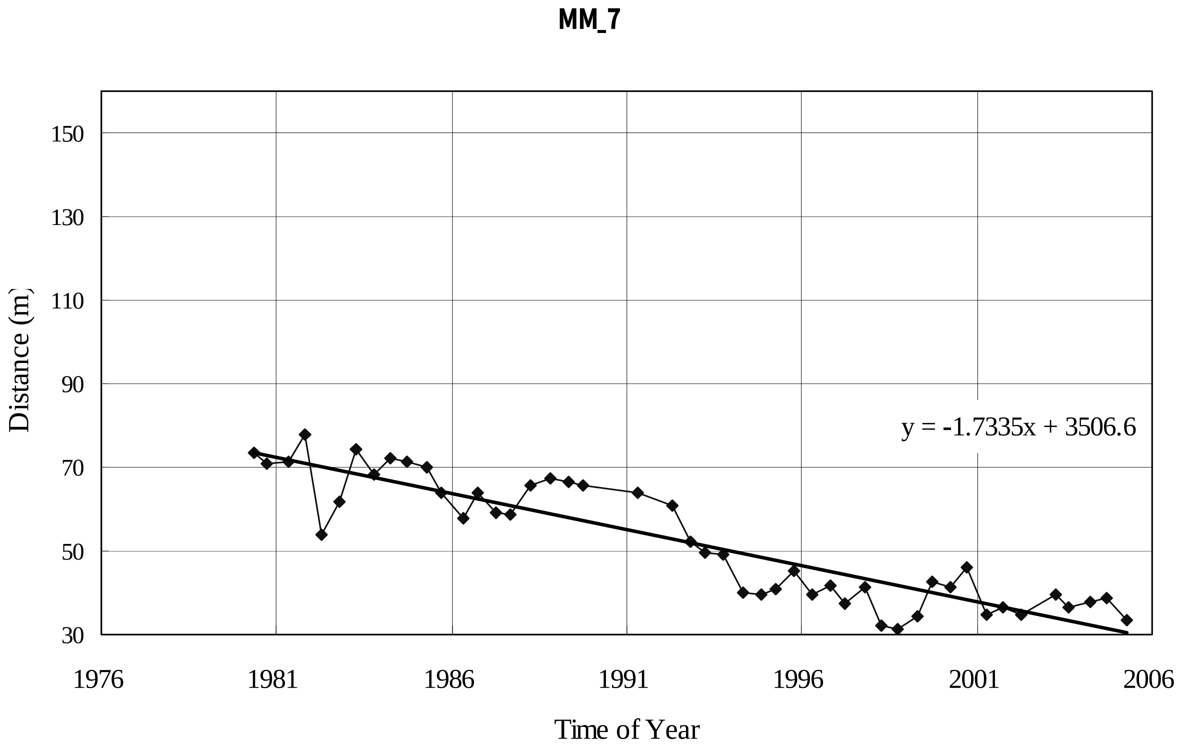

Fig. E.2.28: Regression line of shoreline distance versus time at MM-7

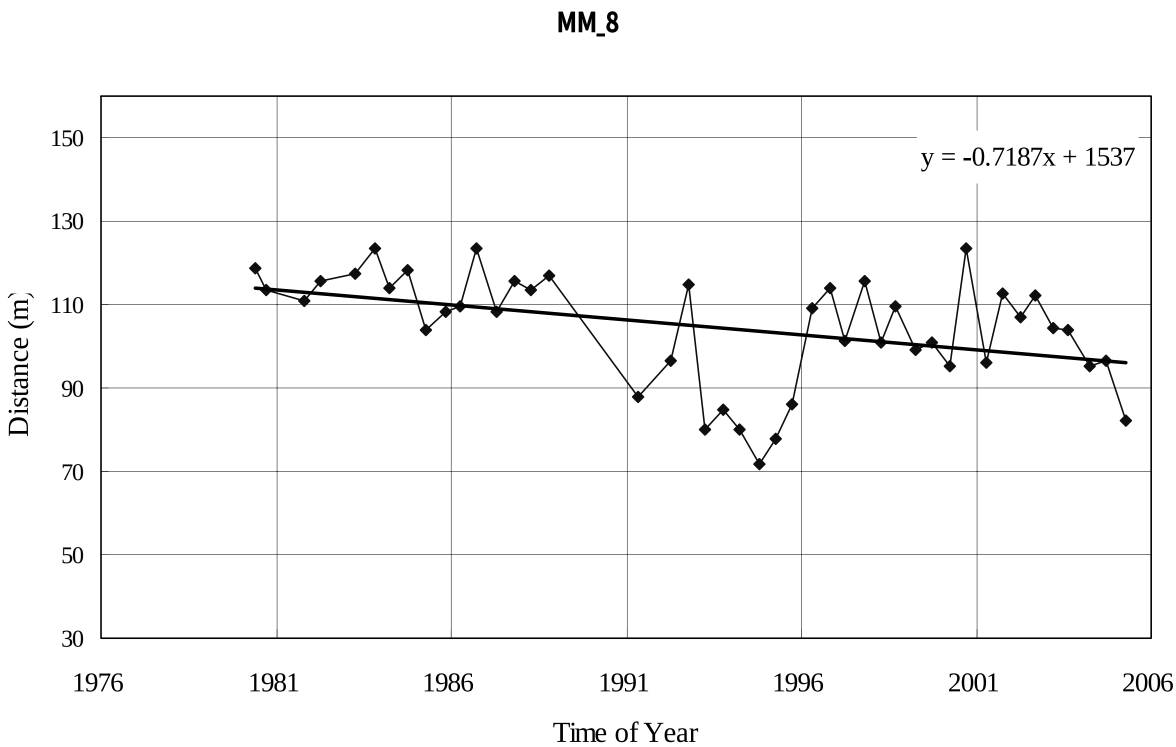

Fig. E.2.29: Regression line of shoreline distance versus time at MM-8

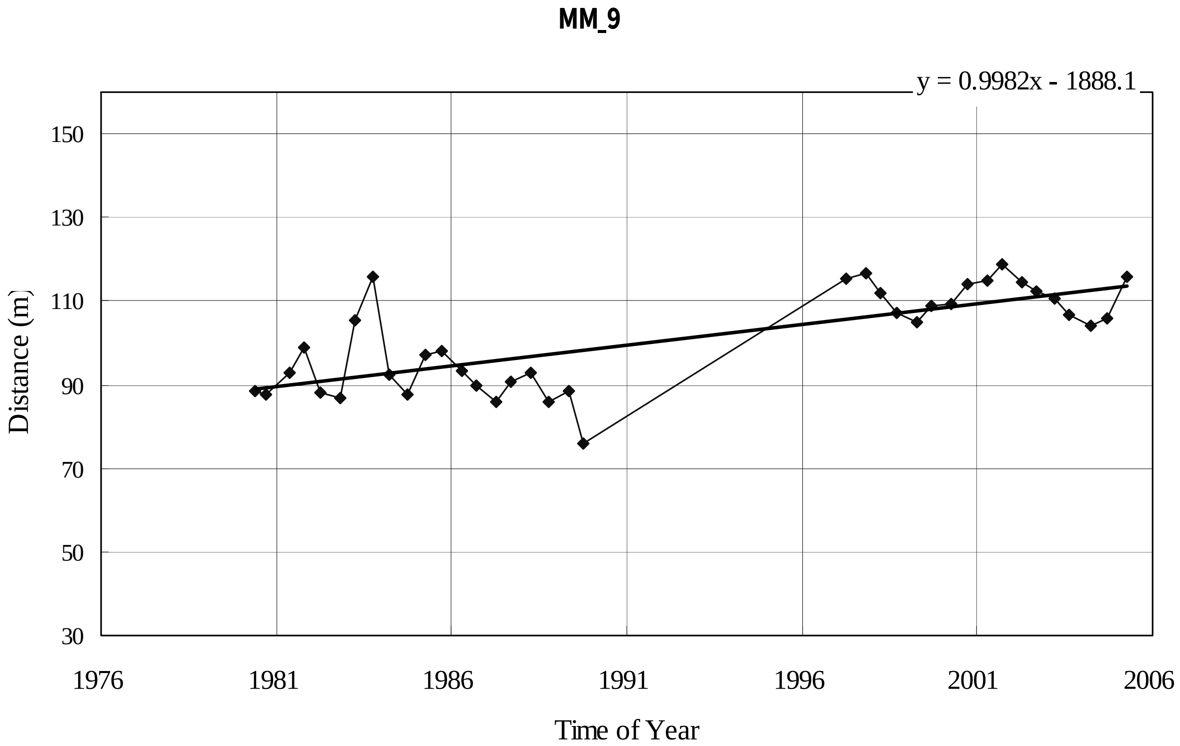

Fig. E.2.30: Regression line of shoreline distance versus time at MM-9

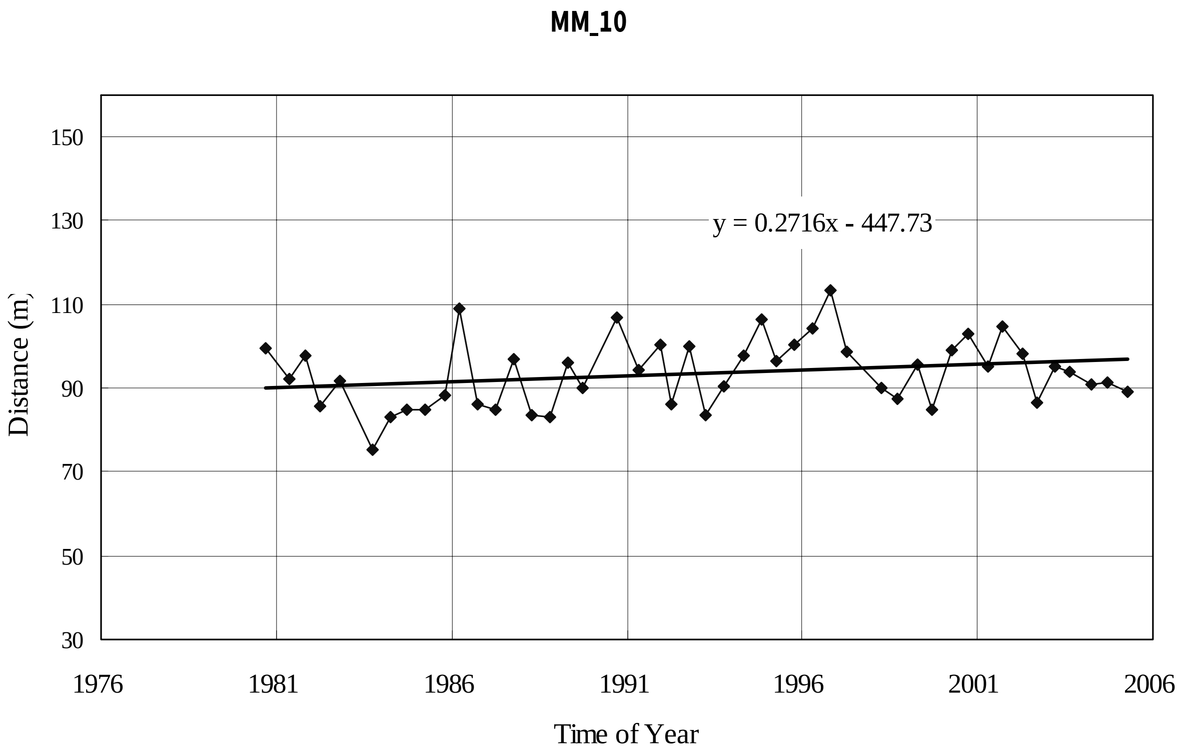

Fig. E.2.31: Regression line of shoreline distance versus time at MM-10

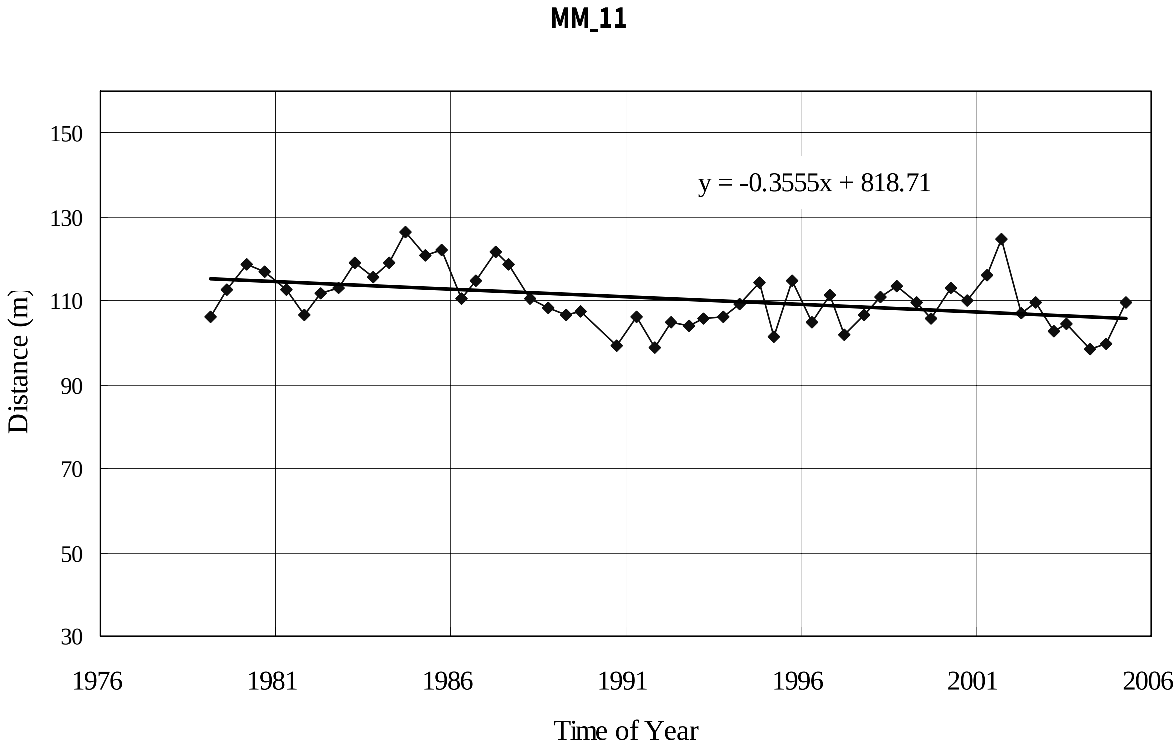

Fig. E.2.32: Regression line of shoreline distance versus time at MM-11

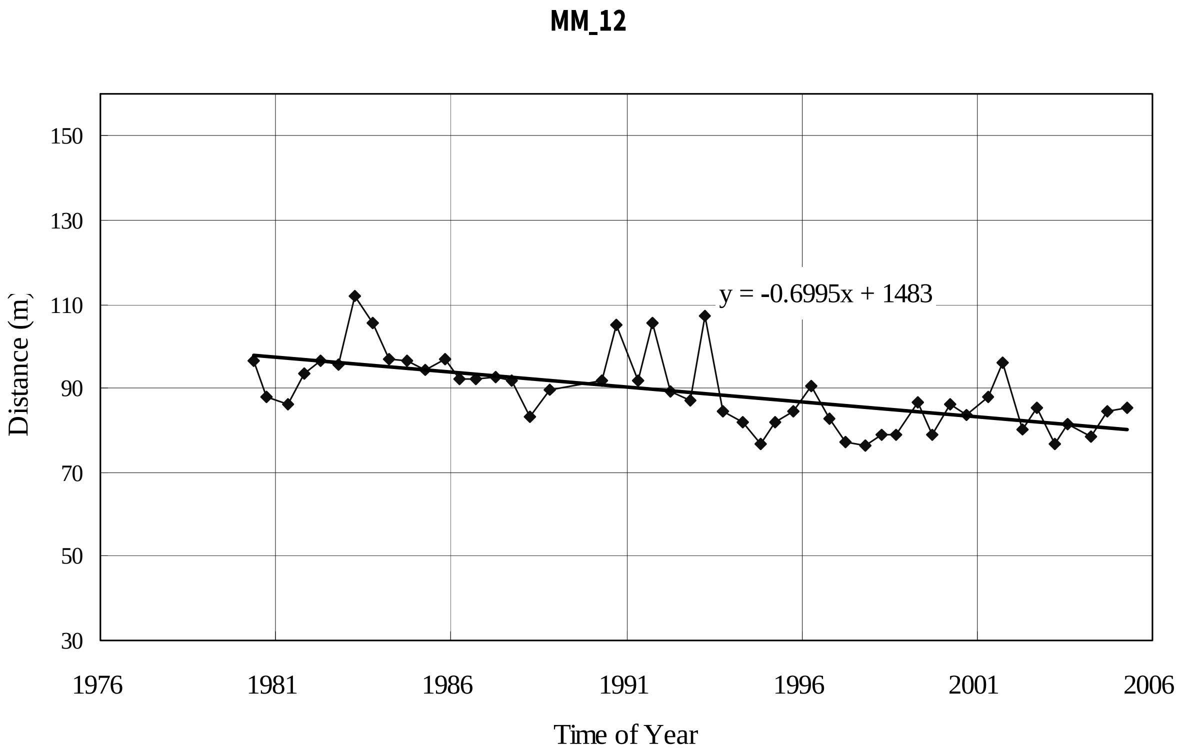

Fig. E.2.33: Regression line of shoreline distance versus time at MM-12

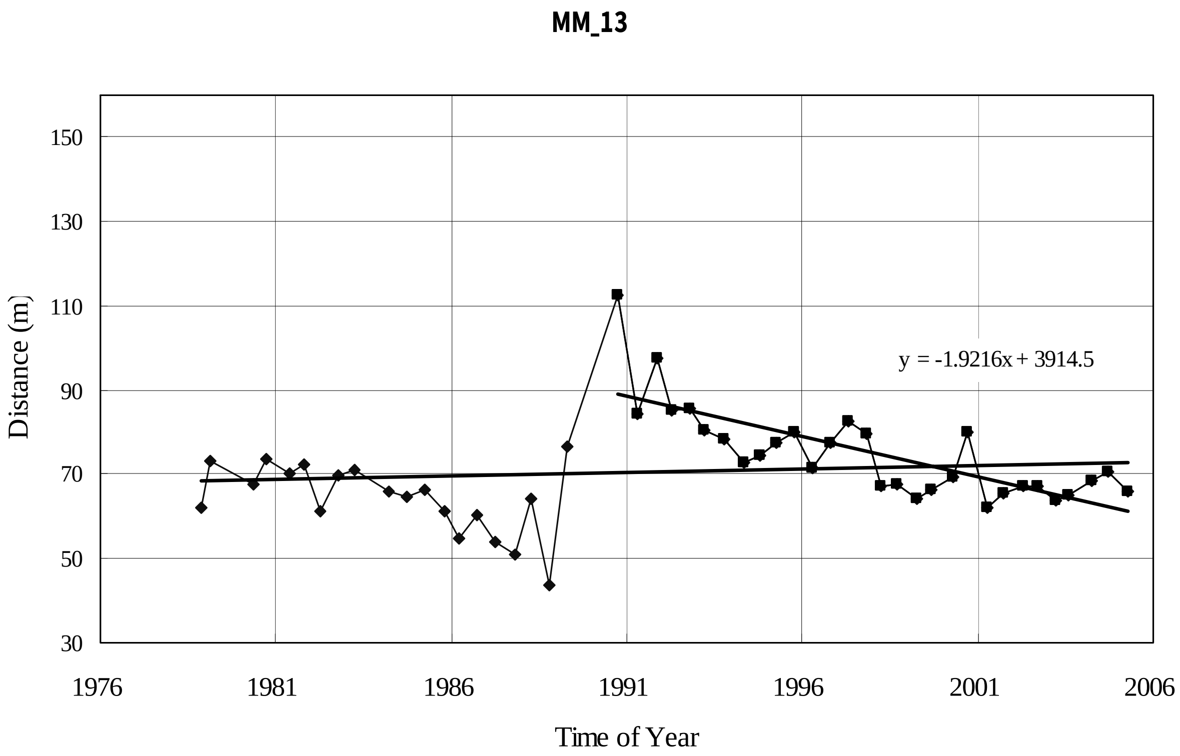

Fig. E.2.34: Regression line of shoreline distance versus time at MM-13

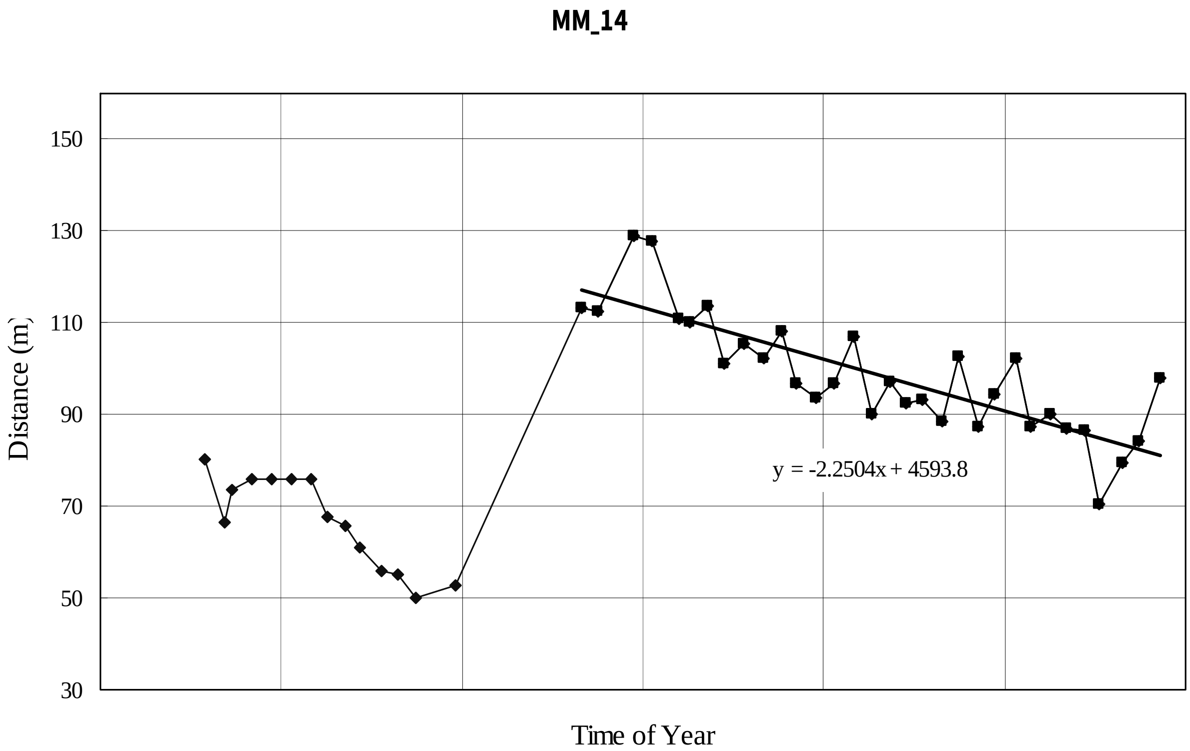

Fig. E.2.35: Regression line of shoreline distance versus time at MM-14

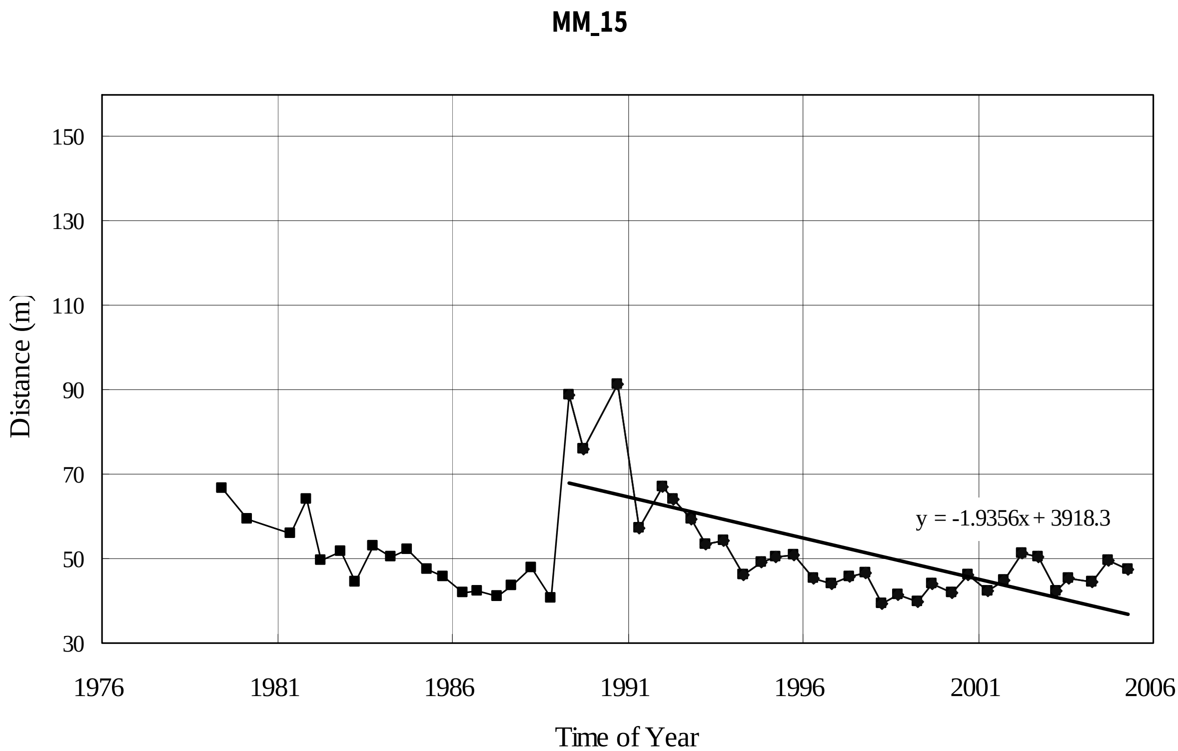

Fig. E.2.36: Regression line of shoreline distance versus time at MM-15

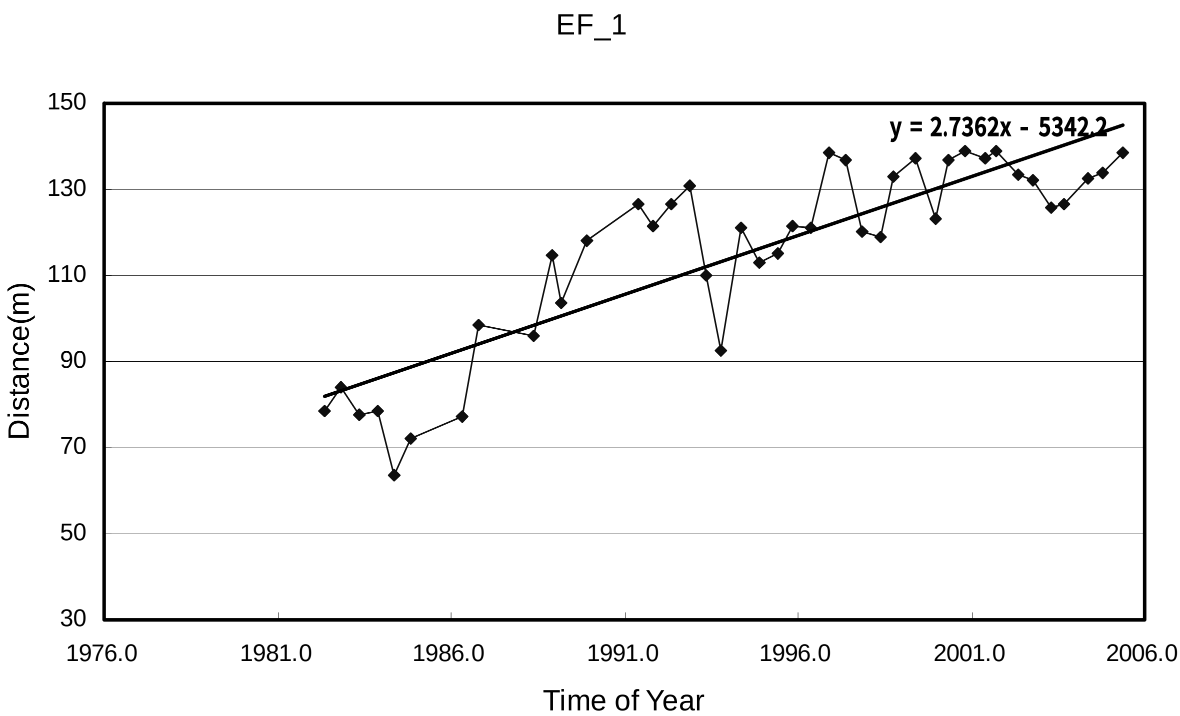

Fig. E.237: Regression line of shoreline distance versus time at EF-1

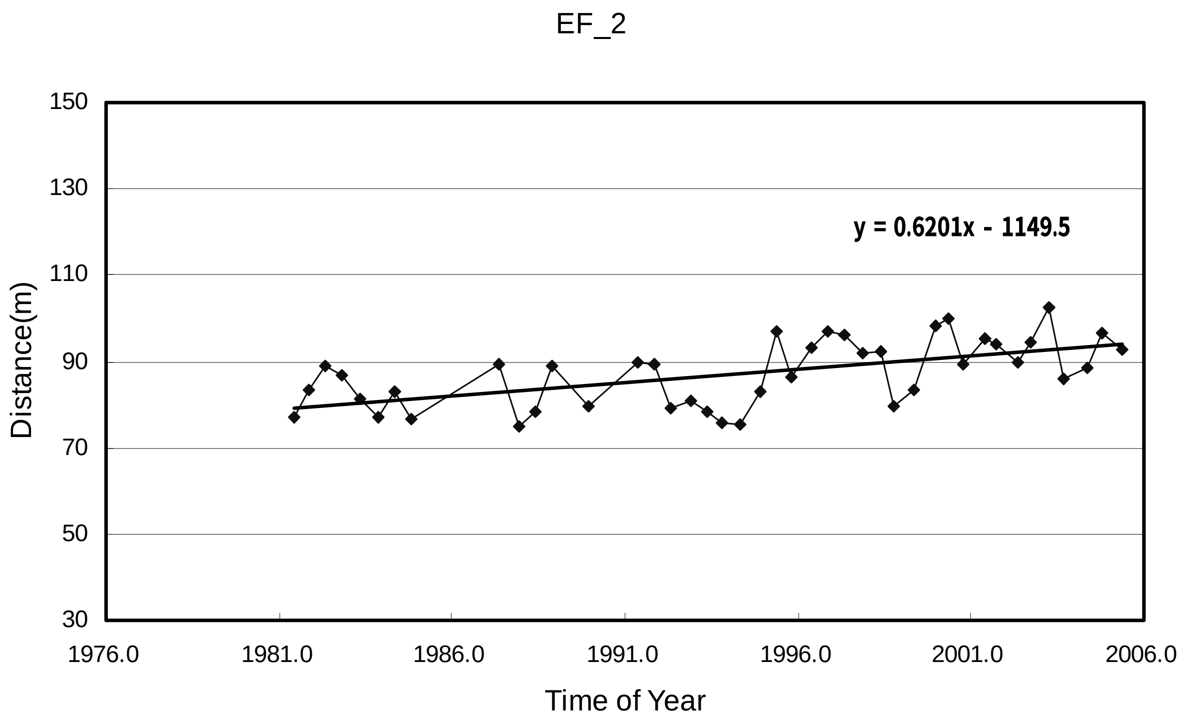

Fig. E.2.38: Regression line of shoreline distance versus time at EF-2

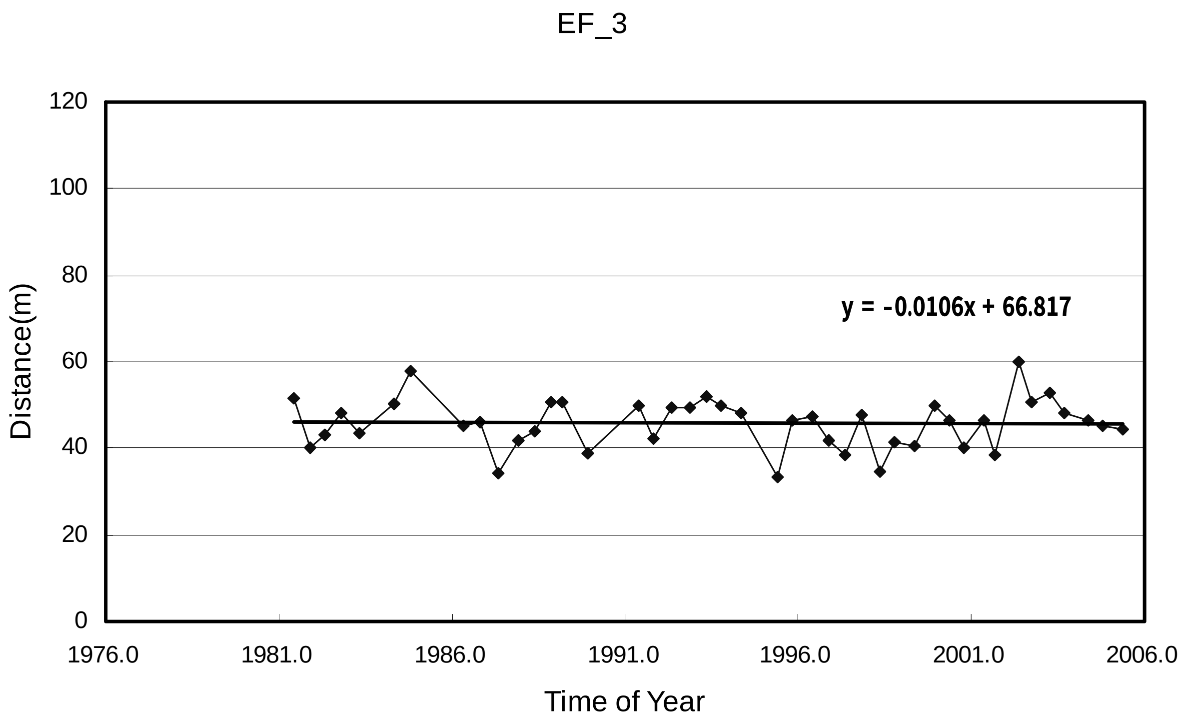

Fig. E.2.39: Regression line of shoreline distance versus time at EF-3

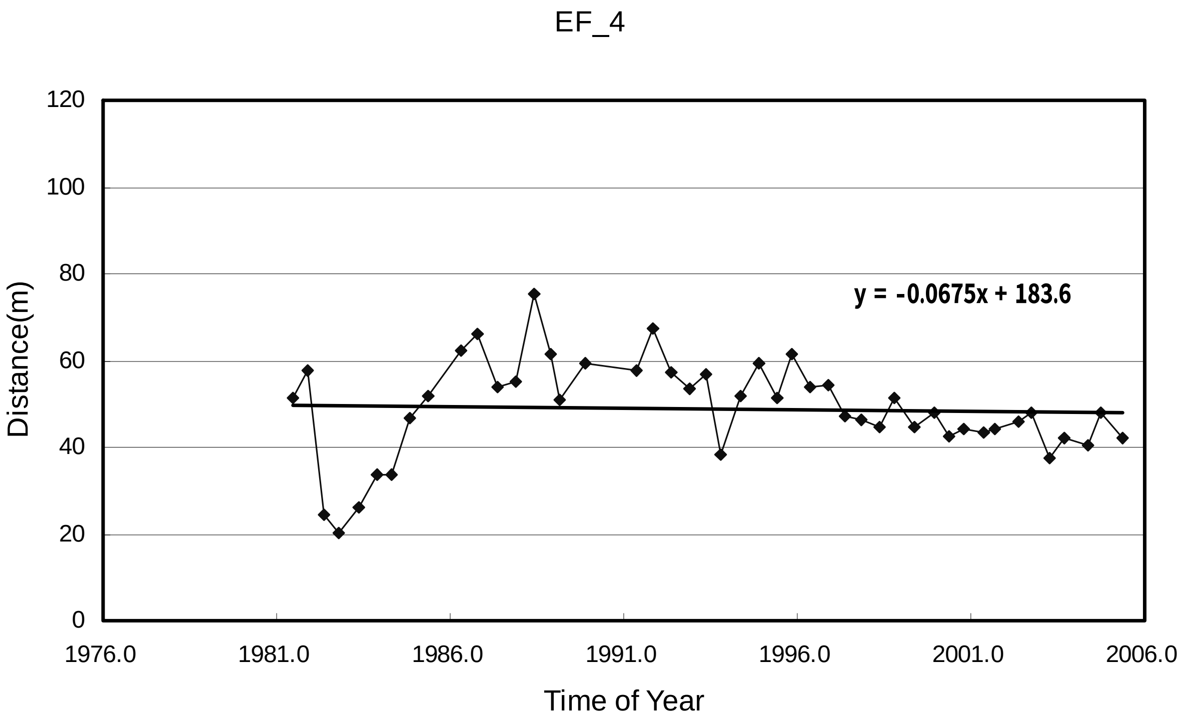

Fig. E.2.40: Regression line of shoreline distance versus time at EF-4

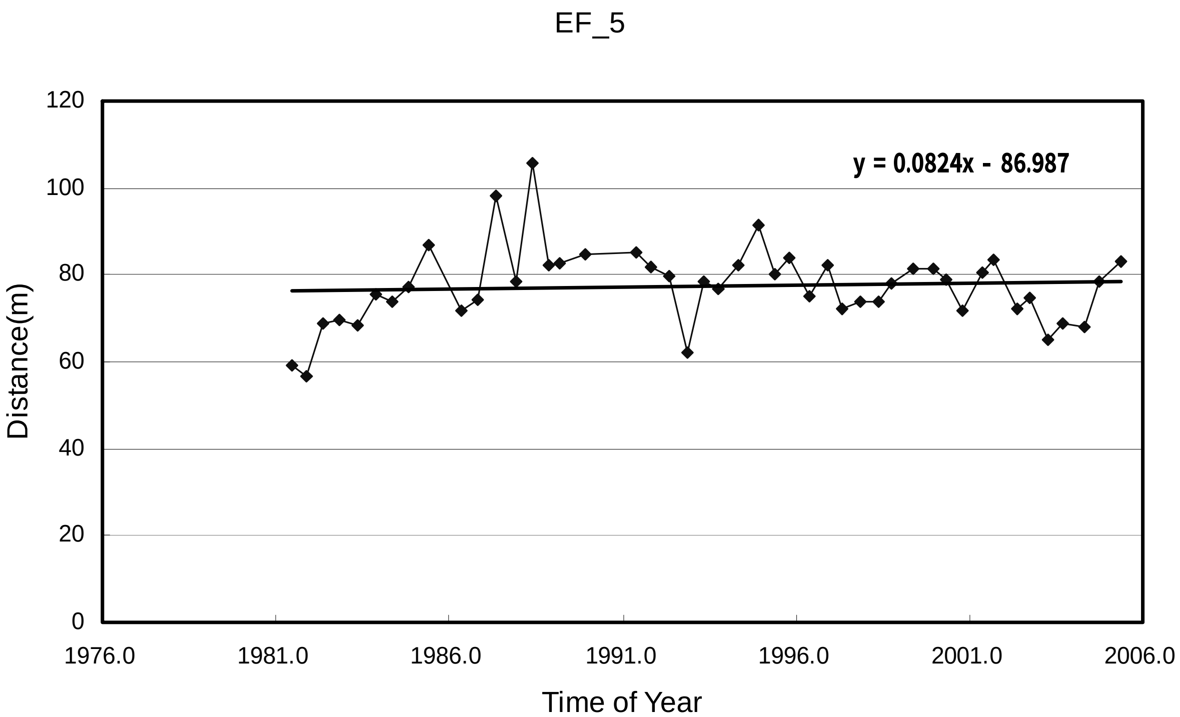

Fig. E.2.41: Regression line of shoreline distance versus time at EF-5

EF_6

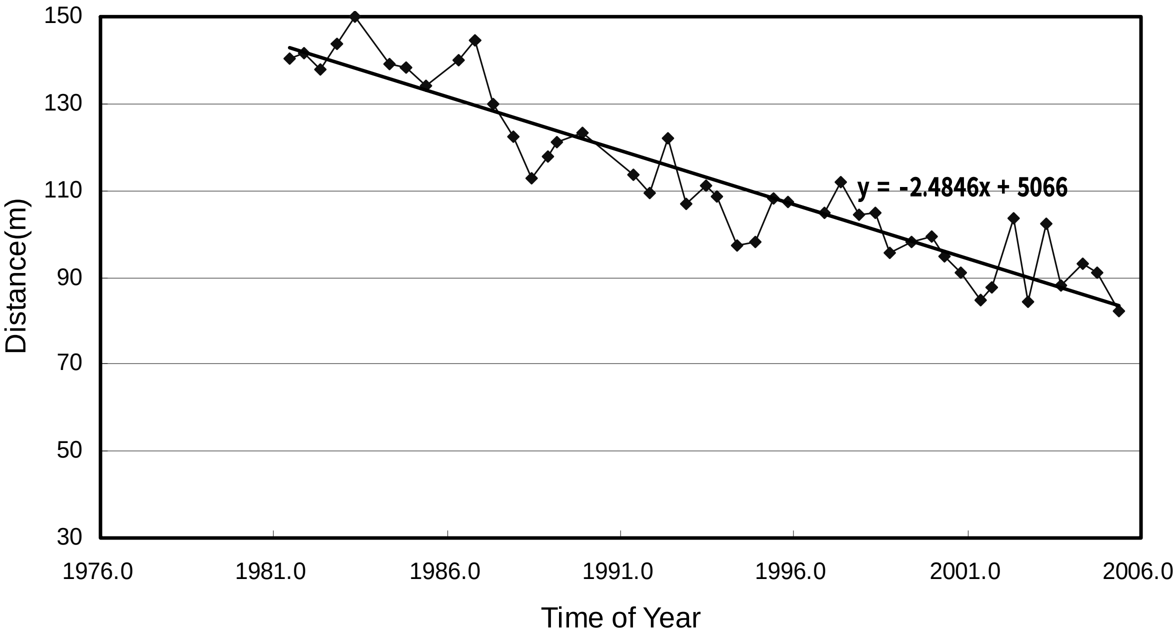

Fig. E.2.42: Regression line of shoreline distance versus time at EF-6

EF_7

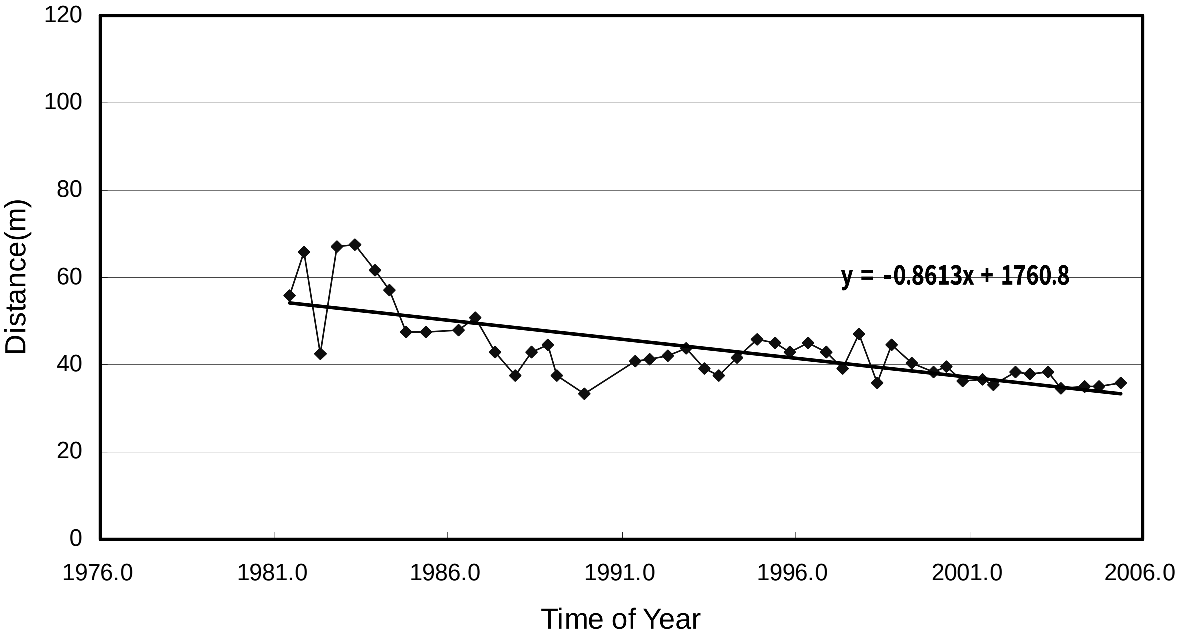

Fig. E.2.43: Regression line of shoreline distance versus time at EF-7

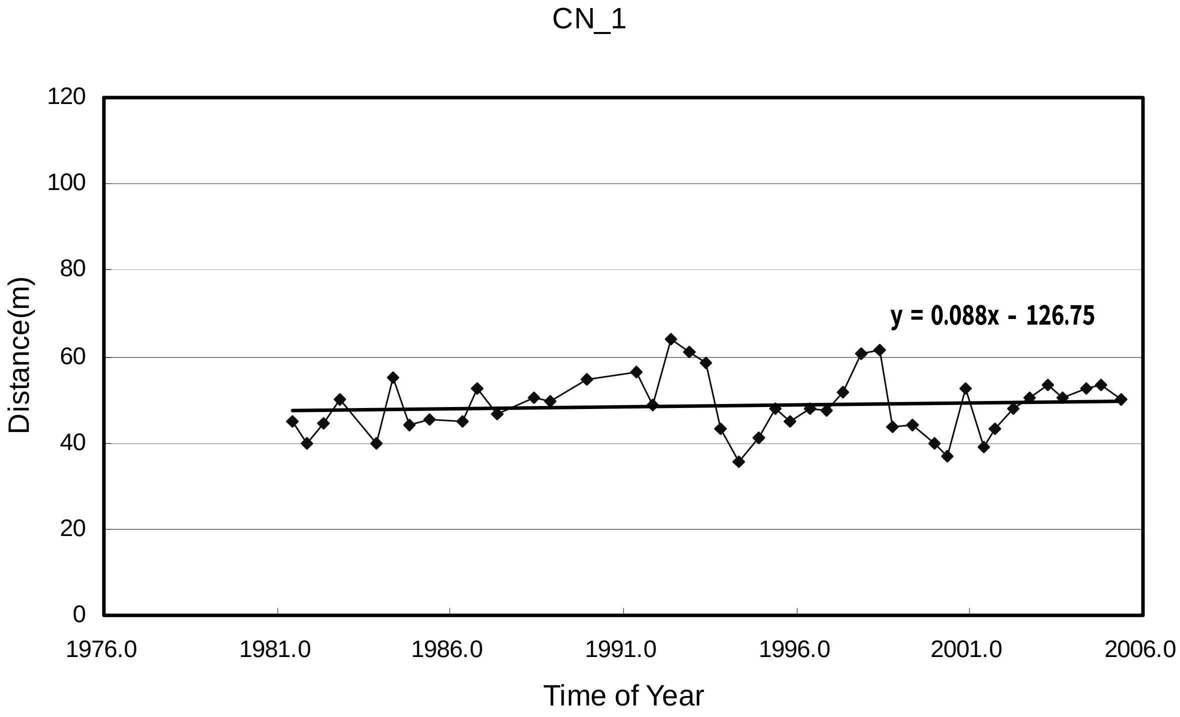

Fig. E.2.44: Regression line of shoreline distance versus time at CN-1

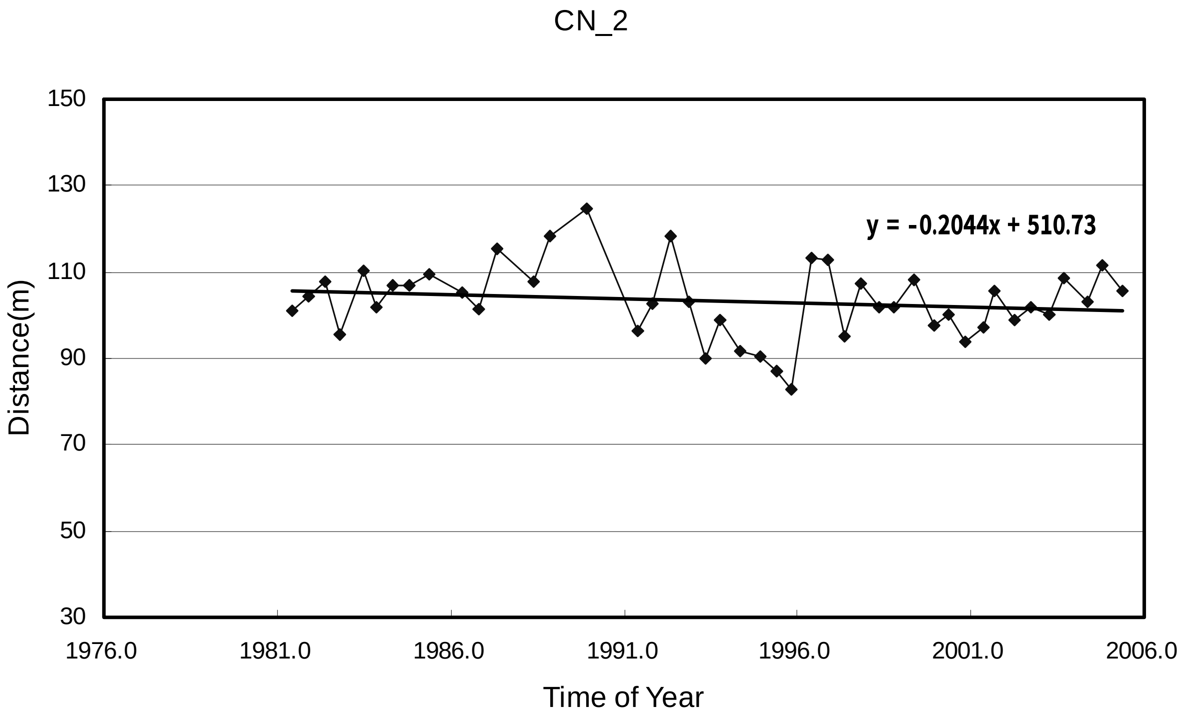

Fig. E.2.45: Regression line of shoreline distance versus time at CN-2

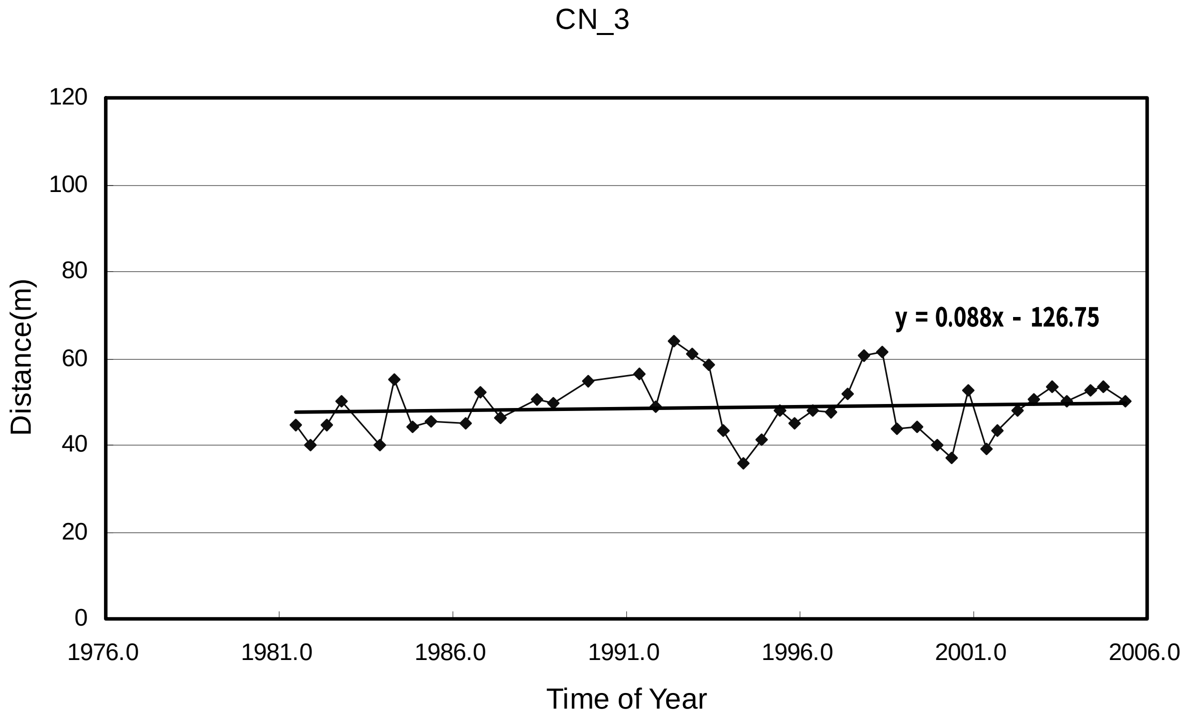

Fig. E.2.46: Regression line of shoreline distance versus time at CN-3

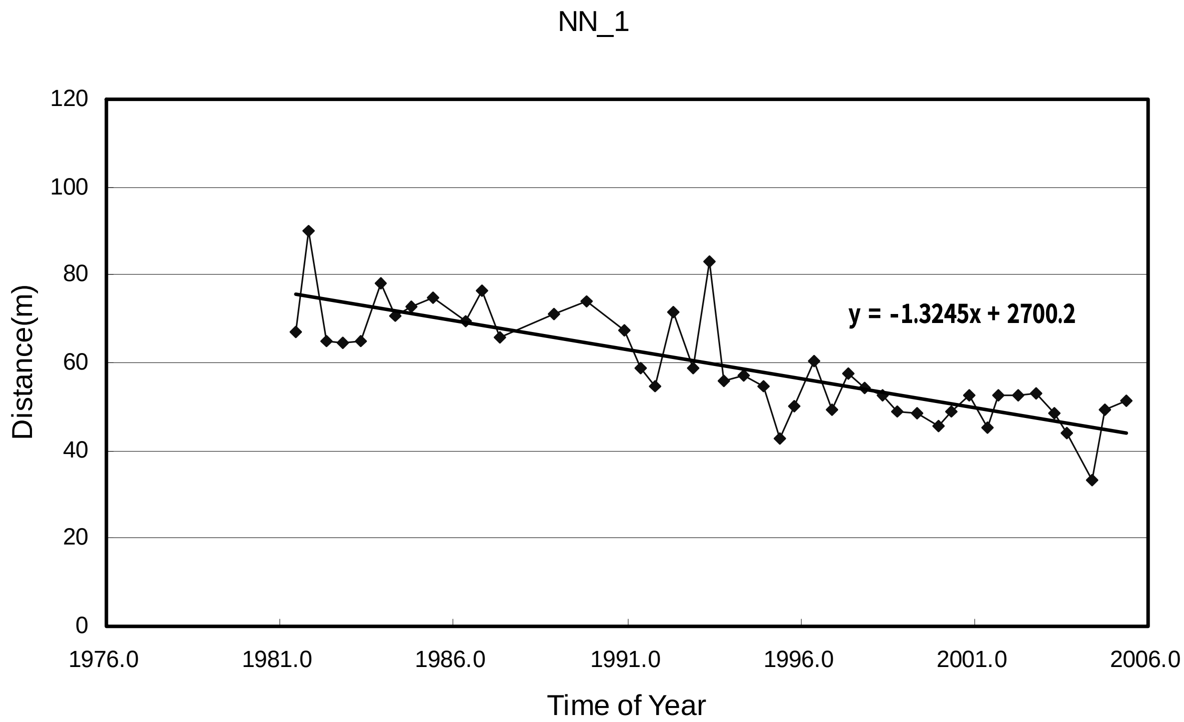

Fig. E.2.47: Regression line of shoreline distance versus time at NN-1

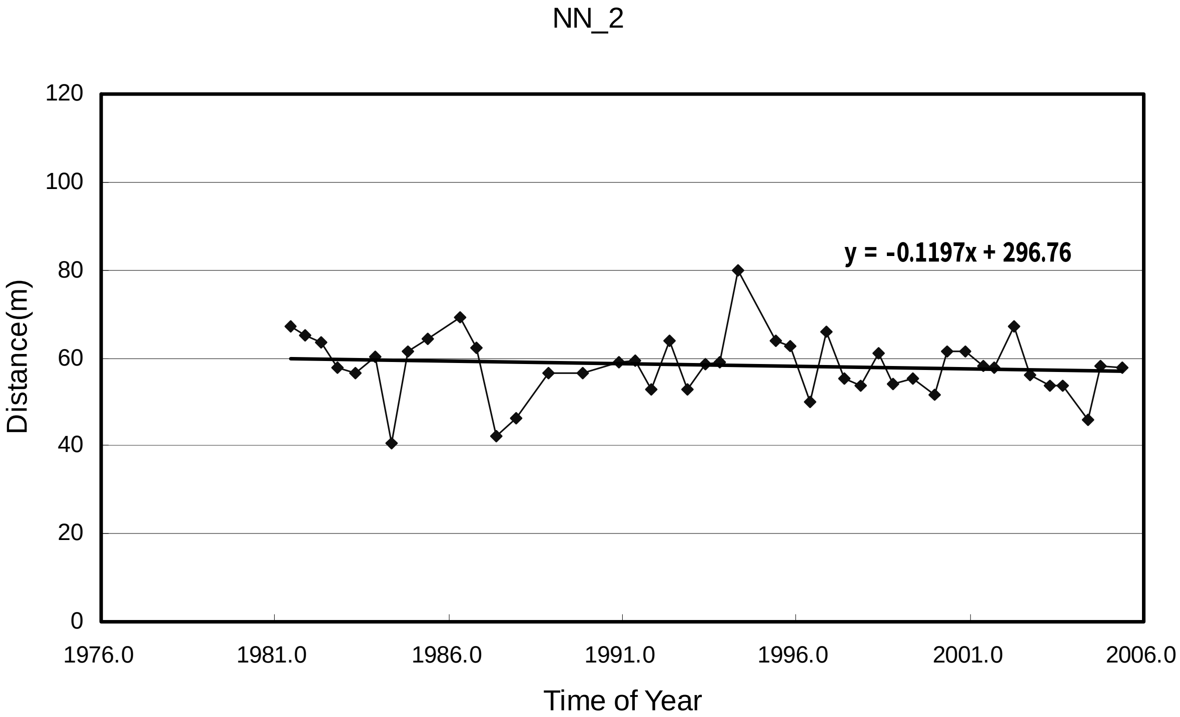

Fig. E.2.48: Regression line of shoreline distance versus time at NN-2

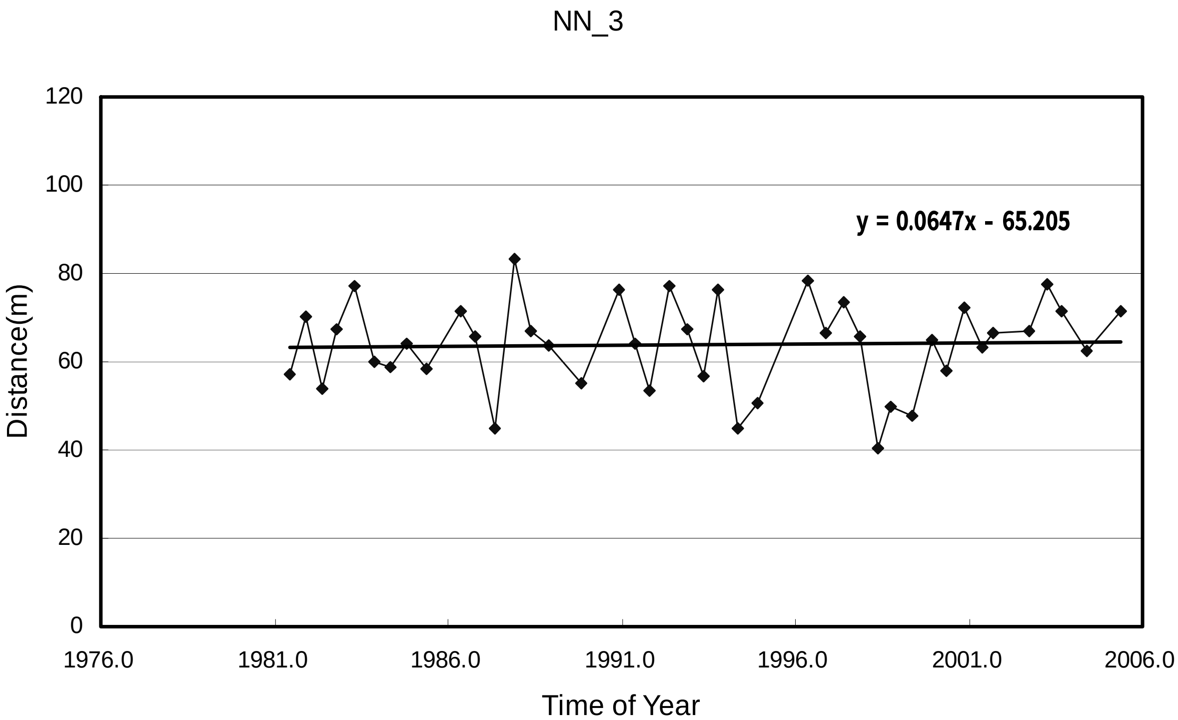

Fig. E.2.49: Regression line of shoreline distance versus time at NN-3

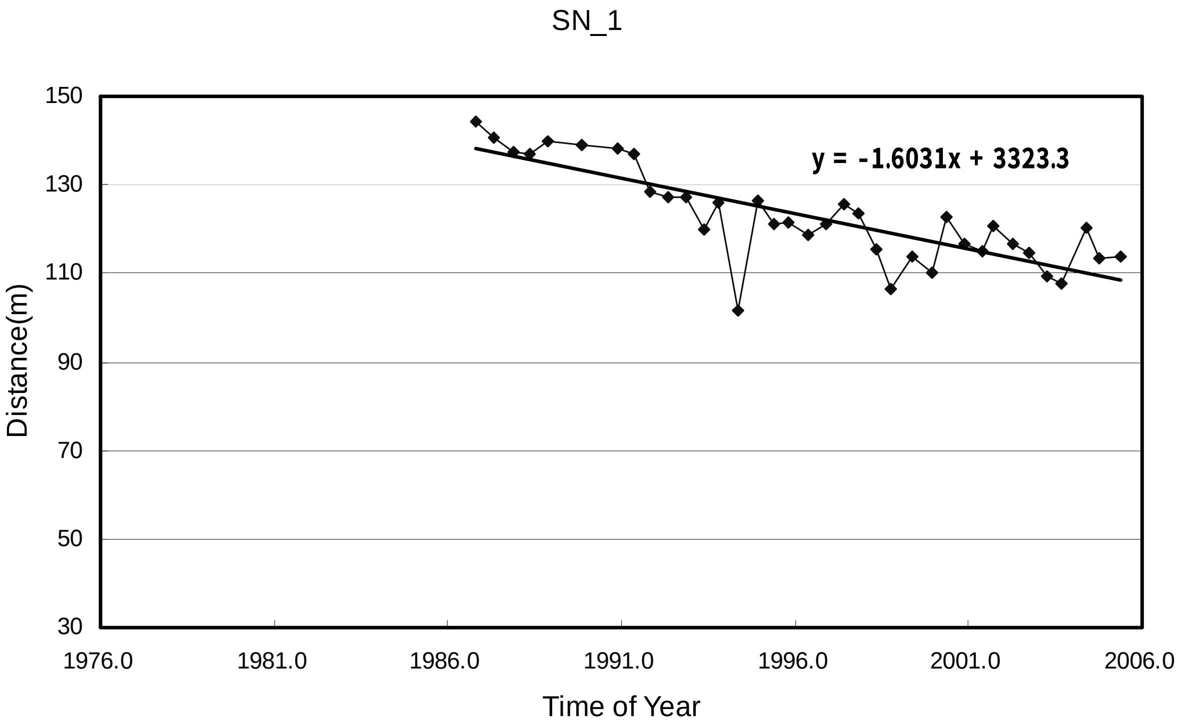

Fig. E.2.50: Regression line of shoreline distance versus time at SN-1

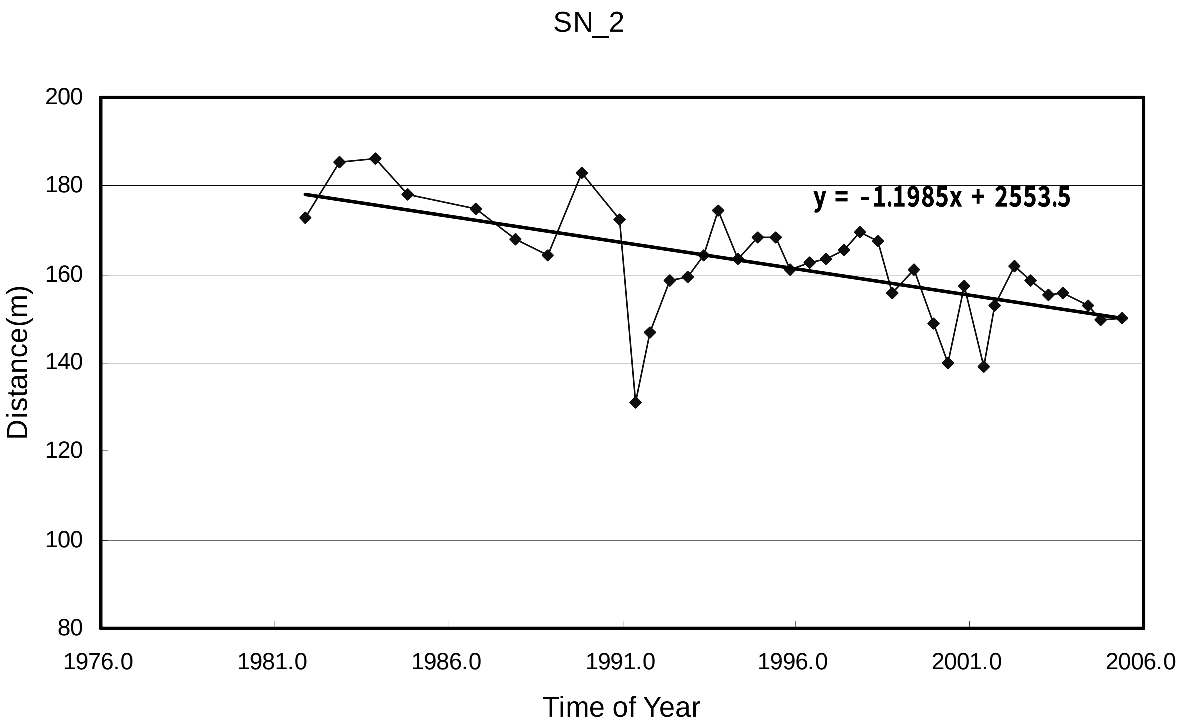

Fig. E.2.51: Regression line of shoreline distance versus time at SN-2

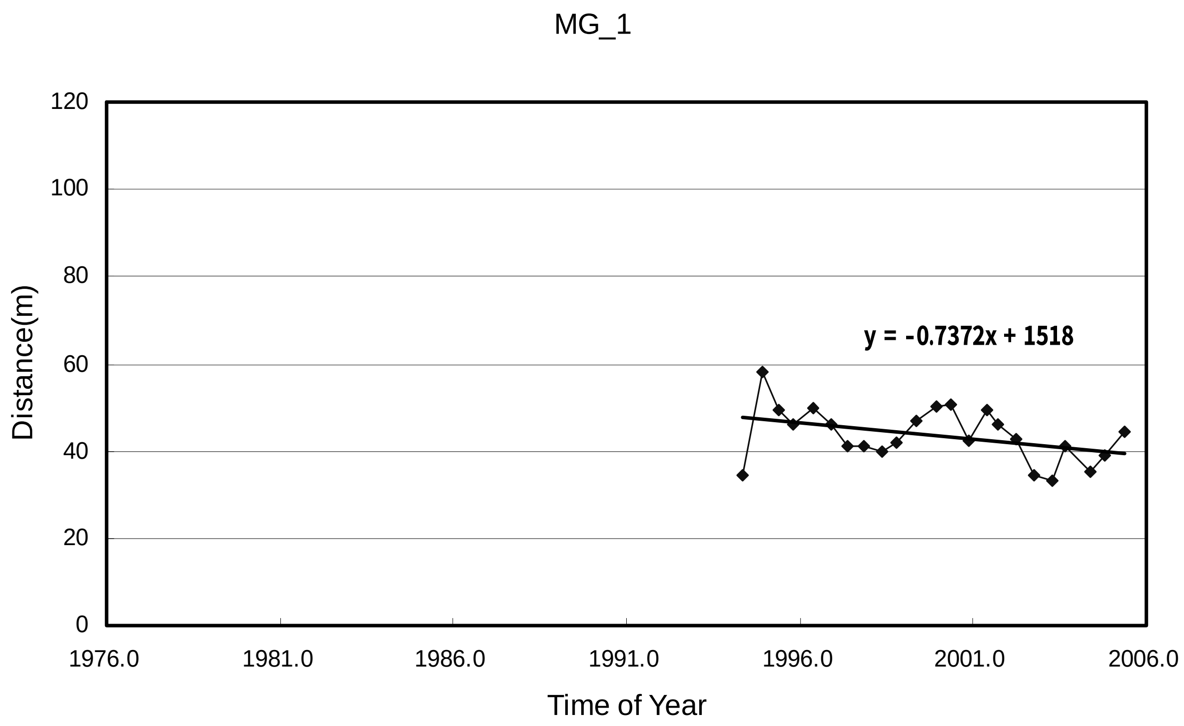

Fig. E.2.52: Regression line of shoreline distance versus time at MG-1

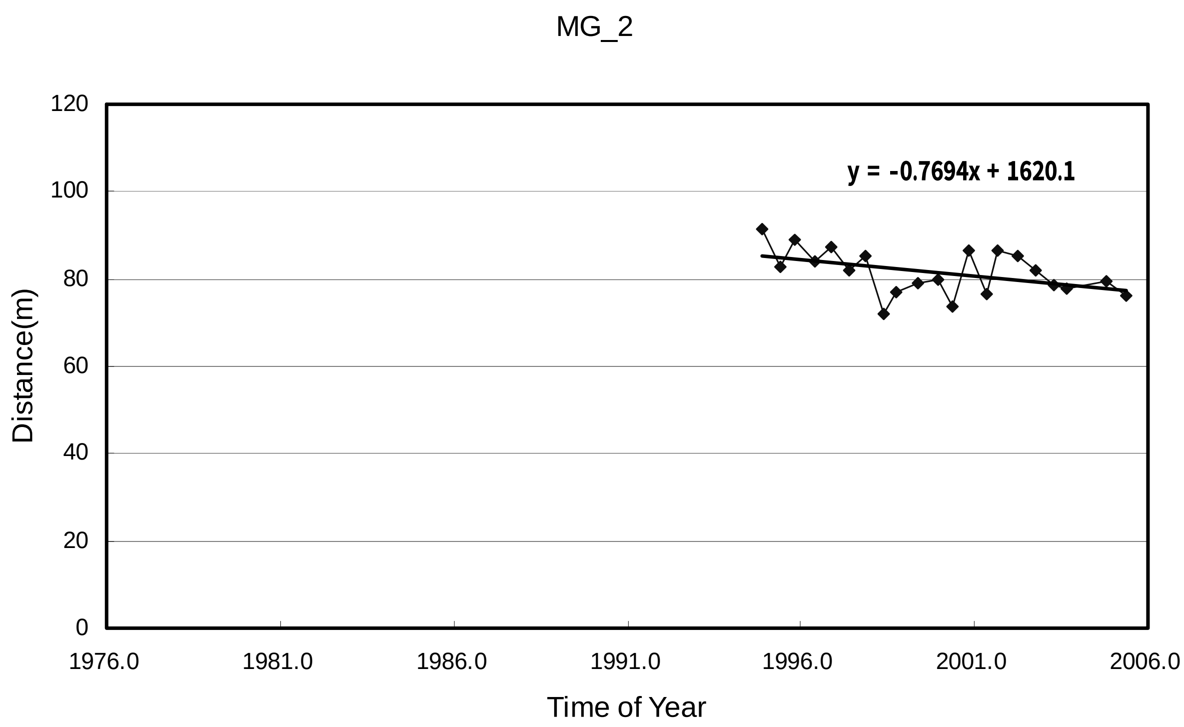

Fig. E.2.53: Regression line of shoreline distance versus time at MG-2

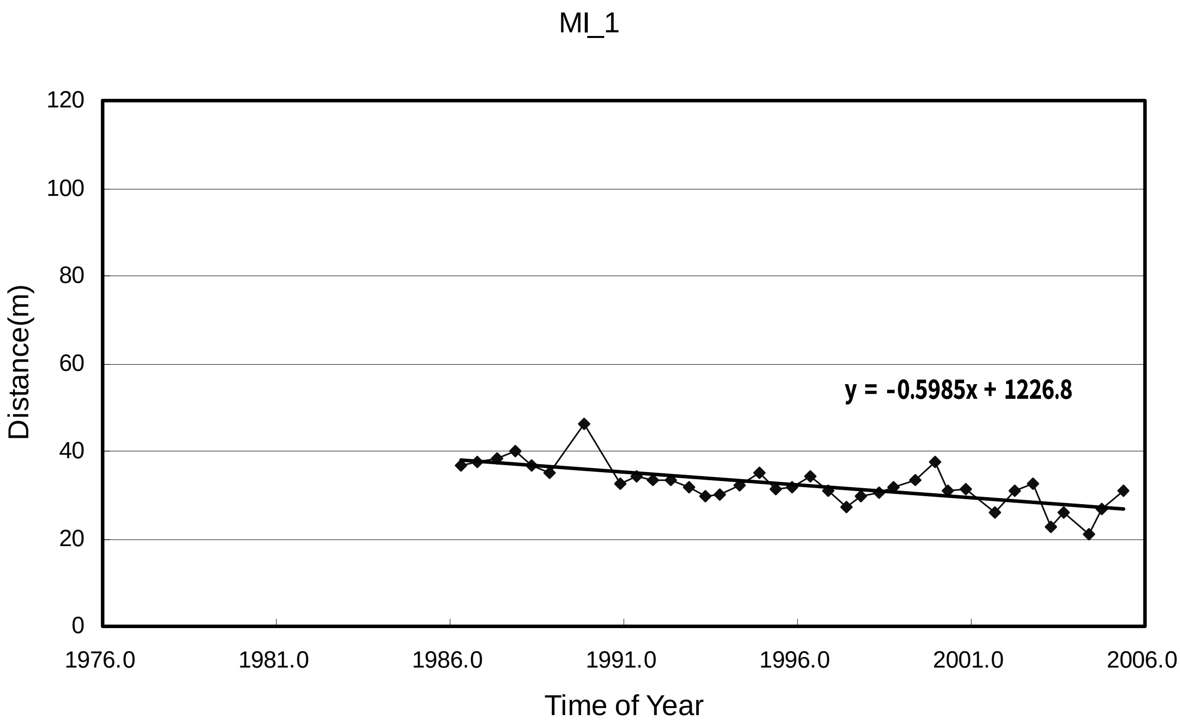

Fig. E.2.54: Regression line of shoreline distance versus time at MI-1

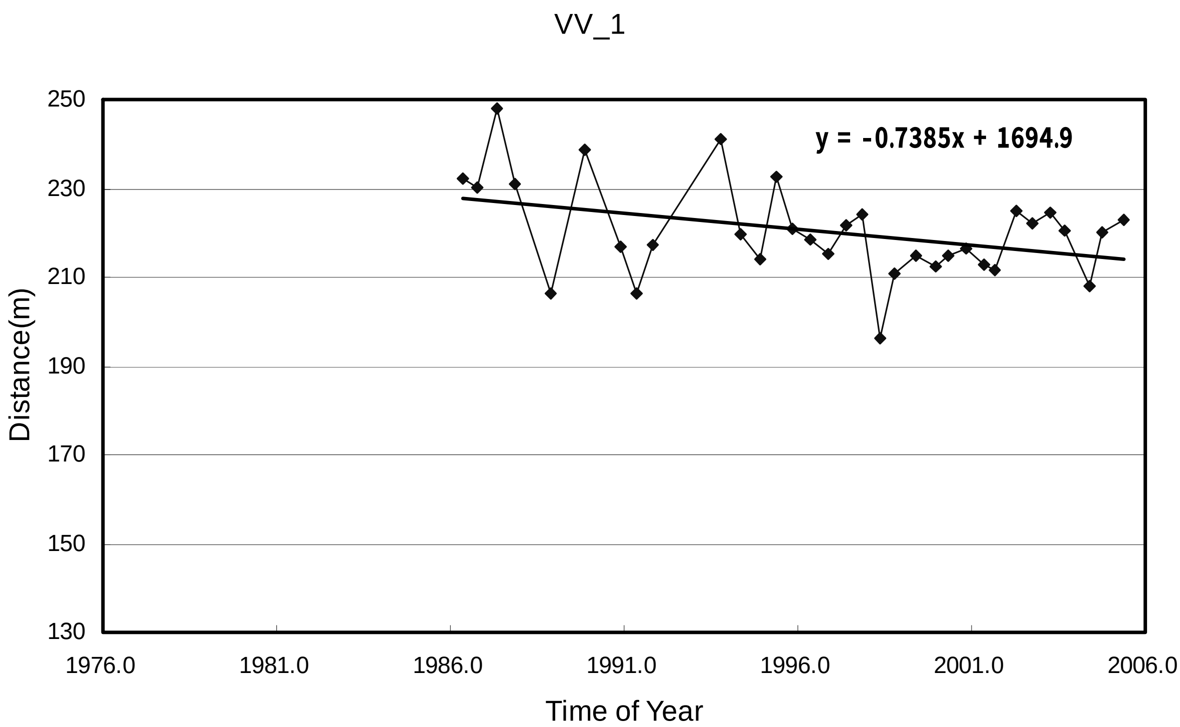

Fig. E.2.55: Regression line of shoreline distance versus time at VV-1

E.3 Grain Size Distribution Curves of Sediment Samples

Test sieve size [mm]	Retained quantity g	Retained quantity %	Total passing %
4	0.459	0.10	99.90
2	4.161	0.90	99.00
1	47.924	10.40	88.59
0.5	135.500	29.41	59.18
0.25	80.231	17.42	41.77
0.1	190.205	41.29	0.48
0.05	1.966	0.43	0.06
Rest in box	0.231	0.05	
Total	460.677		

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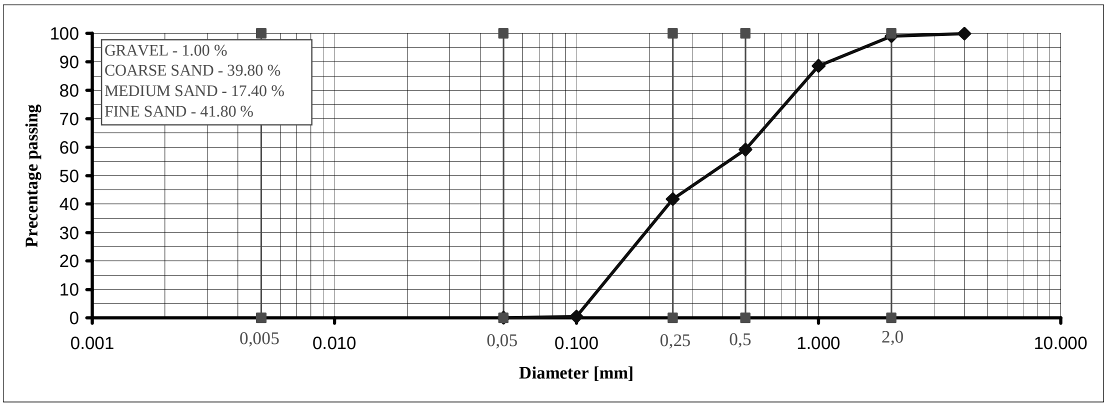
Date started: 09.06.2005
Location: -

Drill No.: -
Tube No.: -
Sample No.: Z-1
Depth [m]: -

DESCRIPTION AND PRETREATMENT		SEDIMENTATION		SIEVING		NOTES
Description	Pretreatment	Mass g	Particle density	Mass g	Process	
Sand	-	-	-	460.7	mechanical	- Tested in accordance with romanian norms STAS 1913/5-85 - * indicates particle density assumed



CLAY	SILT	FINE SAND	MEDIUM SAND	COARSE SAND	GRAVEL

REMARKS
- medium sand is in fact a mixture of sand and shell fragments
- coarse sand and gravel is in fact shell fragments and rarely shell fragments with gravel

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Fig. E.3.1: Grain size distribution curve of sediment sample at the location Z-1

Test sieve size [mm]	Retained quantity g	Retained quantity %	Total passing %
4	14.418	3.47	96.53
2	30.975	7.45	89.08
1	51.303	12.34	76.73
0.5	53.306	12.83	63.91
0.25	27.872	6.71	57.20
0.1	234.610	56.45	0.75
0.05	2.977	0.72	0.03
Rest in box	0.122	0.03	
Total	415.583		

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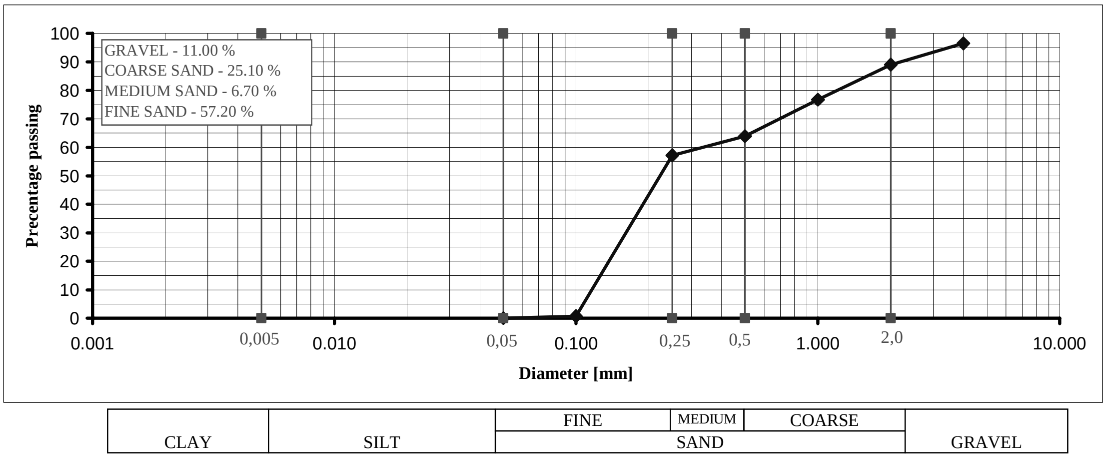
Date started: 09.06.2005
Location: -

Drill No.: -
Tube No.: -
Sample No.: Z-2
Depth [m]: -

DESCRIPTION AND PRETREATMENT		SEDIMENTATION		SIEVING		NOTES
Description	Pretreatment	Mass g	Particle density	Mass g	Process	
Sand	-	-	-	415.6	mechanical	- Tested in accordance with romanian norms STAS 1913/5-85 - * indicates particle density assumed



REMARKS

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Fig. E.3.2: Grain size distribution curve of sediment sample at the location Z-2

Test sieve size [mm]	Retained quantity g	Retained quantity %	Total passing %
4	1.872	0.41	99.59
2	3.015	0.66	98.93
1	4.258	0.93	98.00
0.5	4.676	1.02	96.98
0.25	18.127	3.96	93.02
0.1	420.278	91.88	1.13
0.05	4.960	1.08	0.05
Rest in box	0.160	0.03	
Total	457.346		

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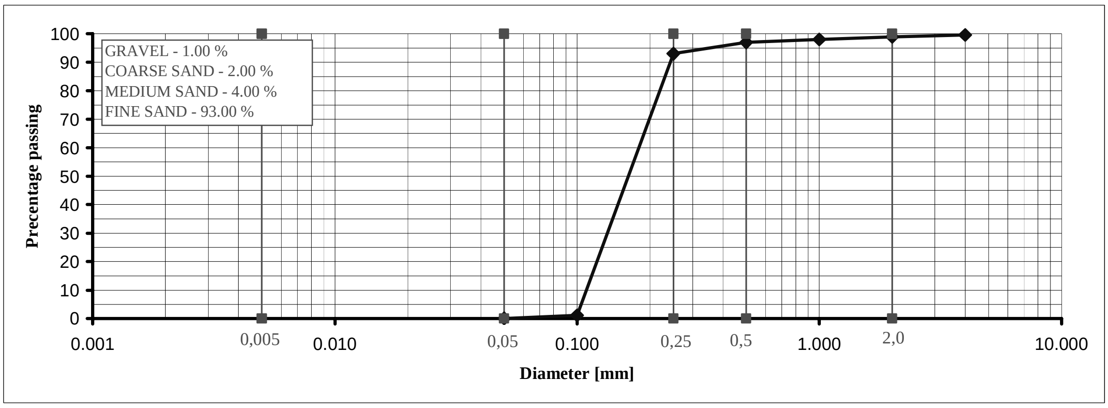
Date started:
Location: -

Drill No.: -
Tube No.: -
Sample No.: Z-3
Depth [m]: -

DESCRIPTION AND PRETREATMENT		SEDIMENTATION		SIEVING		NOTES
Description	Pretreatment	Mass g	Particle density	Mass g	Process	
Sand	-	-	-	457.4	mechanical	- Tested in accordance with romanian norms STAS 1913/5-85 - * indicates particle density assumed



CLAY	SILT	FINE	MEDIUM	COARSE	GRAVEL
		SAND			

REMARKS

- medium sand is in fact a mixture of sand and shell fragments
- coarse sand and gravel is in fact shell fragments and rarely shell fragments with gravel

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Fig. E.3.3: Grain size distribution curve of sediment sample at the location Z-3

Test sieve size [mm]	Retained quantity g	Retained quantity %	Total passing %
4	5.756	1.40	98.60
2	19.332	4.71	93.88
1	32.997	8.04	85.84
0.5	48.411	11.80	74.04
0.25	25.575	6.23	67.80
0.1	273.278	66.62	1.18
0.05	4.742	1.16	0.03
Rest in box	0.112	0.03	
Total	410.203		

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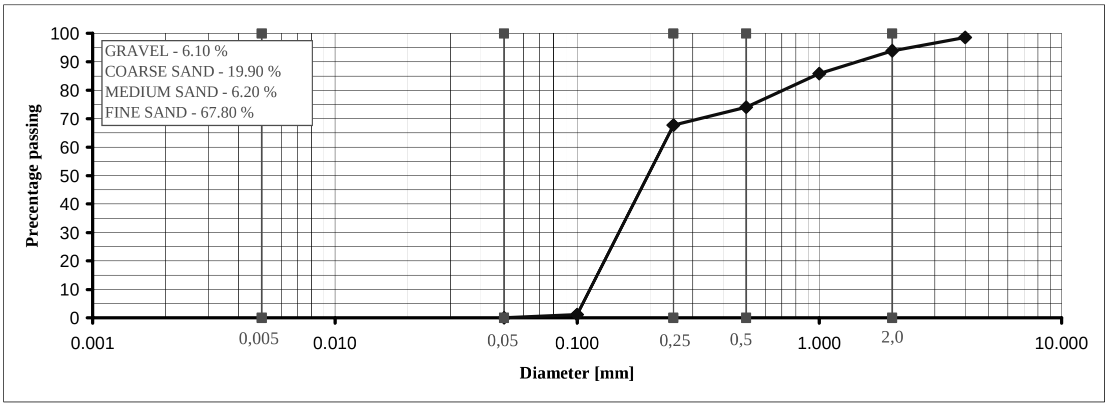
Date started: 09.06.2005
Location: -

Drill No.: -
Tube No.: -
Sample No.: A-1
Depth [m]: -

DESCRIPTION AND PRETREATMENT		SEDIMENTATION		SIEVING		NOTES
Description	Pretreatment	Mass g	Particle density	Mass g	Process	
Sand	-	-	-	410.2	mechanical	- Tested in accordance with romanian norms STAS 1913/5-85 - * indicates particle density assumed



CLAY	SILT	SAND: FINE	SAND: MEDIUM	SAND: COARSE	GRAVEL

REMARKS
- medium sand is in fact a mixture of sand and shell fragments
- coarse sand and gravel is in fact shell fragments and rarely shell fragments with gravel

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Fig. E.3.4: Grain size distribution curve of sediment sample at the location A-1

Test sieve size [mm]	Retained quantity g	Retained quantity %	Total passing %
4	1.520	0.21	99.79
2	1.455	0.20	99.60
1	1.724	0.23	99.36
0.5	2.118	0.29	99.08
0.25	11.790	1.60	97.48
0.1	715.015	96.74	0.74
0.05	5.287	0.72	0.03
Rest in box	0.158	0.02	
Total	739.067		

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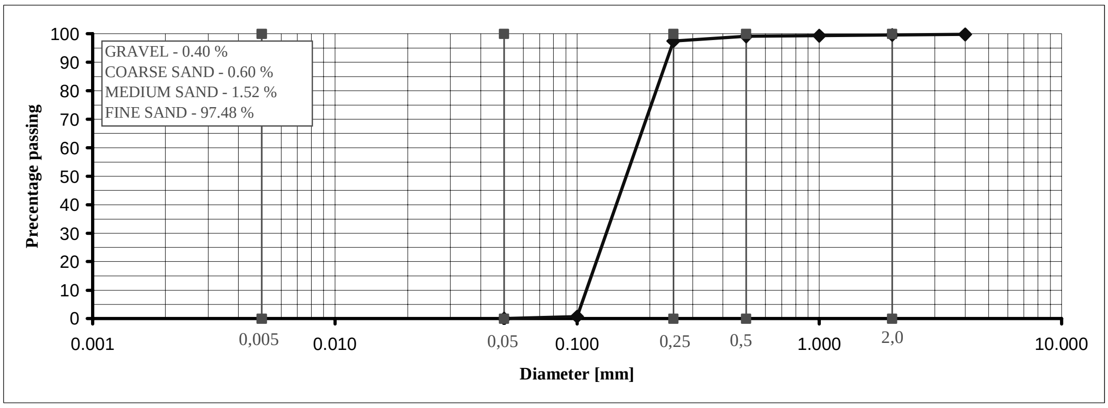
Date started:
Location: -

Drill No.: -
Tube No.: -
Sample No.: A-2
Depth [m]: -

DESCRIPTION AND PRETREATMENT		SEDIMENTATION		SIEVING		NOTES
Description	Pretreatment	Mass g	Particle density	Mass g	Process	
Sand	-	-	-	739.1	mechanical	- Tested in accordance with romanian norms STAS 1913/5-85 - * indicates particle density assumed



CLAY	SILT	FINE	MEDIUM	COARSE	GRAVEL
		SAND			

REMARKS
- medium sand is in fact a mixture of sand and shell fragments
- coarse sand and gravel is in fact shell fragments and rarely shell fragments with gravel

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Fig. E.3.5: Grain size distribution curve of sediment sample at the location A-2

Test sieve size [mm]	Retained quantity g	Retained quantity %	Total passing %
4	9.882	2.13	97.87
2	50.692	10.92	86.95
1	115.731	24.93	62.03
0.5	88.821	19.13	42.90
0.25	36.969	7.96	34.94
0.1	160.803	34.63	0.30
0.05	1.259	0.27	0.03
Rest in box	0.095	0.02	
Total	464.252		

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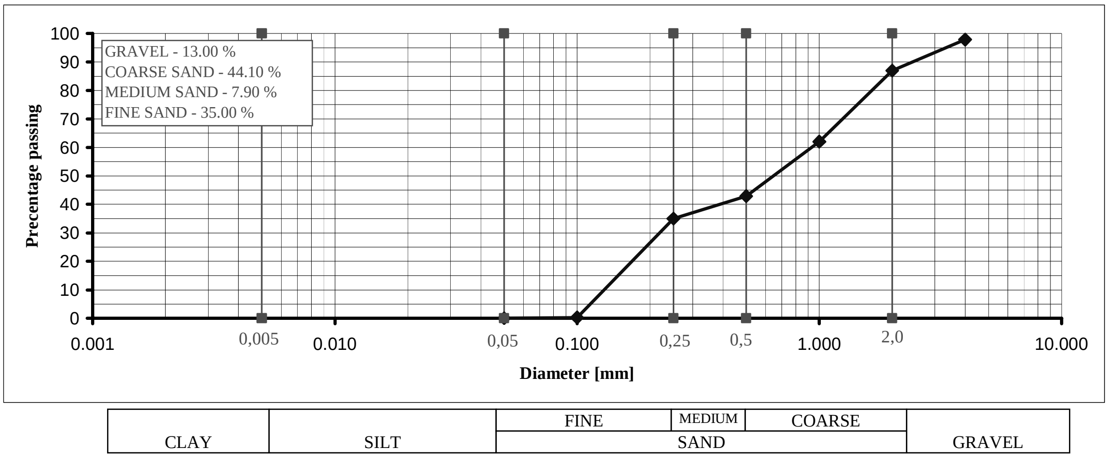
Date started: 09.06.2005
Location: -

Drill No.: -
Tube No.: -
Sample No.: A-3
Depth [m]: -

DESCRIPTION AND PRETREATMENT		SEDIMENTATION		SIEVING		NOTES
Description	Pretreatment	Mass g	Particle density	Mass g	Process	
Sand	-	-	-	464.3	mechanical	- Tested in accordance with romanian norms STAS 1913/5-85 - * indicates particle density assumed



REMARKS
- medium sand is in fact a mixture of sand and shell fragments
- coarse sand and gravel is in fact shell fragments and rarely shell fragments with gravel

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Fig. E.3.6: Grain size distribution curve of sediment sample at the location A-3

Test sieve size [mm]	Retained quantity g	Retained quantity %	Total passing %
4	1.420	0.20	99.80
2	8.088	1.13	98.67
1	46.950	6.58	92.09
0.5	159.482	22.36	69.73
0.25	151.787	21.28	48.45
0.1	339.606	47.60	0.85
0.05	5.781	0.81	0.04
Rest in box	0.265	0.04	
Total	713.379		

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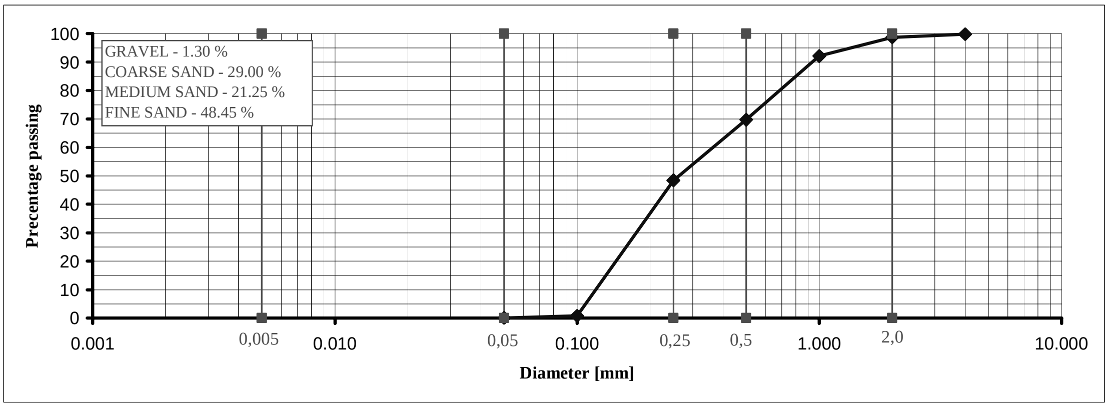
Date started: 09.06.2005
Location: -

Drill No.: -
Tube No.: -
Sample No.: A-4
Depth [m]: -

DESCRIPTION AND PRETREATMENT		SEDIMENTATION		SIEVING		NOTES
Description	Pretreatment	Mass g	Particle density	Mass g	Process	
Sand	-	-	-	713.4	mechanical	- Tested in accordance with romanian norms STAS 1913/5-85 - * indicates particle density assumed



CLAY	SILT	FINE SAND	MEDIUM SAND	COARSE SAND	GRAVEL

REMARKS
- medium sand is in fact a mixture of sand and shell fragments
- coarse sand and gravel is in fact shell fragments and rarely shell fragments with gravel

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Fig. E.3.7: Grain size distribution curve of sediment sample at the location A-4

Test sieve size [mm]	Retained quantity g	Retained quantity %	Total passing %
4	3.465	0.53	99.47
2	3.415	0.52	98.94
1	5.951	0.91	98.03
0.5	41.882	6.44	91.59
0.25	228.281	35.09	56.50
0.1	364.185	55.98	0.53
0.05	3.098	0.48	0.05
Rest in box	0.293	0.05	
Total	650.570		

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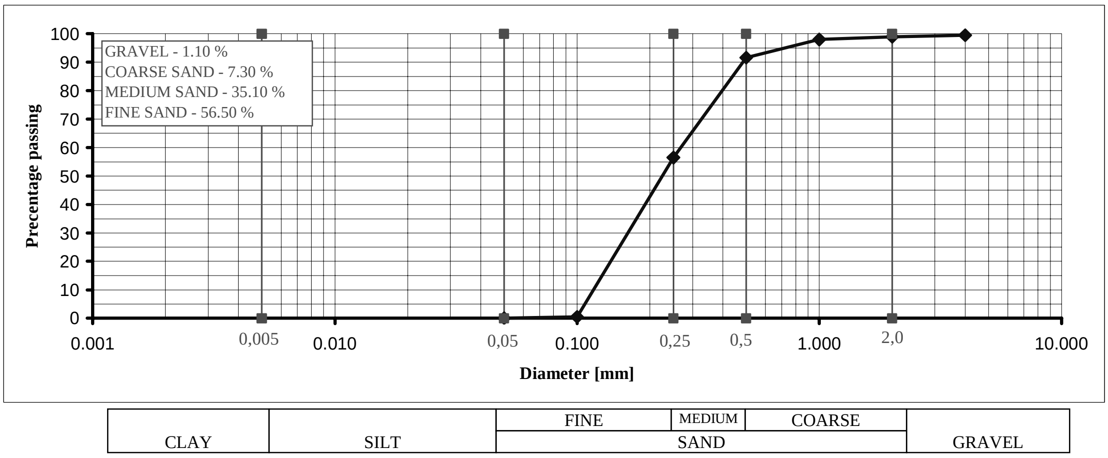
Date started: 09.06.2005
Location: -

Drill No.: -
Tube No.: -
Sample No.: A-5
Depth [m]: -

DESCRIPTION AND PRETREATMENT		SEDIMENTATION		SIEVING		NOTES
Description	Pretreatment	Mass g	Particle density	Mass g	Process	
Sand	-	-	-	650.6	mechanical	- Tested in accordance with romanian norms STAS 1913/5-85 - * indicates particle density assumed



REMARKS
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- coarse sand and gravel is in fact shell fragments and rarely shell fragments with gravel

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Fig. E.3.8: Grain size distribution curve of sediment sample at the location A-5

Test sieve size [mm]	Retained quantity g	Retained quantity %	Total passing %
4	1.310	0.19	99.81
2	2.624	0.38	99.44
1	6.334	0.91	98.53
0.5	43.623	6.26	92.27
0.25	321.623	46.12	46.15
0.1	319.625	45.84	0.31
0.05	2.066	0.30	0.01
Rest in box	0.140	0.02	
Total	697.345		

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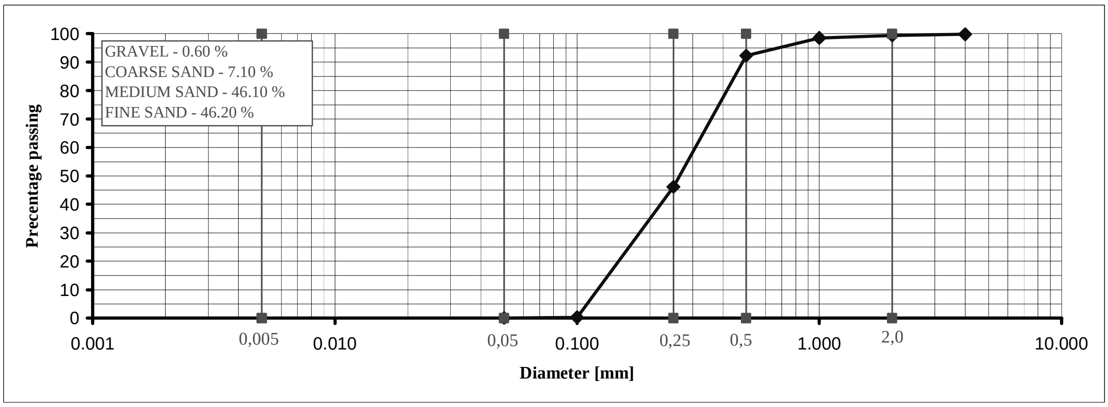
Date started: 09.06.2005
Location: -

Drill No.: -
Tube No.: -
Sample No.: A-6
Depth [m]: -

DESCRIPTION AND PRETREATMENT		SEDIMENTATION		SIEVING		NOTES
Description	Pretreatment	Mass g	Particle density	Mass g	Process	
Sand	-	-	-	697.3	mechanical	- Tested in accordance with romanian norms STAS 1913/5-85 - * indicates particle density assumed



CLAY	SILT	FINE	MEDIUM	COARSE	GRAVEL
		SAND			

REMARKS
- medium sand is in fact a mixture of sand and shell fragments
- coarse sand and gravel is in fact shell fragments and rarely shell fragments with gravel

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Fig. E.3.9: Grain size distribution curve of sediment sample at the location A-6

Test sieve size [mm]	Retained quantity g	Retained quantity %	Total passing %
4	30.020	5.22	94.78
2	55.655	9.68	85.09
1	186.512	32.45	52.65
0.5	187.347	32.59	20.05
0.25	58.367	10.15	9.90
0.1	55.869	9.72	0.18
0.05	0.855	0.15	0.03
Rest in box	0.176	0.03	
Total	574.801		

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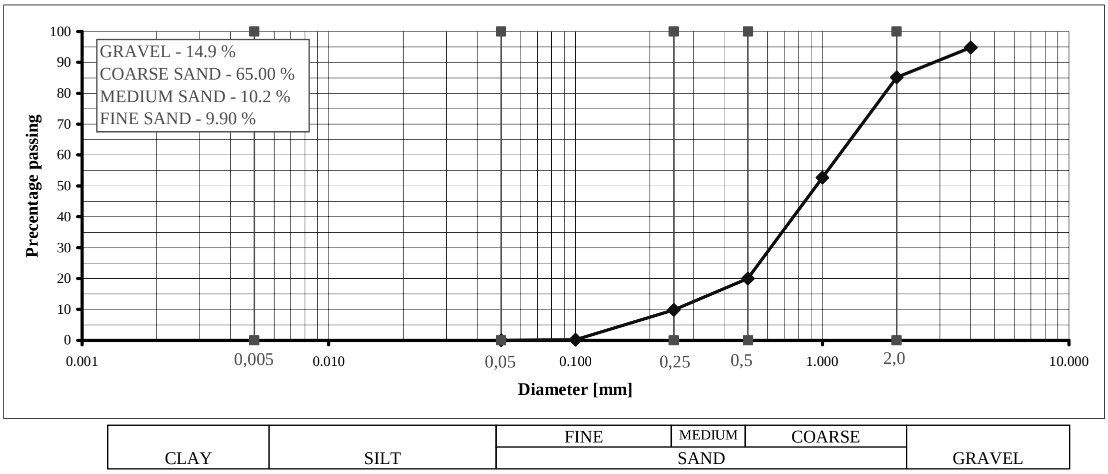
Date started: 09.06.2005
Location: -

Drill No.: -
Tube No.: -
Sample No.: B-1
Depth [m]: -

DESCRIPTION AND PRETREATMENT		SEDIMENTATION		SIEVING		NOTES
Description	Pretreatment	Mass g	Particle density	Mass g	Process	
Sand	-	-	-	574.8	mechanical	- Tested in accordance with romanian norms STAS 1913/5-85 - * indicates particle density assumed



REMARKS
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Fig. E.3.10: Grain size distribution curve of sediment sample at the location B-1

Test sieve size [mm]	Retained quantity g	Retained quantity %	Total passing %
4	9.857	1.66	98.34
2	51.074	8.59	89.76
1	151.236	25.43	64.33
0.5	147.021	24.72	39.61
0.25	95.860	16.12	23.50
0.1	132.140	22.22	1.28
0.05	7.439	1.25	0.03
Rest in box	0.195	0.03	
Total	594.822		

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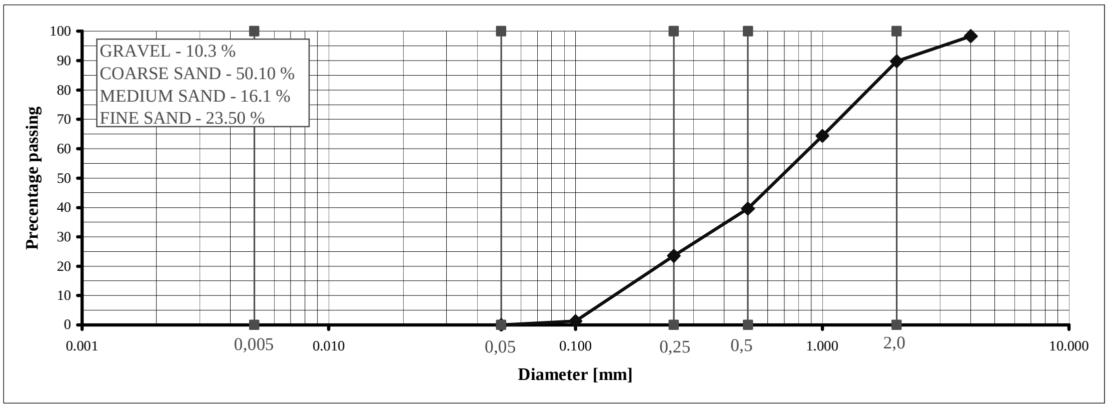
Date started: 09.06.2005
Location: -

Drill No.: -
Tube No.: -
Sample No.: B-2
Depth [m]: -

DESCRIPTION AND PRETREATMENT		SEDIMENTATION		SIEVING		NOTES
Description	Pretreatment	Mass g	Particle density	Mass g	Process	
Sand	-	-	-	594.8	mechanical	- Tested in accordance with romanian norms STAS 1913/5-85 - * indicates particle density assumed



CLAY	SILT	FINE	MEDIUM	COARSE	GRAVEL
		SAND			

REMARKS
- medium sand is in fact a mixture of sand and shell fragments
- coarse sand and gravel is in fact shell fragments and rarely shell fragments with gravel

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Fig. E.3.11: Grain size distribution curve of sediment sample at the location B-2

Test sieve size [mm]	Retained quantity g	Retained quantity %	Total passing %
4	6.735	1.31	98.69
2	31.678	6.17	92.52
1	40.531	7.89	84.63
0.5	59.894	11.66	72.97
0.25	78.571	15.30	57.67
0.1	288.978	56.27	1.40
0.05	6.116	1.19	0.21
Rest in box	1.141	0.22	
Total	513.644		

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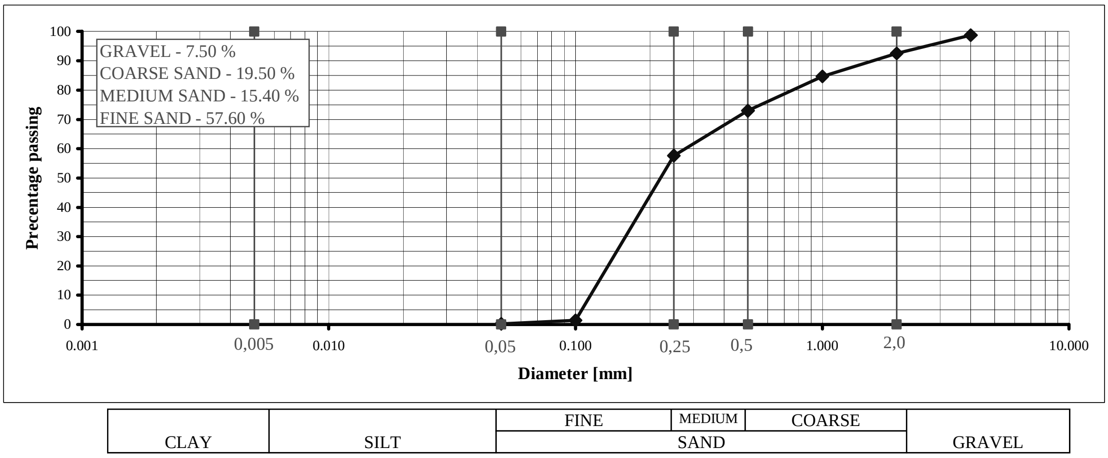
Date started: 09.06.2005
Location: -

Drill No.: -
Tube No.: -
Sample No.: B-3
Depth [m]: -

DESCRIPTION AND PRETREATMENT		SEDIMENTATION		SIEVING		NOTES
Description	Pretreatment	Mass g	Particle density	Mass g	Process	
Sand	-	-	-	513.60	mechanical	- Tested in accordance with romanian norms STAS 1913/5-85 - * indicates particle density assumed



REMARKS
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Fig. E.3.12: Grain size distribution curve of sediment sample at the location B-3

Test sieve size [mm]	Retained quantity g	Retained quantity %	Total passing %
4	5.744	1.43	98.57
2	11.212	2.79	95.78
1	35.172	8.76	87.02
0.5	158.733	39.52	47.51
0.25	165.877	41.29	6.21
0.1	24.750	6.16	0.05
0.05	0.198	0.05	0.00
Rest in box	0.039	0.01	
Total	401.725		

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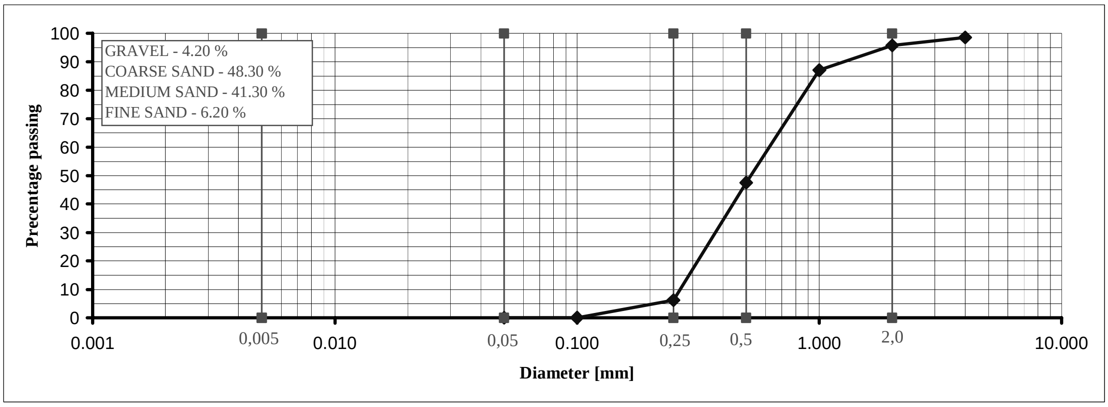
Date started: 09.06.2005
Location: -

Drill No.: -
Tube No.: -
Sample No.: C-1
Depth [m]: -

DESCRIPTION AND PRETREATMENT		SEDIMENTATION		SIEVING		NOTES
Description	Pretreatment	Mass g	Particle density	Mass g	Process	
Sand	-	-	-	401.7	mechanical	- Tested in accordance with romanian norms STAS 1913/5-85 - * indicates particle density assumed



CLAY	SILT	SAND: FINE	SAND: MEDIUM	SAND: COARSE	GRAVEL

REMARKS
- medium sand is in fact a mixture of sand and shell fragments
- coarse sand and gravel is in fact shell fragments and rarely shell fragments with gravel

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Fig. E.3.13 Grain size distribution curve of sediment sample at the location C-1

Test sieve size [mm]	Retained quantity g	Retained quantity %	Total passing %
4	1.655	0.40	99.60
2	6.219	1.52	98.08
1	17.783	4.33	93.75
0.5	79.618	19.40	74.35
0.25	207.106	50.46	23.88
0.1	97.715	23.81	0.07
0.05	0.275	0.07	0.01
Rest in box	0.070	0.02	
Total	410.441		

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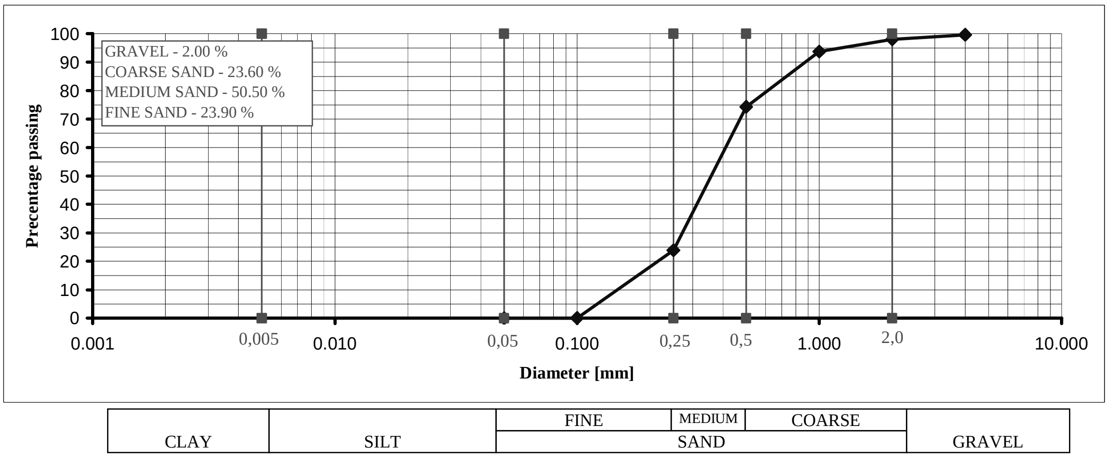
Date started: 09.06.2005
Location: -

Drill No.: -
Tube No.: -
Sample No.: C-2
Depth [m]: -

DESCRIPTION AND PRETREATMENT		SEDIMENTATION		SIEVING		NOTES
Description	Pretreatment	Mass g	Particle density	Mass g	Process	
Sand	-	-	-	410.4	mechanical	- Tested in accordance with romanian norms STAS 1913/5-85 - * indicates particle density assumed



REMARKS
- medium sand is in fact a mixture of sand and shell fragments
- coarse sand and gravel is in fact shell fragments and rarely shell fragments with gravel

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Fig. E.3.14: Grain size distribution curve of sediment sample at the location C-2

Test sieve size [mm]	Retained quantity g	Retained quantity %	Total passing %
4	0.000	0.00	100.00
2	0.450	0.11	99.89
1	1.854	0.45	99.44
0.5	85.974	21.07	78.36
0.25	298.261	73.10	5.26
0.1	21.415	5.25	0.01
0.05	0.067	0.02	-0.01
Rest in box	0.021	0.01	
Total	408.042		

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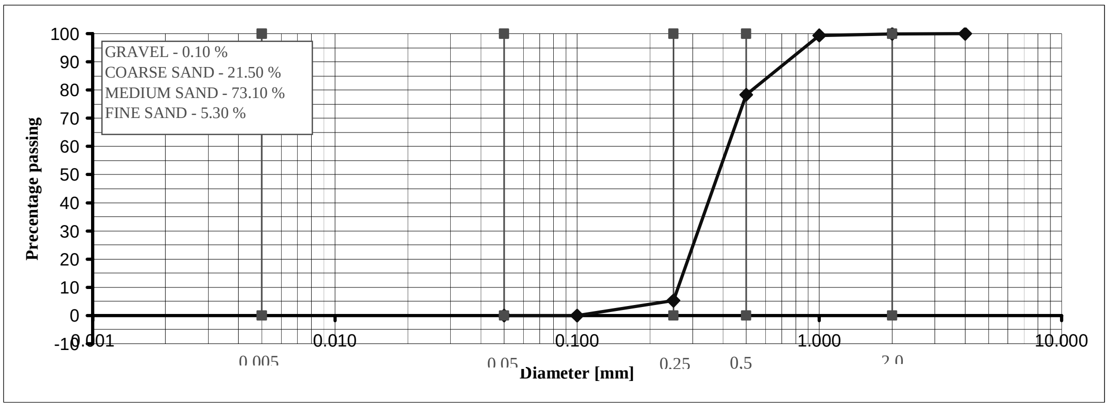
Date started: 09.06.2005
Location: -

Drill No.: -
Tube No.: -
Sample No.: C-3
Depth [m]: -

DESCRIPTION AND PRETREATMENT		SEDIMENTATION		SIEVING		NOTES
Description	Pretreatment	Mass g	Particle density	Mass g	Process	
Sand	-	-	-	408.0	mechanical	- Tested in accordance with romanian norms STAS 1913/5-85 - * indicates particle density assumed



CLAY	SILT	FINE	MEDIUM	COARSE	GRAVEL
		SAND			

REMARKS
- medium sand is in fact a mixture of sand and shell fragments
- coarse sand and gravel is in fact shell fragments and rarely shell fragments with gravel

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Fig. E.3.15: Grain size distribution curve of sediment sample at the location C-3

Test sieve size [mm]	Retained quantity g	Retained quantity %	Total passing %
4	0.252	0.06	99.94
2	0.316	0.08	99.86
1	1.866	0.46	99.39
0.5	69.095	17.21	82.18
0.25	298.002	74.24	7.94
0.1	31.786	7.92	0.02
0.05	0.079	0.02	0.00
Rest in box	0.000	0.00	
Total	401.396		

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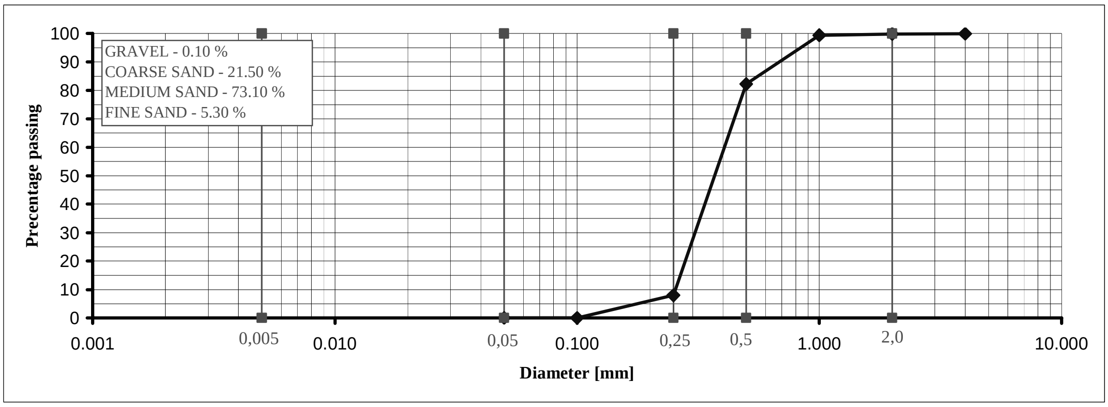
Date started: 09.06.2005
Location: -

Drill No.: -
Tube No.: -
Sample No.: C-4
Depth [m]: -

DESCRIPTION AND PRETREATMENT		SEDIMENTATION		SIEVING		NOTES
Description	Pretreatment	Mass g	Particle density	Mass g	Process	
Sand	-	-	-	401.4	mechanical	- Tested in accordance with romanian norms STAS 1913/5-85 - * indicates particle density assumed



CLAY	SILT	FINE	MEDIUM	COARSE	GRAVEL
		SAND			

REMARKS
- medium sand is in fact a mixture of sand and shell fragments
- coarse sand and gravel is in fact shell fragments and rarely shell fragments with gravel

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Fig. E.3.16: Grain size distribution curve of sediment sample at the location C-4

Test sieve size [mm]	Retained quantity g	Retained quantity %	Total passing %
4	4.009	0.88	99.12
2	26.965	5.94	93.18
1	87.114	19.19	73.99
0.5	218.780	48.19	25.80
0.25	113.750	25.06	0.74
0.1	3.302	0.73	0.02
0.05	0.082	0.02	0.00
Rest in box	0.000	0.00	
Total	454.002		

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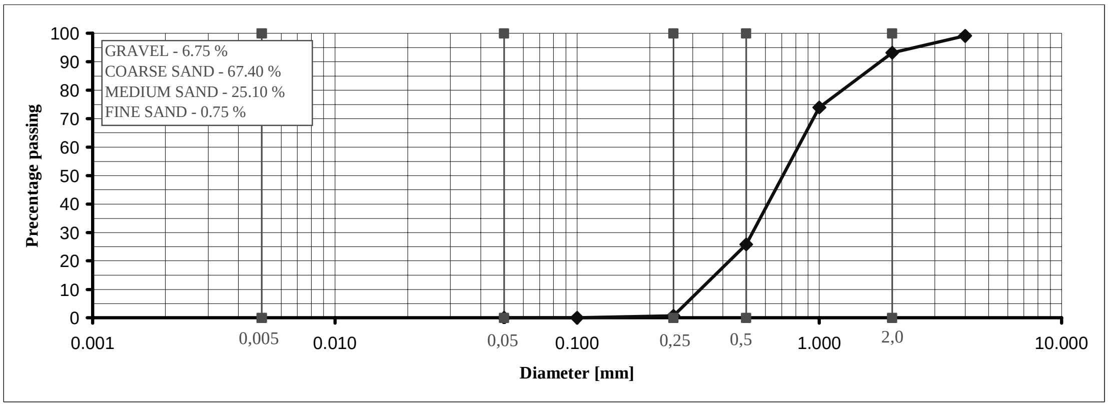
Date started: 09.06.2005
Location: -

Drill No.: -
Tube No.: -
Sample No.: C-5
Depth [m]: -

DESCRIPTION AND PRETREATMENT		SEDIMENTATION		SIEVING		NOTES
Description	Pretreatment	Mass g	Particle density	Mass g	Process	
Sand	-	-	-	454.0	mechanical	- Tested in accordance with romanian norms STAS 1913/5-85 - * indicates particle density assumed



CLAY	SILT	FINE SAND	MEDIUM SAND	COARSE SAND	GRAVEL

REMARKS
- medium sand is in fact a mixture of sand and shell fragments
- coarse sand and gravel is in fact shell fragments and rarely shell fragments with gravel

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Fig. E.3.17: Grain size distribution curve of sediment sample at the location C-5

Test sieve size [mm]	Retained quantity g	Retained quantity %	Total passing %
4	0.000	0.00	100.00
2	0.957	0.19	99.81
1	5.687	1.11	98.70
0.5	102.849	20.16	78.54
0.25	386.686	75.79	2.75
0.1	14.467	2.84	-0.09
0.05	0.059	0.01	-0.10
Rest in box	0.015	0.00	
Total	510.720		

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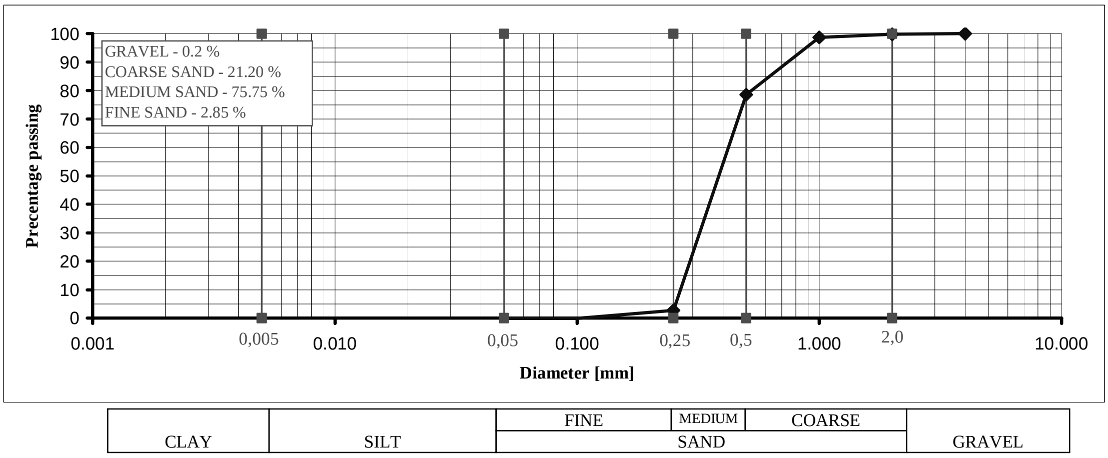
Date started: 09.06.2005
Location: -

Drill No.: -
Tube No.: -
Sample No.: C-6
Depth [m]: -

DESCRIPTION AND PRETREATMENT		SEDIMENTATION		SIEVING		NOTES
Description	Pretreatment	Mass g	Particle density	Mass g	Process	
Sand	-	-	-	510.2	mechanical	- Tested in accordance with romanian norms STAS 1913/5-85 - * indicates particle density assumed



REMARKS
- medium sand is in fact a mixture of sand and shell fragments
- coarse sand and gravel is in fact shell fragments and rarely shell fragments with gravel

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Fig. E.3.18: Grain size distribution curve of sediment sample at the location C-6

Test sieve size [mm]	Retained quantity g	Retained quantity %	Total passing %
4	0.076	0.01	99.99
2	6.237	1.08	98.91
1	37.444	6.48	92.43
0.5	106.124	18.36	74.07
0.25	260.503	45.07	29.00
0.1	166.363	28.78	0.22
0.05	1.074	0.19	0.03
Rest in box	0.136	0.02	
Total	577.957		

PARTICLE SIZE DISTRIBUTION

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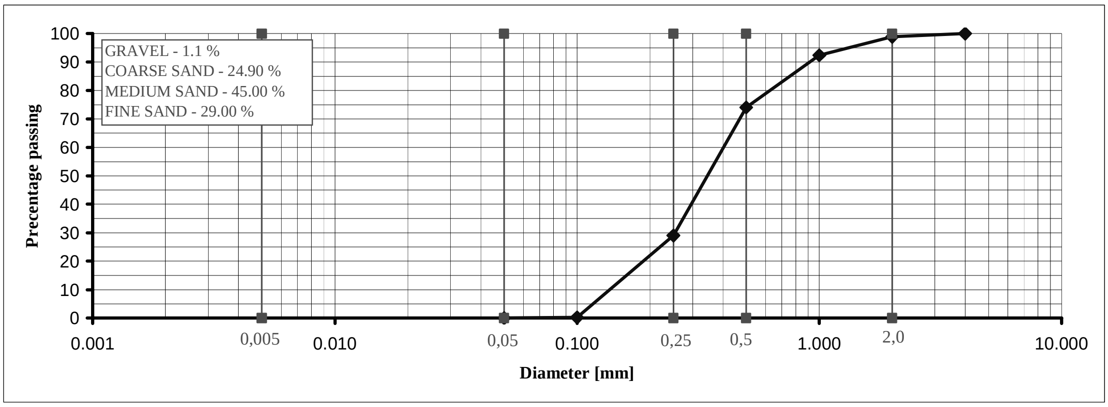
Date started: 09.06.2005
Location: -

Drill No.: -
Tube No.: -
Sample No.: C-7
Depth [m]: -

DESCRIPTION AND PRETREATMENT		SEDIMENTATION		SIEVING		NOTES
Description	Pretreatment	Mass g	Particle density	Mass g	Process	
Sand	-	-	-	578.0	mechanical	- Tested in accordance with romanian norms STAS 1913/5-85 - * indicates particle density assumed



CLAY	SILT	SAND: FINE	SAND: MEDIUM	SAND: COARSE	GRAVEL

REMARKS
- medium sand is in fact a mixture of sand and shell fragments
- coarse sand and gravel is in fact shell fragments and rarely shell fragments with gravel

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Fig. E.3.19: Grain size distribution curve of sediment sample at the location C-7

Test sieve size [mm]	Retained quantity g	Retained quantity %	Total passing %
4	24.498	4.54	95.46
2	10.026	1.86	93.60
1	100.889	18.70	74.90
0.5	218.166	40.45	34.45
0.25	129.104	23.93	10.51
0.1	56.424	10.46	0.05
0.05	0.194	0.04	0.02
Rest in box	0.059	0.01	
Total	539.360		

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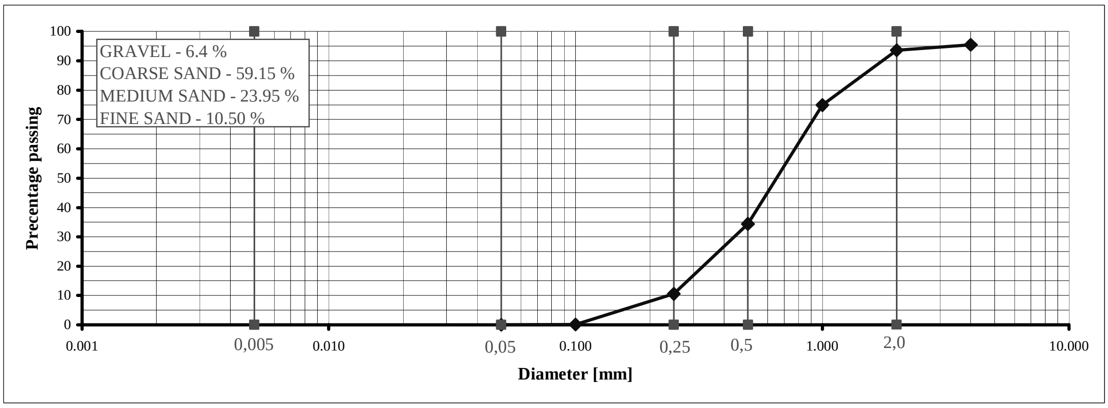
Date started:
Location: -

Drill No.: -
Tube No.: -
Sample No.: D-1
Depth [m]: -

DESCRIPTION AND PRETREATMENT		SEDIMENTATION		SIEVING		NOTES
Description	Pretreatment	Mass g	Particle density	Mass g	Process	
Sand	-	-	-	539.4	mechanical	- Tested in accordance with romanian norms STAS 1913/5-85 - * indicates particle density assumed



CLAY	SILT	FINE SAND	MEDIUM SAND	COARSE SAND	GRAVEL

REMARKS
- medium sand is in fact a mixture of sand and shell fragments
- coarse sand and gravel is in fact shell fragments and rarely shell fragments with gravel

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Fig. E.3.20: Grain size distribution curve of sediment sample at the location D-1

Test sieve size [mm]	Retained quantity g	%	Total passing %
4	0.000	0.00	100.00
2	1.310	0.26	99.74
1	17.735	3.53	96.21
0.5	175.406	34.91	61.30
0.25	243.478	48.46	12.83
0.1	64.035	12.75	0.09
0.05	0.221	0.04	0.04
Rest in box	0.164	0.03	
Total	502.349		

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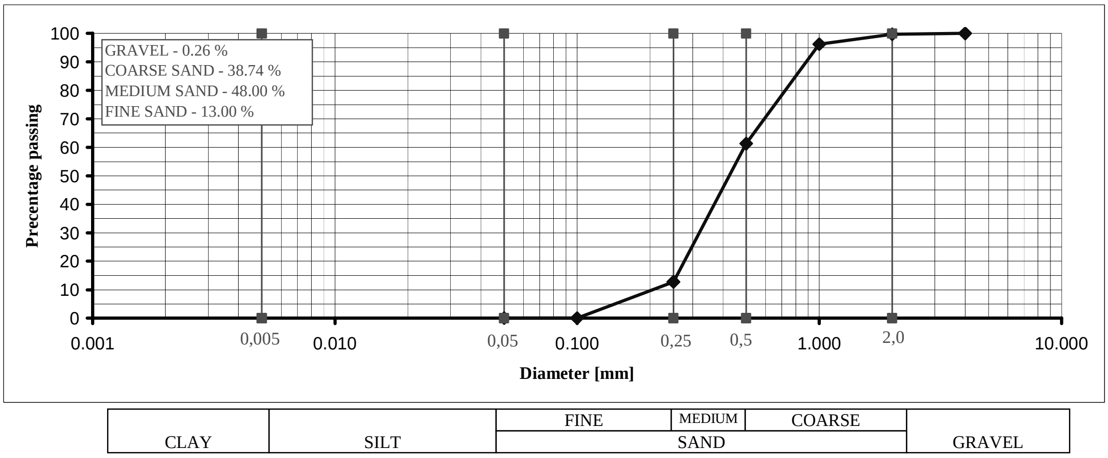
Date started:
Location: -

Drill No.: -
Tube No.: -
Sample No.: D-2
Depth [m]: -

DESCRIPTION AND PRETREATMENT		SEDIMENTATION		SIEVING		NOTES
Description	Pretreatment	Mass g	Particle density	Mass g	Process	
Sand	-	-	-	502.4	mechanical	- Tested in accordance with romanian norms STAS 1913/5-85 - * indicates particle density assumed



REMARKS
- medium sand is in fact a mixture of sand and shell fragments
- coarse sand and gravel is in fact shell fragments and rarely shell fragments with gravel

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Fig. E.3.21: Grain size distribution curve of sediment sample at the location D-2

Test sieve size [mm]	Retained quantity g	Retained quantity %	Total passing %
4	21.388	3.40	96.60
2	24.557	3.90	92.70
1	61.888	9.83	82.86
0.5	177.746	28.25	54.62
0.25	303.110	48.17	6.45
0.1	35.478	5.64	0.82
0.05	3.968	0.63	0.19
Rest in box	1.126	0.18	
Total	629.261		

PARTICLE SIZE DISTRIBUTION

Drill No.: -

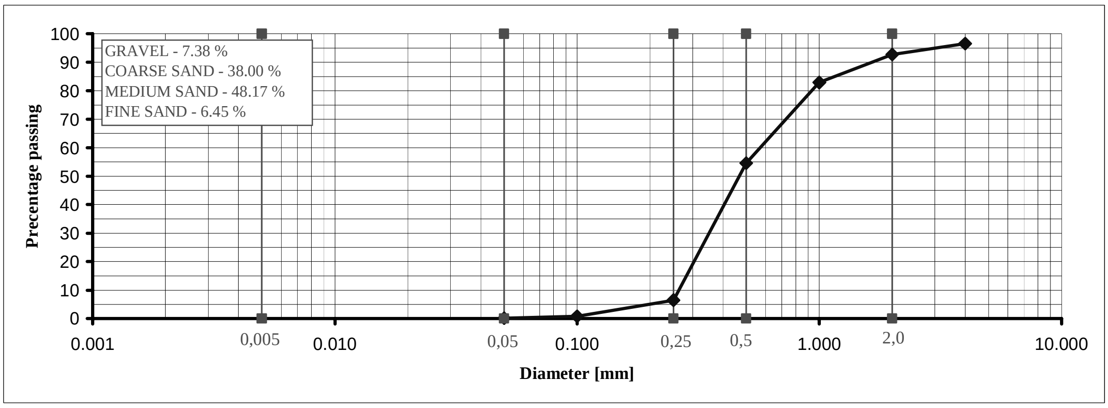
"OVIDIUS" UNIVERSITY OF CONSTANTA Date started: Tube No.: -

CIVIL ENGINEERING FACULTY Location: - Sample No.: E-1

Geotechnical Laboratory Gr.II GTF, ANCFD Depth [m]: -

Licence No. 389/ISC

DESCRIPTION AND PRETREATMENT		SEDIMENTATION		SIEVING		NOTES
Description	Pretreatment	Mass g	Particle density	Mass g	Process	
Sand	-	-	-	629.3	mechanical	- Tested in accordance with romanian norms STAS 1913/5-85 - * indicates particle density assumed



CLAY	SILT	FINE SAND	MEDIUM SAND	COARSE SAND	GRAVEL

REMARKS

- medium sand is in fact a mixture of sand and shell fragments
- coarse sand and gravel is in fact shell fragments and rarely shell fragments with gravel

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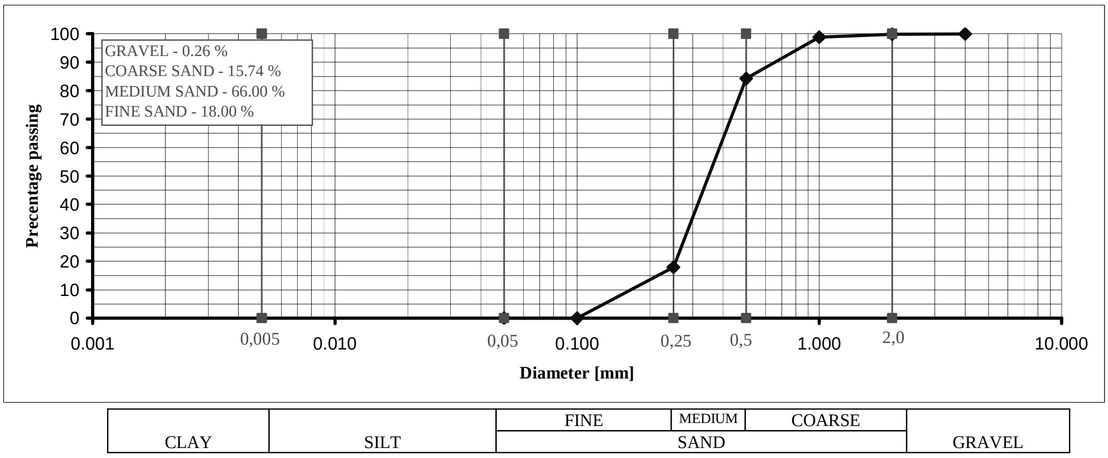
Fig. E.3.22: Grain size distribution curve of sediment sample at the location E-1

Test sieve size [mm]	Retained quantity g	%	Total passing %
4	0.547	0.09	99.91
2	1.080	0.17	99.74
1	5.725	0.91	98.83
0.5	91.890	14.59	84.24
0.25	417.415	66.28	17.96
0.1	112.945	17.93	0.03
0.05	0.150	0.02	0.01
Rest in box	0.007	0.00	
Total	629.759		

PARTICLE SIZE DISTRIBUTION

"OVIDIUS" UNIVERSITY OF CONSTANTA	Date started:	Drill No.: -
CIVIL ENGINEERING FACULTY	Location: -	Tube No.: -
Geotechnical Laboratory Gr.II GTF, ANCFD		Sample No.: E-2
Licence No. 389/ISC		Depth [m]: -

DESCRIPTION AND PRETREATMENT		SEDIMENTATION		SIEVING		NOTES
Description	Pretreatment	Mass g	Particle density	Mass g	Process	
Sand	-	-	-	629.8	mechanical	- Tested in accordance with romanian norms STAS 1913/5-85 - * indicates particle density assumed



REMARKS

- medium sand is in fact a mixture of sand and shell fragments
- coarse sand and gravel is in fact shell fragments and rarely shell fragments with gravel

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Fig. E.3.23: Grain size distribution curve of sediment sample at the location E-2

Test sieve size [mm]	Retained quantity g	Retained quantity %	Total passing %
4	0.000	0.00	100.00
2	0.834	0.15	99.85
1	25.866	4.60	95.25
0.5	428.177	76.19	19.06
0.25	106.092	18.88	0.18
0.1	0.876	0.16	0.03
0.05	0.046	0.01	0.02
Rest in box	0.046	0.01	
Total	561.937		

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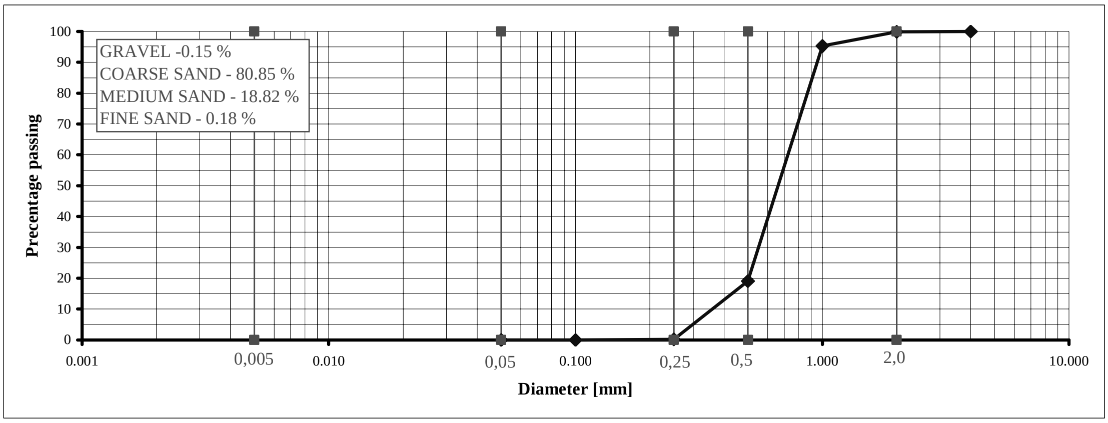
Date started:
Location: -

Drill No.: -
Tube No.: -
Sample No.: E-3
Depth [m]: -

DESCRIPTION AND PRETREATMENT		SEDIMENTATION		SIEVING		NOTES
Description	Pretreatment	Mass g	Particle density	Mass g	Process	
Sand	-	-	-	562.00	mechanical	- Tested in accordance with romanian norms STAS 1913/5-85 - * indicates particle density assumed



CLAY	SILT	FINE SAND	MEDIUM SAND	COARSE SAND	GRAVEL

REMARKS
- medium sand is in fact a mixture of sand and shell fragments
- coarse sand and gravel is in fact shell fragments and rarely shell fragments with gravel

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Fig. E.3.24: Grain size distribution curve of sediment sample at the location E-3

Test sieve size [mm]	Retained quantity g	Retained quantity %	Total passing %
4	8.385	2.72	97.28
2	16.351	5.30	91.98
1	48.253	15.64	76.34
0.5	85.006	27.55	48.79
0.25	94.754	30.71	18.07
0.1	50.936	16.51	1.56
0.05	3.458	1.12	0.44
Rest in box	1.334	0.43	
Total	308.477		

PARTICLE SIZE DISTRIBUTION

Drill No.: -

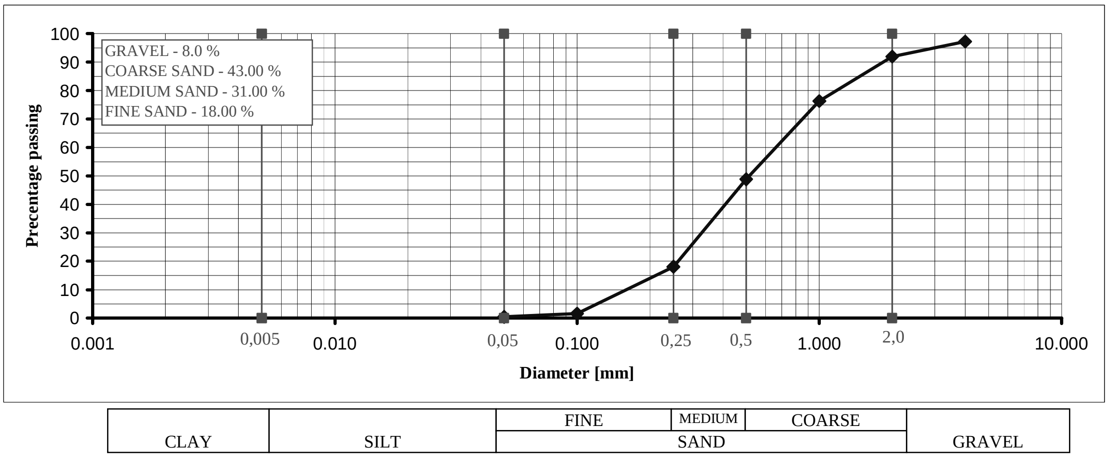
"OVIDIUS" UNIVERSITY OF CONSTANTA Date started: Tube No.: -

CIVIL ENGINEERING FACULTY Location: - Sample No.: E-4

Geotechnical Laboratory Gr.II GTF, ANCFD Depth [m]: -

Licence No. 389/ISC

DESCRIPTION AND PRETREATMENT		SEDIMENTATION		SIEVING		NOTES
Description	Pretreatment	Mass g	Particle density	Mass g	Process	
Sand	-	-	-	308.5	mechanical	- Tested in accordance with romanian norms STAS 1913/5-85 - * indicates particle density assumed



REMARKS
- medium sand is in fact a mixture of sand and shell fragments
- coarse sand and gravel is in fact shell fragments and rarely shell fragments with gravel

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Fig. E.3.25: Grain size distribution curve of sediment sample at the location E-4

Test sieve size [mm]	Retained quantity g	Retained quantity %	Total passing %
4	1.512	0.26	99.74
2	2.880	0.50	99.24
1	24.875	4.30	94.94
0.5	233.137	40.29	54.66
0.25	206.833	35.74	18.92
0.1	105.408	18.21	0.70
0.05	3.413	0.59	0.11
Rest in box	0.588	0.10	
Total	578.646		

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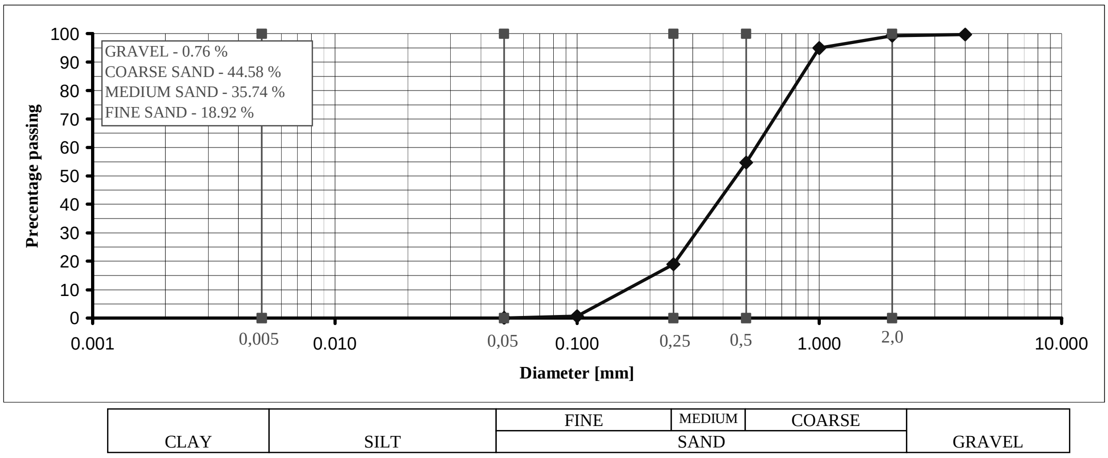
Date started:
Location: -

Drill No.: -
Tube No.: -
Sample No.: E-5
Depth [m]: -

DESCRIPTION AND PRETREATMENT		SEDIMENTATION		SIEVING		NOTES
Description	Pretreatment	Mass g	Particle density	Mass g	Process	
Sand	-	-	-	578.7	mechanical	- Tested in accordance with romanian norms STAS 1913/5-85 - * indicates particle density assumed



REMARKS
- medium sand is in fact a mixture of sand and shell fragments
- coarse sand and gravel is in fact shell fragments and rarely shell fragments with gravel

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Fig. E.3.26 Grain size distribution curve of sediment sample at the location E-5

Test sieve size [mm]	Retained quantity g	Retained quantity %	Total passing %
4	15.014	2.60	97.40
2	48.715	8.43	88.98
1	109.873	19.01	69.97
0.5	176.086	30.46	39.51
0.25	184.831	31.97	7.54
0.1	41.632	7.20	0.34
0.05	1.631	0.28	0.06
Rest in box	0.312	0.05	
Total	578.094		

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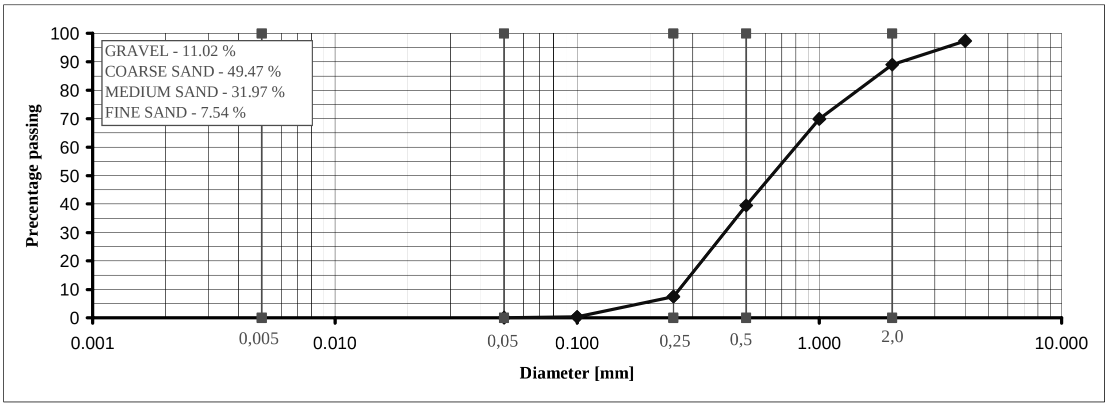
Date started:
Location: -

Drill No.: -
Tube No.: -
Sample No.: F-1
Depth [m]: -

DESCRIPTION AND PRETREATMENT		SEDIMENTATION		SIEVING		NOTES
Description	Pretreatment	Mass g	Particle density	Mass g	Process	
Sand	-	-	-	578.1	mechanical	- Tested in accordance with romanian norms STAS 1913/5-85 - * indicates particle density assumed



CLAY	SILT	SAND (FINE / MEDIUM / COARSE)	GRAVEL

REMARKS
- medium sand is in fact a mixture of sand and shell fragments
- coarse sand and gravel is in fact shell fragments and rarely shell fragments with gravel

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Fig. E.3.27: Grain size distribution curve of sediment sample at the location F-1

Test sieve size [mm]	Retained quantity g	Retained quantity %	Total passing %
4	18.662	3.54	96.46
2	54.345	10.30	86.16
1	45.307	8.59	77.58
0.5	25.934	4.92	72.66
0.25	72.746	13.79	58.87
0.1	304.373	57.69	1.18
0.05	4.582	0.87	0.31
Rest in box	1.567	0.30	
Total	527.516		

PARTICLE SIZE DISTRIBUTION

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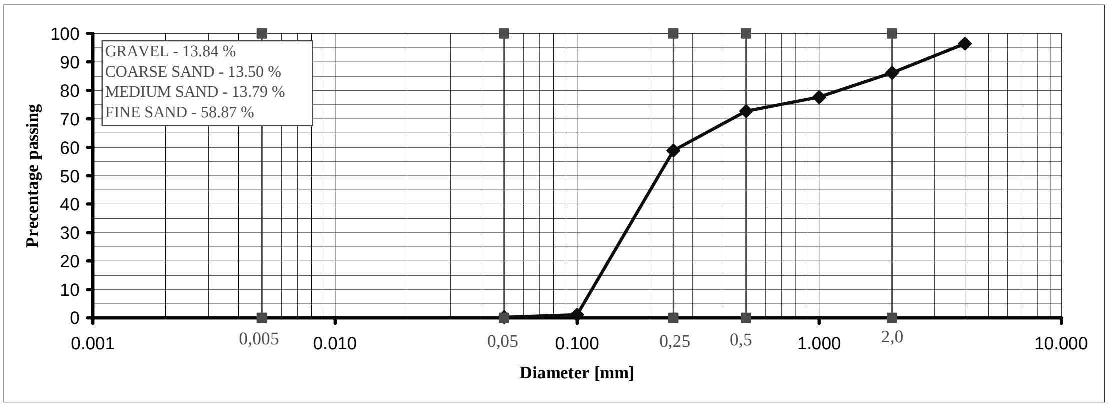
Date started:
Location: -

Drill No.: -
Tube No.: -
Sample No.: F-2
Depth [m]: -

DESCRIPTION AND PRETREATMENT		SEDIMENTATION		SIEVING		NOTES
Description	Pretreatment	Mass g	Particle density	Mass g	Process	
Sand	-	-	-	527.6	mechanical	- Tested in accordance with romanian norms STAS 1913/5-85 - * indicates particle density assumed



CLAY	SILT	FINE	MEDIUM	COARSE	GRAVEL
		SAND			

REMARKS
- medium sand is in fact a mixture of sand and shell fragments
- coarse sand and gravel is in fact shell fragments and rarely shell fragments with gravel

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Fig. E.3.28: Grain size distribution curve of sediment sample at the location F-2

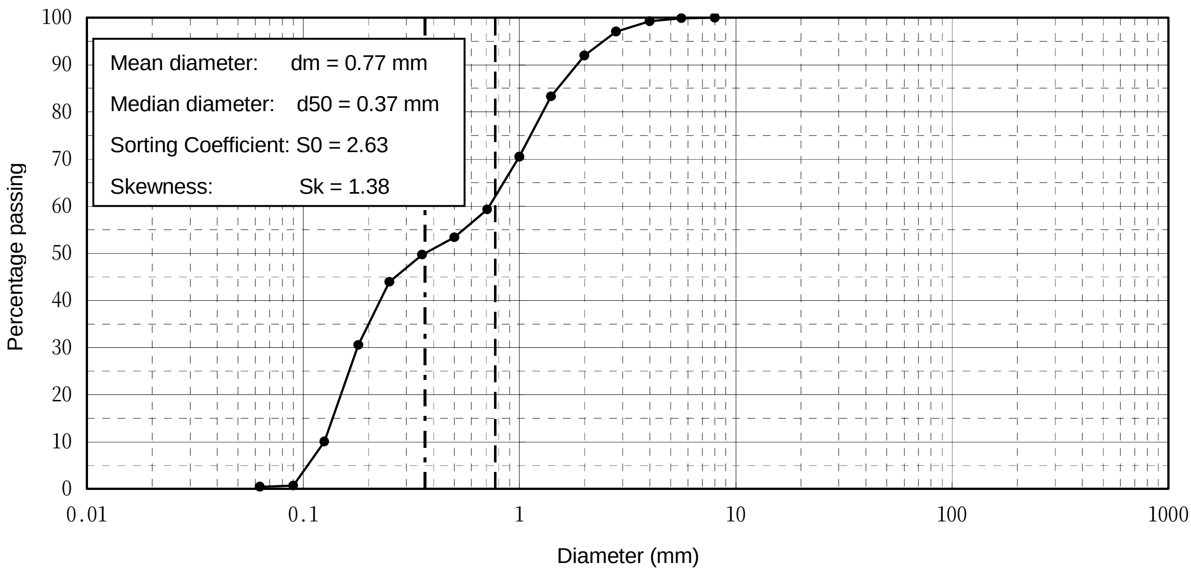

Fig. E.3.29: Grain size distribution curve of sediment sample at the location F-3

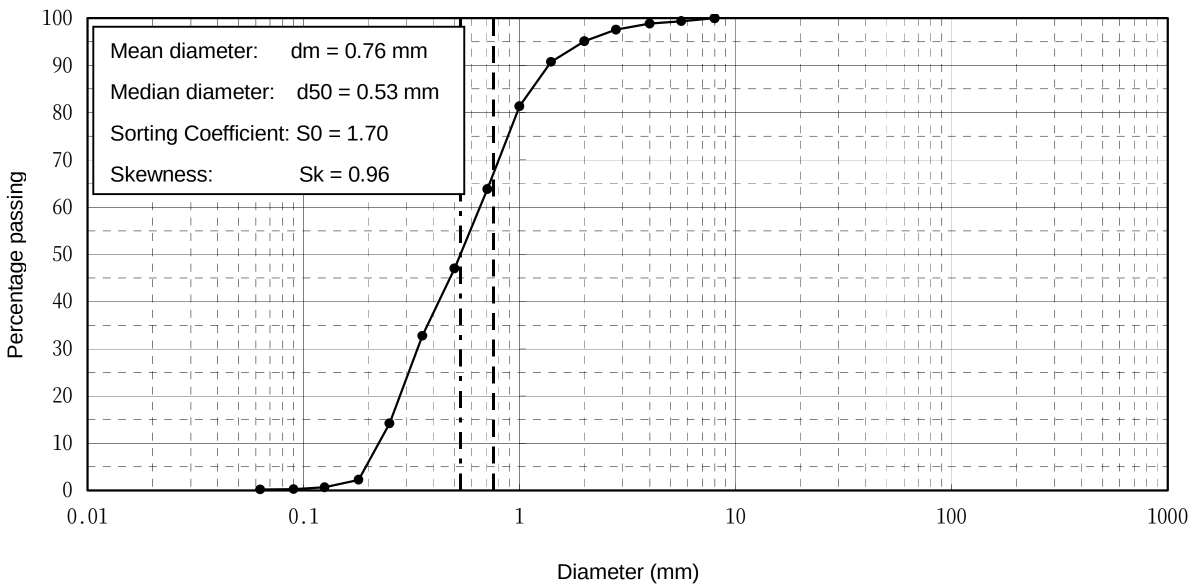

Fig. E.3.30: Grain size distribution curve of sediment sample at the location G-1

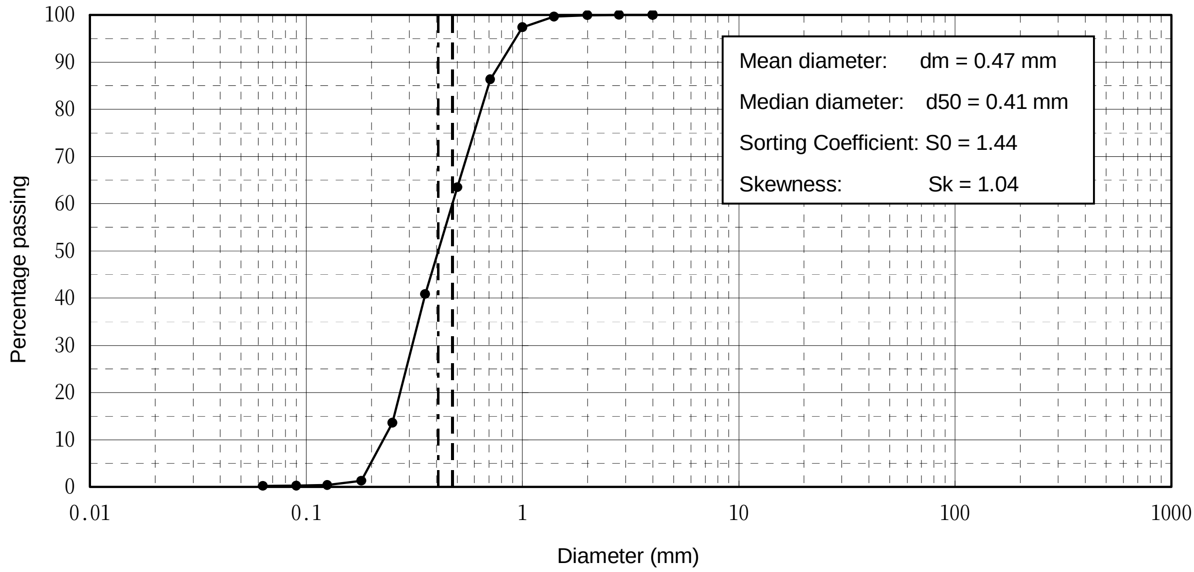

Fig. E.3.31: Grain size distribution curve of sediment sample at the location G-2

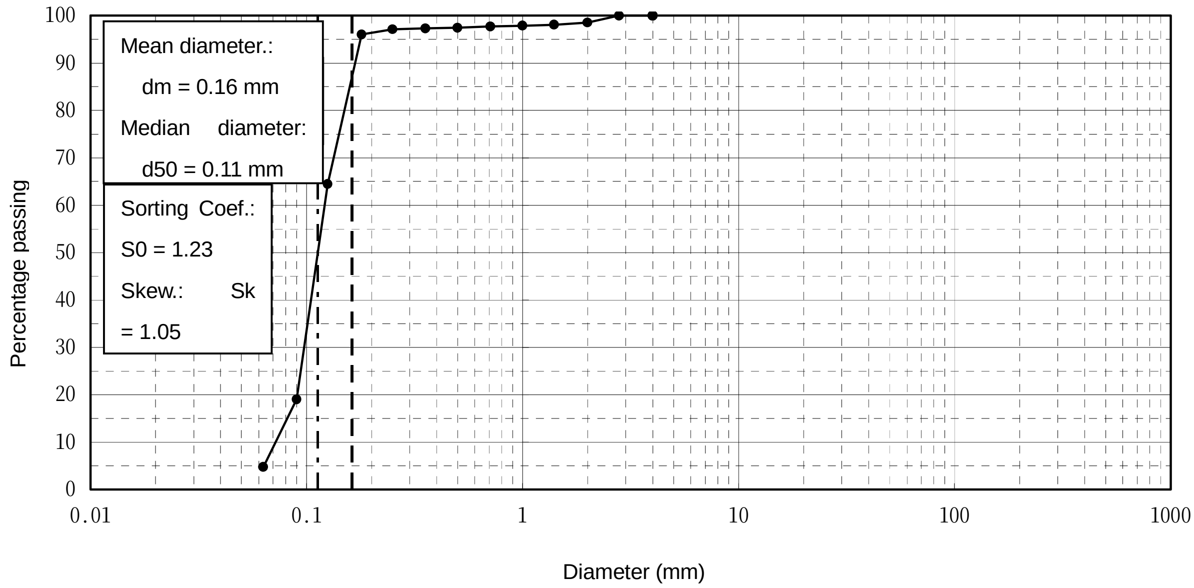

Fig. E.3.32: Grain size distribution curve of sediment sample at the location CC-2

Test sieve size [mm]	Retained quantity g	%	Total passing %
4	0.605	0.15	99.85
2	1.761	0.44	99.41
1	2.980	0.74	98.67
0.5	0.786	0.20	98.47
0.25	6.634	1.65	96.82
0.1	89.752	22.34	74.48
0.05	237.493	59.12	15.36
Rest in box	61.663	15.35	
Total	401.674		

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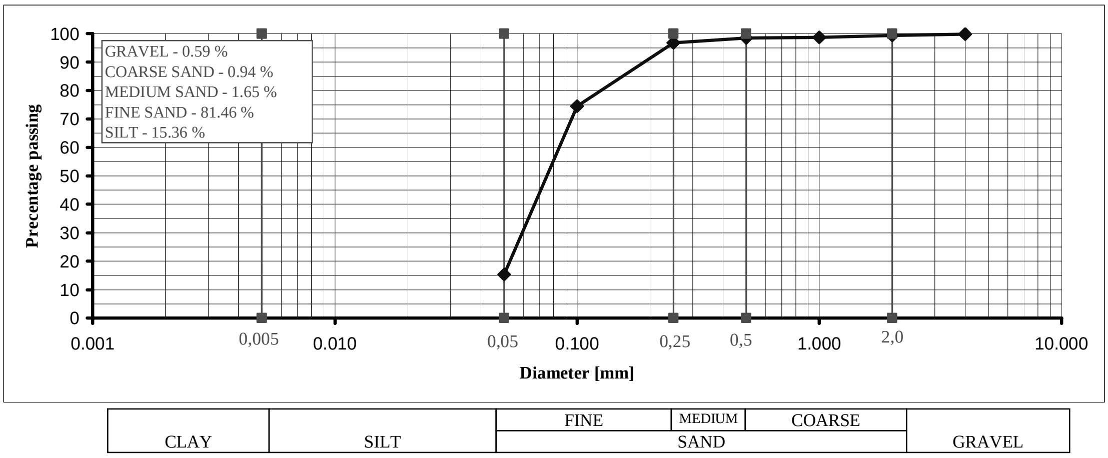
Location: -

Drill No.: -
Tube No.: -
Sample No.: CC-1
Depth [m]: -

DESCRIPTION AND PRETREATMENT		SEDIMENTATION		SIEVING		NOTES
Description	Pretreatment	Mass g	Particle density	Mass g	Process	
Sand	-	-	-	401.7	mechanical	- Tested in accordance with romanian norms STAS 1913/5-85 - * indicates particle density assumed



REMARKS
- medium sand is in fact a mixture of sand and shell fragments
- coarse sand and gravel is in fact shell fragments and rarely shell fragments with gravel

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Fig. E.3.33: Grain size distribution curve of sediment sample at the location CC-1

Test sieve size [mm]	Retained quantity g	Retained quantity %	Total passing %
4			
2			
1			
0.5			
0.25			
0.1			
0.05			
Rest in box			
Total			

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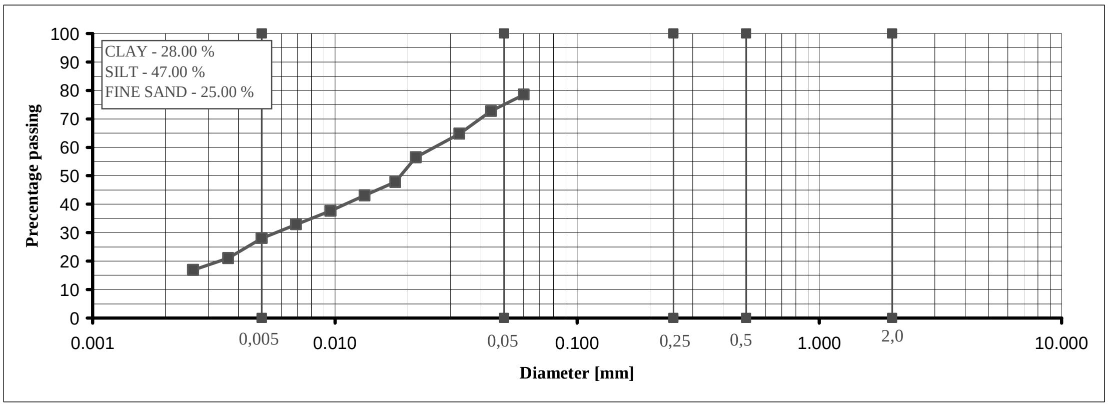
Date started:
Location: -

Drill No.: -
Tube No.: -
Sample No.: AA-1
Depth [m]: -

DESCRIPTION AND PRETREATMENT		SEDIMENTATION		SIEVING		NOTES
Description	Pretreatment	Mass g	Particle density	Mass g	Process	
Sand	Li_2CO_3	50.562	2.624	-	-	- Tested in accordance with romanian norms STAS 1913/5-85 - * indicates particle density assumed



CLAY	SILT	FINE	MEDIUM	COARSE	GRAVEL
		SAND			

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Fig. E.3.34: Grain size distribution curve of sediment sample at the location AA-1

Test sieve size [mm]	Retained quantity g	Retained quantity %	Total passing %
4	0.000	0.00	100.00
2	6.143	1.27	98.73
1	0.867	0.18	98.55
0.5	0.568	0.12	98.44
0.25	8.660	1.79	96.65
0.1	407.904	84.12	12.53
0.05	59.108	12.19	0.34
Rest in box	1.623	0.33	
Total	484.873		

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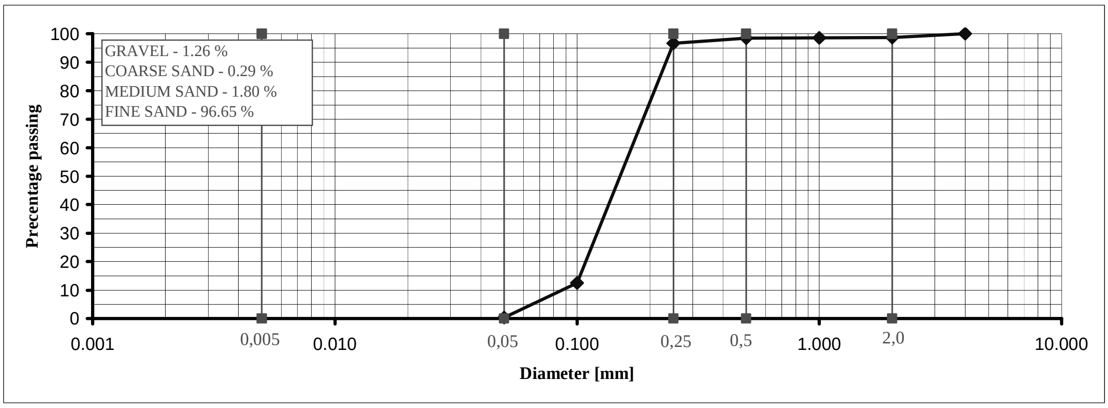
Date started:
Location: -

Drill No.: -
Tube No.: -
Sample No.: AA2-05
Depth [m]: -

DESCRIPTION AND PRETREATMENT		SEDIMENTATION		SIEVING		NOTES
Description	Pretreatment	Mass g	Particle density	Mass g	Process	
Sand	-	-	-	484.9	mechanical	- Tested in accordance with romanian norms STAS 1913/5-85 - * indicates particle density assumed



CLAY	SILT	FINE SAND	MEDIUM SAND	COARSE SAND	GRAVEL

REMARKS

- medium sand is in fact a mixture of sand and shell fragments
- coarse sand and gravel is in fact shell fragments and rarely shell fragments with gravel

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Fig. E.3.35: Grain size distribution curve of sediment sample at the location AA2-5

Test sieve size [mm]	Retained quantity g	Retained quantity %	Total passing %
4	0.853	0.27	99.73
2	6.082	1.94	97.79
1	3.698	1.18	96.62
0.5	4.703	1.50	95.12
0.25	4.368	1.39	93.73
0.1	217.895	69.33	24.40
0.05	73.870	23.50	0.90
Rest in box	2.745	0.87	
Total	314.214		

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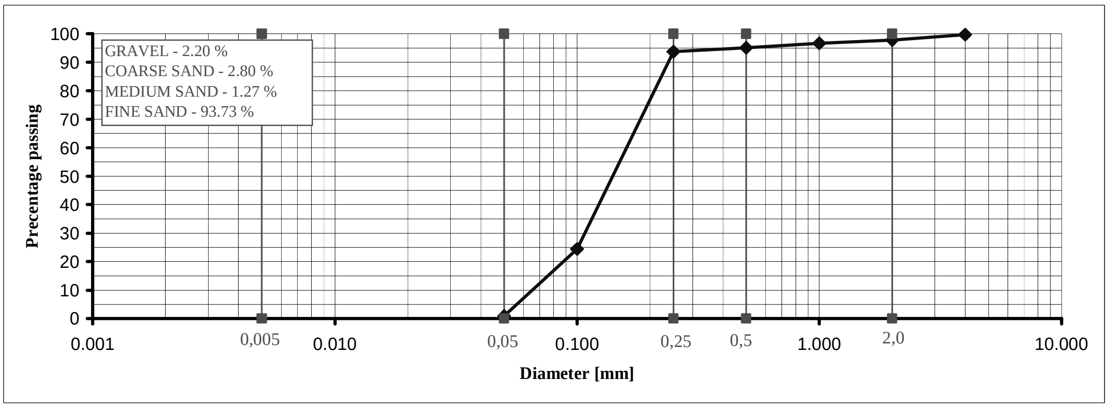
Date started:
Location: -

Drill No.: -
Tube No.: -
Sample No.: AA2-10
Depth [m]: -

DESCRIPTION AND PRETREATMENT		SEDIMENTATION		SIEVING		NOTES
Description	Pretreatment	Mass g	Particle density	Mass g	Process	
Sand	-	-	-	314.3	mechanical	- Tested in accordance with romanian norms STAS 1913/5-85 - * indicates particle density assumed



CLAY	SILT	FINE SAND	MEDIUM SAND	COARSE SAND	GRAVEL

REMARKS

- medium sand is in fact a mixture of sand and shell fragments
- coarse sand and gravel is in fact shell fragments and rarely shell fragments with gravel

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Fig. E.3.36: Grain size distribution curve of sediment sample at the location AA2-10

Test sieve size [mm]	Retained quantity g	Retained quantity %	Total passing %
4			
2			
1			
0.5			
0.25	0.421	2.07	97.93
0.1	2.900	14.29	83.64
0.08	5.906	29.09	54.55
Rest in box	11.065	54.51	
Total	20.292		

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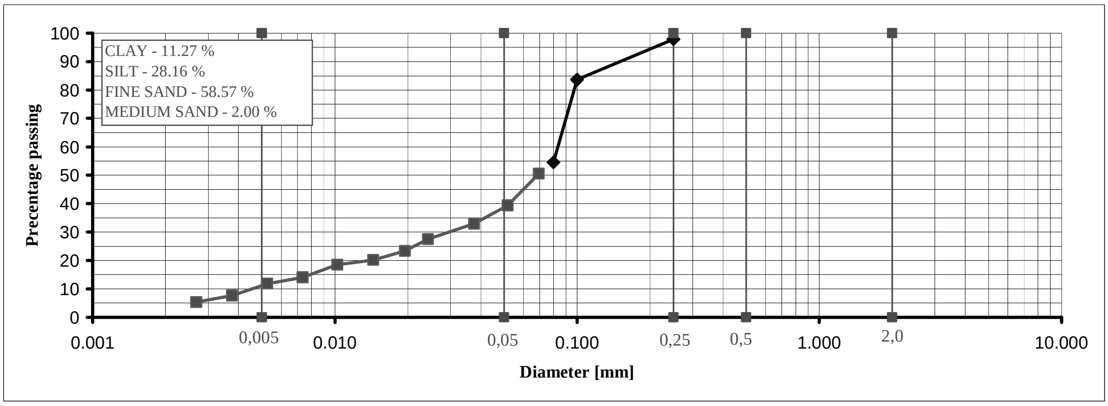
Date started:
Location: -

Drill No.: -
Tube No.: -
Sample No.: AA2-20
Depth [m]: -

DESCRIPTION AND PRETREATMENT		SEDIMENTATION		SIEVING		NOTES
Description	Pretreatment	Mass g	Particle density	Mass g	Process	
Sand	Li_2CO_3	50.054	2.654	20.3	mechanical	- Tested in accordance with romanian norms STAS 1913/5-85 - * indicates particle density assumed



CLAY	SILT	FINE SAND	MEDIUM SAND	COARSE SAND	GRAVEL

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Fig. E.3.37: Grain size distribution curve of sediment sample at the location AA2-20

Test sieve size [mm]	Retained quantity g	%	Total passing %
4	10.094	3.18	96.82
2	11.733	3.70	93.13
1	12.051	3.80	89.33
0.5	11.057	3.48	85.85
0.25	12.815	4.04	81.81
0.1	190.678	60.06	21.75
0.05	64.585	20.34	1.41
Rest in box	4.403	1.39	
Total	317.416		

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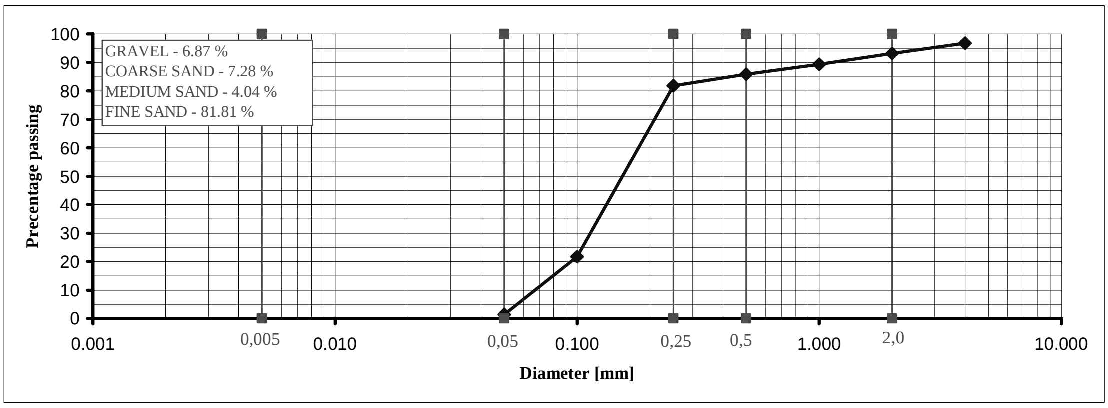
Date started:
Location: -

Drill No.: -
Tube No.: -
Sample No.: AA3-05
Depth [m]: -

DESCRIPTION AND PRETREATMENT		SEDIMENTATION		SIEVING		NOTES
Description	Pretreatment	Mass g	Particle density	Mass g	Process	
Sand	-	-	-	317.5	mechanical	- Tested in accordance with romanian norms STAS 1913/5-85 - * indicates particle density assumed



CLAY	SILT	FINE	MEDIUM	COARSE	GRAVEL
		SAND			

REMARKS

- medium sand is in fact a mixture of sand and shell fragments
- coarse sand and gravel is in fact shell fragments and rarely shell fragments with gravel

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Fig. E.3.38: Grain size distribution curve of sediment sample at the location AA3-05

Test sieve size [mm]	Retained quantity g	Retained quantity %	Total passing %
4	15.027	2.68	97.32
2	27.023	4.81	92.51
1	28.921	5.15	87.36
0.5	25.170	4.48	82.88
0.25	28.078	5.00	77.88
0.1	329.896	58.74	19.14
0.05	98.747	17.58	1.56
Rest in box	8.707	1.55	
Total	561.569		

PARTICLE SIZE DISTRIBUTION

Drill No.: -

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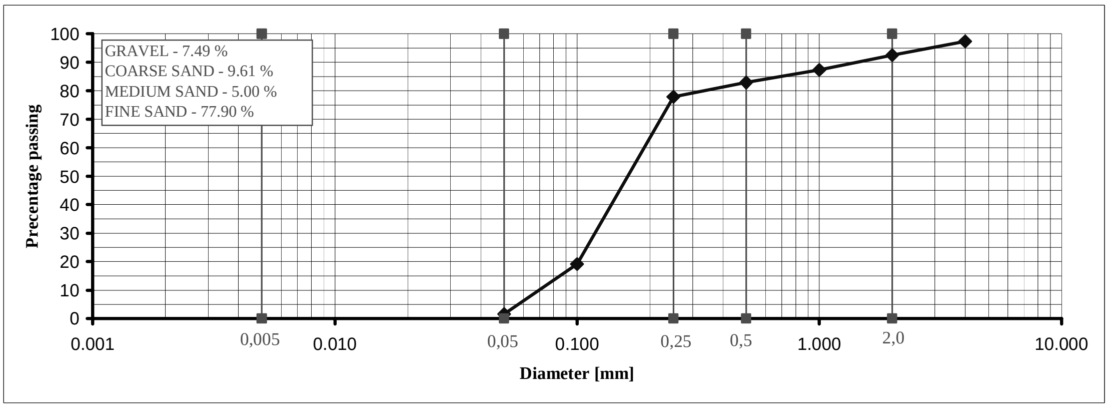
CIVIL ENGINEERING FACULTY Location: - Sample No.: AA3-10

Geotechnical Laboratory Gr.II GTF, ANCFD Depth [m]: -

Licence No. 389/ISC

DESCRIPTION AND PRETREATMENT		SEDIMENTATION		SIEVING		NOTES
Description	Pretreatment	Mass g	Particle density	Mass g	Process	
Sand	-	-	-	561.6	mechanical	- Tested in accordance with romanian norms STAS 1913/5-85 - * indicates particle density assumed



CLAY	SILT	FINE SAND	MEDIUM SAND	COARSE SAND	GRAVEL

REMARKS

- medium sand is in fact a mixture of sand and shell fragments
- coarse sand and gravel is in fact shell fragments and rarely shell fragments with gravel

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Fig. E.3.39: Grain size distribution curve of sediment sample at the location AA3-10

Test sieve size [mm]	Retained quantity g	Retained quantity %	Total passing %
4	72.986	17.79	82.21
2	66.100	16.11	66.09
1	45.297	11.04	55.05
0.5	20.070	4.89	50.16
0.25	20.040	4.89	45.27
0.1	83.141	20.27	25.00
0.05	68.230	16.63	8.37
Rest in box	34.260	8.35	
Total	410.124		

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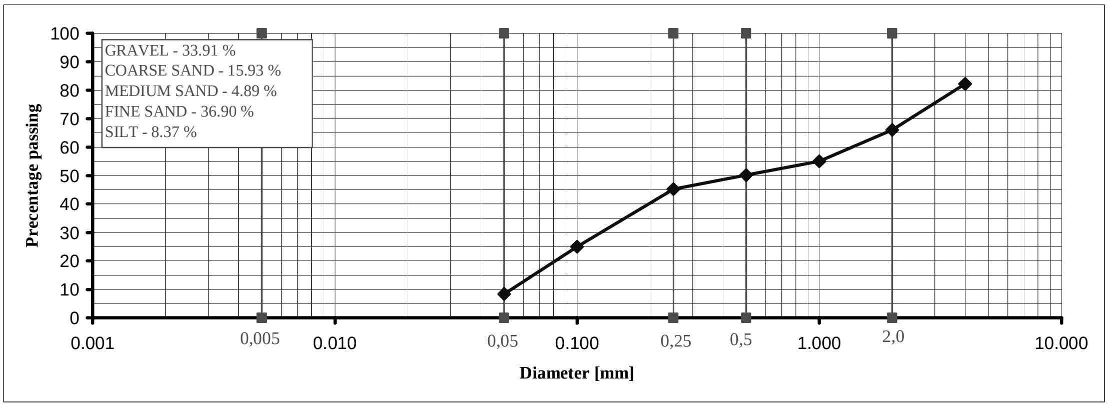
Date started:
Location: -

Drill No.: -
Tube No.: -
Sample No.: AA3-20
Depth [m]: -

DESCRIPTION AND PRETREATMENT		SEDIMENTATION		SIEVING		NOTES
Description	Pretreatment	Mass g	Particle density	Mass g	Process	
Sand	-	-	-	410.2	mechanical	- Tested in accordance with romanian norms STAS 1913/5-85 - * indicates particle density assumed



CLAY	SILT	FINE	MEDIUM	COARSE	GRAVEL
		SAND			

REMARKS
- medium sand is in fact a mixture of sand and shell fragments
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Fig. E.3.40: Grain size distribution curve of sediment sample at the location AA3-20

Test sieve size [mm]	Retained quantity g	Retained quantity %	Total passing %
4	14.746	3.36	96.64
2	19.410	4.43	92.21
1	16.921	3.86	88.35
0.5	10.273	2.34	86.01
0.25	22.373	5.10	80.91
0.1	184.415	42.05	38.87
0.05	162.168	36.97	1.89
Rest in box	8.205	1.87	
Total	438.511		

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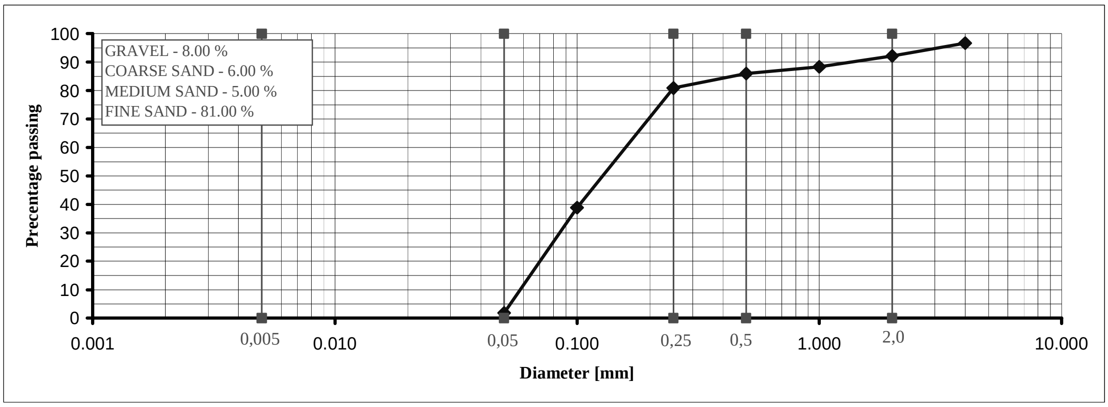
Date started:
Location: -

Drill No.: -
Tube No.: -
Sample No.: AA4-20
Depth [m]: -

DESCRIPTION AND PRETREATMENT		SEDIMENTATION		SIEVING		NOTES
Description	Pretreatment	Mass g	Particle density	Mass g	Process	
Sand	-	-	-	438.6	mechanical	- Tested in accordance with romanian norms STAS 1913/5-85 - * indicates particle density assumed



CLAY	SILT	FINE SAND	MEDIUM SAND	COARSE SAND	GRAVEL

REMARKS
- medium sand is in fact a mixture of sand and shell fragments
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Fig. E.3.41: Grain size distribution curve of sediment sample at the location AA4-20

Test sieve size [mm]	Retained quantity g	Retained quantity %	Total passing %
Rest in box			
Total			

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Date started:
Location: -

Drill No.: -
Tube No.: -
Sample No.: AA5-1
Depth [m]: -

DESCRIPTION AND PRETREATMENT		SEDIMENTATION		SIEVING		NOTES
Description	Pretreatment	Mass g	Particle density g/cm^3	Mass g	Process	
Sand	Li_2CO_3	50.222	2.581	-	-	- Tested in accordance with romanian norms STAS 1913/5-85 - * indicates particle density assumed

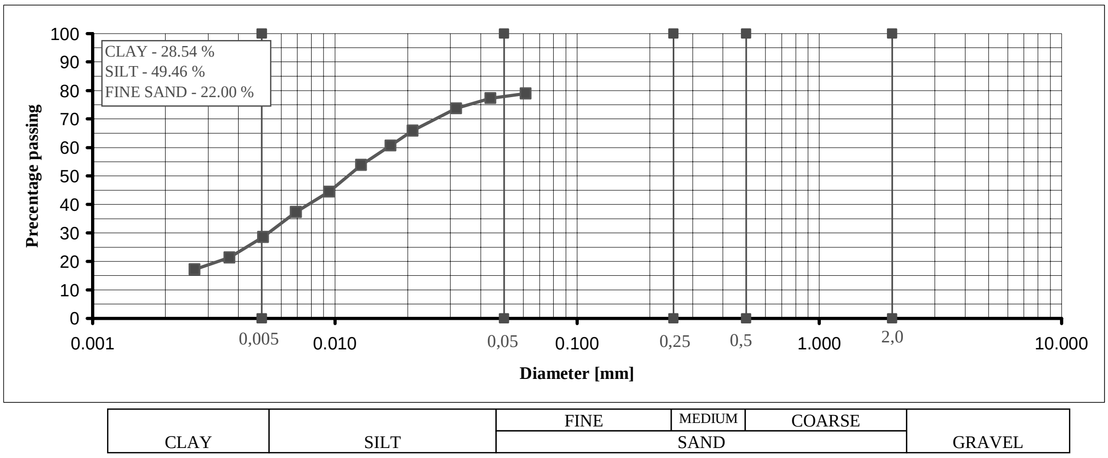

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Fig. E.3.42: Grain size distribution curve of sediment sample at the location AA5-1

Test sieve size [mm]	Retained quantity g	Retained quantity %	Total passing %
4	92.411	26.10	73.90
2	91.811	25.93	47.97
1	73.826	20.85	27.13
0.5	55.558	15.69	11.44
0.25	26.463	7.47	3.96
0.1	12.888	3.64	0.32
0.05	0.901	0.25	0.07
Rest in box	0.168	0.05	
Total	354.026		

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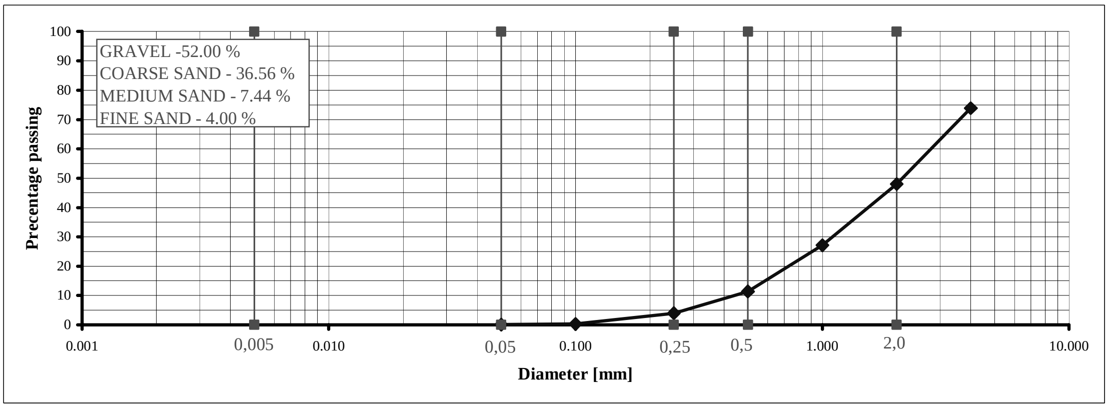
Date started:
Location: -

Drill No.: -
Tube No.: -
Sample No.: AA5-2
Depth [m]: -

DESCRIPTION AND PRETREATMENT		SEDIMENTATION		SIEVING		NOTES
Description	Pretreatment	Mass g	Particle density	Mass g	Process	
Sand	-	-	-	354.10	mechanical	- Tested in accordance with romanian norms STAS 1913/5-85 - * indicates particle density assumed



CLAY	SILT	FINE SAND	MEDIUM SAND	COARSE SAND	GRAVEL

REMARKS
- medium sand is in fact a mixture of sand and shell fragments
- coarse sand and gravel is in fact shell fragments and rarely shell fragments with gravel

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Fig. E.3.43: Grain size distribution curve of sediment sample at the location AA5-2

Test sieve size [mm]	Retained quantity g	Retained quantity %	Total passing %
4	5.673	1.41	98.59
2	1.205	0.30	98.30
1	1.795	0.44	97.85
0.5	2.325	0.58	97.28
0.25	5.538	1.37	95.90
0.1	303.101	75.08	20.82
0.05	80.642	19.98	0.85
Rest in box	3.392	0.84	
Total	403.671		

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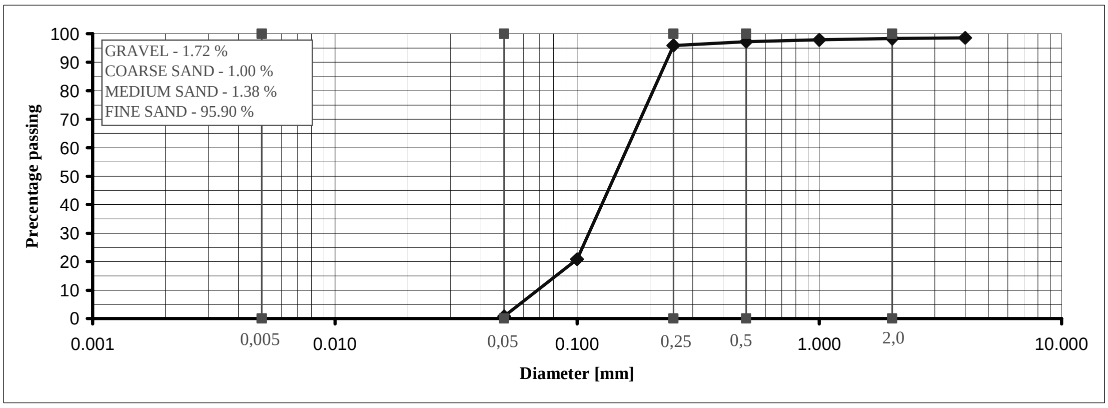
Date started:
Location: -

Drill No.: -
Tube No.: -
Sample No.: BB-1
Depth [m]: -

DESCRIPTION AND PRETREATMENT		SEDIMENTATION		SIEVING		NOTES
Description	Pretreatment	Mass g	Particle density	Mass g	Process	
Sand	-	-	-	403.7	mechanical	- Tested in accordance with romanian norms STAS 1913/5-85 - * indicates particle density assumed



CLAY	SILT	FINE	MEDIUM	COARSE	GRAVEL
		SAND			

REMARKS

- medium sand is in fact a mixture of sand and shell fragments
- coarse sand and gravel is in fact shell fragments and rarely shell fragments with gravel

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Fig. E.3.44: Grain size distribution curve of sediment sample at the location BB-1

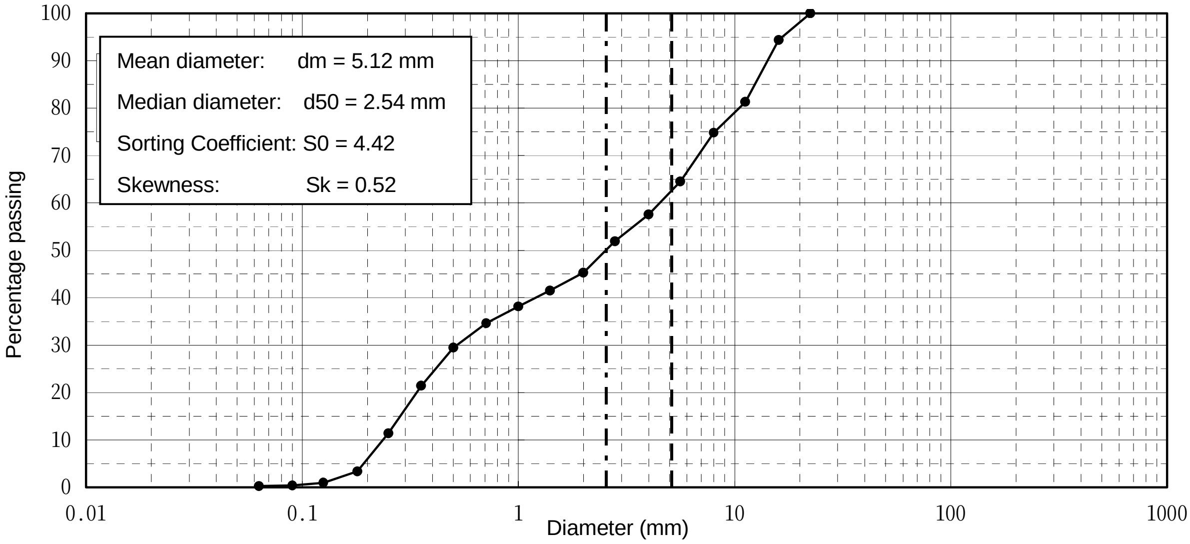


Fig. E.3.45: Grain size distribution curve of sediment sample at the location R-1 (Ostrov, 340km point)

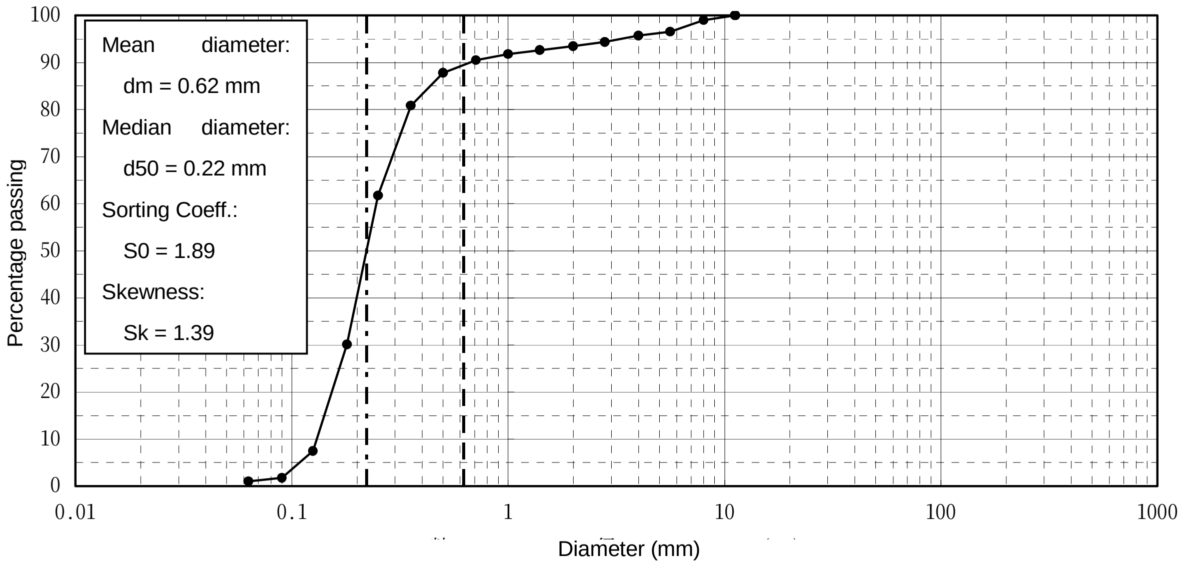


Fig. E.3.46: Grain size distribution curve of sediment sample at the location R-2 (Cochirleni, 305km point)

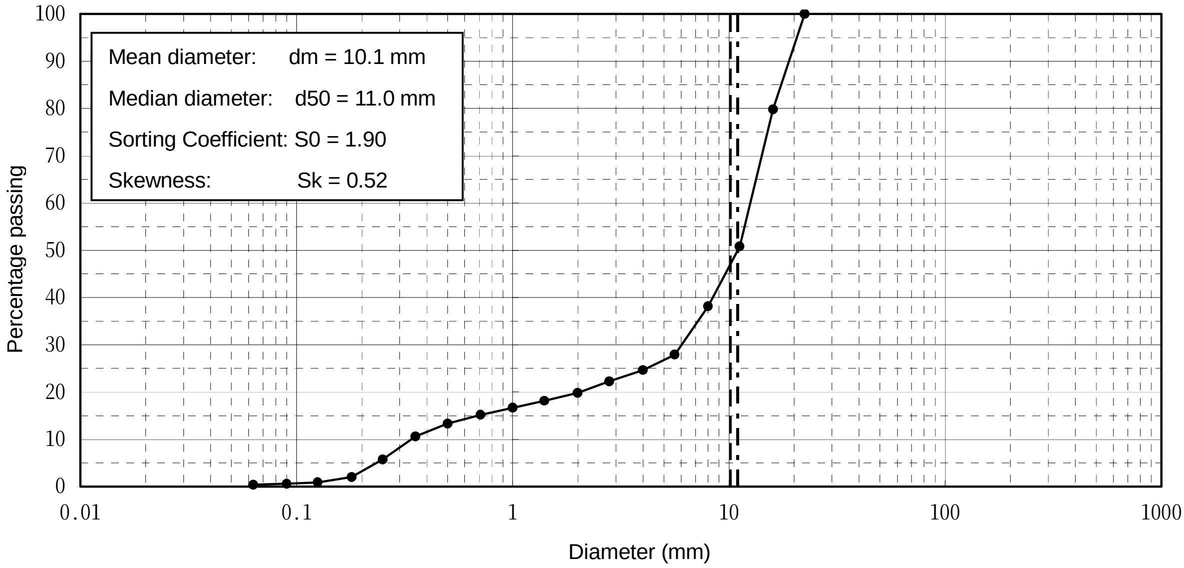


Fig. E.3.47: Grain size distribution curve of sediment sample at the location R-3
(Călăraşi, 340km point)

E.4 Estimate of Volumetric Change of Beach Profiles

E.4.1 Definition of Beach Profile Volume

The National Institute for Marine Research Development "Grigore Antipa" (NIMRD) provided the Team with the data of foreshore profiles together with the data of shoreline position changes, which were analyzed in **E.2**. The foreshore profile data are comprised of the elevations at several locations on foreshore with the distance from the benchmark. Because it was reported that some elevation data near the benchmark were not accurate enough, calculation of the foreshore volume above the mean water level S_D is made with the reference line at the initial shoreline that is taken at the most landward position among the measurements over years.

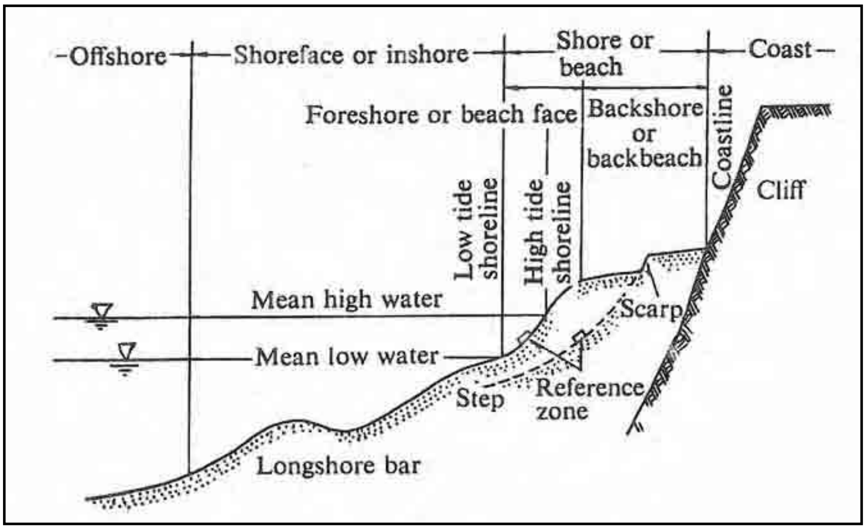


Fig. E.4.1: Illustration of terminology defining various parts of beach profile

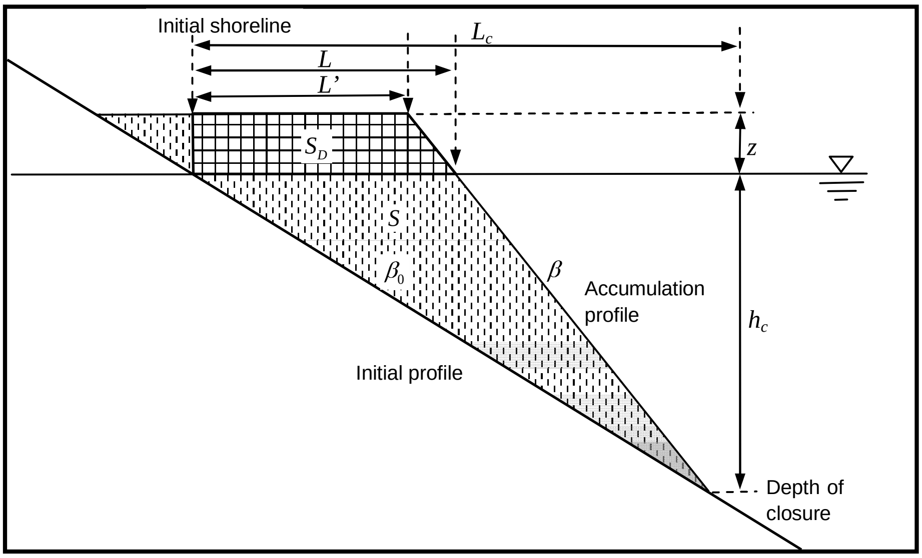


Fig. E.4.2: Definition sketch of beach profile volume

In the present section, several technical terms are used and Fig. E.4.1 illustrates the terminology concerning the beach profile. The data provided by NIMRD include the advance distance of the shoreline from the initial position as sketched in Fig. E.4.2

E.4.2 Relationship between Shoreline Advance and Beach Volume Change

An initial analysis has been made for the relationship between the foreshore volume S_D and the shoreline advance distance L. Figure E.4.3 is the result obtained for the location MM-12 (Mamaia South). The volume S_D can be expressed as a function of the distance L as follows:

$$S_D = aL^2 + bL \tag{E.4.1}$$

where a and b take the values of 0.02 and 0.22, respectively. For other benchmark locations, the values of a and b have been obtained as listed in Table E.4.1.

Table E.4.1: Constants a and b for foreshore volume S_D

Loc.	*a*	*b*	Loc.	*a*	*b*	Loc.	*a*	*b*	Loc.	*a*	*b*
MM-1	0.0100	0.170	MM-10	0.0100	0.1500	EF-4	0.0046	1.3299	NN-3	0.0169	0.3981
MM-2	0.0024	0.427	MM-11	0.0200	0.1900	EF-5	0	0.7446	SN-1	0.0148	0.2225
MM-3	0.0226	-0.0366	MM-12	0.0200	0.2200	EF-6	0.0082	1.0367	SN-2	0.0080	0.4641
MM-4	0.0166	0.1098	MM-13	0.0100	0.3900	EF-7	0.0291	0.1393	MG-1	0.0062	0.3459
MM-5	0.0322	-0.1232	MM-14	0.0100	0.0570	CN-1	0.0001	0.5221	MG-2	0.0242	0.1722
MM-6	0.0114	0.0633	MM-15	0.0007	0.5287	CN-2	0.0098	0.2309	MI-1	0.0196	0.2492
MM-7	0.0119	0.4895	EF-1	0.0100	0.3767	CN-3	0.0168	0.3431	VV-1	0	0.7845
MM-8	0.0186	-0.1077	EF-2	0.0065	0.4053	NN-1	0.0153	0.4896	–	–	–
MM-9	0.0145	-0.0162	EF-3	0.0188	0.3857	NN-2	0.0111	0.4513	–	–	–

Because the volume increase is proportional to the square of the advance distance, the mean height z of the foreshore is considered to be proportional to the shoreline advance distance.

It is assumed that the total beach volume S inclusive of the volume S' below the mean water level increases in a triangular shape with a change in the beach slope β_0 to β as shown in Fig. E.4.1.

It is further assumed that the increase in beach volume is closed at the depth h_c, which is called the depth of closure. Then, the distance L_c between the initial shoreline and the location of the depth of closure is expressed by

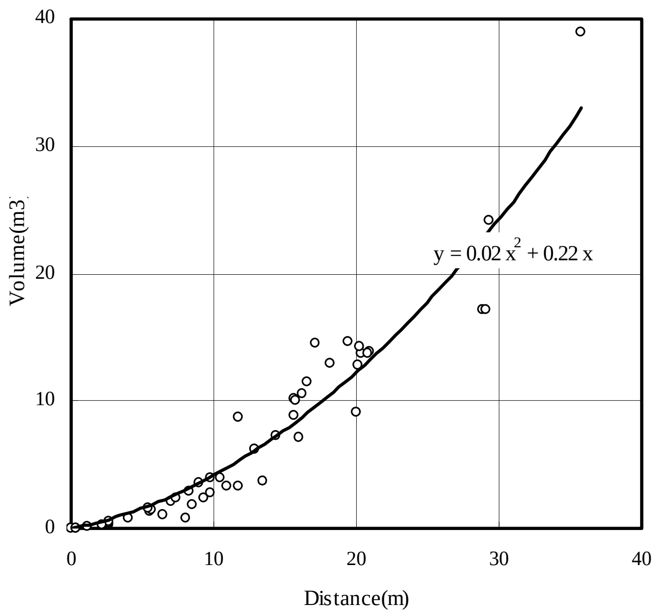


Fig.E.4.3: Shoreline distance vs. foreshore volume at Mamaia South (MM-12)

$$L_c = h_c / \beta_0 \tag{E.4.2}$$

The mean height z of the foreshore above the mean water level is derived by similitude of triangles by introducing the foreshore width L' as below.

$$\left. \begin{aligned} z &= \frac{L - L'}{L_c - L} h_c \\ L' &= L - \frac{z}{h_c}(L_c - L) = L - \frac{z}{\beta} \end{aligned} \right\} \tag{E.4.3}$$

where β denotes the beach slope after the shoreline advance. It is expressed as

$$\beta = \frac{h_c}{L_c - L} = \frac{z}{L - L'} \tag{E.4.4}$$

The foreshore volume S_D can be written in the following form:

$$S_D = \frac{L + L'}{2} z = \frac{L + L'}{2} \cdot (L - L')\beta = \frac{\beta}{2}\left(L^2 - L'^2\right) \tag{E.4.5}$$

The foreshore width L' after the shoreline advance is obtained from Eqs. E.4.4 and E.4.5 as

$$L' = \sqrt{L^2 - 2\frac{S_D}{\beta}} = \sqrt{L^2 - 2\frac{S_D(L_c - L)}{h_c}} \tag{E.4.6}$$

The mean foreshore elevation z is then expressed with the foreshore volume S_d by substituting Eq. E.4.6 into Eq. E.4.4 as follows:

$$z = \beta\left(L - \sqrt{L^2 - 2\frac{S_D}{\beta}}\right) \tag{E.4.7}$$

The total beach volume S after the shoreline advance by the distance L is calculated as

$$S = \frac{1}{2} L h_c \left(\frac{h_c + z}{h_c}\right)^2 \tag{E.4.8}$$

In derivation of Eq. E.4.8, a small backshore volume in the landward side of the initial shoreline is neglected to simplify calculation.

E.4.3 Depth of Closure and Limit Depth of Sediment Transport

For evaluation of the total beach volume S by means of Eq. E.4.8, information is needed on the depth of closure of sediment transport h_c. This depth depends on the wave climate of the locality of interest. In 1978, Hallermeier[1] proposed the following empirical equation for its

[1] Hallermeier, R.J.: Uses for a calculated limit depth to beach erosion, *Proc. 16th Int. Conf. Coastal Eng.*,

estimation based on the analysis of the relationship between the largest annual wave and the water depth beyond which no significant bathymetric change seems to take place by that wave:

$$h_c = 2.28H_e - 68.5\left(\frac{H_e^2}{gT_e^2}\right) \tag{E.4.9}$$

where H_e and T_e are the height and period of the so-called effective wave derived from the records of wave observations over many years. The effective wave height is calculated by

$$H_e = \overline{H} + 5.6\sigma_H \tag{E.4.10}$$

where $\overline{H}$ and σ_H denote the mean and standard deviation of significant wave height over years. The effective wave period is calculated similarly. Waves exceeding this height is said to appear about 12 hours per year.

Another formula for estimation of the depth of closure has been proposed by Sato and Tanaka[2] in 1966:

$$\frac{H_0}{L_0} = 2.4\left(\frac{d}{L_0}\right)^{1/3}\left(\sinh\frac{2\pi h_i}{L}\right)\frac{H_0}{H} \tag{E.4.11}$$

where d is the representative diameter of sediment grains, h_i is the limit depth of sediment transport, H and T are the significant wave height and period, respectively, and the subscript $_0$ refers to deepwater waves. Practical applications of this equation are often made with the wave height at the cumulative probability of 99% or 99.5%, which correspond to about 88 hours or 44 hours per year, respectively. Table E.4.2 lists the results of wave height calculation with Eq. E.4.10 as well as the wave climate data listed in Tables 3.4.1 and 3.4.2 in **3.4**.

The wave periods in Table E.4.1 are estimated with the empirical formula of $T = 3.3H^{0.63}$, which is fitted to the relationship between the heights and periods of extreme waves as shown in Fig. D.5.2.

Table E.4.2: Wave heights for calculation of the depth of closure and limit depth of sediment transport

Definition of waves	Wave height (m)	Wave period (s)
Effective wave height	5.0	9.1
Waves with 99.5% probability	4.3	8.2
Waves with 99% probability	3.7	7.5

The depth of closure is calculated with Eq. E.4.9. The depth h_c is directly calculated with the wave climate data only without the information of sediment grain size. It is obtained as h_c =

ASCE, Hamburg, pp. 1493-1512, 1978.

2 Sato, S. and Tanaka, N.: Field investigation on sand drift at Port Kashima facing the Pacific Ocean, *Proc. 10th Int. Conf. Coastal Eng.*, ASCE, Tokyo, 1966. (cited in "*Technical Standards for Port and Harbour Facilities in Japan with Commentaries*" by OCDI, 2002, p. 157)

9.3 m. The limit depth of sediment transport is calculated with Eq. E.4.11 using the mean value of d = 0.2 mm for the sub-sectors of Navodari–Mamaia, Tomis North, and Tomis South and d = 0.4 mm for the sub-sector of Eforie Nord and further southern sub-sectors (see Fig. 4.3.1 in **4.3**). The result of calculation for the limit depth of sediment transport is calculated as listed in Table E.4.3.

Table E.4.3: Depth of closure and limit depth of sediment transport

Sub-sectors	Sand grain size, d (mm)	Depth of closure, h_c (m)	Limit depth of sediment transport	
			h_i [99.5%] (m)	h_i [99%] (m)
Navodari to Tomis	0.2	9.3	13.8	11.2
Eforie to Vama Veche	0.4	9.3	10.7	8.6

Majority of sediment movement in the field seems to be taking place in the water shallower than the depth of closure given by Eq. E.4.9. It was confirmed during sediment sampling from the seabed that no trace of sand deposition was found at the depth 10 m offshore Eforie and Costineşti, which indicates no cross-shore sediment to that depth. In consideration of these factors, the limit depth of sediment transport is determined at h_c= 9.3 m for the sub-sectors of Navodari to Tomis. For the sub-sectors from Eforie to Vama Veche, it is decided to use h_c = 7.1 m in consideration of the coarse sediment grain size in these sub-sectors by multiplying the ratio 0.77 = 8.6/11.3 to the depth 9.3 m.

E.4.4 Cross-Shore Distance to the Water of Limit Depth of Sediment Transport

The distance from the shoreline to the water with the limit depth of sediment transport is estimated with the mean slope of the inshore zone, which was evaluated from the bathymetric charts prepared by PROIECT S.A. in 1997, as listed in Table E.4.4, where L_c denotes the cross-shore distance. The distance from the shoreline to the depth contour of 4 or 6 m was used to calculate the mean slope.

The cross-shore distance L_c can be regarded as the minimum length of jetty that will completely check the longshore transport of sediment.

Table E.4.4: Representative distance from the shoreline to the limit depth of sediment transport

Sub-sector name	Slope, $\tan\beta$	Depth, h_c (m)	Distance, L_c (m)
Navodari – Mamaia	1/100	9.3	930
Tomis	1/100	9.3	930
Eforie Nord – Sud	1/90	7.1	639
Costineşti	1/50	7.1	355
Olimp – Venus	1/50	7.1	355
Saturn – Mangalia	1/80	7.1	568
2 Mai – Vama Veche	1/70	7.1	497

The estimate of the cross-shore distance for the sub-sectors of Mamaia Center and South needs to be revised, because the wave height incident to the beach of these sub-sectors is reduced from the offshore height by the wave damping effect of the detached breakwaters built in the late 1980s. The breakwaters are made of 25-ton stabilopods and deformed concrete cubes installed at the depth of about 5 m. They were designed with the crest elevation of 2 m and the crest width of 10 m, but they have subsided and stabilopods were dislocated by waves since then. Presently barely one leg of stabilopods is emerging above the mean water level on the average.

The wave height behind the breakwaters is evaluated for the offshore wave of 99% cumulative probability, i.e., H_0 = 3.7 m and T = 7.5 s. The wave will be affected by the random breaking process in front of the breakwater. According to the design diagrams of wave height variation for the bottom slope of 1/100 by Goda[3], the wave height in front of the breakwater is estimated as H_I = 2.85 m. The wave height behind the breakwater H_T can be estimated with the following empirical formula by Iwasaki and Numata:

$$\left.\begin{aligned} H_T &= K_T H_I \\ K_T &= 1/\left[1 + K(H_I\ /\ L)^{0.5}\right]^2 \\ K &= 1.135(B/D)^{0.65} \end{aligned}\right\} \tag{E.4.12}$$

where K_T denotes the transmission coefficient, B is the breakwater width at the mean water level, and D is the representative height of concrete block.

Table E.4.5: Wave height behind Mamaia breakwater and limit depth of sediment transport of the sub-sectors of Mamaia Center and Mamaia South

Conditions	H_I (m)	K_T	H_T (m)	d (mm)	h_c (m)	$\tan\beta$	L_c (m)
As designed (B = 13 m, D = 2.0 m)	2.85	0.3	0.86	0.2	1.5	1/100	150
As of present (B = 1 m, D = 2.0 m)	2.85	0.7	2.00	0.2	4.5	1/100	450

The breakwater width is assumed to be B = 13 m when it was built and B = 1 m at the present, and the representative height of stabilopods is taken as D = 2.0 m. With these conditions, the wave height behind the detached breakwaters and the limit depth of sediment transport for the sub-sectors of Mamaia Center and South are estimated as listed in Table E.4.5.

The limit depth and the cross-shore distance of the sub-sectors other than Mamaia Center and Mamaia South remain as listed in Table E.4.4.

E.4.5 Volumetric Change of Beach Profile

With the data of the depth of closure h_c and the cross-shore distance L_c having been obtained as listed in Tables E.4.4 and E.4.5, the rate of volumetric change of beach profile can now be estimated as follows. Let us take an example of calculation for the location MM-12. The annual rate of shoreline retreat is analyzed as –0.7 m per year as shown in Fig. 4.2.3 in **4.2**. By assuming the trend continues for another 10 years, the shoreline advance distance is L = –7.0 m. The foreshore volume corresponding to this distance is calculated with Eq. E.4.1 as S_D = 2.5 m^3/m; the absolute value of L = 7 m is used because Eq. E.4.1 and the constants a

[3] See Chapter 3 of "*Random Seas and Design of Maritime Structures* (2nd Ed.)," by Y. Goda published in 2000 by World Scientific, Singapore.

and *b* in Table E.4.1 have been obtained with the assumption of accretion process. By substituting L = 7.0, S_D = 2.5 and b = 1/100 into Eq. E.4.7, the mean foreshore elevation z is calculated as z = 0.30 m. The beach volume below the mean water level S' is

$$S' = L\,h/2 = 7.0 \times 4.5/2 = 15.75 \quad \text{m}^3/\text{m}$$

The volumetric change (accretion or erosion) of beach profile is estimated with Eq. E.4.8 as follows:

$$S = S'\left(\frac{h_c + z}{h_c}\right)^2 = 15.75 \times \left(\frac{4.5 + 0.3}{4.5}\right)^2 = 17.95 \quad \text{m}^3/\text{m}$$

Table E.4.6: Estimate of volumetric change of beach profile during 10 years

	L	SD	hc	cotβ	Lc	z	S
MM_1	-5.8	-1.4	9.3	100	930	0.2	-28.3
MM_2	10.2	4.6	9.3	100	930	0.4	51.8
MM_3	-0.24	0.0	9.3	100	930	0.0	-1.1
MM_4	2.8	0.1	9.3	100	930	0.1	13.3
MM_5	-4.1	0.0	9.3	100	930	0.1	-19.4
MM_6	-1.3	-0.1	9.3	100	930	0.1	-6.1
MM_7	-17.3	-12.0	9.3	100	930	0.7	-92.9
MM_8	-7.2	-0.2	9.3	100	930	0.2	-34.7
MM_9	10	1.3	4.5	100	450	0.3	25.5
MM_10	2.7	0.5	4.5	100	450	0.1	6.4
MM_11	-3.6	-0.9	4.5	100	450	0.2	-8.8
MM_12	-7	-2.5	4.5	100	450	0.3	-18.0
MM_13	-19.2	-9.4	4.5	100	450	0.7	-56.9
MM_14	-22.5	-15.9	4.5	100	450	0.8	-71.1
MM_15	-19.4	-10.5	4.5	100	450	0.7	-58.1
EF_1	27.4	17.9	7.1	90	639	1.0	126.7
EF_2	6.2	2.8	7.1	90	639	0.3	24.1
EF_3	-0.1	0.0	7.1	90	639	0.0	-0.4
EF_4	-0.7	-0.9	7.1	90	639	0.2	-2.6
EF_5	0.8	0.6	7.1	90	639	0.1	2.9
EF_6	-24.8	-30.8	7.1	90	639	1.1	-118.8
EF_7	-8.6	-3.4	7.1	90	639	0.4	-33.9
CN_1	0.9	0.5	7.1	50	355	0.2	3.3
CN_2	-2	-0.5	7.1	50	355	0.2	-7.5
CN_3	0.9	0.3	7.1	50	355	0.1	3.3
NN_1	-13.2	-9.1	7.1	50	355	0.9	-59.8
NN_2	-1.2	-0.6	7.1	50	355	0.2	-4.5
NN_3	0.6	0.2	7.1	50	355	0.1	2.2
SN_1	-16	-7.4	7.1	80	568	0.7	-68.1
SN_2	-12	-6.7	7.1	80	568	0.6	-49.9
MG_1	-7.4	-2.9	7.1	80	568	0.4	-29.1
MG_2	-7.7	-2.8	7.1	80	568	0.4	-30.3
MI_1	-6	-2.2	7.1	70	497	0.4	-23.5
VV_1	-7.4	-5.8	7.1	70	497	0.5	-30.3

This value is for the change for 10 years. The volumetric change per year is one tenth of this value, i.e., 1.8 m^3/m/year. Table E.4.6 lists the results of the estimate of volumetric change of beach profiles at the 34 locations along the Study area.

The total volume of the change of beach profile in a sub-sector will be obtained by multiplying the annual change rate to the length of the respective sub-sector.

E.5 Methodology of Shoreline Change Simulation

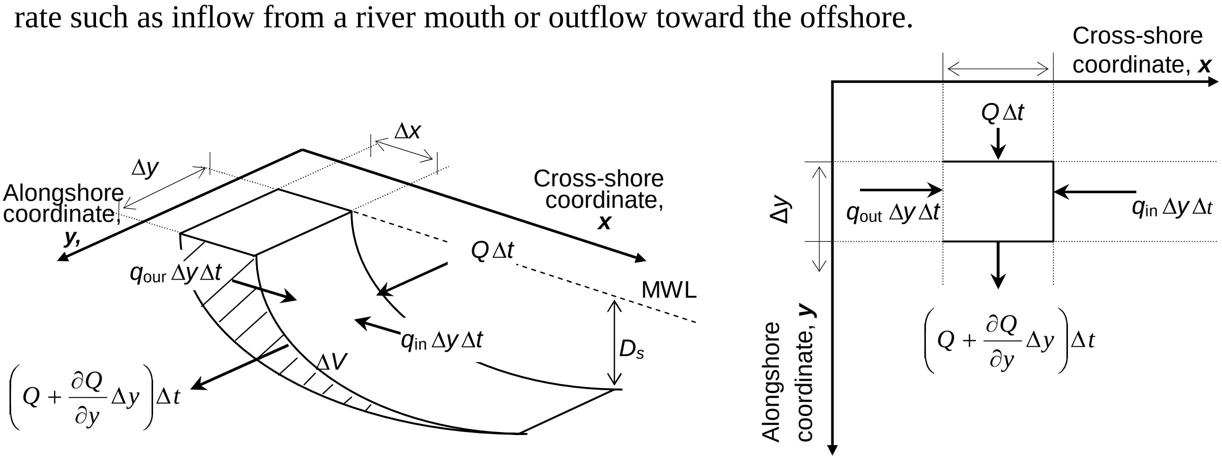
E.5.1 Fundamental Equation of Simulation Model

There have been developed several mathematical models for simulating beach morphological changes by waves and currents. An optimum model is selected in consideration of the availability of field beach data for calibration, the size of coastal zone to be simulated, the computation load and others. One of the sophisticated models is a 3-D model, which is being advanced by several researchers. However, the 3-D model requires the well-surveyed bathymetric data over a lapse of certain years for calibration, which are not available for the Study. Furthermore, it is not practical to use it for the coastal area extending over a few tens of kilometers because of too large computation load.

Most computations of the shoreline change due to wave actions are currently carried out by means of the so-called one-line theory, which has been so named because the advance or retreat of beaches is represented with the position change of the shoreline, i.e. one-line. The inherent assumption is that the beach profile remains unchanged regardless of the movement of the shoreline position and the shoreline change is caused by the alongshore sediment transport. Therefore, the model assumes no presence of cliffs and/or seawalls that interfere with the shoreline position change and prevent free retreat of the shoreline. The fundamental equation of the one-line theory is expressed by Eq. E.5.1.

$$\frac{\partial x_s}{\partial t} + \frac{1}{D_s}\left(\frac{\partial Q}{\partial y} - q\right) = 0 \qquad \text{(E.5.1)}$$

where the coordinate x is taken offshore from the shoreline, y is the alongshore distance, x_s is the temporary location of the shoreline measured from the baseline, D_s is the thickness of sediment layer that is regarded to move together with the shoreline (depth of closure with addition of backshore height), Q is the longshore sediment transport rate converted to in-situ volume with consideration of sand porosity, and q denotes the cross-shore sediment transport rate such as inflow from a river mouth or outflow toward the offshore.



(a) Schematic View (b) Plan view

Fig. E.5.1: Conceptual presentation of shoreline change model

E.5.2 Alongshore Sediment Transport Rate

The alongshore sediment transport rate is estimated with the following formula due to Ozasa and Brampton[4]:

$$Q = \frac{1}{(\rho_s - \rho)g(1-\lambda)}(EC_g)_b\left(K_1 \sin\alpha_b \cos\alpha_b - K_2 \frac{\partial H_b}{\partial y}\cot\beta\cos\alpha_b\right) \tag{E.5.2}$$

$$E_b = \frac{1}{8}\rho g H_b^2 \tag{E.5.3}$$

where ρ_s denotes the density of sand (= 2,650 kg/m^3), ρ is the density of seawater (= 1030 kg/m^3), g is the gravitational acceleration (= 9.8 m/s^2), E is the wave energy density, C_g is the group velocity of waves, H is the wave height, α is the incident wave angle, β is the gradient of seabed, and the suffice $_b$ denotes a quantity at wave breaking.

The constants K_1 and K_2 are to be evaluated with the records to shoreline changes collected at the study area. The term with K_1 represents the sediment transport due to the alongshore component of incident wave energy flux, while the term with K_2 gives a correction owing to the effect of wave-induced currents by the alongshore gradient of wave height.

The wave angle α_b at breaking needs to be measured with reference to the direction of the shoreline, which is somewhat inclined from the coordinate axis y. Using the notation $(\alpha_b)_0$ for the breaking wave angle with the fixed coordinate system, the breaking wave angle relative to the shoreline is given by the following formula:

$$\alpha_b = (\alpha_b)_0 - \tan^{-1}\left(\frac{\partial x_s}{\partial y}\right) \tag{E.5.4}$$

Figure E.5.2 illustrates the relationship between α_b and $(\alpha_b)_0$.

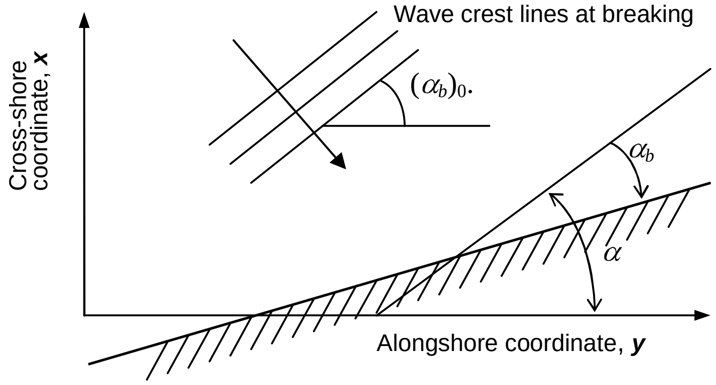


Fig. E.5.2: Sketch of wave angle at breaking

[4] Ozasa, H. and Brampton, A.H. (1979): Numerical modeling for the shore-line changes in front of the seawall, Rep. Port and Harbour Res. Inst., Vol. 18, No. 4, pp. 77-103 (in Japanese)

E.5.3 Computation of Wave Transformation from Offshore to Shoreline

Transformation of waves during their propagation from the offshore to the shoreline is computed in two steps. The first step is the computation of wave shoaling, refraction and diffraction by means of the energy balance equation for directional spectral waves from the offshore to the shallow water zone.

(1) Wave computation from offshore to inshore zones

The energy balance equation is due to Karlsson (1969) as expressed below.

$$\frac{\partial}{\partial X}\left(DV_x\right)+\frac{\partial}{\partial y}\left(DV_y\right)+\frac{\partial}{\partial \theta}\left(DV\right)=0 \tag{E.5.5}$$

where $D(f,\theta)$ denotes the directional spectral density function of waves with f for the frequency and θ for the azimuth. The terms of V_x, V_y and V represent the energy transport velocities in the x, y, and θ directions, respectively, and they are expressed as below.

$$V_y = C_g \sin\theta\,,\ \ V_x = C_g \cos\theta \tag{E.5.6}$$

$$V = \frac{C_g}{C}\left[\frac{\partial C}{\partial x}\sin\theta - \frac{\partial C}{\partial y}\cos\theta\right] \tag{E.5.7}$$

where C is the phase velocity and C_g is the group velocity of waves.

Equation E.5.5 is solved from the offshore side of the computational domain toward the shore under the three dimensional coordinates of x, y, and θ for the combination of multiple frequencies and directional components. At the offshore boundary, the information of directional wave spectrum is needed as the input; usually some standard function is assumed for a given condition of significant wave height, period, and offshore wave direction. Computation based on Eq. 5.5 yields the values of the directional spectral density at each grid point within the computational domain. The significant wave height $H_{1/3}$ is estimated from the integration of the directional spectral density by the following formula:

$$H_{1/3} = 4.004\sqrt{m_0} \tag{E.5.8}$$

where

$$m_0 = \int_0^\infty \int_{\theta_{\min}}^{\theta_{\max}} D(f,\theta)\, df\, d\theta \tag{E.5.9}$$

The principal wave direction and representative wave period are also calculated from the directional spectral density at respective grid points.

Variation of wave height due to diffraction by breakwaters and other barriers is estimated by means of the angular spreading method[5] for the sake of simplicity.

[5] For example, see Section **3.2.4** of "*Random Seas and Design of Maritime Structures,*" by Y. Goda published in 2000 from World Scientific.

(2) Wave computation within the inshore zone

Although the energy balance equation produces detailed information of directional wave spectrum, Eq. E.5.2 for estimate of alongshore sediment transport has been established with the database of regular wave transformation. Thus, the wave computation within the inshore zone is carried out with the concept of regular waves. Along a grid line parallel to the shore and far enough from the wave breaking zone, waves are designated as the shallow water waves and the information of their significant wave height and the primary wave direction on this line is stored at a computer memory. Then, propagation and transformation of waves are traced from this line toward the shoreline as if they are regular waves.

The changes of wave direction and wave height by refraction is estimated with Snell's law, i.e.,

$$\left.\begin{aligned} \theta &= \sin^{-1}\left(\frac{C}{C_i}\sin\theta\right) \\ K_r &= \sqrt{\frac{\cos\theta_i}{\cos\theta}} \end{aligned}\right\} \qquad \text{(E.5.10)}$$

where θ is the direction of refracted waves, K_r is the refraction coefficient, and θ_i is the direction of shallow water waves at the *i*-th grid point from which wave propagation is to be traced. In the area where the effect of wave diffraction is present, the angular spreading method is applied for modification of wave height.

The wave breaking is judged with the criterion of $H_b = 0.8h$, and a search is made to find out the location to satisfy this criterion. The quantities of $(EC_g)_b$ and α_b are evaluated at the location and substituted into Eq. E.5.2 for estimation of alongshore sediment transport rate.

E.5.4 Computation of Shoreline Change

The alongshore sediment transport rate varies from place to place because of the differences in the orientation of shoreline and the wave direction at breaking zone. If there is any unbalance between the input q_{in} and output q_{out} shown in Fig. E.5.1, then the shoreline advance or retreat and the process is analyzed with Eq. 5.1. Computation for solving Eq. 5.1 is carried out with the time step Δt defined by the following:

$$\left.\begin{aligned} \Delta t &= 0.5\frac{(\Delta x)^2}{\varepsilon_{\max}} \\ \varepsilon_{\max} &= \frac{\left(H^2 c_g\right)_b}{D}\left(2K_1 + K_2\frac{\partial H_b}{\partial x}\sin\alpha_b\right) \end{aligned}\right\} \qquad \text{(E.5.11)}$$

Computation is carried out for the period of time as required for months or years.

E.6 Estimate of Sediment Transport Rate along Study Area

E.6.1 Representative Waves

The representative waves for computation of sediment transport and shoreline change have been calculated as the energetic averages of the heights, periods, and directions of wave data of ECMWF by the following formulas:

Representative wave height: $$H_{rep} = \sqrt{\frac{\sum_{i=1}^{N} H_i^2 T_i}{\sum_{i=1}^{N} T_i}} \tag{E.6.1}$$

Representative wave period: $$T_{rep} = \frac{\sum_{i=1}^{N} T_i}{N} \tag{E.6.2}$$

Representative wave direction: $$\theta_{rep} = \frac{\sum_{i=1}^{N} \theta_i H_i^2 T_i}{\sum_{i=1}^{N} H_i^2} \tag{E.6.3}$$

where H_i, T_i, and θ_i are the individual wave data hindcasted every 6 hours by ECMWF and N is the total number of wave data.

The wave data were classified for the northerly and southerly waves, the former with the direction from N0º E to N90ºE and the latter from N90ºE to N180ºE. The overall averages of the northerly and southerly waves have been listed in Table 4.5.1 in **4.5**. Calculation of the representative waves were also made for each month. Table E.6.1 lists the results of the monthly representative height and period of the northerly and southerly waves together with their occurrence frequencies and the ratios of their energy to the energy of the representative waves. The monthly energy ratio is multiplied to the energy of the northerly or southerly waves when the sediment transport rate and shoreline change are calculated in each month by employing the annual averages of the northerly and southerly waves to save the computational load.

Table E.6.1: Monthly representative height and period of the northerly and southerly waves

Month	Northerly waves				Southerly waves			
	Frequency (%)	Height, *H* (m)	Period, *T* (s)	Energy ratio	Frequency (%)	Height, *H* (m)	Period, *T* (s)	Energy ratio
January	63.13	1.92	6.47	0.896	36.87	1.43	6.54	0.649
February	65.90	1.81	6.60	0.809	34.10	1.08	6.20	0.324
March	59.35	1.73	6.41	0.634	40.65	1.16	6.31	0.454
April	30.90	1.44	6.23	0.229	69.10	0.93	6.23	0.490
May	34.43	1.25	5.88	0.187	65.57	0.83	5.81	0.345
June	53.42	0.87	5.54	0.140	46.58	0.72	5.56	0.177
July	56.03	0.96	5.45	0.187	43.97	0.80	5.86	0.221
August	46.27	1.12	5.69	0.223	53.73	0.75	5.91	0.235
September	39.07	1.36	5.92	0.276	60.93	1.06	6.09	0.557
October	47.42	1.51	6.17	0.414	52.58	1.48	6.25	1.008
November	51.38	2.03	6.67	0.841	48.62	1.64	6.72	1.156
December	68.49	2.45	7.02	1.589	31.51	1.34	6.56	0.489
Annual	51.32	1.65	6.17	–	48.68	1.11	6.17	–

The representative waves were given the standard frequency spectral function by Bretschneider and Mitsuyasu and the Mitsuyasu type directional spreading function.[6] The directional spreading parameter of s_{max} = 25 was used. For computation of wave transformation from the offshore to the nearshore by means of the energy balance equation

[6] See **2.3.1** and **2.3.2** of "*Random Seas and Design of Maritime Structures*" by Y. Goda (2000) from World Scientific for their functional forms.

(E.5.5), the frequency spectrum was divided into five components with equal energy and the range of azimuth was divided into 36 with $\Delta\alpha = 5^\circ$.

E.6.2 Computational Conditions of Sediment Transport Rate

The computation of sediment transport rate and shoreline change by means of the one-line theory described in **E.5.1** and **E.5.2** was carried out for the five coastal sectors of Constanța, Eforie, Costineşti, Mangalia, and Limanu independently; see Table 5.2.1 of Volume 1 for the areas included in each sector. The condition of computation and constant values are listed in Table E.6.2

The constant K_1 was given three trial values of 0.077, 0.154, and 0.308 for the Constanța Sector, among which K_1 = 0.154 yielded the simulation result most agreeable with the past survey data. The constant K_2 in Eq. E.5.2 has been fixed at the value of $K_2 = 0.81\ K_1$, based on the team's past experience. For the Eforie and southern Sectors, the value of K_1 = 0.154 for Mamaia was adjusted to K_1 = 0.109 by the square root of the median diameter ratio (0.2 mm in Mamaia versus 0.4 mm in Eforie). Initially the cross-shore sediment outflow rate was not considered, but later it was given the values listed in Table E.6.2 by trial and error procedure. The time step specified by Eq. E.5.11 varied from 50 to 200 minutes.

Table E.6.2: Computational conditions and constant values for sediment transport rate

Item		Constanța Sector	Eforie Sector	Tuzula – Costineşti – Schitu Sectors	Mangalia Sector	Limanu Secotor
Computational domain	Alongshore distance	20,000 m	10,000 m	11,000 m	13,000 m	8,000 m
	Spacing, Δx	20 m	20 m	20 m	20 m	20 m
	Nos. of grid points	1001	501	551	651	401
Computation Period		Jan '76 – Dec '05	Jan '81 – Dec '05	Jan '85 – Dec '05	Jan '95 – Dec '05	Jan '73 – Dec '05
Survey year of initial shoreline		1976	1981	1985	1995	1973
Mean beach slope		1/40	1/40	1/40	1/40	1/40
Depth of closure, h_c		9.6 m	7.5 m	7.3 m	7.6 m	7.6 m
Sediment transport constant	K_1	0.154	0.109	0.109	0.109	0.109
	K_2	$0.81K_1$	$0.81K_1$	$0.81K_1$	$0.81K_1$	$0.81K_1$
Time step (min.)	Northerly waves	50	83	92	67	100
	Southerly waves	92	183	167	200	200
Cross-shore sediment outflow (m^3/m/year)		3.0	1.5	–	1.5	1.4

Note: Depth of closure hc includes the foreshore height added to that listed in **E.4.3**.

E.7 Design of Beach Fill Plan for Mamaia Beach and Other Beaches

E.7.1 Analysis of Foreshore Profiles in Mamaia Beach

(1) Survey results of foreshore profiles

The coastal protection and rehabilitation plan for the Southern Romanian Black Sea shore involves extensive beach fill projects, which are judged indispensable for the coast area under study. Best plans for beach fill should be designed in consideration of the local conditions of beach profiles and incident waves, by referring to relevant information available in various research accomplishments. In the present section, the data on the beaches of Năvodari and Mamaia (hereinafter referred to as “Mamaia Beach”) is examined for the purpose of establishing the methodology of predicting the characteristics of beach profiles. The method is applied for designing a stable profile of beach fill and the dimensions of beach protection facilities in Mamaia Beach. The method is further applied for beach fill plans in other beaches.

First, the data of the foreshore slope and backshore height at the bench marks MM-1 to MM-15, the locations of which are shown in Fig. 4.2.3 of **4.2** of Volume 1, are analyzed from the beach survey data by NIMRD. The survey was made twice a year from 1980 to 2005, and it lists the data of the elevations of profile inflection points and their distances from the bench marks established in the back of beach. A beach fill operation was undertaken in 1989 at the southern part (MM-13 to MM-15) of Mamaia Beach. Profile survey data at respective bench marks were classified into those before and after the beach fill, and the mean profiles were approximated with straight lines, parabolic curves, or 3-degree polynomials, which best fitted to the data. The slope of foreshore and the height (highest elevation) of backshore were calculated with these approximations. Table E.7.1 lists the resultant data and Fig. E.7.1 illustrates them.

The backshore height tends to be high when the wave height is large and the wave period is long. While the backshore height in the sub-sectors of Năvodari North and South and the northern part of Mamaia North is 1.6 m, the mean backshore height in the sub-sectors of Mamaia Center and South is about 2.3 m. The difference reflects the situation that the northeastern waves reaching the northern part of Mamaia Beach become small owing to the wave diffraction effect of the east breakwater of Midia Port.

Table E.7.1: Spatial distribution of the foreshore slope and the backshore height in Mamaia Beach

Bench Mark	Foreshore slope, cot β			Backshore height, h_c (m)			Beach width, B (m)			Sub-sector name
	(1)	(2)	(3)	(1)	(2)	(3)	(1)	(2)	(3)	
MM-1	42.3	26.4		1.6	1.6		105	119		Năvodari North
MM-2	31.5	26.2		1.6	1.6		123	106		Năvodari South
MM-3	35.7	25.4		1.6	1.7		50	56		Mamaia North
MM-4	40.3	36.7		2.6	2.4		135	129		
MM-5	46.0	36.0		2.1	2.1		136	144		
MM-6		15.3			2.3			59		
MM-7	30.3	19.6		2.6	2.4		76	50		
MM-8	22.6	32.6		2.0	1.9		100	117		Mamaia Center
MM-9	42.1	37.4		2.1	2.0		118	97		
MM-10	26.2	25.7		2.3	2.3		89	101		
MM-11	23.3	23.4		2.2	2.2		113	122		
MM-12	21.8	18.3		2.6	2.7		88	101		Mamaia South
MM-13	21.3	19.2	40.0	2.3	2.4	2.3	71	93	70	
MM-14	30.1	26.5	42.1	2.2	2.3	2.2	91	120	78	
MM-15	21.3	16.2	35.3	2.4	2.2	2.1	46	74	56	
Average (MM-4 – MM-15)	**29.6**	**25.6**	**29.1**	**2.3**	**2.3**	**2.1**	**97**	**101**	**68**	

Note: The columns (1), (2), and (3) refer to the periods preceding, succeeding, and soon after the beach fill in 1989, respectively.

(2) Empirical formulas on foreshore profiles

Up to now, three studies are available for the profiles of backshore and foreshore as reported by Rector[7], Swart[8], and Sunamura[9]. The quantities that define the foreshore profile are the horizontal distance X_S from the berm crest to the shoreline at the base level, the berm crest height Y_S, and the foreshore slope $\tan\beta_f = Y_S/X_S$, as sketched in Fig. E.7.2. Rector further define the horizontal distance X_t and the total height Y_t from the berm crest to the inflection point of the inshore profile under the water.

According to Rector, the foreshore slope can be expressed with the offshore wave steepness H_0/L_0 and the sediment diameter on the seabed d_m as in the following:

$$\left.\begin{aligned} &Y_S/L_0 = 0.18(H_0/L_0)^{0.5} &&: \quad H_0/L_0 \le 0.018 \\ &Y_S/L_0 = 0.024 &&: \quad H_0/L_0 \ge 0.018 \\ &\tan\beta_f = \frac{Y_S}{X_S} = 0.3\left(\frac{H_0}{L_0}\right)^{-0.3}\left(\frac{d_m}{L_0}\right)^{0.2} && \end{aligned}\right\} \qquad \text{(E.7.1)}$$

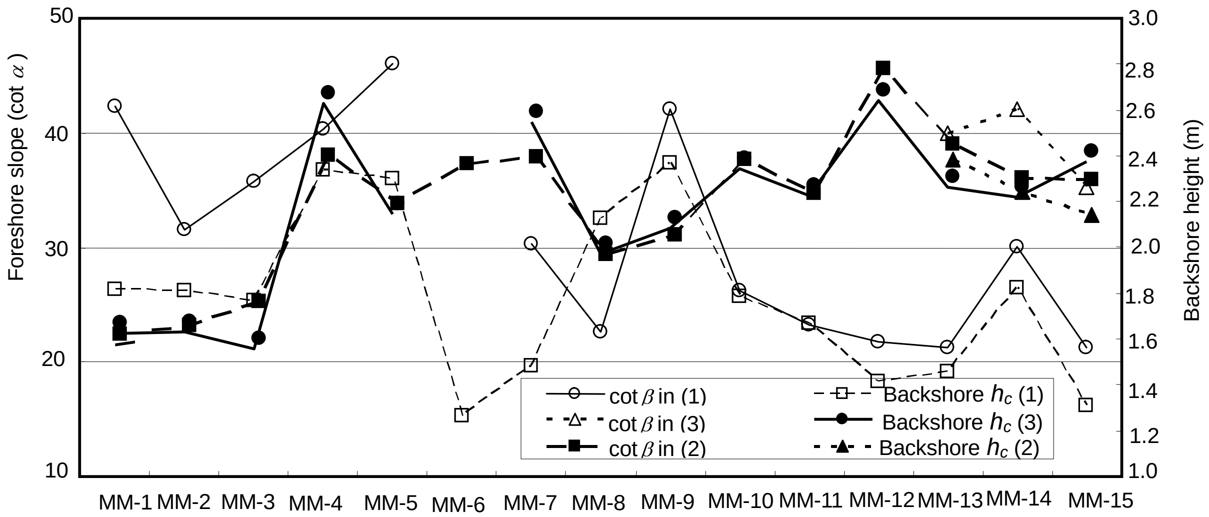


Fig. E.7.1: Spatial distribution of foreshore slope and backshore height in Mamaia Beach (see Table E.7.1 for the explanations for (1), (2), and (3) for the survey period)

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7 Rector, R.L. (1954): Laboratory sturdy of equilibrium profiles of beach, Technical memorandum, No.41,B.E.E. Corps of Engineers.
8 Swart, D.H. (1974) : Offshore transport and equilibrium profiles ; Publication no. 131, Delft Hydraulics Laboratory, 2434 p.
9 Sunamura, T. (1975) : “Static” relationship among beach slope, sand size and wave properties, Geogrphical Review of Japan, Vol.48,No.7.

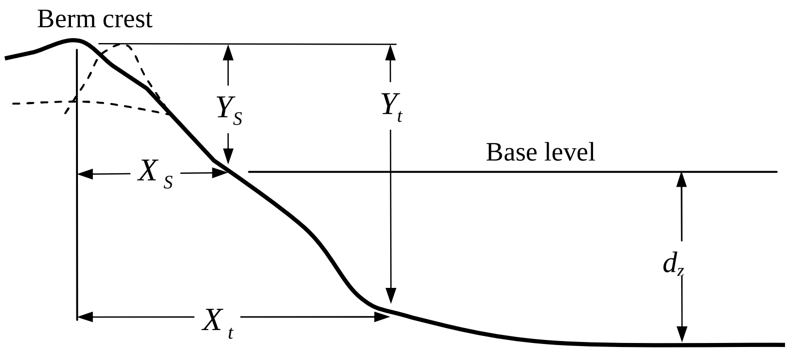


Fig. E.7.2: foreshore beach profile

In a similar approach, Sunamura has defined the berm height and foreshore slope as follows:

$$\left.\begin{aligned} Y_S &= 0.173\left(H_0 L_0\right)^{0.5} \\ \tan\beta_f &= 0.25\left(\frac{d_m}{H_0}\right)^{0.25}\left(\frac{H_0}{L_0}\right)^{-0.15} \end{aligned}\right\} \tag{E.7.2}$$

Swart has given the berm crest height Y_S and the depth of water d_z at which a significant slope change takes place in the inshore as in the following formulas:

$$Y_S = d_m\left[7644 - 7706\exp\left(-0.00143\frac{H_0^{0.488}T^{0.93}}{d_m^{0.786}}\right)\right] \tag{E.7.3}$$

$$d_z = L_0\left[0.0063\exp\left(4.347\frac{H_0^{0.473}}{T^{0.894}d_m^{0.093}}\right)\right] \tag{E.7.4}$$

He also gave empirical formulas for the foreshore profiles based on hydraulic model test data. The foreshore slope can be calculated from the difference between arbitrary two points of the foreshore profile estimated by the empirical formulas. The present analysis employed the berm crest and the location with the elevation of +1 m above the base line for calculation of the foreshore slope by Swart's formulas. It is mentioned here that Rector's formulas were derived from hydraulic model test data, while Sunamura's formula is based on the field measurement data. Sunamura's formula (E.7.2) is said to yield a milder slope of foreshore in general.

(3) Comparison of field data and empirical formulas on foreshore profiles

Application of the above empirical formulas to the field data of Mamaia Beach has been made with the median diameter of d_{50} = 0.2 mm, which is representative of the field data on the average, and the waves of $H_{1/3}$ = 3.7 m and T = 7.5 s, which correspond to the non-exceedance probability of 99% of the ECMWF data. Comparison of the empirical estimates and the field data concerning the foreshore slope and the berm crest height is listed in Table E.7.2.

Table E.7.2: Comparison of empirical estimates and field data on foreshore profile

Item	Estimate			Field data
	Rector	Swart	Sunamura	(average of MM-4 to MM-15 after 1989)
Berm height, Y_S (m)	2.1	1.2	3.1	2.3
Foreshore slope, cot β	17	23	29	25.6

The result of computation indicates that the berm height is well predicted by Rector's formula, while the foreshore slope is best represented by Sunamura's formula.

Next step is the examination of the beach profile after the beach fill operation in 1989. It was carried out by using sand dredged from Lake Siutghiol, the median diameter of which is estimated as around 0.1 mm. The beach fill was made in the area between the bench marks MM-13 to MM-15, and thus the profile of foreshore in this area is used for examination. The result of comparison between the estimate and field data is shown in Table E.7.3.

Table E.7.3: Comparison of empirical estimates and field data on foreshore profile soon after beach fill operation

Item		Estimate			Field data
		Rector	Swart	Sunamura	(average of MM-13 to MM-15 soon after beach fill in 1989)
Berm height, Y_S (m)	$H_{1/3}$ = 2.0 m	2.1	0.6	2.3	2.2
	$H_{1/3}$ = 3.7 m	2.1	0.7	3.1	
Foreshore slope, cot β	$H_{1/3}$ = 2.0 m	17	24	27	39
	$H_{1/3}$ = 3.7 m	20	31	35	

The wave condition is set at $H_{1/3}$ = 2.0 m and T = 7.5 s as the result of wave attenuation by the detached breakwaters, which were constructed at the time of beach fill operation. However, the case without attenuation is also calculated for comparison. The median diameter of sediment is d_{50} = 0.1 mm for the both cases.

The result of computation indicates that Rector's formula for the berm height well reproduces the field situation, while Sunamura's formula for the foreshore profile yields agreement with the field data when large waves are thought to be acting. There are possibilities that the wave attenuation effect of detached breakwaters was less than expectation, actual wave conditions were much severer than the computational case, and/or the median diameter of sediment was smaller than 0.1 mm.

The results of Tables E.7.2 and E.7.3 support the applicability of the empirical formulas by Rector and Sunamura for prediction of the profile of prototype foreshores.

E.7.2 Analysis of Inshore Profiles in Mamaia Beach and Other Beaches

(1) Survey results of inshore profiles

The term of "inshore" refers to the zone between the shoreline and the offshore as sketched in Fig. E.4.1. The seabed slopes of the study area were examined on the basis of the bathymetric maps with depth contours, which were prepared in 1997 for monitoring of shore protection facilities, and the bathymetric chart of Mamaia surveyed in 2004.

Table E.7.4: Inshore slope of several beach areas

Beach area	Slope
Năvodari – Mamaia South	1/100
Tomis	1/100 – 1/125
Eforie Nord – Eforie Sud	1/90
Costinesti	1/50
Olimp – Venus	1/50
Saturn –Mangalia	1/80
2 Mai – Vama Veche	1/70

The inshore slope was calculated as the mean gradient of the seabed between the shoreline and the location with the depth contours of 4 or 6 m in the bathymetric maps. The result of calculation is listed in Table E.7.4.

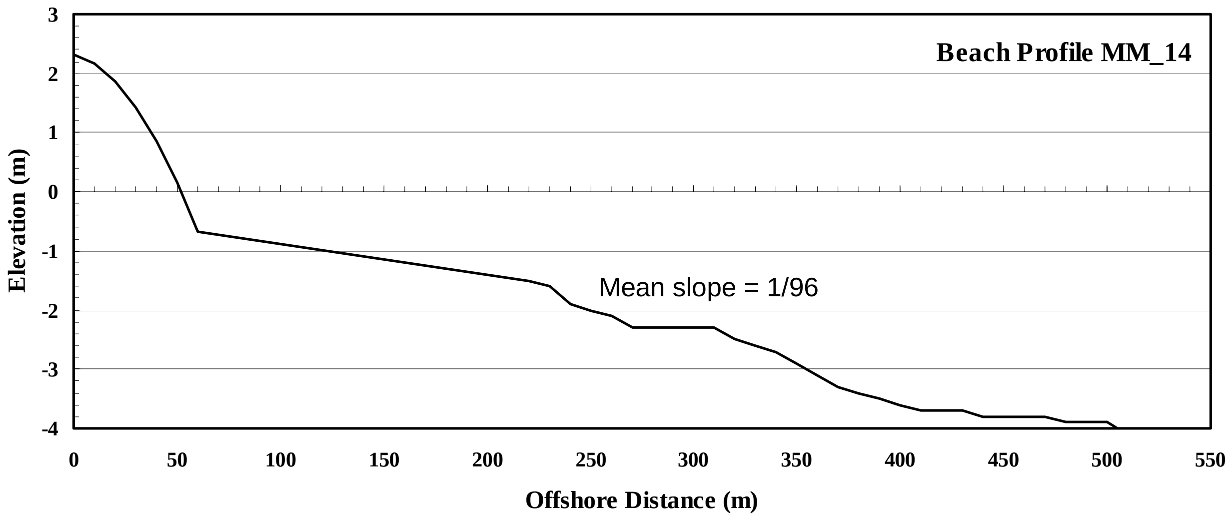


Fig. E.7.3: Inshore profile of Mamaia Beach at MM-14 corresponding to the opening of breakwaters

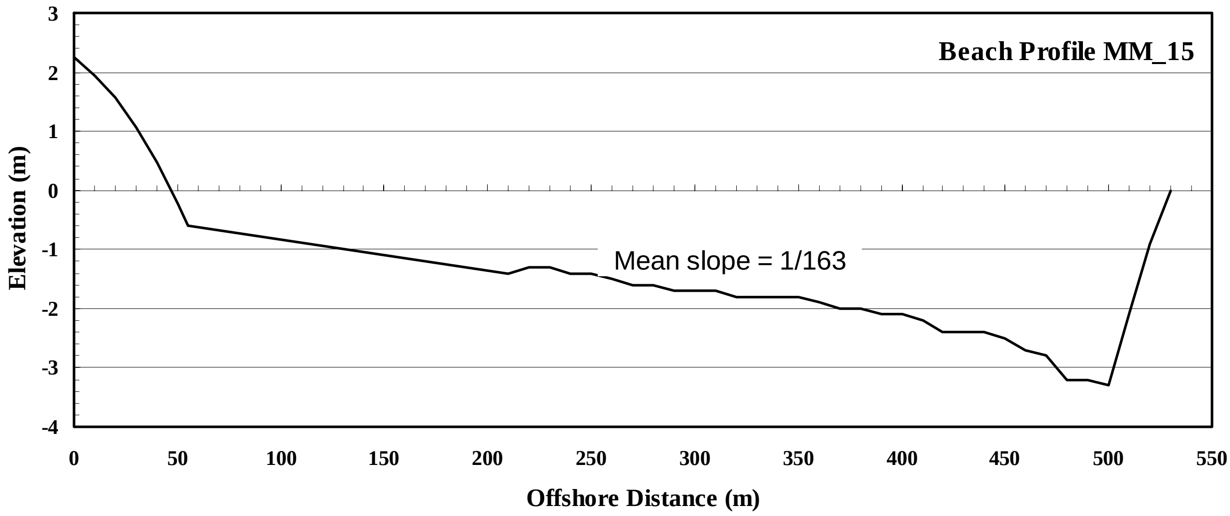


Fig. E.7.4: Inshore profile of Mamaia Beach at MM-15 behind the detached breakwater

Detailed inshore profiles are available in Mamaia Beach at the bench mark locations of MM-14 and MM-15 as shown in Figs. E.7.3 and E.7.4. Bathymetric surveys were carried out for the area deeper than around 1.5 m and thus the sections between the shoreline and the location of 1.5 m deep are represented with straight lines in Figs. E.7.3 and E.7.4. The bench mark MM-14 is situated in the location corresponding to the opening of detached breakwaters, whereas the bench mark MM-14 is situated behind the southernmost breakwater.

The figures of the slope listed in Figs. E.7.3 and E.7.4 have been calculated as the mean slope between the shoreline and the location of 3 m depth. The inshore slope behind the detached breakwater has become much milder than that facing the breakwater opening. The mild slope is considered as the effects of small wave height attenuated by the breakwater and the presence of very fine sediment having remained after the beach fill in 1989.

(2) Empirical formulas on inshore profiles

Equation E.7.4 by Swart is applied for the inshore profile. Dean[10] has proposed the concept of equilibrium beach profile, where the inshore depth *h* is assumed to be proportional to the two-third power of the distance *x* from the shoreline and the proportionality constant is related to the fall velocity of sediment. The shape of the equilibrium beach profile by Dean is expressed as follows:

$$\left.\begin{aligned} h &= Ax^{2/3} \\ A &= 2.25\left(\frac{w_s^2}{g}\right)^{1/3} \end{aligned}\right\} \qquad \text{(E.7.5)}$$

where w_s is the fall velocity of sediment and g is the acceleration of gravity.

Hattori and Kawamata[11] has analyzed the data on the mean slope between the shoreline and the surf zone in an approach similar to Dean and obtained the following formula:

$$\tan\beta = 0.03125\frac{w_s L_0}{gTH_0} \qquad \text{(E.7.6)}$$

(3) Comparison of field data and empirical formulas on inshore profiles of Mamaia

Empirical formulas of the inshore profiles are given the mean sediment diameters of 0.2 mm and 0.1 mm; the former is the average grain size in the field, while the latter corresponds to a candidate sand of beach fill on Mamaia South. Incident waves are set at $H_{1/3}$ = 3.7 m and T = 7.5 s, which correspond to the non- exceedance probability of 99% of the ECMWF data; these are the same as used for the foreshore profile comparison. Because the detached breakwaters are not exercising sufficient wave attenuation owing to deterioration, the above offshore wave conditions are employed for comparison with the field data.

[10] Dean, R.G.(1997) : Equibrium beach profiles, U.S.Atlantic and Gulf Coast, Ocean Eng. Rep. No.12, Univ. of Delaware.
[11] Hattori, M. and Kawamata, R.(1978) : Relationship between wave characteristics and beach profile in surf zones, Proc. 25th Conf. on Coastal Engineering, JSCE, pp218-222 (in Japanese).

Table E.7.5: Comparison of empirical estimates and field data on inshore profile

Median diameter, d_{50}	Inshore slope, cot β			
	Swart	Dean	Hattori and Kawamata	Field data
0.1 mm	41	271	653	163 (MM-15)
0.2 mm	33	76	184	96 (MM-14)

The estimates of the inshore slope by the formulas of Swart, Dean, and Hattori and Kawamata are compared with the filed data in Table E.7.5. The inshore slope by the formulas of Swart and Dean is calculated as the mean value between the shoreline and the location of 3 m depth. Among the three formulas, calculation by Dean's formula with d_{50} = 0.1 mm yields the slope slightly milder than the field data, while that with d_{50} = 0.2 mm yields the slightly steeper slope. The formula by Swart yields too steep slope, while Hattori and Kawamata's formula yields too mild slope. Thus, it is concluded that Dean's formula can provide reasonable estimate of the inshore slope.

E.7.3 Beach Fill Plan for Mamaia Beach

(1) Introduction

The analysis of the shoreline survey records has confirmed a large shoreline retreat rate of about 2 m per year at the beach at the Sub-section of Mamaia South. The numerical simulation with the one-line theory has predicted a maximum shoreline retreat of 100 m behind the southernmost breakwater in the coming twenty years (average of some 70 m in Mamaia South).

Rehabilitation of beach erosion caused by the imbalance of alongshore sediment transport is generally undertaken from the down-drift side as the principle. However, the erosion of Mamaia South beach is very severe, demanding urgent countermeasures. Thus, an immediate beach fill operation and provision of a groin for retaining beach fill sand are judged necessary. For this purpose, the dimensions of the following items are examined in the sub-sections hereinafter:

1) beach width
2) backshore height
3) foreshore slope
4) inshore slope
5) junction location of the foreshore and inshore slopes

(2) Beach width

The Bureau for Ports and Harbours of the Ministry of Land, Infrastructure and Transport of Japan has conducted a survey with questionnaire on the most comfortable beach width at various summer beaches in Japan. Many beach users replied that a beach too wide is uncomfortable when walking across in bare feet over hot sand heated by the sun. Operators of shops, restraints, and bars answered that too wide beaches hinder the beach users' visits to their facilities. Both beach users and operators recommended the beach width 50 m as most optimum one.

Beaches in Japan are mainly utilized for sunbathing and ocean bathing. On the other hand, many areas of Mamaia Beaches are used for playing beach volley ball and football, and a number of parasols and deckchairs are spread on beaches. Thus, beaches in Mamaia would require the beach width larger than the Japanese standard of 50 m. In fact, the data of Table E.7.1 on the average beach width based on the shoreline survey indicates the beach width around 100 m; an exception is the narrow width of 46 m at the location MM-15 before a beach fill operation in 1989.

In consideration of the above data, the beach width at Mamaia South is planed as 100 m.

(3) Backshore height

The average backshore height of Mamaia Beaches is 2.3 m as listed in Table E.7.1, and the backshore height of stable profile by Rector is calculated as 2.1 m. Thus, the backshore height is planned at Y_S = 2.3 m.

(4) Wave height behind detached breakwaters

As discussed in **E.4.4** and listed in Table E.4.4, the detached breakwaters in Mamaia are functioning for wave attenuation even though their function has decreased owing to deterioration of the breakwaters. It is planned to rehabilitate the breakwaters by placing rubble mounds behind them and providing armor blocks on top of the latter. The effective width of breakwater at the mean water level will be increased to B = 17.25 m from the present one of B = 1 m.

Waves incident to the breakwaters have the height of $H_{1/3}$ = 2.85 m for the offshore height of $H_{1/3}$ = 3.7 m because of attenuation by random breaking. The formulas for estimation of the wave transmission are those by Iwasaki and Numata of Eq. E.4.12. Because these formulas are for the one dimensional case, further consideration for the effect of the breakwater opening is necessary. The wave energy through the detached breakwater with the length A is transmitted with the height reduced by the transmission coefficient K_T, while the energy flux through the opening S enters the area behind the detached breakwater system without attenuation. Thus, the energy averaged transmission coefficient $(K_T)_{\mathrm{mean}}$ can be evaluated by the following:

$$(K_T)_{mean} = \sqrt{\frac{S + AK_T^2}{S + A}} \tag{E.7.8}$$

The average wave height in the area behind the detached breakwater system is thus estimated as listed in Table E.7.6 for the present and rehabilitated breakwater conditions.

Table E.7.6: Average wave height behind the present and rehabilitated breakwaters in Mamaia

Breakwater	Incident Height, H_I (m)	Transmission coef., K_T	Transmitted height, H_T (m)	Mean transmission coef., $(K_T)_{\mathrm{mean}}$	Mean transmitted height, $(H_T)_{\mathrm{mean}}$ (m)
Present (B = 1 m, D = 2 m)	2.85	0.2	0.57	0.72	2.05
Rehabilitated (B = 17.25 m, D = 2 m)		0.7	2.00	0.86	2.45

(5) Foreshore and inshore profiles

Because the slopes of foreshore and inshore depend on the sediment grain size, four median diameters of 0.1, 0.2, 0.3, and 0.4 mm are used for calculation of the foreshore and inshore slopes for examination of the grain size effect. The wave height of $(H_T)_{mean}$ = 2.05 m after rehabilitation of the breakwaters is used. The backshore height is set at Y_S = 2.3 m as discussed in **(3)**. The foreshore slope is estimated by Sunamura's formula (Eq. E.7.2) that gave best results, while the inshore slope is estimated by Dean's approach (Eq. E.7.5). Table E.7.7 lists the result of calculation. Result of the estimated beach profiles is also shown in Fig. E.7.5 for the cases of the mean diameters of 0.1 and 0.4 mm, together with the present profile.

As clearly shown in Fig. E.7.5, a beach fill with sand of d_{50} = 0.1 mm will produce the inshore profile making contact with the detached breakwater at the depth of about 1.5 m. On the other hand, a beach fill with sand of d_{50} = 0.4 mm will make contact with the existing beach profile at the distance of about 130 m. The required volume of beach fill with sand of d_{50} = 0.4 mm is about one-third of the volume with sand of d_{50} = 0.1 mm for the location MM-15. In case of the location MM-14 shown in Fig. E.7.3, the inshore slope of filled beach with sand of d_{50} = 0.1 mm cannot make contact with the existing beach profile. An underwater sill will be required to contain the nourished sand. Because of a larger volume of beach fill sand and additional structure of underwater sill, the beach fill with sand of d_{50} = 0.1 mm will be more expensive.

Table E.7.7: Foreshore and inshore slopes with and without detached breakwaters

Slope		Median diameter of beach fill sand, d_{50} (mm)			
		0.1	0.2	0.3	0.4
Foreshore, cot β_f	no breakwater	35	29	26	24
	with breakwater	27	23	21	19
Inshore slope, cotβ		270	76	42	29

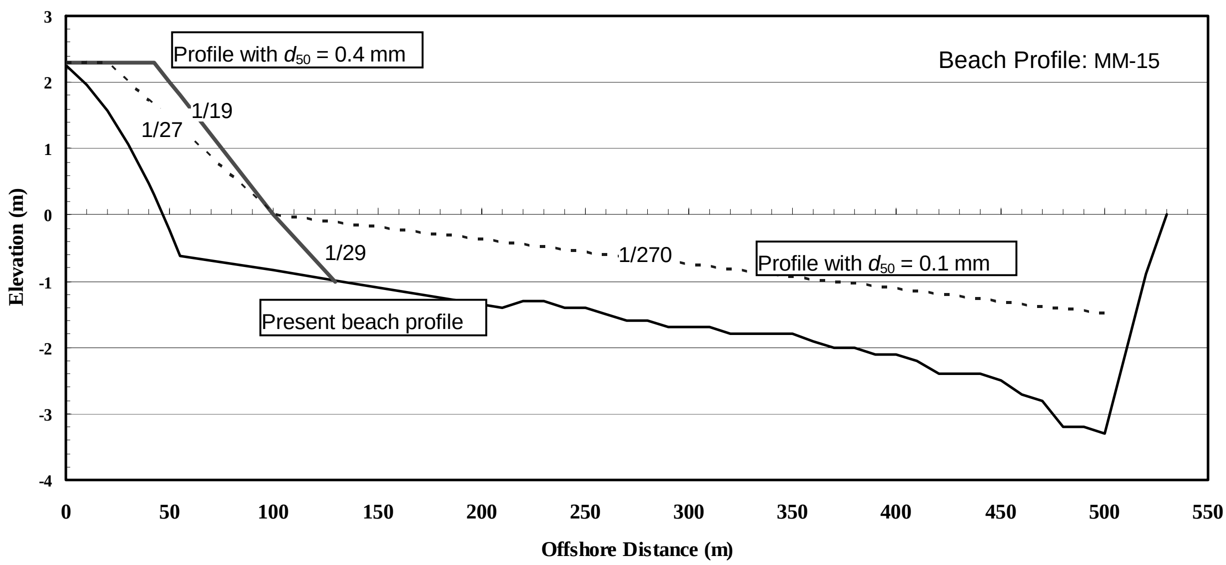


Fig. E.7.5: Profiles of foreshore and inshore with beach fill with fine and coarse sand

The candidates for beach fill sand are the seabed sand in the east of Midia Port and the riverbed sand of the Danube as described in **5.7**. The former has the median diameter of 0.1 mm, while the latter's diameter is around 0.4 mm or greater. Therefore, it is recommended to use the riverbed sand from the both viewpoints of construction cost and stability of filled beach profiles.

(6) Junction point between foreshore and inshore profiles

Design of beach fill plan requires determination of the junction point between the foreshore and inshore profiles, because their stable slopes are different. The stable slope is a function of sand grain size. One of the possible sites of sand supply is the sand bars around Cochirleni (km 305 – km 306) of the Danube. The median diameter of the sand in this area is about d_{50} = 0.25 mm. In the following, examination is made of the stable beach slope for sand of this median diameter.

The backshore height is calculated as Y_S = 2.11 m according to Rector's formula (E.7.1), and therefore a round figure of Y_S = 2.2 m is adopted here. The foreshore slope can be calculated by Sunamura's formula of the following:

$$\tan\beta_f = 0.25\left(\frac{d}{H_0}\right)^{0.25}\left(\frac{H_0}{L_0}\right)^{-0.15} \qquad \text{(E.7.9)}$$

The formula yields the foreshore slope of $\tan\beta_f$ = 1/21.7, and thus a round figure of 1/20 is adopted.

For determination of the inshore slope, Dean's formula (E.7.5) for the equilibrium profile is referred to. The beach profile corresponding to the median diameter d_{50} = 0.25 mm is shown in Fig. E.7.6. The equivalent beach slope, which is defined with the straight line connecting the shoreline and the seabed at the respective distance, is also shown in this diagram.

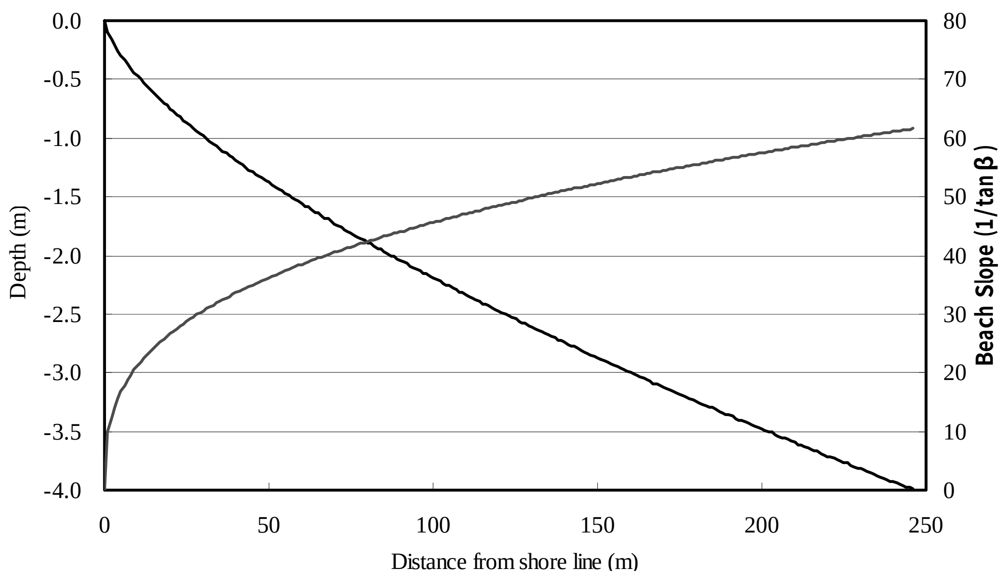


Fig. E.7.6: Equilibrium beach profile and the equivalent beach slope for sand with d_{50} = 0.25mm

Because the equivalent beach slope is steeper than 1/60 within the shore distance of 230 m, the slope of 1/60 is employed for the planning of beach fill profile.

A beach fill plan is usually made with the junction point at the elevation with the amplitude of long-period oscillations above the high water level (HWL). The design water level of Constanța Port is HWL, which is +0.6 m above the mean water level (MWL).

The amplitude of long-period oscillations a_L can be estimated with Eq. E.7.10, in which the proportionality constant A is assigned the value 0.023 on the basis of filed observation at the Hazaki Ocean Research Facility of the Port and Airport Research Institute, Japan.

$$a_L = H_0' \frac{A}{\sqrt{H_o'/L_0}} \tag{E.7.10}$$

With the incident wave condition of $H_{1/3}$ = 3.7 m and T = 7.5 s, Eq. E.7.10 yields a_L = 0.4 m and the elevation of the junction point at MWL+1.0 m.

On the other hand, beach reconnaissance from Năvodari to Mamaia South in June 2005 clearly indicated the inflection point of beach profile at the depth −0.5 to −1.0 m. In consideration of these factors, it is designed to connect the foreshore and inshore profiles at the elevation of the mean water (MWL).

(7) Summary of beach fill plan for Mamaia South

The elements of the beach fill plan for Mamaia South are summarized below.

1)	beach width	: 100 m
2)	backshore height	: 2.3 m
3)	foreshore slope	: 1/20
4)	inshore slop	: 1/60
5)	junction point of the foreshore and inshore profiles	: ±0.0 m

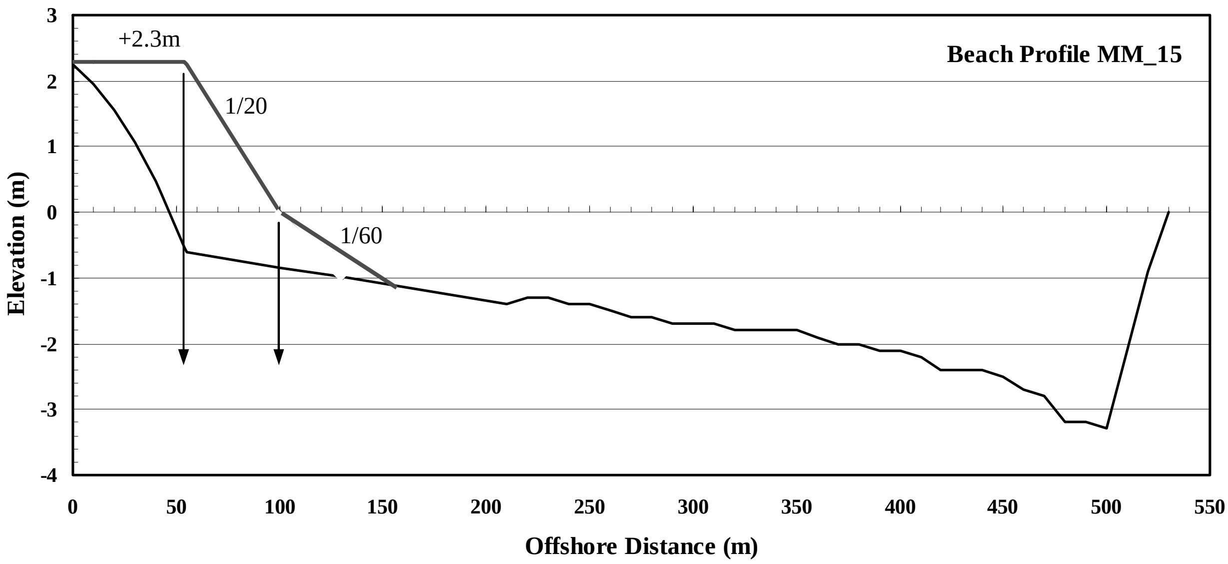


Fig. E.7.7: Planned profile of beach fill at the location MM-15

The planned profile of beach fill is superposed on the present beach profile at MM-15 as shown with red lines in Fig. E.7.7. The beach fill profile at the location MM-14 will extend slightly offshore compared with the profile shown in Fig. E.7.7, because the present inshore profile at MM-14 is steeper than that at MM-15.

E.7.4 Supplementary Structures for Mamaia Beach

(1) Wave attenuation function of detached breakwaters

Examination is made here whether the planned beach fill would be stable by using the criteria by Horikawa et al.[12] on the advance and/or retreat of the shoreline, which was derived from field and laboratory data. Horikawa et al. proposed to classify the shore into the following three types:

Shore type I: shoreline retreats and accretion occurs in the offshore side.
Shore type II: transitional shore without clear indication of shoreline advance or retreat.
Shore type III: shoreline advances and erosion occurs in the offshore side.

The three types can be judged with the value of the constant *C* of Eq. E.7.11 in such a way that the value *C* being greater than 18 corresponds to the type I and the constant *C* being smaller than 18 corresponds to the type III.

$$H_0/L_0 = C(\tan\beta)^{-0.27}(d/L_0)^{0.67} \qquad \text{(E.7.11)}$$

where $\tan\beta$ is the inshore slope and d is the sediment diameter. The offshore wave height H_0 and wavelength L_0 are those of waves occurring several times a year.

Table E.7.8: C value versus $H_{1/3}$

Wave height, $H_{1/3}$ (m)	0.57	1.00	1.33	1.50	2.00
C value	7.7	13.6	18.0	20.3	27.1

Calculation of the C value with Eq. E.7.10 is made by using the range of wave height from $H_{1/3}$ = 0.57 m that is the one-dimensional transmitted wave height and $H_{1/3}$ = 2.00 m that is the average height in consideration of the breakwater opening. The wave period is $T_{1/3}$ = 7.5 s and the median diameter of sediment is d_{50} = 0.4 mm. The result of calculation is listed in Table E.7.8 and graphically shown in Fig. E.7.8.

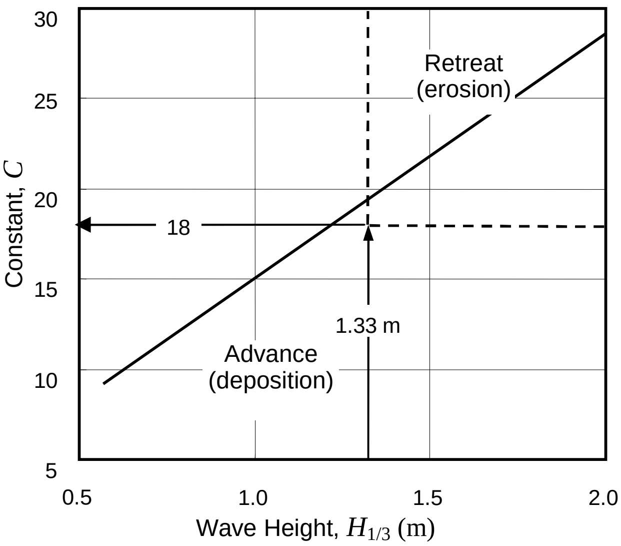


Fig. E.7.8: Range of shoreline retreat and advance by the C value

As seen in Fig. E.7.8, the shoreline of the filled beach in Mamaia is expected to advance when the wave height is less than 1.33 m, while it will retreat for larger

[12] Horikawa, K. , Sunamua,T., Kondo, K. and Okada,S(1975 : A consideration of two-demensional beach transformation due to waves, Proc. Of the 22nd Conference on Coastal Engneering, JSCE, pp.329-334.

wave height. The average wave height of 2.0 m suggests a tendency of shoreline retreat in general. However, the shoreline behind the center of a detached breakwater will advance with formation of a salient topography and the plan shape of filled beach will be stabilized.

The above stability examination of beach fill is based on the condition that the one-dimensional wave transmission coefficient of the detached breakwater is K_T = 0.2 for the incident waves of $H_{1/3}$ = 3.7 m and T =.7.5 s, the storm condition of which will occur several times a year. By maintaining the above transmission coefficient through appropriate rehabilitation of detached breakwaters whenever required, the filled beach will be able to keep its stability.

(2) Sand-retaining groin

At the northern end of the beach fill area, a groin should be built to prevent the moving-out of filled sand. First, the water depth at the head of the groin needs to be determined. If the alongshore sediment transport is to be checked completely, the groin must be extended to the depth of closure. In that case, the groin length will become very large and may not be practical. Therefore, two factors will be considered here for determination of the groin length.

The first factor is the seasonal variation of the shoreline position. According to the shoreline change analysis with the one-line model, the shoreline position is estimated to vary over 20 m at most, depending on the season.

The second factor is a deformation of beach profile by storm waves with erosion of foreshore and reduction of inshore slope as sketched in Fig. E.7.9. Its quantitative estimate is difficult, however, and thus an alternative approach is made here. The sediment on the present inshore is considered to be composed of sand with the median diameter of about 0.2 mm. After some lapse of time, the filled sand with d_{50} = 0.25 mm will be mixed with the present beach sand. The resultant mixed sand will have the median diameter of about 0.225 mm. According to Dean's formula (E.7.5), the equilibrium beach profiles and the inshore slopes with d_{50} = 0.225 and 0.25 mm are calculated as shown in Fig. 7.10.

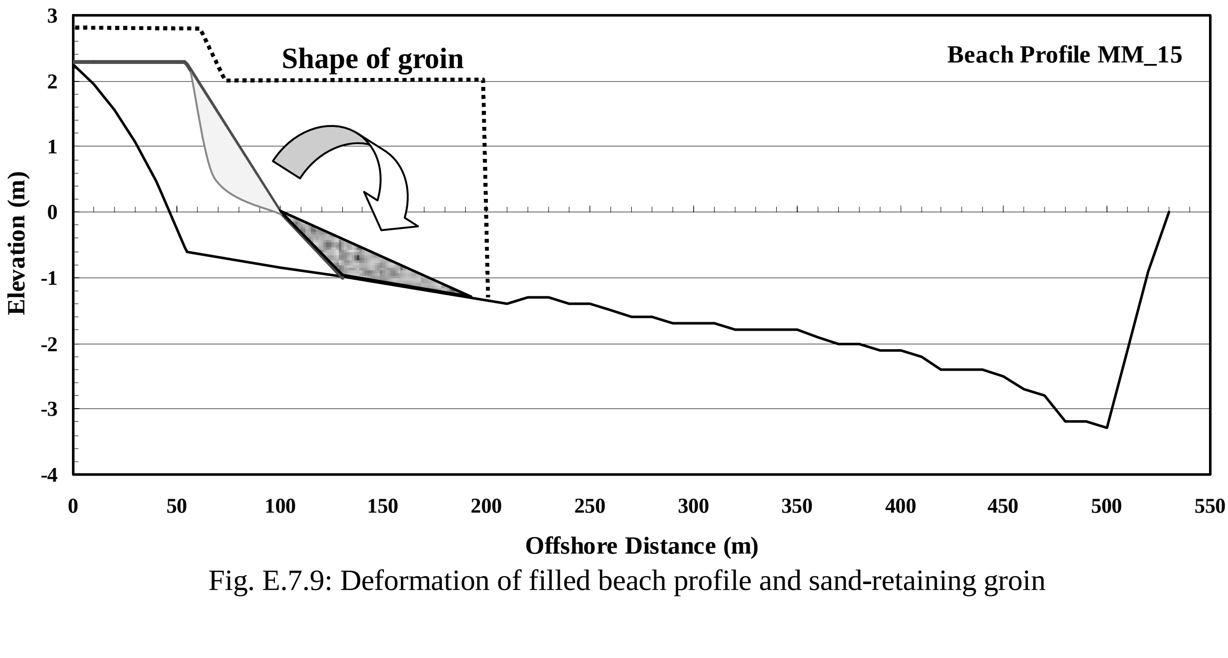


Fig. E.7.9: Deformation of filled beach profile and sand-retaining groin

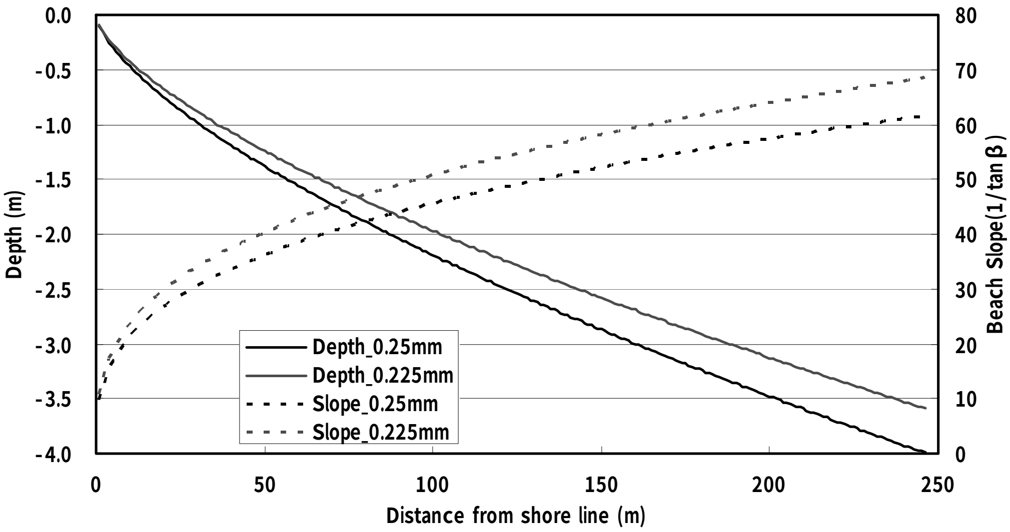


Fig. E.7.10: Equilibrium beach profiles and inshore slopes with d_{50} = 0.225 and 0.25 mm

The distance from the shoreline to the location of the designated water depth is read off from Fig. E.7.10 as listed in Table E.7.9.

Table E.7.9: Changes in the distances to various water depths by sand grain size

Diameter, d_{50} (mm)	Water depth, h (m)			
	1.0	1.5	2.0	2.5
0.225	37	66	102	143
0.25	29	56	84	118

As it has been shown in Fig. E.7.7, the toe of the filled beach with d_{50} = 0.25 mm and the planned slope of 1/60 is located at the distance of 70 m from the new shoreline. The water depth at the toe is about 1.2 m. According to Table E.7.9, the change in the median diameter owing to mixing of the original and filled sand from 0.25 mm to 0.225 mm will extend the toe location by about 10 m at the depth around 1.5 m. This extension length is taken as the alternative estimate of the deformation of filled beach profile.

The length of the sand-retaining jetty is planned as 210 m from the edge of the promenade or 110 m from the new shoreline by beach fill, which is the result of the planned distance of 70 m to the fill toe being added by the above beach extension length of 10 m, the seasonal change of 20 m, and an allowance of 10 m as a safety margin.

The longitudinal section of a groin usually has a high crest on the backshore and a low crest in the inshore as sketched in Fig. E.7.7. The crest elevation in the inshore is mostly determined from the convenience of execution of the designed cross section. The groin will be constructed by extending a mound of quarry runs toward the offshore with the crest elevation of around +1.0 m above the datum level (DL) and by covering the surface with two layers of armor stones of some 500 kg. Thus, the crest elevation in the inshore will be around DL+2.0 m.

The crest elevation on the backshore is determined by taking the wave run-up height in consideration. According to Mase,[13] the 2% exceedance run-up height can be estimated with Eq. 7.12.

$$\left.\begin{aligned} R_{2\%} &= 1.86 H_0 \xi_0^{0.71} \\ \xi_0 &= \tan\beta \left(H_0 / L_0\right)^{-0.5} \end{aligned}\right\} \tag{E.7.12}$$

The above formula yields $R_{2\%}$ = 1.27 m for the average wave of $H_{1/3}$ = 2.0 m and T = 7.5 s behind the detached breakwaters and the foreshore slope tanβ = 1/30. Since the mean high water level is HWL = DL+0.6 m, the above run-up height is added to HWL with some allowance. The process yields the crest elevation of DL+2.3 m (= 0.6 + 1.27 + 0.43), which is the same as the backshore height of beach fill.

E.7.5 Beach Fill Plans for Other Sectors

(1) Planning conditions

The incident wave condition of $H_{1/3}$ = 3.7 m and T = 7.5 s used for Mamaia Beach is also employed in the beach fill plans. for the other sectors. However, the wave height is reduced by 15% for the Eforie Nord Sector with the result of $H_{1/3}$ = 3.15 m, because of the wave sheltering effect of the breakwater of Constanța Port. Beach fill sand is the riverbed sand of the Danube (d_{50} = 0.4 mm).

Table E.7.10: Present inshore slope and beach characteristics of several beach areas

Beach area	Inshore slope	Beach slope	Beach width (m)
Năvodari – Mamaia South	1/100	1/20 – 1/45	30 – 100
Tomis	1/100 – 1/125	1/15	20 – 30
Eforie Nord – Eforie Sud	1/90	1/30	0 – 30
Olimp – Venus	1/50	1/7 – 1/20	10 – 70
Saturn –Mangalia	1/80	1/10 – 1/30	0 – 70

The inshore slopes of several beach areas, which were estimated from the bathymetric maps of 1997, have been listed in Table E.7.4. They are listed again in Table E.7.10, together with the beach width and the beach slope, which were obtained by simplified survey during the beach reconnaissance; the beach slope is defined as the mean slope between the shoreline and the end of the backshore. It is cautioned however that the inshore slope needs to be re-examined by means of detailed bathymetric survey in the feasibility study for beach fill project, as evidenced by the existing slope of Mamaia being much gentler than 1/150.

(2) Beach width

The beaches at Tomis, Eforie Nord and Eforie Sud are visited by many people, and some of these beaches are fringed by cliffs that have the danger of collapse when beaches are eroded. Therefore, these beaches are planned to have a width of 100 m in beach fill plans, same as that in Mamaia.

[13] Mase, H.(1989): Random wave run-up height on gentle slope, J. Waterway, port, Coastal, and Ocean Eng., ASCE, Vol.115, No.5, pp.649-661.

The beaches south of Olimp do not have the problem of endangered cliffs and the number of visitors is less than the beaches from Mamaia to Eforie Sud. Therefore these beaches are planned to have a width of 50 m in beach fill plans.

(3) Beach fill parameters

The profile of beach fill is specified by the four parameters of backshore height, foreshore slope, inshore slope, and junction point between foreshore and inshore. These parameters have been determined in the same manner as used for Mamaia Beach. Because some beaches are going to be to provided with submerged breakwaters, the incident wave heights to these beaches were calculated by assuming the average wave transmission coefficient of $(K_T)_{mean}$ = 0.74, which is based on the one-dimensional transmission coefficient of K_T = 0.3 and the equal lengths of breakwaters and opening (Eq. 7.8). Though the beach width was set at 100 m for Eforie Nord, it was reduced to 50 m for the Agigea area because of a small number of beach visitors. For the area between Olimp and Venus, the beach width is increased to 60 m, because the planned width of 50 m is insufficient to attain the backshore height of 2.2 m. Similarly, the beach width of Saturn and Mangalia is set at 55 m. The junction point between the foreshore and inshore is set at the elevation of DL±0.0 m. Table E.7.11 lists the parameters of beach fill plan from Tomis to Saturn – Mangalia .

Table E.7.11: Parameters of beach fill plan

Beach area	H_0’ (m)	Backshore height [a], Y_S (m)	Foreshore slope[b], cot β_f	Inshore slope[c], cot β	Beach width (m)
Tomis	2.74	2.2	22	30	100
Agigea	2.33	2	21	30	50
Eforie Nord	2.33	2.2	21	30	100
Eforie Middle and Eforie Sud	2.74	2.2	22	30	100
Olimp – Venus	3.70	2.2	25	30	60
Saturn –Mangalia	2.74	2.2	22	30	55

Note: a) by Rector’s formula, b) by Sunamura’s formula, and c) by Dean’s method.

(4) Estimate of beach fill volume

For estimation of the volume of beach fill sand, the information of the existing beach profile is necessary. Although it was available in Mamaia South, the situation is different in other beaches where the available information was the maps of shore protection facilities with contours prepared in 1997 and the simple survey during beach reconnaissance. The bench mark shoreline surveys by NIMRD provide valuable information, but they are for the area with appreciable beach widths only.

An attempt has been made to estimate the existing beach profile by synthesizing the above sources of information. The coast south of Olimp is composed of many pocket beaches formed by groins etc., but the average beach profiles excluding these pocket beaches are assumed. For Mangalia, the northern part where the beach become narrow is taken for estimation.

Table E.7.12 is the result of estimate of the representative profiles of existing beaches. It also lists the required volume of beach fill sand, which is derived by taking the difference between the profiles of existing beach and planned beach fill (with the parameters in Table E.7.10) and multiplying it by the lengths of respective beaches. The total volume of beach fill for the all coast is estimated at about 3.2 million cubic meters.

Table E.7.12: Estimate of total volume of beach fill sand

Beach area	Existing beach condition					Unit fill volume (m^3/m)	Beach length (m)	Beach fill volume (m^3)
	Backshore height (m)	Backshore width (m)	Foreshore width (m)	Forshore slope, cot β_f	Inshore slope cot β			
Mamaia	–	–	–	–	–	150	1,200	180,000
Tomis	–	0	20	15	100	245	3,500	857,500
Agigea	–	0	10	15	100	83	750	62,250
Eforie Nord	–	0	10	15	90	301	1,550	466,550
Eforie Middle	–	0	15	12	90	278	1,550	430,900
Eforie Sud	–	0	15	12	90	278	2,300	639,400
Olimp	1.5	0	10	7	50	190	1,450	275,550
Jupiter, Aurora, Venus	1.5	5	–	10	50	108	2,400	269,200
Saturn	0	0	0	–	80	98	1,100	107,800
Mangalia	2.0	10	–	10	80	50	1,000	50,000
Total								3,214,050

ANNEX F:

NEW FACILITIES FOR SHORE PROTECTION AND REHABILITATION

Annex F: New Facilities for Shore Protection and Rehabilitation

F.1 Facility Layout Maps

Figures F.1.1 to F.1.7 show the layout of shore protection facilities, which are presented in **5.7** of Volume 1. The isobaths drawn in the figures are based on the bathymetric maps prepared by PROIECT S.A., CONSTANȚA in 1997. The present isobaths are expected to be different from those in these figures. When future projects are implemented with the facility plan in the Study, field studies of bathymetric and topographic surveys and inspection of existing structures should be carried out for reevaluation of the appropriateness of the proposed facility plan.

As explained in **5.7.4** of Volume 1, the coastal protection plan at the Eforie Sector has been changed from the original proposal made in February 2006 in reponse to the report of the strategic environmental assessment (SEA), which was submitted in February 2007. Figures F.1.3 and F.1.4 represent the original proposal, and thus the proposed facilities are not the same as those shown in Fig. 5.7.8. However, Figs. F.1.3 and F.1.4 are retained here to record the details of the original proposal.

F.2 Preliminary Design of Standard Cross Sections

(1) Principles of structural design

The structural types of proposed facilities have been selected in consideration of the following fundamental conditions and on-site situation:

1) Construction materials that are easily purchased on site are chosen to reduce the construction cost.
2) In Romania marine construction works are not executed frequently and mobilization of working vessels for marine construction is rather difficult. It is expected that a few domestic vessels may not be operational for all the time. Therefore, selection is made for the facilities of structural types that can be built with construction equipment on land.
3) Jetties are designed to have the capability to accommodate people for walking, fishing etc. just like many of the existing groins.
4) Detached breakwaters, jetties, and submerged breakwaters should have sufficient resistance against wave actions, because quite a number of existing groins and jetties mainly built with stones have been damaged by waves.

The main construction material is stone. There are several limestone quarries at a distance of 50 to 60 km from Constanța in the north and some others exist in further north. They can produce a large amount of limestone and it is possible to get stones in 1 to 4 ton size. Granite, which is of better quality than limestone, can be obtained at a quarry in Sibiwara only. Existing groins and jetties are all built with limestone blocks, so that the structural type of rubble mound with limestone blocks is chosen as the principal design.

With regard to the second condition, the construction method of extending a rubble mound structure from the shore is selected as a standard procedure, except for submerged, detached breakwaters, which need to be constructed by mobilizing floating vessels.

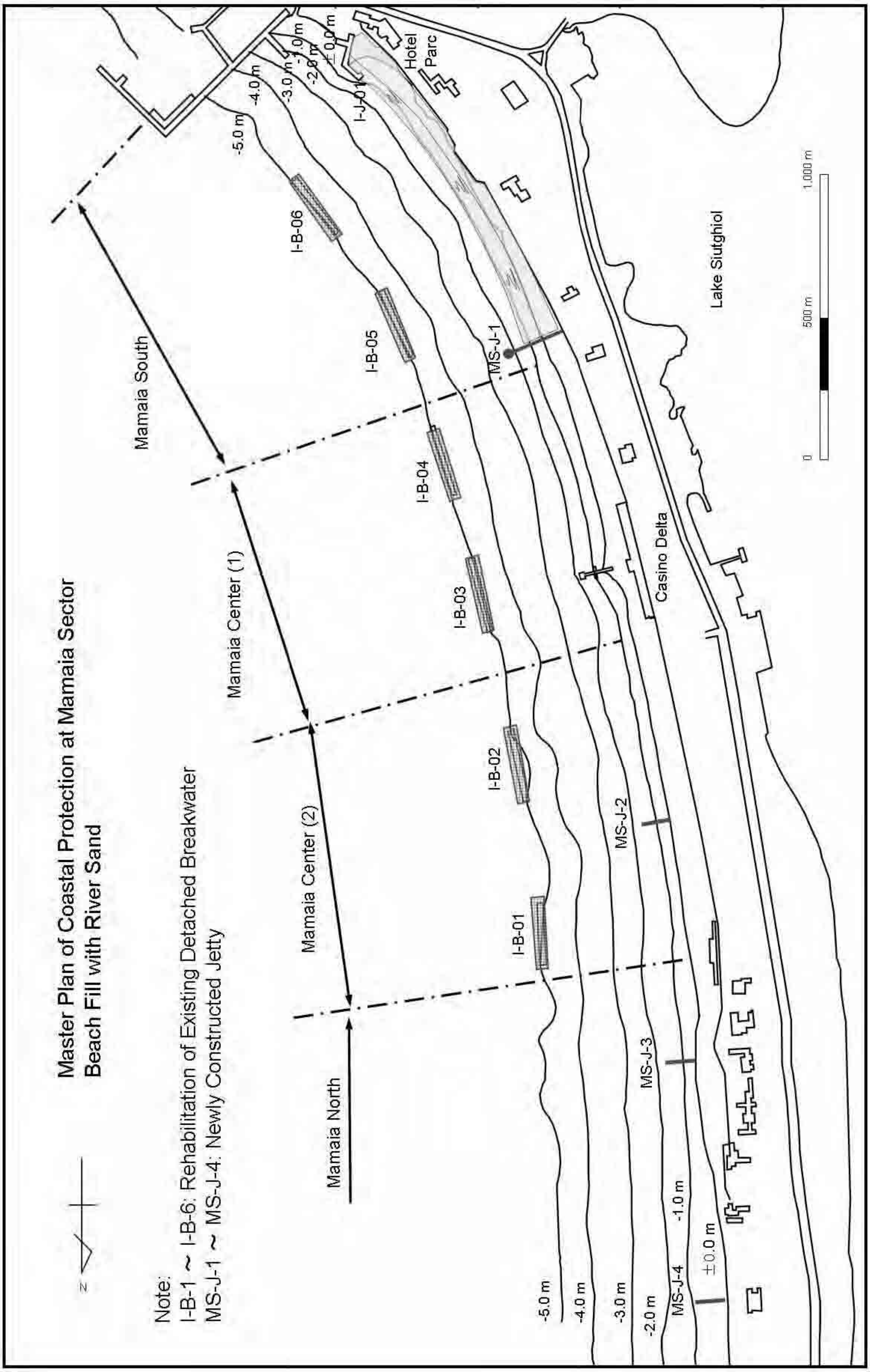


Fig. F.1.1 (1): Master pan of coastal protection at Mamaia sub-sectors (beach bill with river sand)

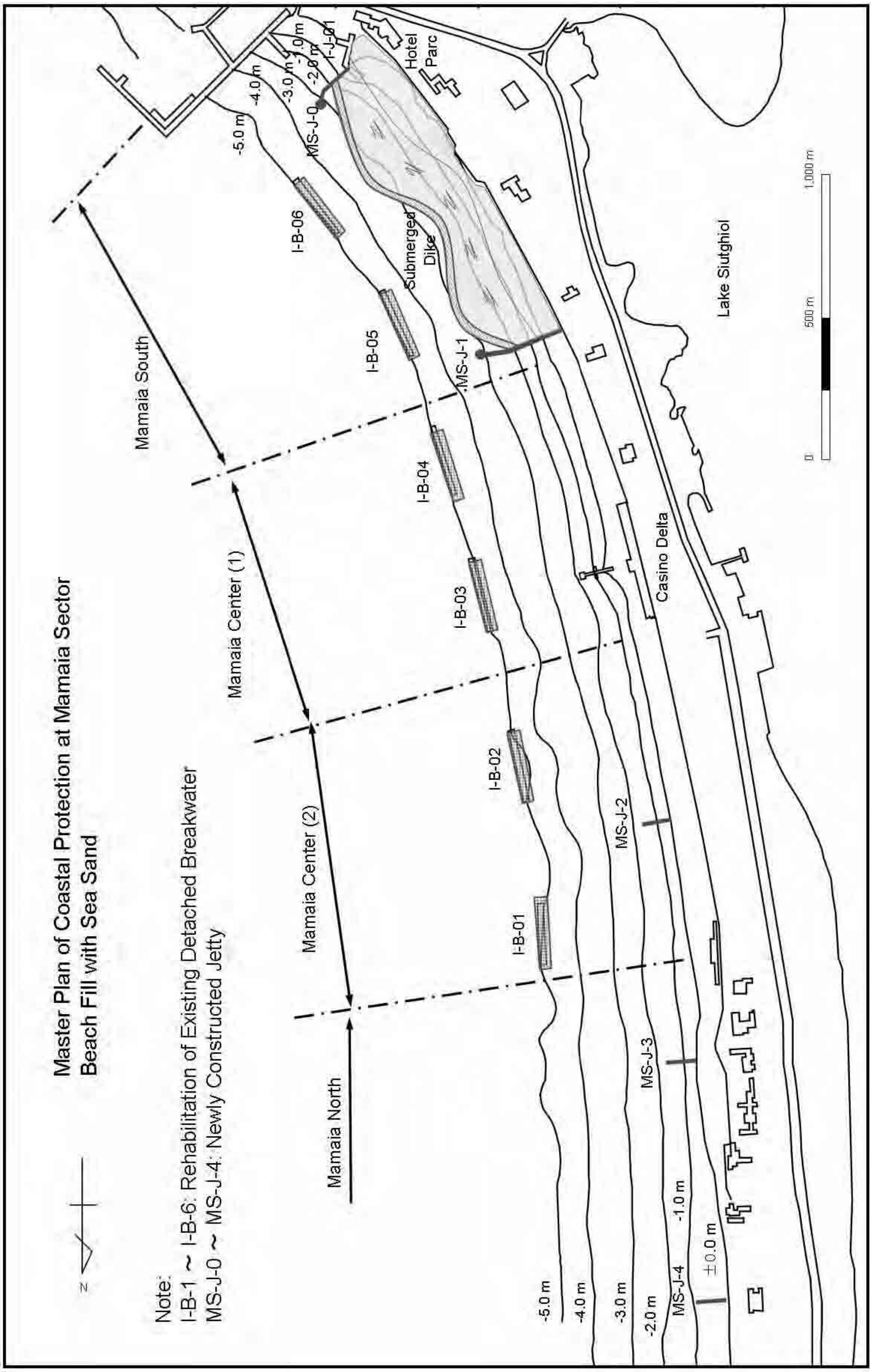


Fig. F.1.1 (2): Master plan of coastal protection at Mamaia sub-sectors (beach fill with sea sand)

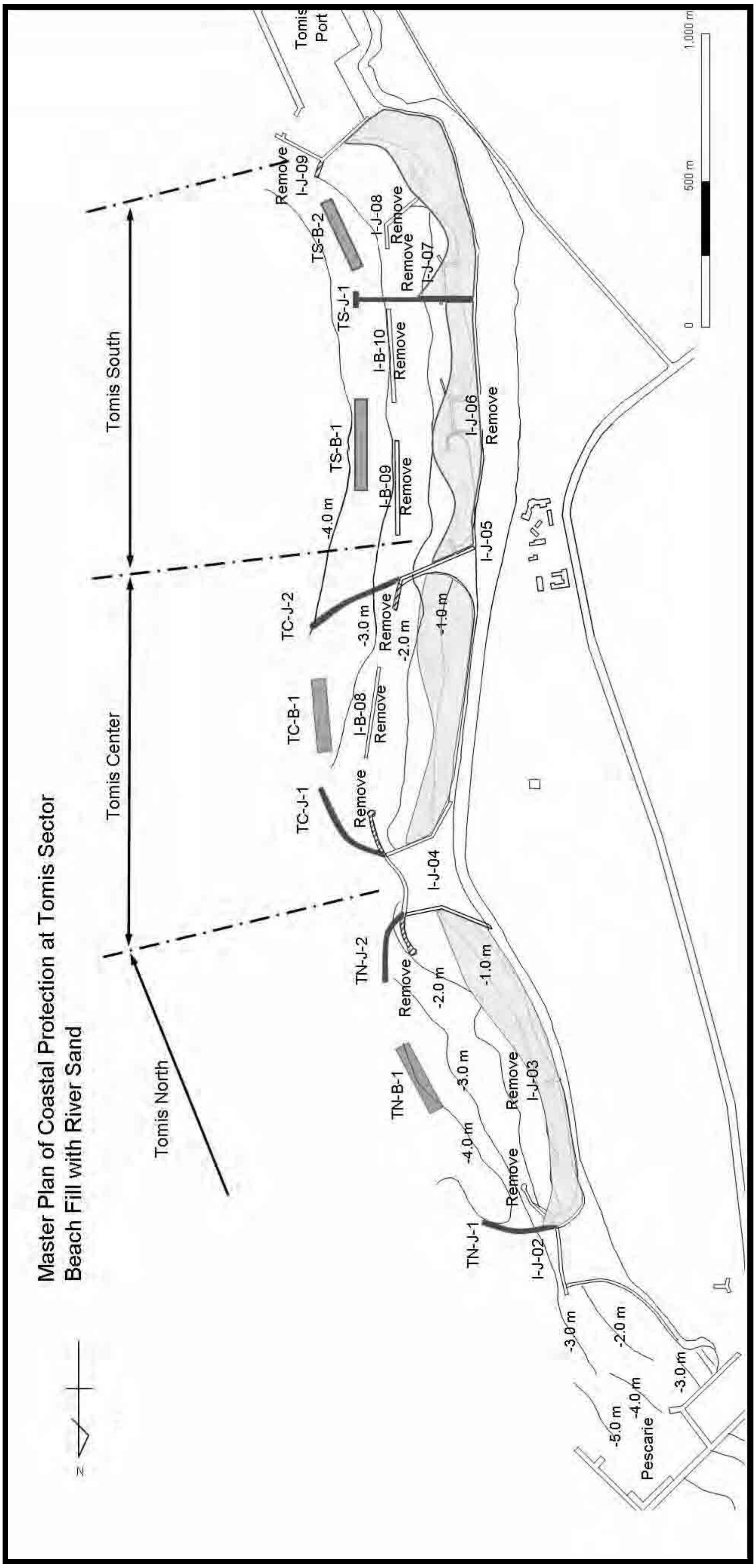


Fig. F.1.2: Master Plan of coastal protection at Tomis sub-sectors (beach fill with river sand)

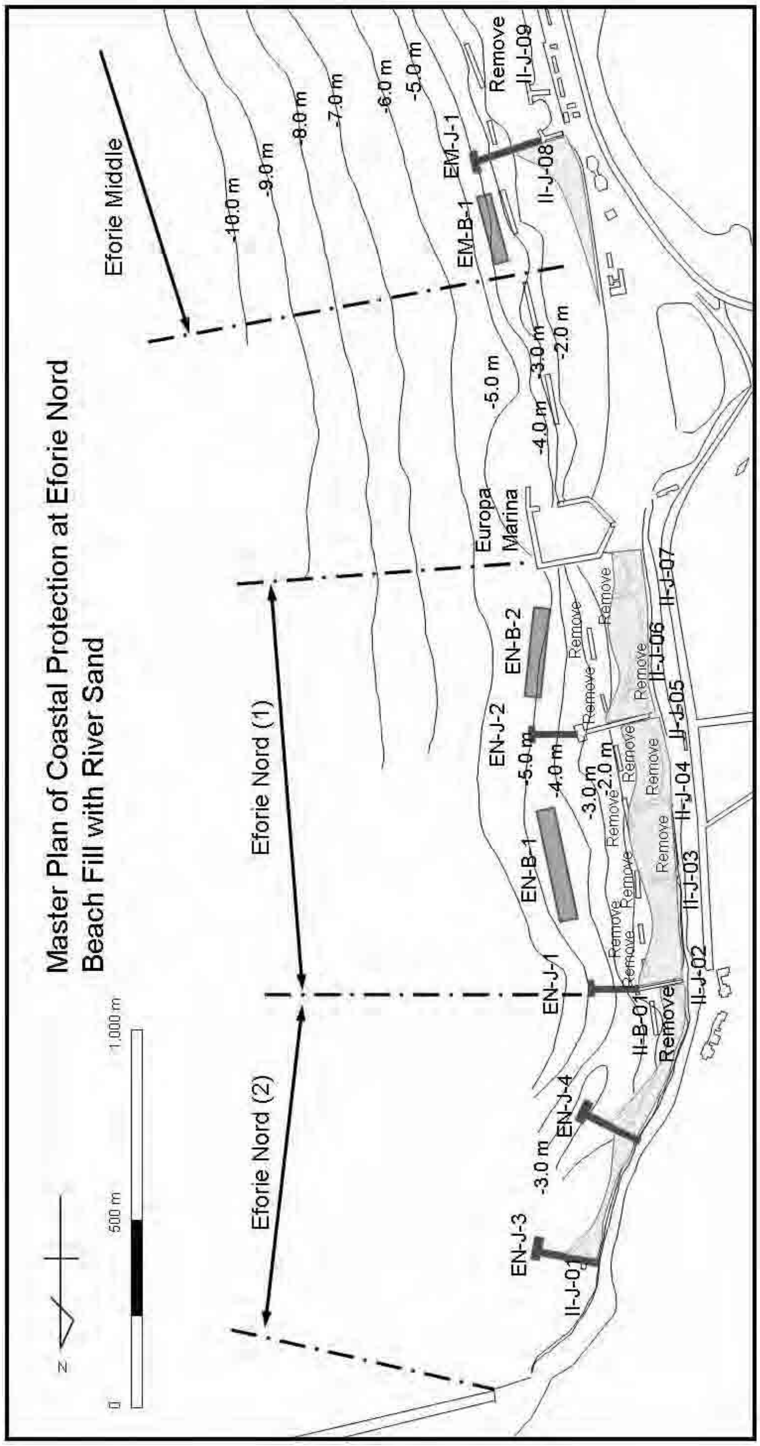


Fig. F.1.3 (1): Master Plan of coastal protection at Eforie Nord (beach fill with river sand)

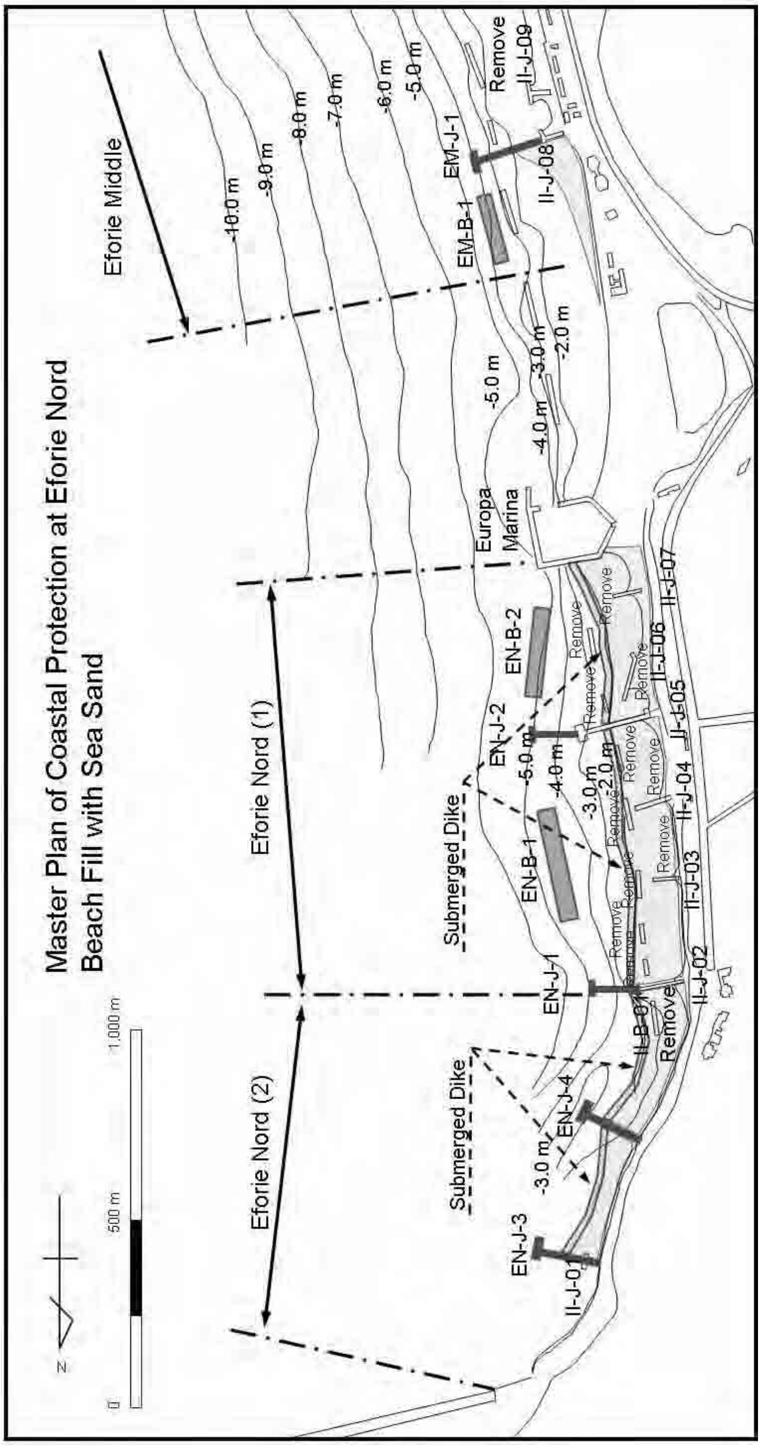


Fig. F.1.3 (2): Master plan of coastal protection at Eforie Nord (beach fill with sea sand)

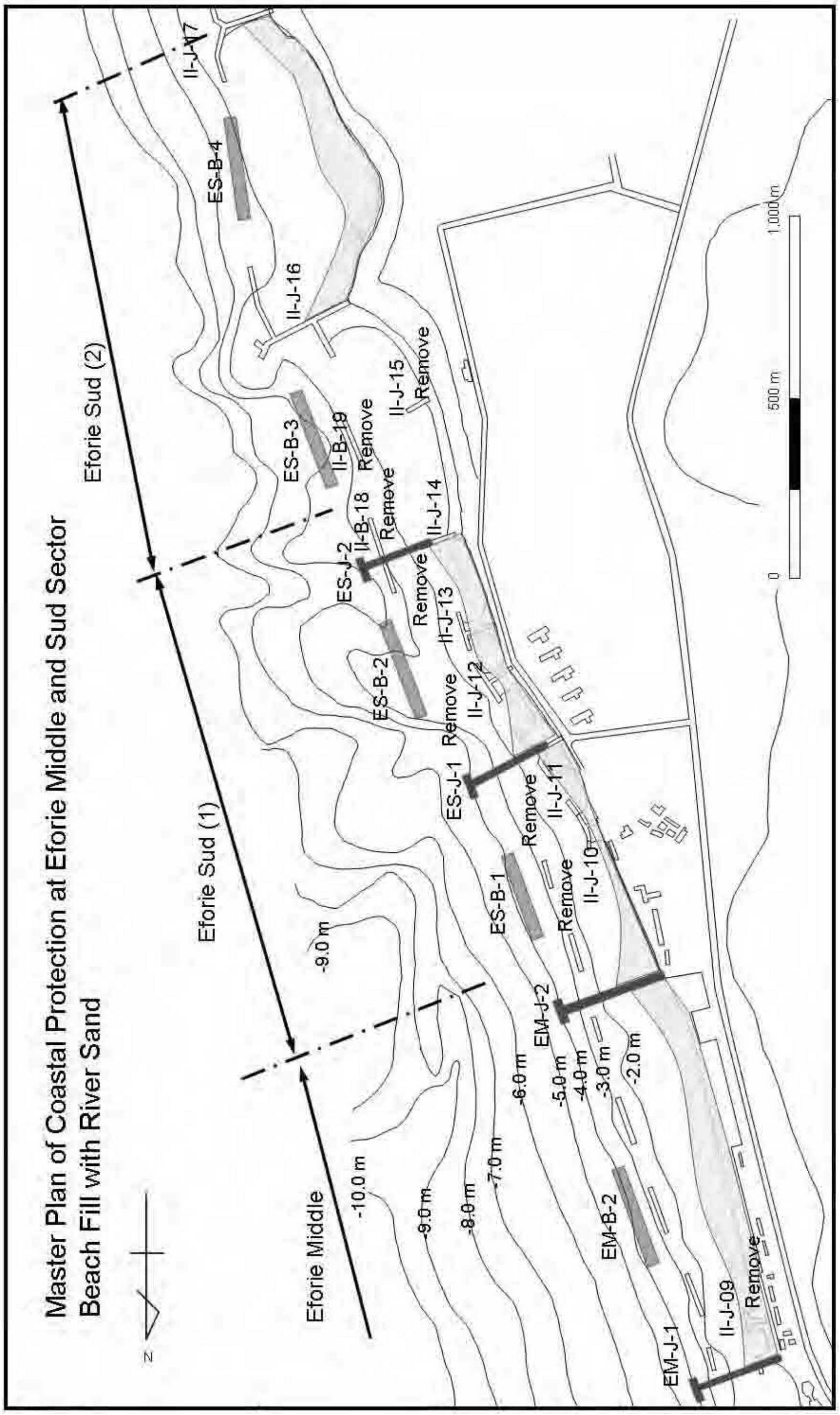


Fig. F.1.4: Master Plan of coastal protection at Eforie Middle and Sud Sector (beach fill with river sand)

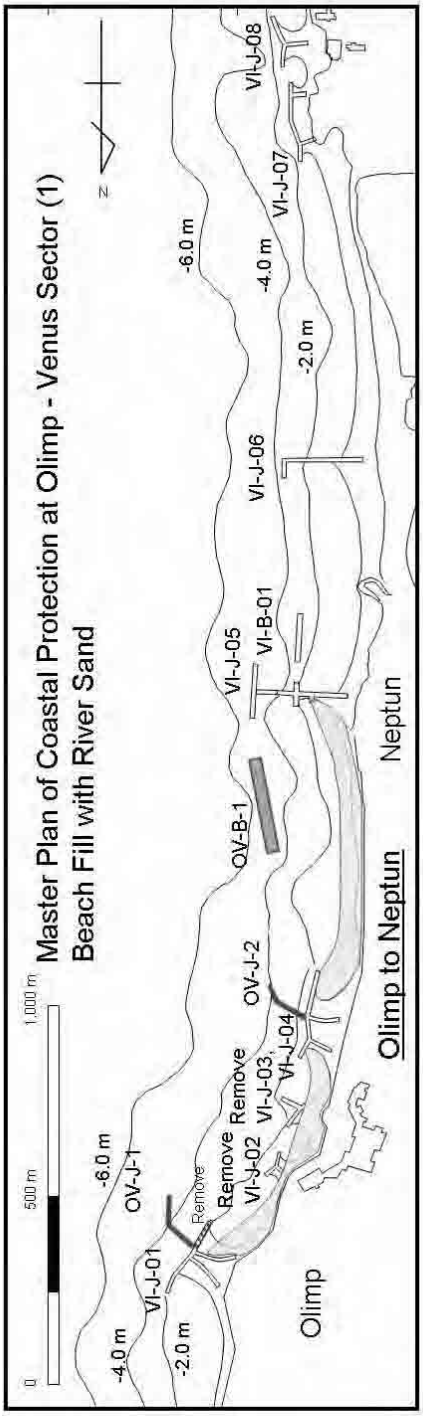


Fig. F.1.5: Master plan of coastal protection at Olimp-Venus sub-sectors (1)

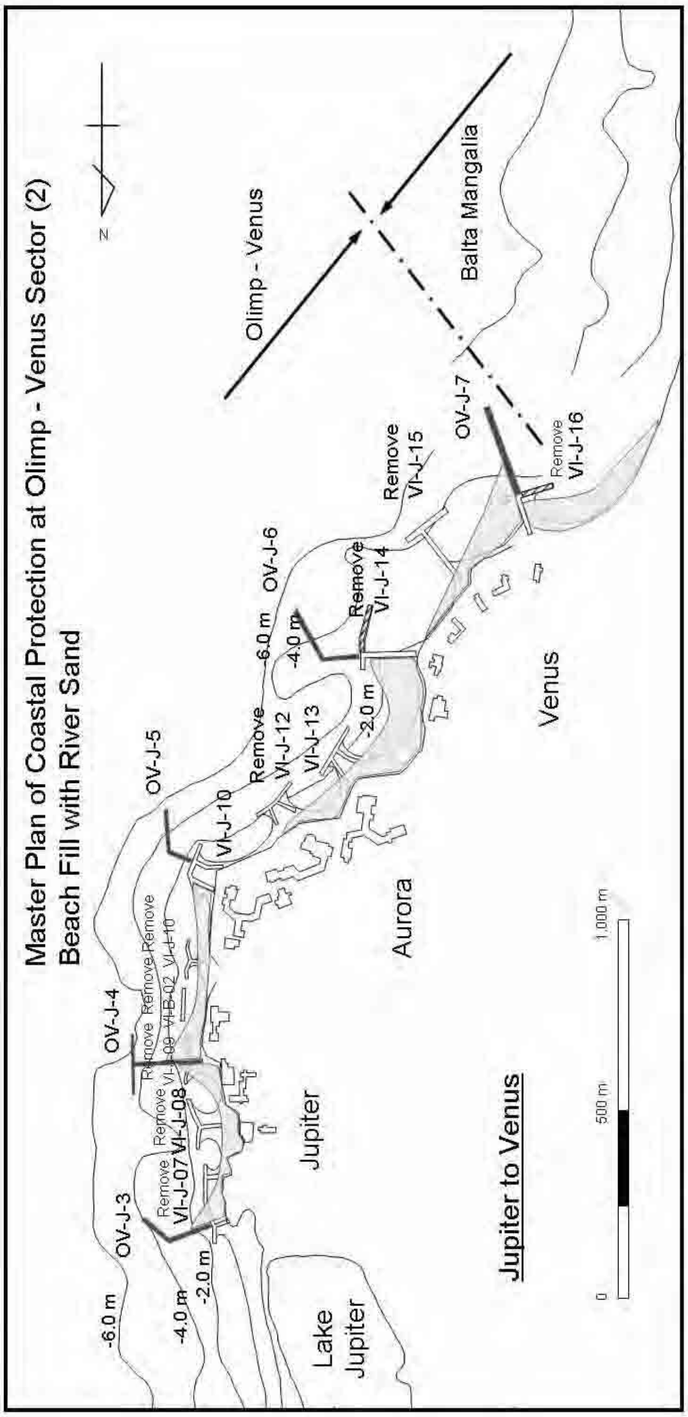


Fig. F.1.6: Master plan of coastal protection at Olimp-Venus sub-sectors (2)

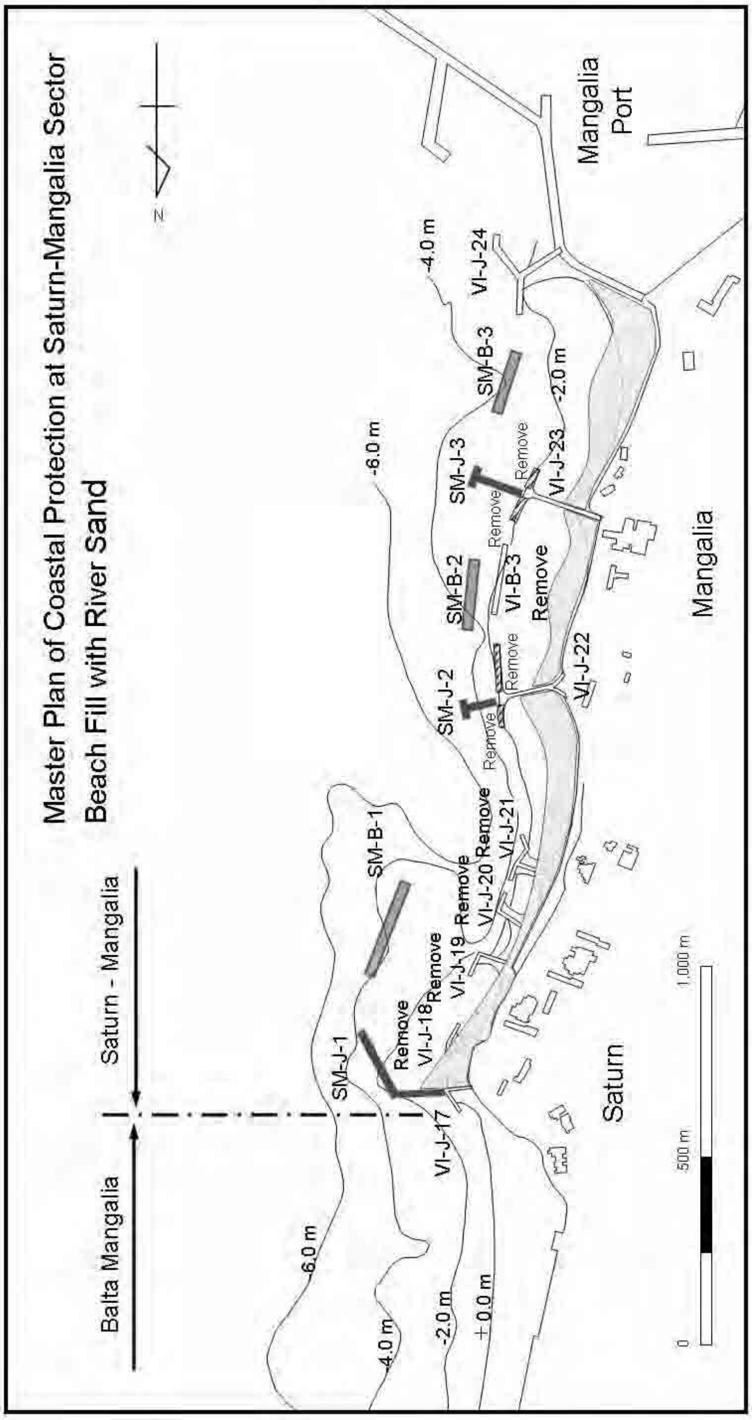


Fig. F.1.7: Master plan of coastal protection at Saturn and Mangalia

Most of existing groins are paved on their crowns with cast-in-place concrete of 20 to 30 cm thick and 3 to 4 m wide, upon which people can walk. However, the majority has been damaged with breakage, cracking, and/or scouring of foundation rubble stones. Therefore, pavement will be made with 1.0 m thick concrete to the isobath of about –2.0 m.

One reason of the damage of existing facilities by wave actions is an insufficient armoring of the slopes of rubble mound structures. Stabilopods are mostly placed in one layer and not exercising their interlocking functions. In the structural designs of proposed facilities, armor blocks are placed in two layers. For rubble armored structures, a gentle slope of 1 on 2 is adopted so that relatively small rubble stones can withstand the wave actions.

Standard cross sections of the proposed facilities based on the above design principles are shown in Figs. F.2.1 to F.2.12. In the following sub-sections, brief description is given for each structure.

(2) Rehabilitation of detached breakwater at Mamaia (Fig. F.2.1)

The breakwater was originally built as a mound type of breakwater made of concrete cube blocks with its seaward slope armored by 20-ton stabilopods. The crest elevation was +2.0 m above the datum level, but presently the crest is composed of a few legs of stabilopods exposed above the water, probably because of the general settlement or rolling down of stabilopods. Thus, the wave attenuation function of the breakwater is greatly reduced.

The most economical method for the rehabilitation of the breakwater will be to provide a mound of rubble stones behind the present deteriorated structure and to place 4.5-ton stabilopods in two-layer on top of it. Because incident waves will break on the seaward slope of the existing breakwater, waves attacking the newly placed 4.5-ton stabilopods will lose its energy and exercise less force on them. The stability number of stabilopods expressed in the K_D value of the Hudson formula is said to be 18. The significant wave height in the water of 5 m deep is estimated as $H_{1/3}$ ˜ 4 m, and the calculation based on the Hudson formula indicates the stability of 4.5-ton stabilopods at this water depth.

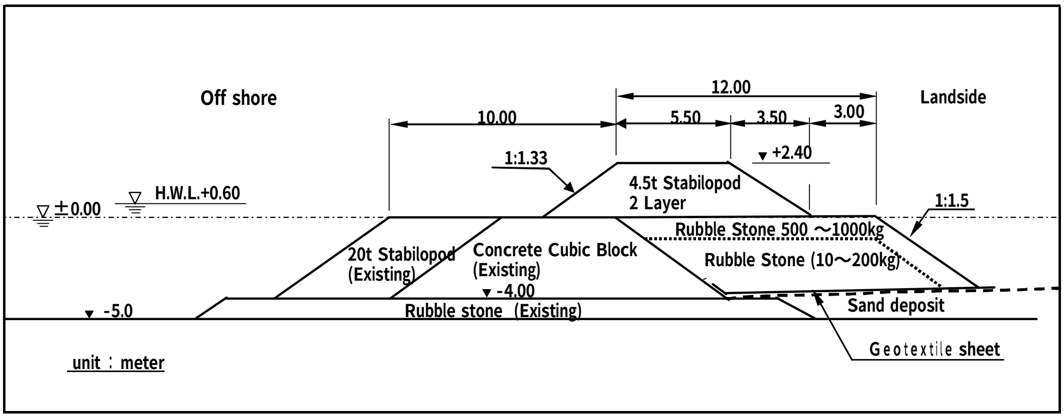


Fig. F.2.1: Cross section of rehabilitation of detached breakwater at Mamaia

(3) Sand-retaining groin at Mamaia South, MS-J-1 (Figs. F.2.2 and F.2.3)

The cross section shown in Fig. F.2.2 is applied for the section between the head of groin and the onshore distance of 100 m from the head. The side slopes are armored with two-layered rubble stones of 500 kg in weight, but the head itself is proposed to be armored with 4.5-ton stabilopods because of intensive wave actions there. The core section of rubble mound is designed with the crest elevation of +1.0 m to enable easy construction works with power shovels and other construction equipment. The gradient of side slopes is set at 1 on 2 for stability of armor stones and easy access of people to water. The side slopes are provided with underwater aprons of 5 m wide for foot protection against scouring. A walkway of 3 m wide is provided on the crown section, which will be built by cast-in-place concrete of 1.0 m thick.

For the trunk section between the shore and the point of 100 m from the groin head, the width of the foot protection apron is reduced to 2.0 m, because of weaker wave actions there, as shown in Fig. F.2.3.

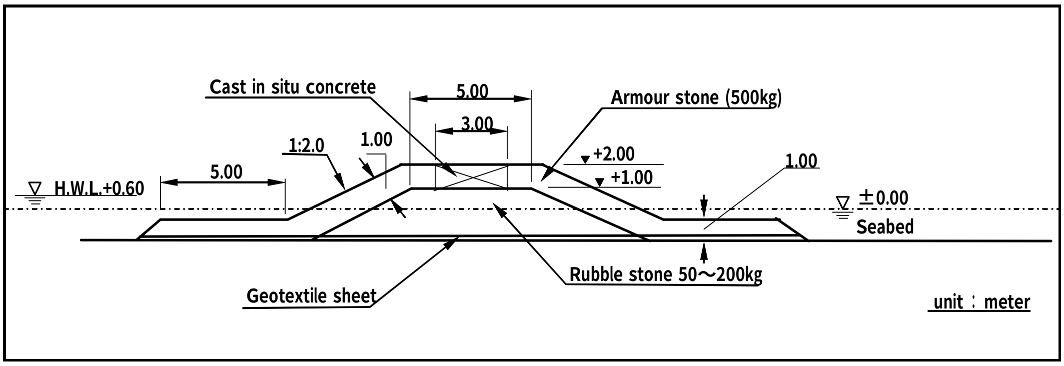


Fig. F.2.2: Cross section of sand-retaining groin at Mamaia South, MS-J-1 (1)
(offshore section of the length 100 m from the groin head)

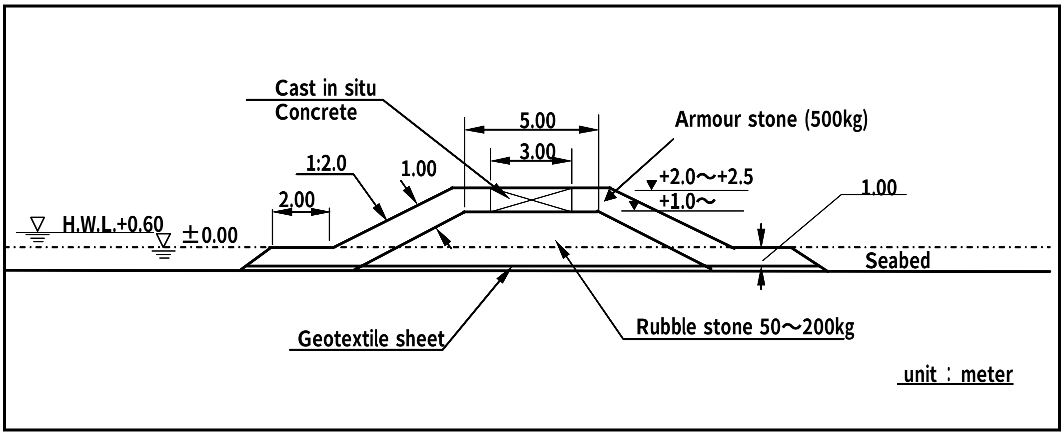
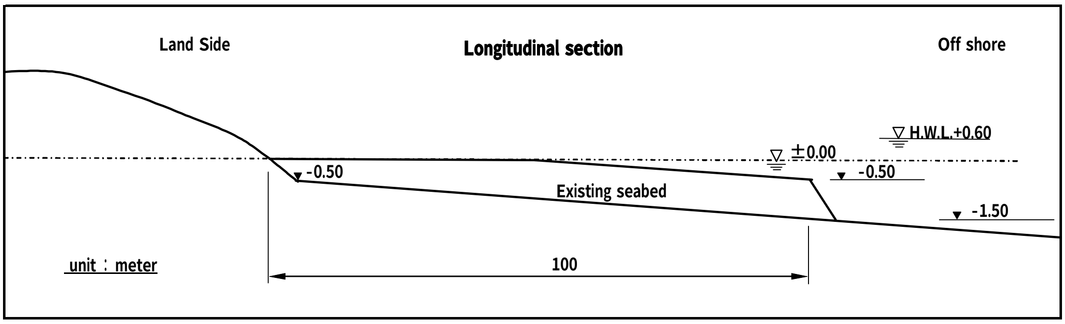


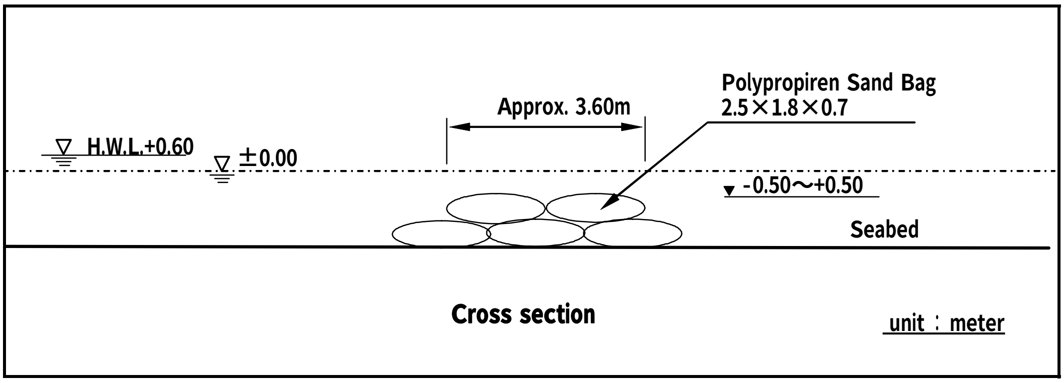
Fig. F.2.3: Cross section of sand-retaining groin at Mamaia South, MS-J-1 (2)
(trunk section between the shore and the point of 100 m from the groin head)

(4) Submerged groins at Mamaia, MS-J-2 to J-4 (Fig. F.2.4)

These groins are given the objectives of reducing the speed of longshore currents, which are the major factor responsible for alongshore sediment transport and beach deformation, either erosion or accretion. Because the net sediment transport in Mamaia Beach is toward the north as shown in Fig. 4.5.4, the sub-section of Mamaia Center is expected to experience intensive erosion by stopping of sediment supply from the south by the sand-retaining groin MS-J-1. By installing three submerged groins of MS-J-2 to J-4, the rate of the northward sediment transport will be reduced. The groins can be of simple structure because they are not intended to fully stop longhsore currents. Thus two layers of polypropiren sand bags filled with sand is selected as a structural type, and its crest is set at –0.5 to ±0.0 m. The length of these groins is planned to be 100 m.



(a) Longitudinal section



(b) Cross section

Fig. F.2.4: Groin MS-J-2, 3, and 4 at Mamaia

(5) Jetty type A (Fig. F.2.5)

This type of structure is applied for jetties at the head portion and the trunk section in relatively large water depth. The surface is armored with two-layered 4.5-ton stabilopods. The gradient of the slope is determined from the stability consideration, but the gradient of 1 on 1.33 is often employed for stabilopods.

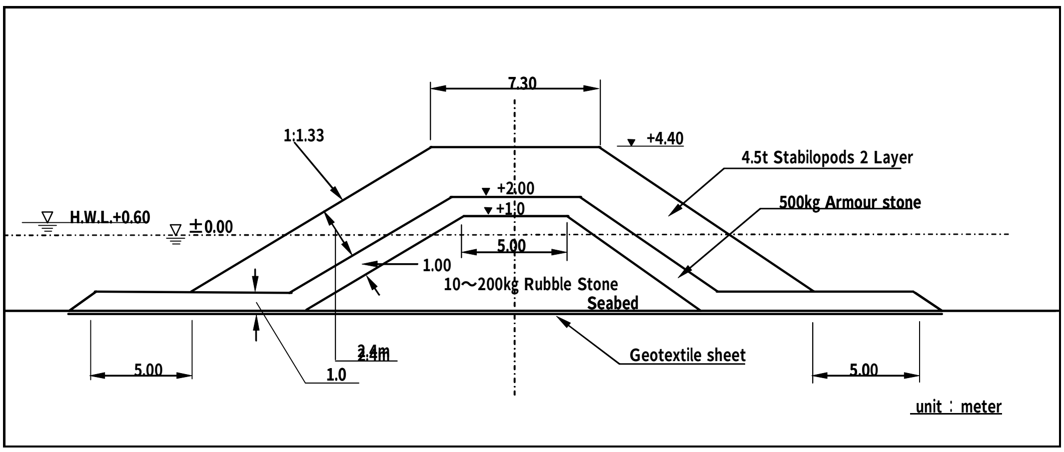


Fig. F.2.5: Cross section of jetty type A

(6) Jetty type B (Fig. F.2.6)

This type of structure is used for the portion of jetties that is located at the inner side of the curved section and not exposed to direct wave attacks. The seaward slope is armored with two-layered 4.5-ton stabilopods, but the landward slope is armored with rubble stones of 1 to 3 tons in weight against the impact of overtopped water and/or waves diffracted by the head of jetty.

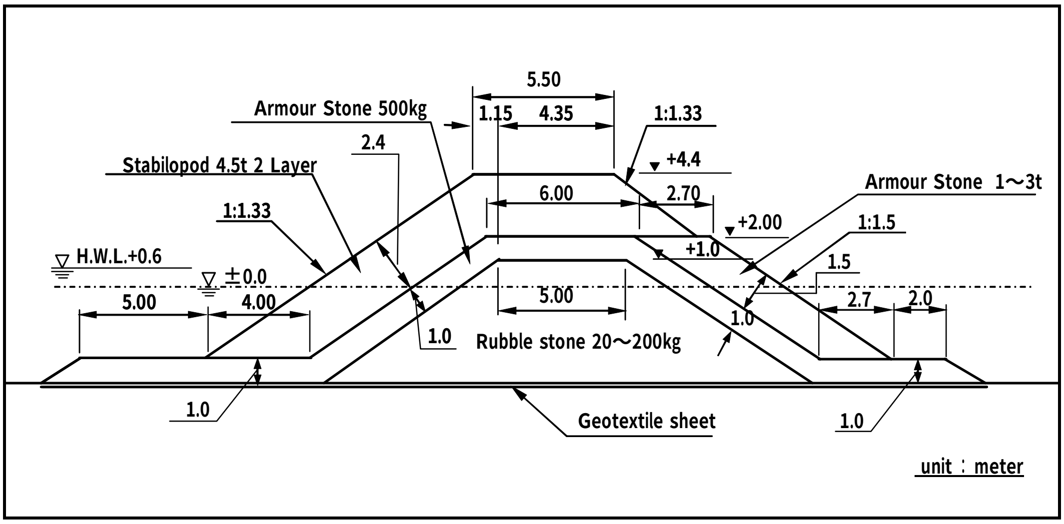


Fig. F.2.6: Cross section of jetty type B

(7) Jetty type C-1 (Fig. F.2.7)

This type of structure is used for the head section of jetties in the area where wave actions are relatively weak. The rubble stones of 1 to 3 tons in weight are placed in two layers for armoring. Actual size of rubble stones is to be determined by considering the degree of wave deformation based on detailed bathymetric surveys.

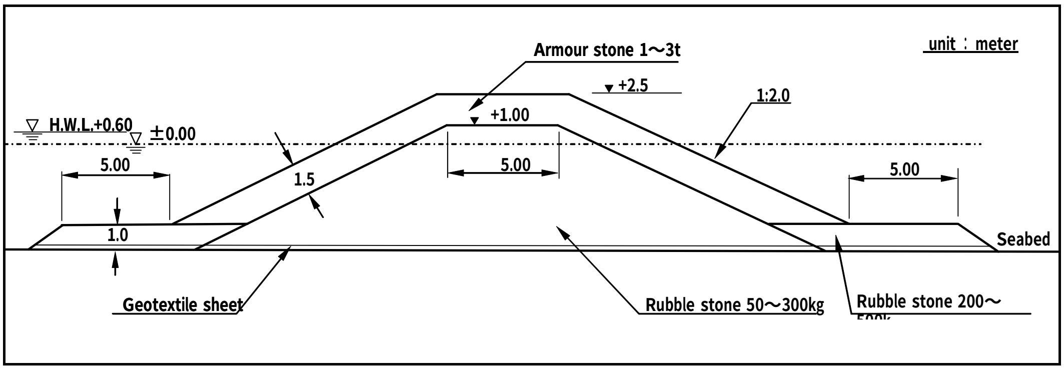


Fig. F.2.7: Cross section of jetty type C-1

(8) Jetty type C-2 (Figs. F.2.8)

This is the structure to be employed for the trunk sections of jetties in shallow water, where wave actions become weak. In the cost estimation of the proposed facilities of the coastal protection plan, this type of structure is used for the portion of jetties in water shallower than 2 m. As seen in Fig. F.2.8, a walkway is provided with thick concrete, but such a walkway is not provided in the jetty type C-1, because the wave actions are too strong for the walkway to maintain its integrity.

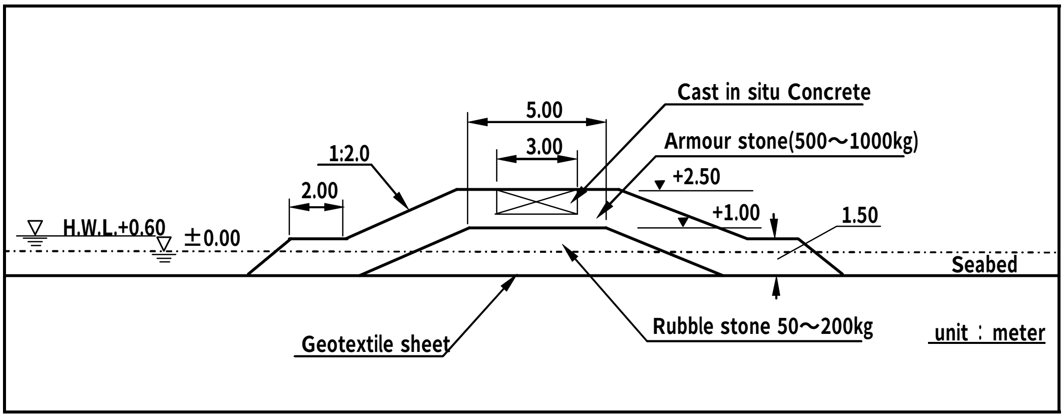


Fig. F.2.8: Cross section of jetty type C-2

(9) Jetty type E for rehabilitation of Jetty II-J-02 and Jetty II-J-05 (Fig. F.2.9)

The existing jetties II-J-02 and II-J-05 in Eforie Nord are preserved for the length of 100 m as the trunk sections of the new jetties EN-J-1 and EN-J-2, but they are in the deteriorated state, which require rehabilitation. The jetties II-J-02 and II-J-05 are made of rubble mound with concrete pavement of 0.3 m thick. The present damaged pavement is to be removed and a new walkway will be built with cast-in-place concrete of 1.0 m thick.

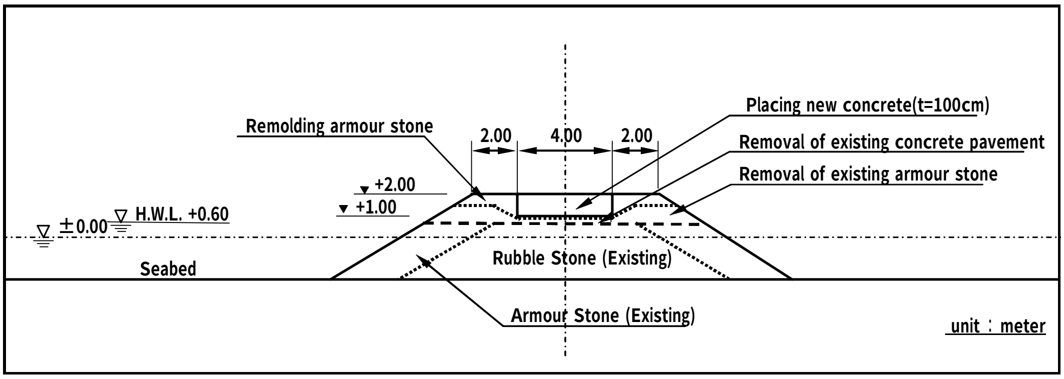


Fig. F.2.9: Cross section of type E [rehabilitation of jetty II-J-02(100 m), II-J-05(100 m)]

(10) Jetty type F for rehabilitation of Jetty II-J-05 (Figs. F.2.10)

The offshore portion of the existing jetty II-J-05 was built with concrete blocks and thin concrete pavement (0.3 m). However, the concrete pavement has been deteriorated by cracking, breakage, and washed-away. The crest elevation is about +1.0 m above the datum level, which is too low and allows wave overtopping and overflow. Thus, it is proposed to remove the whole deteriorated pavement and to cast fresh concrete directly on top of existing concrete blocks to the thickness of 0.8 m. The crest elevation will become +1.5 m, which is the same as the present wing section at the jetty head. Furthermore, mounds of rubble stones will be placed in the both sides of the existing concrete blocks for protection against scouring.

(11) Submerged breakwaters (Fig. F.2.11)

Submerged breakwaters or submerged, detached breakwaters are installed offshore for the purpose of wave attenuation through the process of wave breaking over their crests. Thus, less wave energy reaching to the shore, beaches will become more stable against wave-induced deformation process of erosion and accretion. Their crests are designed to be located underwater, and they are invisible from people strolling on the beaches. It is a preferable structure from the aesthetic viewpoint, but the crest must be wide enough to ensure the required degree of wave attenuation. In the present study, the crest elevation is set at –0.5 m below the datum level in consideration of the lowest recorded water level of –0.30 m (see **3.3**).

The core portion should be built with various sizes of stones, using small stones at the bottom layer next to the seabed and increasing the size toward the surface. The top of submerged breakwaters is to be covered with concrete blocks specially designed for submerged breakwaters. The required size of concrete blocks depends on the wave conditions and the individual shapes of blocks. Manufacturer's recommendation for the block size should be respected.

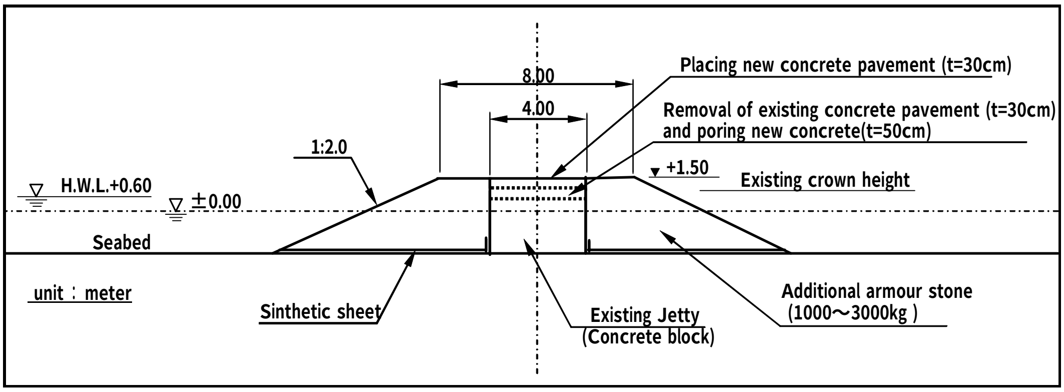


Fig. F.2.10: Cross section of type F [rehabilitation of jetty II-J-05 (100m)

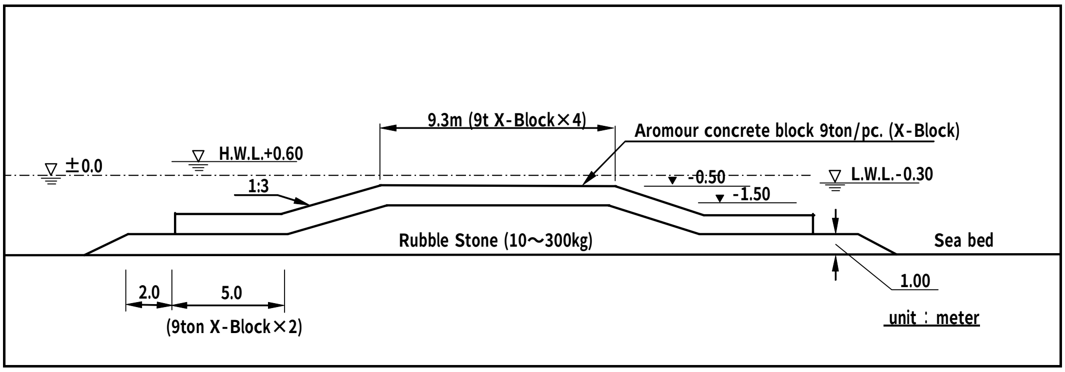


Fig. F.2.11: Cross section of submerged breakwater

(12) Sand-retaining underwater dike (Fig. F.2.12)

This facility is employed when beach fill is carried out by using sea sand mined from the seabed off Midia Port or Sulina Channel, in case that authorization of river sand mining from the Danube is not issued before execution of the coastal protection projects at Mamaia South and Eforie Nord. Because the sea sand is of fine grain size and the beach profile with sea sand becomes very gentle, the filled beach section requires an underwater dike to retain sand within the beach area.

The location of dike installation and its crest elevation depend on the beach fill plan. The cross section shown in Fig. F.2.12 is a temporary one and will be modified during the feasibility study stage.

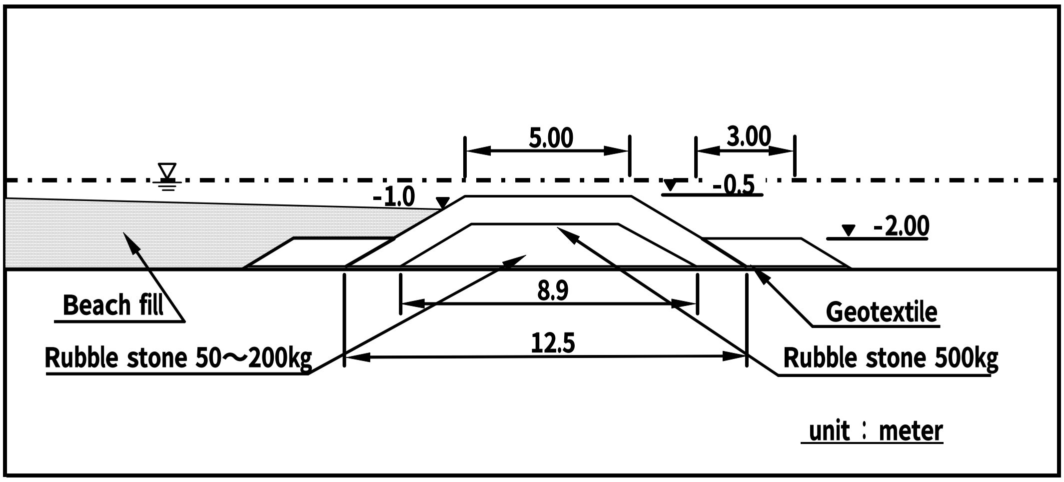


Fig. F.2.12: Cross section of underwater dike for beach fill

F.3 Conditions of Tentative Cost Estimate

The tentative estimate of construction cost listed in Table 5.9.1 of Volume 1 has been made under the following conditions:

1) The prices of materials, labor cost, rental fees of construction equipment, etc. are based on quotations at the market price in 2005.

2) The cost of river sand is evaluated as delivered at the Basarabi wharf along the Danube – Black Sea Canal for the projects at Mamaia and Tomis, while river sand for the projects south of Eforie is thought to be delivered at Agigea (South Constanța Port). Sea sand is assumed to be dredged by a trailing suction hopper dredge at the eastern area off Midia Port, stored in her hopper, carried to the offshore of a beach fill site, and ejected to the fill area through floating pipelines activated by the dredge's pump.

3) The loss of nourished sand is estimated as 10% in ten years according to several cases in Japan, or 1% per year. This loss of nourished sand is to be re-supplied in each phase of coastal protection plan, and the cost of re-supplying sand is included in the maintenance cost.

4) The cost of operation and administration is estimated as 3% of the initial investment cost, according to several past examples.

5) The cost of removing present facilities is based on the available drawings of standard cross section collected during the Study, because the detailed survey of existing structures were not undertaken.

6) The cost of rehabilitation of the existing facilities in the Mangalia Sector from Olimp to Mangalia for those requiring rehabilitation is assumed to be one half of the jetty type B (Fig. F.2.6), which has some similarity with these existing facilities.