TD
711
P47
2006

STUDY OF A SELF-SCOURING OUTFALL IN LAKE ONTARIO BY PHYSICAL AND
COMPUTER MODELING

by

Shanil Persaud
B.Eng. (Civil)
Ryerson University, 2004

A thesis

presented to Ryerson University

in partial fulfillment of the
requirements for the degree of
Masters of Applied Science
in the Program of
Civil Engineering

Toronto, Ontario, Canada, 2006

©Shanil Persaud 2006

PROPERTY OF
RYERSON UNIVERSITY LIBRARY

UMI Number: EC53528

INFORMATION TO USERS

The quality of this reproduction is dependent upon the quality of the copy submitted. Broken or indistinct print, colored or poor quality illustrations and photographs, print bleed-through, substandard margins, and improper alignment can adversely affect reproduction.

In the unlikely event that the author did not send a complete manuscript and there are missing pages, these will be noted. Also, if unauthorized copyright material had to be removed, a note will indicate the deletion.

UMI Microform EC53528
Copyright 2009 by ProQuest LLC
All rights reserved. This microform edition is protected against
unauthorized copying under Title 17, United States Code.

ProQuest LLC
789 East Eisenhower Parkway
P.O. Box 1346
Ann Arbor, MI 48106-1346

AUTHOR'S DECLARATION

I hereby declare that I am the sole author of this thesis.

I authorize Ryerson University to lend this thesis to other institutions or individuals for the purpose of scholarly research.

Signature

I further authorize Ryerson University to reproduce this thesis by photocopying or by other means, in total or in part, at the request of other institutions or individuals for the purpose of scholarly research.

Signature

Borrower's Page

Ryerson University requires the signatures of all persons using or photocopying this thesis.
Please sign below, and give address and date.

Name	Signature	Address	Date

ABSTRACT

Shanil Persaud, 2006. Modeling Study of a Self-Scouring Outfall in Lake Ontario by Physical and Computer Modeling. M.A.Sc., Civil Engineering

Coastal outfalls that discharge storm water and/or sewerage into bodies of water are part of a collection of critical municipal infrastructure that must be kept functioning properly at all times so as to avoid expensive frequent maintenance and environmental problems. The Green Road coastal outfall pipe, located in an embayment on the shores of Lake Ontario in the City of Hamilton, is subject to sediment plugging by waves that transport sediments from an eroding bluff to the east into the study outfall pipe, thereby reducing its hydraulic discharge capacity. To alleviate the problem of outfall blockage, a 1:15 scale undistorted physical (hydraulic) model was designed and built at the National Water Research Institute (NWRI) to study the performance of a self-scouring outfall (SSO), a structure that utilizes a combination of wave run-up slopes, converging walls and steep outlet channels to promote self-cleansing with respect to sediment to prevent direct sediment intrusion into the outfall pipe. A commercial computational fluid dynamics (CFD) model FLUENT, was used to study the internal hydrodynamics of the complex outfall structure. Results from the physical model determined that the performance of the SSO is a function of wave direction, water level, wave height and period, as well as sediment characteristics. Model results indicate that a SSO built on the shores of Lake Ontario would be able to scour a greater amount of sediment in conjunction with a high water level and wave height of 75.07m and 1.95 m, respectively. A modified SSO design reduced sedimentation on the wave run-up slopes by more than 25 % and was 100% effective in preventing direct sediment intrusion. The structure performed exceptionally better than a traditional outfall in terms of sediment handling. Therefore, it is expected a municipality can save a great deal of money on cleanouts by installing a self-scouring outfall to prevent outfall plugging.

ACKNOWLEDGEMENTS

The thesis presented here today was a result of months of hard work by the author and a host of other individuals who care deeply about solving the most intricate and complex problems found in nature. There are a number of people whom I would like to thank for their support, encouragement, advice and hard work.

I would like to thank first and foremost my supervisor Dr. Grace Luk of Ryerson University for her constructive advice in assembling my thesis, expertise and support over the last 5 years, and for providing me with encouragement to apply to graduate school. A special amount of gratitude and appreciation is extended to the engineers at Environmental Hydraulics Group, a division of Genivar Ontario Inc. who have been instrumental in helping me with this project, I would like to thank Dr. Alan Fok, Hydraulic Specialist; Jean-Luc Daviau, Project Coordinator; Shelly Kuan, Model Testing Engineer and Mohiuddin Mohammad, Hydraulics Engineer for a great amount of assistance during this amazing project. I would also like to thank Mark Stirrup (Project Manager) at Hatch Mott MacDonald and Susan Jacob (City Engineer), City of Hamilton for allowing me to publish the results of the study in my thesis. I am also very grateful for the constructive advice that was provided to our team by Dr. B.G. Krishnappan and Cheng He, Research Scientists at the NWRI's CCIW lab. I would also like to thank John Cooper and Bill Warrender of the CCIW for their technical assistance during wave generation and data logging. I would also like to thank Robin Luong, Civil Technician at Ryerson University for all his help during the semester to sieve samples. Last but certainly not least, I would like to extend a great deal of thanks to my family and close friends for their endless love and support throughout this long endeavour. I wish you all the best.

Table of Contents	Page
Author's Declaration	ii
Borrower's Page	iii
Abstract	iv
Acknowledgments	v
Table of Contents	vi
List of Tables	ix
List of Figures	x
List of Symbols	xiv

CHAPTER 1 INTRODUCTION.....	1
1.1 Scope of Study.....	2
1.2 Objectives.....	2
1.3 Expected Results.....	3
1.4 Project Team and Acknowledgement.....	3
CHAPTER 2 LITERATURE REVIEW	5
2.1 A Brief Overview of Physical Modelling in Coastal Engineering	5
2.2 A Physical Model	7
2.2.1 Credibility of Model Results and Similitude.....	8
2.3 Model Wave Considerations.....	9
2.3.1 The Reflection of Waves.....	10
2.3.2 Model Waves Breaking on a Beach.....	11
2.3.3 Wave Refraction, Diffraction and Shoaling.....	12
2.3.4 Generation of Laboratory Waves	13
2.4 Laboratory Effects in Physical Models.....	16
2.5 Scale Effects in Physical Models.....	17
2.6 Mobile Bed Sediment Models	18
2.6.1 Scaling of Model Fluid and Sediment	20
2.6.2 Data Collection for Physical Model Study	20
2.6.3 Scale Selection of Model.....	21
2.7 Computational Fluid Dynamic Simulations	21
CHAPTER 3 DESIGN OF THE PHYSICAL MODEL.....	23
3.1 Observations from Initial Site Visit.....	23
3.2 Determination of the Model Scale	28

3.3	Wave Analysis for Model Design.....	31
3.3.1	Refraction and Shoaling.....	34
3.3.2	Design of Model Wave Period.....	36
3.3.3	Design of Wave Height.....	37
3.4	Sediment Sampling in the Study Area.....	39
3.4.3	Design of Model Sediment.....	48
3.5	Sounding to determine model bathymetry	51
3.6	Selection of Model Water Level.....	55
3.7	Construction of Model Containment Wall.....	58
3.7.1	Wave Generator.....	60
3.7.2	Grid Alignment	62
3.8	Construction of Seawall and Embayment	63
3.8.1	Design and Construction of the Fixed-Bed	64
3.8.2	Movable-Bed and Embayment Construction	68
3.8.3	New Rock Geometry	71
3.9	Self-Scouring Outfall - I.....	75
3.9.1	Self-Scouring Outfall - II.....	80
3.10	Alignment of SSO	83
3.11	Wave Absorbers.....	87
3.12	Wave Staff Measurements.....	90
CHAPTER 4 RESULTS AND ANALYSIS OF THE PHYSICAL MODEL		95
4.1	Experimental Results from the Physical Model.....	95
4.2	Calibration of the Physical Model.....	96
4.2.1	Analysis and Discussion of Calibration Results.....	101
4.2.2	Simulation of a Littoral Current.....	103
4.2.3	Wave Absorbers and Basin Dynamics.....	105
4.2.4	Sediment Transport	106
4.2.5	Duration of Model Simulations.....	106
4.3	Performance of the Self-Scouring Outfall	107
4.3.1	Performance Review of Self-Scouring Outfall.....	108
4.3.2	SSO-I Simulation Analysis for Test 2 and 3.....	111
4.3.3	SSO-I Simulation Analysis for Test 4 and 5.....	120
4.4	Calibration Test II	125
4.5	Testing and Analysis of SSO-II for Test 5 and 8.....	130
4.5.1	Testing and Analysis of SSO-II for Test 3 and 9.....	135
4.5.2	Hydrodynamic Forces inside the SSO	138
4.6	Phase II Layout of Physical Model.....	140
4.6.1	Results of New SSO-II Orientation for Test 10 and 11.....	143
4.6.2	The Usage of Shell Blast for Test 11.....	149

4.7	Major Findings of the Physical Model Study	151
4.7.1	Results of Calibration Test.....	152
4.7.2	Testing Results of SSO-I	152
4.7.3	Testing Results of SSO-II.....	153
4.7.4	Phase-II Model Layout.....	153
4.7.5	Overall Performance of the Physical Model	154

CHAPTER 5 MATHEMATICAL MODELING OF A SELF-SCOURING OUTFALL 155

5.1	A Brief Overview of FLUENT.....	155
5.2	Applicability of Fluent To This Study	156
5.3	Description of Gambit	157
5.3.1	Meshing the SSO Model.....	158
5.4	Description of the FLUENT VOF Model.....	161
5.5	Turbulence Model Used in FLUENT	164
5.6	Selection of Model Parameters	165
5.7	Overview of the SSO Flow Experiment.....	165
5.7.1	Model Verification	166
5.7.2	Velocity Hydrodynamics in the SSO	168
5.7.3	Turbulence inside the SSO.....	169
5.7.4	Pressure Surfaces within the SSO.....	172

CHAPTER 6 CONCLUSIONS 174

6.1	Conclusions	174
6.2	Recommendations.....	176

APPENDIXES..... 178

Appendix A1:	Wave Approach Data from Buoy 45139	178
Appendix A2:	Refraction and Wave Breaking Calculations.....	180
Appendix A3:	Phase 1 & 2 - Model Layout.....	183
Appendix A4:	Characteristics of Embayment Sediment.....	187
Appendix A5:	Sieve Analysis of Samples	188
Appendix A6:	Specific Gravity of Brick Sand and Shell Blast	194
Appendix A7:	Model Wave Gauge Data (On CD-ROM).....	195
Appendix A8:	Selection of Model Parameters.....	196

REFERENCES 209

List of Tables

<u>Table</u>	<u>Title</u>	<u>Page</u>
2.2	THE ADVANTAGES AND DISADVANTAGES OF USING PHYSICAL MODELS	6
3.1	WAVE CHARACTERISTICS IN PROTOTYPE	34
3.2	RESULTS OF WAVE REFRACTION ANALYSIS	35
3.3	SUMMARY TABLE OF MODEL PERIOD CALCULATIONS	37
3.4	SUMMARY TABLE OF MODEL WAVE HEIGHT CALCULATIONS	37
3.5	NUMERICAL RESULTS FOR WAVE BREAKING ANALYSIS	39
3.6	SEDIMENT CHARACTERISTICS	47
3.7	CRITICAL MODEL PARAMETERS	50
3.8	SUMMARY OF POTENTIAL MODEL DIMENSIONS	53
3.9	WATER LEVEL CALCULATIONS	57
3.10	SAMPLE CALCULATION OF GRID SCALE	62
3.11	SUMMARY OF FIXED BED MEASUREMENTS	68
3.12	EMBAYMENT PARAMETERS FOR CALIBRATION SETUP	71
3.13	SHEET PILE DIMENSION SUMMARY TABLE FOR MODEL	74
3.14	EMBAYMENT SUMMARY TABLE	75
3.15	MODEL SLOPES FOR VARIOUS COMPONENTS	83
3.16	POINT SELECTION SUMMARY SHEET FOR THE ALIGNMENT OF THE SSO	84
3.17	PARAMETERS FOR WAVE ABSORBERS	89
3.18	SUMMARY OF MODEL AND PROTOTYPE DIMENSIONS	94
3.19	MODEL DESIGN SUMMARY TABLE	94
4.1	MODEL AND PROTOTYPE VALUES FOR CALIBRATION TEST 1 AND 2.	98
4.2	WAVE PRODUCTION FOR VARIOUS MODEL RUN DURATIONS.	107
4.3	TEST DESCRIPTION FOR PHYSICAL MODEL INVESTIGATION OF SSO	109
4.4	SEDIMENT DEPOSITION ON CRITICAL SLOPES OF SSO-I AND SSO-II.	110
4.5	INPUT PARAMETERS AND ACTUAL MODEL RESULTS FROM TEST 2 AND 3	111
4.6	INPUT PARAMETERS AND ACTUAL MODEL RESULTS FROM TEST 2 AND 3	120
4.7	INPUT PARAMETERS AND ACTUAL MODEL RESULTS FROM TEST 2 AND 3.	126
4.8	DURATION OF MODEL SIMULATIONS ACCORDING TO FROUDE CONDITION.	128
4.9	INPUT PARAMETERS FOR TESTS 5 (SSO-I) AND TEST 8 (SSO-II).	131
4.10	INPUT PARAMETERS FOR TESTS 3 (SSO-I) AND TEST 9 (SSO-II).	135
4.11	INPUT PARAMETERS FOR TESTS 10 (SSO-II) AND TEST 11 (SSO-II).	143
4.12	INPUT PARAMETERS FOR TESTS 12 (SSO-II)	149
5.1	OVERVIEW OF PRECISION PANEL	197
5.2	OVERVIEW OF THE MODEL PANEL	198
5.3	OVERVIEW OF MULTIPHASE MODEL PANEL	199
5.4	OVERVIEW OF VISCOUS MODEL PANEL	200
5.5	OVERVIEW OF MATERIALS PANEL	201
5.6	OVERVIEW OF PHASES PANEL	201
5.7	OVERVIEW OF OPERATING CONDITIONS	202
5.8	OVERVIEW OF VELOCITY INLET	203
5.9	OVERVIEW OF OUTFLOW	204
5.10	OVERVIEW OF SOLUTION CONTROLS	205
5.11	OVERVIEW OF SOLUTION INITIALIZATION	206
5.12	OVERVIEW OF ITERATION SELECTION	207
5.13	COMPARISON OF EXPERIMENTAL AND COMPUTER MODELING RESULTS	207

List of Figures

<u>Figure</u>	<u>Title</u>	<u>Page</u>
2. 1	THE ADVANTAGES OF THE USING PHYSICAL MODELS.	5
2. 2	PHYSICAL MODEL CONSISTING OF A FIXED AND MOBILE-BED	8
2. 3	PERFORATED WAVES GUIDES USED TO ABSORB/REFLECT WAVES	10
2. 4	WAVE REFRACTION, DIFFRACTION AND SHOALING	12
2. 5	WAVE DIFFRACTION	13
2. 6	PISTON WAVE GENERATOR	15
2. 7	FLAP-TYPE WAVE GENERATOR	15
2. 8	PLUNGER GENERATOR	15
2. 9	MUTLI-DIRECTION WAVE GENERATOR	15
3. 1	DESIGN PROCEDURE FOR SSO MODEL STUDY	24
3. 2	EXISTING CONDITION OF THE GREEN ROAD OUTFALL	25
3. 3	THE PARTIALLY SUBMERGED SHEET PILES	25
3. 4	SILTATION IN FRONT OF THE EMBAYMENT AREA	26
3. 5	SEDIMENT FREE OUTFALL	26
3. 6	OUTFALL HAS BEEN PLUGGED BY SEDIMENT	26
3. 7	CALM WATER IN EMBAYMENT	27
3. 8	TURBID WATER IN EMBAYMENT	27
3. 9	AN EXAMPLE OF A LAKE BUOY	31
3. 10	WAVE ARCS FOR 30°	32
3. 11	WAVE ARCS FOR 60°	32
3. 12	WAVE APPROACH DIRECTIONS	33
3. 13	REFRACTED WAVE PATTERN OF APPROACHING WAVES	34
3. 14	THE REFRACTION OF WAVE APPROACHING THE COAST	36
3. 15	WAVE BREAKING FOR A PERIOD OF FOUR SECONDS	38
3. 16	WAVE BREAKING FOR A PERIOD OF SIX SECONDS	38
3. 17	LOCATION OF SEDIMENT SAMPLE	40
3. 18	PONAR DREDGER	41
3. 19	GRAIN-SIZE ANALYSIS FROM EAST BEACH	42
3. 20	GRAIN-SIZE ANALYSIS FOR SEDIMENT IN WATER – EAST BEACH	43
3. 21	GRAIN-SIZE ANALYSIS FROM INSIDE.BERM LOCATION	44
3. 22	GRAIN-SIZE ANALYSIS FROM EMBAYMENT LOCATION	45
3. 23	GRAIN-SIZE ANALYSIS FOR SEDIMENT – NORTHEAST OF OUTFALL	46
3. 24	GRADATION CURVE OF CONSTRUCTION SAND USED FOR MOBILE-BED	51
3. 25	SURVEYING INSTRUMENT	52
3. 26	MEMBER OF THE SURVEY TEAM	52
3. 27	SMALL WATER CRAFT	52
3. 28	LOCATIONS OF SOUNDING POINTS	54
3. 29	HISTORICAL VARIATIONS OF LAKE ONTARIO WATER LEVEL	55
3. 30	AVERAGE HIGH AND LOW LONG-TERM WATER LEVEL IN LAKE ONTARIO	55
3. 31	OPTIMAL FLOOR ELEVATION	56
3. 32	HIGH WAVE PROPAGATION AND HIGH WATER LEVEL IN WAVE TANK	58
3. 33	DIMENSIONS OF PHYSICAL MODEL	60
3. 34	INSTALLATION OF THE WATERPROOFING MEMBRANE	60
3. 35	KELK WAVE GENERATOR	61
3. 36	KELK WAVE GENERATOR CONTROL SYSTEM	61
3. 37	MODEL GRID SYSTEM	62
3. 38	CONSTRUCTION PHASE OF SEAWALL	63
3. 39	COMPLETED SEAWALL	64
3. 40	PEG LOCATION FOR PHASE 1 LAYOUT (WAVES FROM THE NORTHEAST)	65
3. 41	PEG LOCATION FOR PHASE 2 LAYOUT (WAVES FROM THE EAST)	66
3. 42	CRITICAL COMPONENTS IN THE PHYSICAL MODEL	67

3. 43	DIFFERENT SIZES OF COARSE AGGREGATE	67
3. 44	FIXED BED AND PARTIALLY COMPLETED TRANSITION ZONE	68
3. 45	CONSTRUCTION PHASE OF MOVABLE-BED	69
3. 46	CONTAINMENT WALL	69
3. 47	ELEVATION PEGS IN A PARTIALLY COMPLETED MOVABLE BED	69
3. 48	EMBAYMENT AREA PRIOR TO THE INSTALLATION OF ARMOUR STONE	70
3. 49	CALIBRATION TEST CONFIGURATION OF EMBAYMENT AREA	70
3. 50	NEW ARMOUR STONE MATERIAL AND METAL SHEET PILES	72
3. 51	ARMOUR STONE GEOMETRY FOR PHASE I	72
3. 52	METAL SHEET PILES IN THE FIELD	73
3. 53	SCHEMATIC DRAWING OF SHEET PILES	74
3. 54	EXISTING SELF-SCOURING OUTFALL IN THE CITY OF TORONTO	75
3. 55	VARIOUS COMPONENTS OF THE SELF-SCOURING OUTFALL	76
3. 56	SCHEMATIC DRAWING FOR MODEL SSO – I	77
3. 57	DIFFUSER	78
3. 58	NE WAVES INTERACTING WITH THE SSO-I	79
3. 59	PLAN-VIEW OF THE INTERNAL MECHANICS OF SSO-I	79
3. 60	SCHEMATIC DRAWING OF MODEL SSO-II	80
3. 61	THE HYDRODYNAMICS WITHIN THE OUTFALL	81
3. 62	ELEVATION OF CRITICAL COMPONENTS	82
3. 63	ELEVATION AND MEASUREMENTS	82
3. 64	OUTFALL ALIGNED WITH OUTERMOST SHEET PILES	86
3. 65	OUTFALL ALIGNED WITH SMOOTHER COASTLINE	86
3. 66	OUTFALL ALIGNED WITH BERM 10.05 M LAKE WARD	86
3. 67	INNER GEOMETRY OF WAVE ABSORBER	87
3. 68	PLACEMENT OF MODEL WAVE ABSORBER	87
3. 69	PLACEMENT OF WAVE ABSORBERS IN THE MODEL	88
3. 70	WAVES LEAVING THE STUDY AREA	90
3. 71	WAVE STAFF MEASUREMENT APPARATUS	91
3. 72	SIDE VIEW OF WAVE STAFF	91
3. 73	ELECTRONIC MEASURING DEVICES	91
3. 74	SPACE BETWEEN WAVE STAFF AND FLOOR	91
3. 75	WAVE STAFF PLACEMENT IN THE MODEL	92
3. 76	POST-CONSTRUCTION PLACEMENT OF WAVE STAFFS	93
4. 1	CONDITION OF THE OUTFALL PRIOR WAVE SIMULATIONS	97
4. 2	FIRST CALIBRATION TEST FOR A PERIOD OF 1.03 SECONDS	99
4. 3	SECOND CALIBRATION TEST FOR A PERIOD OF 1.55 SECONDS	100
4. 4	INITIAL CONDITION OF EMBAYMENT	100
4. 5	CLEAN OUTFALL PIPE	100
4. 6	CONDITION OF EMBAYMENT AFTER TESTS	101
4. 7	SEDIMENT INSIDE OUTFALL PIPE	101
4. 8	LOCATION OF WAVE STAFFS	103
4. 9	SEDIMENT IN THE EMBAYMENT	103
4. 10	VARIOUS COMPONENTS OF SSO-I	108
4. 11	HYDRODYNAMICS OF THE SSO-I	111
4. 12	WAVE HEIGHT DATA FOR A 1.03 SECOND PERIOD	114
4. 13	WAVE HEIGHT DATA FOR 1.55 SECOND PERIOD	114
4. 14	TEST 2 – PRIOR TO WAVE ACTIVITY	115
4. 15	TEST 3 – PRIOR TO WAVE ACTIVITY	115
4. 16	TEST 2 - AFTER MODEL TEST	116
4. 17	TEST 3 - AFTER MODEL TEST	116
4. 18	TEST 2: BERM FORMATION	116
4. 19	TEST 3: BERM FORMATION	116
4. 20	THE MOVEMENT OF SEDIMENT ALONG THE COAST	118
4. 21	THE MOVEMENT OF SEDIMENT ALONG THE COAST	119
4. 22	TEST 4 – PRIOR TO WAVE ACTIVITY	120

4. 23	TEST 5 – PRIOR TO WAVE ACTIVITY	120
4. 24	WAVE HEIGHT DATA FOR A 1.03 SEC PERIOD	121
4. 25	WAVE STAFF DATA FOR A 1.55 SEC PERIOD	122
4. 26	TEST 4 – AFTER WAVE SIMULATION	123
4. 27	TEST 5 – AFTER WAVE SIMULATION	123
4. 28	ERODED SEDIMENT EXPOSING THE ELEVATION PEGS	124
4. 29	THE MOVEMENT OF SEDIMENT FROM EAST TO WEST	125
4. 30	WAVE HEIGHT DATA FOR A PERIOD OF 1.03 SECONDS	126
4. 31	CONSTRUCTION OF NEW EMBAYMENT	127
4. 32	NEWLY COMPLETED EMBAYMENT	127
4. 33	NEW EMBAYMENT AFTER WAVE ACTIVITY	127
4. 34	EROSION AND DEPOSITION OF SEDIMENT	127
4. 35	THE DEPTH OF SEDIMENT IN THE EMBAYMENT	129
4. 36	THE DEPTH OF SEDIMENT AS A FUNCTION OF DISTANCE	129
4. 37	NEW CRITICAL COMPONENTS IN THE SELF-SCOURING OUTFALL	130
4. 38	WAVE HEIGHT DATA FOR A 1.55 SECOND PERIOD	131
4. 39	TEST 5 – INITIAL CONDITION – SSO-I	132
4. 40	TEST 8 – INITIAL CONDITION – SSO-II	132
4. 41	TEST 5: SSO –I, AFTER TEST	132
4. 42	TEST 8: SSO-II, AFTER TEST	133
4. 43	TEST 5 – PLAN VIEW OF SSO-I	134
4. 44	TEST 8 – PLAN VIEW OF SSO-II	134
4. 45	WAVE HEIGHT DATA FOR A 1.55 SECOND PERIOD	135
4. 46	TEST 3: SSO-I – INITIAL CONDITION	136
4. 47	TEST 9: SSO-II – INITIAL CONDITION	136
4. 48	TEST 3: SSO-I AFTER WAVE ACTIVITY	137
4. 49	TEST 9:SSO-II AFTER WAVE ACTIVITY	137
4. 50	TEST 3:SSO-I – PLAN VIEW	137
4. 51	TEST 9:SSO-II – PLAN VIEW	137
4. 52	TEST 9:SSO-II – OUTFALL HYDRODYNAMICS	139
4. 53	TEST 9:SSO-II – FLOWS WITHIN OUTFALL	139
4. 54	TEST 3:SSO-I FLOWS WITHIN OUTFALL	139
4. 55	NEW ORIENTATION OF SSO	141
4. 56	CONSTRUCTION OF NEW LAYOUT	142
4. 57	COMPLETED BATHYMETRY	142
4. 58	POSITION OF WAVE ABSORBERS DOING A WAVE SIMULATION	142
4. 59	TEST 10:SSO-II -WAVE HEIGHT DATA FOR A PERIOD OF 1.55 SECONDS	144
4. 60	TEST 11:SSO-II - WAVE HEIGHT DATA FOR A 1.55 SECOND PERIOD	144
4. 61	TEST 10 –INITIAL CONDITIONS	145
4. 62	TEST 11 – INITIAL CONDITIONS	145
4. 63	TEST 10 – LOW WATER LEVEL – HWH	145
4. 64	TEST 11 – HIGH WATER LEVEL – HWH	145
4. 65	WAVE FRONT APPROACHING SSO	147
4. 66	SHEET PILES BREAKING WAVE FRONT	147
4. 67	WAVE FRONT HAS WRAPPED ITSELF AROUND THE SHEET PILE AREA	147
4. 68	TEST 10: LOW WATER LEVEL – HWH	148
4. 69	TEST 11: HIGH WATER LEVEL – HWH	148
4. 70	TEST 10 – BERM FORMATION	148
4. 71	TEST 11 – BERM FORMATION	148
4. 72	SIEVE ANALYSIS FOR SHELL BLAST MATERIAL	150
4. 73	PLACEMENT OF SHELL BLAST IN MODEL	151
4. 74	SHELL BLAST AFTER THE SIMULATION	150
5.3	WIRE FRAME VOLUME MODEL OF SSO PRIOR TO MESHING.	157
5.4	TETRAHEDRAL MESHING SCHEME.	158
5.5	MESH ELEMENTS FOR X-DIRECTION.	159
5.6	MESH ELEMENTS FOR Y-DIRECTION.	159

5.7	MESH ELEMENTS FOR Z-DIRECTION.	159
5.8	MESHED SSO CONSISTING OF 19,424 ELEMENTS.	160
5.9	SCHEMATIZATION OF SSO STRUCTURE.	161
5.10	VELOCITY VECTORS WITHIN THE SSO.	168
5.11	“FILLED” VIEW OF TURBULENCE IN SSO.	170
5.12	INTERIOR VIEW OF TURBULENCE WITHIN THE SSO.	170
5.13	PRESSURE ZONES IN SSO.	172
5.14	PRECISION PANEL.	197
5.15	SOLVER PANEL.	198
5.16	MULTIPHASE SOLVER PANEL.	199
5.17	VISCOUS MODEL PANEL.	200
5.18	MATERIALS PANEL.	201
5.19	PHASES PANEL.	201
5.20	OPERATING CONDITIONS PANEL.	202
5.21	B.C. PANEL.	203
5.22	VELOCITY INLET DATA PANEL.	203
5.23	OUTFLOW PANEL.	204
5.24	SOLUTION CONTROLS PANEL.	205
5.25	SOLUTION INITIALIZATION PANEL.	206
5.26	ITERATION PANEL.	207
5.27	ITERATION TIME STEPPING DIAGRAM.	208
5.28	RESIDUAL CONVERGENCE HISTORY FOR 30 MINUTE SIMULATION.	208

Lists of Symbols

Symbol	Description	S.I. Units
A	coefficient	Dimensionless
σ_k and σ_ϵ	turbulent Prandtl numbers for k and ϵ	Dimensionless
β	beach slope	Dimensionless
$C_{3\epsilon}$	constant	Dimensionless
d	depth contour or depth of water	m
E_q	Specific heat of phase and temperature	J/g·K
E	energy	J
f	function of,	Dimensionless
F_e	elastic compression force	N
F_i	inertial force	N
F_g	gravitational force	N
F_n	net force	N
F_u	viscous force	N
F_σ	surface tension force	N/m
Fr	Froude number	Dimensionless
g	gravitational acceleration	m/s ²
G_b	the generation of turbulence kinetic energy due to buoyancy	m ² /s ³
G_k	the generation of turbulence kinetic energy due to the mean velocity gradient	m ² /s ³
γ	submerged sediment specific weight	KN/m ³
h_b	wave breaking water depth	m
H_b	deep water wave height	m
H	wave height	m
k	turbulence	m ² /s ²
k_s	bottom roughness	m
L_b	deep water wave length	m
L	length	m
λ	length parameter	Dimensionless
m	mass	kg
N_L	length scale	Dimensionless
p	pressure	Pa
ρ	density	kg/m ³
π	dimensionless parameters	Dimensionless
Re	Reynolds number	Dimensionless
S_k	Turbulence Sources for the k- ϵ Model	kg/m-s ³
S_ϵ	Turbulence Sources for the k- ϵ Model	kg/m-s ⁴
t	time	s
T	wave period	s
T	temperature	°C
τ	shear stress	N/m ²

ϵ	dissipation rate	m^2/s^3
μ	dynamic viscosity	$\text{N}\cdot\text{s}/\text{m}^2$
V	velocity	m/s
ν	kinematic viscosity	m^2/s
x	x coordinate distances	m
y	y coordinate distances	m
Y_M	the contribution of the fluctuating dilatation in compressible turbulence to the overall dissipation rate	$\text{kg}/\text{m}\cdot\text{s}^3$
z	z coordinate distances	m

Chapter 1 Introduction

Coastal outfall structures are often given the important task of discharging stormwater from an urban catchment into a body of water for disposal. All too often, the outlets of these structures are susceptible to sediment blockage caused by wave energy which transports large quantities of sediment from the lake into the outlet pipe. When the outfall becomes plugged with sediment its hydraulic discharge capacity is reduced, rendering it ineffective. Open-ended outfall pipes are common hydraulic structures located throughout the world to discharge a variety of sources such as sewerage, stormwater, industrial waste or a combination of these.

The Green Road stormwater outfall is located on the shores of Lake Ontario in Stoney Creek, Hamilton, Ontario and severe problems of sediment intrusion have been reported in the past. It was observed by an engineering survey team that four processes are contributing to outfall blockage: an eroding bluff to the east, a littoral current that transports sediment from east to west, waves that transport sediment directly into the embayment area and a convex shoreline that causes approaching wave energy to converge and travel into the embayment area.

Currently, the staff of the City of Hamilton must remove sediment from the outfall pipe at a frequency of two to ten times per year to prevent stormwater from backing-up onto the roads. The cleanout frequency of the outfall is a function of the intensity and number of storm events that occur over the lake in any given year. It has been concluded by city officials that it is not economically feasible to increase the elevation of the Green Rd. outfall pipe or undertake other major expensive initiatives to rectify the problem.

It has been proposed that a self-scouring outfall installed in-line with the existing outfall would prevent direct sediment intrusion into the outlet pipe and will require less maintenance than the present outfall. A self-scouring outfall (SSO) is a unique structure that employs sloping channels to break/deflect wave energy and also narrow outlet channels which facilitate the removal/entry of urban stormwater/lake water. At present, there is only one full scale functioning self-scouring outfall in Ontario and it is located in

the Eastern Beach area of Toronto. All available literature has indicated there is no data concerning the performance evaluation of this structure. It was found during the literature review there is very little information in general concerning the performance of a self-scouring outfall and the current thesis is an attempt to present a thorough analysis of the structure through physical and mathematical modelling.

1.1 Scope of Study

In this thesis, a 1:15 scale undistorted physical model was designed and constructed at the prestigious National Water Research Institute (NWRI) in Burlington, Ontario. The performance of the self-scouring outfall in the physical model was analyzed for its ability to scour sediment from its surfaces by using a combination of wave up-rush slopes, converging walls and steep outlet channels to prevent direct sediment intrusion into an existing outlet pipe. In this study, waves were generated from two directions to simulate the wave action found in Lake Ontario. Similarly, two model outfalls (SSO-I and SSO-II) were constructed and tested. The second model (SSO-II) had modified geometry that incorporated design modifications to improve the performance of the outfall. Data from a lake buoy was used to reproduce the annual historical high and low water levels found in Lake Ontario and corresponding wave heights. Upon completion of the physical model study, a numerical model (FLUENT 6.1) was used to observe the internal flow hydrodynamics of the outfall.

1.2 Objectives

The objective of this thesis was to study the performance of a self-scouring outfall by utilizing a physical model and a computational fluid dynamic (CFD) software program (FLUENT) at Ryerson University. While conducting the physical model investigation visual observations and measurements were made to numerically quantify performance and to ascertain whether or not sediment had accumulated on the surfaces of SSO-I & SSO-II. Sediment erosion and deposition in the vicinity of the SSO structure was also examined to determine whether or not model results agreed with observations and first-hand accounts of a berm formation in the vicinity of the SSO structure. FLUENT 6.1 CFD software was also used to simulate a flow discharging from the SSO-II inlet pipe.

Various models in FLUENT were used to simulate velocity and turbulence profiles to identify potential problem spots of the flow as it travelled through the structure. The observations from FLUENT were correlated with a physical model scouring test to see if results agreed with each other.

1.3 Expected Results

It is expected during the calibration test that sand deposition will occur in the outlet embayment pipe. This event would be a direct indication the model is reproducing real world conditions because outlet plugging occurs at least 2-10 times per year in the Green Road embayment. According to observations in the field it can also be concluded that a high water level in conjunction with a high wave height will ultimately result in outfall blockage.

It is predicted that SSO-II will demonstrate better sediment handling characteristics and prevent the migration of sediment into the outlet pipe by utilizing a sophisticated and superior hydraulic design that has been modified from SSO-I. It is also expected the usage of CFD software to simulate the complex hydrodynamics within the outfall will prove beneficial as it provides researchers with an added dimension of insight into how well the outfall will perform under all operating conditions.

1.4 Project Team and Acknowledgement

Environmental Hydraulics Group (EHG), a global leader in hydraulic transient analysis spearheaded this amazing project by managing and initiating all aspects of data acquisition, model testing and final outfall design. Coordination between all parties was handled by EHG along with a field investigation to acquire wind and wave climate data. EHG collected sediment samples from around the embayment site and used a special dredging tool to sample the bottom sediment found in Lake Ontario. The self-scouring outfall model was constructed in-house by EHG staff and was also tested and modified by Environmental Hydraulics Group.

As a graduate student I did following for my thesis project:

- Aided the consultants by conducting sieve analysis for sediment samples from the field and by using Microsoft Excel to analyze numerical data and produce graphical figures for my report
- Conducted a literature review for information that was relevant to the operation and performance of the self-scouting outfall
- Assisted in the verification of Phase I bathymetry and the actual calculation of Phase II bathymetry
- Worked with consultants to construct the physical model for testing of the SSO (the SSO was constructed by EHG)
- Helped to acquire data after model simulations, for example:
 1. Measurement of sediment depth
 2. Quantifying the accumulation of sediment on the SSO and in the sheet pile area
 3. Capturing still photos before and after tests and the making of videos during the test
 4. Analysis of wave data after each model simulation
- Prepared AutoCAD drawings of: Model layout for Phase 1&2, Plan view drawing for SSO-1&2, Embayment rock geometry, Alignment of SSO and Placement of wave staffs
- Researched and learned how use Gambit and Fluent computational fluid dynamics software to model the interior hydrodynamics by simulating a flow leaving the outfall
- Used all available data to produce my thesis

Chapter 2 Literature Review

The following chapter presents a review of the fundamental literature related to physical and numerical modelling. This literature assists in understanding the various workings of a self-scouring outfall (SSO), a new type a coastal outfall structure that is meant to block direct sediment intrusion into an existing coastal outfall pipe.

2.1 A Brief Overview of Physical Modelling in Coastal Engineering

Physical modelling of sedimentary transport processes employing the usage of movable-beds is a well established laboratory modelling technique dating from the 1870's when Louis J. Fargue first reproduced a scaled model of the Garonne River (Hughes, 1993). Others (for example Froude and Reynolds) made significant contributions around the same time by conducting experiments in shipping flumes and rivers to advance the science of hydraulic modelling (Hughes, 1993). Physical modelling in coastal engineering had its beginnings in Europe but has gained a strong foothold throughout the world as an essential modelling tool. Physical modelling addresses the vital need to understand such complex phenomenon as wave mechanics and sediment transport and the effect these processes have on coastal structures. Svendsen (1985) summarized the important advantages of physical modelling in Figure 2.1.

Figure 2. 1 The advantages of the using physical models.

Hughes (1993) listed additional points in Table 2.1 outlining the benefits and drawbacks of physical models.

Table 2. 1 The advantages and disadvantages of using physical models

#	Advantages	Disadvantages
1.	A physical model automatically integrates the appropriate governing equations so that simplifying assumptions made in analytical and numerical models are not necessary.	Scale effects occur in models that are smaller than the prototype and result from the inability to simulate all relevant variables in correct relationship to each other. Viscous force is a common scale effect analogous to coastal models.
2.	The small size of the model permits easier data acquisition at a reduced cost when compared to the collection of field data which can be time consuming and expensive.	Laboratory effects occur when it is not possible to create realistic forcing conditions. An example would be the usage of unidirectional waves in the model compared to directional waves in the prototype.
3.	Ability to run multiple simulations, experimental control and ability to model rare environmental conditions	Forcing functions and boundary conditions acting in nature are not included in a physical model. An example could be wind shear which alters the near shore circulation currents.
4.	Ability to get visual feedback from the model.	In rare situations a physical model can be more expensive to operate than a numerical model.
5.	Experimenters can immediately use qualitative observations to modify testing process.	
6.	When physical models are designed and built correctly they add credibility to the decision making process.	
7.	Coastal laboratory modelling techniques will continue to play an important roll because of inherent limits of deterministic fluid mechanics due to turbulence.	
8.	The application of new techniques like Laser Doppler Velocimetry gives researchers the opportunity to study the physical relationship of fluid flow beyond what is currently known.	
9.	The accuracy of mathematical modelling is a function of how accurate the mathematical relationships are on which they are based. Mathematical models point out important deficiencies and physical models allow the researcher to carefully study the physics of the phenomenon in a controlled environment.	
10.	Scale models remain the best analog computer for their ability to reproduce complex boundary conditions.	
11.	Researchers can use physical models to quickly gain insight into complex flow relationships without having to rely on a complete mathematical description of the process.	

2.2 A Physical Model

There are a numerous applications of physical models in engineering and in many other technical disciplines. It has been reported by Hughes (1993) that the majority of physical models in coastal engineering operate at a reduced scale when compared to the prototype. Also, it is a well known fact that as model size decreases it becomes harder to study the flow regimes associated with a particular process. There are a variety of reasons to build a full-scale model to study a phenomenon, however, the cost often associated with such a venture is far greater than using a scaled down model to simulate the prototype conditions. Price (1978) stated one of the greatest benefits of a physical model is that it gives the researchers the opportunity to visualize and observe the process attentively.

At present, there exist three broad categories of physical models: validation, design and process model. A validation model is unlikely to resemble the prototype since it is constructed to minimize scale effects and to represent simplified assumptions in a numerical model (Dalrymple, 1989). According to Kamphuis (1991) design models are used extensively for engineering applications despite their high cost and need for extensive field data and process models study the physics of a physical model in laboratory setting so that the researcher can derive a new understanding of a process.

The present study will utilize a fixed-bed and movable-bed to study the scouring potential of a movable-bed model composed of granular material (sand) which can react to hydrodynamic forces (water waves) to simulate prototype conditions. When movable-bed models are designed correctly the data obtained from these models can have great credibility when compared to prototype conditions. Shown in Figure 2.2 are the fixed-bed model used to promote wave generation and the movable-bed used for simulating sediment transport.

Figure 2. 2 Physical model consisting of a fixed and mobile-bed

2.2.1 Credibility of Model Results and Similitude

Qualitative data produced by physical models must be interpreted with great care to ensure test results agree with prototype conditions. A physical model has the capability to simulate prototype conditions better than either equations or dimensional analysis. These models can be regarded as powerful engineering tools that have the capability to automatically calibrate complex boundary relationships inherent to three-dimensional sediment transport processes. Models results in good agreement with prototype conditions will prove to be more economic because costly redesigns can be avoided.

According to Hughes (1993) a correctly designed model should emulate the prototype in all respects. Proper similitude (agreement between prototype and model conditions) can only be achieved when all forces acting on the model and prototype are in correct proportion to one another. Forces that are not in proportion to the model and prototype should be small enough so as not to greatly affect the model results. Similitude in coastal hydraulic models can be established by using similitude by calibration, differential equations, dimensional analysis and scale series. Similitude by calibration

was adopted in this study to make use of extensive field data. The basic idea behind similitude by calibration is to use prototype conditions known to exist in the past and try to reproduce those same conditions in the model (Hughes, 1993).

The ability of a coastal movable-bed model to achieve similitude is a function of geometric, kinematic and dynamic similarity. Simply stated, similarity between all components of similitude can only be achieved if the numerical ratios between the model and prototype are kept the same. Coastal movable-bed models which include structures are often undistorted because it is necessary to preserve identical vertical and length scales to facilitate simple model design and data acquisition. The Froude scaling criterion is used to model the majority of coastal and hydraulic model flows in which inertial forces are balanced by gravitational forces, which occurs in flows with a free surface.

2.3 Model Wave Considerations

There has been much work performed over the years by Dalrymple (1989) and Le Mehaute (1989) concerning the hydrodynamics of model waves. The self-scouring outfall physical model employs the usage of a fixed concrete bed for wave generation and a mobile bed for sediment transport, both of which work in unison. Dalrymple (1985) classified wave motion occurring in a coastal region as either “short” or “long.” To elaborate, short waves have a period of 1-20 seconds and can be used to study wind/wave interaction, swell effects on coastal structures and beach erosion (Dalrymple, 1993). Long waves have periods in the order of minutes to days (Dalrymple, 1993) and can be used to study such events as tsunamis and tidal effects (Hughes, 1993).

According to wave period classification data presented earlier by Dalrymple, (1993) the ideal model wave for the self-scouring outfall study would entail the generation of short waves which would satisfy both prototype conditions and experimental work by others. Hughes (1993) goes on to say that model wave basins allow for the study of wave refraction, diffraction, shoaling and breaking. Le Mehaute (1976) stated that most short waves used in physical models can be considered non-dissipative or fully turbulent. Hughes (1993) summarizes the previous statement by saying “Scaling of short-wave models assumes that waves experience negligible energy

loss due to friction prior to breaking; and when energy is lost, it is lost entirely through turbulent dissipation processes in association with wave breaking mechanisms.”

2.3.1 The Reflection of Waves

A self-scouring outfall is a unique structure that utilizes a combination of wave up-rush slopes, converging walls and steep outlet channels to allow for the flushing of storm water and the diversion and reflection of approaching wave fronts. Depending on the situation, the reflection of short waves in a model may be greater or less than in the prototype (Hughes, 1993). Research conducted by Le Mehaute (1976) stipulates that reflected waves in a model may be smaller than in a prototype because of smooth model surfaces and increased friction.

Le Mehaute (1976) also argues that reflection coefficients in model rip-rap (armour stones) are higher when compared to the prototype because model rip-rap is less porous due to viscous effects. The solution to this problem is to geometrically increase the size of the rip-rap to compensate for such effects. Additionally, wave reflection from boundaries in a physical model are not desirable because this does not occur under prototype conditions, thus model boundary wave reflection can be minimized by using wave absorbers to channel incoming waves to outside the study area (as shown in Figure 2.3).

Figure 2. 3 Perforated waves guides used to absorb/reflect waves

Hughes (1993) also cautions researchers that if wave reflection is a significant process in the model basin then the model scale should range from 1:10 – 1:20 to gain a more insightful picture of the event.

2.3.2 Model Waves Breaking on a Beach

When building a physical model of a coastal region, it is important to scale the critical components like sand, beach slopes, and structures to reduce errors when conducting simulations. Hughes (1993) reported that when model waves break on a beach the entrained air bubbles are larger in the model because of surface tension. Literature seems to suggest the breaking water depth formula presented by Goda (1975) correlates well with field data when compared to others from Sunamura (1983) and Komar and Gaughan (1973), this formula is presented below.

$$\frac{H_b}{L_b} = A \left[1 - \exp \left\{ \frac{-1.5\pi h_b}{(1 + 15(\tan \beta)^{4/3})} \right\} \right] \quad [2.1]$$

Where, h_b = wave breaking water depth; A = coefficient; β = beach slope; L_b = deep water wave length and H_b = deep water wave height

Miller (1972) conducted experimental work on wave breaking and concluded that surface tension does indeed play a critical role and should be considered in theoretical work when dealing with wave breaking. However, Stieve (1985) validated Le Mehaute's notion that energy dissipation during waving breaking will be in similitude. To test this theory he built an undistorted model of a 1:40 plane sloping beach that obeyed all Froude scaling criterions, the beach was subjected to waves of different heights. Stieve concluded that parameters generated from waves ranging from 0.1 m to 1.5 m did not deviate from the Froude scaling criterion. In retrospect, it can be said with confidence that waves entrained with air have no significant dynamic influence between model and prototype.

2.3.3 Wave Refraction, Diffraction and Shoaling

Model waves created during wave generation must often traverse a mobile bed in which sediment scour is initiated by bottom friction resulting from wave energy interacting with the sediment bed. Models that closely obey bathymetric deviations in the prototype lakebed are subjected to possible occurrences of wave refraction, diffraction and shoaling. Simply stated, wave refraction refers to the bending of water waves resulting from varying underlying water depths underneath. As a wave front enters shallow water its velocity or wave celerity decreases because the bottom of the wave orbital interfaces with bottom sediment which slows the wave. However, as the wave slows its height increases due to the shallow water and the amount of energy it possesses also increases in proportion to the wave height. Similarly, as a wave traverses over deep water its velocity will increase because there is no bottom friction opposing its motion. Figure 2.4 below illustrates how a wave front bends as it passes over shallow water.

Figure 2. 4 The celerity of wave front A slows as it approaches the shore (or headland) because of shallow water and the celerity of wave front B increases as it enters the bay resulting in the bending of the entire wave front

Additionally, headlands (small peninsulas) are usually plagued by intense erosion due to the formation of large waves, while an embayment is the site of significant sediment deposition. Another coastal engineering process worth mentioning is wave diffraction. Wave diffraction is the process by which energy spreads laterally, perpendicular to the dominant direction of wave propagation (Dean & Dalrymple, 1991).

Figure 2.5 below shows what happens to an approaching wave front as it hits a surface piercing object like breakwater. Diffraction occurs in the sheltered portion or the “Shadow zone” as the wave disturbance impinges on the structure.

Figure 2.5 The coastal structure causes approaching waves to diffract laterally as they enter the sheltered side of the breakwater

Wave shoaling occurs in much the same conditions as wave refraction. Wave shoaling develops over shallow water as the wave propagates towards the shore its celerity and wave length decrease but the wave energy per unit area increases along with the wave height. When conducting wave shoaling analysis, the wave period remains the same. Typically, when the wave crest becomes too steep it curls forward and breaks, this results when the wave height is about equal to the water height (Centre for Applied Coastal Research, University of Delaware, 2003).

2.3.4 Generation of Laboratory Waves

When building a physical model of a coastal region it is necessary to produce water waves to simulate the prototype condition. Often in nature the wind plays a pivotal role in generating waves by interfacing with a liquid medium. While it is possible to create water waves via wind action in a hydraulics laboratory, the tank where the waves are produced must be enclosed and the wind/wave flume must be of sufficient length in order to generate the appropriate size wave (Hughes, 1993). The most efficient and cost effective method to generate water waves in a laboratory is therefore by mechanical means. This can be accomplished by employing a simple blunt face that straddles the

length of the wave basin and subsequently displaces water by translating back and forth to generate oscillations when connected to a motor or hydraulic piston. A variety of wave makers have been developed over the last 50 years, including piston, wedge & flap and three-dimensional etc.

According to Hughes (1997), one of the first types of wave makers developed was known as a “piston-type” wave maker because it generated uniform waves by moving the wave board in a sinusoidal motion with a given amplitude and period of oscillation. Many researchers used this type of wave generator to conduct vital coastal engineering research as it simulated real-world waves accurately and conformed well to linear wave theory (Hughes, 1993). Another type of wave generator is the “flap-type” wave maker which is hinged at the bottom and is allowed to oscillate or “flap” to create sinusoidal waves along the length of the wave tank. The flap-type machine is effective at producing deep water waves because the orbital excursions are minimal at the hinge (Hyun, 1976).

A third kind of wave generator is the “wedge-shaped” plunger wave generator. This type of wave maker can be used in place of a piston driven or flap-type wave maker to generate waves in one direction. A plunger wave maker generates waves by oscillating vertically to displace a mass of water thereby creating waves. Dean and Dalrymple (1991) refer to a three-dimensional wave maker that looks and acts like a snake. A multi-directional wave maker or a “snake” wave maker employs sophisticated computer controls to oscillate or mimic the movement of a snake in order to produce waves that can propagate in any direction of a wave basin. Figures 2.6-2.8 show the surface profiles created by different wave makers. An example of a multi-directional wave maker is shown in Figure 2.9.

All wave makers discussed can generate simple periodic or random waves and often their performance is function of the task at hand. The piston type wave makers that use a wide KELK board to displace water have traditionally performed well at generating periodic waves which correspond to linear wave theory. The new wave makers have the added advantage of sophisticated computer controls that can accept computer wave algorithms to reproduce ocean type currents, and waves that can propagate to any location in the wave basin. The paddles utilized on these wave makers can monitor wave reflection and make adjustments accordingly to generated waves.

Figure 2.6 Piston wave generator.

Figure 2.7 Flap-type wave generator.

Figure 2.8 Plunger-type wave generator.

Figure 2.9 Mutli-direction wave generator.

Although each of the wave generators shown above could be used to generate waves in a laboratory setting, the choice of a wave generator is often a decision not made by the researcher. Hydraulic facilities in the province of Ontario, Canada are sparsely located throughout the region and the commute time to use any one of these facilities must be analyzed in terms of economics and applicability of the wave generator to the study. For this particular study, the prestigious hydraulic laboratory at the National Water Research Institute was selected to study the physical model. The NWRI has a piston driven wave generator that is capable of reproducing both periodic and random waves; these wave functions are adequate for this study.

2.4 Laboratory Effects in Physical Models

Literature from Hughes (1993) and Kamphuis (1991) indicates that laboratory effects in short-wave physical models are related to three conditions:

- The physical constraints of boundaries on the flow.
- Unintentional nonlinear effects brought about by using mechanical means of wave and current generation.
- Simplification of prototype forcing conditions (e.g., prototype wave condition as unidirectional).

According to Kamphuis (1991) the error from physical models listed above are analogous to problems in numerical modeling caused by:

- Numerical approximation to equations.
- Roundoff and truncation errors.
- Availability of computer speed and memory.

A laboratory effect which must be closely watched and analyzed is the interaction that a wave has with the boundary of a wave flume. An experienced researcher will know how to interpret such a phenomenon in order to obtain the most pertinent data from the model and exclude any data that is not relevant to the simulation. Hughes (1993) also states that researchers must also be aware of wave reflections from stationary structures. To clarify, waves that interact with a coastal structure in nature have an opportunity to reflect harmlessly back out to the sea; in a wave basin or wave flume, however, waves that hit a structure are reflected back into the study area and may potentially hit the wave board. To minimize this undesirable event the following suggestions have been compiled by Hughes (1993):

- Energy dissipating beaches comprised of gentle slopes
- Conduct experiments in a series of wave bursts to minimize re-reflected waves interfering with model simulations.

- Usage of active wave absorption at the wave board to detect and absorb unwanted wave energy.

Dalrymple (1977) stated that basin models can produce inaccurate results primarily because of their finite width and side effects resulting from model boundaries which ultimately influence the hydrodynamics of the simulation. Furthermore, obliquely approaching waves can create long shore currents that result in rip currents along the basin side wall (Dalrymple, 1977). This type of problem can cause large scale circulation problems in a wave basin forcing the researcher to utilize areas of the basin not affected by boundary effects. The problem of circulation can be alleviated by using training walls to guide the model waves and utilizing wave absorbers to absorb along-shore waves and channel them to outside the study area while at the same time introducing an equal volume of water into the study area in order to conserve water mass (Dalrymple, 1977).

2.5 Scale Effects in Physical Models

It is stated by Hughes (1993) that most scale effects in coastal hydraulic models result from the assumption that gravity is the dominant physical force balancing inertial forces. Unfortunately, when researchers inappropriately choose the Froude scaling criterion other physical forces like viscosity, elasticity, surface tension, etc. will be incorrectly scaled. Kamphuis (1991) explains that scale effects in physical models are analogous to decreased accuracy in numerical models because complex coastal processes are being represented by simplified mathematical equations. Research done by Costa (1981, 1984, 1990) on the secondary forces present in model simulations, revealed that viscous, elastic and surface tension forces are over represented in model simulations when compared to prototype conditions. Over the years engineers have learned to live with scale effects since it is impossible to have complete similitude. Often, many scale effects can be quantified by researchers such that critical analysis can be conducted to decide upon the validity of the simulation data (Le Mehaute, 1990).

2.6 Mobile Bed Sediment Models

Movable-bed physical models are far more complex than fixed-bed models because they deal with complex hydrodynamic processes like wave motion, current motion, sediment transport and boundary effects to name just a few. The mathematical formulations required to describe these processes are very complex so it is necessary to place a reasonable degree of caution on the validity of the output data. A movable-bed model is one in which the model bed is composed of granular non-cohesive material which can be transported within the model boundary by hydrodynamic forces caused by waves and currents (Hughes, 1993). Kamphuis (1985), made a statement below that reiterates the benefits of using a physical model to solve and understand a coastal engineering problem.

“Field work is often prohibitively expensive or impossible to perform because parameters are not readily manipulated. Numerical modeling is dependent on physical interactions which are poorly understood and virtually impossible to formulate successfully in mathematical terms.”

Dean (1985) listed two criteria that should be met when simulating sand transport in a physical model.

- Knowledge of the character of the dominant forces, and
- An understanding of the dominant response mechanism of the sediment.

Three-dimensional movable-bed models can often be used to study various combinations of long shore transport, onshore/offshore transport, scouring near structures and on surfaces (SSO). Three-dimensional physical models that have been used to study various processes, such as:

- Littoral drift induced by oblique wave approach.
- Effect of coastal structures on adjacent shores.
- Scour holes in the vicinity of structures (Hughes, 1993).

Sediment in a movable-bed model is typically transported by a combination of wind waves and currents (Hughes, 1993). Sediment is free to travel via bedload transport whereby sediment grains are lifted into the water column and moved by fluid as suspended a load (Hughes, 1993). Oddly enough, all of the literature that has been reviewed thus far cautioned the researcher to be very careful when designing and building a hydraulic model. Further, caution is warranted when interpreting the output data produced by the model. Comments on the use of hydraulic models acknowledge both the value & limitations of their use:

Movable-bed models of the coastal zone should be used with discretion. While they can provide valuable insight into complicated interaction of coastal processes, they can also lead to catastrophic consequences due to over confident interpretation (Noda, 1972).

With all the limitations they present, coastal movable bed models constitute valuable instruments to predict the effects of maritime structures on an adjacent bed (Motta, 1986).

A physical model is a very powerful instrument that must be used with caution, the credibility of such a model is often a function of the quality of prototype data. Physical models validated via prototype conditions have a greater accuracy and can be used with confidence to simulate real-world conditions. In the past, two methods were used to derive scale ratios: dimensional analysis and differential equations. Hughes (1993) lists two suggestions below to expedite the process:

- Assume that the sediment process is reacting primarily to a unidirectional flow situation with waves added.
- Assume that sediment is reacting primarily to waves with currents added.

2.6.1 Scaling of Model Fluid and Sediment

When designing and constructing a coastal movable-bed model there are several general similitude difficulties that have to do with scaling requirements (Hughes, 1993). For example, it is often difficult to obtain model fluid that has been scaled properly. The reason for this is because a high cost is often associated with obtaining a large volume of fluid that is scaled down to the appropriate model-fluid density; for this reason, water is the only choice for the researcher. Furthermore, it should also be stated that if certain liquids are reduced in density their characteristics will most likely change such that a new liquid may result. Another difficulty in achieving similitude is to obtain the correct sediment size. It is quite possible that cohesive sediment may result if the grain size is scaled according to the geometric length (grain diameter $< 0.08\text{mm}$) (Hughes, 1993). Cohesive sediment would invoke different sediment transport processes not representative of prototype conditions.

2.6.2 Data Collection for Physical Model Study

In order to design and build a physical model that emulates prototype conditions it is necessary to obtain accurate and relevant data from the field. Hudson (1979) suggested researchers to obtain four pieces of essential data:

1. Detailed sediment-size distribution over the entire area of interest
2. Computational estimates of yearly and monthly net long-shore transport based on wave statistics, refraction, etc.
3. All available survey data
4. Accurate wave and sediment transport measurements

Hughes (1993) suggests measurements and cautions that engineers and scientists should use upon completion of a model run:

- Document bed evolution with periodic measurement of model bathymetry
- Drain water slowly to preserve bed features

- Place white string on contours and take photographs from overhead cameras
- During simulation measure: waves, currents, longshore transport rates, suspended sediment concentrations
- Videotape model simulations

2.6.3 Scale Selection of Model

According to Hughes (1979) when selecting the scale of the movable-bed model the following should be considered.

- Determine the principal mode of sediment transport in the region to be studied
- Non-cohesive sand lower limit
- Size of available wave tank and basins
- Prototype wave and current measurements
- Estimates of long shore transport rates in prototype
- Size of prototype area to be studied
- Type and size distribution of prototype bottom material
- Size and capabilities of model test facility
- Available material to be used for model sediment
- Funds and time available for study
- Qualifies personnel to conduct the study

2.7 Computational Fluid Dynamic Simulations

Computational fluid dynamics (CFD) uses sophisticated mathematical equations and computer software processing technology to compute the dynamics of fluid flows (FLUENT, 2006). CFD software empowers the user to create sophisticated geometry and apply fluid flow physics and chemistry to the prototype to predict how the fluid dynamics applies to a physical process. In the study of a self-scouring outfall FLUENT software will be utilized to simulate the hydrodynamics within the structure. CFD software is excellent for analyzing the flow patterns within the outfall because no technical literature is currently available on the system performance of a self-scouring outfall. Harwood and Saul (1999) list several advantages below in using CFD software.

- There is no need for laboratory testing facilities

- Model geometry and input parameters can be quickly changed in the program whereas in a physical model time and money must be allotted to modify the model
- Flow parameters such as pressure and velocity are calculated at all points in the CFD software which might not be possible in the physical model

This chapter reviewed fundamental principles that must be taken into account when designing and constructing a physical model. They are:

- Judging the credibility of model results.
- Model wave considerations.
- Laboratory effects in a physical model
- Scale effects in a physical model.
- Necessary field data for constructing a physical model.
- The usage of computational fluid dynamic software.

These principles have been adopted in this study, to ensure results are reliable/valid and accurate.

Chapter 3 Design of the Physical Model

The following chapter will outline the procedures, processes and construction methods that were utilized to design the physical model of a self-scouring outfall at the National Water Research Institute. In order to design an accurate model it was necessary to do extensive fieldwork during the first phase of the project so that prototype conditions were meticulously reproduced in the lab to increase the credibility of model simulations. The second part of the design phase dealt with the actual construction of the physical model, calibration, and model test runs. A brief summary of the design procedure is outlined in Figure 3.1.

3.1 Observations from Initial Site Visit

The outfall under investigation was located in Stoney Creek, Hamilton, Ontario at the foot of Green Road on the shores of Lake Ontario. Visual observations were made of the embayment area during the initial site visit to ascertain the impact of wave driven sedimentation. At the site, metal sheet piles were driven into the lakebed in a rectangular pattern to form a barrier in front of the gravel beach. It was assumed the purpose of these sheet piles was to dissipate wave driven sediment from entering the embayment area during turbulent water conditions and to prevent outfall plugging. Upon closer observation, the metal sheet piles were sawed to the low water level as a possible fix to mitigate the sediment build-up in the embayment area. During the preliminary stages of this project it was decided to model the metal sheet piles during simulations because the cost associated with their removal in the field was too high.

The primary objective of the site visit was to gather as much data as possible so that prototype conditions could be reproduced in the lab. Presented on the following pages are photos that illustrate the condition of the outfall. Figures 3.2 and 3.3 were taken on October 13, 2006 and illustrate the different components that make up the Green Road outfall.

Figure 3. 1 Design procedure for SSO model study.

Figure 3. 2 Existing condition of the Green Road Outfall, a coarse gravel beach and large armour stones compose the embayment.

Figure 3. 3 The partially submerged sheet piles shown in front of the armour stones were used to break incoming waves.

Photos taken during the field visit provided the researchers with a snap shot of the events caused by a fast moving weather system. Upon closer observation of Figure 3.4, a visible plume of murky water was sighted at the foot of the embayment. A probable explanation was that turbulent wave activity caused sediment to become suspended in the water column as significant shear disturbed the bottom sediment on the lake bed. In Figure 3.5, protective horizontal outfall bars are visible in the clean embayment; however, as the remnants of Hurricane Katrina swept across Southern Ontario a large quantity of sediment accumulated in the embayment in a matter of days incapacitating the outfall (Figure 3.6). The strong winds from the storm generated large waves able to transport large quantities of sediment into the embayment, which reduced its hydraulic discharge capacity by up to 50% during this single storm event.

Figure 3. 4 Strong waves have caused siltation in front of the embayment area.

Figure 3. 5 Outfall is sediment free.

Figure 3. 6 Outfall has been plugged by sediment.

When comparing the photos below, it was observed the lake water in Figure 3.7 was calm and tranquil during low water level conditions but high winds quickly generated intense waves (Figure 3.8). The site visit enabled engineers to visualize the types of powerful storm events the new self-scouring outfall was going to be subjected to without the aide of physical or numerical models.

Figure 3. 7 The lake water in front of the embayment is calm and clear when high waves are nonexistent.

Figure 3. 8 High waves scour the bottom sediment which results in sediment transport into the embayment.

The photos above serve to underline the urgent need for rectification of the outfall blockage problem. If the stormwater is unable to exit the outfall, it would likely pond at the source(s). An accumulation of stormwater on the roadway would subsequently

present a hydroplaning hazard for automobiles and flood damage risks to residential property owners.

3.2 Determination of the Model Scale

In literature reviewed it was stated by Dean (1985) that movable-bed models should be undistorted, adhere to the Froude scaling law and have a scale factor of less than 30. Environmental Hydraulics Group (EHG), the engineering consulting firm that helped to conduct this study built a 1:30 scale model of a self-scouring outfall & concluded a 1:30 scale model would be too small to effectively study the hydrodynamics of the outfall. Instead, the optimal scale size of the model was determined based on available lab-space, funds, sediment characteristics, reviewed literature and field observations. It was decided by the design team to build 1:15 scale undistorted physical model that satisfied the conditions of geometric, kinematics and dynamic similarity as presented in the literature review. The scale ratio of this particular model is shown below.

$$N_x = \frac{X_p}{X_m} \quad [3.1]$$

$$N_L = \frac{L_p}{L_m} = \frac{15m}{1m} = 15$$

A 1:15 scale physical model was an ideal size for a coastal model that consisted of a structure and a movable-bed because it was large enough to ensure that any viscous influence would be offset by the high Reynolds number. The survey team decided that an area approximately 125 m wide by 225 long in the field was an adequate size for the reproduction of sediment transport in the lab when the scale factor was applied to these lengths. As an example of the selected model scale, a length of 60 m in the prototype (field) would be 4 m in the model if the scale ratio was 1:15. By simple division using the linear scale and the need to maintain geometric similarity between the model and the prototype, the model area dimensions were found to be roughly 8 m wide by 15 m long.

3.2.1 Froudian Modelling

The majority of coastal engineering movable-bed models were designed using the Froude scaling criterion because:

- 1) Gravity forces predominated in most free surface flows.
- 2) Inertial forces were always present in the flow problems.

Geometric, kinematic and dynamic similarity were to be maintained between the model and the prototype so that the following proportionalities were valid:

$$\lambda_L = \frac{L''}{L'}; \lambda_T = \frac{T''}{T'}; \lambda_M = \frac{M''}{M'} \quad [3.2]$$

However, due to technical and monetary constraints, it is sometimes not feasible to maintain complete similitude between the model and prototype. Often, selection of the basic model scales (λ_{a1} , λ_{a2} , λ_{a3}) are subject to great scrutiny and severe restrictions (Yalin, 1971). The acceptance of the two dynamic similitude criteria associated with the fluid and particle transport would require the model fluid to be different than the full-scale prototype fluid and for the model to have a different sediment particle density than the prototype. The requirements of a special fluid and sand particles were difficult to fulfill because of the high cost and the vast quantity that was required. These limitations resulted in the selection of water as the model fluid, such that the scales for viscosity and density were equal to unity

$$\lambda_\mu = 1 \text{ and } \lambda_\rho = 1$$

Another parameter used was acceleration due gravity as,

$$\lambda_g = 1$$

It was not possible to achieve complete similitude between the model and prototype because certain quantities cannot be scaled down. For example, gravity remained the same between the model and prototype and water was the only practical fluid that could be used, the inability of a model to simulate these parameters correctly was the reason why scale effects become an integral part of all hydraulic models. Since gravity, viscosity and density are the same between the model and prototype certain tradeoffs were made because models cannot simulate gravity-driven and fluid viscosity-related phenomenon simultaneously.

In coastal engineering hydraulic models, waves and currents are gravity driven and are fundamental to the processes being studied. Therefore it was desirable to simulate gravity correctly and accept that viscosity would not be properly reproduced. In hydraulic model tests concerning sea waves it was found by Goda (2000) that viscosity and surface tension were relatively small and could be safely neglected. The Froude scaling law is reproduced below.

$$Fr = \frac{V}{\sqrt{gL}} \quad [3.3]$$

Where V = velocity, g = acceleration due to gravity and L = length. The Froude modelling criterion states that inertial forces (numerator) are balanced by the gravitational forces (denominator). A property of the Froude law dictates that the time and velocity scale should be equal to the square root of the length scale. For similitude, the ratio of (Froude Number) will be maintained at unity (EHG Report, 2005). The time scale formula is shown below.

$$\frac{T_m}{T_p} = \sqrt{L_r} \quad [3.4]$$

Where T_p = prototype time, T_m = model time and $L_r^{1/2}$ is the model scale. The calculation for the wave period which is related to Froude time scale must be calculated using equation 3.4. All other quantities such as the wave height, water level and horizontal and vertical lengths can be calculated using the linear scale.

3.3 Wave Analysis for Model Design

Model wave design involved the analysis of data which was obtained from the Western Lake Ontario Buoy C45139 (Figure 3.9) located at 43.26° N and 79.54° W. The buoy provides the user with the wave height and wave period data as well as other statistical information about the wave frequency and wave direction. During this study it was assumed that significant incident wave heights range from 1 to 2 m (prototype), with wave periods from 4 to 6 seconds (prototype). The complete table of all the statistical information pertaining to wave height and frequency was placed in Appendix A1. Presented below are a series of graphs (Figures 3.10 & 3.11) that help to demonstrate how often a wave from a particular direction encroached on the site.

Figure 3.9 An example of a lake buoy.

Figure 3. 10 The figure illustrates the frequency and direction of various waves from 30° Arcs.

Figure 3. 11 The figure illustrates the frequency and direction of various waves from 60° Arcs.

Shown in Figures 3.10 and 3.11 are the frequencies of waves approaching the north and east passing buoy C45139 for 30 and 60 degree arcs. For this study, it was more practical to use 60 degree arc data because this approach covers a wider spectrum

and was more practical for this study. Upon closer observation of Figure 3.11 and careful examination of the orientation of the future SSO relative to the coastline and embayment, it was decided to reproduce model waves from the northeast and eastern directions. This was because waves that approach from the northeast direction can push sediment directly into the embayment area as shown in Figure 3.12. Furthermore, approaching waves from the east can push sediment directly into the eastern outlet channel of self-scouring outfall-II. It is believed that some excellent study results can be produced from waves simulated from the chosen directions because they represent two worse case scenarios of sediment-laden waves attacking the structure from predominate directions.

Figure 3. 12 The various wave approaches are shown relative to the embayment area shown in the red circle. Waves from the northeast and east can push sediment directly into the embayment [Source: EHG Green Road Outfall Report, 2005].

3.3.1 Refraction and Shoaling

It was discussed in the previous chapter as waves travel from deep water to the near surf zone they experience wave refraction and shoaling. Water waves that have entered a region of shallow water with strong bottom contours will often have their wave crestlines bent in pattern that resembles the depth contours of the sea bottom. Research by Goda (2000) has shown that wavelengths of waves become shorter and long-crested (nearly 2-D), relative to the local wavelength as is shown Figure 3.13.

Figure 3.13 Refracted wave pattern of approaching waves near the shoreline
[Source: Goda, 2000].

The refracted wave pattern displayed in Figure 3.13 indicated the angle of the wave crest-line has been altered at the 2000 m marking on the x-axis by bottom contours. In the design of a coastal movable-bed model it was necessary to verify whether or not refraction and shoaling (change in wave height due to varying depth) was cause for concern. If the relative water depth (water depth/wave length, h/L) was calculated to be less than 0.5 then approaching waves were affected by bed contours. Table 3.1 has been produced to show that bed contours on Lake Ontario did not seriously impact the model design by causing waves to change direction.

Table 3.1 Wave Characteristics in Prototype

Wave Characteristics	Prototype – Condition 1	Prototype – Condition 2
Water Depth, h (m)	4.0	4.0
Wave Period, T (s)	6.0	4.0
Breaks at depth (m) / Wave Height (m)	1.75 1.25	1.6 1.0
Wave Length, L (m)	34.75	20.85
Relative Water Depth, h/L	0.11 - OK	0.3 - OK

Outlined in Table 3.1 are two wave refraction scenarios that could potentially occur in the field. As numerical calculations concluded wave refraction was not a significant process in the field, therefore process was insignificant in model. To clarify, the water depth was kept constant for both conditions 1 and 2 in Table 3.1 and the period was varied. Goda (2000) has produced a series of wavelength and celerity tables to quickly assist researchers in finding a particular wavelength. The breaking water depth and height were also shown to indicate these values tend to decrease with a smaller wave period. Formulae for the breaking water depth can be found in Goda (2000) (pg. 428). The last row of Table 3.1 has indicated that bottom contours did not significantly affect the direction and propagation of the wave front in any abnormal way and wave refraction was not cause for concern.

A refraction analysis was conducted for waves of significant height ($H_{1/3} = 1$ m) propagating from deep water to the study area. By definition, significant wave height is the average height from trough to crest of one third highest waves valid for an indicated time period. The approaching deep water waves whose crests (the curved tops or ridges of an oscillatory wave) make a 32° angle with a 30 m bottom contour were analyzed for this study and the results were summarized in Table 3.2.

Table 3. 2 Results of wave refraction analysis

Wave Period, (s)	Direction of Wave	Wave Height, m	Comment
4 sec – Prototype	27 degrees	0.9 m	A wave that is approaching from an angle of 32° and is refracted to an angle of 27° is not significant because this deviation only represents a 5° change in direction.
6 sec - Prototype	20 degrees	0.9 m	A wave that is approaching from an angle of 32° and is refracted to an angle of 20° is significant, but because a wave period of 6 seconds is not dominant, this scenario is not cause for concern.

According to buoy data, waves with a 6 second period were far less frequent than 4 second waves. Because the shorter wave period was more dominant, it was believed that this type of wave was most likely to transport sediment into the study area. For this particular case, wave refraction was negligible but for cases where wave refraction occurs

in shallow water with bottom contours the result of the wave direction and height was difficult to predict. Furthermore, it was assumed in this study that short-period waves that propagate into the study area arrived unchanged. Presented below is a graph illustrating how the approaching wave crest bends in order to accommodate the change in water depth. A wave crest approach angle of 32 degrees was obtained from local bathymetric charts of the study region and was used to simulate wave refraction in the study area (Figure 3.14). It was believed the refraction analysis shown in Figure 3.14 could be used as a method correlate how well wave patterns in the model agreed with theoretical calculations.

Figure 3. 14 The refraction of wave approaching the coast.

3.3.2 Design of Model Wave Period

The design of the model wave period was calculated using the Froude time scale because it was possible to use available prototype data from Lake Buoy C45139. The numerical calculations for a 4 and 6 second model wave period were presented in Table 3.3.

Table 3.3 Summary table of model period calculations

Prototype Period, T_p (sec)	Model Period, T_m (sec)
4.0 sec	$T_m = \frac{T_p}{N_L^{1/2}} = \frac{4}{15^{1/2}} = 1.033 \text{ sec}$
6.0 sec	$T_m = \frac{T_p}{N_L^{1/2}} = \frac{6}{15^{1/2}} = 1.549 \text{ sec}$

3.3.3 Design of Wave Height

By using data provided by buoy C45139 it was possible to ascertain the spectrum of incoming waves heights that were propagating into the study during a 15 year period. For this study it was assumed that the significant wave heights that bring sediment into the study area range from 1 m to 2 m in the prototype. Shown in Table 3.4 below are the calculations for two model wave heights that were derived from buoy prototype data.

Table 3.4 Summary table of model wave height calculations

Prototype Wave Height, H_p (m)	Model Wave Height, H_m (sec)
1.43 m	$H_m = \frac{H_p}{N_L} = \frac{1.43m}{15} = 0.095m$
1.95 m	$H_m = \frac{H_p}{N_L} = \frac{1.95m}{15} = 0.13m$

The model wave heights in Table 3.4 were calculated by dividing the prototype wave height by the model scale, the linear geometric scale was used to calculate this value because wave height is not governed by the Froude condition. Designing the physical model required analysis prior to construction, hence, all aspects governing the hydrodynamics of the model are examined throughout this chapter. Presented in Figures 3.15 and 3.16 are a series of graphs that indicate the water depth at which water waves will break for a period of 4 and 6 seconds.

Figure 3.15 Wave breaking for a period of four seconds.

Figure 3.16 Wave breaking for a period of six seconds.

It was observed in Figures 3.15 and 3.16 that as the wave crest approached the coastline the height of the wave slowly decreased and then suddenly increased as it broke on the beach. The breaking depth of a wave front was crucial in the model design for it provided insight into the amount of area and water depth needed to construct the model. Table 3.5 below, was created using Goda's (1975) wave breaking equation to determine the water depths at which waves of various size would break. The graphical results obtained from Figures 3.15 and 3.16 and the numerical calculations shown in Table 3.5 seem to agree well with each other.

Table 3.5 Numerical results for wave breaking analysis

Size of Wave	Period, (T) = 4 seconds		Period, (T) = 6 seconds	
	H _b Prototype, m	Model, m $H_{b(mod el)} = \frac{H_{b(proto)}}{N_L}$	H _b Prototype, m	Model, m $H_{b(mod el)} = \frac{H_{b(proto)}}{N_L}$
Breaking water depth for a 1m wave	1.50	0.10	1.72	0.11
Breaking water depth for a 1.5m wave	2.14	0.14	2.40	0.16
Breaking water depth for a 2m wave	2.81	0.19	3.05	0.20

Table 3.5 provided researchers with an idea how much water depth was needed for a specific size wave to break on the shore which aided the designers in determining the length and slope of the mobile bed.

3.4 Sediment Sampling in the Study Area

A sediment sampling initiative was undertaken by Environmental Hydraulics Group to determine the various sizes of sediment found at different locations within the embayment area and in the vicinity. Sediment sampling was necessary because it provided researchers with information concerning the origins of sediment found in the embayment. It was believed the majority of sediment transported into the embayment originated from an eroding bluff east of the study outfall. Sediment from this bluff was most likely transported by a littoral current travelling from east to west. Shown below in Figure 3.17 are the locations that were used to extract sediment samples from the beach or lakebed in the field.

Figure 3. 17 The letters on the figure indicate the approximate location of the sample extraction point, these letters also correspond to the sieve analysis gradation charts that are to follow.

In order to duplicate the prototype sediment it was necessary to obtain a total of 15 sediment samples from various locations such as the lake bed, embayment and beaches. A greater number of samples were taken from the embayment and beach locations, so as to gain a greater insight into sediment transport in the vicinity of the outfall. A small device called a Ponar which utilizes two buckets in a dredging action to dig into the lake bed was used to acquire the samples from the lake bed. The Ponar is shown in Figure 3.18.

Figure 3. 18 A Ponar, similar to the one above was used to gather samples from the lakebed (courtesy of Ricky Hydrological Company).

Sediment gradation charts were created in Figures 3.19 to 3.23 to determine the size and type of the respective sediment that occupied a particular location in the embayment area, these figures also corresponded to sample location points shown in Figure 3.17. The D_{50} was calculated for each sediment sample to determine the sediment grain size that fifty-percent of the sand passed through. The soil parameters for each sediment sample and photographic images are presented in Table 3.6. Additionally, the uniformity coefficient (C_u) was calculated to give researchers an indication of the curve shape. This C_u value also indicated to researchers that 60% of the sample was finer than the D_{60} . A large C_u meant that the size distribution of particles was wide. For example, a C_u value of 1 indicated to researchers the grain size was uniform (dune sand), whereas for glacial till the C_u could reach 30.

The coefficient of curvature (C_c) was used to judge the gradation of the soil. A soil was deemed to be well graded if the C_c value was between 1 and 3, whereas a C_c value of 4 was indicative of gravel and a C_c value of 6 equated to sand.

Coefficient of Uniformity, C_u – Provides an indication of the shape of the gradation curve and the finer content

Coefficient of Curvature, C_c – Provides an indication of the soil gradation

Grain-Size Analysis A "From Beach - East Side"

Figure 3. 19 Grain-size analysis from east beach.

Grain-Size Analysis B "From Beach - East Side, Water Depth = 30 cm, 2m From Shore"

Figure 3. 20 Grain-size analysis for sediment in water adjacent to east beach.

Grain-Size Analysis C "2 m Inside of Berm, Sample Depth = 25 cm"

Figure 3.21 Grain-size analysis from inside berm location.

Grain-Size Analysis D "From Embayment Berm Middle"

Figure 3. 22 Grain-size analysis from embayment location.

Grain-Size Analysis E

Figure 3. 23 Grain-size analysis for sediment that was acquired from the lake bed northeast of the outfall.

Table 3.6 Characteristics of sediment samples gathered around the Green Rd. outfall embayment.

ID	Sample Location	Description of Sediment	D ₅₀ , mm	C _u	C _c	Image of Sediment
A	East side of beach close to embayment	Sand, medium and coarse	2.4	3.90	0.86
B	From shallow lake bed near east beach	Gravel and coarse sand	6.1	2.80	0.87
C	Inside berm	Sand, medium and coarse	2.3	4.52	1.09
D	Middle of berm near to embayment	Sand, medium and coarse	1.8	3.97	0.91
E	East side — Deep water	Sand with fines	0.09	-	-

Table 3. 6 Sediment characteristics

3.4.3 Design of Model Sediment

The construction and testing of a coastal movable-bed model required a great deal of sediment for adequate moulding of the bed contours. It was often difficult to acquire movable-bed material that had a particular size and density and was geometrically similar to the prototype sediment because there was a high cost associated with processing this material. Many believe that using sand as model sediment is preferable for coastal sediment transport models (Dalrymple 1989). When observing the gradation charts shown in Figures 3.19 – 3.23, it was evident that many different particle sizes compose the Green Rd. Outfall embayment and surrounding areas.

The predominantly fine sediment found in the embayment was modelled in the lab because it was believed this particular diameter of sediment was transported from east to the west by a littoral current and was ultimately being pushed into the embayment by wave energy. It was decided that non-cohesive construction sand would be used in the model to simulate the transport of prototype sediment because it was readily available, economical, and had been successfully used by other researchers in the past. The usage of finely ground nutshell from the aircraft industry was a potential sediment choice for researchers but this particular product was deemed far too costly to be used in this study and researchers opted to use readily available construction sand.

By using the given model parameters and by making reasonable assumptions, an attempt has been made to calculate the model particle size that could be used in the model. The Shield's diagram was used to determine if sediment transport was occurring within the model.

*Assumptions:

1. Prototype sediment and model sediment are geometrically similar such that they both have similar grain size distribution curves.

Given: (Prototype conditions)

Flow depth, $h' = 5.25$ m (low water level)

Bed slope, $S' = 0.01$

Median sediment diameter, $D'_{50} = 2.3$ mm

Specific weight of material / Specific weight of water, $\gamma'_s/\gamma = 2.69$

Acceleration due to gravity, $g = 9.81$ m/s²

Kinematic viscosity, $\nu = 10^{-6} \text{ m}^2/\text{s}$ ($\approx 20^\circ\text{C}$)

Solution:

To begin, the values on the x and y-axis of the Shield's diagram (Figure 3.77) were calculated to determine if sediment transport was occurring.

Calculating the shear velocity yielded,

$$v_* = \sqrt{gS'h'} = \sqrt{9.81 * 0.01 * 5.25} = 0.718 \text{ m/s} \quad [3.5]$$

The value on the x-axis shown in the Shield's Diagram was calculated below:

$$X'_1 = R_* = \frac{v_*' D'_{50}}{\nu} = \frac{0.718 \text{ m/s} * 0.023 \text{ m}}{10^{-6}} = 16,506 \quad [3.6]$$

The value on the y-axis shown in the Shield's Diagram was calculated below:

$$X'_2 = F_* = \frac{\rho v_*'^2}{\gamma'_s D'_{50}} = \frac{\gamma}{\gamma'_s} \cdot \frac{S'h'}{D'_{50}} = \frac{1}{2.69} \cdot \frac{0.01 * 5.25 \text{ m}}{0.023} = 0.8486 \quad [3.7]$$

The sediment transport point was plotted on the Shield's diagram by using the values generated by equations 3.6 and 3.7. It can be seen on Figure 3.77 that $F_* > 0.05$ which indicated to researchers that the flow was rough and turbulent, satisfying the Froudian modelling criterion.

Figure 3.77 The Shield's diagram indicated that sediment transport was occurring. The model scale λ_l satisfied the condition below

$$\lambda_l \succ \left(\frac{70}{X'_1} \right)^{\frac{2}{3}} = \left(\frac{70}{16506} \right)^{\frac{2}{3}} = \frac{1}{38.17} = 0.0262 \quad [3.8]$$

The model scale was selected as follows

$$\lambda_l = \frac{1}{38} = 0.0263 \quad [3.9]$$

The relation below was presented by Yalin, (1971)

$$\left\{ \begin{array}{l} \lambda_v = \lambda_{v*} = \sqrt{(\lambda_l)} \\ \lambda_l = \lambda_D = \lambda_k, \\ \lambda_s = \lambda_p = \lambda_{p_s} = 1 \end{array} \right\} \quad [3.10]$$

By using the value produced in equation [3.9] the following was obtained

$$\lambda_v = \lambda_{v*} = \frac{1}{\sqrt{38}} = \frac{1}{6.164} \quad [3.11]$$

and

$$\lambda_D = \frac{1}{38} \quad [3.12]$$

Therefore, the model characteristics are

Table 3. 7 Critical Model Parameters

Prototype Dimensions	Model Dimensions
Bed slope, $S' = 0.01 = 1\%$	Bed slope, $S^* = 0.01 = 1\%$
Acceleration due to gravity, $g' = 9.81 \text{ m/s}^2$	Acceleration due to gravity, $g^* = 9.81 \text{ m/s}^2$
Kinematic viscosity, $\nu' = 10^{-6} \text{ m}^2/\text{s} (\approx 20^\circ\text{C})$	Kinematic viscosity, $\nu^* = 10^{-6} \text{ m}^2/\text{s} (\approx 20^\circ\text{C})$
Median sediment diameter, $D'_{50} = 2.3 \text{ mm}$	$D_{50} = 0.23/38 = 0.0061 \text{ cm} = 0.06 \text{ mm}$
Specific weight of material / Specific weight of water, $\gamma'_s/\gamma = 2.69$	Specific weight of material, $\gamma_s^* = 2.69 \text{ ton/m}^3$
Flow depth, $h' = 5.25 \text{ m}$ (low water level)	Flow depth, $h'' = 525/38 = 13.8 \text{ cm}$ (low water level)
Adopted Model Scale = 1:15	
$D_{50(p)} = 2.3 \text{ mm}$	$D_{50(m)} = \frac{D_{50(p)}}{N_L} = \frac{2.3\text{mm}}{15} = 0.15 \cong 0.2\text{mm}$

Based on the Shield's Diagram calculations a model scale of 1:38 would suffice for sediment transport. According to literature reviewed in chapter 2, a larger model scale size

was thought to facilitate greater visualization of the sediment and flow hydrodynamics associated with the SSO. In this study a larger scale of 1:15 was adopted as the model scale and was applied to all linear dimensions. It is often difficult and time consuming to engineer sediment with a specific median diameter because large quantities are required for a movable-bed coastal model. Often, the solution is to use readily available commercial sand which has similar geometric properties to the prototype but differs in some respects like density. A particle distribution curve was produced in Figure 3.24 to calculate the median sediment diameter for the model sediment, both the prototype and model sediment were found to be geometrically similar to each other.

Figure 3. 24 Gradation curve of construction sand used for mobile-bed.

3.5 Sounding to determine model bathymetry

The model sand was designed in section 3.4 and was used as mobile-bed sediment for the model bathymetry. In order for the model to inherit prototype conditions it was necessary to survey the embayment area, surrounding coastline and lake bed so that hydrodynamic

processes that occurred in field would develop in the model. Bathymetric data for the region under investigation was acquired from the Canadian Hydrographic Survey (CHS) or NOAA at 1 m contour intervals. Unfortunately, the data from the CHS and NOAA was not accurate enough to design a 1:15 scale model so it was necessary to utilize sounding and survey techniques in the field to acquire useful data.

Figure 3. 25 Surveying instrument.

Figure 3. 26 Member of the survey team.

Figure 3. 27 Small water craft that was used to measure horizontal and vertical distances.

A diagram was created in Figure 3.28 to list the sounding locations relative to the Green Road embayment area and respective beaches. The area covered by the survey had a width of 125 m and a length of 225 m. The acquisition of the sounding data was crucial for accurate representation of bottom contours because the bed slope was a function of this

parameter. Table 3.8 below summarizes the dimensions of the future model that was to be constructed.

To acquire the most accurate data, a theodolite was used to measure distances and angles of various locations around the site. Water depths were measured by using a long yellow polyethylene string to measure several horizontal distances that radiated from the theodolite at the outfall. Approximately fifty survey points were measured so that researchers could reproduce the lake contours in the lab these survey points were spaced a minimum distance of 10 m to a maximum distance of 30 m apart, and the spacing for particular direction was a direct function of the amount of accuracy that was deemed necessary for a particular region in the model. Observing Figure 3.28 it can be seen that a greater amount of points predominates in the eastern direction because it was believed the majority of sediment was coming from this direction. Shown below in Figures 3.25-3.27 are a series of photos that illustrate different stages of the survey operation.

Table 3.8 Summary of potential model dimensions

Prototype dimension	Model dimensions
$N_{p(\text{width})} = 125 \text{ m}$	$N_{m(\text{width})} = \frac{N_{p(\text{width})}}{N_L} = \frac{125m}{15} = 8.33m$
$N_{p(\text{length})} = 225 \text{ m}$	$N_{m(\text{length})} = \frac{N_{p(\text{length})}}{N_L} = \frac{225m}{15} = 15m$

The vertical and horizontal lengths calculated in Table 3.8 were optimized to reduce costs, calculations were performed in section 3.6 to ensure adequate wave generation and sediment transport was possible when various linear measurements such as length, width and height were changed.

Figure 3. 28 Locations of sounding points to determine water depths and horizontal distances.

3.6 Selection of Model Water Level

Like many other bodies of water the water level in Lake Ontario fluctuates with seasonal variations. The Canadian Hydrographic Society has recorded the water level in Lake Ontario since 1918 and the variations of this data were plotted in Figure 3.29.

Figure 3.29 Historical variations of Lake Ontario water level (Courtesy of CHS).

The variations in the Lake Ontario water level are marked by extreme high and low water levels throughout the year. The selection of the appropriate model water levels should reflect average water level values that occur throughout the year and not extreme values that are in the minority. It was decided by EHG that in order to properly test the model it was necessary to use two water levels: high and low. Shown below in Figure 3.30 are the seasonal variations of the water level over a 2 year period.

Figure 3.30 Average high and low long-term water level in Lake Ontario.

The red highlighted water levels shown in Figure 3.30 were used throughout the model testing to represent both high and low water levels. The CCIW floor was used as the reference floor elevation to calculate all vertical distances. Figure 3.31 shown below illustrates the numerous case studies that were analyzed in order to find the optimal reference floor elevation. The calculations for the reference floor elevation are shown below.

Figure 3. 31. Various scenarios that were used to calculate the optimal floor elevation.

There are three case studies presented in Figure 3.31. Case study one used an initial floor elevation of 50.0 m to produce a water depth of 1.67 m between the floor and high water level (HWL). This depth does not include free board (FB) and wave breaking (WB) space required for normal model operation. Case number one was far too expensive to construct because the containment wall would be over two metres high and the mobile bed slope would be far too steep resulting in premature wave breaking. Case number two used a higher floor elevation which reduced the containment wall height but resulted in a low water level, this can affect wave generation and sediment transport. Case number three used a floor elevation of 69.27 m which has been calculated by taking into account the thickness of the model self-scouring outfall Plexiglas base and finer details. The advantages of using this 3rd model includes many:

- Sufficient water depth for wave generation and sediment transport
- Optimal containment wall height
- Adequate mobile bed slope for wave propagation and wave breaking

As stated previously, two water levels were used for the model simulations and they are calculated below in Table 3.9.

Table 3.9 Water level calculations

Prototype Water Level	Model Water Level
$H_{p-HWL} = 75.07 \text{ m}$	$H_{m-HWL} = \frac{HWL_p - CCIW_{floor}}{N_L} = \frac{75.07m - 69.27m}{15} = 0.387m$
$H_{p-LWL} = 74.52 \text{ m}$	$H_{m-LWL} = \frac{LWL_p - CCIW_{floor}}{N_L} = \frac{74.52m - 69.27m}{15} = 0.350m$

where

H_{p-HWL} & H_{p-LWL} - Prototype high and low water level

H_{m-HWL} & H_{m-LWL} - Model high and low water level

$CCIW_{floor}$ - Calculated reference floor elevation

A high and low water elevation of 38.7 cm and 35 cm was used in the physical model to reproduce high and low water levels found in Lake Ontario. The usage of the high and low water levels found in Lake Ontario is representative of the water levels the new self-scouring outfall would be subjected to in the embayment.

3.7 Construction of Model Containment Wall

The high water level was calculated in section 3.6 and was found to be 38.7 cm and the maximum wave height that was measured in the model was found to be 13 cm from trough to peak. The superimposition of a 13 cm wave (high wave height) in deep water (depth = 38.7 cm) against a 60 cm high wall in Figure 3.32 was intuitive as it indicated to researchers the maximum amount freeboard that was available visually. Upon completion of a cost to benefit analysis it was concluded the optimum model containment wall height was 60 cm because there existed sufficient freeboard for the generation of maximum water level and wave height.

Figure 3.32 High wave propagation and high water level in wave tank

Closer inspection of Figure 3.32 indicated there existed only 14.5 cm of freeboard space available between the wave peak and the top of the containment wall. Therefore, it was concluded by researchers that only calculated wave heights from the testing regiment should be adopted to test the outfalls and extreme high water levels should be strongly avoided. In order to retain a large volume of water for testing, a containment wall was constructed in Figure 3.33 by a private contractor to house the wave generator, fixed-bed, movable-bed and SSO model. The wall was exactly three cinder blocks high and 0.2 m thick and produced an internal rectangular area of 8.8 m by 11.8 m or 104 m². The cinder blocks were of construction grade

measuring 0.2 wide by 0.4 long by 0.2 high. When this wall was completed in Figure 3.34, the contractor applied a waterproofing membrane on the wet side of model containment wall to prevent the wave pool water from leaking out. Upon completion of the wall, a reinforcing steel rod and plate system was installed at 1.2 m intervals around the perimeter of the wall, a nut attached to a steel rod was later tighten to make the wall function as one piece to prevent overturning.

Figure 3.33 Dimensions of physical model.

Figure 3.34 Installation of the waterproofing membrane in the interior of the model.

3.7.1 Wave Generator

A piston-type wave generator manufactured by Kelk was used in the physical model study to produce unidirectional sinusoidal waves. Although not the most sophisticated in design, the Kelk wave generator was sufficient for this study's application. The Kelk wave generator shown in Figure 3.35 has a 5 m wide baffle board situated at the front of the machine to displace water, installed at the rear are multiple holding tanks that are used to balance

opposing forces. The hydraulic pump and KELK control system shown in Figure 3.36 were used to send hydraulic fluid to dual pistons in the wave generator that produced horizontal sinusoidal oscillations of the 5 m baffle board. A signal generator was also attached to the hydraulic pump so that researchers could duplicate the characteristic wave amplitude and wave period inherent to the Green Road embayment region of Lake Ontario. The combination of these four pieces of hardware enabled researchers to successfully generate model waves in agreement with prototype waves.

Figure 3.35 Kelk wave generator.

Figure 3.36 Kelk wave generator control system and hydraulic pump for wave generator.

3.7.2 Grid Alignment

Upon completion of a land and water survey of the Green Rd. outfall region, the measurements were transferred to an 8.5 by 11 inch sheet of paper. A scale system was set up on the paper to determine how much 1 centimetre represented in the prototype, through further numerical manipulations the scale factor ($N_L = 15$) was used to determine all dimensions in the model. An example is shown below in Table 3.10

Table 3.10 Sample calculation of grid scale

Step 1	If 6.5 cm = 56.25 m (on the 8.5 by 11 inch sheet of a paper), then
Step 2	1 cm = 8.654 m (which means 1 cm on the 8.5 by 11 inch sheet of paper equals 8.654 m in the prototype)
Step 3	$8.654 \text{ m} / 15 = 0.577 \text{ m}$ (By dividing the prototype value by the scale factor (15) produces the model value of 0.577 m)

The procedure shown above in Table 3.10 was used to determine all model dimensions. In order to convert and process the large number of prototype measurements an excel spreadsheet was quickly created to calculate all model values in a quick and efficient manner, this spreadsheet can be found in Appendix A3. A Cartesian coordinate system was setup in Figure 3.37 to establish an origin from which all horizontal and vertical measurement points were going to radiate from.

Figure 3.37 Model grid system.

In Figure 3.37 a series of horizontal and vertical lines were superimposed on the photo of the CCIW floor to illustrate the simplicity of using a grid system, at the ends of the lines were a series of alphanumerical designations that were pasted on the walls at all points on the grid to facilitate easier peg placement when moulding the bed.

3.8 Construction of Seawall and Embayment

Aerial photos previously shown in Figure 3.12 illustrate how the Green Rd. outfall was situated in an embayment flanked on the eastern and western sides by seawalls that extended along the coastline for erosion protection. The construction of the physical model shown in Figure 3.38 incorporated prototype style seawalls that were constructed out of interlocking landscaping bricks measuring 0.22 m long by 0.09 m wide by 0.07 m high. Figure 3.38 is a snapshot of the partially constructed seawall that was photographed during the construction phase. Upon completion of the seawall the structure was inspected for imperfections such as openings which were sealed with mortar consisting of 1 part Type 10 Portland cement to 3 parts sand, all of which mixed with water.

Figure 3. 38 Construction phase of seawall.

It was confirmed during the survey that the majority of the sediment was coming from the north east direction, therefore it was decided that the eastern region of the embayment was more dynamic in terms of sediment transport and should constitute the majority of the model area. The orientation of the seawalls were interpolated from aerial photographs of the region using numerical scaling and careful measurement. Upon completion of the seawall in Figure 3.39, construction sand was distributed on the south side (dry side) of the east seawall to support the wall against overturning as a result of hydrostatic pressure-a potential problem on the north side (wet side). It was decided by researchers that waves were not permitted to overtop the seawall during high water level (75.07 m) and high wave height (0.13 m), to maintain a reasonable degree of control 6 rows of landscaping bricks were used (yellow dashed lines) to develop sufficient vertical height.

Figure 3. 39 Completed seawall with fill on the south side to prevent overturning.

3.8.1 Design and Construction of the Fixed-Bed

A fixed bed was utilized for the construction of the physical model because researchers believed it was an integral component that was critical for generating stable waves and essential for hydraulic similitude. The construction of the fixed bed was not immediately undertaken because a detailed survey of the lake bathymetry northeast of Green Rd. embayment had to be conducted to determine the depth of the seafloor relative to the water surface. The elevation pegs shown in Figures 3.40 and 3.41 were measured during the land and lake survey and were used to construct the phase 1 and 2 model bathymetry for waves

propagating from the northeast and eastern direction, respectively. To minimize redundancy, only construction photos from phase 1 are shown in this chapter.

Figure 3.40 Peg location and elevations (shown in red) for phase 1 layout (Waves from the northeast).

The fixed bed was constructed prior to the installation of the movable-bed because a seamless transition between the fixed-bed and movable-bed was required. Figure 3.42 below is a pre-construction drawing created by EHG which outlines the general slope of the fixed and movable bed. It should be stated that in some regions of the movable-bed the slope can exceed 4%; the slope in specific regions is for the most part governed by the lake and coastal bathymetry shown in Figures 3.40 and 3.41.

Figure 3.42 Critical components in the physical model including bed slopes [Courtesy of EHG].

The coarse aggregate shown in Figure 3.43 was used for constructing the fixed-bed. Elevation pegs were placed upright on the CCIW floor to indicate the vertical distance for a specific location. When all the pegs were installed a layer of coarse aggregate was placed on the floor and levelled to match the grade of the elevation pegs. It was necessary for all the stones to settle and not move when vertical pressure was applied to them so compaction was utilized to help the rocks settle into their final resting position. Shown in Figure 3.44 is the completed fixed bed.

Figure 3.43 Different sizes of coarse aggregate.

Figure 3.44 Completed fixed bed and partially completed transition zone.

When all the stones were secured in their surrounding, a grout mixture consisting of 1 part cement to 1 part sand was mixed together with water to yield a grout paste which had the consistency of maple syrup. The grout was poured onto the fixed bed to form a thin layer; a trowel was later used to smooth any inconsistencies on the bed. A transition layer was created between the fixed and mobile bed and was modelled according to the pictorial representation in Figure 3.42. A summary of the fixed-bed area measurements is presented in Table 3.11.

Table 3.11 Summary of fixed bed measurements

Parameter	Length
Width	6.1 m
West Depth	≈ 4 m
East Depth	≈ 3.5 m

3.8.2 Movable-Bed and Embayment Construction

The construction of the movable-bed entailed depositing sand into depressions that formed between the fixed-bed and seawall. Many cubic metres of sand were brought into the model to mould the movable-bed according to the lake bathymetry outlined in Figures 3.40 or 3.41. A smooth movable-bed surface was facilitated by dragging a short piece of wood with a straight edge over the sand to produce level contours that were free of depressions or ridges.

The amount of labour intensive work that was required during the preliminary stages of this project can be seen in Figure 3.45 as mounds of dirt are strategically positioned along the width of the model for future moulding initiatives. A specially built rigid retaining wall shown in Figure 3.46 was used to prevent the movable-bed foundation sand from escaping during wave generation. Visible red-tipped elevation pegs are presented in Figure 3.47; these were instrumental in moulding the bed accurately.

Figure 3.45 Construction phase of movable-bed.

Figure 3.46 Containment wall.

Figure 3.47 Red-tipped elevation pegs in a partially completed movable bed.

With respect to Figure 3.47, a gently sloping beach of 2-3% was created to produce a seamless transition between the lake and coastal beach, similar to field conditions. A beach slope that was too high would only supply unwanted sediment to the embayment area during

wave breaking and would most likely contribute to beach erosion. A detailed survey was conducted of the embayment area for as the focal point of this project, all aspects of this region needed to be flawlessly reproduced in the physical model. Graphical aides have been superimposed on Figure 3.48 to indicate the initial condition of the embayment prior to the installation of armour stone and sheet piles, shown in the completed embayment in Figure 3.49. The configuration shown in Figure 3.49 was used to calibrate the model but was later changed for testing purposes because it was observed during model testing that crushed stones forming the geometry of the embayment were moving during wave activity. The movement of such rocks in the model was undesirable as this did not occur in the field. To remedy this error, the crushed rocks were replaced with heavier landscaping stones incapable of movement for the prescribed water level, wave height and wave celerity (as depicted in Figure 3.50).

Figure 3. 48 Embayment area prior to the installation of armour stone.

Figure 3. 49 Calibration test configuration of embayment area.

The outlet pipe shown in Figure 3.49 was scaled to the appropriate model size and the coarse aggregate that was used for the fixed bed was used as armour stone to flank both side of the outfall in the same figure. For future tests, the armour stone was replaced with large blocks that did not move. A summary table of critical embayment components that were used during model calibration are shown in Table 3.12.

Table 3. 12 Embayment parameters for calibration setup

Parameter	Prototype, p	Model, m
Circular Outlet Pipe	Diameter = 1.8 m	$Diameter_m = \frac{D_p}{N_t} = \frac{1.8m}{15} = 0.12m$
Armour stone	Rocks should be sufficiently large and heavy such that approaching waves cannot move rocks.	(waves should not move stones)
Sheetpiles	Porous structure which allows/hinders the approaching mass of water.	Porous structure which allows/hinders the approaching water mass (made to resemble existing prototype sheet piles.)

3.8.3 New Rock Geometry

During model tests, it was observed that water waves were displacing coarse aggregate that formed the geometry of the embayment. This type of situation was undesirable because it deviated from observed prototype behaviour in the field. After evaluating the problems associated with small coarse aggregate it was decided that large heavy cubical bricks were to be used as armour stone to dissipate any approaching wave energy. The newly installed armour stone and metal sheet piles can be viewed in Figure 3.50. In order to further stabilize the armour stone configuration a grout mixture with the consistency of chocolate milk was created and was applied to the rock surfaces and allowed to flow through the cracks to form a stable rock mass.

To test the self-scouring outfall adequately, two wave directions were simulated. Waves from a north easterly direction were produced (Phase 1) and waves from an easterly direction (Phase 2) were also created. It should be noted that schematic drawing shown in Figure 3.51 was created as a result of a detailed design/survey procedure that was undertaken to measure the horizontal and vertical distances of the rock geometry/configuration so that the same rock mass could be easily reproduced in phase 2.

Figure 3. 50 New armour stone material and metal sheet piles.

Figure 3. 51 Armour stone geometry for phase 1.

Additionally, the red circles shown in Figure 3.51 represented the outer perimeter of the armour stones that were situated on the movable-bed and the green circles represent the toe of the armour stones were used to mark the beginning of the upward slope. For this model, an armour stone slope of 1:3 was adopted because this number was representative of conditions found in the field. To achieve similitude, all components of the outfall were painstakingly measured and scrutinized to ensure all structures in the model agreed with the prototype.

The Green Rd. outfall has metal sheet piles located at the front of the embayment (in the lake), installed years ago as a preventative measure to block sediment from entering the embayment. It was decided by researchers that the metal sheet piles were to be modelled in the lab because the cost associated with their removal was far too expensive. During the field visit, intricate measurements were taken of the initial sheet pile condition and are shown in Figure 3.52, these measurements were later converted to model values in Figure 3.53 and were later used to build the model sheet pile area that was shown in Figure 3.50.

Figure 3. 52 Metal sheet piles in the field.

Figure 3.53 Schematic drawing of sheet piles that were used in the model, colour shades symbolize changes in elevation.

Table 3.13 was produced to list the calculated model sheet pile elevations that were used to construct the sheet pile area in the physical model.

Table 3.13 Sheet pile dimension summary table for model

Location according to Figure 3.47	Prototype, m	Model, m
Left hand side (pink)	$H = 75.16 \text{ m}$	$H_m = \frac{H_p - CCIW_{floor}}{N_L} = \frac{75.16 - 69.27}{15} = 0.392 \text{ m}$
Bottom left hand corner	$H_p = 75.10 \text{ m}$	$H_m = 0.389 \text{ m}$
Bottom right	$H_p = 74.94 \text{ m}$	$H_m = 0.378 \text{ m}$
Bottom right hand corner	$H_p = 75.16 \text{ m}$	$H_m = 0.393 \text{ m}$
Lower right hand side	$H_p = 75.01 \text{ m}$	$H_m = 0.383 \text{ m}$
Top right hand side	$H_p = 75.16 \text{ m}$	$H_m = 0.393 \text{ m}$
Model width of sheet pile area		$H_m = 0.42 \text{ m}$
Model depth of sheet pile area		$H_m = 0.26 \text{ m}$

An embayment grade of about 4.5 % was adopted because it provided a seamless transition between the pipe invert elevation and the outermost elevation peg in front of the sheet piles. Also, landscaping bricks that were used to construct the seawall were once again employed in Figure 3.50 to construct wing walls on both sides of the existing outlet pipe. A summary was produced in Table 3.14 to list the critical design values of the embayment area.

Table 3. 14 Embayment summary table

Component	Model Length, m
Slope of east and west armour Stone	1 : 3
Slope of embayment	4.5%
Wing wall material	Landscaping bricks – (L=0.22, W=0.09, H=0.07)

3.9 Self-Scouring Outfall - I

A self-scouring outfall is a unique structure that has the ability to prevent sediment intrusion into an outlet pipe and reduce the frequency of cleanouts by utilizing sloping channels to promote wave breaking and sediment scour. There are currently no published guidelines that outline acceptable design limits for a self-scouring outfall. The City of Toronto has one of the only known working prototypes of a self-scouring outfall located in the Eastern Beaches area of Lake Ontario. EHG staff visited this site and observed the performance of this outfall, shown below is photo of this outfall (Figure 3.54 and Figure 3.55).

Figure 3. 54 Existing self-scouring outfall in the City of Toronto.

Figure 3.55 Various components of the self-scouring outfall in City of Toronto

Detailed testing results and performance analysis concerning the SSO are presented and analyzed thoroughly in the proceeding chapter, thus this section is only concerned with the design parameters of the physical model. With respect to the wave actions shown in Figure 3.55, as waves from lake propagate towards the SSO they have an opportunity to break on the wave run-up slope or travel over the slope and scour sediment that has accumulated on the converging down slope. Storm water that exits via the grate opening from the existing outlet pipe is beneficial because storm water has an opportunity to harmlessly flush sediment on the converging down slope through the outlet channel and into the lake.

Building on previous knowledge of outfall design and extensive experience in hydraulics, EHG was able to build a preliminary model shown in Figure 3.58. From the model, an AutoCAD drawing was generated by carefully measuring all dimensions of the 1:15 scale undistorted self-scouring outfall so that the digital file could be modified at a later time. Figure 3.56 is a plan view drawing of SSO-I, the symmetrical nature of the structure poses many dilemmas for the designer because a flow spreader needed to be designed to divide the stormwater flow that was discharged from the outlet pipe during storm events. To alleviate this problem, a triangular diffuser structure shown in Figure 3.57 was developed to divide the flow exiting the existing outfall pipe. At the present time there is no information

to suggest this type of flow divider structure will be used in the prototype outfall since there are vast arrays of cheaper alternatives that do not require extensive fabrication to produce. The completed self-scouring outfall model shown in both Figures 3.58 and Figure 3.59 was constructed out of Plexiglas, silicone, hot glue and reinforced waterproof cardboard and was built strongly enough to withstand the significant wave pressures from approaching waves. Although SSO-I demonstrated good performance and is similar to the prototype SSO found in Toronto, designers and researchers believed that by designing a second outfall (SSO-II) was the only way to implement design modification and improve performance.

From a design standpoint, the geometry inherent to SSO-I in Figures 3.58 and 3.59 was drastically altered to incorporate the new modifications of SSO-II.

Figure 3.56 Schematic drawing for model SSO – I.

Figure 3. 57 Note that only half of the diffuser has been drawn because the structure is symmetric.

Figure 3.58 Water waves approaching from the northeast direction are interacting with the SSO-I.

Figure 3.59 Plan-view of the internal mechanics of SSO-I.

Approaching water waves that encroached on the SSO-I structure shown in Figures 3.58 and 3.59 had an opportunity to break on the wave run-up slope or overtop the slope and travel down the steep grade of converging down slope and leave through the outlet opening.

Waves were also free to propagate with sediment into the outlet channel and leave in the same manner after breaking. Please note the hydrodynamics of the outfall will be discussed in later sections.

3.9.1 Self-Scouring Outfall - II

Modifications were made to SSO-I, as shown in Figure 3.60. The new features were intended to improve the scouring performance of the outfall. The improvements that were made to the SSO-I were developed in light of a site visit to the Toronto Eastern Beaches SSO, a brainstorming session that occurred between EHG staff, and observations of the internal hydrodynamics of the model outfall during testing.

Figure 3. 60 Schematic drawing of model SSO-II.

An additional photo has been inserted in Figure 3.61 to provide a three-dimensional perspective of SSO-II and to help understand the hydrodynamics within the structure.

Figure 3. 61 This figure illustrates the hydrodynamics within the outfall as stormwater exits and waves propagate towards the structure.

Figure 3.61 shown above contains the design modifications that were made to improve the performance of the self-scouring outfall. These improvements are listed below:

- Merge the wave run-up slope and converged down slope face in order to decrease the grade of the converged down slope
- Installation of a trapezoidal wooden block between the wave-run up slope and converged down slope
- A slope was built within the sewer headwall to accelerate the flow leaving the circular outlet pipe
- The orientation of the east and west outlet channel was rotated by 110 degrees clockwise and anticlockwise, respectively

From literature, it was found that the angle of repose for most sandy material was between 26 and 34 degrees (Nielson, 1992). Furthermore, for beach sand, a value of 31 degrees was typical (Hanes and Inman, 1985b). In the design of the wave run-up slope a value of 30 degrees was adopted for the angle of repose. Figures 3.62 and 3.63 that are to follow were prepared by EHG and illustrate the elevation of various components found in SSO-II.

Table 3.15 has been prepared to summarize the various slope angles found on different components of Figure 3.63, elevations and lengths were not listed in the table because these can be read directly from the figure.

Table 3. 15 Model slopes for various components

ID#	Component	Slope, °
1	Wave run-up slope	30
2	Wave run-up benching slope	24.5
3	Converged down slope	8
4	Top of outlet channel	23
5	Bottom of outlet channel	5

3.10 Alignment of SSO

The alignment of the self-scouring outfall was a critical point in the project because this task required extensive analysis to ensure the outfall was located at the best possible position to maximize its performance. A comprehensive analysis regarding the alignment of the SSO was conducted by EHG, resulting in the alignment of the outfall with:

- The outermost sheet piles.
- The smoother coastline.
- A berm that is 10.05 m lake ward from the present outfall.

In order to judge which outfall alignment should govern a criterion system was setup to facilitate this procedure. The best outfall alignment was based on hydrodynamics, morphology, aesthetics, and maintenance.

Table 3.16 was created on the following page to calculate the best alignment option by assigning a numerical weight to each sub criteria which was then used to calculate the total weighted score.

Table 3.16 Point Selection Summary Sheet for the Alignment of the SSO

Criteria	Sub Criteria	Weight	Alignment with sheet piles		Alignment with smoother coastline		Alignment with a berm 10.05m lake ward	
			Score	Weighted Score	Score	Weighted Score	Score	Weighted Score
Hydrodynamics	1. -No energy convergence in embayment because outfall is outside of embayment	4	4	16	2	8	3	12
	2. -Protection needed on both E. & W sides (Additional work)	3	3	9	1	3	1	3
	3. -Return to existing natural coastline	5	1	5	5	25	1	5
	4. -Front of SSO is inside armour stone	5	1	5	5	25	1	5
	5. -No Significant protection from armour stones	5	1	5	5	25	1	5
	6. -Some modifications of wing walls as per design	3	3	9	3	9	1	3
	7. -Located at the convex nature of coastline	3	3	9	3	9	3	9
	8. -Wave activity inside of embayment equals convergence of energy	5	1	5	3	15	5	25
	9. -Formation of small embayment in front	2	2	4	2	4	2	4
	10. -SSO location is an interruption of the natural coastline	2	1	2	2	4	1	2
Morphology	1. -All sediment movement and deposition is outside the outfall	4	4	16	4	16	4	16
	2. -The updrift side of the shoreline advances almost to the top of the wing wall because of convex shoreline	4	4	16	2	8	2	8
	3. -Current beach may be wider along armour stone	4	4	16	4	16	2	8
	4. -Downdrift of SSO beach erosion may extend very far, depending on wave direction	3	3	9	3	9	3	9
	5. -Located at convex nature of coastline	3	3	9	3	9	3	9
	6. -Aerial photos indicate E. & W. of embayment are stable (no erosion)	5	3	15	5	25	3	15
	7. -Alignment of E. sea beach is natural evolution developed coastline	5	3	15	5	25	3	15
	8. -Sediment deposition is beyond this line	5	3	15	5	25	3	15
	9. -Unable to predict morphology	2	2	4	2	4	1	2
Aesthetic	1. -May not be unpleasant since SSO has artistic appearance	4	4	16	4	16	4	16
	2. -The SSO may be an eye sore in this location	3	1	3	3	9	1	3
	3. -Location is acceptable	5	5	25	5	25	5	25
Maintenance	1. -Easy to maintain	5	5	25	5	25	5	25

	2.	-Longer access road required	3	3	9	3	9	3	9
	3.	-Easy Access	5	5	25	5	25	2	10
	4.	-Need to extend sewer	1	1	1	1	1	1	1
	Total			→	288	→	374	→	259

Weighting Criteria for Selection of SSO Alignment

- Score of **1** → ● "Weak"
- Score of **2** → ● "Moderate"
- Score of **3** → ● "Good"
- Score of **4** → ● "Very Good"
- Score of **5** → ● "Excellent"

It can be seen from Table 3.16 that scenario two has a highest tabulated score, indicating that by aligning the self-scouring outfall with the *naturally smoother coastline* was the best option. The reasons for doing such will not be discussed since these reasons are clearly stated in the table. Figures 3.64 to 3.66 shown below are schematic representations of the different alignment options that were analyzed for this project.

Figure 3. 64 Outfall aligned with outermost sheet piles. Figure 3. 65 Outfall aligned with smoother coastline.

Figure 3. 66 Outfall aligned with berm 10.05 m lake ward

3.11 Wave Absorbers

The usage of wave absorbers within the model was critical because they helped to reduce wave reflection and provided an avenue for absorber-bound waves to exit. In this particular project wave absorbers were placed along both sides of the model and behind the wave generator to absorb/reflect incoming waves. Figure 3.67 shows the inner geometry of the wave absorber and Figure 3.68 illustrates how the wave absorbers were positioned in the model.

Figure 3. 67 Inner geometry of wave absorber.

Figure 3. 68 Placement of model wave absorbers.

The wave absorber shown in Figure 3.67 was constructed by using perforated channels aligned at 45 degrees to reduce internal weight and to direct water out of the model. The wave absorber was approximately 4 kg and was constructed from a light weight plastic material. In order to ensure that the wave absorber remained stationary during wave generation it was necessary to place significant weight on top of the absorbers. This reduced movement and also ensured the wave absorber boundary was always rough and turbulent. The wave absorbers on the west side of the model that were closest to the wave generator were placed on their side to produce a non-perforated smooth surface (i.e., wall). This feature increased the number of wavelengths created and permitted waves to propagate close to the SSO before they broke to dissipate their energy. Wave absorbers were also placed behind the wave generator to reduce wave reflection as the 5 m wide baffle board oscillated. Figure 3.69 shows a schematic layout of the wave absorbers in the model.

Figure 3. 69 Placement of wave absorbers in the model.

The wave absorbers that were shown in the previous figures were strategically positioned so that waves in the study zone could travel through the perforated channels and harmlessly return to wave generator. To minimize reflection and maximize absorption, a 1:3 gravel slope was constructed outside the study area between the containment wall and wave absorbers to dissipate rogue waves that penetrated through the wave absorbers. The coarse aggregate that was used to build the fixed-bed was once again employed to construct the gravel slope on the inside of the containment wall. Table 3.17 summarizes the characteristics of the wave absorber and any design parameters that were used to position the wave absorbers and construct the gravel slope. An example of flows leaving the study zone and returning to wave generator is shown in Figure 3.70.

Table 3. 17 Parameters for wave absorbers

Parameters for Wave Absorber	Quantity
Length =	3.0 m
Width =	0.6 m
Height =	0.5 m
Weight =	≈ 4 kg
Parameters for Gravel Slope	Quantity
Slope	1:3
Type of aggregate	Coarse (20 – 40 mm)

Figure 3. 70 Waves leaving the study area via the wave absorber and travelling toward the wave generator.

3.12 Wave Staff Measurements

The acquisition of buoy data enabled calculations of model wave height and period. In order to validate the results from the model it was necessary to measure such characteristics as wave height and model wave period using wave staffs. During this project, five capacitance-type single wire wave gauges were used to measure the instantaneous surface fluctuations of the water waves in real time. LabView software was used to create a program that could interpret the electrical impulses from wave staffs and convert them into meaningful data. A sample data stream of the wave staff measurements was presented in the results sections. The wave staffs used in this project were supported by aluminium tripod mounts resting on the CCIW floor for stability. A heavy lead block was placed on top each wave staff to prevent large waves from overturning the structure and damaging the sensitive apparatus. Presented in Figures 3.71 to 3.74 are the various components of the wave staff.

Figure 3. 71 Wave staff measurement apparatus.

Figure 3. 72 Side view of wave staff.

Figure 3. 73 Close look at electronic measuring devices.

Figure 3. 74 Space between wave staff and floor.

It was believed by researchers that the placement of the wave staffs relative to the SSO was important because it was necessary to obtain accurate measurements in shallow regions of water as well as in deep regions of water so that model results could be corroborated with field data. Figures 3.75 and 3.76 show the final placement of wave staffs in the study zone.

Figure 3. 75 Preconstruction schematic drawing of wave staff placement in the model.

Figure 3. 76 Post-construction placement of wave staffs in the physical model.

To validate model results a wave staff gauge was placed directly in front of the wave generator more than one wave length away. This was done because as waves enter shallow and deep regions their respective heights were found to be a function of bottom contours. The placement of the wave staffs in the model completed the design phase of this project. Table 3.18 on the following page summarized all the numerical values that were used to design and build the physical model. Furthermore, all design values that have been obtained to this point have only been used for the construction of phase one which reproduced waves approaching from the northeast direction. Phase two entails waves approaching from the east, to model this phenomenon the seawall was rebuilt and bathymetry was recalculated to simulate waves approaching from an easterly direction. More detail on the construction of phase two is not provided as the construction process was identical to phase one. (Results obtained from phase two are presented and analysed in later chapters.) All design work related to phase two is presented in Appendix A3.

Table 3. 18 Summary of Model and Prototype Dimensions

Parameter	Prototype	Model
Model scale, N_L	Model scale	$N_L = 1:15$ (undistorted model)
Period 1, T	$T_p = 4.0$ sec	$T_m = 1.033$ sec
Period 2, T	$T_p = 6.0$ sec	$T_m = 1.549$ sec
Wave height 1, H	$H_p = 1.43$ m	$H_m = 0.095$ m
Wave height 2, H	$H_p = 1.95$ m	$H_m = 0.130$ m
Sediment diameter, D_{50}	$D_{50-p} = 2.3$ mm	$D_{50-m} = 0.2$ mm (construction sand)
Width of study area, N_{width}	$N_{width-p} = 125$ m	$N_{m-width} \approx 8.8$ m
Length of study area, N_{length}	$N_{length-p} = 225$ m	$N_{m-length} \approx 11.8$ m
Water level 1, H_{HWL}	$H_{HWL-p} = 75.07$ m	$H_{m-HWL} = 0.387$ m
Water level 2, H_{LWL}	$H_{LWL-p} = 74.52$ m	$H_{m-LWL} = 0.350$ m
Circular outfall embayment pipe, D	$D_p = 1.8$ m	$D_m = 0.12$ m

Table 3. 19 Model Design Summary Table

Cinder blocks for containment wall	Length = 0.4m, Width = 0.2m, Height = 0.2m
Waterproofing membrane for internal wall	Manufacturer: Grace, Composition: Epoxy compound
Landscaping bricks for seawall	Length = 0.22m, Width = 0.09m, Height = 0.07m
Binding material for seawall	1 part Type 10 Portland Cement : 3 parts construction sand + water
Wave Generator	Model: KELK, Type: Unidirectional piston type
Fixed bed material	Coarse aggregate = 20 – 40 mm
Fixed bed dimensions	Width = 6.1m, Length = 4 m
Binding material for fixed bed	1 part Type 10 Portland Cement : 1 part construction sand + water
Armour stone material	Landscaping bricks broken into small pieces (0.1cm x 0.1cm)
Armour stone slope	1:3
Armour stone placement	Refer to Figure 3.50 and 3.51
Embayment slope	4.5%
Wingwall construction material	Landscaping bricks
Design of sheet pile area	Refer to Figure 3.52 & 3.53 and Table 3.13
Suggested design values of SSO	Refer to Figure 3.62 and 3.63
Alignment of SSO	Refer to Figure 3.64 – 3.66
Wave absorbers	Length = 3m, Width = 0.6m, Height = 0.5 m, Weight ≈ 4 kg, Perforated channels aligned at 45°
Gravel slope – adjacent to containment wall	Material: 20-40 mm coarse aggregate (same as fixed-bed), Filter fabric for regions adjacent to movable-bed, Slope: 1:3
Placement of wave staffs	Refer to Figure 3.75
Liquid medium	Potable water from municipal distribution system
Duration of each simulation	Time = 10 minutes

Chapter 4 Results and Analysis of the Physical Model

Designing the Green Road embayment physical model required extensive field work because adequate prototype data had to be collected to ensure characteristics like bathymetry and sediment composition were accurately reproduced in the lab. The design of a hydraulic model entails performing a great deal of optimization work in order to reduce costs and save time. In this section, figures, tables and graphs were presented and were subjected to critical analysis to determine the performance of the self-scouring outfall. Additionally, sediment transport graphs and wave-staff data are also presented to correlate experimental results to prototype conditions as a method of model validation.

4.1 Experimental Results from the Physical Model

The design and construction of the physical model was an intricate process that required a great deal of thought, mathematical calculations and optimization in order to accurately reproduce prototype conditions. The study of a self-scouring outfall is for the most part a qualitative study that uses logical deductions to decipher gathered data as well as observed model behaviour. This type of study is usually more expensive, has a smaller sample size and has variables that are harder to measure. The usage of a physical model was ideally suited for this type of study because it allowed researchers to observe a multitude of three-dimensional processes like sediment transport, wave motion and outfall hydrodynamics, all which were occurring at the same time.

The results presented in this section have not been calculated using complex mathematical equations, but rather, have been observed or recorded by electronic measuring devices. On a note of caution, researchers were vigilant when observing the performance of the self-scouring outfall because of laboratory effects and scale effects that are inherently present in physical models. It is often hard to definitively say that a specific scenario that occurred in model simulations will play out in the prototype because of the inability of researchers to accurately reproduce minute details.

4.2 Calibration of the Physical Model

The calibration of the physical model was a critical step that was not to be bypassed because it ensured that the model was able to simulate prototype conditions to a reasonable degree of accuracy. In the literature review, various types of calibration options were listed for calibrating a physical model, subsequently, *calibration by similitude* was adopted in this study because extensive field data was available to corroborate with model results. In order to properly calibrate the model and conduct model runs, two hydraulic parameters were used to test model; that is, the model used:

- Two different *Water levels*.
- Two different *Wave height* (with corresponding periods).

The calibration procedure did not include the self-scouring outfall structure because the objective of this operation was to reproduce the initial prototype conditions of no self-scouring outfall in the embayment. The first of two model calibration tests (Test#1&2) simulated different wave heights generated in Lake Ontario during a high water level condition. To calibrate the model, the Green Road embayment was reproduced to include a correctly scaled circular outfall pipe, similar to what was found in the field. The embayment wing walls were reproduced using coarse aggregate on both sides of the outlet pipe to simulate the larger armour stone found in the prototype.

The bathymetry was moulded to recreate the bottom contours found in Lake Ontario and wave guides were set along the sides to absorb/reflect the waves. For reasons discussed in the design section, bamboo sheet piles were initially used in the model to reproduce the steel sheet piles in the field. The bamboo sheet piles were later replaced with rulers that were 30 cm long, these provided greater rigidity and had sufficient vertical length to prevent movement during wave action. Figure 4.1 is a photo of the completed embayment area prior to wave bombardment.

Figure 4. 1 The figure illustrates the condition of the outfall prior wave simulations.

Calibration of the physical model was no simple task because it involved the acquisition of prototype data which was time consuming and expensive. The usage of sophisticated sensitive monitoring equipment was a great necessity when calibrating the model because minor fluctuations in such parameters as water depth were detected effortlessly by five wave staff gauges. To judge whether or not the model had been calibrated it was necessary to remember and understand the fundamental problem plaguing the Green Road coastal outfall: the outfall is susceptible to sediment blockage. Because water waves have been transporting sediment into its circular opening, the outfall's hydraulic discharge capacity is reduced, rendering it ineffective.

To determine whether or not the model has been calibrated properly, seven benchmark indicators were used to validate the model:

- Model wave height from wave gauges
- Water level in model corresponding to prototype water level data of Lake Ontario
- The occurrence of sediment deposition in the embayment
- No serious bed deformations (by visual inspection)
- Development of a littoral current (movement of water ball)
- Formation of a berm in front of the embayment and east and west seawall
- Still photos and video footage

It was not possible to monitor all the benchmark indicators since a considerable amount of time was allotted to taking photos and making videos for future analysis. For example, during the first simulation a ball measuring 1.5 cm was constructed of candle wax and trace amounts of silica sand and was allowed to travel through the water column during wave simulations. From a technical point of view, if the model was built according to prototype specifications the wax ball should translate from east to west during simulations due to a littoral current that was believed to be transporting a bulk amount of sediment in the field. The wax ball was only used for the initial simulations to arrive at the appropriate time duration to run the model. Table 4.1 was prepared to compare both theoretical and model test results that were produced by numerical calculations and actual data that was gathered during calibration Test 1 and 2 from recording devices. (Similar tables are used throughout this chapter to list the parameters that were used in each simulation.

Table 4. 1 Model and prototype values for calibration test 1 and 2.

Test #	Proto Wave Period, s	Calculated Model Wave Period, s	Actual Model Incident Wave Period, s	Proto Wave height, m	Calculated Model Wave Height, m	Actual Incident _{Avg.} Model Wave Height, m	Proto Water Level, m	Model Water Level, m
Calibration Test # 1	4.0	1.033	1.03	1.43	0.095	0.08	75.07	0.387
Calibration Test # 2	6.0	1.549	1.55	1.95	0.13	0.183	75.07	0.387
Comments -No self-scouring outfall structure was used for the calibration test -Waves from the northeast were simulated								

Wave staff gauges were strategically positioned throughout the model to record the wave height at a particular location (exact location in *Design Section*). A total of 5 capacitance type wave staffs were used to study and record the instantaneous surface fluctuations of the water level. Shown below in Figures 4.2 and 4.3 are two sets of sinusoidal line graphs that display up to 5 seconds of the model simulation for a particular test. The fluctuations in water levels, and times they occurred, can be read on the Y-axis and X-axis, respectively. Each wave staff can be uniquely identified by a distinct colour and number that corresponds to its particular location on the model bed. In order to validate the performance of the model the wave period was calculated for one wavelength by measuring the horizontal distance between peaks and was later compared with measured buoy prototype data to authenticate the test. The legend can also be used to identify a specific wave staff.

Figure 4.2 First calibration test for a period of 1.03 seconds.

Figure 4. 3 Second calibration test for a period of 1.55 seconds.

The performance of the self-scouring outfall was judged on the basis of how well prototype conditions were simulated during model test runs by analyzing graphs, tables and photographic footage. Presented below are a series of photographs that illustrate the condition of the embayment prior to wave activity (Figures 4.4 & 4.5) and after both calibration tests (Figures 4.6 & 4.7).

Figure 4. 4 Initial condition of embayment.

Figure 4. 5 Clean outfall pipe.

Figure 4. 6 Condition of embayment after tests.

Figure 4. 7 Sediment inside outfall pipe.

4.2.1 Analysis and Discussion of Calibration Results

A series of two calibration tests were performed to ensure the model was functioning within acceptable parameters. According to wave data presented in Table 4.1 and shown in Figures 4.2 and 4.3, Calibration Tests 1 and 2 produced an average incident wave height of 8 cm in Test 1 which showed good agreement with the calculated model wave height value of 9.5 cm. The wave staff data collected from calibration test 2 indicated that the model produced an average incident wave height that exceeded the stipulated testing parameters listed in Table 4.1. It was believed by researchers that deviations in wave height for the first set of calibration tests were acceptable because wave height data from the computer had to be quickly analyzed, and if necessary, adjusted at the wave generator controls to compensate for incorrect wave heights. As the project progressed it was observed that the accuracy of the generated model waves also increased.

Upon draining the wave basin at the end of Calibration Tests 1 and 2, a noticeable translation of the embayment rocks was visually observed (apparent when comparing Figures 4.6 to Figure 4.4). By analyzing the wave height data found in Figure 4.3 and recorded video footage, it was concluded that the large waves produced during the second calibration test and the long duration of the model simulation were the primary causes for the movement of the embayment rocks. It should be stated that in the field, embayment rocks were never observed to move. To alleviate this problem a solid mass of rock was

formed that resisted repetitive wave bombardment. Future model tests (#2-12) employed the use of landscaping blocks that were thought to be incapable of movement because of their heavy mass and large surface area that was affective at dissipating approaching wave energy. These large blocks were deemed to be acceptable in representing armour stones found in the prototype because movement was no longer observed.

According to visual observations from the model simulations, photographic footage and numerical data from recording devices, researchers were optimistic that the calibration test was a great success because the model behaved exactly according to the program input parameters. To elaborate, all generated waves established a minimum of two wavelengths, scoured bottom sediment on the movable-bed and later broke on the shoreline to scour additional sediment. It was initially feared that waves would break too early; to alleviate this concern, calculations were carried out in the design section to ensure propagating waves broke in a specific depth of water close to the shoreline, similar to prototype conditions.

Additionally, it was discovered during simulations that model waves adjacent to the east seawall were smaller than western seawall waves (shown in Figures 4.2, 4.3 and corresponding wave staff locations shown in Figure 4.8). Smaller waves produced near the eastern seawall were attributed to the shallow water depth that prevails in that region and also the bottom friction incurred when the bottom of the wave orbital scrapes the model lake bottom as the propagating wave front travels towards the shoreline. Waves to the west of the model are in deep water and arrive at the self-scouring outfall unscathed by significant bottom friction which explains their greater height.

Figure 4. 8 A Schematic demonstrating the location of wave staffs relative to deep and shallow water (not to scale).

4.2.2 Simulation of a Littoral Current

During the field survey it was accepted with a reasonable degree of certainty that a littoral current was transporting a bulk amount of sediment from an eroding bluff to the east into the Green Road embayment located to the west. The sediment shown in Figure 4.9 was found on the east beach ($D_{50} = 2.4\text{mm}$) and similar sediment was also unexpectedly discovered in the embayment area ($D_{50} = 1.8\text{-}2.3\text{mm}$). This was strong evidence that sediment in the embayment originated from the east and also served to validate all previous assumptions that were made about the origin of the sediments.

Figure 4. 9 Sediment found near the embayment is similar to east beach sediment shown above.

It was thought to be a reasonable assumption that a littoral current was transporting eroded bluff sediment from east to west, and wave driven currents pushed some of this sediment onto the east beach shore and eventually into the embayment, this would seem to explain the similar sediment found on both the east beach and in the embayment. A probable explanation for the large quantity of sediment in the embayment was the shape of the coastline. From aerial photography (Lower left-hand corner of Figure 3.12) it was evident the embayment was situated on a convex shoreline, vulnerable to wave refraction, the bending of water waves. Figure 4.75 illustrates the bending of the wave crest as the water depth changes from shallow to deep. The wave rays show how the variation in water depth focuses water waves to transport sediment into the embayment.

Figure 4.75 Changes in depth cause waves to bend.

The sediment transport and deposition phenomenon that was observed in the field can be extrapolated to explain why the sediment in the physical model is moving from one location to another. During the first set of calibration tests, wax balls were positioned at strategic locations in the wave basin on the east side of the movable-bed and were allowed to move freely during model tests. The results of this small experiment helped to validate the model, for upon draining the basin it was observed that the majority of the wax balls were situated on the west side of the model near wave staffs 4 and 5. The movement of the wax balls and the occurrence of sediment transport from east to west serve to support the presence of a littoral current.

4.2.3 Wave Absorbers and Basin Dynamics

According to reviewed literature it is often desirable to use a large physical model or one that approaches the size of the prototype to facilitate the study of similar model hydrodynamics that would occur in nature. Unfortunately, large size models were not found to be cost effective because they required considerable space and time to construct. A 1:15 scale undistorted model was designed and constructed for the current project to simulate waves propagating over a rigid and movable-bed into the Green Rd. Embayment. Often, in nature, wave absorbers are located along coastal regions to dissipate wave energy and are not positioned perpendicular to incoming waves, the latter being a necessity in our model. Wave absorbers were placed along both sides of model to promote wave absorption or reflection to train water waves.

During the calibration tests it was observed that all absorber-bound waves were effortlessly assimilated and channelled back towards the wave generator without interfering with basin dynamics (Figure 3.70). In some cases, research suggests that a wave absorber can change the wave dynamics and consequently affect the sediment transport process if the model area is too small. This type of behaviour was not observed during model simulations and is not cause for concern. In the current model, wave absorbers were positioned close to the wave generator (on the fixed-bed) and were turned on their side so that no waves would be absorbed but rather reflected. The reason for this change was simple; by analyzing wave data produced in Figure 4.2 it was clear that waves passing wave staff 1 were significantly higher than waves passing over wave staff 5, by turning the wave absorber on its side resulted in a larger wave height in the region of wave staff 5. The idea of turning a wave absorber into a wave reflector helped to reduce the dissimilarities between the two wave staff measurements. It should be stated the objective of this study was not to analyze basin dynamics but rather to confirm that the basin dynamics were not deviating from prototype data, such an occurrence would call to question the credibility of the self-scouring outfall data.

4.2.4 Sediment Transport

A key benchmark used to analyze if the model was functioning correctly was to observe sediment movement and to see if grains were moving from east to west and ultimately into the embayment area. Upon closer inspection of Figure 4.7 it was observed that movable-bed sediment was on the invert of the of the SSO outlet pipe. The occurrence of sediment entering the embayment was interpreted as a positive sign as it suggested the following:

- Sediment geometry and density were compatible with the model
- Turbulent wave activity was capable of scouring and lifting the bottom sediment into the water column for transport
- Moulding of mobile-bed bathymetry was acceptable for model simulations
- Wave absorbers were not interfering with basin hydrodynamics

4.2.5 Duration of Model Simulations

It was decided by researchers that an acceptable model run duration for calibration test-1 would be 20 minutes based on the model area and amount of available movable-bed sediment. However, after noticing that a bulk amount of sediment was being transported between time intervals 2-10 minutes it was decided to revise the model duration to a maximum of 10 minutes. Table 4.2 outlines the main advantages of reducing the model duration time. When a model period of 1.03 and 1.55 seconds was used for a 20 minute duration too many waves were produced and the model bed suffered serious bed-deformations. These bed deformations had the potential to seriously affect the wave and sediment dynamics within the study zone. Long model durations are advantageous in some respects because they provide more time to study complex processes under investigation (e.g., the scouring and sediment removal capability of the self-scouring outfall). But long model durations should be used with caution as the problem of a long time duration can seriously incapacitate a hydraulic model.

To clarify, as model waves propagate towards the shoreline they scour the bottom sediment generating sediment in the water column which eventually makes its way to the

embayment area or contributes to the formation of a berm. When too many model waves are created, the amount of sand that is being removed from the model lakebed is not being replaced fast enough, leading to serious bathymetric deviations which are not representative of prototype conditions. Furthermore, sediment that is transported into the embayment has nowhere to go and subsequently accumulates in the embayment area or is re-circulated in the model. It can be said with confidence that failure to maintain a sediment balance in the model when using long time durations will cause poor model performance for reasons that were outlined above.

Table 4. 2 Wave production for various model run durations.

Duration, min	Period, sec	Number of Waves Produced
20	1.03	$= \left(\frac{20 \text{ min} * 60 \text{ sec}}{1.03} \right) \approx 1165$
20	1.55	≈ 774
10	1.03	≈ 583
10	1.55	≈ 387

4.3 Performance of the Self-Scouring Outfall

The self-scouring outfall structure that is to be constructed in the field utilizes complex geometry to scour sediment by using a combination of wave up-rush slopes, converging walls and steep outlet channels to promote self-cleansing with respect to sediments (Mohammad et al. 2006). The following sub-sections that are to follow attempt to analyze the performance of the self scouring outfall by qualitatively observing if the outfall has sediment on any of its slopes and whether or not the sediment distribution pattern in front of the SSO has been modified. A checklist of self-scouring outfall components was compiled to ensure that researchers systematically analyzed the performance of the outfall. The checklist included examination of:

- Sediment accumulation on the east and/or west outlet channel at the entrance
- Sediment accumulation on the east and/or west converging down slope
- Sediment accumulation on the east and/or west wave run-up slopes
- Undermining of structure

- Sediment removal/deposition in the sheet pile area
- Sediment deposition on the grouting area in front of the east and/or west outlet channels

A photo of SSO-I (Figure 4.10) has been provided below to illustrate the various components that were listed above.

Figure 4.10 Various components of SSO-I have been labelled to facilitate greater understanding.

4.3.1 Performance Review of Self-Scouring Outfall

Twelve model test runs were carried out at the Canada Centre for Inland Waters, two of which were calibration tests. Table 4.3 is presented on the following page, details of each test can be quickly found in this table.

Table 4. 3 Test description for physical model investigation of self-scouring outfall.

Test NO.	Test Description	PROTO WAVE		MODEL WAVE		Water Level	Comments
		Height	Period	Height	Period		
1.	Model with existing conditions (high water level). Waves from NE direction. No SSO in the model. (Test date: Wed Nov 23, 2005)	1.20m 2.75m	4.0s 6.0s	0.080m 0.183m	1.03s 1.55s	75.07m	Model calibration for tests
2.	Bathymetry and coastline as test 1. SSO in the embayment and high water level in the model. Waves from NE direction. (Test date: Tues Dec 20, 2005)	1.64m	4.0s	0.109m	1.03s	75.07m	Effects of wave climate on SSO
3.	Bathymetry and coastline as test 1. SSO in the embayment and high water level in the model. Waves from the NE direction. (Test date: Thurs Dec 22, 2005)	2.31m	6.0s	0.154m	1.55s	75.07m	Effects of wave climate on SSO
4.	Bathymetry and coastline as test 1. SSO in the embayment and low water level in the model. Waves from NE direction. (Test date: Tues Dec 13, 2005)	1.32m	4.0s	0.088m	1.03s	74.52m	Effects of wave climate on SSO
5.	Bathymetry and coastline as test 1. SSO in the embayment and low water level in the model. Waves from NE direction. (Test date: Wed Jan 4, 2006)	1.85m	6.0s	0.123m	1.55s	74.52m	Effects of wave climate on SSO
6. Run B	Bathymetry and coastline as test 1. SSO in the embayment and low water level in the model. Waves from the NE Direction. (Test date: Tues Jan 17, 2006)	1.95m	6.0s	0.13 m	1.55s	74.52m	Effects of wave climate on SSO.
7a. Run A	Bathymetry and coastline as test 1. No SSO in the embayment and high water level. Waves from the NE Direction. (Test date: Thurs Jan 19, 2006)	1.43m	4.0s	0.095 m (≈0.1 m)	1.03s	75.07 m	-Test is 10 mins. -Existing existing Embayment with metal sheet piles.
7b. Run A	Bathymetry and coastline as test 1. No SSO in the embayment and high water level. Waves from the NE Direction. (Test date: Thurs Jan 19, 2006)	2.1 m	6.0s	0.138 m	1.55s	75.07m	-Test is 10 mins. -Existing existing Embayment with metal sheet piles
8. Run C	-Low water, High waves New SSO (Test date: Tues Jan 24, 2006)	2.0m	6.0s	0.133 m	1.55s	74.52 m	Test Duration: = 10 minutes
9. Run D	-High water, High waves New SSO (Test date: Thurs Jan 26, 2006)	2.0m	6.0s	0.133 m	1.55s	75.07 m	Test Duration: = 10 minutes
10.	-Low water, High waves New SSO New Bathymetry (Test date: Mon Feb 6, 2006)	2.03m	6.0s	0.135 m	1.55s	74.52 m	Test Duration: = 10 minutes
11.	-High water, High waves, New SSO New Bathymetry (Test date: Thurs Feb 9, 2006)	2.28m	6.0s	0.152 m	1.55s	75.07 m	Test Duration: = 10 minutes
12. (demo)	-Low water, High Waves New SSO New Bathymetry (Test date: Tues Feb 14, 2006)	2.12m	6.0s	0.141 m	1.55s	74.52 m	Test Duration: = 10 minutes -Shell blast in sheet pile area

Table 4.3 is a summary of critical data such as wave period, wave height and water level that can be used for comparing different parameters during the twelve test runs. It is important to remember that not only were the hydraulic parameters like water level and wave height altered throughout the testing regiment but a modified SSO design was also tested to gauge its superiority. (Table 4.4 is similar to Table 4.3 in many respects because it explicitly states the amount of sediment that has been deposited on the critical slopes of the SSO for all the tests carried out to date).

Table 4. 4 Sediment deposition on critical slopes of SSO-I and SSO-II.

Test #	Water level, m	Wave height, m	Wave Period, sec	Sediment coverage on E. Slope, %	Sediment coverage on W. Slope, %	Sediment on E. down slope, Y/N	Sediment on W. down slope, Y/N	Sediment on E. outlet channel, Y/N	Sediment on W. outlet channel, Y/N
Test Conditions for waves approaching from the northeast direction									
1 - Calibration	-	-	-	-	-	-	-	-	-
2:SSO-I	75.07	0.109	1.03	<25	25-50	N	N	N→Partially clean	Y
3:SSO-I	75.07	0.154	1.55	<25	Negligible	N	N	Y→Thin layer	Y
4:SSO-I	74.52	0.088	1.03	0	<25	N	Y-small amount	N	Y
5:SSO-I	74.52	0.131	1.55	0	50	N	Y-trace amount	Y	Y→full
6:SSO-I	74.52	0.123	1.55	<25	50	Y-trace amount	Y- trace amount	N	Y-full
7 - Calibration	75.07	0.145	1.55	-	-	-	-	-	-
8: SSO-II	74.52	0.138	1.55	0	25-50	N-trace amount	Y-small amount	N- trace amount	N
9: SSO-II	75.07	0.133	1.55	0	0	N	N	N	N
Test Conditions for waves approaching from the east direction									
10: SSO-II	74.52	0.135	1.55	50	50-75	Y-full	Y-trace amount	Y-thin layer	Y-thin layer
11: SSO-II	75.07	0.152	1.55	0	25-50	Y-full	N	Y-thin layer	Y-thin layer
12: SSO-II	74.52	0.141	1.55	25-50	25	Y-full	N	Y-thin layer	Y-trace amount

The following two subsections discuss the same SSO design but with regulated hydraulic parameters. The second calibration test precedes these two subsections and discusses the performance of SSO-I and SSO-II in terms design modifications.

4.3.2 SSO-I Simulation Analysis for Test 2 and 3

As is customary in this study, a comparison table has been prepared for each set of tests that outlines the input parameters that are to be used for the simulation, listed in Table 4.5 are the governing parameters for this model run.

Table 4.5 Input parameters and actual model results from test 2 and 3

Test #	Proto Wave Period, s	Calculated Model Wave Period, s	Actual Model Incident Wave Period, s	Proto Wave height, m	Calculated Model Wave Height, m	Actual Incident _{Avg.} Model Wave Height, m	Proto Water Level, m	Model Water Level, m
Test # 2	4.0	1.033	1.03	1.43	0.095	0.109	75.07	0.387
Test # 3	6.0	1.549	1.55	1.95	0.13	0.154	75.07	0.387
Comments -Self-scouring outfall-I was installed in the embayment and was subjected to low and high waves from the northeast direction								

The self-scouring outfall design shown in Figure 4.11 was used for tests 1-6 and was considered to be similar to a design used in the Eastern Beaches region in the City of Toronto (shown in the *Design Section*).

Figure 4.11 Hydrodynamics of the SSO-I

The self-scouring outfall shown in Figure 4.11 is a symmetric structure that has no continuously moving parts, electrical motors or chemical storage bays for such things as chemical coagulants to aid in floc formations. The structure utilizes a combination of slopes and converging walls to facilitate sediment removal. Ideally, as water waves

propagate towards the structure they have an opportunity to travel up the 30 degree wave run-up slope where they may break and dissipate their energy, or continue to travel up and over the wave run-up slope and deposit/scour sediment on the converging down slope. Due to the steep grade of the converging down slope all water is channelled to the outlet channel where it can exit harmlessly into the lake. In some instances sediment that has deposited on the converging down slope may not scour because of a high sediment depth or non turbulent conditions in the zone. The design of SSO-I allows water waves to freely travel into both the east and west outlet channel where they may either deposit or scour sediment on the channel floor. Additionally, storm water that has been discharged from the existing outlet pipe can be accelerated down a slope and later divided by a flow splitter where it can exit via a perforated grate and scour sediment on the converging down slope.

Currently, there is no known available literature that critiques the performance of the prototype self-scouring outfall in the City of Toronto. SSO-I incorporated a similar design to the SSO outfall in the City of Toronto and the advantage of this research is its applicability in gauging the performance of that particular outfall design to conclude whether or not SSO-I requires modification. This data will subsequently be used to modify SSO-I to increase sediment removal efficiency. Simulations involving SSO-II will later be looked at for their efficiency. Upon completion of each simulation, approximately one-half-day is spent remoulding the bed for future tests. It is imperative the mobile-bed bathymetry be kept the same throughout the testing phase to ensure the same wave heights are always generated for their respective hydraulic parameters. As is customary, wave staff data will be presented for all testing regiments to ensure the bathymetry has been moulded correctly and that no unwanted deviations in sediment composition have occurred. A deviation in sediment composition would entail hard sand. The occurrence of this situation equates to sand that has been left standing for far too long and has now formed a tough exterior at the surface because of atmospheric drying.

Often, wave action will cause sediment scour to occur on the mobile bed which causes certain particle sizes to settle before others sizes when basin draining occurs. This non-uniform sand composition that results on the bed is undesirable because only a

specific range of particles sizes will be observed attacking the SSO during testing. To alleviate this problem, sediment was deliberately disturbed by churning and mixing it prior to each test so that a wide distribution of sediment particle sizes composed the top layer of the mobile bed. Figures 4.12 and 4.13 show the wave staff graphs for tests 2 and 3.

Figure 4. 12 Wave height data for a 1.03 second period.

Figure 4. 13 Wave height data for 1.55 second period.

According to the wave staff data presented in Figures 4.12 and 4.13, the model generated satisfactory size waves to test the self-scouring outfall and showed good agreement with prototype waves. The wave spectrum shown in Figures 4.12 and 4.13 represented a small portion of the 10 minute testing period but was enough to provide an idea of whether or not the model was simulating prototyping waves adequately. Typically, when analyzing the wave data, the 3-5 minute mark was adopted as a starting point to isolate a small spectrum for analysis. Often, it is necessary to give the sediment and the wave energy time to reach an equilibrium before wave height measurements can commence (The entire wave spectrum for a 10 minute test can be found in Appendix A).

Wave staff data from tests 2 and 3 were not compared to calibration tests 1 and 2 wave data because researchers and technicians were resolving minor problems during this period. However, future wave staff data presented thereon will take advantage of data presented in this particular subsection to gauge differences or similarities in wave heights and periods.

The photos that are to follow in Figures 4.14 and 4.15 show the condition of the outfall prior to and after wave simulations. In some instances, graphs were constructed to show the movement of sediment from one region to the other in the model.

Figure 4. 14 Test 2 – Prior to wave activity.

Figure 4. 15 Test 3 – Prior to wave activity.

The initial conditions of SSO-I for tests 2 and 3 have been presented above to adequately demonstrate that the tests were conducted on an equal footing. There was no visible sediment on any of the slopes and the east and west grouting area was sediment free. The

sheet pile area was brought to grade each time a new test was conducted via an elevation peg positioned in the middle of the sheet pile area.

Figure 4.16 Test 2: After model test.

Figure 4.17 Test 3: After model test.

Figure 4.18 Test 2: Berm formation

Figure 4.19 Test 3: Berm Formation.

The water level of 75.07 m (high water level) was kept the same for wave simulations of tests 2 and 3 and only the wave height was varied (input parameters are shown in Table 4.5). When observing Figures 4.16 and 4.17, there was a greater amount of noticeable sediment deposition on the west wave run-up slope in Figure 4.16 when compared to the same slope in 4.17. It was apparent that a larger size wave was capable of scouring a greater amount of sediment on the wave up-rush slopes.

The problem associated with this scenario was two fold: first because a larger more turbulent wave has the potential to undermine the structure by causing greater erosion to occur at the toe of the structure, and secondly: larger waves can deposit a greater amount of sediment on various slopes. Although inspection of the east grouting area in both Figures 4.16 and 4.17 suggested sporadic sand deposition on the top surface,

for the most part this area was found to be relatively clean. By once again carefully scrutinizing the west grouting areas in Figures 4.16 and 4.17 it was observed that there was sediment deposition on both areas but significantly more on the west grouting area in Figure 4.17, sediment deposition was attributed to the higher wave height. It was noticed that the converging down slope in Figure 4.16 was clean; the lack of sediment on the slope is most likely a result of the less turbulent conditions produced by the low wave height of 10.9 cm. Trace amounts of sediment were faintly visible on the east and west converging slope of Figure 4.17, but from a technical standpoint the deposition is believed to be negligible.

The east outlet channel shown in Figure 4.16 was found to be very clean and only trace amounts of sediment were visible on the same outlet channel in Figure 4.17. Furthermore, a thick layer of sediment was visible on the west outlet channel of Figure 4.16 and only a trace amounts were visible on the same type of slope in Figure 4.17. The reason for this scenario was simple: because a larger more turbulent wave has greater energy, it can scour a greater amount of bottom sediment not only on the movable-bed but also on the surfaces of the SSO. In all instances, the larger generated waves deposited elevated amounts of sediment in the embayment. These larger waves had a greater potential to scour sediment on smooth surfaces like the grouting area and SSO surfaces.

As the wave basin was slowly drained at the conclusion of the test, a large berm that traversed the width of the model became visible in both Figures 4.18 and 4.19. The berm formation resulted from inbound waves that broke and scoured sediment close to the seawall. It is believed scoured sediment was transported back to the wave pool due to the beach slope. This sediment then formed a raised ridge called or a berm. It was observed throughout the testing regiment that breaking waves in the eastern half of the model transported a large amount of sediment by wave and current actions. Numerical measurements were taken of sediment height and locations along the width of the model, plotted graphically in Figures 4.20 and 4.21 to show the sediment distribution.

Figures 4.20 and 4.21 validate the model by numerically confirming that currents and waves were transporting sediment from the east to the west. The elevation measurements shown in Figure 4.20 and 4.21 were measured on the centreline of the

berm that formed and were compared to the initial elevation measurements at the same location before testing had begun. It can be seen in Figure 4.20 that a great deal of sediment was deposited between 2.5m - 5.5 m which corresponds to the location of the SSO study area. In some instances the elevation at specific locations on movable-bed increased by 10-15 cm as a result of berm formation!

Figure 4. 20 The movement of sediment along the coast.

Figure 4.21 The movement of sediment along the coast.

Additionally, confirmed numerically by Figure 4.21, larger waves did transport a greater amount of sediment into the study area when compared to a smaller one. The observations that were made from still photos and numerical data in Figures 4.20 and 4.21 can be extended to explain certain events that occurred in the field. To elaborate, on August 31 2006, the remnants of Hurricane Katrina swept across Lake Ontario dumping over 100 mm of rain and reeking havoc with maximum sustained winds of 55 km/h (Environment Canada, 2005).

The powerful winds that were generated from this storm event caused large waves to develop in Lake Ontario near the study area which ultimately resulted in significant sediment deposition in the Green Road embayment (see Figures 3.4 and 3.5). It was suspected that powerful waves generated from the remnants of Hurricane Katrina were responsible for transporting a large volume of sediment into the Green Rd. embayment and blocking the outfall opening by more than 50%. It is the author's opinion, that although it is not possible to control the development of storm cells, it is possible to control outfall blockage. The ability to control outfall blockage is the motivation behind the City of Hamilton's considerable investment to test the viability of the SSO design.

4.3.3 SSO-I Simulation Analysis for Test 4 and 5

For comparison purposes Table 4.6 was created to facilitate greater understanding when analyzing still images from model simulations. Similar to the previous subsection, a series of figures were presented to outline the following:

- The condition of the outfall prior to wave bombardment (Fig. 4.22 and 4.23)
- The condition of the outfall after the simulation had ended (Fig. 4.26 and 4.27)

Table 4. 6 Input parameters and actual model results from test 2 and 3

Test #	Proto Wave Period, s	Calculated Model Wave Period, s	Actual Model Incident Wave Period, s	Proto Wave height, m	Calculated Model Wave Height, m	Actual Incident _{AVG} Model Wave Height, m	Proto Water Level, m	Model Water Level, m
Test # 4	4.0	1.033	1.03	1.32	0.095	0.088	74.52	0.35
Test # 5	6.0	1.549	1.55	1.97	0.13	0.131	74.52	0.35
<p style="text-align: center;">Comments</p> <p>-Self-scouring outfall-I was installed in the embayment and was subjected to low and high waves from the northeast direction during low water level</p>

Figure 4. 22 Test 4 – Prior to wave activity.

Figure 4. 23 Test 5 – Prior to wave activity.

The condition of the outfall prior to wave activity was shown in Figures 4.22 and 4.23. Every effort was made to mould the sheet pile area to the same depth each time by using a combination of rulers and levels to promote greater accuracy. The east and west grouting areas were cleaned by removing all large quantities of sediment that accumulated on the surfaces during the previous tests. The small narrow channels and slopes within the SSO were also been cleaned by a make-shift mop that was specially

designed to fit into the small channels. The sheet pile material was changed in Test 5 as metal rulers were more representative of the sheet piles used in the field because they could be easily adjusted in the vertical direction. Sheet piles used in Test 5 were also incapable of vertical and horizontal movement during wave activity which was advantageous. Wave staff data for Tests 4 and 5 were presented in Figures 4.24 and 4.25 because it was the belief of the author that numerical data that was accurately recorded by each wave staff should be presented to help validate the results of each physical model test.

Figure 4.24 Wave height data for a 1.03 sec period.

Figure 4.25 Wave staff data for a 1.55 sec period.

With respect to Figures 4.24 and 4.25, the water levels in both tests were kept the same (74.07 m – low water level), only the wave height was varied. According to the wave height data that was presented in Figures 4.24 and 4.25 suitable size waves were generated for both tests which closely matched their prototype counterparts. The data produced by each wave staff had a great deal of credibility because all the wave staffs produced similar data which meant that model bathymetry was acceptable and no serious deviations in model wave height data occurred. It should be noted that due to differences in model water depth in shallow and deep regions some wave staffs may record wave heights that differ by as much as 50%.

Figure 4.26 Test 4 – After wave simulation.

Figure 4.27 Test 5 – After wave simulation.

The simulation results obtained from tests 4 and 5 did not deviate greatly from results obtained during tests 2 and 3. Upon closer observation of Figure 4.27 it was seen that a lower water level produced less erosion at the toe of the structure when compared to a higher water level shown in Figure 4.17. In terms of sedimentation on wave run-up slopes, it can be said decisively that a larger wave has the ability to transport a greater amount of sediment onto the west wave run-up slope. In general, it was observed from previous tests that large waves transport more sediment into SSO area as suggested by Figures 4.26 and 4.27.

A low wave height was used in Test 4 (0.088 m) to simulate a 1.32 m wave in Lake Ontario. There was no sediment deposition on east outlet channel in Figure 4.26, which is likely due to the self-scouring capability of the outfall. The eastern outlet channel in Test 5 had a thin layer of sediment on its surface that was most likely deposited by turbulent flows that predominate on eastern half of the structure. The east converging down slope in Figure 4.26 was for most part clean but the west converging slope in the same figure showed significantly more sediment deposition. Contrastingly, the east and west converging down slope in Figure 4.27 showed little sign of significant sediment deposition. It was possible that large waves generated in Test 5 were able to scour most of the sediment that had accumulated on the converging down slope with the exception of the location where the converging down slope transitions with the outlet channels; this could potentially explain the clean state of the channels.

The west grouting area in Figure 4.27 experienced a greater amount of sedimentation when compared to the same area in Test 4. However, the east grouting area in both Tests 4 and 5 saw considerably less sedimentation than their western counterparts. At this point in the study it is safe to hypothesize that larger model waves regardless of the water level have greater potential to deposit more sediment into the embayment area. Figure 4.28 vividly portrays the sediment erosion capability of the waves from Test 5 and same phenomenon has been reproduced graphically by numerical data in Figure 4.29.

Figure 4. 28 Wave simulated during test 5 has eroded sediment exposing the elevation pegs.

Figure 4.29 The movement of sediment from east to west.

From visual observations and the numerical data presented in Figures 4.28 and 4.29, it is evident that a great deal of sediment was transferred from the shallow zone to deep region of the model in front of the SSO by a strong littoral current. The process described above was very similar to what occurs in the field and helps to add credibility to the model simulation. Data for Test 6 was not presented because the results from this test seem follow the general pattern of sediment erosion and deposition that was discovered in all the test results from the previous subsections. However, Table 4.4 does present a brief summary of the observations that were recorded in Test 6.

4.4 Calibration Test II

A second calibration test was carried out using phase 1 geometry. The wing wall material composed of granular coarse aggregate was replaced with rigid landscaping bricks to simulate the larger embayment bricks in the field shown in the Figure 3.1 of the *Design Section*. Calibration Test 2 was the prelude to the modified SSO design which

draws from performance deficiencies of SSO-I and incorporates critical suggestions from hydraulic experts in the field. Calibration Test 2 was composed of two parts: Part 1 tested the new embayment material and Part 2 involved the installation and testing of SSO-II. Table 4.7 clearly outlines the input parameters that were used for each test along with recorded wave height data shown in Figure 4.30.

Table 4. 7 Input parameters and actual model results from test 2 and 3.

Test #	Proto Wave Period, s	Calculated Model Wave Period, s	Actual Model Incident Wave Period, s	Proto Wave height, m	Calculated Model Wave Height, m	Actual Incident _{Avg} Model Wave Height, m	Proto Water Level, m	Model Water Level, m
Test # 7a	4.0	1.03	1.03	1.58	0.095	0.105	75.07	0.387
Test # 7b	6.0	1.55	1.55	2.18	0.13	0.145	75.07	0.387
Comments -For test 7a, new wing wall material was used and small waves were simulated during high water level -For test 7b, SSO-II was installed in the embayment and was subjected large waves during high water level

Figure 4. 30 Wave height data for a period of 1.03 seconds.

In order to accommodate the new embayment material a major demolition and reconstruction took place. Figures 4.31 and 4.32 clearly depict the progress of the work at various stages.

Figure 4.31 Construction of new embayment.

Figure 4.32 Newly completed embayment.

Figure 4.31 shows the removal of all coarse aggregate and the installation of survey lines to locate geographical points in the embayment. The implementation of the new embayment material required a great deal of work but it was the opinion of researchers that this new type of material was by far more representative of the prototype conditions found in the field. Figure 4.32 shows the completed embayment prior to wave activity. The armour stone geometry was not altered and the neither was the mobile-bed slope approaching the outer most sheet piles. A smooth transition was created from the outlet pipe to the outermost sheet piles to reduce the likelihood of premature wave breaking due to abrupt changes on the slope. Figures 4.33 and 4.34 effectively show the condition of the outfall after the second calibration test.

Figure 4.33 New embayment after wave activity.

Figure 4.34 Erosion and deposition of sediment.

It can be seen in both Figures 4.33 and 4.34 that sediment has been eroded and deposited inside the embayment and in front of the sheet piles. Closer observations of Figure 4.34 shows the formation of a berm in front of the sheet pile area and significant

sediment erosion inside the sheet pile area. According to Figure 4.30, a wave height averaging 10.5 cm was generated, the erosion characteristics and berm formation of this test were indicative of previous tests that were conducted by researchers. It was observed that the east grouting area was relatively clean whereas the west grouting area had significant sediment. It would appear that incident waves propagated towards the structure and broke inside the sheet pile area where they carved a crater and deposited sediment in front of the sheet pile area. In some respects the outermost sheet piles may be acting like wall capable of reflecting waves transporting sediment.

The outlet pipe shown in Figure 4.33 had very little sediment deposited on the invert when compared to the outlet pipe invert that was shown in the first calibration test. A greater amount of internal pipe sedimentation during the first calibration test was most likely a result of strong waves that displaced the coarse aggregate embayment rocks in turn allowing waves to propagate unopposed into the outlet pipe and depositing an elevated amount of sediment. With respect to Figure 4.33, the embayment was a scaled down replica of the prototype, where it was reasonable to assume the amount of time it took for sediment to accumulate into model outlet pipe was a function of the Froude Condition. Table 4.8 expands on the parameters of the Froude condition.

Table 4.8 Duration of model simulations according to Froude condition.

t_p – duration of prototype storm event, min	t_m – duration of model storm event, min
120 min = 2 hours	$t_m = \frac{t_p}{\sqrt{N_L}} = \frac{120}{\sqrt{15}} \cong 31 \text{ min}$
360 min = 6 hours	$\cong 93 \text{ min} = 1.6 \text{ hours}$
720 min = 12 hours	$\cong 186 \text{ min} = 3.1 \text{ hours}$

According to the Froude Condition, if researchers wished to simulate a two hour prototype storm the model would have to be run for 31 minutes. To simulate a 12 hour storm event the model would have to operate for 3.1 hours. When the remnants of Hurricane Katrina swept across Lake Ontario the duration of the storm event was about 12 hours. This system deposited a significant amount of sediment into the embayment during this time period rendering the outfall ineffective (Figure 3.5). It is impossible for

researchers to simulate a 12 hour prototype storm in the model because sediment cannot be continuously supplied and removed during the 3.1 hour model simulation. It was difficult to use a prototype event to calibrate the model (similitude by calibration) because the model could not function sufficiently long; therefore, it was decided to confirm whether or not sediment had deposited in the embayment during model simulations. Upon completion of the calibration test, sand depths were measured along three parallel lines that stretched from the outlet pipe to the outer most sheet piles. Figure 4.35 shows the location of sediment depth measurements and Figure 4.36 is a graph that was produced using data from the depth measurements.

Figure 4.35 The depth of sediment along the West, Centre and East line.

Figure 4.36 The depth of sediment as a function of distance.

Figure 4.36 illustrates the transport of sediment from the mobile-bed area into the embayment. Clearly, it can be seen that a significant amount of sediment was deposited close to the outlet pipe between 0-58 cm when compared to the original sand depth (blue line). By tracing the elevation of the centre line it can be seen that a great deal of sediment has been eroded from the sheet pile area by wave breaking. This sediment was potentially transported to the outlet pipe or towards the wave generator. Once again, measurements confirm the validity of the model. To recap, the calibration test consisted of two tests which utilized two different wave heights shown in Table 4.7 and the water level was kept the same.

4.5 Testing and Analysis of SSO-II for Test 5 and 8

A new self-scouring outfall shown in Figure 4.37 was designed and installed in the model Green Road embayment area for testing. The critical modifications made include:

- New east and west outlet orientation.
- New east and west blunt surface in front of the east and west grouting area.
- Shallow converging down slope grade.
- Installation of a removal trapezoidal gate to deflect incoming water mass.
- Steep outlet channels.

Figure 4. 37 New critical components in the self-scouring outfall.

A series of two sets of tests were presented in the following subsections to compare SSO-I and SSO-II, these tests employed identical hydraulic parameters. Table 4.9 has been created below to show the input parameters that were used for each test.

Table 4.9 Input parameters for Tests 5 (SSO-I) and Test 8 (SSO-II).

Test #	Proto Wave Period, s	Calculated Model Wave Period, s	Actual Model Incident Wave Period, s	Proto Wave height, m	Calculated Model Wave Height, m	Actual Incident _{avg} Model Wave Height, m	Proto Water Level, m	Model Water Level, m
Test # 5	6.0	1.549	1.55	1.97	0.095	0.131	74.52	0.35
Test # 8	6.0	1.549	1.55	2.07	0.13	0.138	74.52	0.35
Comments -SSO-I and SSO-II were tested under identical conditions to judge which design was superior -waves were from the northeast								

The wave staff data presented in Test 5 was not included in this section in order to avoid redundancy, however, the wave height data for Test 8 is presented below.

Figure 4.38 Wave height data for a 1.55 second period.

According to the wave height data presented in Figure 4.38 it would appear that an adequate wave height was generated. Once again, readers are cautioned that differences in water depth and abrupt changes in slope can affect the height of the wave. The initial conditions of SSO-I and SSO-II were presented in Figures 4.39 and 4.40 prior to wave activity.

Figure 4. 39 Test 5 – Initial condition – SSO-I. Figure 4. 40 Test 8 – Initial condition – SSO-II.

Figures 4.39 and 4.40 were shown above to indicate the condition of the outfall prior to wave bombardment and to ensure the bathymetry in the sheet pile area was moulded to specifications. Every effort has been made in the lab to effectively duplicate the prototype conditions, in particular, the sediment elevation in the sheet pile area. It is unclear what kind of affect incorrect sheet pile sediment bathymetry would have on the model. It is also worthy to note that although this error is quite small, the consequences of incorrect sheet pile sediment height could be greater in field because of scaling relationships. A list of possible scenarios of incorrect sediment height in the *sheet pile area* was created.

- High sediment level: Disproportionate amount of sediment on the converging down slope from waves that overtop the trapezoidal gate
- High sediment level: A greater amount of sediment deposition outside the sheet pile area
- Low sediment height: Undermining of structure

Photos were taken of the SSO-I (Figure 4.41) and SSO-II (Figure 4.42) after the wave simulation ended, they are shown below.

Figure 4.41 Test 5: SSO-I, after test.

Figure 4.42 Test 8: SSO-II, after test.

The above tests were compared using the same hydraulic parameters and only the design of the outfall was altered. When comparing the east wave run-up slope in both Figures 4.41 and 4.42 it was clear that both surfaces were relatively clean, however the same slope in SSO-II (right-hand side) appears to have more sediment erosion at the toe of the structure. The west wave run-up slope on SSO-II has visibly less sediment on it when compared to the same slope in SSO-I.

As the propagating water mass moves towards SSO-II and travels up the east and west wave run-up slope the *trapezoidal gate* forces the water mass to break and reflect back out to the lake harmlessly with most of the sediment it was transporting. As a comparison, any water mass that travelled up the *east* or *west* wave run-up slope on SSO-I was free to enter the structure and deposit a bulk amount of sediment on the wave run-up slopes, converging down slopes or outlet channels because there were no means of stopping the water mass from entering the bowels of the structure. The implementation of the trapezoidal gate prevented the water mass from entering the structure and this gate has the added advantage of being removed to facilitate cleaning as necessary.

Figures 4.43 and 4.44 are two plan view photos that were prepared to illustrate the condition of the converging down slope and outlet channels. The sediment distribution pattern on the slopes of SSO-I is quite typical with regard to the design. Nearly all the tests which used the SSO-I design experienced sedimentation on the west outlet channel and the west converging down slope, not to mention in a variety of other places as well.

The new self-scouring outfall design employed the installation of a trapezoidal gate that was used to deflect the incoming water masses. The trapezoidal gate shown in Figure 4.44 helped to reduce amount of sediment that could have potentially entered the outfall and plugged the outlet channel. Although the trapezoidal gate played a pivotal role in reducing sediment intrusion into the outfall, it can also be said that the orientation of the outlet channel is one of the most innovative design implementations on this outfall.

Figure 4.43 Test 5 – Plan view of SSO-I.

Figure 4.44 Test 8 – Plan view of SSO-II.

Rotating the outlet channels by 110 degrees prevented northeast propagating waves from directly entering the structure and depositing sediment on the slopes of the outlet channel. According to Figure 4.44 this design modification has undoubtedly paid off because the east and west outlet channels in this figure were found to be relatively clean. The converging down slope area was also very clean is attributed to the installation of the trapezoidal gates and new outlet orientation.

By examining Figure 4.42 it was evident the area in front of the east and west grouting had significantly less sediment deposition. This occurrence was a result of incoming waves being reflected off the blunt east and west outlet channel surface shown in Figure 4.37. All waves that were reflected off this surface or similar ones will erode sediment on surfaces perpendicular to it. It can be said with a great deal of confidence that due to the orientation of the SSO structure and angle of the approaching wave front the eastern section of the SSO has greater access to more turbulent flows when compared to the western half of the structure. The occurrence of turbulent flows in the eastern half of the structure will become more evident in the photos presented in the next subsection.

4.5.1 Testing and Analysis of SSO-II for Test 3 and 9

A model simulation was carried out for Test 9 using a high water and high wave height to test the design of a newly constructed outfall. The results of this test were compared to Test 3 which utilized the same hydraulic parameters as Test 9 but used the SSO-I design. The input parameters from these two tests have been neatly organized in Table 4.10 for easy viewing. Also, wave staff data for Test 9 was shown below in Figure 4.45.

Table 4. 10 Input parameters for Tests 3 (SSO-I) and Test 9 (SSO-II).

Test #	Proto Wave Period, s	Calculated Model Wave Period, s	Actual Model Incident Wave Period, s	Proto Wave height, m	Calculated Model Wave Height, m	Actual Incident _{Avg} Model Wave Height, m	Proto Water Level, m	Model Water Level, m
Test # 3	6.0	1.549	1.55	2.31	0.095	0.154	75.07	0.387
Test # 9	6.0	1.549	1.55	2.00	0.13	0.133	75.07	0.387
<p style="text-align: center;">Comments</p> <p>-SSO-I and SSO-II were tested under identical conditions to judge which design was superior</p> <p>-waves were from the northeast</p>

Figure 4. 45 Wave height data for a 1.55 second period.

The wave height data presented in Figure 4.45 indicated that a slightly smaller wave had been produced for this test when compared to the average incident wave height generated in Test 3. This was not a significant problem because according to buoy data from Lake Ontario larger waves bombard the shoreline during large storm events but these large waves only occur during extreme conditions. All wave data presented in Figure 4.45 seemed to follow the general trend that has been observed in previous wave data graphs. The similarity of the wave data graphs presented thus far indicates the moulding the lake bathymetry after each test was accurately executed enabling technicians to generate model waves flawlessly. Once again, presented below are a series of photos that outline the condition of the outfall prior to wave driven sediment attacks (Figures 4.46 and 4.47).

Figure 4. 46 Test 3: SSO-I – Initial condition.

Figure 4. 47 Test 9: SSO-II – Initial Condition.

Figures 4.46 and 4.47 indicate both the sheet pile areas and surrounding bathymetry have been moulded to prototype bathymetry specifications. This type of precision is necessary in order to ensure model similitude governs. Any deviations in bathymetry between the two tests would alter how waves encroach on the structure and this occurrence would taint model results and analysis. Figures 4.48 and 4.49 illustrate the accumulation of deposited sediment on the slopes of SSO-I and SSO-II after waves have impacted the structure.

Figure 4.48 Test 3: SSO-I After wave activity.

Figure 4.49 Test 9: SSO-II After wave activity.

Figure 4.50 Test 3: SSO-I – Plan view.

Figure 4.51 Test 9: SSO-II – Plan view.

The new self-scouring outfall design (SSO-II) continues to outperform SSO-I. When observing Figure 4.48 it can be seen there was a trace amount of sediment at the toe of the east and west wave run-up slope whereas the same slopes in Figure 4.49 (SSO-II) had no sediment. In term of erosion, both SSO-I and SSO-II exhibited significant erosion at the toe of the eastern half of the structure. Erosion has occurred at the foot of the structure because large waves (13.3 – 15.5 cm) were generated during the tests and also, the orientation of the SSO relative to the approaching wave front does invite scour as waves have tendency to break in this location.

The east grouting area in front of SSO-I was relatively clean when compared to the western grouting area. One possible explanation for this event was that incoming wave fronts violently traverse over the east grouting area before breaking on the structure, in the process, the wave mass scours much of the sediment on the surface of the east grouting. The west grouting in SSO-I (Figure 4.48) had significantly more sediment on its surface because propagating water masses break and loose much of their energy on

the eastern half of the structure and as a result sediment a greater amount of sediment is deposited on the west grouting area.

The east and west grouting surfaces in Figure 4.49 was relatively clean as incoming waves reflect and retreat back out to the lake via the new east and west blunt surfaces created by changing the orientation of the outlet channels. Upon closer observation of Figure 4.49, it can be seen there was less sediment on the east grouting surface compared to the west. This phenomenon was attributed to the location of where the waves were breaking, the orientation of the outfall and the ability of the wave front to scour the sediment on the grouting surface.

Comparing the colouration of the converging down slope in Figures 4.50 and 4.51 it can be seen that SSO-II was doing a significantly better job of preventing sediment from penetrating the entrance of the trapezoidal gate and outlet channels. It is also apparent that all flow surfaces in Figure 4.51 (SSO-II) are clean and free of sediment. Conversely, SSO-I in Figure 4.50 has sediment deposition on the surface of the west outlet channel and a thin layer on the east.

It is important that the outlet channels be kept relatively clean during normal operation because deposited sediment from storm events becomes stagnant and dry when left in the outlet channel for a long amount of time, it is difficult for waves and sewer flows to scour this deposited sediment, potentially leading to a build up of sediment or plugging of the outlet channel. The likelihood of this build-up plugging event is quite small, however all possible worse case scenarios were tested and the outfall performed quite well in each.

4.5.2 Hydrodynamic Forces inside the SSO

Up to this point, waves from northeast have been generated to determine how well the self-scouring outfall performed when subjected to different hydraulic parameters. The next subsection focuses on the modified self-scouring outfall, waves were now generated from the eastern direction and new bathymetry was also used. Before continuing, a group of photos shown in Figures 4.52- 4.54 have been assembled to

provide the reader with an idea of the types hydrodynamic forces that affect the internal geometry of the self-scouring outfall during various simulations.

Figure 4. 52 Test 9:SSO-II – Outfall hydrodynamics. Figure 4. 53 Test 9:SSO-II – Flows within outfall.

Figure 4. 54 Test 3:SSO-I Flows within outfall.

The intense turbulent flows within SSO-I in Figure 4.54 resulted from a high water level (75.07 m) and high wave height (15.4 cm). If one looks closely at this figure, it can be seen that a red circles has been drawn around an area that receives very little cleaning during wave activity. The reason for this is simple, as the propagating wave front makes contact with the blunt vertical surface in the middle of SSO structure the water mass is divided, creating a pocket on the west wave run-up slope that receives little or no turbulent flows. The east wave run-up slope seems to be very active and this fact may help or hinder it since the cleansing potential is a function of many things like wave

height, water level, amount of sediment, duration of the storm event and direction of the approaching wave.

It can be seen in Figure 4.52 that as the wave front approaches, the east grouting surface receives a great deal of cleansing with respect to intense turbulence from the water mass. The west grouting surface on the other hand receives slightly less turbulent flows due to the angle of the approaching wave front and also because the water mass is divided by the blunt vertical surface in front of the SSO. The east wave run-up slope, converging down slope and east outlet channel in Figure 4.53 will always experience turbulent flow regimes because this section was always bombarded by intense wave activity.

The trapezoidal gate and new outlet orientation in Figure 4.53 seems to be preventing significant water flows from entering the western half of the structure which reduces the likelihood of sedimentation on surfaces. Once again, looking closely at Figure 4.53 it can be seen that waves have the ability to overtop the trapezoidal gate and deposit/scour sediment. In the opinion of researchers this scenario was beneficial because as overtopping waves travel over the trapezoidal gate they have a certain amount of potential energy which can be used to scour sediment that has accumulated on the shallow grade of the converging down slopes and steep outlet channels. Additionally, sediment that has accumulated on the converging down slope surface can be scoured by sewer flows leaving the perforated grate opening.

4.6 Phase II Layout of Physical Model

Water waves generated in Lake Ontario can arrive into the embayment area from the northeast, east-northeast and eastern direction. It was concluded in the design section that waves approaching from the northeast and east would be reproduced during the study because waves from these two directions can push sediment directly into the existing embayment and outfall. Phase 2 (waves from east direction) involved the creation of new lake bathymetry to encompass a wider shoreline and building of a new seawall and embayment. A schematic drawing of the new model orientation (waves from eastern

direction) is shown in Figure 4.55 and the existing phase 1 bathymetry is also shown in same figure (wave from northeast).

Figure 4. 55 New orientation of SSO (shown in blue).

Figure 4.55 illustrates the old layout of the model coastline in violet (wave from NE) and the new layout (wave from E) has been superimposed on the same figure in a blue colour by rotating the entire coastline and SSO by 45 degrees. A great deal of optimization work concerning the vertical and horizontal placement of the coastline and SSO had to be completed before construction could begin. In order to maximize the amount of available movable-bed area researchers had to reduce the length of the fixed bed to an absolute minimum value but at the same time still make sure there was enough fixed-bed length for wave generation. The optimization work was primarily done by hand using tracing paper and bathymetric diagrams of Lake Ontario and was later perfected on AutoCAD. Figures 4.56 – 4.58 depict various stages of construction and critical components like wave absorber/reflectors that were used to test the self-scouring outfall.

Figure 4.56 Construction of new layout.

Figure 4.57 Completed bathymetry.

Figure 4.58 Position of wave absorbers during a wave simulation.

It is important to note that embayment geometry, sheet pile placement and the layout of armour stones were reproduced in phase two using schematic drawings that

were drafted for the phase one layout (waves from the northeast direction). A minimum fixed-bed distance of 3 m was used to promote stable wave generation. During model runs this distance seemed to be adequate because waves near the seawall were not prematurely breaking close to wave generator but were observed as propagating towards the SSO structure.

4.6.1 Results of New SSO-II Orientation for Test 10 and 11

The newly constructed bathymetry was used to test the modified self-scouring outfall (SSO-II) for waves propagating from the east. Similar to all other tests, a table has been included below that displays all the parameters that were used in this test (Table 4.11).

Table 4. 11 Input parameters for Tests 10 (SSO-II) and Test 11 (SSO-II).

Test #	Proto Wave Period, s	Calculated Model Wave Period, s	Actual Model Incident Wave Period, s	Proto Wave height, m	Calculated Model Wave Height, m	Actual Incident _{Avg} Model Wave Height, m	Proto Water Level, m	Model Water Level, m
Test # 10	6.0	1.549	1.55	2.03	0.095	0.135	74.52	0.35
Test # 11	6.0	1.549	1.55	2.28	0.13	0.152	75.07	0.387
Comments								
-SSO-II was tested using phase 2 bathymetry and was tested by producing waves from the east								

For the most part, Tests 10 and 11 use similar hydraulic parameters and the only difference being the model water level. A high wave height was used for both tests to promote intense sediment transport in the vicinity of the SSO. Figures 4.59 and 4.60 illustrate wave height data graphs that have been used throughout this study to verify the average incident model wave height. It should be pointed out that only four wave gauges were used to record data for the phase two layout because model space was limited.

Figure 4.59 Test 10:SSO-II -Wave height data for a period of 1.55 seconds.

Figure 4.60 Test 11:SSO-II - Wave height data for a 1.55 second period.

According to Figures 4.60 and 4.61 it appears that the appropriate wave height was being produced for both tests. It is important to note that variations in water depth at certain

wave staff locations may cause the machinery to record different wave heights as a result of wave shoaling. The initial conditions of SSO-II are presented below in Figures 4.61 and 4.62 for Test 10 and 11.

Figure 4. 61 Test 10 –Initial conditions.

Figure 4. 62 Test 11 – Initial conditions.

Images of the self-scouring outfall in Figures 4.61 and 4.62 show the model prior to Tests 10 and 11 wave simulations. Displaying the outfall prior to wave activity is important when comparing the initial condition of the outfall's critical components to the tested outfall subjected to wave activity. Researchers visually inspected the outfall for defects before the simulation began; the absence of problems allowed the simulation to commence. Figures 4.63 and 4.64 shown below illustrate the condition of the outfall upon draining the wave basin.

Figure 4. 63 Test 10 – Low water level – HWH Figure 4. 64 Test 11 – High water level – HWH

The photos above show remarkable evidence of how the water level and wave height can contribute to sedimentation. In Test 10, a low water level and high wave height (HWH) were used to simulate wave conditions in Lake Ontario and it can be seen by looking at

the red perforated lines in Figures 4.63 and 4.64 that the east and west wave run-up slope are more than 50% covered with sediment. Whereas in Test 10, the east wave run-up slope was clean and the west wave run-up had significant sedimentation. It is believed that a high water level and a high wave height have helped to cleanse all the slopes in Test 11. It should be pointed out that a slightly higher wave was generated in Test 11 and from knowledge of previous tests with similar hydraulic parameters it can be concluded that this combination of a "high wave" and "high water level" has a greater scour potential. It was also noticed that minor erosion occurred at the toe of both the east and west wave run-up slope, more so in Test 11 because of the higher water level.

The observations made during Test 11 for SSO-II can be extended to explain why the prototype outfall plugs more often during certain times of the year. The Lake Ontario water level data presented in the *Design Section* indicated that Lake Ontario is at its highest water level between the months of April and August. It is reasonable to assume storm events that generate large waves in Lake Ontario during these particular months will undoubtedly push sediment into the embayment and outfall.

The model sheet pile area shown in both Figures 4.63 and 4.64 was for the most unscathed by wave activity. A possible reason for this could be that sheet piles provided a physical barrier by preventing eastern approaching waves from eroding the sediment within the sheet pile area. By examining the eastern and western grouting area in Figure 4.63 it can be seen there was significant sediment deposition on both slopes. It was quite possible that because the water level was low, the scouring potential was reduced because the incoming wave mass broke and dissipated its energy before reaching the grouting area. The most plausible explanation for sediment on the grouting is however likely attributed to the fact that approaching waves from the east must travel over the east and west grouting to leave to the study area, whereby much of their sediment is deposited.

After testing, it was noticed in Figure 3.64 the eastern grouting area had less sediment deposition than the western grouting area. This was mostly likely due a higher water level which enabled fully developed waves to reach the SSO. However, as the wave front encountered the eastern most sheet piles, much of the energy of the wave front was lost causing the water mass to lose speed and momentum resulting in the deposition of sediment on the western grouting surface.

A series of photos shown in Figures 4.65 and 4.66 were compiled during model testing to illustrate the direction and ferocity of the wave front as it made contact with the structure.

Figure 4. 65 Wave front approaching SSO.

Figure 4. 66 Sheet piles breaking wave front.

Figure 4. 67 Wave front has wrapped itself around the sheet pile area.

Figures 4.65 – 4.67 clearly explain what happens to the wave front as it encounters the eastern half of the structure and passes through the sheet pile area. Much of the energy of the approaching wave front was lost when the first barrier of sheet piles forced the water mass to break and deposit/scour sediment. Figures 4.68 and 4.69 provide a bird's eye view of the SSO condition after Test 10 and 11.

Figure 4. 68 Test 10: Low water level – HWH.

Figure 4. 69 Test 11: High water level – HWH.

It can be said conclusively by comparing Figures 4.68 and 4.69 that a high water level has a greater ability to scour sediment on eastern outlet slope for reasons already discussed. The west converging down sloped in each figure appears to be relatively clean because the following reasons:

- The wave front is approaching from the east.
- The diffuser assemble and grates provide a physical barrier for incoming water.
- The trapezoidal gate acts as barrier by preventing waves from bypassing the wave run-up slope.

Two observations can be decisively made about waves approaching from the east:

1. The east converging down slope will most likely be covered in sediment
2. The west converging down slope will remain relatively clean as it is well protected

Figures 4.70 and 4.71 below show the formation of a berm in front of the SSO and respective seawalls.

Figure 4. 70 Test 10 – Berm formation.

Figure 4. 71 Test 11 – Berm formation.

The formation of a berm in both Figures 4.70 and 4.71 is good indication that the model was designed well because past testing results yielded the same occurrence. It has also been reported to researchers that individuals who have lived in the vicinity of the Green Road outfall for a number of years have confirmed the occurrence of a berm formation.

4.6.2 The Usage of Shell Blast for Test 11

To avoid redundancy, wave staff data for the final test (Test 12) is not presented as results were identical to Test 10. All observations made in Test 10 concerning the sedimentation and erosion on slopes can be used to explain what occurred in Test 12. A summary table below outlines the input parameters used during the test (Table 4.12).

Table 4.12 Input parameters for Tests 12 (SSO-II)

Test #	Proto Wave Period, s	Calculated Model Wave Period, s	Actual Model Incident Wave Period, s	Proto Wave height, m	Calculated Model Wave Height, m	Actual Incident _{Avg} Model Wave Height, m	Proto Water Level, m	Model Water Level, m
Test # 12	6.0	1.549	1.55	2.12	0.095	0.141	74.52	0.35
Comments								
-Shell Blast was placed in the sheet pile area and in front of the armour stone for observations								

It was suggested by other researchers to use particles with different characteristics such as density and geometry. Unfortunately, only one type of model sediment had to be used throughout the study to avoid mixing of study sand with experimental sand. To elaborate, if study sand and experimental sand were to mix it would be impossible to separate the two sediments in the study basin because of their size and quantity. All of the future tests would not be reflective of prototype conditions because experimental sand would have been mixed in with the “study sand” producing tainted results of the SSO performance.

Therefore, it was decided by researchers that a new type of sediment was going to be used for the final test so that mixing of study sand and experimental sediment sand would not be an issue. The material used was called Shell Blast. It has been used primarily as a sand blasting cleaning tool in the aircraft industry. Some analysis work was carried on the Shell Blast to gain a better understanding of how it would behave in

water. The material was found to have a specific gravity of 1.45 which was slightly lower than construction sand ($G_s=2.5$), the sieve analysis chart of the Shell Blast material was shown in Figure 4.72.

Figure 4. 72 Sieve analysis for Shell Blast material.

The Shell Blast was found to have a D_{50} of 0.35 mm which was slightly higher than construction sand ($D_{50} \approx 0.2$ mm). The Shell Blast was placed around the armour stone and sheet pile area; results of the simulation were shown in Figures 4.73 and 4.74.

Figure 4. 73 Placement of Shell Blast in model.

Figure 4. 74 Shell Blast after the simulation.

When the wave simulation was completed researchers attentively searched for the Shell Blast in and around the model area but were unable to find any trace of it right away. After excavating about 25-100mm of sediment in the sheet pile area (dependent on location) a thin red layer of fine particles was visible, but was completely submerged under the model sand. Only a small amount of Shell Blast was tested because it must first be submerged in water for many hours prior to use. It would appear very little of the Shell Blast was scoured from the sheet pile area because the majority of it was buried under the model sand.

It was hard to get an idea of the mobility of this type of material unless a larger quantity was to be used. It should be pointed out that small trace amounts of the Shell Blast were found at the most western part of the model, likely transported by the initial wave crests that propagated through the structure.

4.7 Major Findings of the Physical Model Study

The following section briefly summarizes all the major findings of this project into five subsections so that important results and observations could be quickly retrieved. These five subsections consist of the calibration test, SSO-I testing, SSO-II testing, phase-II model layout and a summary of the overall model performance.

4.7.1 Results of Calibration Test

- The small gravel rocks used initially were replaced with larger and heavier bricks that resisted movement when subjected to wave energy.
- A model duration time of 20 minutes was found to be too long and was later changed to 10 minutes to avoid movable-bed deformations.
- The calibration tests were deemed to be successful because sediment entered the embayment and was deposited on the pipe invert, similar to prototype conditions.
- It was concluded the amount of the sand deposited in the embayment was a function of how well researchers were able to mould the movable-bed such that it resembled the contours of the prototype bed. Any deviations in model bed contours such as slopes, ridges or depressions could result in too much or too little sediment transported into the embayment. It was also believed deposited sand in the embayment was a function of the model run time in association with the Froude time scale, water level, and wave height and corresponding wave period.

4.7.2 Testing Results of SSO-I

- The converging down slopes and the outlet slopes were optimised because the outlets were too shallow to break incoming waves and could not sufficiently accelerate the outbound waves and outfall discharge. Also, the converging down slope was too steep as a result of the outlet slopes.
- The “Open Concept Design” of SSO-I allowed for wave propagation directly into the outfall structure, resulting in higher than average sediment deposition.
- The western outlet channel was always covered by sediment, whereas, the eastern outlet channel was clear 60% of the time.
- The western and eastern converging down slopes were sediment free 40% and 80% of the time, respectively.
- The east wave run-up slope was clean more than 75% of the time, whereas, the west wave run-up slope was only clean 50-75% of the time.

4.7.3 Testing Results of SSO-II

- Converging down slopes and outlet channels were optimized by increasing the slope of the outlets and decreasing the grade of the down slopes. Modifying the slopes help to reduce sedimentation on surfaces.
- Implementation of the newly designed trapezoidal gate significantly reduced sedimentation on the converging down slope and was also effective at deflecting the water mass away from the structure.
- Outlet channels were rotated by 110° clockwise and anticlockwise with respect to the east and west outlet channels in order to prevent sedimentation by using the newly constructed outlet wall to block incoming waves.
- Newly created trapezoidal gates and blunt surfaces prevented significant sediment from accumulating in front of the structure because wave fronts were deflected by the vertical walls.
- Model simulations suggested that the SSO should be built on a strong foundation to prevent undermining by turbulent wave activity.
- East and west outlet channels were sediment free 100% of the time.
- East and west converging down slope had no sediment deposition 100% and 75% time, respectively.
- East wave run-up slope was always clean; however, the west wave run-up slope was covered with sediment 50-100% of the time.

4.7.4 Phase-II Model Layout

- New orientation of the outfall allowed generated waves from the east to enter directly through the eastern SSO outlet to deposit a significant amount of sediment on the east converging down slope.
- Both east and west outlets were covered with a thin layer of sediment.
- Western converging down slope was free of sediment 100% of time, whereas, the eastern converging down slope was always covered by sediment.

- Western wave run-up slope was shown to be more vulnerable because it was always covered by sediment; however, the eastern wave run-up slope was sediment free more than 50% of the time.

4.7.5 Overall Performance of the Physical Model

The overall performance of the hydraulic movable-bed model was excellent. Generated waves were effortlessly produced by a piston-type Kelk wave generator that worked flawlessly throughout the model study. It was observed throughout the experiment that model boundaries did not affect the performance of the model by generating unwanted waves in the wave basin. The formation of a berm and sediment deposition in the embayment confirmed that sediment transport was occurring in the model. In general, SSO-II exhibited superior sediment handling performance over SSO-I because of the newly implemented design modifications. The degree of scour on model surfaces and in the vicinity of the SSO is believed to be a direct function of water level and wave height. It was observed that a high water level in conjunction with a high wave height created a great deal of sediment scour and erosion within the SSO and in front of the structure. The self-scouring outfall prevented sediment from entering the existing outlet pipe and was also able to scour the vast majority of sediment that accumulated on its surfaces.

Chapter 5 Mathematical Modeling of a Self-Scouring Outfall

The primary objective of this thesis is to design and construct a physical model to test and analyze a new type of structure called a self-scouring outfall. Upon completion of the design/build phase of the project, it was discovered that a computational fluid dynamics program called FLUENT could be used to examine the complex hydrodynamics within the outfall structure. It was subsequently decided to use FLUENT software to simulate flows leaving the outfall because this process was beyond the scope of the physical model study. Only SSO-II was to be modelled in FLUENT because it performed exceptionally better than its predecessor and was selected by the City of Hamilton to be constructed in the Green Road embayment. FLUENT's powerful mathematical algorithms were used to provide additional insight into the velocity patterns within SSO-II and to discern areas of turbulent flow and pressure variations.

5.1 A Brief Overview of FLUENT

FLUENT is a powerful computational fluid dynamics (CFD) program that is capable solving the most difficult fluid flow and heat transfer problems that reside in complex geometry. FLUENT software develops its competitive edge from its friendly user interface, robust equation solver and unstructured mesh flexibility. The program was written in C computer language to enhance solver controls, memory allocation and data structures. FLUENT software is what is known as a solver, meaning that model geometry must be created using a separate program (Gambit) and then exported to FLUENT for analysis. Gambit is the pre-processor program that provides a graphical user interface for users to design structures for analysis. Gambit also utilizes a large array of meshing tools that can be used to apply meshing schemes to both two-dimensional and three-dimensional structures.

5.2 Applicability of Fluent To This Study

FLUENT is a leading edge software used throughout the industry by many of the most prestigious companies in the world. This program has applications in every engineering discipline and can be used to simulate such things as a simple flow in pipe, waves breaking on the legs of an off-shore oil platform (Figure 5.1) and air flow over a Ferrari Formula-1 racing car (Figure 5.2).

Figure 5.1 Wave impacting oil platform
(Courtesy of FLUENT)

Figure 5.2 Aerodynamics of racing car.
(Courtesy of FLUENT)

The self-scouring outfall utilizes complex geometry to promote sediment scour on its surfaces. The SSO to be constructed in the field will be installed inline with an existing 1.8 m diameter outfall. In order to test the 1:15 scale SSO-II model, a known flow was discharged from a circular outlet pipe in the SSO model and was allowed to flow through a bed of sediment in the structure so that researchers could observe the scouring potential generated by sewer flows. A replica of the SSO model was later created in FLUENT and same flow with no sediment inside the structure was allowed to flow through the structure for comparison purposes. The observations from this test provided an indication of the scouring performance of the various slopes in the structure by observing the velocity vectors. FLUENT software was used to:

- Check the flow velocity in different areas of the SSO.
- Identify turbulent areas in the SSO.
- Identify areas of low and high pressure.

5.3 Description of Gambit

Gambit allows designers to create mesh models for CFD problems and many other scientific applications that can be used in almost any technical application. All user input is via a graphical user interface (GUI) that is straightforward and easy to use. Gambit is a powerful geometry program with a lot of functions, but this section will only briefly outline what was done to arrive at the meshed SSO model. In order for the SSO to be meshed it had to be hand measured so that all wall boundaries and slope elevations would be known. These points were later entered into Gambit via the GUI and were connected to form rigid edges and faces. When all faces in the SSO were created they were all united to form a single volume. A picture of the completed wire frame volume model is shown below in Figure 5.3.

Figure 5.3 Wire frame volume model of SSO prior to meshing.

5.3.1 Meshing the SSO Model

The meshing algorithm used in Gambit has a wide arsenal of meshing schemes that can be employed to mesh fairly multifaceted geometries. The SSO is very complex structure that utilizes a diffuser assembly to divide the stormwater flow after it has been discharged from the existing outlet pipe. Reproducing the diffuser structure on the computer was difficult, for it was observed during the initial meshing trials that the Gambit compiler halted computations during the analysis of the diffuser section due to tolerance issues. This problem was most likely attributed to the curvaceous nature of the diffuser run-up slope. It was decided to simplify the geometry of the diffuser by making its lines more linear, thereby easier for computer to process. The user has the ability to choose from a list of meshing schemes but it is often best to allow Gambit to choose the most appropriate meshing scheme as it scrutinizes the structure for flaws that could render other meshing schemes inappropriate. The TGrid meshing scheme was used, a scheme known as a hybrid mesh because it not only consists of tetrahedral elements but also other shapes as well. Figure 5.4 below illustrates the tetrahedral meshing scheme.

Figure 5. 4 Tetrahedral meshing scheme.

There are three characteristics in the Gambit meshing algorithm that dictate what type of mesh will be generated:

1. **Elements** – shape of the elements used to model the volume
2. **Type** – Meshing algorithm for the overall mesh pattern
3. **Smoother** – A smoothing algorithm

It was believed that due to limitations in computer memory, the first meshing scheme should consist of the fewest possible elements in order to reduce memory usage and to expedite the mesh generation calculation process. An interval size 0.016 was used to yield an unstructured mesh volume of 19,424 elements. Typically, a smaller interval size produces a denser mesh with more volume elements. Shown in Figures 5.5 – 5.7 are the mesh elements for SSO-II in the x, y and z direction's, respectively.

Figure 5. 5 Mesh elements for x-direction.

Figure 5. 6 Mesh elements for y-direction.

Figure 5. 7 Mesh elements for z-direction.

In the feasibility stage of this project it was debated whether or not the SSO model should be modeled in 2-D or 3-D. It was later decided that only a three-dimensional model would do an effective job of simulating the flows in the outfall because of steep elevation drops the flow experiences and winding channels that are inherent to the structure. A three dimensional model approximates the future SSO prototype with excellent geometric similarity and it is likely that CFD simulations should produce results that are comparable to the physical model results. Figure 5.8 illustrates a completely meshed SSO.

Figure 5. 8 Meshed SSO consisting of 19,424 elements.

According to Gambit documentation, it is advisable to assign boundary conditions to the model prior to exporting the mesh to FLUENT because this task is easier accomplished in Gambit via the GUI rather than in FLUENT. In Gambit there are two types of zone specification that govern the operation of the model: Boundary Types and Continuum Types. *Boundary types* define various characteristics of the model both at its internal and external boundaries, and *continuum types* define the internal medium (solid or fluid) which may inhabit a particular region of the model. The study outfall consists of an inlet and two outlets, the exact description of these boundaries will be discussed more

explicitly in the FLUENT section of this chapter that deals with parameter selection. The remainder of the surfaces were assigned designations in accordance with the naming procedure in Gambit and these sections will be discussed in greater detail in the FLUENT section. Upon completion of the boundary assignments the SSO was rendered to gain a more realistic appearance (Figure 5.9).

Figure 5. 9 Schematization of SSO structure.

5.4 Description of the FLUENT VOF Model

The Volume of Fluid model (VOF) is one of three multi-phase models that can be used in FLUENT to solve problems involving stratified (multiphase) or free-surface flows. The VOF model is capable of modeling two or more immiscible fluids by using a finite difference method to solve a set of momentum equations whereby the volume fraction is tracked throughout the phase or control volume. The VOF model can be used to predict the following:

- A jet break-up.
- Motion of bubbles in a liquid.
- Movement of water after a dam break.
- Steady or transient tracking of a liquid-gas interface.

The VOF model functions on the assumption that 2 or more fluids are not interpenetrating. For each new phase that is added in the model, a fraction of the computational cell is allocated to represent the new phase. As an example, if the q th fluid's volume fraction in the cell is denoted by α_q , then the following three conditions are possible (Fluent, 2003):

- $\alpha_q = 0$: the cell is empty (of the q th fluid)
- $\alpha_q = 1$: the cell is full (of the q th fluid)
- $0 < \alpha_q < 1$: the cell contains the interface between the q th fluid and one or more other fluid

Where, α_q is volume fraction of the q th phase in the control volume.

When conducting simulations, the interfaces have tendency to move between phases so FLUENT uses a volume fraction equation to track the movement of the interfaces which is based on the solution of continuity equation for the volume fraction of one or more phases. For a two phase system, where phase 2 is denoted by α_2 the equation takes on the following form:

$$\frac{\partial \alpha_2}{\partial t} + \vec{v} \cdot \nabla \alpha_2 = \frac{S_{\alpha_2}}{\rho_2} \quad [5.1]$$

Where, S is the source and $\vec{v}$ and ρ is the overall velocity vector and density, respectively.

Additionally, the density term shown in equation 5.1 is produced below for the second phase.

$$\rho = \alpha_2 \rho_2 + (1 - \alpha_2) \rho_1 \quad [5. 2]$$

A single momentum equation is solved throughout the model domain by FLUENT to calculate the shared velocity field for all phases. The momentum equation below [5.3] is dependent on the volume fraction of all phases through the properties ρ and μ . It should be pointed out however that one of the limitations of shared field's approximation is that when large velocity differences exist between phases, the velocity computed near the phase can be affected (Fluent, 2003).

$$\frac{\partial}{\partial t}(\rho \bar{v}) + \nabla \cdot (\rho \bar{v} \bar{v}) = -\nabla p + \nabla \cdot [\mu(\nabla \bar{v} + \nabla \bar{v}^T)] + \rho \bar{g} + \bar{F} \quad [5. 3]$$

Where p is pressure, μ is dynamic viscosity, g is the gravitational acceleration and F is the force vector.

The energy equation the VOF model uses is also shared among phases and it is shown below.

$$\frac{\partial}{\partial t}(\rho E) + \nabla \cdot (\bar{v}(\rho E + p)) = \nabla \cdot (K_{eff} \nabla T) + S_h \quad [5. 4]$$

According to FLUENT user documentation the VOF model treats energy (E), and temperature, T , as mass-averaged variables. This expression is shown below.

$$E = \frac{\sum_{q=1}^n \alpha_q \rho_q E_q}{\sum_{q=1}^n \alpha_q \rho_q} \quad [5. 5]$$

Where,

E_q - for each phase is based upon the specific heat of the phase and temperature

ρ and k_{eff} – effective thermal conductivity are shared by all phases

S_h – source term, contains contributions from radiation, as well as any other volumetric heat sources.

The above equations constitute the most critical elements of VOF model, other minor schemes of the VOF panel will be discussed during the selection of model parameters for model simulation.

5.5 Turbulence Model Used in FLUENT

FLUENT software provides many turbulence models for the user to choose from because no single turbulence model is universally accepted for the multitude of flow problems encountered in engineering. Simply stated, turbulent flows are characterised by fluctuating velocity fields which have tendency to mix transported quantities like momentum, energy, and species concentration causing them to fluctuate as well. When choosing between turbulence models the following characteristics should be considered:

- The physics encompassed in the flow.
- Established practice for a specific class of problem.
- Level of accuracy required.
- Computational power.
- Amount of time available for the simulation.

It is also valuable to understand the capabilities and limitations of each model. In this study the *standard k - ϵ* turbulence model was adopted to simulate turbulence because it is known in the industry as a work horse for solving engineering flow calculations involving turbulence. The *standard k - ϵ* model (Launder and Spalding, 1972) is a two equation model in which two separate transport equations are solved; that is turbulent velocity and the length scales are independently solved. The *standard k - ϵ* model is based on model equations for turbulence (k) and its dissipation rate (ϵ). The difference in the way the two variables are derived is great, for k is derived from the exact equation and the transport equation for ϵ was derived using physical reasoning. The k - ϵ model is useful only for turbulent flows, the transport equations that produced k and ϵ are shown below.

$$\frac{\partial}{\partial t}(\rho k) + \frac{\partial}{\partial x_i}(\rho k u_i) = \frac{\partial}{\partial x_j} \left[\left(\mu + \frac{\mu_t}{\sigma_k} \right) \frac{\partial k}{\partial x_j} \right] + G_k + G_b - \rho \varepsilon - Y_M + S_k \quad [5.6]$$

And

$$\frac{\partial}{\partial t}(\rho \varepsilon) + \frac{\partial}{\partial x_i}(\rho \varepsilon u_i) = \frac{\partial}{\partial x_j} \left[\left(\mu + \frac{\mu_t}{\sigma_\varepsilon} \right) \frac{\partial \varepsilon}{\partial x_j} \right] + C_{1\varepsilon} \frac{\varepsilon}{k} (G_k + C_{3\varepsilon} G_b) - C_{2\varepsilon} \rho \frac{\varepsilon^2}{k} + S_\varepsilon \quad [5.7]$$

Where,

G_k – the generation of turbulence kinetic energy due to the mean velocity gradient

G_b – the generation of turbulence kinetic energy due to buoyancy

Y_M – the contribution of the fluctuating dilatation in compressible turbulence to the overall dissipation rate

$C_{3\varepsilon}$ – constants

σ_k and σ_ε - turbulent Prandtl numbers for k and ε

S_k and S_ε – are user-defined source terms

5.6 Selection of Model Parameters

A detailed procedure outlining the selection of model parameters can be found in Appendix A8.

5.7 Overview of the SSO Flow Experiment

A physical model investigation employing the use of a fixed and mobile-bed was conducted at the NWRI-CCIW laboratory to study the performance of a self-scouring outfall. A CFD analysis using FLUENT 6.1 software was undertaken at the conclusion of the physical model tests to observe the hydrodynamics within the outfall when a flow was discharged from the existing outlet pipe. To examine this problem more closely researchers deposited a 5 cm layer of sediment on the channels and slopes within the

model SSO to investigate the scouring potential of discharged stormwater travelling through the SSO to the lake. A rubber hose with a known diameter and flow was inserted into the model SSO inlet pipe and was allowed to scour the sediment. This test was performed twice and was recorded via video camera for future observations. It was the hope of researchers to simulate this experiment in FLUENT so that flows and velocities could be reproduced. CFD software also has the added benefit of turbulence models that can identify critical zones that may need to be modified structurally to prevent concrete surface erosion.

5.7.1 Model Verification

FLUENT 6.1 is a commercial CFD software program that employs the use of sophisticated computer algorithms to manipulate partial differential equations into discrete equations which can be solved using a finite difference method. All commercial modelling software is validated to a certain degree, but it is still a worth while endeavour to verify that the model is producing the correct simulated results. During the physical model experiment, the flow discharging from the rubber hose was measured and found to be 0.37 L/s. The flows leaving both the east and west outlet are equal because the structure is symmetrical, therefore the outlet flows can be calculated by simple division of the inflow. The known inflow value from the experiment will be compared to the simulated inflow CFD value to ascertain if the two values are similar. Table 5.13 shows the results of this endeavour.

Table 5. 1 Comparison of experimental and computer modeling results

Type of Simulation	Inflow (L/s)	East Outlet (L/s)	West Outlet (L/s)
Physical model	0.3778 $\approx$ 0.4	0.1889 $\approx$ 0.2	0.1889 $\approx$ 0.2
FLUENT CFD Software	0.3088 $\approx$ 0.3	0.1544 $\approx$ 0.15	0.1544 $\approx$ 0.15
Difference (%)	18.3	18.3	18.3

It can be seen when observing the values in Table 5.13 that the computer software inflow value is not in agreement with experimental value calculated for inflow. These two values consistently differ 18.3%. Some of the possible reasons for this discrepancy are:

- Selection of incorrect flow parameters.
- Backflow is occurring at the inlet.
- Wall treatment may be necessary for roughness.
- Solution needs a larger time step so that flow can develop.

Although there is a slight deviation between the experimental inflow and CFD inflow value, it should not affect or hamper efforts to observe the flow hydrodynamics within the self-scouring outfall.

5.7.2 Velocity Hydrodynamics in the SSO

A flow was simulated leaving the inlet pipe and observations were made concerning potential “weak spots” in the SSO that could pose potential problems. Figure 5.25 shows velocity vectors throughout the SSO.

Figure 5.10 Velocity vectors within the SSO.

According to Figure 5.25, the highest velocity was calculated at the inlet pipe, as it should be. As the flow travelled through the flow spreader zone it managed to maintain a high rate of speed (≈ 0.0208 m/s or 20.8 cm/s) as it entered the converging down slope section. However, the velocity of the flow in front of the trapezoidal gate decreased because the slope was only 8% in that zone. The farthest area towards the back on the converging down slope experienced the lowest velocity (432.0×10^{-7} m/s or less than 1mm/s) or stagnation because very little flow was able to penetrate that area because either the:

1. Flow spreader directed a bulk amount of water close to the trapezoidal gate.
2. The curvature of the SSO wall towards the back of the converging down slope was acting as a sediment trap.

It is the opinion of the author that the intensity of the stormwater flow leaving the inlet pipe and travelling through the SSO is a function of storm events or snow melt. Sediment that has accumulated on the converging down slope can be scoured in one of four ways:

1. High flow volume from the inlet produced by storm events will scour sediment.
2. Waves that overtop the trapezoidal gate will have certain amount of potential energy to scour sediment on the shallow slope.
3. Inbound lake waves could enter through the outlet channel and scour sediment.
4. Backhoe may need to clean zone.

With respect to Figure 5.25, as the flow travelled down the steep outlet channel (slope = 23%) it was once again accelerated on the newly modified slope to scour sediment and direct it towards the lake.

5.7.3 Turbulence inside the SSO

The self-scouring outfall utilizes steep outlet channels, converging downs slopes and complex geometry to propel outgoing flows and hinder incoming wave masses that transport sediment. As the flow leaves the inlet pipe, and travels through the SSO structure it collides with various wall surfaces dissipating turbulent kinetic energy. FLUENT can track the turbulent kinetic energy via the *Standard k- ϵ model* selected. A series of figures have been prepared to illustrate which locations exhibited the greatest amount of turbulent kinetic energy (Figure 5.26 and Figure 5.27).

Figure 5.11 “Filled” view of turbulence in SSO.

Figure 5.12 Interior view of turbulence within the SSO.

Figure 5.26 clearly indicates the most turbulent region in SSO was the region where the flow was divided into two streams by a sharp vertical crease. As the flow exited the flow spreader zone and entered the converging down slope section it was directed into the western section of the trapezoidal gate. It was observed when the flow vacated the converging down slope and tried to enter the outlet channel a small weak vortex was created. This vortex was not large enough to scour the sediment on the top part of the outlet channel and converging down slope. Instead, as the flow entered the long steep outlet channel, it was initially reflected from side to side at the top outlet channel, the turbulence quickly subsided as the flow impinged on the lower part of the outlet channel and entered the short outlet channel before exiting into the lake.

The blunt outlet face shown in Figure 5.26 adjacent to the lake received a great amount of turbulence as the flow was accelerated down the steep outlet channel. The water mass was allowed to violently impact the outlet wall surface because of an abrupt 110 degree change in channel orientation (flow direction is shown in black dashed lines). Figure 5.27 shows the internal turbulence contours around the inlet pipe and other critical components in the SSO.

5.7.4 Pressure Surfaces within the SSO

Computational fluid dynamic software (CFD) has many added benefits to traditional hand calculations. The flexibility of CFD software allows the user to obtain valuable data in an efficient manner without consuming a great amount time. The static pressure option was selected in the FLUENT control panel to illustrate the zones of low and high pressure in Figure 5.28.

Figure 5. 13 Pressure zones in SSO.

The figure shown above was provided as “check” to make sure the area of highest pressure was at the inlet. As the flow travelled through the SSO it was reflected off many surfaces and lost a great deal of energy before it exited into the lake. The hydrostatic pressure within the SSO was dependent on density (constant), acceleration due to gravity (constant) and height of the water column. The water level and velocity were at their highest in the flow spreader and converging down slope area but as the velocity pressure

decreased (function of density and velocity) a low pressure zone was created at the mouth of the outlets.

In conclusion, computational fluid dynamics software has done an excellent job in predicting potential trouble spots in the SSO as well validating the model design. CFD software provided an added degree of flexibility to the user because many simulations can be run and various options can be selected to acquire a vivid insight into specific processes. CFD software will continue to evolve to tackle complex three-dimensional flow and sediment problems but in the opinion of the author nothing beats visual observations of sediment transport and actual water waves impacting a structure in a hydraulic model!

Chapter 6 Conclusions

6.1 Conclusions

Thousands of coastal outfalls discharge a wide variety of sewerage into bodies of water. Unfortunately, all too often a large number of these outfalls become victim to sediment intrusion because of powerful coastal inbound waves transporting bulk quantities of sediment into the outlet entrance. These waves seriously reduce the hydraulic discharge capacity of the outfall, at times even rendering it useless. Currently, the Green Rd. outfall located in the City of Hamilton requires costly sediment removal at a frequency of 2-10 times per year.

It was decided by city engineering staff the best course of action was to conduct a hydraulic model study to observe how a new type of structure called a self-scouring outfall (SSO) would perform when subjected to similar conditions as the coastal outfall in the City of Hamilton. Self-scouring outfalls utilize wave run-up slopes for wave breaking, steep outlet channels and converging down slopes to promote sediment scour. A physical modeling study was conducted at the National Water Research Institute, Canada Centre for Inlands Water laboratory to investigate how the self-scouring outfall performed when subjected to high and low water levels and small and large waves from two different directions.

An intensive field investigation was conducted to survey the site using sophisticated instruments and to gather relevant information about sediment distribution, sediment size and water depth around the Green Road embayment area so that bathymetric charts could be produced to duplicate prototype conditions during model construction. The computational fluid dynamics program FLUENT 6.1 was also utilized to study and observe the hydrodynamics of water being discharged from the SSO inlet pipe. (Simulated water flows through the inlet were not studied during the physical model study because this aspect of the study was beyond the scope of the project.) FLUENT software utilizes a number of mathematical equations to track and report flow

behaviour within a structure so that users can acquire a visual image of potential trouble spots.

It was concluded that numerical calculations would be ineffective to predict the performance of the self-scouring outfall and that a physical model investigation was the only choice to adequately study the performance of the SSO in the Green Road embayment because many three-dimensional processes like wave breaking, sediment transport, sewer flows and turbulence were all occurring at the same time in the embayment where the SSO was to be situated. Findings from the Phase-I model simulations indicated that SSO-I performed well but minor modifications were needed to reduce sedimentation on its slopes because the outfall orientation allowed waves to directly enter and deposit sediment. Observations from tests utilizing SSO-II indicated that less sediment was deposited on the slopes of SSO-II, however, minor cleaning of the outlet channels may be periodically required depending on the frequency and duration of storm events in the Green Road embayment region.

Model simulations employing the new SSO-II outlet orientation indicated less sediment was deposited in front of the SSO when using any combination of hydraulic parameters used during testing. Water waves were observed to reflect off the vertical/blunt surface produced by the new outlet orientation, deposited sediment in front of the vertical wall was scoured towards the lake. Computer simulations using the volume of fluid and Standard K- ϵ turbulence model in FLUENT 6.1 suggests the rear portion of the converging down slope may experience less sediment scour because of the geometry of the structure, certain SSO walls may need to be fortified so that excessive turbulence does not erode the concrete. It was concluded SSO-II performed exceptionally well because there was no outfall blockage during simulations and less sediment accumulated on all slopes of the SSO and in front of the structure. Furthermore, waves approaching from east as opposed to waves from the northeast have a greater opportunity to enter the structure because they can propagate directly into the structure and deposit sediment; this is thought to be worst case scenario.

6.2 Recommendations

A coastal mobile-bed physical model is able to simulate a multitude of three-dimensional processes that are all occurring at the same time. Often, it is difficult to measure exact quantities of transported sediment during model simulations and approximations must sometimes be used to arrive at quantities. The physical model constructed for this study provided excellent insight into wave/sediment interactions with the self-scouring outfall. In order to further advance the study of the SSO and to acquire data that is easier to measure it is recommended that the SSO be tested in a wave flume of sufficient length and width. The benefits are as follow:

- Accurate verification of incident wave at all approach points
- Ability to use a variety of sediment, such that model sediment has similar geometry and density to prototype sediment
- Far less sediment is required in wave flume because of a the smaller volume
- Construction of a wave flume is cheap but is a function of available lab space
- Acquisition of data is easier because of the smaller space and rectangular nature of the wave flume
- Easier to optimize SSO slopes for testing
- Mathematical relationships can be established to predict outfall cleanouts by using hydraulic parameters(water height and wave height) and sediment deposition and erosion in front of the SSO and inside the SSO structure

It is recommended that SSO-II be installed in the Green Road embayment because it prevents sediment from plugging the inlet pipe negating the need for frequent cleanouts. Occasionally, the SSO will require cleaning to remove large sediment, but it is the belief of researchers that the self-scouring outfall will significantly reduce the maintenance frequency of the embayment, thereby saving the City of Hamilton money. This thesis outlines the design process that was used to design a 1:15 scale undistorted physical model to test a new structure called a self-scouring outfall. An analysis was undertaken to ascertain how the outfall would perform in Lake Ontario when subjected to different water levels, wave heights and wave approach directions. Computational fluid

dynamic software provided another dimension of analysis that was used to identify how the outfall would perform during a certain flow condition. This thesis is intended to promote a better understanding how a self-scouring outfall helps to prevent sediment from plugging a municipal coastal outfall.

Appendixes

Appendix A1: Wave Approach Data from Buoy 45139

Wave occurrences by 30° Arcs

% Obs			Total	0 - 1m	1 - 2m	2 - 3m	3 - 4m	4 - 5m	>1m	>2m	
NNE	0 - 29	8.3%	2944	2856	87	1	0	0	88	1	
	% of Total			8.1%	0.2%	0.0%	0.0%	0.0%	0.2%	0.0%	
	% of Class								6.1%	1.3%	
NE	30 - 59	8.1%	2882	2699	181	2	0	0	183	2	
	% of Total			7.6%	0.5%	0.0%	0.0%	0.0%	0.5%	0.0%	
	% of Class								12.7%	2.5%	
ENE	60 - 89	9.0%	3178	2854	290	31	3	0	324	34	
	% of Total			8.1%	0.8%	0.1%	0.0%	0.0%	0.9%	0.1%	
	% of Class								22.5%	43.0%	
ESE	90 - 119	8.0%	2850	2600	230	12	7	1	250	20	
	% of Total			7.3%	0.6%	0.0%	0.0%	0.0%	0.7%	0.1%	
	% of Class								17.3%	25.3%	
NNW	330 - 359	8.7%	3100	3056	44	0	0	0	44	0	
				8.6%	0.1%	0.0%	0.0%	0.0%	0.1%	0.0%	
									3.0%	0.0%	
Equal % per 30°			8.3%	35450=total observations for 360°				Class total 360°		1443	79

Wave occurrences by 60° Arcs

	% Obs	Total	0 - 1m	1 - 2m	2 - 3m	3 - 4m	4 - 5m	>1m	>2m
N 60° arc	16.7%	6044	5912 16.7%	131 0.4%	1 0.0%	0 0.0%	0 0.0%	132 0.4%	1 0.0%
NE 60° arc	16.8%	5943	5554 15.7%	369.5 1.0%	18 0.1%	1.5 0.0%	0 0.0%	389 1.1%	19.5 0.1%
								27.0%	24.7%
ENE 60° arc	17.1%	6044	5503.5 15.5%	495.5 1.4%	38 0.1%	6.5 0.0%	0.5 0.0%	540.5 1.5%	45 0.1%
								37.5%	57.0%
E 60° arc	17.0%	6028	5454 15.4%	520 1.5%	43 0.1%	10 0.0%	1 0.0%	520 1.5%	43 0.1%
								36.0%	54.4%

Equal % per 30° 16.7%

Appendix A2: Refraction and Wave Breaking Calculations

Change in wave direction and height due refraction on slopes with staight, parallel depth contours including shoaling

$$g = 9.81 \text{ m/s}^2$$

Wave Period, T(s)	Water Depth, h(m)	h/gT^2	Coefficients, $k_r k_s$	Crest angle with contour	Wave Height 1, H(m)	Wave Height 2, H(m)	Wave Height 3, H(m)
6.00	30.0	0.0849		32.0	1.00	1.50	2.00
6.00	20.0	0.0566	0.9520	31.5	0.95	1.43	1.90
6.00	15.0	0.0425	0.9300	30.5	0.93	1.40	1.86
6.00	10.0	0.0283	0.8900	26.5	0.89	1.34	1.78
6.00	5.0	0.0142	0.9000	21.0	0.90	1.35	1.80
6.00	4.0	0.0113	0.9200	18.5	0.92	1.38	1.84
6.00	3.0	0.0085	0.9500	17.0	0.95	1.43	—
6.00	2.0	0.0057	1.0300	14.0	1.03	1.55	—
6.00	1.5	0.0042	—	—	—	—	—

Wave Breaking Calculations, From Goda (1975)

Given:

$$\text{Beach Slope} = \tan \beta = \tan(1^\circ) = 0.0170 \quad (\text{Bottom slope})$$

$$\text{Deep Water Wave Height, } H_o \text{ (m)} = 1.0000$$

$$\text{Wave Period, } T \text{ (sec)} = 6.0000$$

$$\text{Deep Water Wave Length, } L_o \text{ (m)} = 56.1600 \quad (L_o = 1.56T^2)$$

$$\frac{H_b}{L_o} = A \left[1 - \exp \left\{ -1.5 \frac{\pi H_b}{L_o} \left(1 + 1.5 (\tan \beta)^{\frac{4}{3}} \right) \right\} \right]$$

$$A, \text{ coefficient} = 0.1700$$

$$h_b, \text{ breaking water depth (m)} = ?$$

$$H_o, \text{ (m)} = 2.0$$

$$H_b \text{ calculation from Sunamura (1983)} \quad H_b = H_o (\tan \beta)^{0.2} \left(\frac{H_o}{L_o} \right)^{-0.25}$$

$$H_b = 1.0(0.017)^{0.2} \left(\frac{1.0}{56.16} \right)^{-0.25} = 2.0381$$

$$h_b = 3.049$$

	Prototype, m	Model, m
Breaking water depth for 1 m wave, m =	1.72	0.11
Breaking water depth for 1.5 m wave, m =	2.40	0.16
Breaking water depth for 2.0 m wave, m =	3.05	0.20

$$g = 9.81 \text{ m/s}^2$$

Wave Period, T(s)	Water Depth, h(m)	h/gT^2	Coefficients, k_s	Crest angle with contour	Wave Height 1, H(m)	Wave Height 2, H(m)	Wave Height 3, H(m)
5.00	30.0	0.1223		32.0	1.00	1.50	2.00
5.00	20.0	0.0815	0.9900	32.0	0.99	1.49	1.98
5.00	15.0	0.0612	0.9720	32.0	0.97	1.46	1.94
5.00	10.0	0.0408	0.9320	30.0	0.93	1.40	1.86
5.00	5.0	0.0204	0.8800	25.0	0.88	1.32	1.76
5.00	4.0	0.0163	0.8900	22.5	0.89	1.34	1.78
5.00	3.0	0.0122	0.9150	19.5	0.92	1.37	–
5.00	2.5	0.0102	0.9300	18.4	0.93	1.40	–
5.00	2.0	0.0082	0.9750	16.0	0.98	–	–
5.00	1.5	0.0061	–	–	–	–	–

Wave Breaking Calculations, From Goda (1975)

Given:

Beach Slope = $\tan\beta = \tan(1^\circ) = 0.0170$ (Bottom slope)
 Deep Water Wave Height, H_o (m) = 1.0000
 Wave Period, T (sec) = 5.0000
 Deep Water Wave Length, L_o (m) = 39.0000 ($L_o = 1.56T^2$)

A , coefficient = 0.1700

h_b , breaking water depth (m) = ?

H_o , (m) = 2.0

H_b calculation from Sunamura (1983)

1.8605

$h_b = 2.905$

	Prototype, m	Model, m
Breaking water depth for 1 m wave, m =	1.61	0.11
Breaking water depth for 1.5 m wave, m =	2.26	0.15
Breaking water depth for 2.0 m wave, m =	2.91	0.19

$$g = 9.81 \text{ m/s}^2$$

Wave Period, T(s)	Water Depth, h(m)	h/gT^2	Coefficients, $k_r k_s$	Crest angle with contour	Wave Height 1, H(m)	Wave Height 2, H(m)	Wave Height 3, H(m)
4.00	30.0	0.1911	Deep Water	32.0	1.000	1.50	2.00
4.00	20.0	0.1274	Deep Water	32.0	1.000	1.50	2.00
4.00	15.0	0.0956	0.9950	32.0	0.995	1.49	1.99
4.00	10.0	0.0637	0.9750	32.0	0.975	1.46	1.95
4.00	5.0	0.0319	0.8900	27.5	0.890	1.34	1.78
4.00	4.0	0.0255	0.8800	26.0	0.880	1.32	1.76
4.00	3.0	0.0191	0.8800	24.5	0.880	1.32	1.76
4.00	2.0	0.0127	0.9100	20.0	0.910	1.37	–
4.00	1.75	0.0111	0.9200	18.5	0.920	–	–
4.00	1.5	0.0096	–	–	–	–	–

Wave Breaking Calculations, From Goda (1975)

Given:

Beach Slope = $\tan\beta = \tan(1^\circ) = 0.0170$ (Bottom slope)
 Deep Water Wave Height, H_o (m) = 1.0000
 Wave Period, T (sec) = 4.0000
 Deep Water Wave Length, L_o (m) = 24.9600 ($L_o = 1.56T^2$)

A , coefficient = 0.1700

h_b , breaking water depth (m) = ?

H_o , (m) = 2.0

H_b calculation from Sunamura (1983)

$$1.6641$$

$$h_b = 2.810$$

	Prototype, m	Model, m
Breaking water depth for 1 m wave, m =	1.50	0.10
Breaking water depth for 1.5 m wave, m =	2.14	0.14
Breaking water depth for 2.0 m wave, m =	2.81	0.19

Appendix A3: Phase 1 & 2 - Model Layout

Phase 1

Bottom Elevation = 69.27 m

Sections	Offset from X			Offset from Y			Elevation		
	Fig. (cm)	Proto (m)	Model (m)	Fig. (cm)	Proto (m)	Model (m)	Proto (m)	Model (m)	
A' - A'	0	0	0	2.95	24.04	1.60	73.76	29.93	1
	9.4	76.61	5.11	4.2	34.23	2.28	74.12	32.33	2
			↑	5.65	46.05	3.07	Embayment	Embayment	3
				7.9	64.39	4.29	73.81	30.27	4
				9.4	76.61	5.11	73.65	29.20	5
				12	97.80	6.52	73.47	28.00	6
				13.45	109.62	7.31	73.32	27.00	7
				15.5	126.33	8.42	73.00	24.87	8
AA	0.8	6.52	0.43	2.8	22.82	1.52	77.43	54.39	1
B' - B'	0.7	5.705	0.38	5.10	41.57	2.77	Embayment	Embayment	1
	6.6	53.79	3.59	7.85	63.98	4.27	74.52	35.00	2
				9.05	73.76	4.92	74.57	35.33	3
				10.25	83.54	5.57	74.59	35.47	4
				11.50	93.73	6.25	74.58	35.40	5
				12.70	103.51	6.90	74.5	34.87	6
				13.90	113.29	7.55	74.24	33.13	7
				15.15	123.47	8.23	74.02	31.67	8
BB	1.10	8.97	0.60	2.30	18.75	1.25	73.3	26.87	1
	5.70	46.46	3.10	3.50	28.53	1.90	73.6	28.87	2
				5.00	40.75	2.72	Embayment		3
				6.95	56.64	3.78	Armour Stone ends; Beach starts		4
CC	2.50	20.38	1.36	3.15	25.67	1.71	Embayment	Embayment	1
DD	3.60	29.34	1.96	2.30	18.75	1.25	72.1	18.87	1
				4.90	39.94	2.66	71.90	17.53	2
				6.15	50.12	3.34	73.04	25.13	3
				8.10	66.02	4.40	74.55	35.20	4
				10.60	86.39	5.76	Sea Wall	Sea Wall	5
EE	4.6	37.49	2.50	4.80	39.12	2.61	71.77	16.67	1
				8.10	66.02	4.40	72.94	24.47	2
				11.10	90.47	6.03	74.58	35.40	3
				14.60	118.99	7.93	Sea Wall	Sea Wall	4
FF	5.80	47.27	3.15	4.85	39.53	2.64	71.60	15.53	1
				8.75	71.31	4.75	72.60	22.20	2
				14.35	116.95	7.80	74.02	31.67	3
				0.00	0.00	0.00	0.00		4
Additional						6.28			5
GG	8.2	66.83	4.46	4.85	39.53	2.64	71.27	13.33	1
				8.10	66.02	4.40	71.25	13.20	2
				11.55	94.13	6.28	72.10	18.87	3
				0.00	0.00	0.00	0.00		4

Phase 1

Sections	Offset from X			Offset from Y			Elevation		
	Fig. (cm)	Proto (m)	Model (m)	Fig. (cm)	Proto (m)	Model (m)	Proto (m)	Model (m)	
HH	10.30	83.95	5.60	4.90	39.94	2.66	71.05	11.87	1
				9.20	74.98	5.00	71.00	11.53	2
				13.80	112.47	7.50	71.69	16.13	3
				1.70	13.86	0.92	0.00		4

II	12.40	101.06	6.74	4.85	39.53	2.64	70.76	9.93	1
				10.30	83.95	5.60	70.76	9.93	2
				0.00	0.00	0.00	0.00		3
				0.00	0.00	0.00	0.00		4

JJ	14.00	114.10	7.61	4.90	39.94	2.66	70.35	7.20	1
				11.20	91.28	6.09	70.42	7.67	2
				0.00	0.00	0.00			3
				0.00	0.00	0.00			4

KK	16.2	132.03	8.80	4.90	39.94	2.66	70.1	5.53	1
				12.50	101.88	6.79	70.15	5.87	2
				0.00	0.00	0.00			3
				0.00	0.00	0.00			4

LL	17.70	144.26	9.62	4.90	39.94	2.66	69.93	4.40	1
				13.30	108.40	7.23	70.08	5.40	2
				0.00	0.00	0.00	0		3
				0.00	0.00	0.00	0		4

1 - 1	2.95	24.04	1.60	4.65	37.90	2.53	Outfall Geometry		
2 - 2	2.55	20.78	1.39	4.85	39.53	2.64			
3 - 3	1.05	8.56	0.57	5.90	48.09	3.21			

W - 1	1.25	10.19	0.68	4.10	33.42	2.23	77.428	54.39	1	West Sea Wall
W - 2	0.98	7.99	0.53	3.50	28.53	1.90	77.428	54.39	2	
W - 3	0.75	6.11	0.41	2.85	23.23	1.55	77.428	54.39	3	
W - 4	0.90	7.34	0.49	3.65	29.75	1.98	77.428	54.39	4	
W - 5	0.80	6.52	0.43	4.50	36.68	2.45	77.428	54.39	5	
W - 6	0.00	0.00	0.00	3.30	26.90	1.79	77.428	54.39	6	
W - 7	0.00	0.00	0.00	2.60	21.19	1.41	77.428	54.39	7	
W - 8	0.00	0.00	0.00	5.30	43.20	2.88	77.428	54.39	8	

E - 1	0.00	0.00	0.00	6.50	52.98	3.53	77.428	54.39	1	East Sea Wall
E - 2	2.50	20.38	1.36	6.40	52.16	3.48	77.428	54.39	2	
E - 3	5.20	42.38	2.83	16.20	132.03	8.80	77.428	54.39	3	

Sections	Offset from X			Offset from Y			Elevation		
	Fig. (cm)	Proto (m)	Model (m)	Fig. (cm)	Proto (m)	Model (m)	Proto (m)	Model (m)	

S - 1	0.95	7.74	0.52	5.80	47.27	3.15	75.05	38.53	1	Pipe Invert
S - 2	1.10	8.97	0.60	5.95	48.49	3.23	75.05	38.53	2	Pipe Invert
S - 3	3.10	25.27	1.68	4.90	39.94	2.66			3	Sheet Piles
S - 4	2.62	21.35	1.42	4.32	35.21	2.35	75.01	38.27	4	Sheet Piles
S - 5	2.80	22.82	1.52	4.60	37.49	2.50	75.00	38.20	5	Sheet Piles
S - 6	2.47	20.13	1.34	4.85	39.53	2.64	75.00	38.20	6	Level
S - 7	2.25	18.34	1.22	5.50	44.83	2.99	74.45	34.50	7	Sheet Piles
S - 8	1.95	15.89	1.06	4.75	38.71	2.58	75.01	38.27	8	Sheet Piles

Phase 2

Phase two - layout 2

Bottom Elevation = 69.27 Factor = 8.654

Sections	Offset from X			Offset from Y			Elevation		
	Fig. (cm)	Proto (m)	Model (m)	Fig. (cm)	Proto (m)	Model (m)	Proto (m)	Model (m)	
A	1.8	15.58	1.04	1.90	16.44	1.10	71.27	13.3	1
				5.45	47.16	3.14	72.47	21.3	2
				7.00	60.58	4.04	73.28	26.7	3
				7.95	68.80	4.59	73.70	29.5	4
				W. Wall					
B	4.42	38.25	2.55	1.90	16.44	1.10	71.21	12.9	1
				3.05	26.39	1.76	71.50	14.9	2
				6.15	53.22	3.55	72.60	22.2	3
				7.40	64.04	4.27	73.30	26.9	4
				8.10	70.10	4.67	73.84	30.5	5
				W. Wall					
C	3.45	29.86	1.99	1.90	16.44	1.10	71.15	12.5	1
				3.15	27.26	1.82	71.44	14.5	2
				5.75	49.76	3.32	72.10	18.9	3
				7.35	63.61	4.24	72.58	22.1	4
				8.00	69.23	4.62	73.60	28.9	5
				8.40	72.69	4.85	74.17	32.7	6
				Armour Stone					
D	4.1	35.48	2.37	1.90	16.44	1.10	71.08	12.1	1
				3.20	27.69	1.85	71.36	13.9	2
				5.65	48.90	3.26	71.65	15.9	3
				Embayment					
E	6.00	51.92	3.46	1.90	16.44	1.10	71.00	11.5	1
				6.05	52.36	3.49	71.69	16.1	2
				7.25	62.74	4.18	71.84	17.1	3
				7.85	67.93	4.53	72.18	19.4	4
				8.25	71.40	4.76	72.65	22.5	5
				8.75	75.72	5.05	73.12	25.7	6
				9.00	77.89	5.19	73.80	30.2	7
				9.30	80.48	5.37	74.48	34.7	8
				Armour Stone					
F	6.85	59.28	3.95	4.40	38.08	2.54	71.41	14.3	1
				6.40	55.39	3.69	71.60	15.5	2
				7.30	63.17	4.21	71.97	18.0	3
				8.00	69.23	4.62	72.48	21.4	4
				8.80	76.16	5.08	72.99	24.8	5
				9.30	80.48	5.37	73.76	29.9	6
				9.70	83.94	5.60	74.57	35.3	7
				Beach/E. Wall					

Phase 2

Sections	Offset from X			Offset from Y			Elevation		
	Fig. (cm)	Proto (m)	Model (m)	Fig. (cm)	Proto (m)	Model (m)	Proto (m)	Model (m)	
G	7.8	67.50	4.50	2.10	18.17	1.21	71.11	12.3	1
				5.50	47.60	3.17	71.44	14.5	2
				6.80	58.85	3.92	71.74	16.5	3
				7.75	67.07	4.47	72.30	20.2	4
				8.85	76.59	5.11	72.85	23.9	5
				9.50	82.21	5.48	73.71	29.6	6
				10.10	87.41	5.83	74.56	35.3	7
				Beach/ E.Wall					
H	8.75	75.72	5.05	1.90	16.44	1.10	70.97	11.3	1
				4.50	38.94	2.60	71.23	13.1	2
				6.20	53.65	3.58	71.51	14.9	3
				7.40	64.04	4.27	72.11	18.9	4
				8.90	77.02	5.13	72.71	22.9	5
				9.75	84.38	5.63	73.65	29.2	6
				10.10	87.41	5.83	74.58	35.4	7
				Beach/ E.Wall					
I	10.45	90.43	6.03	2.80	24.23	1.62	70.96	11.3	1
				5.20	45.00	3.00	71.11	12.3	2
				6.95	60.15	4.01	71.25	13.2	3
				9.00	77.89	5.19	72.46	21.3	4
				10.20	88.27	5.88	73.51	28.3	5
				11.50	99.52	6.63	74.55	35.2	6
				E. Wall					
J	12.65	109.47	7.30	3.85	33.32	2.22	70.66	9.3	1
				6.30	54.52	3.63	71.00	11.5	2
				9.15	79.18	5.28	72.12	19.0	3
				10.80	93.46	6.23	73.12	25.7	4
				12.50	108.18	7.21	74.11	32.3	5
				E. Wall					

Appendix A4: Characteristics of Embayment Sediment

Appendix A4 - Results of sediment sampling from Green Rd. Embayment

ID	Location	D ₅₀ , mm	D ₁₀ , mm	D ₃₀ , mm	D ₆₀ , mm	C _u = D ₆₀ /D ₁₀	C _c = (D ₃₀) ² /(D ₆₀ X D ₁₀)	Description of Sediment
A	East side of Beach	2.4	0.82	1.5	3.2	3.90	0.86	Sand, Medium and Coarse
B	From shallow lake bed near beach	6.1	2.5	3.9	7	2.80	0.87	Gravel and Coarse Sand
C	Inside berm	2.3	0.63	1.4	2.85	4.52	1.09	Sand, Medium and Coarse
D	Middle of berm from embayment	1.8	0.63	1.2	2.5	3.97	0.91	Sand, Medium and Coarse
E	East side in deep water	0.09	no data	no data	0.095	#/VALUE!	#/VALUE!	Sand with Fines

Appendix A5: Sieve Analysis of Samples
Sieve Analysis for MacViro Consultants Inc.

Location: From beach - East Side

Percent Passing

100

Description of soil: Sand, medium and coarse.

Sample No. : 1

Weight of oven dry sample, W =

800 grams

A

U.S. Standard Sieve Series	Sieve Opening, mm	Mass Retained, M	Percent Retained	Cummulative Percent Retained	Cummulative Percent Passing
1"	25.4	0	0.00	0.00	100.00
3/4"	19.0	27.51	3.44	3.44	96.56
3/8"	9.51	38.45	4.81	8.25	91.76
4	4.75	141.66	17.71	25.95	74.05
10	2.000	230.8	28.85	54.80	45.20
20	0.850	261.27	32.66	87.46	12.54
40	0.425	96.81	12.10	99.56	0.44
100	0.150	2.11	0.26	99.83	0.17
200	0.075	0.12	0.02	99.84	0.16
Pan	-	0.03			

Location: From beach - East Side

Description of soil: Sand, medium and coarse.

Sample No. : 2

Weight of oven dry sample, W =

800 grams

U.S. Standard Sieve Series	Sieve Opening, mm	Mass Retained, M	Percent Retained	Cummulative Percent Retained	Cummulative Percent Passing
1"	25.4	0	0.00	0.00	100.00
3/4"	19.0	0	0.00	0.00	100.00
3/8"	9.51	72.64	9.08	9.08	90.92
4	4.75	143.82	17.98	27.06	72.94
10	2.000	252.24	31.53	58.59	41.41
20	0.850	251.18	31.40	89.99	10.02
40	0.425	78.43	9.80	99.79	0.21
100	0.150	1.19	0.15	99.94	0.06
200	0.075	0.07	0.01	99.95	0.05
Pan	-	0.02			

Average of Both Samples				
U.S. Sieve Series	Sieve Opening, mm	Sample 1	Sample 2	Avg. % Passing
1"	25.4	100.00	100.00	100.00
3/4"	19.0	96.56	100.00	98.28
3/8"	9.51	91.76	90.92	91.34
4	4.75	74.05	72.94	73.50
10	2.000	45.20	41.41	43.31
20	0.850	12.54	10.02	11.28
40	0.425	0.44	0.21	0.32
100	0.150	0.17	0.06	0.12
200	0.075	0.16	0.05	0.11
Pan	-			

Location: From beach - East Side, ≈ 30 cm water depth, 2 m from shore

Description of soil: Gravel and coarse sand.

Sample No. : 1

Weight of oven dry sample, W = 700 grams

B

U.S. Standard Sieve Series	Sieve Opening, mm	Mass Retained, M	Percent Retained	Cummulative Percent Retained	Cummulative Percent Passing
1"	25.4	0	0.00	0.00	100.00
3/4"	19.0	20.36	2.91	2.91	97.09
3/8"	9.51	145.17	20.74	23.65	76.35
4	4.75	269.52	38.50	62.15	37.85
10	2.000	239.43	34.20	96.35	3.65
20	0.850	23.71	3.39	99.74	0.26
40	0.425	1.33	0.19	99.93	0.07
100	0.150	0.11	0.02	99.95	0.05
200	0.075	0.18	0.03	99.97	0.03
Pan	-	0.01			

Location: From beach - East Side, ≈ 30 cm water depth, 2 m from shore

Description of soil: Gravel and coarse sand.

Sample No. : 2

Weight of oven dry sample, W = 700 grams

U.S. Standard Sieve Series	Sieve Opening, mm	Mass Retained, M	Percent Retained	Cummulative Percent Retained	Cummulative Percent Passing
1"	25.4	0	0.00	0.00	100.00
3/4"	19.0	0	0.00	0.00	100.00
3/8"	9.51	158.49	22.64	22.64	77.36
4	4.75	267.37	38.20	60.84	39.16
10	2.000	251.69	35.96	96.79	3.21
20	0.850	21.04	3.01	99.80	0.20
40	0.425	1.18	0.17	99.97	0.03
100	0.150	0.08	0.01	99.98	0.02
200	0.075	0.1	0.01	99.99	0.01
Pan	-	0.03			

Average of Both Samples				
U.S. Sieve Series	Sieve Opening, mm	Sample 1	Sample 2	Avg. % Passing
1"	25.4	100.00	100.00	100.00
3/4"	19.0	97.09	100.00	98.55
3/8"	9.51	76.35	77.36	76.86
4	4.75	37.85	39.16	38.51
10	2.000	3.65	3.21	3.43
20	0.850	0.26	0.20	0.23
40	0.425	0.07	0.03	0.05
100	0.150	0.05	0.02	0.04
200	0.075	0.03	0.01	0.02
Pan	-			

Location: 2 m inside from berm, Sample Depth ≈ 25 cm

Description of soil: Sand, medium to coarse.

Sample No. : 1

Weight of oven dry sample, W = 876.77 grams

C

U.S. Standard Sieve Series	Sieve Opening, mm	Mass Retained, M	Percent Retained	Cummulative Percent Retained	Cummulative Percent Passing
1"	25.4	0.00	0.00	0.00	100.00
3/4"	19.0	14.30	1.63	1.63	98.37
3/8"	9.51	57.23	6.53	8.16	91.84
4	4.75	106.76	12.18	20.33	79.67
10	2.000	284.26	32.42	52.76	47.24
20	0.850	254.38	29.01	81.77	18.23
40	0.425	84.90	9.68	91.45	8.55
100	0.150	61.60	7.03	98.48	1.52
200	0.075	9.53	1.09	99.57	0.43
Pan	-	1.13			

Location: 2 m inside from berm, Sample Depth ≈ 25 cm

Description of soil: Sand, medium to coarse.

Sample No. : 2

Weight of oven dry sample, W = 822.25 grams

U.S. Standard Sieve Series	Sieve Opening, mm	Mass Retained, M	Percent Retained	Cummulative Percent Retained	Cummulative Percent Passing
1"	25.4	0.00	0.00	0.00	100.00
3/4"	19.0	0.00	0.00	0.00	100.00
3/8"	9.51	38.05	4.63	4.63	95.37
4	4.75	117.94	14.34	18.97	81.03
10	2.000	310.30	37.74	56.71	43.29
20	0.850	253.15	30.79	87.50	12.50
40	0.425	65.74	8.00	95.49	4.51
100	0.150	32.17	3.91	99.40	0.60
200	0.075	3.68	0.45	99.85	0.15
Pan	-	0.23			

Average of Both Samples				
U.S. Sieve Series	Sieve Opening, mm	Sample 1	Sample 2	Avg. % Passing
1"	25.4	100.00	100.00	100.00
3/4"	19.0	98.37	100.00	99.18
3/8"	9.51	91.84	95.37	93.61
4	4.75	79.67	81.03	80.35
10	2.000	47.24	43.29	45.27
20	0.850	18.23	12.50	15.37
40	0.425	8.55	4.51	6.53
100	0.150	1.52	0.60	1.06
200	0.075	0.43	0.15	0.29
Pan	-			

Location: From Embayment Berm Middle
Description of soil: Sand, medium to coarse.

Sample No. : 1

Weight of oven dry sample, W = 800 grams

D

U.S. Standard Sieve Series	Sieve Opening, mm	Mass Retained, M	Percent Retained	Cummulative Percent Retained	Cummulative Percent Passing
1"	25.4	0.00	0.00	0.00	100.00
3/4"	19.0	0.00	0.00	0.00	100.00
3/8"	9.51	8.07	1.01	1.01	98.99
4	4.75	63.81	7.98	8.99	91.02
10	2.000	265.23	33.15	42.14	57.86
20	0.850	315.36	39.42	81.56	18.44
40	0.425	111.71	13.96	95.52	4.48
100	0.150	34.36	4.30	99.82	0.18
200	0.075	0.39	0.05	99.87	0.13
Pan	-	0.10			

Location: From Embayment Berm Middle
Description of soil: Sand, medium to coarse.

Sample No. : 2

Weight of oven dry sample, W = 800 grams

U.S. Standard Sieve Series	Sieve Opening, mm	Mass Retained, M	Percent Retained	Cummulative Percent Retained	Cummulative Percent Passing
1"	25.4	0.00	0.00	0.00	100.00
3/4"	19.0	0.00	0.00	0.00	100.00
3/8"	9.51	19.52	2.44	2.44	97.56
4	4.75	66.19	8.27	10.71	89.29
10	2.000	272.74	34.09	44.81	55.19
20	0.850	297.05	37.13	81.94	18.06
40	0.425	102.88	12.86	94.80	5.20
100	0.150	40.08	5.01	99.81	0.19
200	0.075	0.58	0.07	99.88	0.12
Pan	-	0.08			

Average of Both Samples				
U.S. Sieve Series	Sieve Opening,mm	Sample 1	Sample 2	Avg. % Passing
1"	25.4	100.00	100.00	100.00
3/4"	19.0	100.00	100.00	100.00
3/8"	9.51	98.99	97.56	98.28
4	4.75	91.02	89.29	90.15
10	2.000	57.86	55.19	56.53
20	0.850	18.44	18.06	18.25
40	0.425	4.48	5.20	4.84
100	0.150	0.18	0.19	0.19
200	0.075	0.13	0.12	0.13
Pan	-			

Location: E1 <NE>, 3-4 m water depth, 7 m from shore

Description of soil: Sand with fines.

Sample No. : 1

Weight of oven dry sample, W = 150 grams

E

U.S. Standard Sieve Series	Sieve Opening, mm	Mass Retained, M	Percent Retained	Cummulative Percent Retained	Cummulative Percent Passing
1"	25.4	0.00	0.00	0.00	100.00
3/4"	19.0	0.00	0.00	0.00	100.00
3/8"	9.51	0.00	0.00	0.00	100.00
4	4.75	0.00	0.00	0.00	100.00
10	2.000	0.10	0.07	0.07	99.93
20	0.850	0.46	0.31	0.37	99.63
40	0.425	1.17	0.78	1.15	98.85
100	0.150	9.19	6.13	7.28	92.72
200	0.075	86.97	57.98	65.26	34.74
Pan	-	49.77	33.18	98.44	1.56

Location: E1 <NE>, 3-4 m water depth, 7 m from shore

Description of soil: Sand with fines.

Sample No. : 2

Weight of oven dry sample, W = 150 grams

U.S. Standard Sieve Series	Sieve Opening, mm	Mass Retained, M	Percent Retained	Cummulative Percent Retained	Cummulative Percent Passing
1"	25.4	0.00	0.00	0.00	100.00
3/4"	19.0	0.00	0.00	0.00	100.00
3/8"	9.51	0.00	0.00	0.00	100.00
4	4.75	0.00	0.00	0.00	100.00
10	2.000	0.48	0.32	0.32	99.68
20	0.850	0.58	0.39	0.71	99.29
40	0.425	1.23	0.82	1.53	98.47
100	0.150	8.77	5.85	7.37	92.63
200	0.075	91.33	60.89	68.26	31.74
Pan	-	46.58	31.05	99.31	0.69

Average of Both Samples				
U.S. Sieve Series	Sieve Opening,mm	Sample 1	Sample 2	Avg. % Passing
1"	25.4	100.00	100.00	100.00
3/4"	19.0	100.00	100.00	100.00
3/8"	9.51	100.00	100.00	100.00
4	4.75	100.00	100.00	100.00
10	2.000	99.93	99.68	99.81
20	0.850	99.63	99.29	99.46
40	0.425	98.85	98.47	98.66
100	0.150	92.72	92.63	92.67
200	0.075	34.74	31.74	33.24
Pan	-	1.56	0.69	

Sieve Analysis for MacViro Consultants Inc.

Location: CCIW

Percent Passing

100

Description of soil: Brick Sand

Sample No. : 1

Weight of oven dry sample, W =

1000 grams

F

U.S. Standard Sieve Series	Sieve Opening, mm	Mass Retained, M	Percent Retained	Cummulative Percent Retained	Cummulative Percent Passing
4	4.8	0	0.00	0.00	100.00
10	2.0	2.48	0.25	0.25	99.75
30	0.6	92.42	9.24	9.49	90.51
40	0.425	69.03	6.90	16.39	83.61
50	0.300	137.44	13.74	30.14	69.86
100	0.150	308.02	30.80	60.94	39.06
200	0.075	182.33	18.23	79.17	20.83
Pan	—	201.34	20.13	99.31	0.69

Location: CCIW

Description of soil: Brick Sand

Sample No. : 2

Weight of oven dry sample, W =

1000 grams

U.S. Standard Sieve Series	Sieve Opening, mm	Mass Retained, M	Percent Retained	Cummulative Percent Retained	Cummulative Percent Passing
4	4.8	0	0.00	0.00	100.00
10	2.0	2.88	0.29	0.29	99.71
30	0.6	91.51	9.15	9.44	90.56
40	0.425	70.63	7.06	16.50	83.50
50	0.300	141.52	14.15	30.65	69.35
100	0.150	329.69	32.97	63.62	36.38
200	0.075	116.61	11.66	75.28	24.72
Pan	—	244.24	24.42	99.71	0.29

Average of Both Samples

U.S. Sieve Series	Sieve Opening,mm	Sample 1	Sample 2	Avg. % Passing
4	4.8	100.00	100.00	100.00
10	2.0	99.75	99.71	99.73
30	0.6	90.51	90.56	90.54
40	0.425	83.61	83.50	83.55
50	0.300	69.86	69.35	69.60
100	0.150	39.06	36.38	37.72
200	0.075	20.83	24.72	22.77
Pan	—	0.69	0.29	0.49

Specific Gravity Table

Trial # (Nut Shell)	1	2
Mass of flask, g	169.38	162.26
Mass of flask + water, g	667.44	660.22
Temperature of water, °C	21.8	22.3
Density of water, g/ml	0.997846	0.997732
Volume of flask, ml	499.1351	499.0917
Mass of flask + water + soil, g	682.35	675.49
Temperature of water, °C	25	25.4
Density of water, g/ml	0.997077	0.996973
Volume of water, ml	464.3272	464.6366
Volume of soil, ml	34.80791	34.4551
Mass of soil, g	50	50
Density of solids, g/ml	1.436455	1.451164
Specific gravity of solids	1.439556	1.454462

Specific Gravity Average = 1.45

Trial # (Garden)	1	2
Mass of flask, g	161.24	162.13
Mass of flask + water, g	659.23	659.09
Temperature of water, °C	22.8	23
Density of water, g/ml	0.997616	0.997570
Volume of flask, ml	499.1798	498.1706
Mass of flask + water + soil, g	690.63	690.34
Temperature of water, °C	23.5	23.5
Density of water, g/ml	0.997419	0.997419
Volume of water, ml	480.6305	479.4475
Volume of soil, ml	18.54934	18.7231
Mass of soil, g	50	50
Density of solids, g/ml	2.695514	2.670498
Specific gravity of solids	2.701954	2.677003

Specific Gravity Average = 2.69

Trial # (Brick Sand)	1	2
Mass of flask, g	169.26	170.13
Mass of flask + water, g	666.96	667.07
Temperature of water, °C	23	23
Density of water, g/ml	0.997570	0.997570
Volume of flask, ml	498.9124	498.1505
Mass of flask + water + soil, g	698.46	698.73
Temperature of water, °C	23.5	23.6
Density of water, g/ml	0.997419	0.997425
Volume of water, ml	480.44	479.8354
Volume of soil, ml	18.47234	18.31512
Mass of soil, g	50	50
Density of solids, g/ml	2.706749	2.729985
Specific gravity of solids	2.713343	2.736635

Specific Gravity Average = 2.72

Appendix A7: Model Wave Gauge Data (On CD-ROM)

Appendix A8: Selection of Model Parameters

The following section identifies which parameters in the VOF and Standard K- ϵ models were selected by user to run the simulation. Many of the variables selected were accompanied by a short critique in a table explaining why these values were chosen. The selection of model variables was presented by affixing the actual program panel into the document. The various panels are:

1. Precision Panel
2. Solver Panel
3. Multiphase Model Panel
4. Viscous Model Panel
5. Materials Panel
6. Phases Panel
7. Operating Conditions Panel
8. Boundary Conditions Panel
9. Controls Panel
10. Initialization Panel
11. Iteration Panel

5.6.1 Precision Selection Panel

Shown below in Figure 5.10 is the precision panel which was initially displayed when the user started the program.

Figure 5. 14 Precision panel.

Table 5. 2 Overview of Precision panel

Name on Panel	Overview of Selection
Precision Panel	<ul style="list-style-type: none">-three-dimensional double precision calculation (3ddp) was selected to provide the greatest numerical accuracy-it was believed by researchers that the complex SSO geometry warranted additional precision to handle the unstructured tetrahedral mesh, channel orientation and channel slopes

5.6.2 Solver Selection Panel

Figure 5. 15 Solver panel.

Table 5. 3 Overview of the Model panel

Name on Panel	Overview of Selection
Solver	<ul style="list-style-type: none"> -the segregated solver was chosen for this study because this was a multiphase problem that involves both air and water -this particular solver was only for incompressible flows -the governing equations are solved sequentially (segregated from one another)
Space	<ul style="list-style-type: none"> -three dimensional space was employed for the SSO construction because this "space" bears greater similitude to the prototype -an unstructured 3-D tetrahedral hybrid mesh was created in Gambit for SSO simulations in FLUENT
Formulation	<ul style="list-style-type: none"> -the implicit formulation was selected by default because the segregated solver was chosen -the implicit formulation is stable with respect to the time step size because it uses a different time evolution scheme to evaluate every term in the differential equation over a time step Δt
Time	<ul style="list-style-type: none"> -the Unsteady time calculation was selected because the depth and the velocity of the flow vary with time
Transient Controls	<ul style="list-style-type: none"> -this option was switched off because it increased the running time of the iteration, however, in some cases the NITA option can increase computing speed by up to 6 times (FLUENT, 2006)
Unsteady Formulation	<ul style="list-style-type: none"> -1st-Order Implicit was selected, this option is sufficient for most problems as it yields fast convergence but it is less accurate than its counterpart -Initially 2nd-Order was selected but was later changed because of convergence problems
Porous Formulation	<ul style="list-style-type: none"> Superficial Velocity was selected to prevent the velocity from decreasing if it crossed a porous boundary, such as a piece of filter fabric or a perforated stone bed

5.6.3 Multiphase Selection Panel

Figure 5. 16 Multiphase solver panel.

Table 5. 4 Overview of Multiphase Model panel

Name on Panel	Overview of Selection
Model	<ul style="list-style-type: none"> -the Volume of Fluid model (VOF) was chosen to reproduce multiple phases (air and water) -VOF model is good for surface tracking applied to fixed Eulerian mesh -Eulerian model is good for granular flows involving sediment -Mixture model allows flows to be interpenetrating and to move at different velocities (slip velocity)
Number of Phases	<ul style="list-style-type: none"> - two phases were used in the VOF model: Phase 1 = Air, Phase 2 = Water
VOF Scheme	<ul style="list-style-type: none"> -Euler Explicit scheme was selected because a time dependent scheme was selected and also because this scheme yielded better results for hybrid meshes. -Implicit will quickly produce a steady state solution, Geo-Reconstruct is used for time accurate transient behaviour
Courant Number	<ul style="list-style-type: none"> -This number controls the time stepping scheme, time step is proportional to the Courant Number -large Courant Number will yield fast convergence (within a range, determined by linear stability theory) -Initially the Courant Number should be set to a small value (0.1-0.5), once transients have settled, the Courant Number can be increased
Solve VOF Every Iteration	<ul style="list-style-type: none"> -this option was not selected because it is not necessary to solve the volume fraction equation at every iteration within a time step because a small time step was used for this study (0.005 seconds)
Open Channel Flow	<ul style="list-style-type: none"> -the flow in the SSO is similar to an open channel flow
Body Force Formulation	<ul style="list-style-type: none"> -good for calculations involving surface tension

5.6.4 Viscous Model Selection Panel

Figure 5. 17 Viscous model panel.

Table 5. 5 Overview of Viscous Model panel

Name on Panel	Overview of Selection
Model	-the standard <i>k-ε</i> model was selected for its robustness, economy and reasonable accuracy for a wide range of flow problems
k-epsilon	- <i>Standard k-ε model</i> was chosen because it has proven itself time and time again to model turbulence -the model constants have been determined by experiments with air and water for fundamental shear flow (experiments performed by others) - <i>RNG</i> improves accuracy for rapidly strained and swirl flows - <i>Realizable</i> utilizes a new formulation for turbulent viscosity, this model is superior for analyzing rotational flows
Near-Wall Treatment	- <i>Standard Wall Functions</i> was used because the flow is not subjected to severe pressure gradients -Other Wall Treatment Options are good for high Reynolds Number flows and situations when resolving the laminar sub-layer is necessary
Turbulent Viscosity	-this option is not available with the current panel settings

5.6.5 Materials Selection Panel

Figure 5. 18 Materials panel.

Table 5. 6 Overview of Materials panel

Name on Panel	Overview of Selection
Air	-by default air is selected as one of the model materials -the density, viscosity and material type are all accepted as they are
Water	-a new material named water was added from the FLUENT database -Properties: Material type = fluid, Density = 998.2 kg/m ³ , Dynamic Viscosity = 1.003 x 10 ⁻³ kg/m-s

5.6.6 Phases Selection Panel

Figure 5. 19 Phases panel.

Table 5. 7 Overview of Phases panel

Name on Panel	Overview of Selection
Air	-air was selected as the primary phase because it is above the water (representative of nature)
Water	-water was selected as the secondary phase because it is below air

5.6.7 Operating Conditions Selection Panel

Operating Conditions

Pressure

Operating Pressure (pascal)
101325

Reference Pressure Location

X (m) 0
Y (m) 0
Z (m) 0

Gravity

☒ Gravity

Gravitational Acceleration

X (m/s²) 0
Y (m/s²) -9.81
Z (m/s²) 0

Variable-Density Parameters

☒ Specified Operating Density

Operating Density (kg/m³)
1.225

OK Cancel Help

Figure 5. 20 Operating Conditions panel.

Table 5. 8 Overview of Operating Conditions

Name on Panel	Overview of Selection
Operating Pressure	<ul style="list-style-type: none"> -the default operating pressure (atmospheric pressure) was used for calculations -the operating pressure is important for incompressible gas calculations because it directly determines density from the ideal gas law
Reference Pressure Location	<ul style="list-style-type: none"> -the Cartesian coordinates of (0,0,0) indicate the pressure will always be the same at this location
Gravity	<ul style="list-style-type: none"> -the gravity option was used because the flow is travelling on a surface with slopes -a negative sign is affixed to the Y-direction to indicate gravitation acceleration points downward
Variable-Density Parameters	<ul style="list-style-type: none"> -this option was turned on and the default density value for air was accepted -the operating density is set to the lighter phase, this excludes the build-up of hydrostatic pressure within the lighter phase, improving round-off accuracy for the momentum balance

5.6.8 Boundary Conditions for the Velocity Inlet Selection Panel

Figure 5.21 B.C. panel.

Figure 5.22 Velocity inlet data panel.

Table 5.9 Overview of Velocity Inlet

Name on Panel	Overview of Selection
Zone Name	-the <i>Inlet</i> zone was assigned to act as a “Velocity Inlet” – Fig. 5.17 -the zone name in Fig. 5.18 stipulates which component in the SSO is being worked on
Phase	- mixture is chosen as it represents both phases
Velocity Specification Method	-this option allows the user to provide the velocity of the flow and Cartesian coordinates of the flow direction
Reference Frame	-Absolute reference frame is selected because mesh is not moving, inlet is stationary
Velocity Magnitude (m/s)	-if the flow from physical model tests is known, then velocity can be computed (a rubber hose with constant flow was used during the physical model tests to simulate inlet discharge into the SSO) $V=Q/A=(0.0003778 \text{ m}^3/\text{s})/(\pi \cdot 0.12 \text{ m}^2/4)=0.0334 \text{ m/s}$
Turbulence Specification Method (T.I.)	-Intensity and hydraulic diameter were selected because these values can be easily calculated -If T.I. <1%, low turbulence, if T.I. > 10% high turbulence
Turbulence Intensity (%)	$Re=VD_p/\mu=(0.0334)(0.12)(998.2)/(1.003 \times 10^{-3})=3989$ $T.I.=(0.16Re^{-1/8}) \cdot 100=5.68 \%$
Hydraulic Diameter (m)	-a model inlet pipe <i>Hydraulic Diameter</i> of 0.12 m was used for the simulation ($D_m = D_p/N_L = 1.8 \text{ m}/15 = 0.12 \text{ m}$) Where, D_m = model inlet pipe diameter, D_p = prototype inlet pipe diameter and N_L = length scale
Discrete Phase B.C. Type	- <i>Escape</i> was selected because it symbolizes the departure of he particles from the inlet

5.6.9 Outflow Selection Panel

The screenshot shows a software window titled "Outflow" with a close button in the top right corner. Inside the window, there are four input fields: "Zone Name" with the text "east_outlet", "Phase" with the text "mixture", "Flow Rate Weighting" with the value "1", and "Discrete Phase BC Type" with a dropdown menu showing "escape". At the bottom of the window, there are three buttons: "OK", "Cancel", and "Help".

Figure 5. 23 Outflow panel.

Table 5. 10 Overview of Outflow

Name on Panel	Overview of Selection
Zone Name and Phase	<ul style="list-style-type: none"> -the boundary conditions for the east outlet are essentially the same as the west outlet -the phase was chosen to be mixture because both air and water exit through outlet
Flow Rate Weighting	<ul style="list-style-type: none"> -since the SSO is a symmetrical structure the flow rating will be set to 1, indicating no flow restriction have been placed on the outlet -a flow rate weighting of 0.5 would translate to a flow reduction of 50%
Discrete Phase BC Type	<ul style="list-style-type: none"> - <i>Escape</i> was selected because it symbolizes the departure of the particles from the outlet
-Boundary conditions for the west outlet will not be presented because this outlet has identical boundary conditions and flow characteristics as the east outlet	

5.6.10 Solution Selection Panel

Figure 5. 24 Solution Controls panel.

Table 5. 11 Overview of Solution Controls

Name on Panel	Overview of Selection
Equations	-these governing equations are automatically selected by FLUENT for analysis
Discretization	<p>Pressure</p> <ul style="list-style-type: none"> -FLUENT allows the user to choose the appropriate discretization scheme for the convection terms of each governing equation -the <i>Standard</i> pressure scheme was used because it is acceptable for most problems -the <i>Body Force</i> option for pressure was not selected because it deals with large body forces -the <i>Presto</i> and <i>Second Order Scheme</i> option were also not used because the first deals with high speed rotating flows and the later with compressible flows <p>Momentum, Volume Fraction, Turbulence Kinetic Energy and Turbulence Dissipation rate</p> <ul style="list-style-type: none"> -the <i>First Order Upwind</i> was selected by default because the segregated solver was used but this parameter was later changed to <i>Second Order Upwind</i> -the <i>First Order Upwind</i> yields better convergence but is less accurate when the flow is not aligned with the grid and also when the mesh contains triangular and tetrahedral elements -the <i>Second Order Upwind</i> provides better accuracy and it can tackle all the situation that the <i>First Order Upwind</i> cannot
Pressure Velocity Coupling	<ul style="list-style-type: none"> -<i>Simple</i> was selected -<i>Simple</i> and <i>Simplex</i> provide similar results, these options, however have a greater advantages because of the increases in <i>Under-Relaxation Factors</i> than can be applied (these options are good for laminar flows in which convergence is limited by pressure velocity coupling) -<i>PISO</i> was selected because it can handle transients flows, however, this option requires additional computational power when compared to one of the other options like <i>Simple</i> or <i>Simplex</i> -<i>PISO</i> is also good for highly skewed meshes (not present in SSO)
Under-Relaxation Factors	<ul style="list-style-type: none"> -the segregated solver uses these factors to control the update of computer variables at each iteration -for most flow problems these factors do not need modification, the default values are applicable to the largest number of possible cases -it is advisable to start with default values, if residuals continue to increase then lower the factors so that convergence can occur

5.6.11 Solution Initialization Panel

Solution Initialization

Compute From
inlet

Reference Frame
☒ Relative to Cell Zone
☐ Absolute

Initial Values

Gauge Pressure (pascal) 0

X Velocity (m/s) 0

Y Velocity (m/s) 0.02636436

Z Velocity (m/s) -0.02050562

Init Reset Apply Close Help

Figure 5. 25 Solution Initialization panel.

Table 5. 12 Overview of Solution Initialization

Name on Panel	Overview of Selection
Compute From	<ul style="list-style-type: none"> -the user must provide FLUENT with an initial guess as to where the flow will start -the inlet was selected to initialize the solution because it is the natural starting point of the flow
Reference Frame	-all values will be computed relative to the inlet zone
Initial Values	-Fluent automatically calculates the values in all fields using information that has been previously supplied, the user is also free to alter this information

5.6.12 Iteration Panel

Figure 5. 26 Iteration panel.

Table 5. 13 Overview of Iteration selection

Name on Panel	Overview of Selection
Time Step Size (s)	-a small time step size (Δt) was used because unsteady flow problems typically have high transient start-ups -once transient decay to somewhat of a steady state, Δt can be increased to decrease the computational time
Number of Time Steps	-FLUENT literature suggests the ideal number of iterations per time step is between 10-20 -to roughly calculate the number of time steps required, one must observe the amount of iterations that FLUENT needs to converge at each time step, if FLUENT needs substantially more, the time step is too large
Time Stepping Method	-specifies the number of fixed-time steps to be performed before the size of the time step starts to change
Options	-this choice was not turned on because it was not necessary to monitor the time average (mean) and root-mean-squares of instantaneous values during the calculation
Iterations	-parameter sets the maximum number of iteration per time step -Figure 5.23 illustrates the iteration set up on the following page

During the computational process, FLUENT can graphically monitor convergence dynamically by checking the residuals. According to the FLUENT documentation, once the solver has completed an iteration the residual sum of each conserved variable is computed and stored (FLUENT, 2003). If one possesses a computer with infinite precision the residual will go to zero as the solution converges. However, on an actual

computer the residuals decay to a small value (“Round-off”) then stop changing (“Level Out”). Figure 5.24 illustrates the residual convergence history for 30 minute simulation.

Figure 5.27 Iteration time stepping diagram.

Figure 5.28 Residual convergence history for 30 minute simulation.

Initially, a small time step of 0.005 seconds was used for the simulation so that accurate results could be obtained. However, when the transients began to decay ($t=6$ seconds or ≈ 2300 iterations) the iteration process was halted and the time step was changed to 0.5 seconds and allowed to “Level Off.”

REFERENCES

Abbot, M.B. 1988. Numerical modeling for coastal and ocean engineering. C. Chapter in "Handbook on Coastal and Ocean Engineering," J.B. Herbich (Ed), Gulf Publishing, New York.

Ali, K.H.M., Burrows, R., Wose, A.E. (1995) Sedimentation in long sea outfalls. *Water Science and Technology*. 32(2): 123-132.

American Society of Civil Engineering. 1975. Symposium on modeling techniques, ASCE, New York.

Atkins, H.N. 2003. Highway Materials Soils, and Concretes – 4th Edition, Prentice Hall, New Jersey.

Bayram, A., Larson, M. 2000. Analysis of scour around a group of vertical piles. *ASCE Journal of Waterway, Port, Coastal, and Ocean Engineering*. 126(4): 215-220.

Brennan, M.S. 2003. CFD simulations of gravity sluices. Third International Conference on CFD in the Minerals and Process Industries. 275-280.

Campisano, A., Creaco, E., Modica, C. 2004. Experimental and numerical analysis of the scouring effects of flushing waves on sediment deposits. *Journal of Hydrology*. 299(3-4): 324-334.

Canadian Hydrographic Service, Environment Canada (2005) Lake Levels. Available on-line at http://chswwww.bur.dfo.ca/danp/bulletin_e.html

Cox, J.C., Bocamazo, L.M., Williams, G.L., Ruthven, H.E., Cornett, A.M., Chase, S.A. 2003. Designing an outfall extension through a beach renourishment for Deal Lake, New Jersey. In: Coastal Structures 2003 – Proceedings of the Conference. ASCE, Portland, OR. 1324-1333.

Dalrymple, R.A. 1985. Physical modelling in coastal engineering, A.A. Balkema, Netherlands.

Dalrymple, R.A. 1989. "Physical modeling of littoral processes," in Recent Advances in Hydraulic Physical Modeling, R. Martins, Ed., Kluwer Academic Publishers, Dordrecht, The Netherlands, pp 567-588.

Das, B.M. 1982. Soil mechanics laboratory manual. Engineering Press, San Jose, California.

Dean, R.G., Dalrymple, R.A. 1991. Water wave mechanics for engineers and scientists, World Scientific Printing Co. Pte. Ltd., Singapore.

Duer, M.J. 1998. Use of variable orifice 'duckbill' valves for hydraulic and dilution optimization of multiport diffusers. *Water Science and Technology*. **38**(10): 277-284.

EHG Report, Mohammad, M., Daviau, J.L. 2006. Draft No. 1 – Scale model for self-scouring outfall. Toronto, Ontario. pp1-12.

Fluent. 2003. FLUENT 6.1 documentation. Fluent Ltd., Lebanon, N.H.

Goda, Y. 2000. Random seas and design of maritime structure, World Scientific Printing Co. Pte. Ltd., Singapore.

Grace, R.A. 2005. Marine outfall performance I: Introduction and flow restoration. *ASCE Journal of Performance of Constructed Facilities*. **19**(4): 347-358.

Grace, R.A. 2005. Marine outfall performance II: Stabilization and case studies. *ASCE, Journal of Performance of Constructed Facilities*. **19**(4): 359-369.

Hatipoglu, F., Avci, I. 2003. Flow around a partly buried cylinder in a steady current. *Pergamon, Ocean Engineering*. **(30)**: 239-249.

He, C., Marsalek, K., Rochfort, Q., Krishnappan, B.G. 2006. Case Study: Refinement of hydraulic operation of a complex CSO storage/treatment facility by numerical and physical modeling. *ASCE, Journal of Hydraulic Engineering*. **132**(2): 131-139.

He, C., J. Marsalek and Q. Rochfort. 2004. Numerical modeling of enhancing suspended solids removal in a CSO facility. *Water Quality Research Journal of Canada* **39**(4): 457-465.

Hudson, R.Y., Herrmann, F.A., Sager, R.A., Whalin, R.W., Keulegan, G.H., Chatham, C.E., and Hales, L.Z. 1979. "Coastal Hydraulic Models," Special Report No. , US Army Engineer Waterways Experiment Station, Vicksburg, Mississippi.

Huges, S.A., Fowler, J.E. 1995. Estimating wave-induced kinematics at sloping structures. *Journal of Waterway, Port, Coastal, and Ocean Engineering*. **121**(4): 209-215.

Hughes, S.A. 1993. Physical models and laboratory techniques in coastal engineering, World Scientific Printing Co. Pte. Ltd., Singapore.

Ismail, N.M., Wiegel, R.L. 2003. Secondary flows and sediment problems near coastal marine outfalls. Coastal Structures 2003 – Proceedings of the Conference. ASCE, Portland, OR. 764-775.

Kamphuis, J.W. 2000. Introduction to coastal engineering management, World Scientific Printing Co. Pte. Ltd., Singapore.

Kamphuis, J.W. 1996. Physical modeling of coastal processes,” Chapter in “Advances in Coastal and Ocean Engineering, Volume 2,” World Scientific Press, pp. 79-114. (EC)

Kamphuis, J.W. 1991. “Physical Modeling,” In Handbook of Coastal and Ocean Engineering, J.B. Herbich, Ed., Vol 2, Gulf Publishing Company, Houston, Texas.

Kamphuis, J.W. 1988. Physical modeling. C. Chapter in “Handbook on Coastal and Ocean Engineering,” J.B. Herbich (Ed), Gulf Publishing, New York.

Kamphuis, J.W. 1982. The coastal mobile bed model, Coastal Engineering Research Centre Report No. 75, Department of Civil Engineering, Queen’s University, Kingston.
Kamphuis, J.W. 1973. Short wave models with fixed bed boundary layer. ASCE, Journal of the waterways harbors and coastal engineering division. 99(4): 471-483.

Keulegan, G.H. 1966. “Model Laws for Coastal and Estuarine Models,” in Estuary and Coastline Hydrodynamics, A.T. Ippen, Ed., McGraw-Hill Book Company, Inc., New York, pp 691-710

Kraus, N.C. 1996. History and heritage of coastal engineering. A collection of paper on the history of coastal engineering in countries hosting the international coastal engineering conference 1950 – 1996. History of coastal engineering in Canada, Kamphuis, J.W. , ASCE, U.S.A.

Langhaar, H.L 1951. Dimensional Analysis and Theory of Models, John Wiley & Sons, New York.

Larsen, T. 1995. Diffuser design for marine outfalls in areas with strong currents, high waves and sediment transport. Water Science and Technology. 32(2): 249-255.

- Larsen, T. 1987. The influence of waves on the hydraulics of sea outfalls. In: Proceedings of Coastal Engineering Conference. 3:1278-1284.
- Le Mehaute, B. 1976. "Similitude in Coastal Engineering," Journal of the Waterways, Harbors and Coastal Engineering Division, American Society of Civil Engineers, Vol. 102, No. WW3, pp 317-335
- Leutheusser, H.J. 1984. Symposium on scale effects in Modelling Hydraulic Structures, Sprint Druck, Stuttgart, Germany.
- Louder, B.E., Spalding, D.B. 1972. Lectures in mathematical models of turbulence, Academic, London.
- Maiti, S., Sen. D. 1999. Computation of solitary waves during propagation and runup on a slope. Pergamon, Ocean Engineering. (26): 1063-1083.
- Mays, L.W. 2001. Water Resources Engineering, John Wiley & Sons, United States of America.
- Miller, R.L. 1972. "The Role of Surface Tension in Breaking Waves," Proceedings of the 13th Coastal Engineering Conference, American Society of Civil Engineers, Vol 1, pp 433-449
- Mohammad, M., Daviau, J.L., Stirrup, M., Jacob, S., Luk, G. 2006. Self-scouring on lake Ontario: scale model and design issues. Presented at the: 41st Central Ontario Symposium on Water Quality Research. pp.1-15.
- Mohammad, M., Daviau, J.L., Stirrup, M., Jacob, S., Luk, G. 2006. Self-scouring outfall on lake Ontario: scale model and design issues. Presented at the: WEAO 35th Annual Technical Symposium and OPCEA Exhibition. pp.1-16.
- Munsun, B.R., Young, D.F., and Okiishi, T.H. 1990. Fundamentals of Fluid Mechanics, John Wiley & Sons, New York.
- Nicholas, A.P., Smith, Sambrook, G.H. 1999. Numerical simulation of three-dimensional flow hydraulics in a braided channel. Hydrological Processes. (13): 913-929.

Noble, M.A., Sherwood, C.R., Xu, J., Dartnell, P., Robertson, G., Martini, M. 1996. Transport processes near coastal ocean outfalls. Ocean Conference Record (IEEE). 3: 1996-2000.

Novak, P., Cabelka, J. 1981. Models in hydraulic engineering – physical principles and design applications, Pitman Advances Publishing, Melbourne.

Price, W.A. 1979. “Models – Can We Learn from the Past (theme speech),” Proceedings of the 16th Coastal Engineering Conference, American Society of Civil Engineering, Vol 1, pp 25-36.

Richard, W.P., John, C.A., David, B., Sian, J. 1996. Development of design methodology for self-cleansing sewers. Water Science and Technology. 33(9): 195-205.

Salaheldin, T.M., Imran, J., Chaudhry, M.H. 2004. Numerical modeling of three dimensional flow field around circular piers. ASCE, Journal of Hydraulic Engineering. 130(2): 91-100.

Sharp, J.J. 1981. Hydraulic modelling, Butterworth & Co., Canada.

Soulsby, R.L., Damgaard, J.S. 2005. Bedload sediment transport in coastal waters. Coastal Engineering. 52(8): 673-689.

Stieve, M.J.F. 1985. “A Scale Comparison of Waves Breaking on a Beach,” Coastal Engineering, Vol. 9, pp 151-158.

Svendsen, I.A. 1985. “Physical Modelking of Water Waves,” in Physical Modelling in Coastal Engineering, R.A. Dalrymple, Ed., A.A. Balkema, Rotterdam, The Netherlands, pp 13-47

Warnock, J.E. 1950. “Hydraulic Similtude,” in Engineering Hydraulics, edited by H. Rouse, John Wiley & Sons, New York, pp 136-176.

Washburn, L., Jones, B.H. 1996. Field observations and modelling of dispersion processes around ocean outfalls. Ocean Conference Record (IEEE). 1: 307-312.

Yalin, M.S. 1989. "Fundamentals of Hydraulic Physical Modelling," in Recent Advances in Hydraulic Physical Modelling, R. Martins, Ed., Kluwer Academic Publishers, Doesrecht, The Netherlands, pp 567-588

Yalin, M.S. 1977. Mechanics of sediment transport – 2nd edition, Pergamon Press, Toronto, Ontario, Canada.