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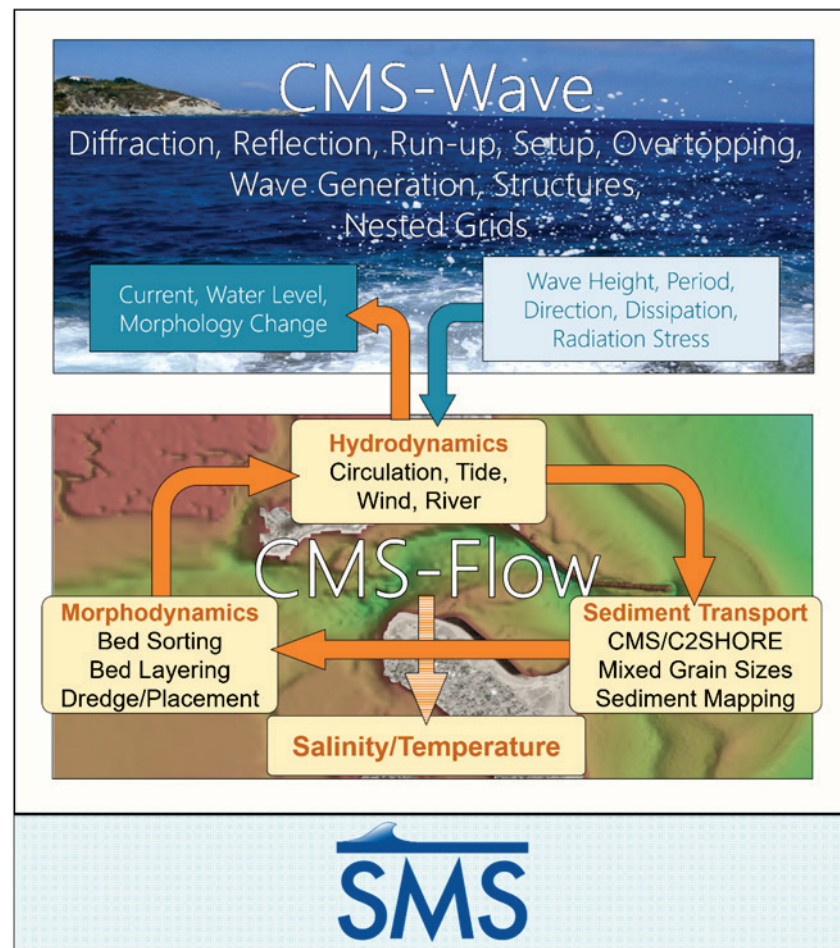


Coastal Inlets Research Program

A Revisit and Update on the Verification and Validation of the Coastal Modeling System (CMS)

Report 1—Hydrodynamics and Waves

Honghai Li, Lihwa Lin, Cody L. Johnson, Yan Ding, Mitchell E. Brown, September 2022
Tanya M. Beck, Alejandro Sánchez, and Weiming Wu



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A Revisit and Update on the Verification and Validation of the Coastal Modeling System (CMS)

Report 1–Hydrodynamics and Waves

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Report 1 of 2

Approved for public release; distribution is unlimited.

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Under Funding Account Code U430420; AMSCO Code 060000

Abstract

This is the first part of a two-part report that revisits and updates the verification and validation (V&V) of the Coastal Modeling System (CMS). The V&V study in this part of the report focuses on hydrodynamic and wave modeling. With the updated CMS code (Version 5) and its latest graphical user interface, the Surface-water Modeling System (Version 13), the goal of this study is to revisit some early CMS V&V cases and assess some new cases on model performance in coastal applications. The V&V process includes the comparison and evaluation of the CMS output against analytical solutions, laboratory experiments in prototype cases, and field cases in and around coastal inlets and navigation projects. The V&V results prove that the basic physics incorporated are represented well, the computational algorithms implemented are accurate, and the coastal processes are reproduced well. This report provides the detailed descriptions of those test simulations, which include the model configuration, the selection of model parameters, the determination of model forcing, and the quantitative assessment of the model and data comparisons. It is to be hoped that, through the V&V process, the CMS users will better understand the model's capability and limitation as a tool to solve real-world problems.

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Preface

This study was conducted by the Coastal and Hydraulics Laboratory (CHL) of the US Army Engineer Research and Development Center (ERDC) under the Coastal Inlets Research Program, Funding Account Code U430420; AMSCO Code 060000. The technical monitor was Dr. Tanya M. Beck.

The work was performed by the Coastal Engineering Branch (CEB) of the Navigation Division, ERDC-CHL. At the time of publication of this report, Ms. Lauren M. Dunkin was chief of CEB; Ms. Ashley Frey was chief of the Navigation Division; and Mr. Charles E. Wiggins was the technical director for Navigation Research and Development. The deputy director of ERDC-CHL was Mr. Keith W. Flowers, and the director was Dr. Ty V. Wamsley.

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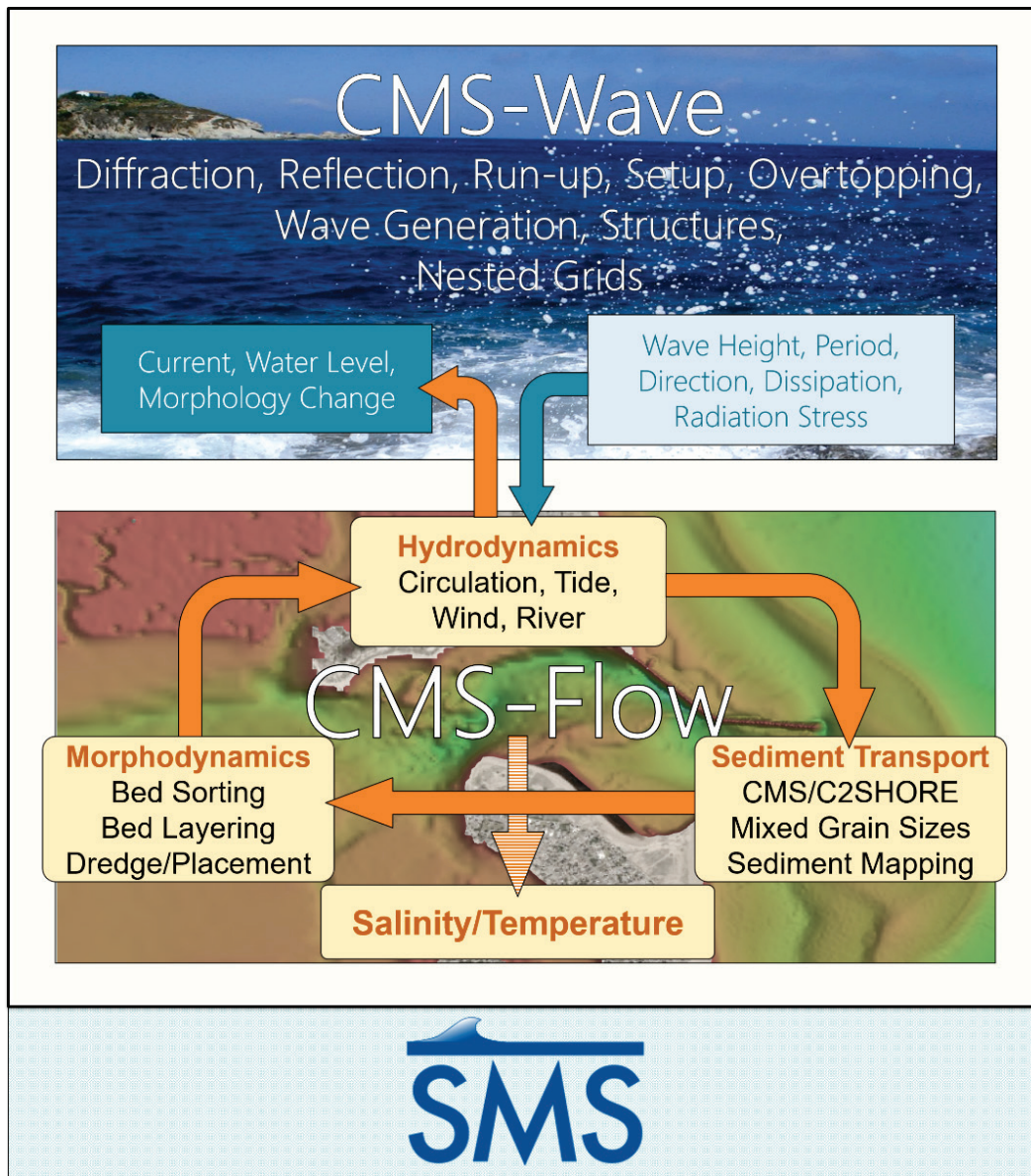
1 Introduction

The Coastal Modeling System (CMS) is an integrated numerical modeling system for simulating nearshore waves, currents, water levels, sediment transport, and morphology change (Militello et al. 2004; Buttolph et al. 2006a; Lin et al. 2008; Reed et al. 2011; Lin et al. 2011a; Sánchez and Wu 2011; Sánchez et al. 2011; Sánchez et al. 2014). The system is designed for coastal inlets and navigation applications including channel performance and sediment exchange between inlets and adjacent beaches. Modeling provides planners and engineers with essential information for improving the usage of US Army Corps of Engineers (USACE), Operation and Maintenance Funds. The Coastal Inlets Research Program (CIRP) is developing, testing, improving, and transferring the CMS to USACE districts and industry, and assisting users in engineering studies.

1.1 Background

The CMS includes a flow model (CMS-Flow), which calculates hydrodynamics and sediment transport, and a wave model (CMS-Wave). These models are coupled together within the Surface-water Modeling System (SMS), which provides a graphical user interface (Aquaveo 2021). The overall framework of the CMS and its components are presented in Figure 1.

Figure 1. The CMS framework.



CMS-Flow is a two-dimensional depth-averaged nearshore circulation model. It calculates currents, water levels, and physical processes such as advection, turbulent mixing, combined wave-current bottom friction, Coriolis force, and the influence of coastal structures (Buttolph et al. 2006a; Wu et al. 2011a,b). CMS-Flow also calculates salinity, temperature, sediment transport, and morphology change as discussed in Sánchez et al. (2011), and Li and Brown (2017). CMS-Wave is a spectral wave transformation model and solves the wave-action balance equation using a forward marching Finite Difference Method (Mase et al. 2005; Lin et al. 2008). CMS-Wave includes physical processes such as wave shoaling,

refraction, diffraction, reflection, wave-current interaction, wave breaking, wind wave generation, white capping of waves, and the influence of coastal structures. The CMS takes advantage of the SMS interface versions 8.2 through 13.1 for grid generation, model setup, plotting, and post-processing of modeling results. The SMS also provides a link between the CMS and the Lagrangian Particle Tracking Model (MacDonald et al. 2006).

There are two independent numerical solvers in early released versions of the CMS (version 4 and earlier), the implicit solver and the explicit solver. The implicit solver uses the SIMPLEC algorithm (Van Doormaal and Raithby 1984) on a non-staggered grid to handle the coupling of water level and velocity. Primary variables u -, v -velocity, and water level are stored on the same set of grid points. Fluxes at cell faces are determined using a Rhie and Chow (1983) type momentum interpolation method (Wu et al. 2011a,b). The explicit solver uses a staggered grid with velocities at the cell faces and the water levels and water depths at the cell centers (Buttolph et al. 2006a). Recently, the two solvers have been merged into one computational code in the new released version of the CMS (version 5).

1.2 Objective

Early verification and validation (V&V) of the CMS were conducted a decade ago (Demirbilek and Rosati 2011). With the release of the merged CMS code and the updated SMS, a new round of model V&V becomes necessary. The process checks the numerical implementation of the governing equations and determines to what degree the model accurately represents physical processes in real world applications.

Through the V&V process, a comprehensive and quantitative assessment of the CMS modeling is performed using a wide range of idealized, laboratory, and field test cases. All the idealized and laboratory experimental test cases are selected from the early V&V study (Lin et al. 2011b; Sánchez et al. 2011), and the basic physics and computational algorithms implemented in the CMS are examined. Most of the field cases are selected from coastal studies conducted by USACE in recent years.

The V&V report consists of two parts. Part I focuses on hydrodynamic and wave modeling and Part II on sediment transport modeling. This is Part I of the two-part reports, which includes study purpose, model setup, and the assessment of the results for each test case. Major findings and default

parameters are provided as guidance to users for practical applications of the CMS. A data package of all test cases will be posted to the CIRP website <https://cirp.usace.army.mil/CMS> with the publication of the report.

1.3 Approach

Three categories of data sources are used in this V&V study to evaluate the performance of the CMS:

- Category 1: Verification with analytical/empirical solutions
- Category 2: Validation with data from laboratory experiments
- Category 3: Validation with data from field studies.

Typical applications of the CMS include the analyses of navigation channel performance; wave, current, and wave-current interaction in channels and in the vicinity of navigation structures; and sediment management transport issues around coastal inlets and adjacent beaches. Some examples of CMS applications are Batten and Kraus (2006), Buttolph et al. (2006b), Zarillo and Brehin (2007), Li et al. (2009), Beck and Kraus (2010), Byrnes et al. (2010), Rosati et al. (2011), Reed and Lin (2011), Wang and Beck (2011), Watts et al. (2011), Beck and Legault (2012), Li et al. (2015), Sánchez et al. 2016, Lin et al. (2018), Demirbilek et al. (2019), Li et al. (2019), and Li (2021).

1.4 Report organization

This report is organized into five chapters. Chapter 1 presents an overview of the CMS V&V study. Chapter 2 discusses the verification of the CMS with analytical solutions and idealized cases (Category 1). Chapters 3 and 4 present the validation of the CMS against laboratory (Category 2) and field (Category 3) data, respectively. In Chapters 2 – 4, test cases are identified by Category “C” and Example number “Ex” as C1-Ex1, etc. Chapter 5 summarizes the study and provides recommendations for CMS users. The appendix provides a description of the goodness-of-fit statistics applied herein.

2 Analytical Solutions and Idealized Examples

2.1 Overview

This section evaluates the performance of CMS for modeling the fundamental process of estuary, riverine, coastal free-surface hydrodynamics. The evaluation is carried out by comparing the CMS computed solutions to situations that have been simplified to allow for direct analytical solutions to the primitive variables of the nonlinear shallow water equations. The cases are labeled based on category and example (e.g., the first test case is labeled C1-Ex1 to denote that it is the first example in the Category 1).

The test cases by category are as follows:

1. Wind-stress driven setup in a flat basin
2. Wind-driven flow in a circular basin
3. Tidal propagation in a quarter annulus
4. Transcritical flow over a bump
5. Long-wave runup over a frictionless slope
6. Wave generation and growth in limited fetch
7. Nonlinear wave-wave interactions
8. An idealized infinite wedge.

2.2 Test C1-Ex1: Wind setup in a flat basin

2.2.1 Purpose

This test is a basic validation of the CMS ability to produce stable steady-state solutions. The governing equations are simplified to express a balance between the water surface gradient and wind stress terms. The specific model features to be tested are the following:

- Spatially and temporally constant wind field input
- Water surface gradient formulation
- Land-water boundaries.

2.2.2 Problem and analytical solution

This case assumes a closed basin and that non-linearity, bottom friction, diffusion, surface waves, and Coriolis force may be neglected. This reduces the momentum equation to a balance between water surface gradient and wind stress, expressed as

$$\rho g h \frac{\partial \eta}{\partial y} = \rho_a C_d |W| W \quad (1)$$

where g is the acceleration due to gravity, ρ is the water density, $h = \zeta + \eta$ is the total water depth or water column thickness, ζ is the still water depth, η is the water surface displacement from the still water level, C_d is the drag coefficient, y is the coordinate of the axis along which the wind is blowing, ρ_a is the air density, and W is wind speed. This equation models a steady wind blowing across a large body of water such as a lake or continental shelf, and assuming a constant drag coefficient, may be solved for η by definite integration (Dean and Dalrymple 1984) as

$$\eta = \sqrt{\frac{2\rho_a C_d |W| W}{\rho g} (y + C) + \zeta^2} - \zeta \quad (2)$$

where C is a constant of integration that can be calculated by setting $\eta = 0$ at the axis' mid-point. For wind speeds less than 30 m/sec^{*†}, the CMS drag formulation is given as

$$C_d = \left(\frac{\kappa}{14.56 - 2 \ln W} \right)^2 \quad (3)$$

where κ is the von Kármán constant (Hsu 1988).

* For a full list of the spelled-out forms of the units of measure used in this document, please refer to *US Government Publishing Office Style Manual*, 31st ed. (Washington, DC: US Government Publishing Office 2016), 248-52, <https://www.govinfo.gov/content/pkg/GPO-STYLEMANUAL-2016/pdf/GPO-STYLEMANUAL-2016.pdf>.

† For a full list of the unit conversions used in this document, please refer to *US Government Publishing Office Style Manual*, 31st ed. (Washington, DC: US Government Publishing Office 2016), 345-7, <https://www.govinfo.gov/content/pkg/GPO-STYLEMANUAL-2016/pdf/GPO-STYLEMANUAL-2016.pdf>.

2.2.3 Model setup

To verify this simplified case, a constant water depth of 5 m is defined on a regular Cartesian grid with irregularly shaped boundaries. The computational grid has 60 columns and 70 rows with a constant resolution of 500 m. The irregular geometry is used to check for discontinuities near the land-water boundaries. Under the model's simplifications, the water surface solution should be symmetric about the axis along the wind's direction and linearly increase with that axis' coordinate. The water levels and currents are initialized at a value of zero and the steady-state solution is achieved by running the simulation for 48 hr. Table 1 lists model setup details.

Table 1. CMS-Flow settings for wind setup case.

Parameter	Value
Numerical Scheme	Implicit
Time-step	600 sec
Simulation Duration	48 hr
Wind Speed	10 m/sec
Drag coefficient	0.00161
Mixing terms	Off
Bottom friction	Off
Wall friction	Off
Coriolis effect	Off

2.2.4 Results and discussion

The numerically integrated water surface elevation (WSE) solutions for a northern and western wind are shown in Figures 2 and 3, respectively. The x-markers indicate sample points for a comparison with the analytical solution shown in Figures 4 and 5 for north and west winds, respectively. To quantify the skill of the model, the numerical solution at all points in the domain is compared to the analytical solution taken as the reference value. The Normalized Root Mean Square Error (NRMSE), Root Mean Square Error (RMSE), Normalized Mean Absolute Error (NMAE), squared correlation coefficient (R^2), and Bias are given in Table 2.

Figure 2. Numerical solution for water surface setup from northern wind.

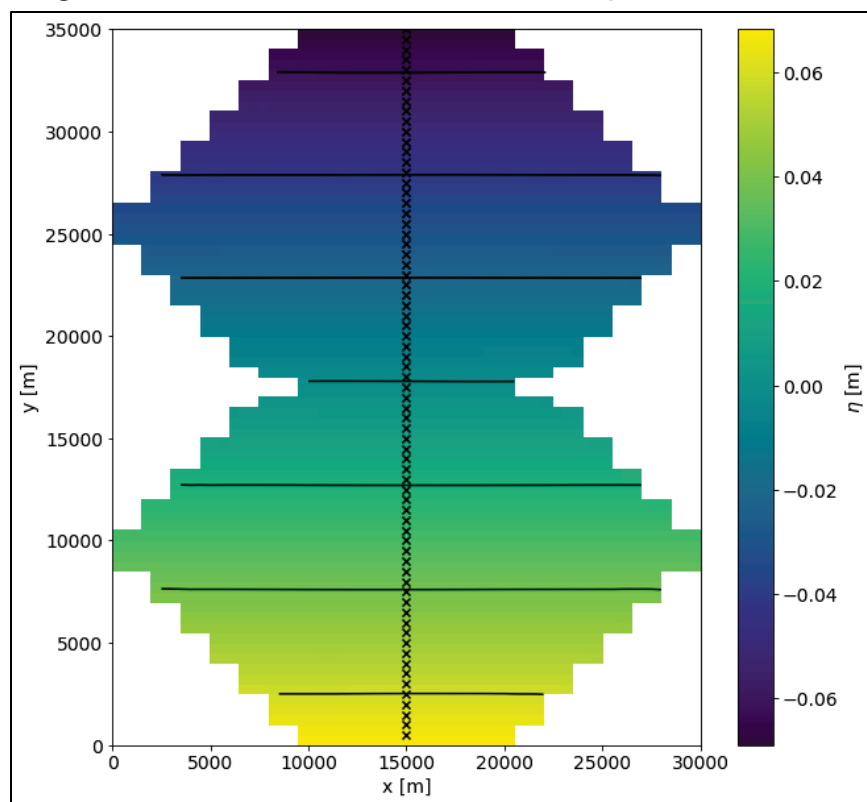


Figure 3. Numerical solution for water surface setup from western wind.

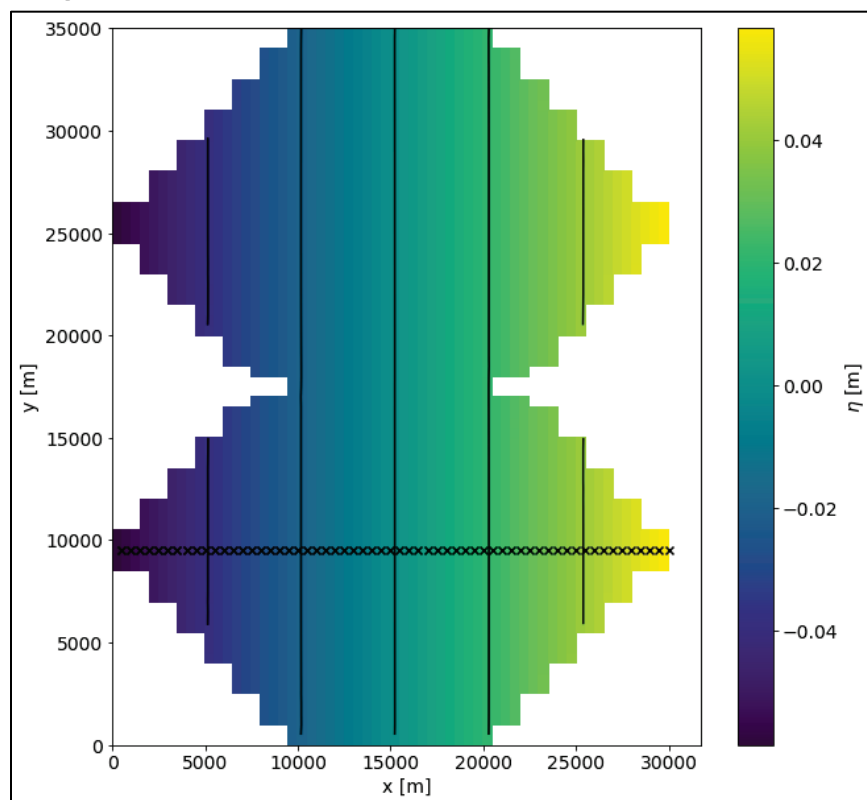


Figure 4. Numerical and analytical solution for water surface setup from northern wind.

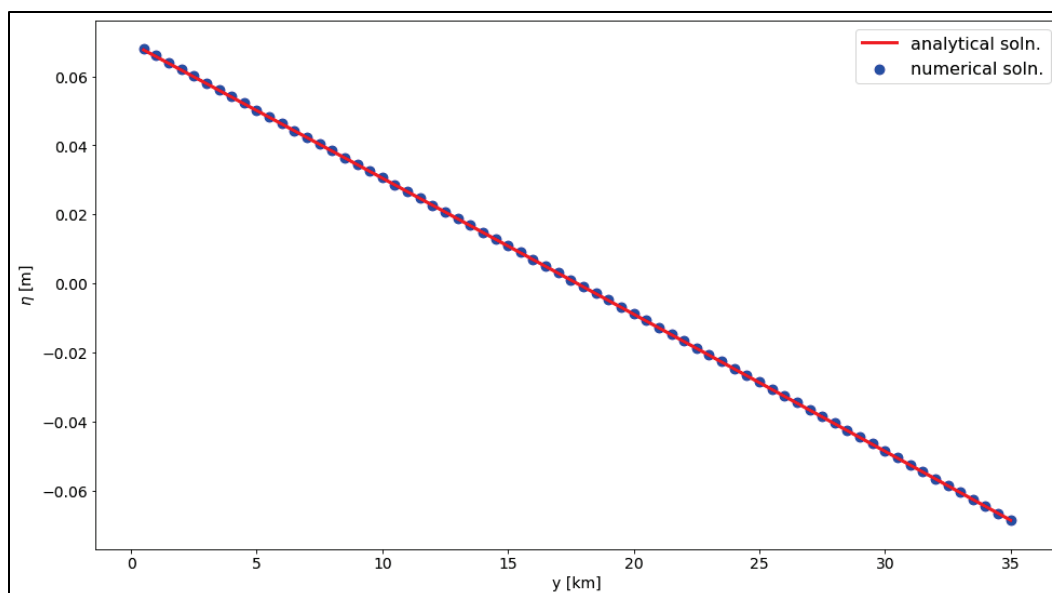


Figure 5. Numerical and analytical solution for water surface setup from western wind.

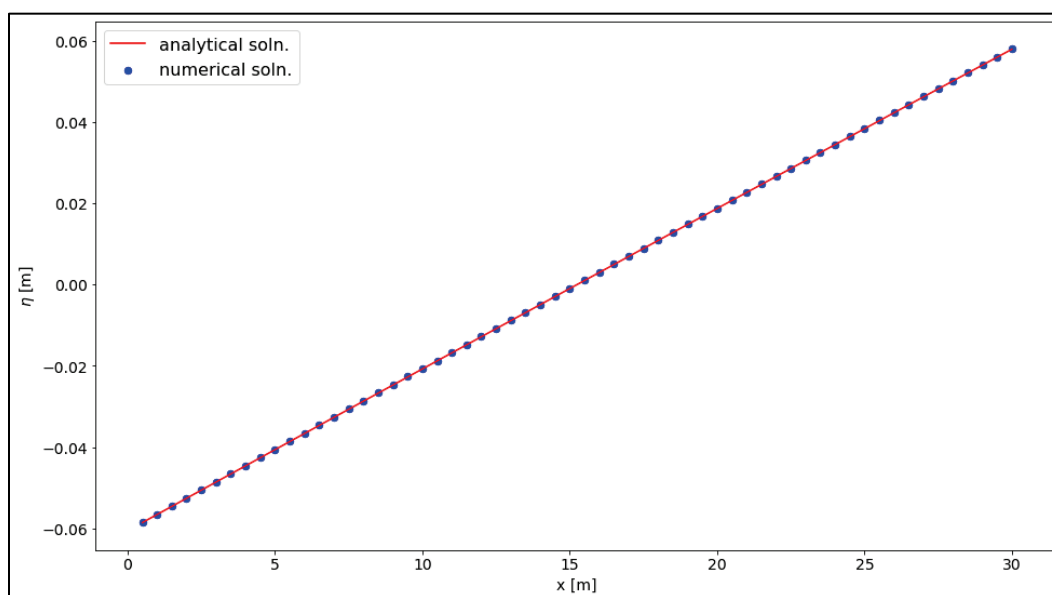


Table 2. Model accuracy statistics for numerical solution.

Statistics	Northern Wind	Western Wind
NRMSE [-]	0.00109	0.00061
RMSE [m]	0.00015	0.00007
NMAE [-]	0.00425	0.00314
R^2 [-]	0.99998	0.99999
Bias [m]	0.00014	0.00070

2.2.5 Conclusions and recommendations

The steady-state solution for an idealized wind-driven setup with flat bed and irregular geometry was simulated, and the model's performance was assessed with the model skill statistics. The model skill statistics show excellent agreement with the analytical solution with $\text{NRMSE} < 0.1\%$, $\text{NMAE} < 0.4\%$, and $R^2 > 0.9999$. These cases demonstrate the CMS capability to balance wind stress and water surface gradient forcing within steady-state solutions.

2.3 Test C1-Ex2: Wind-driven flow in a circular basin

2.3.1 Purpose

This case is designed to verify a steady-state linear hydrodynamics when forced with spatially variable winds, damped with a linear bottom friction, and including the Coriolis force.

2.3.2 Problem

The governing equations are linearized by neglecting advection and diffusion while also employing a linear bottom friction term instead of the more commonly used quadratic formulation. Further, atmospheric pressure gradients and wave-induced stresses are neglected, yielding an analytically solvable system, expressed as

$$\frac{\partial(hU_j)}{\partial x_j} = 0 \quad (4)$$

$$-\epsilon_{ij}f_c hU_j = -gh \frac{\partial \eta}{\partial x_i} + \tau_i^s - \tau_i^b \quad (5)$$

where i, j indicate the Cartesian axes, ϵ is the permutation parameter ($\epsilon = 0$ for $i = j$; $\epsilon = 1$ for $i, j = 1, 2$; and $\epsilon = -1$ for $i, j = 2, 1$), f_c is the Coriolis frequency, U_j is the depth-averaged current velocity, τ_i^s is the wind stress at the free surface, and τ_i^b is bottom friction. If the water surface displacement is small compared to the depth (i.e., $h \gg \eta$, then h may be considered constant). After substitution of linear bottom friction, $\tau_i^b = \hat{\kappa} h U_i$ where $\hat{\kappa}$ is the linear bottom friction coefficient, and a gradient wind expression for the wind stress term, $\tau_i^s = \epsilon_{i2} x_j W / R$, where W is the wind speed gradient and R is the radius of the circular basin, the governing equations become

$$\frac{\partial U_i}{\partial x_i} = 0 \quad (6)$$

$$-\epsilon_{ij} f_c U_j = -g \frac{\partial \eta}{\partial x_i} - \hat{\kappa} U_i + \epsilon_{i2} \frac{x_2 W}{Rh} \quad (7)$$

2.3.3 Analytical solution

Dupont (2001) presented the analytical solution in circular coordinates, which, transformed to Cartesian coordinates, are given as

$$\eta = \begin{cases} \frac{W x_1 x_2}{2ghR}, & f_c = 0 \\ \frac{W f_c}{gh\hat{\kappa}R} \left[\frac{R^2}{8} + \frac{1}{4} \left(\frac{2\hat{\kappa} x_1 x_2}{f_c} - x_1^2 - x_2^2 \right) \right], & f_c \neq 0 \end{cases} \quad (8)$$

The current velocity fields, which do not depend on the Coriolis frequency, are given by

$$U_i = \frac{\epsilon_{ij} x_j W}{2h\hat{\kappa}R} \quad (9)$$

2.3.4 Model setup

A circular basin with a radius of 50 km is set up with a quadtree computational grid (Sanchez et al. 2011). The grid resolution varies from 2000 m in the basin's interior to 125 m near the land boundary with a total of 22,144 cells (Figure 6). Steady-state solutions are achieved by running the simulation for 216 hr with an initial ramp of 1 hr. Both cases with and without the Coriolis Effect are tested. (See Table 3 for relevant model parameters.)

Figure 6. Quadtree computational grid for circular basin.

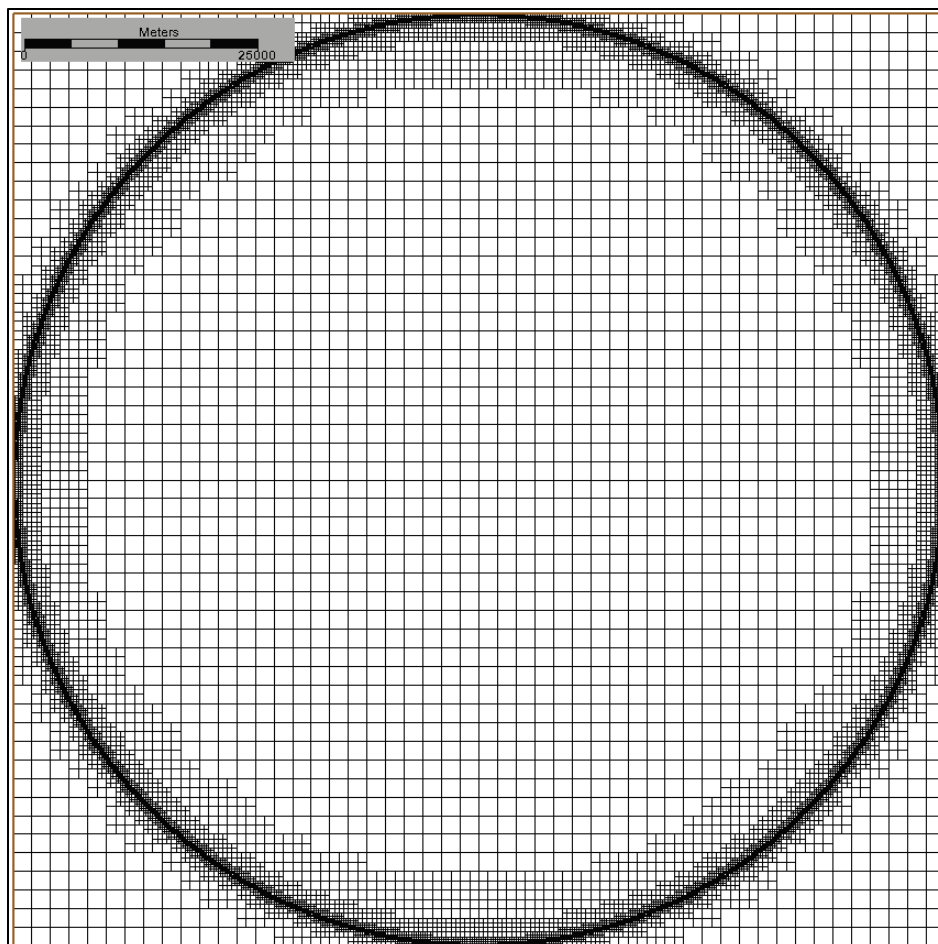


Table 3. CMS-Flow settings for wind setup case.

Parameter	Value
Numerical Scheme	Implicit
Time-step	1800 sec
Simulation duration	216 hr
Ramp duration	1 hr
Water depth	100 m
Linear bottom friction coefficient	0.01
Coriolis frequency	1.0e-4 Hz
Wind gradient	1.0e-4 m ² sec ⁻²
Mixing terms	Off
Advection terms	Off
Wall friction	Off

2.3.5 Results and discussion

2.3.5.1 Without Coriolis force

The analytical and numerical solutions for the steady-state current field and water level for the case without the Coriolis force is shown in Figure 7. The circulation and water level setup calculated with the analytical solution is accurately reproduced in the numerical results. The accuracy assessment statistics are given in Table 4. The numerical water levels show excellent agreement with the analytical solution. The numerical current velocities, however, while still matching analytical solution, show some disagreement and exhibit NRMSE values near 10%.

Figure 7. Water levels and current fields for numerical (left) and analytical (right) solutions without the Coriolis force.

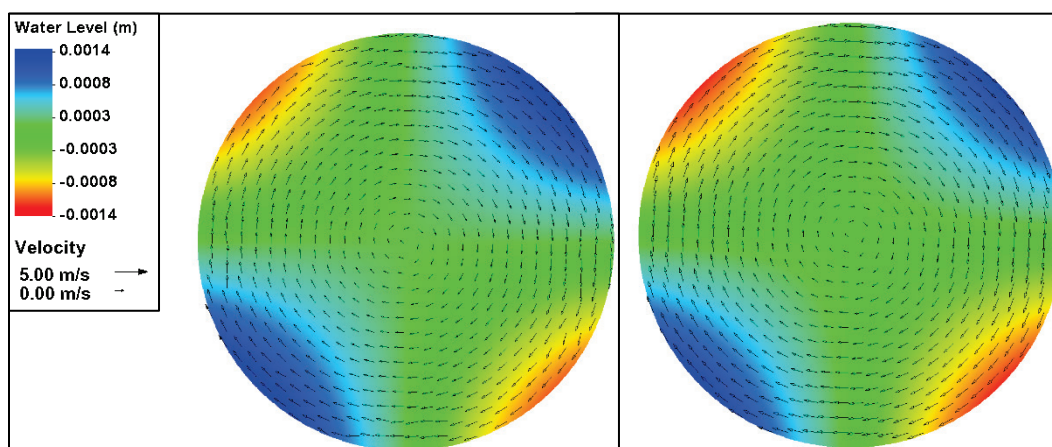


Table 4. Model accuracy statistics for no Coriolis force case.

Statistics	Water Level	u velocity	v velocity
NRMSE [-]	0.00994	0.10924	0.12404
RMSE [m or ms ⁻¹]	0.00003	0.00001	0.00001
NMAE [-]	0.03340	0.00060	-0.00047
R^2 [-]	0.99910	0.91418	0.89189
Bias [m or ms ⁻¹]	0.00025	0.00000	0.00000

2.3.5.2 With the Coriolis force

A comparison between numerical and analytical solutions for the case with the Coriolis force is given in Figure 8 and the accuracy statistics are given in Table 5. The numerical results for the water level compare very

well to the analytical solution but performs slightly worse than the case without the Coriolis forces. Since the current fields are independent of the Coriolis force, the numerical results for currents are nearly identical to the case without the Coriolis force. The Coriolis frequency used corresponds to the Coriolis effect within the northern hemisphere, which results in a rightward deflection of currents. Consequently, there is net transport of water towards the center of the circular basin, resulting in a water level setup given by $\eta = \frac{Wf_c R}{8gh\hat{\kappa}}$, whereas $\eta = 0$ for the case without the Coriolis force.

Figure 8. Water levels and current fields for numerical (left) and analytical (right) solutions with the Coriolis force.

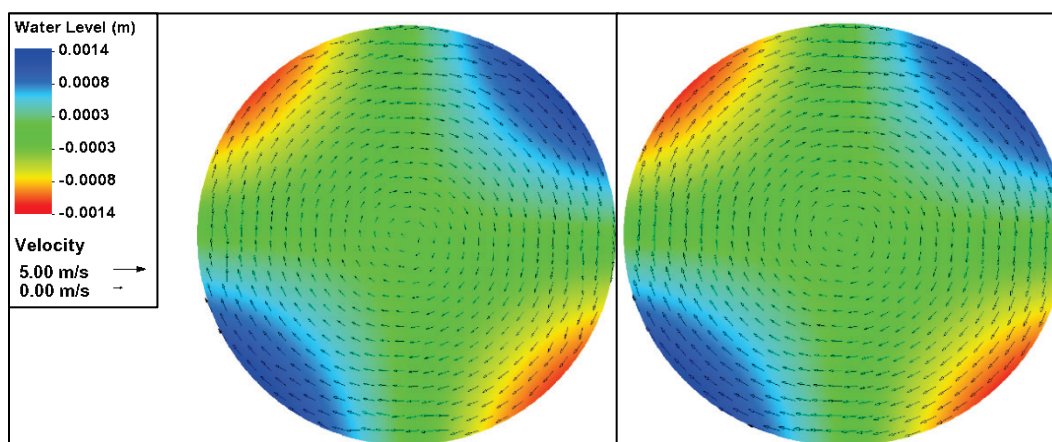


Table 5. Model accuracy statistics for with Coriolis force case.

Statistics	Water Level	u velocity	v velocity
NRMSE [-]	0.04523	0.10934	0.12401
RMSE [m or ms ⁻¹]	0.00012	0.00001	0.00001
NMAE [-]	0.15660	0.00062	-0.00051
R^2 [-]	0.98086	0.91408	0.89185
Bias [m or ms ⁻¹]	0.00012	0.00000	0.00000

2.3.6 Conclusions and recommendations

The numerical simulations of the steady-state linearized hydrodynamics due to wind forcing within a circular basin are compared to the analytical solution, both with and without the Coriolis force. In general, the model accuracy statistics demonstrates good agreement between numerical results and the analytical solution.

2.4 Test C1-Ex3: Tidal propagation in a quarter annulus

2.4.1 Purpose

This verification test assesses the CMS ability to simulate long wave (tidal) propagation and reproduce results from a linearized version of the shallow water equations. The case also tests performance and symmetry in a circular domain with curved boundaries and uniform tidal forcing. Bottom friction and turbulent mixing are not included and therefore numerical dissipation is only introduced by the numerical discretization scheme.

2.4.2 Problem

Lynch and Gray (1978) presented a general solution for the linearized shallow water equations in polar coordinates for an annular domain with a sloping bed, bottom friction, and arbitrary tidal forcing on the outer curved boundary. Here, the problem is simplified through using a linearly sloping bed and by neglecting bottom friction and horizontal momentum diffusion. Further, only one tidal constituent is used to impose a uniform water level boundary condition (Figure 9). When the boundary condition is independent from the position on the boundary, then the interior solution is also independent from angular position, and the tidal wave is identical along every radius of the annulus (Table 6).

Figure 9. Computational domain for tidal propagation in a quarter annulus.

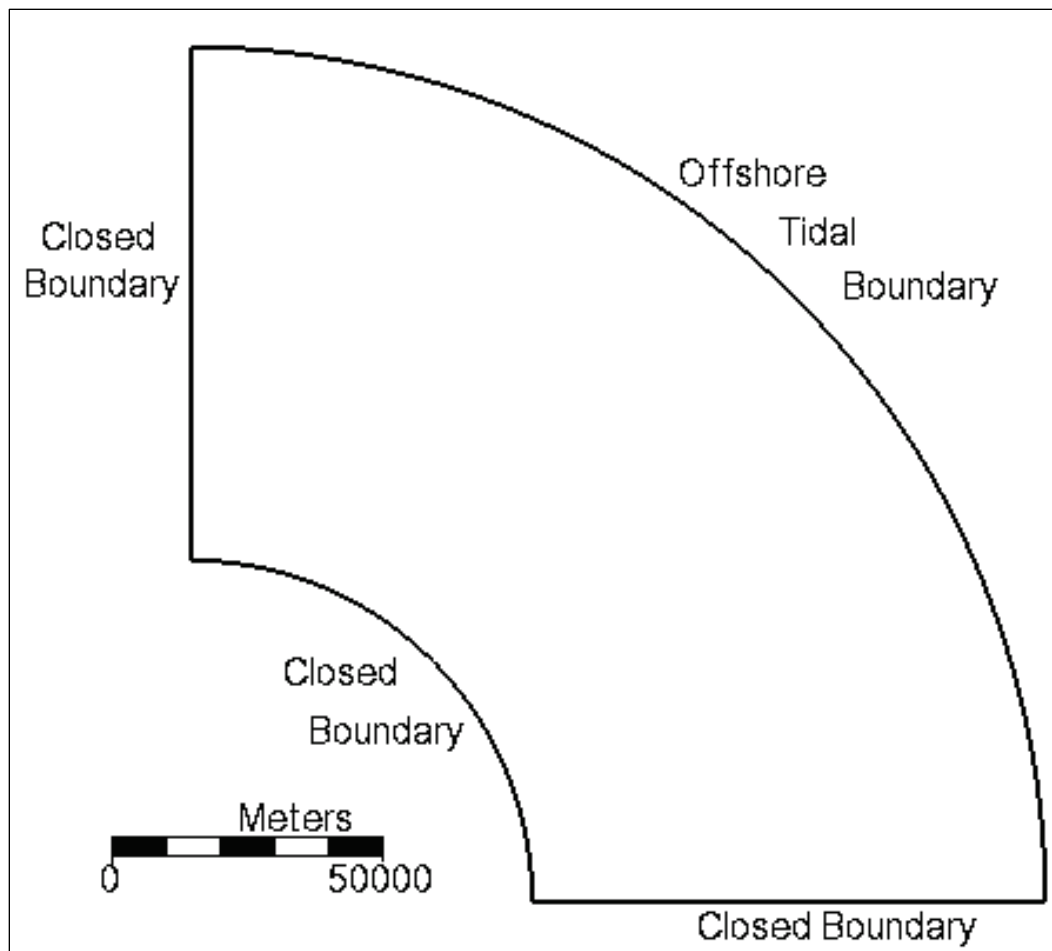


Table 6. Quarter annulus tidal propagation problem definition.

Parameter	Value
Boundary tidal wave amplitude	0.3048 m
Tidal period (M2)	12.42 hr
Annulus inner radius	60.96 km
Annulus outer radius	152.4 km
Annulus inner water depth	10.02 m
Annulus outer water depth	25.05 m
Bottom friction	None
Mixing terms	Off
Coriolis force	Off

2.4.3 Model setup

Three telescoping levels are used to construct the quadtree computational grid (Figure 10) with 4, 2, and 1 km cells used for the grid resolution. The inner and outer boundaries are refined the most while the interior has the coarsest resolution. The computational grid contains a total of 1,160 active cells. Model settings are tabulated in in Table 7.

Figure 10. Computational quadtree grid and depth used for tidal propagation in a quarter annulus.

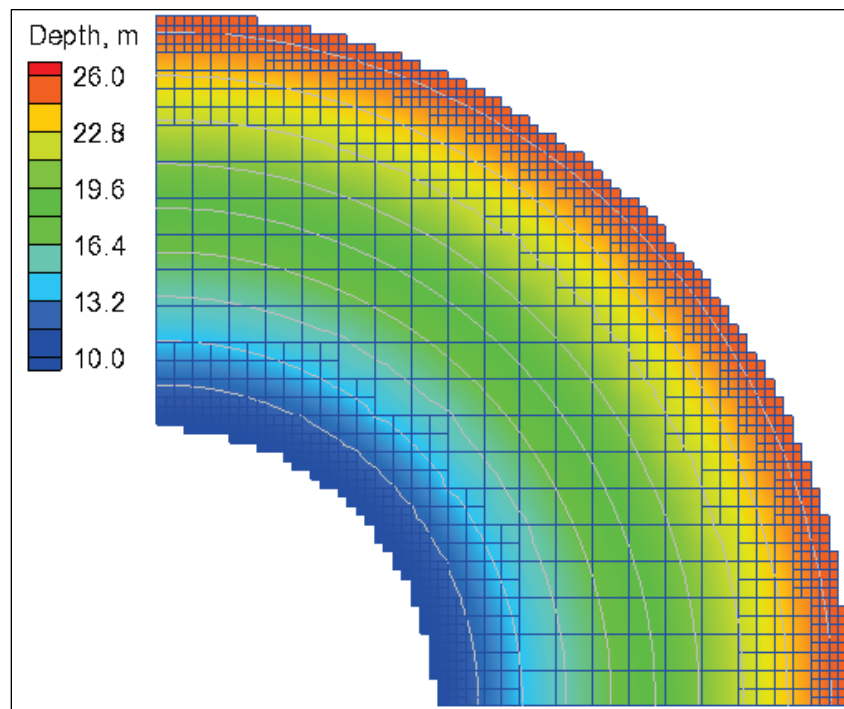


Table 7. CMS-Flow settings for quarter annulus case.

Parameter	Value
Temporal solution scheme	implicit
Time-step	600 sec
Simulation duration	120 hr
Ramp duration	24 hr
Bottom friction	None
Wall friction	Off
Advection terms	Off
Mixing terms	Off
Coriolis force	Off

2.4.4 Results and discussion

Figure 11 shows a comparison between the analytical and numerical solutions for the water level located at the center of the inner edge of the quarter annulus. The goodness-of-fit statistics are listed in Table 8. No significant numerical dispersion, diffusion, or instabilities are evident in Figure 11. The statistics all indicate excellent performance for this case.

Figure 11. Comparison of analytical (blue line) and numerical (orange markers) water surface solution for the center of the annulus' inner boundary.

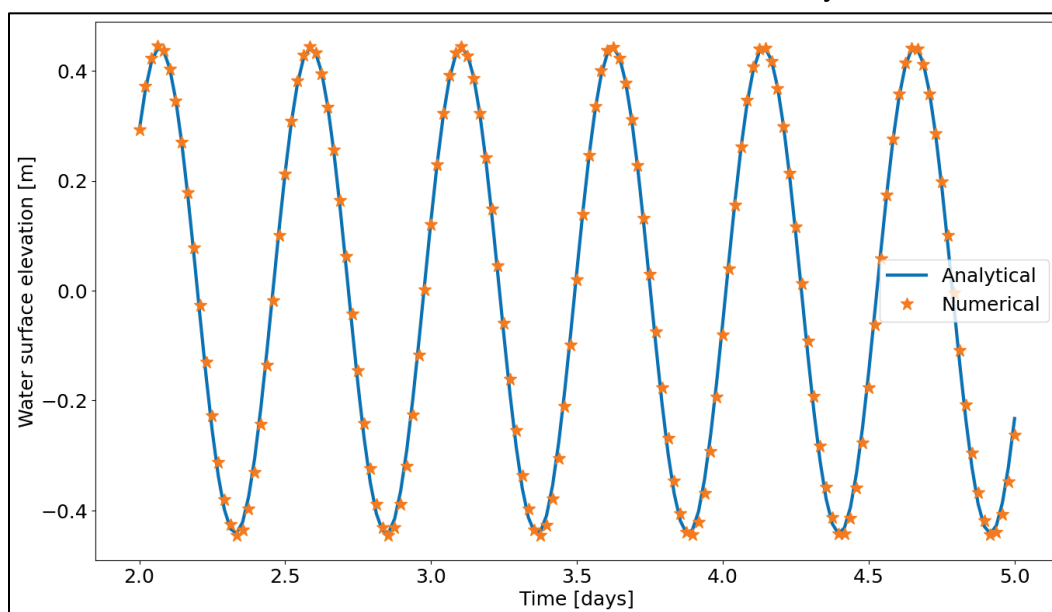


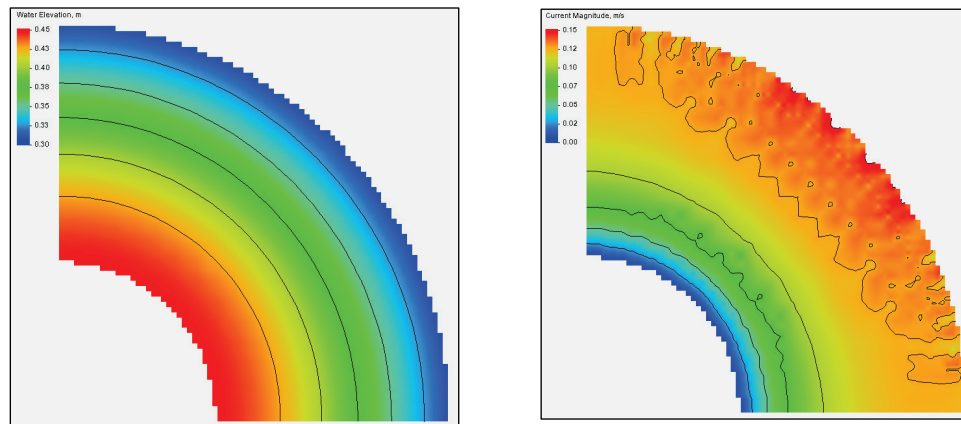
Table 8. Model accuracy statistics for tidal propagation in quarter annulus case.

Statistics	Water Level
NRMSE [-]	0.019
RMSE [m]	0.017
NMAE [-]	0.003
R^2 [-]	0.997
Bias [m]	0.001

Snap shots of the WSE and current magnitude are shown in Figure 12. The contours in the WSE are smooth and radially symmetric, which should be the case because this problem's tidal forcing is independent from angular position. The current magnitude, however, does exhibit errors near the offshore boundary. This is due to the imposing circular boundary conditions on a rectilinear grid approximation to a curve. In practice, however, offshore

boundaries are typically imposed on straight boundaries, and this problem does not frequently occur. This is evidenced by the field cases detailed within this report.

Figure 12. (left) Snapshot of WSE at 62 hr and (right) current magnitude at 65.5 hr. Black lines are contours.



2.4.5 Conclusions and recommendations

CMS-Flow simulates linear long (tidal) wave propagation in a curved domain with sufficient accuracy. The case tested here is simplified in that bottom friction, advection, diffusion, and the Coriolis force are neglected. The water level NRMSE, NMAE, and R^2 were 1.9%, 0.3%, and 0.997, respectively. The curved boundary introduces some inaccuracies into the near-boundary velocity field, which are dissipated further into the domain. However, for practical application, water level boundary conditions should be specified on straight boundaries. If curved boundaries are necessary, then velocities as well as water levels should be imposed on the boundary.

2.5 Test C1-Ex4: Transcritical flow over a bump

2.5.1 Purpose

The test case verifies the ability of the model to simulate the transition from subcritical flow to supercritical flow and back to subcritical flow. The transition from supercritical flow back to subcritical flow contains a hydraulic jump and thus this case also assesses the model's capability of reproducing shocks. Due to the neglect of bottom friction, an analytical solution is available to verify the numerical solution.

2.5.2 Problem

The problem is one dimensional and exhibits a parabolic bump in the bed level defined by

$$z_b = \begin{cases} 0.2 - 0.03(x - 10)^2, & \text{if } 8 \text{ m} < x < 12 \text{ m} \\ 0, & \text{elsewhere} \end{cases} \quad (10)$$

where z_b is the bed elevation with respect to the still water level and x is the horizontal coordinate distance from the inflow boundary. A constant flux is defined at $x = 0$ m, and the water level is held constant at $x = 25$ m. Bottom friction, mixing terms, and wall friction are neglected in this case. (See Table 9 for the relevant hydrodynamic parameters.)

Table 9. CMS-Flow settings for transcritical flow case.

Parameter	Value
Numerical Scheme	Implicit
Time-steps	0.05s -- 0.062 sec
Simulation duration	2 hr
Ramp duration	1 hr
Flow discharge	0.18 m ² sec ⁻¹
Downstream depth	0.33 m
Bottom friction	None
Mixing terms	Off
Wall friction	Off

2.5.3 Analytical solution

The analytical solution to the shallow-water equations with the presence of a shock is found by applying the Bernoulli principal on both sides of the hydraulic jump and locating the hydraulic jump based on a Rankine-Hugoniot relation (Delestre et al. 2014). The system of equations is given by

$$\begin{cases} h^3 + \left(z_b - \frac{q_0^2}{2gh_c^2} - h_c - z_{b,max} \right) h^2 + \frac{q_0^2}{2g} = 0 & \text{for } x < x_{shock} \\ h^3 + \left(z_b - \frac{q_0^2}{2gh_L^2} - h_L \right) h^2 + \frac{q_0^2}{2g} = 0 & \text{for } x > x_{shock} \\ q_0^2 \left(\frac{1}{h_1} - \frac{1}{h_2} \right) + \frac{g}{2} (h_1^2 - h_2^2) = 0 \end{cases} \quad (11)$$

where q_0 is the inflow boundary condition, h_c is the critical depth for supercritical flow, $z_{b,max}$ is maximum elevation of the bed level (i.e., the bump's crest), h_L is the depth at the outflow water level boundary condition, h_1 is the depth approaching the shock from the left, and h_2 is the depth approaching the shock from the right. The Cardan formula is used to solve the above cubic equations for h at all x points in the domain, and the real solution which is closest to its neighbor is selected.

2.5.4 Model setup

The implicit numerical scheme is chosen and applied to a $25 \text{ m} \times 0.3 \text{ m}$ domain with a constant grid spacing of 0.1 m . The central line of grid points is used for comparison with the analytical solution. A steady flux boundary condition is used at the upstream boundary and the water level is pinched at the downstream. The model is ramped from initial conditions of $h = 0.33 \text{ m}$ and $q = 0 \text{ m}^2/\text{sec}$ over the course of an hour. (See model setup parameters in Table 9.)

2.5.5 Results and discussion

The comparison between the analytical and the steady-state numerical solutions for water level is shown in Figure 13. The accuracy statistics can be found in Table 10. The NRMSE is approximately 5.4%, and the R^2 is approximately 0.92, indicating that the model performs adequately when simulating the transcritical flow and hydraulic jump. The comparison in water level within Figure 13 indicates that the transition from subcritical flow to supercritical flow is captured well, but that the flow around the hydraulic jump, particularly upstream of the shock, exhibits some noise. The jump in the numerical solution occurs slightly before the jump in the analytical solution and is not as smooth. Further, the water level is very slightly overestimated directly downstream of the jump where the flow returns to subcritical conditions.

Figure 13. Comparison of analytical and numerical solutions for water level in transcritical flow over a bump with a hydraulic jump.

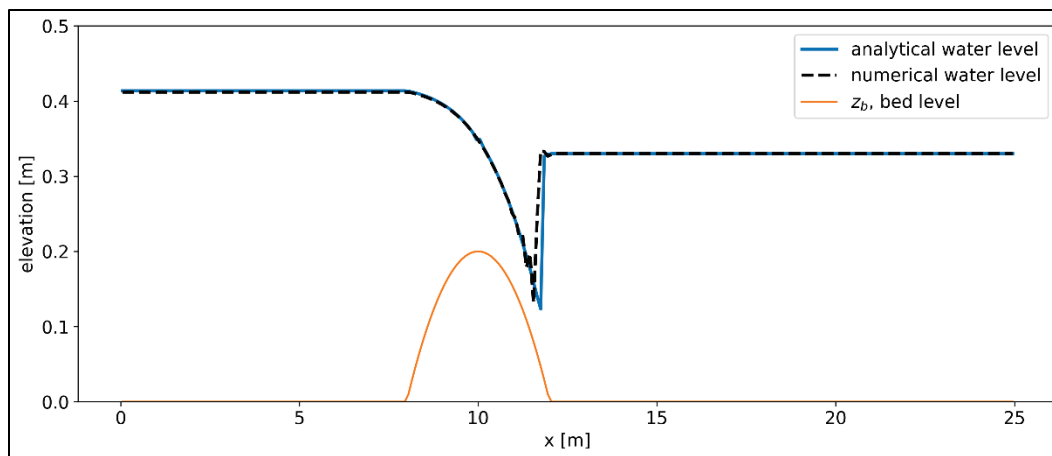


Table 10. Model accuracy statistics for transcritical flow over a bump case.

Statistics	Water Level
NRMSE [-]	0.05388
RMSE [m]	0.01509
NMAE [-]	0.00142
R^2 [-]	0.91535
Bias [m]	0.00051

2.5.6 Conclusions and recommendations

CMS-Flow is verified for simulating transcritical flow over a bump with the presence of a hydraulic jump. The implicit scheme does an adequate job capturing the flow regimes but suffered from slight instabilities in the vicinity of the hydraulic jump. The implicit scheme, however, requires a relatively small time-step to solve for the physics of this problem and therefore loses the computation efficiency usually attributed to implicit schemes. Further, the implicit scheme may be responsible for the discontinuities near the hydraulic jump.

2.6 Test C1-Ex5: Long-wave runup over a frictionless slope

2.6.1 Purpose

The CMS wetting-drying algorithm and long-wave runup simulation are assessed by comparing the computed water level and shoreline position to

the Carrier et al. (2003) analytical solution of a tsunami-like wave running up a frictionless slope.

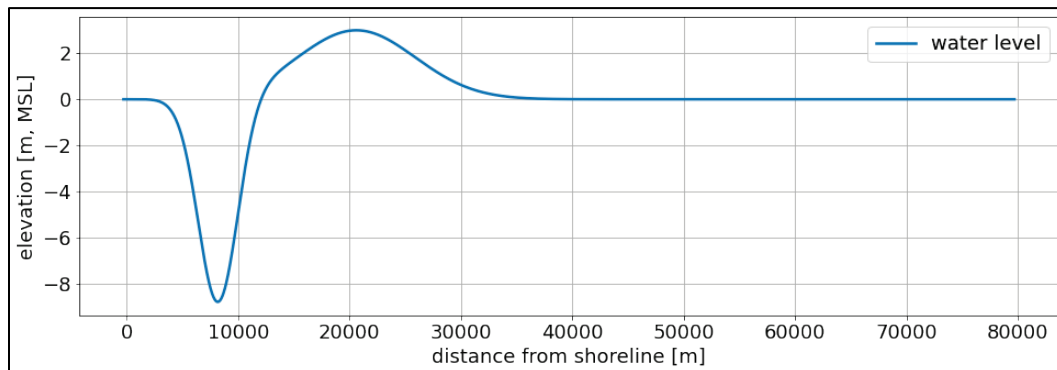
2.6.2 Initial condition

The bed level has a constant 1:10 slope with an initial shoreline position located at $x = 0$ m. The velocity field is initially at rest. The initial water level (see illustration in Figure 14) is a leading depression N-wave (characteristic of the tsunami-like waves generated through submarine landslides) given by a combination of Gaussian humps as

$$\eta(x, 0) = a_1 e^{-k_1(x-x_1)^2} - a_2 e^{-k_2(x-x_2)^2} \quad (12)$$

where, $a_1 = 3$, $a_2 = 9$, $k_1 = 1.7778 \cdot 10^{-8}$, $k_2 = 1.6 \cdot 10^{-7}$, $x_1 = 20604.5$ m, and $x_2 = 8192$ m.

Figure 14. Leading depression N-wave given as initial water level for Test C1-Ex5.



2.6.3 Model setup

The system is solved implicitly with a time-step of 0.1 sec. The small time-step for an implicit solver is required to capture the moving shoreline. The model is essentially one-dimensional with a variable grid size in the x -dimension. The x grid resolution is 3 m for $x < 300$ m and increases smoothly to 10 m at the offshore boundary with a constant aspect ratio of 1.05. The computational grid is stationary, and the wet-dry interface is treated as a moving, internal boundary. Within CMS, a grid cell is deemed dry when the water level is less than a user-specified threshold depth. The interface between wet and dry cells is treated computationally like a closed boundary with wall friction. The relevant model set up parameters are given in Table 11.

Table 11. Model set up parameters for long-wave runup case.

Parameter	Value
Numerical Scheme	Implicit
Time-steps	0.1 sec
Simulation duration	360 sec
Ramp duration	0.0 sec
Wetting/drying depth	1 cm
Wall friction	Off
Bottom friction	Off
Mixing terms	Off

2.6.4 Results and discussion

Figure 15 shows a comparison between the numerical and analytical solutions for the water level relative to the shoreline slope at several times. Very good agreement can be seen between the solutions. The overall shape of the free surface is captured well by CMS as the leading depression N-wave interacts with the frictionless slope (i.e., an initial rundown event is followed by runup of lesser magnitude). The goodness-of-fit statistics, shown in Table 12, quantitatively demonstrate the good performance of CMS in modeling nonlinear long-wave dynamics.

Figure 15. Water level near the shoreline at elapsed times 0 sec, 160 sec, 175 sec, and 220 sec. The numerical solution is presented as the solid line while the analytical solution is given as the broken line.

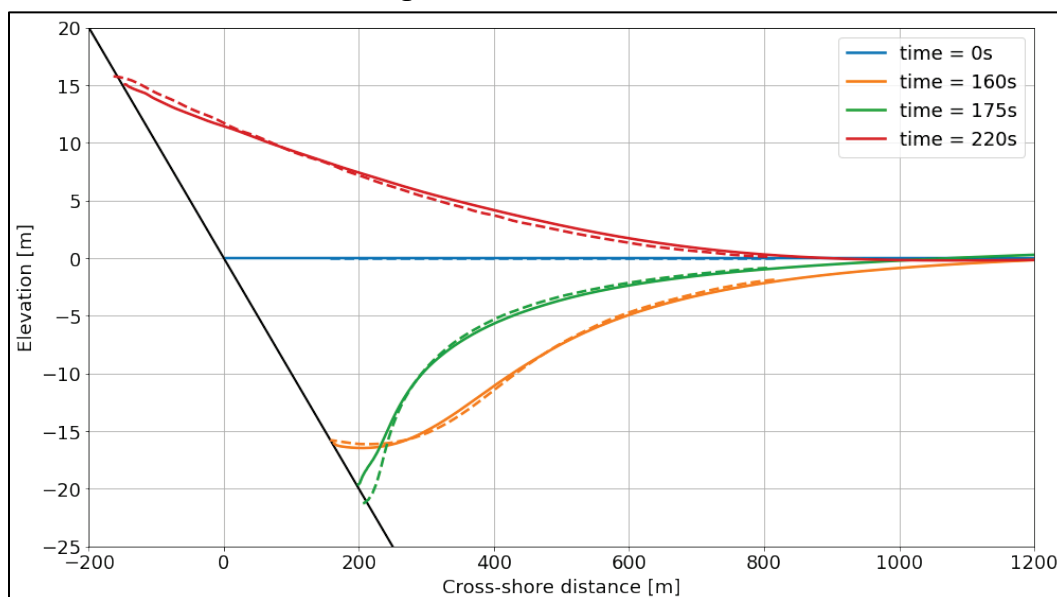
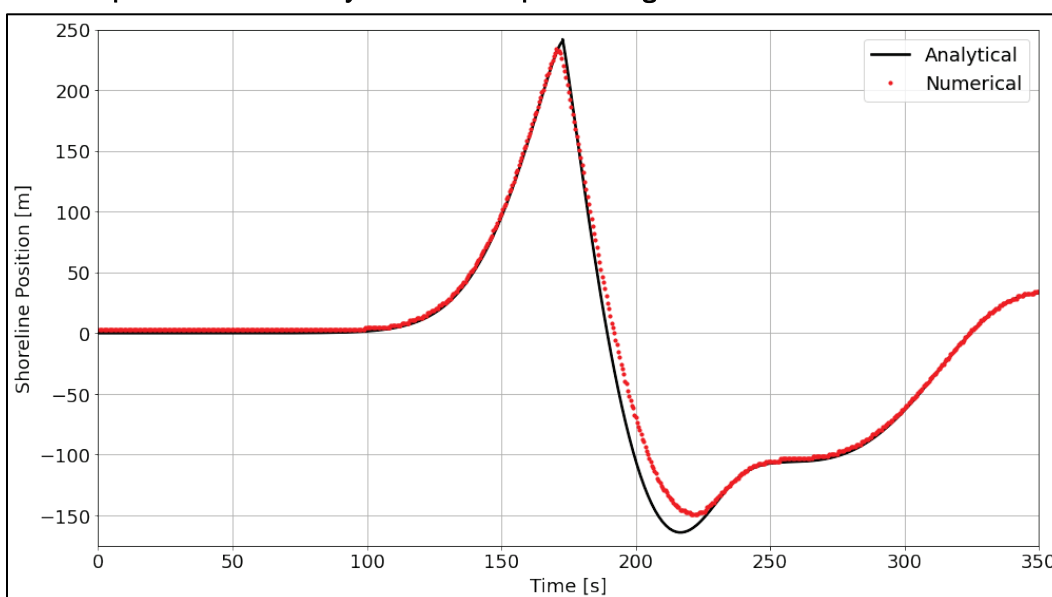


Table 12. Model accuracy statistics for with Coriolis force case.

Statistics	t=160 sec	t=175 sec	t=220 sec
NRMSE [-]	0.017	0.037	0.026
RMSE [m or ms ⁻¹]	0.252	0.679	0.376
NMAE [-]	-0.007	-0.004	0.026
R^2 [-]	0.998	0.983	0.994
Bias [m or ms ⁻¹]	-0.058	-0.020	0.156

The largest discrepancy from the analytical solution occurs after the maximum rundown and increases toward the point of maximum runup. This is illustrated in Figure 16 as a plot of shoreline position as a function of time. This plot shows that CMS misses some areas of inundation as the wave begins to run up the slope. However, the difference is relatively small, and the rest of the computed shoreline position time series matches the analytical solution very well. Note that CMS is a numerical model purposely designed for engineering practice. It specializes in tidal flow, wind wave-current interaction, and sediment transport within inlets and navigable coastal embayment. Wetting/drying is not a critical process for these situations, but nonetheless this test case establishes that CMS is successful at capturing nearshore hydrodynamics, which do depend strongly on this process.

Figure 16. Shoreline position as a function of time. The numerical solution is presented as red points while the analytical shoreline position is given as the solid black line.



2.6.5 Conclusions and recommendations

CMS-Flow performance is tested for nonlinear hydrodynamics in the case of a tsunami-like wave's interaction with a frictionless slope. In particular, the CMS wetting/drying algorithm and implicit scheme's fidelity to nonlinear hydrodynamics are shown to perform well when compared to the analytical solution. This is born out through excellent goodness-of-fit statistics between the analytical and numerical water levels, as well as a comparison of the numerical and analytical shoreline position time series.

2.7 Test C1-Ex6: Wave generation and growth in limited fetch

2.7.1 Purpose

The purpose of this test is to compare CMS-Wave prediction to wave generation and growth curves for fully developed seas given in the Shore Protection Manual (SPM) (USACE 1984) under the fetch-limited condition. The wave generation and growth curves are based on the Sverdrup-Munk-Bretschneider (SMB) method, developed originally in the 1950s for deep-water wave growth and forecasting. The present SMB diagram shown in the SPM has improved for shallow-water constant depth applications.

2.7.2 Model setup and parameters

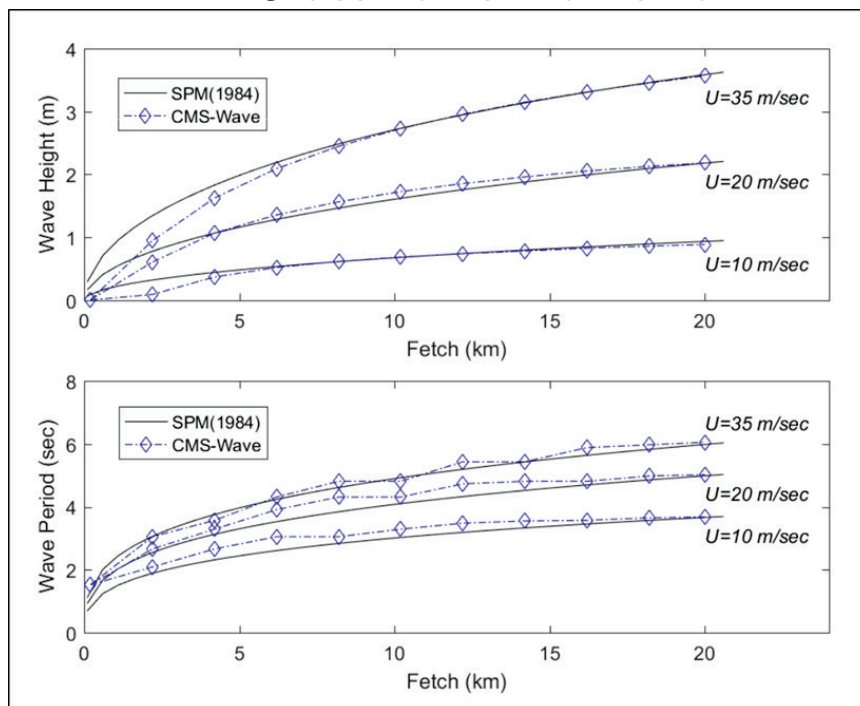
CMS-Wave is set up for a flat-bottom seabed of constant depth 20 m (Lin et al. 2008). The model domain is 2 km $\times$ 20 km consisting of 10 $\times$ 100 cells with constant cell size 200 m $\times$ 200 m. The test cases include a constant wind of 10, 20, and 35 m/sec with wind direction along the long axis (20 km) of the grid. The wave energy input at the upwind boundary of modeling area is set to zero. The wave generation is calculated on a spectral grid of 30 frequency bins (0.12 to 0.35 Hz with 0.008 Hz increment) and 35 direction bins (covering a half-plane with 5° spacing).

2.7.3 Results and discussion

Figure 17 shows comparison of the calculated wave height (top panel) and wave period (lower panel) with results from the SPM. Calculated wave height values agree well in these simulations for fetch greater than 5 km. For a fetch length less than 5 km at high wind speeds, CMS-Wave

predicts lower wave heights than the SPM method, which is not necessarily an error since the SPM method is only a coarse approximation. Over the short fetch lengths considered in this test, calculated wave heights by CMS-Wave increase linearly with the length of fetch, which agrees with the SPM method.

Figure 17. Comparison of calculated wave generation and SMB curves, wave height (top panel) and period (lower panel).



Tables 13 and 14 listed the statistics for accuracy of model wave height and period, respectively.

Table 13. Model wave height accuracy statistics for the limited fetch generation under three different wind speeds (U) of 10, 20, and 30 m/sec.

Statistic	$U = 10$ m/sec	$U = 20$ m/sec	$U = 35$ m/sec
RMSE (m)	0.042	0.045	0.034
NRMSE (%)*	1.15	2.10	3.70
R^2	0.90	0.92	0.96
Bias (m)	-0.04	0.04	-0.02

*Normalized by model wave height at the fetch equal to 20 km.

Table 14. Model wave period accuracy statistics for the limited fetch generation under three different wind speeds (U) of 10, 20, and 30 m/sec.

Statistic	$U = 10$ m/sec	$U = 20$ m/sec	$U = 35$ m/sec
RMSE (sec)	0.25	0.36	0.29
NRMSE (%)*	4.17	7.13	7.51
R^2	0.88	0.71	0.74
Bias (sec)	0.06	0.15	0.12

*Normalized by model wave period at the fetch equal to 20 km.

2.7.4 Conclusions and recommendations

CMS-Wave performance for fetch-limited wave generation and growth curves in fully developed seas is compared with results from the SPM. The comparison covers the surface wave range from shallow to intermediate depth. Model wave periods generally agree well with the SMB. Model wave heights are somewhat underestimated within the fetch of 5 km as compared with the SMB. Future studies compare model wave height and period in fetch-limited range with field measurements are recommended.

2.8 Test C1-Ex7: Nonlinear wave-wave interactions

2.8.1 Purpose

The purpose of this test is to compare nonlinear wave-wave interactions as calculated by CMS-Wave to an analytical solution available in the open literature. Jenkins and Phillips (2001) proposed a simplified formulation to represent nonlinear wave-wave interactions as a second-order diffusion operator that conserves wave energy and wave action. Because the formulation is independent of the dispersion relation, it is applicable in deep and shallow water. The Jenkins and Phillips formulation has been extended to deep and shallow waters and implemented in the wave-action balance equation of the CMS-Wave to calculate nonlinear wave-wave interactions efficiently (Lin et al. 2010).

2.8.2 Model setup and parameters

CMS-Wave is set up with a flat seabed of constant depth 1,000 m. The model domain is 2 km $\times$ 200 km consisting of 10 $\times$ 1,000 cells with

constant cell size $200 \text{ m} \times 200 \text{ m}$. The incident spectral waves include three significant heights of 0.45, 0.5, and 0.6 m with the JONSWAP (Hasselmann et al. 1973) peak enhancement parameters of 1, 2, and 5, respectively. Wind forcing is not included in these simulations. The peak period of incident deep water wave is 10 sec, and the mean wave direction is along the long axis (200 km) of the grid. The directional distribution is a cosine bell function with a power of 20. The wave spectrum is specified in CMS-Wave on 30 frequency bins (0.04 to 0.33 Hz with 0.01 Hz increment) and 35 direction bins (covering a half-plane with 5° spacing). The default statistical parameters can be found for this test case in the Lin et al. (2011b) study.

2.8.3 Results and discussion

Figure 18 and Figure 19 show the comparison of directionally integrated wave energy transfer rate S_{nl} from CMS-Wave with exact computations in the examples of Hasselmann et al. (1985) for the spectral peak enhancement parameter $\gamma = 2$ and $\gamma = 5$, respectively. The calculated results are consistent with the observed and theoretical results in that the nonlinear wave-wave interactions cause wave energy to transfer from high to low frequencies, a well-accepted process referred to as frequency downshifting. However, the simplified formulation does not fully reproduce energies at high frequencies (i.e., in the more dynamic and unstable range involving wave breaking and energy exchange with atmospheric forcing).

Figure 18. Comparison of directionally integrated wave energy transfer rate S_{nl} for JONSWAP spectrum with the spectral peak enhancement factor $\gamma = 2$.

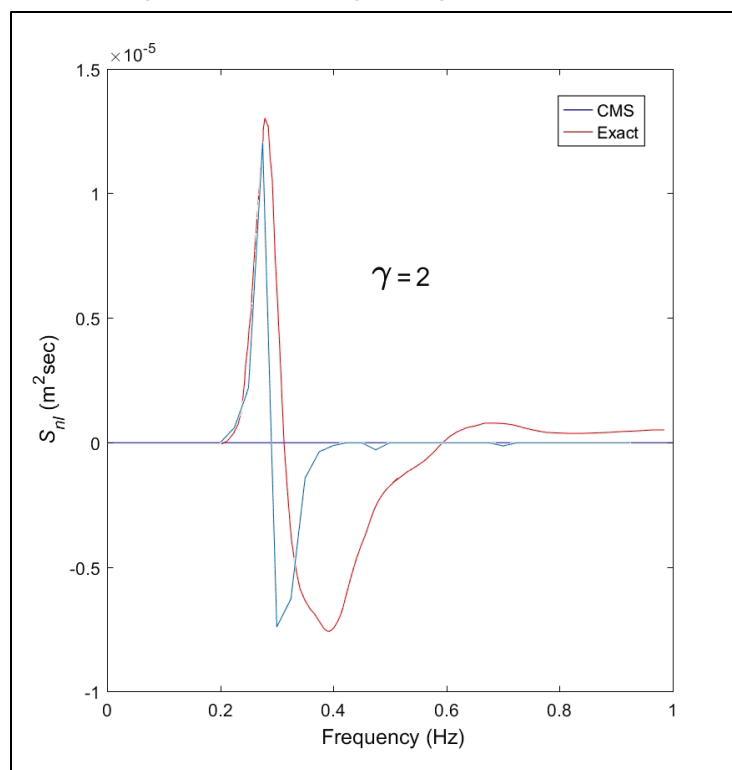


Figure 19. Comparison of directionally integrated wave energy transfer rate S_{nl} for JONSWAP spectrum with the spectral peak enhancement factor $\gamma = 5$.

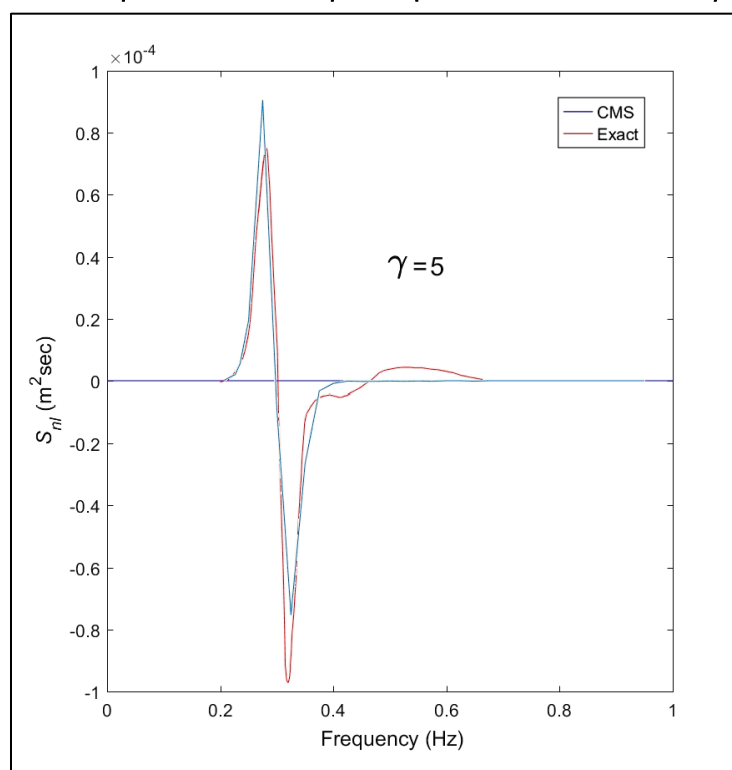


Table 15 listed the statistics for accuracy of model wave energy transfer rate for the spectral peak enhancement parameter $\gamma = 2$ and $\gamma = 5$.

Table 15. Statistics of model wave energy transfer rate accuracy for $\gamma = 2$ and $\gamma = 5$.

Statistic	$\gamma = 2$	$\gamma = 5$
NRMSE (%)*	15.6	2.37
R ²	0.35	0.77

*Normalized by the range (maximum - minimum) of model wave energy transfer rate.

2.8.4 Conclusions and recommendations

A theoretically based formulation to efficiently calculate nonlinear wave-wave interactions in CMS-Wave is tested and compared with exact computations in the examples by Hasselmann et al. (1985). The proposed formulation has performed well in idealized and real application examples (Lin et al. 2011a). This nonlinear wave-wave interaction effect is recommended in CMS-Wave for long fetches, which allow both time and space for efficient nonlinear wave energy transfer to occur as demonstrated in both idealized and real applications.

2.9 Test C1-Ex8: An idealized infinite wedge

2.9.1 Purpose

The purpose of this test is to compare CMS-Wave calculations to a wave diffraction solution for monochromatic, unidirectional incident waves interacting with a fully-reflective vertical wedge of any angle on a flat bottom (Chen 1987).

2.9.2 Model setup and parameters

The wave model domain is a square grid of 2 km $\times$ 2 km with constant cell spacing of 10 m and a uniform water depth of 1,000 m. The vertex of the wedge is located at the center cell of the grid. The vertical wedge is represented by dry cells with an elevation of 2 m. The incident wave height is 1 m, and the wave period is 8 sec (0.125 Hz), which corresponds to a wavelength of approximately 100 m. The spectral wave transformation is computed on 10 frequency bins (0.06 to 0.15 Hz with

0.01 Hz increments) and 35 direction bins (covering a half-plane with 5° spacing). The incident total wave energy density is placed in one directional and frequency bin to represent the monochromatic unidirectional waves for the boundary condition.

2.9.3 Results and discussion

Numerical simulations are conducted for a semi-infinite fully reflective breakwater (wedge angle = 0°) normal to incident wave direction and the same breakwater with incident waves from a 135° oblique direction. Figure 20 and Figure 21 show the comparison of model wave height with the analytical solution (Chen, 1987) for incident waves with 90° and 135° approach directions, respectively. Because the incident wave height is 1 m, the wave height contours shown in Figure 20 and Figure 21 are the same as normalized by the incident wave height. The model calculates the resultant wave direction by averaging the spectral components from different directions.

The model results show the diffracted and reflected energy levels in the lee and front of the breakwater similar to the analytical solution. The model cannot calculate the standing wave pattern caused by wave reflection as shown in the analytical solution.

Table 16 listed the statistics of model wave height accuracy for idealized semi-infinite fully reflective wedge with incident wave of 1 m and 8 sec from two approach directions: (1) 90°, normal to the wedge and (2) 135°, oblique to the wedge.

Table 16. Statistics of model wave height accuracy for idealized fully-reflective semi-infinite wedge with incident wave height of 1 m and 8 sec coming from two approach directions: (1) 90°, normal to the wedge, and (2) 135°, oblique to the wedge.

Statistic	(1) wave approach angle = 90°	(2) wave approach angle = 135°
NRMSE (%)*	4.04	5.15
R ²	0.83	0.78

*Normalized by the incident wave height of 1 m, with wave period equal to 8 sec.

Figure 20. (a) Model wave height and (b) analytical wave height for incident waves of 1 m, 8 sec, and 90° approach angle, normal to fully reflective infinite wedge.

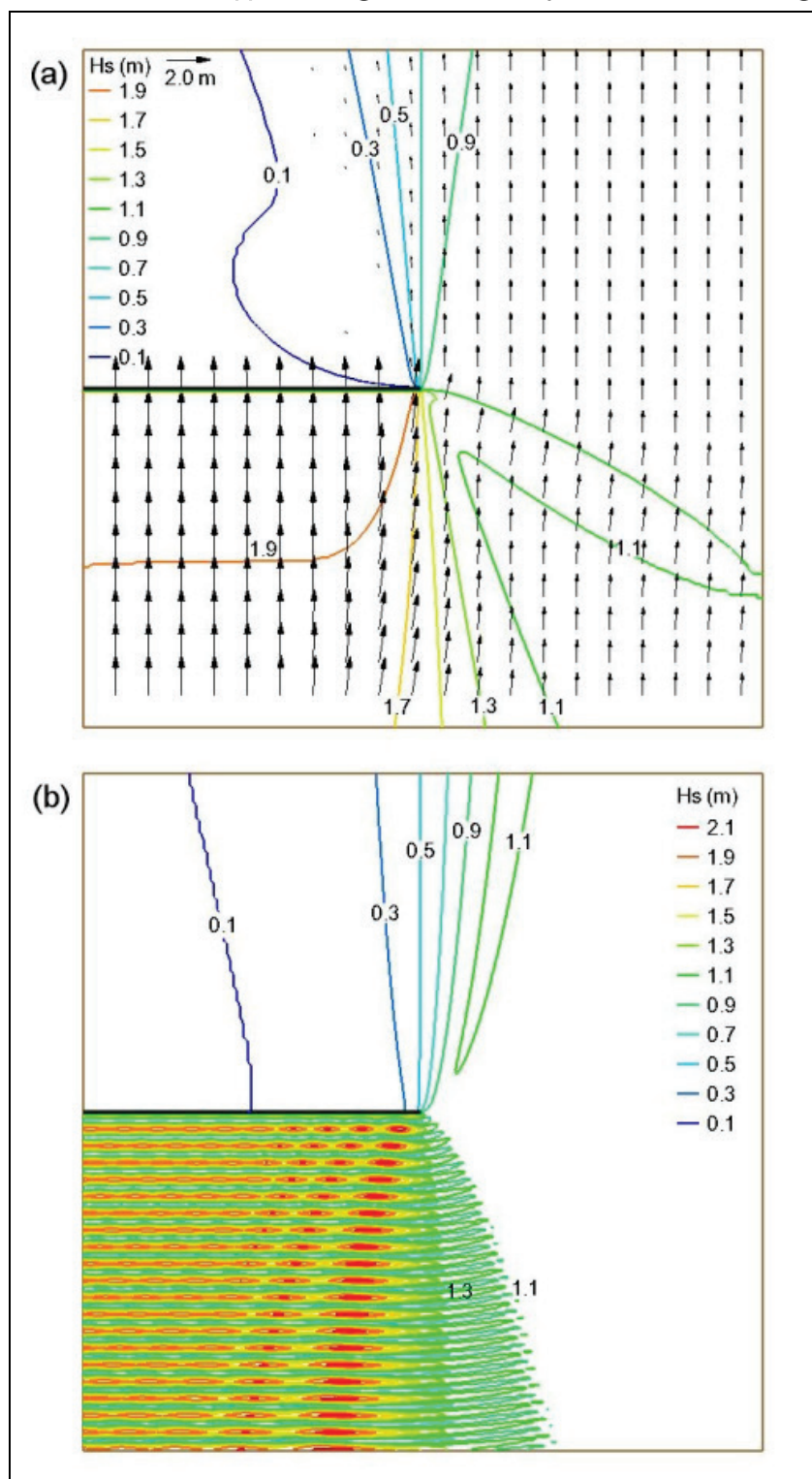
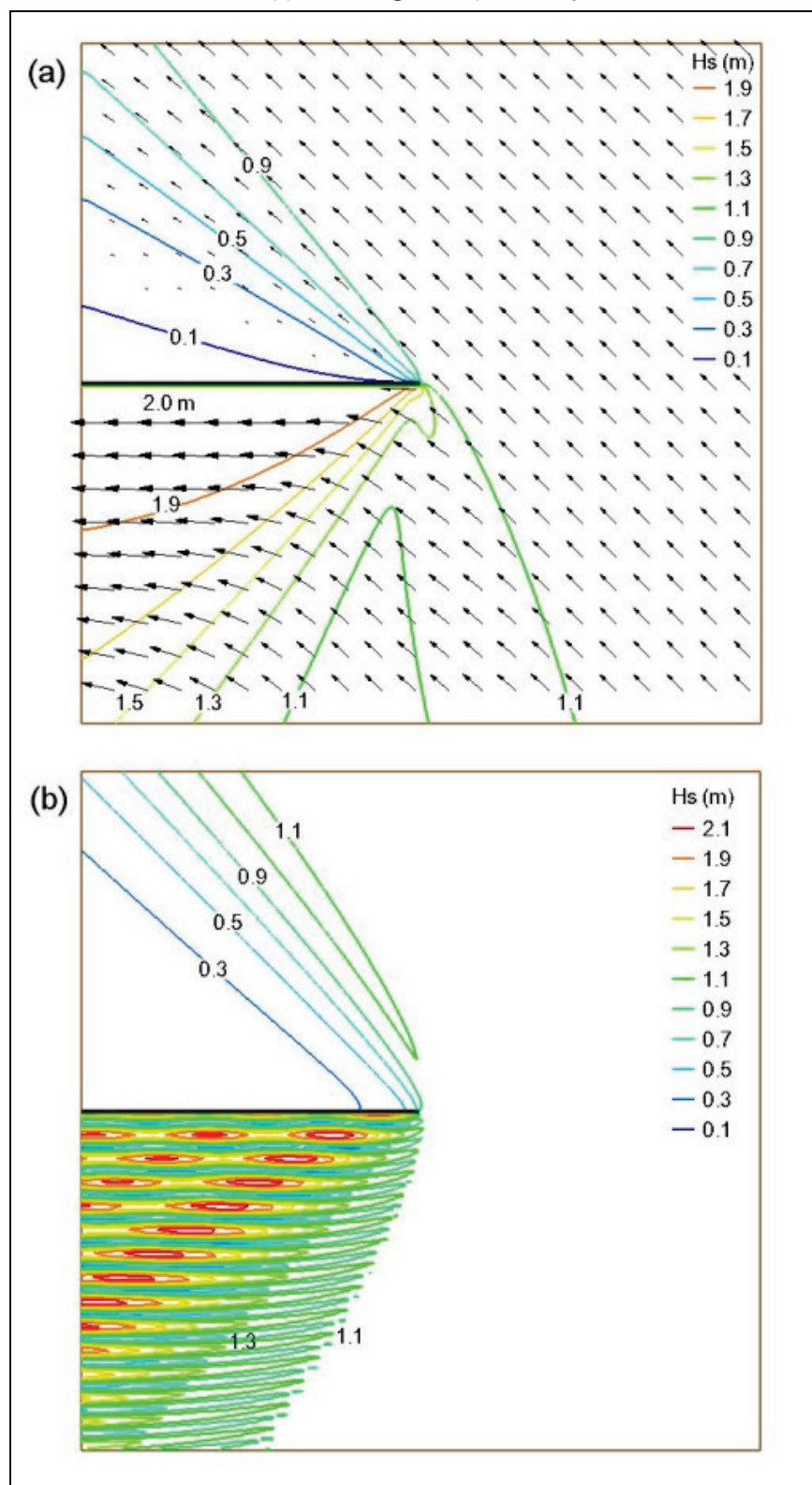


Figure 21. (a) Model wave height and (b) analytical wave height for incident waves of 1 m, 8 sec, and 135° approach angle oblique to fully reflective infinite wedge.



2.9.4 Conclusions and recommendations

CMS-Wave is tested for diffracted and reflected energy levels in the lee and front of idealized breakwater configuration with the analytical solution. Model results show good wave diffraction and scattering around the idealized breakwater. However, the model cannot calculate the standing wave pattern caused by wave reflection in the front of the breakwater. A more advanced wave model (e.g., a Boussinesq-type model) is recommended for standing wave calculation in the front of the breakwater.

3 Laboratory Cases

3.1 Overview

This chapter presents test cases from Category 2 (i.e., validation of the CMS implementation of numerical methods compared to observation from laboratory experiments). The controlled environment of the laboratory makes results from these cases useful for calibrating parameters as well as basic validation of the CMS physics' formulations. Category 2 test cases are the following:

1. Rectangular flume with a spur dike
2. Rectangular flume with a sudden expansion
3. An idealized elliptic shoal
4. Idealized jettied inlet with equilibrium breach profile and oblique incident regular waves
5. Wave breaking experiments on a planar beach
6. Wave runup on impermeable uniform slope
7. Cleveland Harbor, OH, physical model.

3.2 Test C2-Ex1: Steady flow in a rectangular flume with a spur dike

3.2.1 Purpose

An experimental case of steady-state flow in a flume with a spur dike is simulated with CMS-Flow. Measurements of the flow velocity are made in cross-flume transects at regular spacing downstream from the dike. The velocity profiles are intended to exhibit the CMS ability to simulate the recirculation zone behind the dike. The specific model capabilities being tested are non-uniform Cartesian grid, inflow discharge boundary condition, wall boundary condition, outflow water level boundary condition, and the subgrid eddy viscosity (turbulence) model (Smagorinsky 1963).

3.2.2 Experiment setup and parameters

Rajaratnam and Nwachukwu (1983) investigated steady flow in a 37 m long, 0.92 m wide rectangular flume. A 0.1524 m long thin plate was used to simulate a groin-like structure in their laboratory experiment. The CMS

validation presented here is compared to experiment A1 in their laboratory experiments. The water depth for this case is 0.189 m with an input discharge of 0.0453 m³/sec.

3.2.3 Model setup and parameters

The computational grid contains 152 nodes in the longitudinal and 36 nodes in the lateral direction with a variable grid resolution between 0.01 and 0.05 m (Figure 22). The finest resolution is located around the spur dike protrusion into the flume with additional refinement near the walls and within the recirculation zone downstream from the dike. Table 17 presents a summary of the relevant model parameters.

Figure 22. CMS-Flow computational grid. Finest resolution is located at the spur dike.

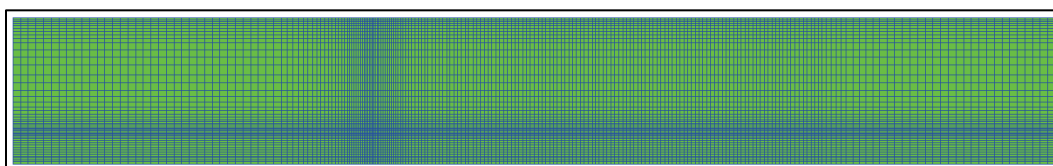


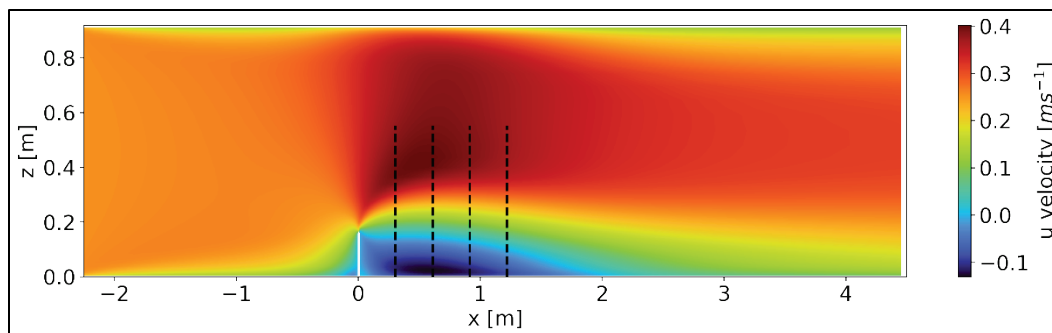
Table 17. Model accuracy statistics for with-Coriolis force case.

Parameter	Value
Solution Scheme	Implicit
Time-step	1 min
Simulation Duration	3 hr
Ram duration	2.5 hr
Manning's I	0.02
Wall friction	On
Turbulence model	Subgrid

3.2.4 Results and discussion

The computed steady-state velocities are interpolated to the observational cross sections located downstream from the spur dike. Figure 23 shows the location of the cross sections and the computed steady-state u velocity component. The cross sections are spaced at distances equal to $\frac{x}{b} = 2, 4, 6, 8$, where x is the along-flume coordinate originating at the spur dike and increasing down-flow and b is the length of the spur dike.

Figure 23. The CMS-Flow computed steady-state u velocity component and cross-section locations (dashed lines).



In the laboratory experiments, velocities are observed at several cross-flume locations and at two vertical locations. Here, CMS is compared to those acquired at 85% of the water depth. Figure 24 shows a comparison of the observed along-flume velocity component (u component [i.e., in the x direction]) and the computed velocity at each cross section. The goodness-of-fit statistics for this comparison is presented in Table 18.

Figure 24. Computed (blue line) versus observed (red circles) along-flume velocity component at the cross section in Figure 22. From left to right, the panels are the cross sections located at $x/b = 2, 4, 6$, and 8 .

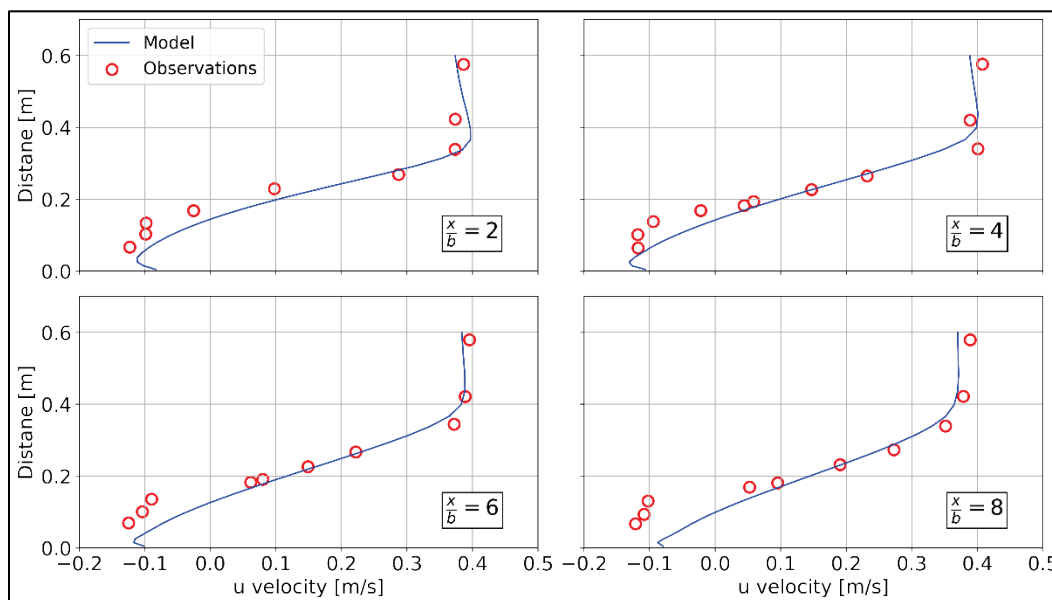


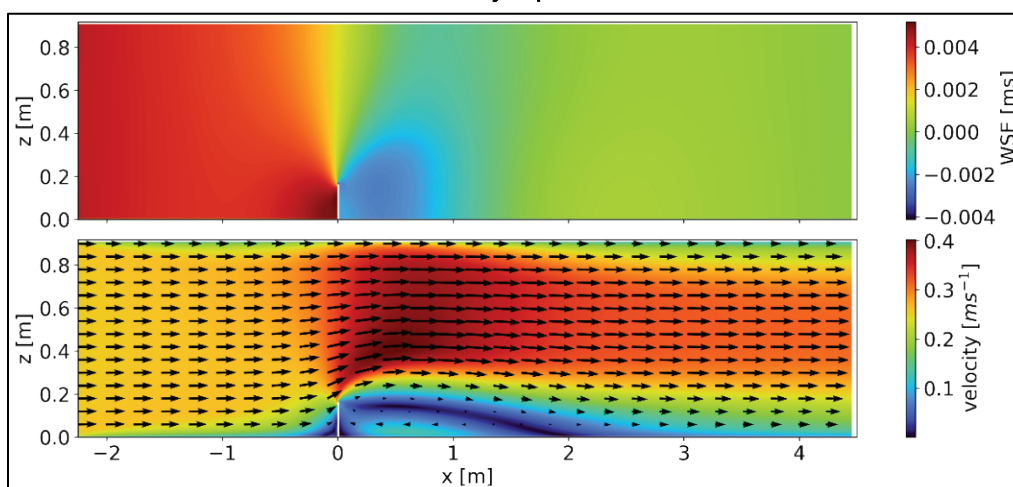
Table 18. Velocity goodness-of-fit statistics between CMS results and the Rajaratnam and Nwachukwu (1983) experiment.

Statistic	$x/b=2$	$x/b=4$	$x/b=6$	$x/b=8$
RMSE, m/sec	0.0470	0.0427	0.0453	0.0647
NRMSE, %	9.24	8.13	8.72	12.68
NMAE, %	6.16	3.64	4.73	6.39
R^2	0.981	0.991	0.995	0.990
Bias, m/sec	0.0313	0.0191	0.0246	0.0326

3.2.5 Conclusions and recommendations

CMS-Flow performance is assessed through a comparison with flume test observations from the spur dike experiments of Rajaratnam and Nwachukwu (1983). The computed flow velocity field (Figure 25) compares well with the flume cross sections, displaying RMSE values in the range of 0.0427 m/sec to 0.0647 m/sec, NMAE from 3.64% to 6.39%, and R^2 from 0.981 to 0.995. Further tests that vary the turbulence model and their relevant parameters are required to test the CMS-Flow sensitivity. In the right scenarios, the non-uniform Cartesian grid may be preferable to the telescoping grid because it is easier to set up.

Figure 25. Computed water level (top) and current velocity (bottom) for spur dike laboratory experiment.



3.3 Test C2-Ex2: Steady flow in a rectangular flume with a sudden expansion

3.3.1 Purpose

The phenomena of flow rapidly expanding as it exits the constriction of an inlet is a common occurrence in coastal hydrodynamics. This case validates the CMS ability to simulate the rapid expansion of steady-state flow in a laboratory flume. Model performance is assessed with a comparison between the simulated and observed cross-flume profiles of the along-flume velocity component. The specific model features that are being validated are telescoping grid with variable aspect ratio, output flow water level boundary condition, inflow flux boundary condition, wall boundary condition, and mixing-length eddy viscosity (turbulence) model (Wu et al. 2011a,b).

3.3.2 Experiment setup and parameters

Xie (1996) performed the experiment in a rectangular flume that was 18 m long with an upstream inflow section 0.6 m wide that abruptly expanded to 1.2 m wide 1.7 m downstream. The experimental setup is shown in Table 19.

Table 19. Experimental hydrodynamic conditions for the Xie (1996) experiment.

Parameter	Value
Inflow	0.03854 m ³ /sec
Bed slope	1/1000
Downstream depth	0.115 m

1.4.1 Model setup and parameters

A three-level telescoping grid is created with elongated cells aligned with the predominate flow direction (Figure 26).

Figure 26. Computational grid for the experimental flume in Xie (1996).



The model setup parameters are given in Table 20. The water level is initialized at zero (still water condition) for the entire grid. A discharge boundary condition is applied at the upstream inflow boundary, and a constant water level is imposed at the downstream outflux boundary. A 0.5 hr ramp is allowed, and the model is run to steady-state at 1 hr. The mixing-length turbulence model is chosen for this case (Wu et al. 2011a,b). A horizontal shear viscosity of 0.3 is used, which is near the CMS default of 0.4. A Manning's coefficient of 0.015 is chosen and is consistent with the concrete bottom of the flume.

Table 20. Model set up parameters for the Xie (1996) experiment.

Parameter	Value
Solution Scheme	implicit
Time-step	30 sec
Simulation duration	1 hr
Ramp duration	0.5 hr
Manning's coefficient	0.015
Turbulence model	Mixing-length
Bottom shear viscosity coefficient	0.067 (=default)
Horizontal shear viscosity coefficient	0.3 (default=0.4)

1.4.2 Results and Discussion

Xie (1996) measured the current velocity along six cross-flume transects spaced at 1 m intervals beginning at the sudden expansion. Figure 27 shows a comparison of the observed and computed x -component velocities (i.e., along-flume velocity component). The magnitude and pattern of the computed velocities in general agree well with the measurements. This is also evident in the goodness-of-fit statistics presented in Table 21.

Figure 27. Comparison between the CMS computed velocity and the observed velocity in Xie (1996). The velocity values have been shifted by the position of the flume cross section where the observations were made.

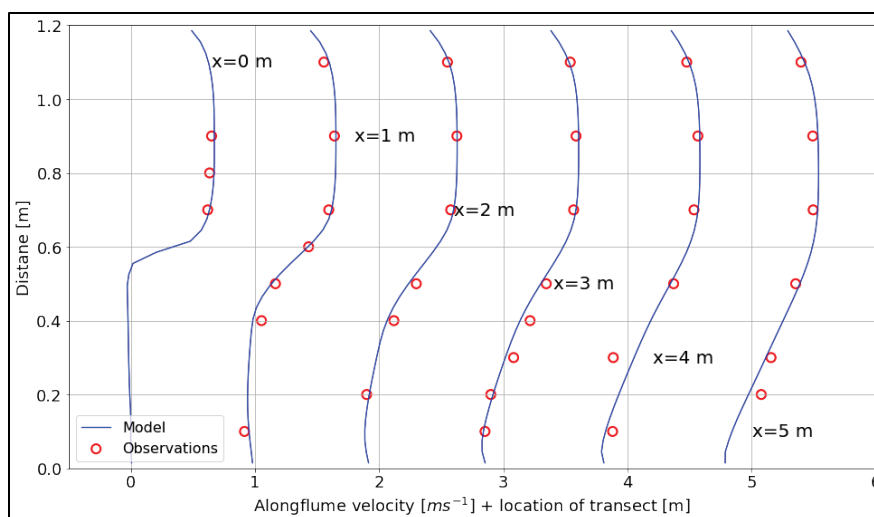


Table 21. Velocity goodness-of-fit statistics between CMS results and the Xie (1996) experiment. *The small number of observations at $x = 0$ m make the calculation of NRMSE and R^2 sensitive to small errors.

Statistic	Cross-section Location					
	$x = 0$ m*	$x = 1$ m	$x = 2$ m	$x = 3$ m	$x = 4$ m	$x = 5$ m
RMSE, m/sec	0.026	0.038	0.037	0.042	0.075	0.054
NRMSE, %	82.3	5.2	5.0	5.8	10.9	12.9
NMAE, %	77.30	0.11	-1.36	-2.67	3.64	0.18
R^2	0.997	0.993	0.994	0.991	0.973	0.987
Bias, m/sec	0.024	0.001	-0.010	-0.020	0.025	-0.001

1.4.3 Conclusions and recommendations

The performance of CMS-Flow for steady-state conditions with a rapid expansion in a rectangular flume is analyzed and compared to experimental results from Xie (1996). The mixing-length turbulence model is used, and the computed flow field compared well both quantitatively (Table 18) and qualitatively (Figure 26). RMSE values range between 0.026 m/sec and 0.075 m/sec while the R^2 values range between 0.987 and 0.997. The spatial pattern of re-circulation in the experimental flume is captured well in the computed results. The good results indicate that stretching the telescoping grid cells does not affect simulation accuracy. The computational costs saved by elongating the grid cells in the

predominate flow direction can be significant, therefore this practice is recommended.

3.4 Test C2-Ex3: An idealized elliptic shoal

3.4.1 Purpose

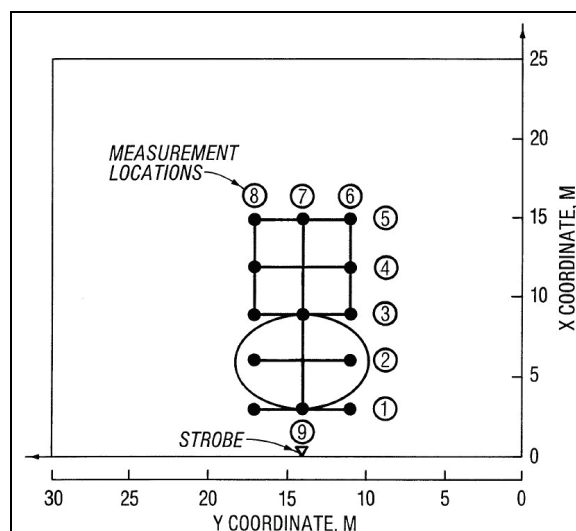
The purpose of the test is to compare model wave diffraction and refraction with a physical experiment by Vincent and Briggs (1989). The laboratory experiment includes waves over a mound on a flat bottom. The mound has an elliptical perimeter of $(x^*/3.05 \text{ m})^2 + (y^*/3.96 \text{ m})^2 = 1$ where x^* and y^* are local coordinates in meters with the shoal center at $x^* = y^* = 0$. The water depth (m) over the shoal is given by

$$0.9144 \text{ m} - 0.6096 \text{ m} \sqrt{1.5625 \text{ m} - (x^*/3.05 \text{ m})^2 + (y^*/3.96 \text{ m})^2}$$

with the smallest depth equal to 0.1524 m at the center of shoal. The surrounding flat bottom has a constant depth of 0.4572 m.

The elliptical shoal experiment is conducted in a wave basin of 29 m long by 35 m wide. The long axis (y^* -axis) of the mound is parallel to the wave generator approximately aligned with the global y -axis. The incident wave normal direction is parallel to the global and local axis. The center of the shoal is located at $x = 6.1 \text{ m}$ and $y = 13.72 \text{ m}$ in the global coordinate system. Figure 28 shows the locations of the elliptical shoal and nine transect lines for wave measurements.

Figure 28. Locations of elliptical shoal and measurement transects.



The elliptical shoal experiment consists of 18 incident wave conditions to investigate the effects of different factors, including directional spreading, wave randomness, and wave breaking over the shoal. For model tests of numerical modeling of combined diffraction-refraction (D-R), three incident wave conditions, named as B2, N2, and N4, for narrow frequency spectra are simulated. While B2 is associated with a symmetric and broad directional spreading, N2 and N4 are associated with a relatively narrow directional distribution. Table 22 presents the incident wave height H_s , spectral peak period T_p , spectral peak-enhancement factor γ , mean direction θ_o , and directional spreading standard deviation θ_s for B2, N2, and N4 (Vincent and Briggs 1989). Incident wave condition N4 is for a non-breaking test with a wave height three times smaller than B2 and N2.

Table 22. Model Incident wave conditions for B2, N2, and N4.

Case ID	H_s (m)	T_p (sec)	γ	θ_o (deg)	θ_s
B2	0.0775	1.3	20	0	30
N2	0.0775	1.3	20	0	10
N4	0.0254	1.3	20	0	10

3.4.2 Model setup and parameters

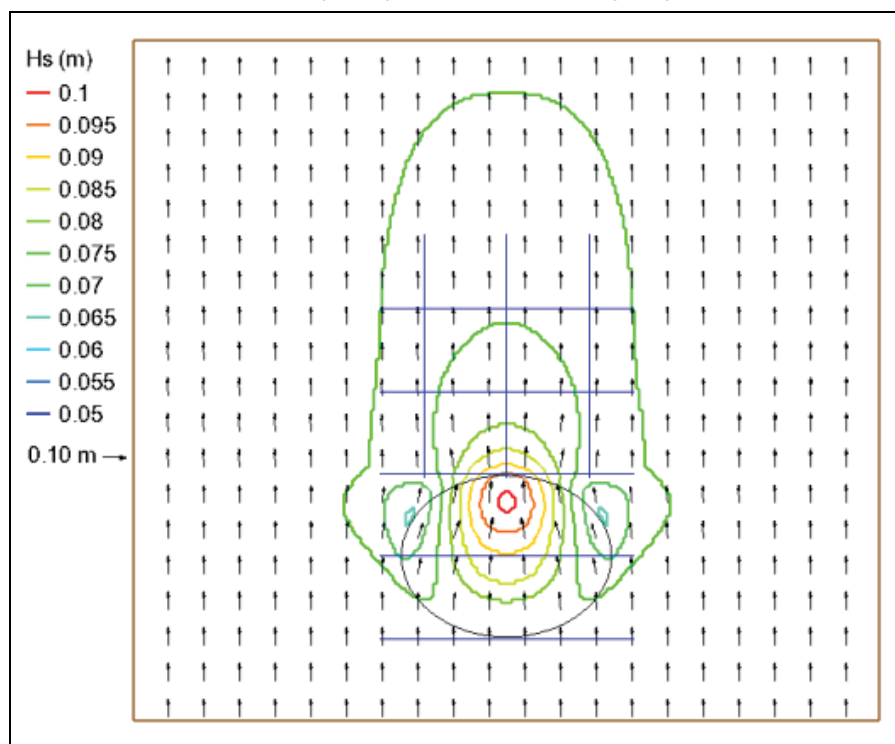
The model simulations are performed at the laboratory scale. The model domain is a rectangular grid of 25.1 m $\times$ 27.5 m, consisting of 251 $\times$ 275 cells with cell size of 0.1 m $\times$ 0.1 m. The spectral wave transformation is computed on 27 frequency bins (0.6 to 0.99 Hz with 0.015 Hz increment)

and 35 direction bins (covering a half-plane with 5° spacing). The simulations include cases with wave refraction only and with combined D-R.

3.4.3 Results and discussion

Figure 29 and Figure 30 show the model wave height and direction fields for combined D-R with the outline of elliptical shoal and transect positions for B2 and N2, respectively. The model results show stronger wave energy focusing behind the shoal due to combined D-R for a narrower frequency spectrum. Figure 31 and Figure 32 show the model calculated wave height and data at Transects 3 and 4 for B2 and N2, respectively. Figure 33 provides the comparison of model wave height and data at Transect 4 for N4 (wave measurement for N4 available only at Transect 4).

Figure 29. Model wave height contours and wave vector fields for B2 with elliptical shoal outline (black) and transect lines (blue).



The model results show small difference between runs with refraction only and with combined D-R. This indicates that wave processes over the shoal is dominated mainly by refraction, and diffraction can be negligible. For B2, the model wave height agrees with data at Transects 3 and 4. For N2, the model calculates higher wave heights at the middle of Transect 3 and

lower wave heights at the middle of Transect 4. For N4, for which the wave data are available only at Transect 4, the model also predicts lower wave heights at the middle of Transect 4.

The difference between model calculated wave height and data behind the shoal may be due to several reasons. First, the model wave height conversion from wave spectrum based on the linear wave theory is only approximate behind the shoal where wave activity is highly nonlinear. Second, wave data analyzed by conventional Fourier transform or zero-crossing wave height method may not be reliable for strong nonlinear waves behind the shoal. Third, the narrow directional spreading of spectral waves in N2 and N4 may not be accurately produced in the physical or numerical model. These factors cause the difference between model wave height and data.

Figure 30. Model wave height contours and wave vector fields for N2 with elliptical shoal outline (black) and transect lines (blue).

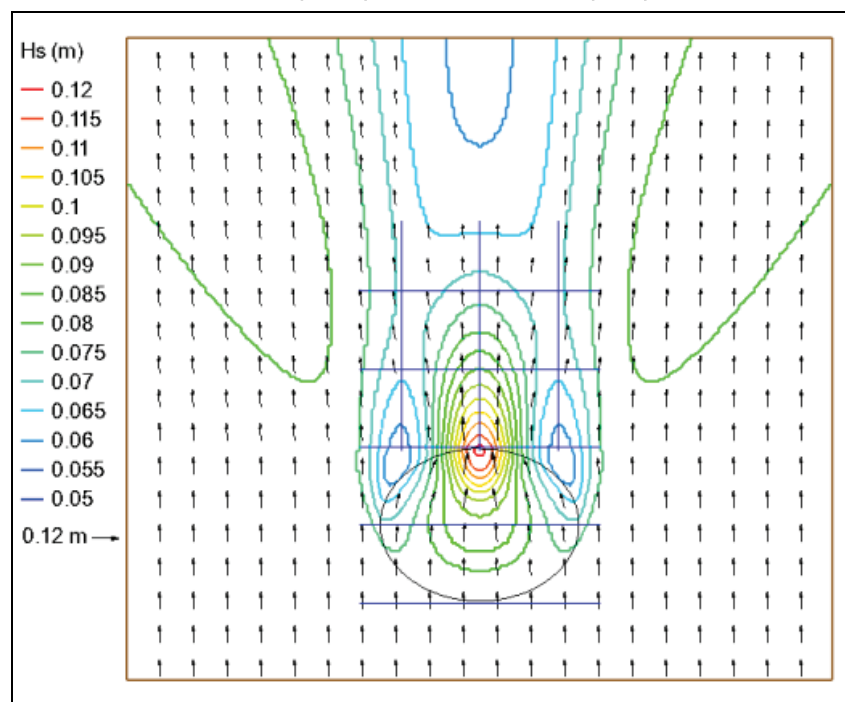


Figure 31. Model wave height and data comparison at Transects 3 and 4 for B2.

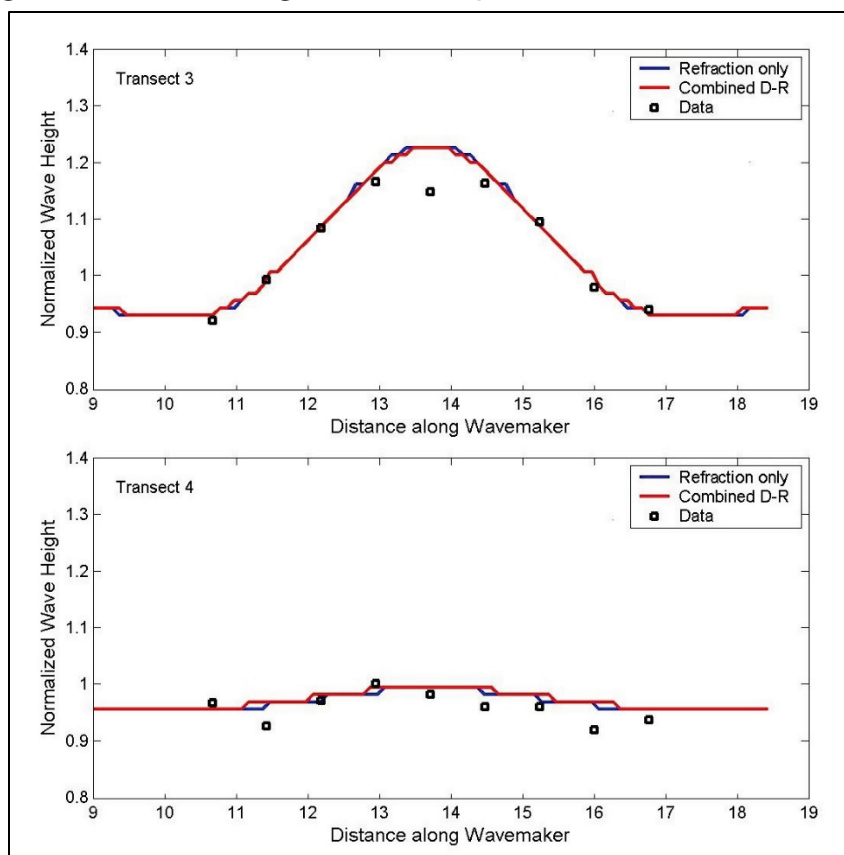


Figure 32. Model wave height and data comparison at Transects 3 and 4 for N2.

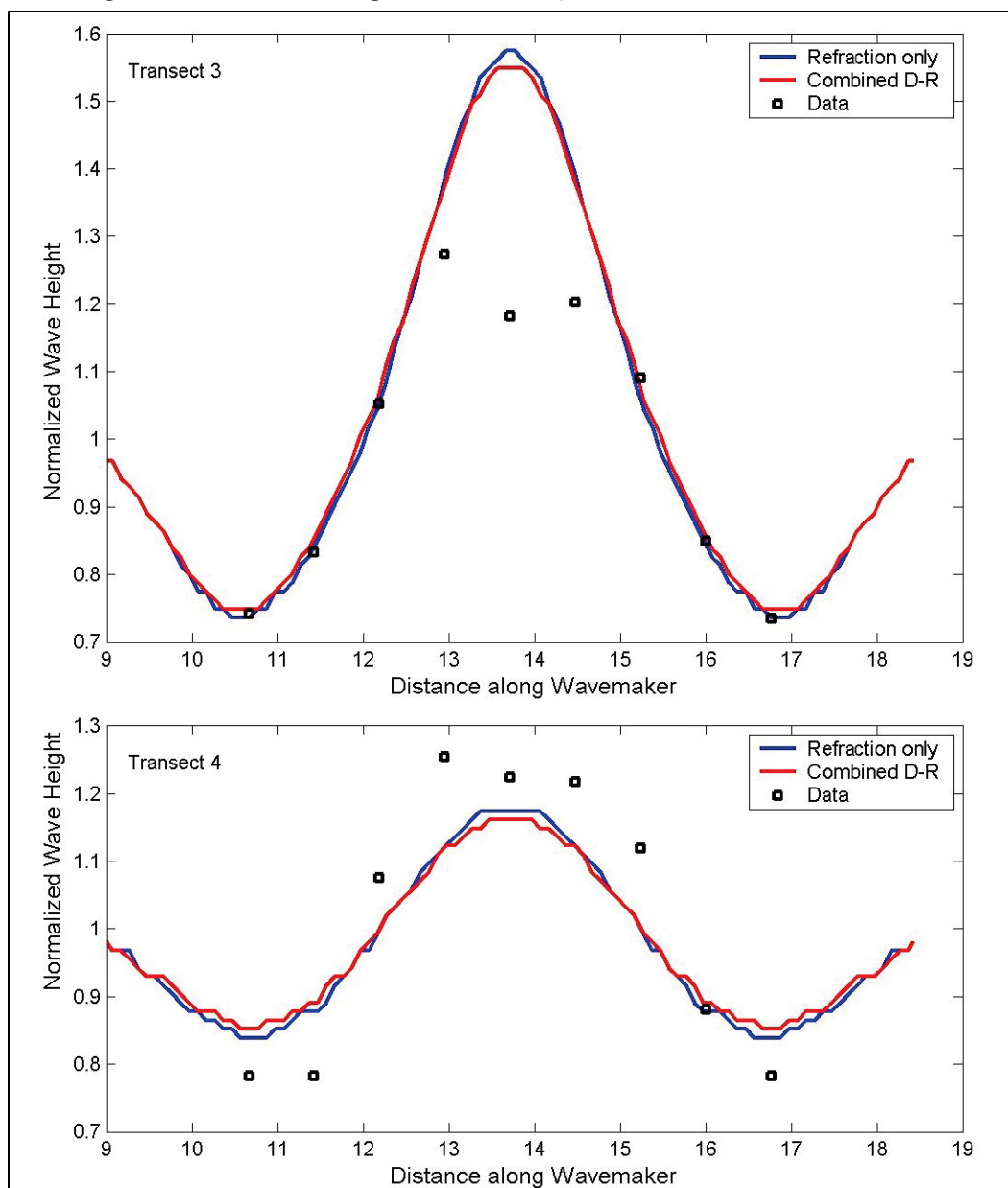


Figure 33. Model wave height and data comparison at Transect 4 for N4.

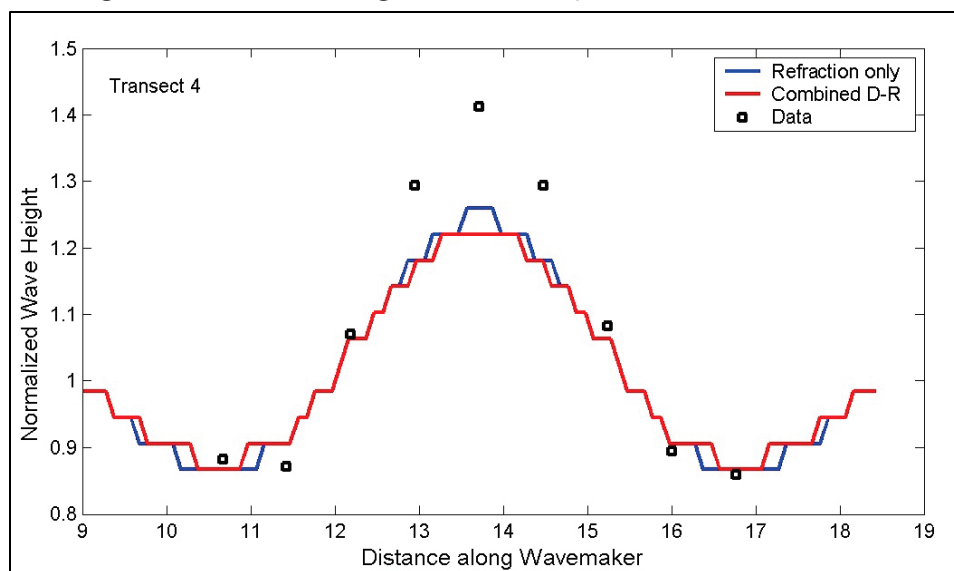


Table 23 lists the model wave height accuracy statistics with combined refraction and diffraction along 9 transects in the idealized elliptic shoal experiment for Cases B2, N2, and N4.

Table 23. Model wave height accuracy statistics with combined refraction and diffraction along 9 transects in the idealized elliptic shoal experiment for Cases B2, N2, and N4.

Statistic	Case B2	Case N2	Case N4
RMSE (m)	0.018	0.022	0.019
NRMSE (%)*	23.2	28.4	74.8
R ²	0.83	0.51	0.42

*Normalized by model incident wave height.

3.4.4 Conclusions and recommendations

The combined wave D-R in CMS-Wave is tested and compared with a physical experiment of idealized shoal in the shallow water by Vincent and Briggs (1989). Model results show small difference between runs with refraction only and with combined D-R. This indicates that wave processes over the shoal is dominated mainly by refraction, and diffraction can be negligible. Model results for the broad directional spreading agree with data behind the shoal. In the case of incident waves with narrow directional spreading, the model tends to underestimate wave height

behind the shoal. This difference between model-calculated wave height and data behind the shoal may come from the fact that the spectral wave model cannot accurately predict highly nonlinear and narrow directional spreading waves behind the shoal.

3.5 Test C2-Ex4: Idealized jettied inlet with equilibrium beach profile and oblique incident regular waves

3.5.1 Purpose

The purpose of this validation case is to evaluate the CMS for wave-induced hydrodynamics in the vicinity of a physical model for an idealized inlet with two absorbing jetties. The specific model features to be tested are interaction of wave and current, wave-adjusted lateral boundary conditions, and Stokes velocities in the continuity and momentum equations.

3.5.2 Physical experiment

Seabergh et al. (2005) conducted a physical model study to collect both current and wave measurements in the vicinity of an idealized dual jetty inlet located in the CHL. The idealized inlet experiments were performed in a 46 m wide by 99 m long concrete basin with 0.6 m high walls. Figure 34 shows the layout of the facility and basin area. A 1:50 undistorted Froude model scale was used to represent the dimensions of a medium-sized US Atlantic coast inlet. The ocean side parallel contours correspond to an equilibrium profile $h = Ax^{2/3}$, where h is the still water depth, x is the cross-shore coordinate from the shoreline and A is a grain size dependent empirical coefficient (equal to $0.1615 \text{ m}^{1/3}$ in this case). (For further details on the physical model and previous modeling results with CMS, please refer to Seabergh et al. (2005) and Lin and Demirbilek (2005).) Fully reflective and absorbing jetties are constructed for inlet geometries studied in the physical model. However, all of the tests shown here are for the absorbing jetties since they represent those typically found in coastal applications. The incident wave conditions for the test cases used here are shown in Table 24. The three cases are chosen to cover a range of wave heights from low, medium, to high waves.

Figure 34. Physical model setup for the idealized inlet case (from Seabergh et al. 2005).

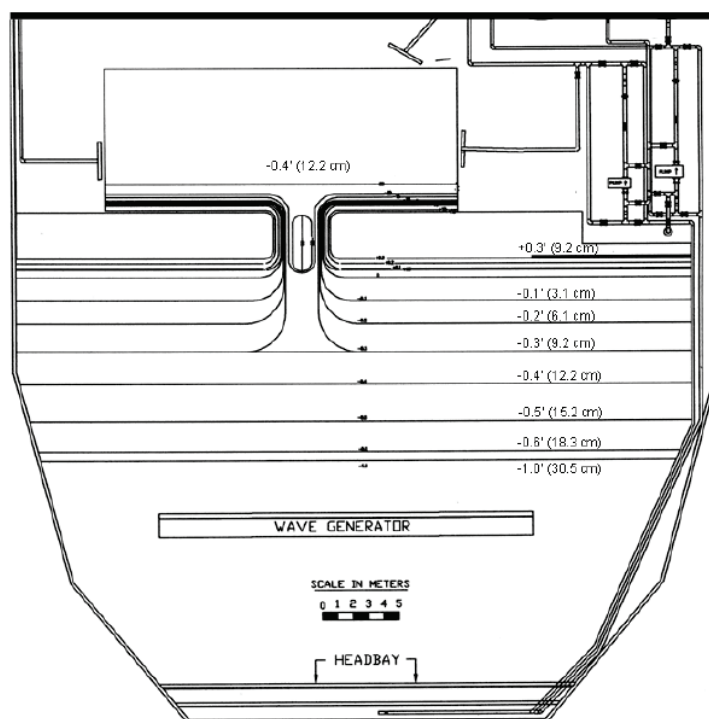


Table 24. Wave conditions (prototype scale) of three test cases from Seabergh et al. (2005).

Case	Wave height*, m	Wave period, sec	Wave Direction**
1	1.65	11.0	20°
2	2.0	11.0	20°
3	3.25	8.0	20°

*Measured at the first offshore station approximately 50 m (prototype) from the jetty tips (see Figure 30).

**Clockwise from shore normal.

3.5.3 Model setup

The computational grid and bathymetry in the prototype scale for both CMS-Flow and CMS-Wave are shown in Figure 35. The grid has 31,422 active cells and a constant resolution of 10 m. A list of the basic model setup parameters is given in Table 25. A constant zero water level boundary condition is assigned to the offshore boundary of CMS-Flow. A wall boundary condition is used at all boundaries inside the bay.

Figure 35. CMS computational grid showing the model bathymetry. Black circles indicate current velocity and wave height measurement stations used in this study.

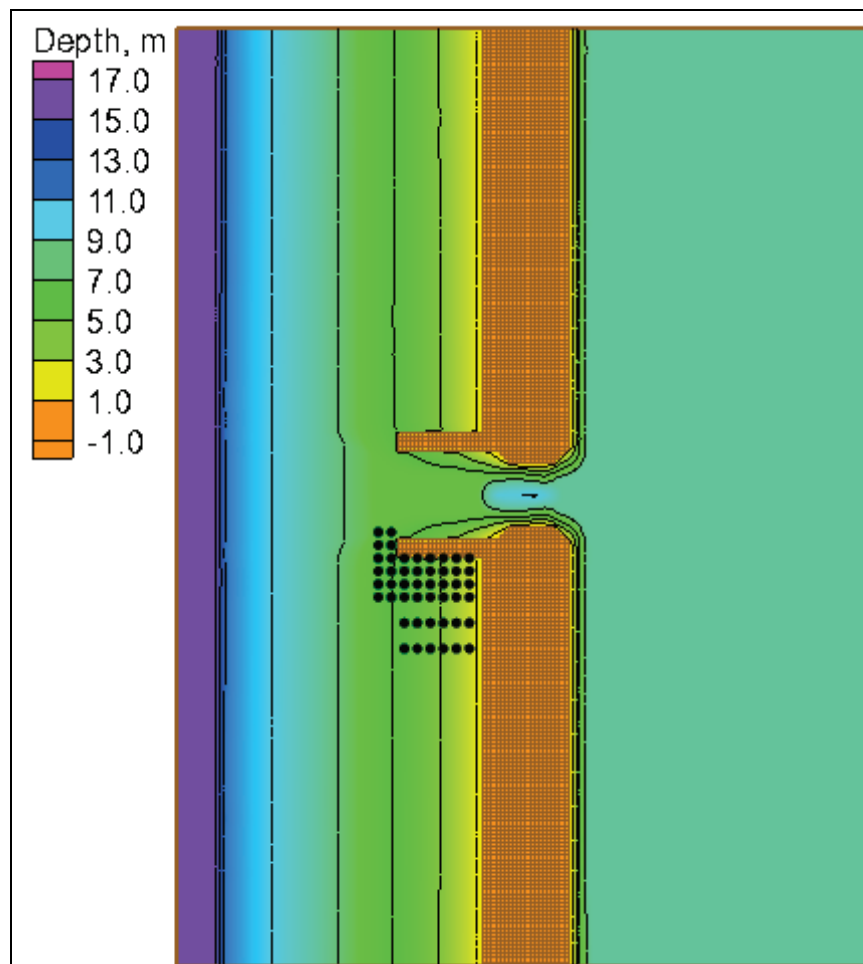


Table 25. CMS settings for the Seabergh et al. (2005) experiment.

Parameter	Value
Flow time-step	10 min
Simulation duration	4 hr
Ramp period duration	3 hr
Manning's n for both flow and wave	0.025
Steering interval	1 hr
Roller effect	On
Roller dissipation coefficient	0.05 (default for regular waves)
Stokes velocities	On
Wave reflection coefficient	0.0
Breaking criterion	Battjes and Janssen
Wave-to-flow coupling	Linear temporal interpolation

Default CMS settings are used where possible with the Manning's coefficient being the only calibrated parameter ($n = 0.025$) for this case study. The roller dissipation coefficient β_D is set to the recommended value for regular waves ($\beta_D = 0.05$). Both parameters are held constant for all test cases. Including the roller is very important for regular waves because it improves the prediction of the longshore current. The wave- and depth-averaged hydrodynamics equations are solved for depth-uniform currents according to Phillips (1977) and Svendsen (2006). The formulation includes Stokes velocities in both the continuity and momentum equations and provides a better prediction of cross-shore currents.

3.5.4 Results and discussion

The measured and calculated wave heights and wave-induced nearshore currents are presented in plan-view vector plots and also in cross-shore transects as discussed below. Note that within the cross-shore transects, the wave height and cross-shore profiles are offset by a number as indicated to the left of each transect, which are plotted using different colors. Demirbilek et al. (2009) reported similar results for the wave height using a previous version of CMS. Due to the inclusion of the surface roller and Stokes velocities, the current velocities reported here are significantly improved with respect to Demirbilek et al. (2009).

3.5.4.1 Case 1 ($H = 1.65$ m, $T = 11$ sec)

The calculated wave height magnitudes and directions agree well with the measurements with a NMAE of 12.71% (Figure 36 and Figure 37 and Table 21). The wave model tends to over-predict wave refraction near the structure and shoreline. The breaker is located at approximately the third cross-shore measurement station from the shoreline and is predicted well by the model (Figure 37). Measured and computed current velocities for Case 1 are compared in Figure 36 and Figure 38. The velocity field is characterized by a narrow longshore current approximately 75 m wide which is deflected seaward by the south jetty. The NRMSE and NMAE values for the longshore current are approximately 26% and 20%, respectively while for the cross-shore current, they are 28% and 18%, respectively (Table 26). Most of the longshore current is located within the first two measurement stations from the shoreline. The calculated cross-shore currents agree well with the measurements except near the jetty

where underestimation occurred at transects number 3 and number 4, and overestimation at number 5.

Figure 36. Measured and calculated wave height (left) and mean current (right) vectors for Case 1. Background colors indicate the local water depth corresponding to the right color bar.

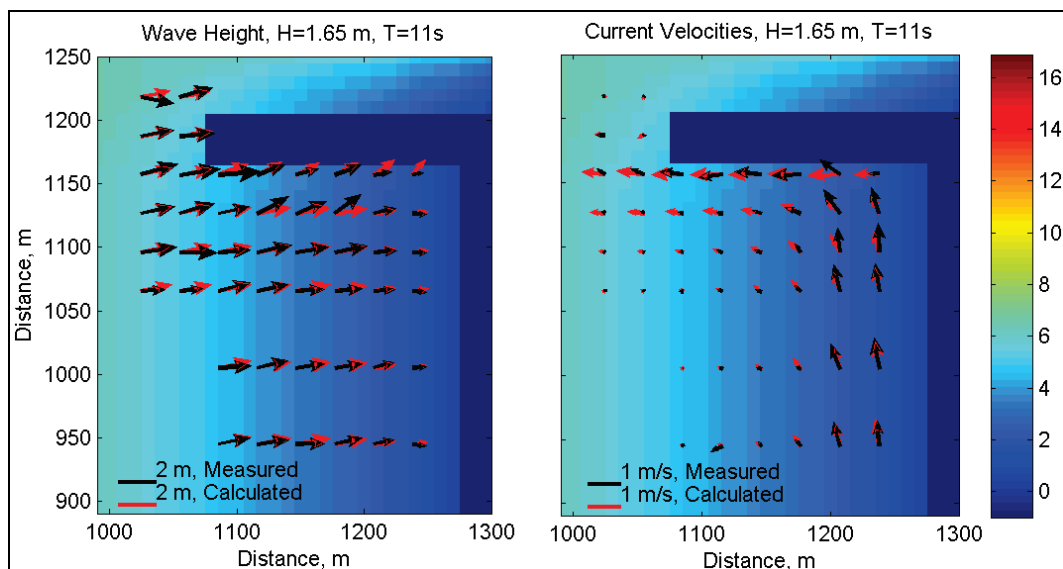


Figure 37. Cross-shore transects of measured and calculated wave heights for Case 1 ($H = 1.65$ m, $T = 11$ sec). For display purposes, wave heights are shifted by the number indicated on the left-hand side of each transect.

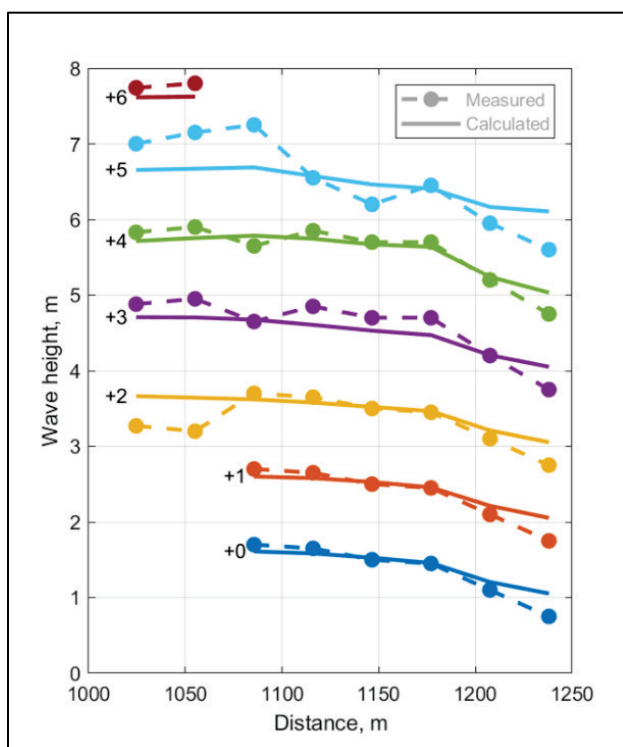


Figure 38. Cross-shore transects of measured and calculated longshore (left) and cross-shore (right) currents for Case 1 ($H = 1.65$ m, $T = 11$ sec). For display purposes, current velocities are shifted by the number indicated on the left-hand side of each transect.

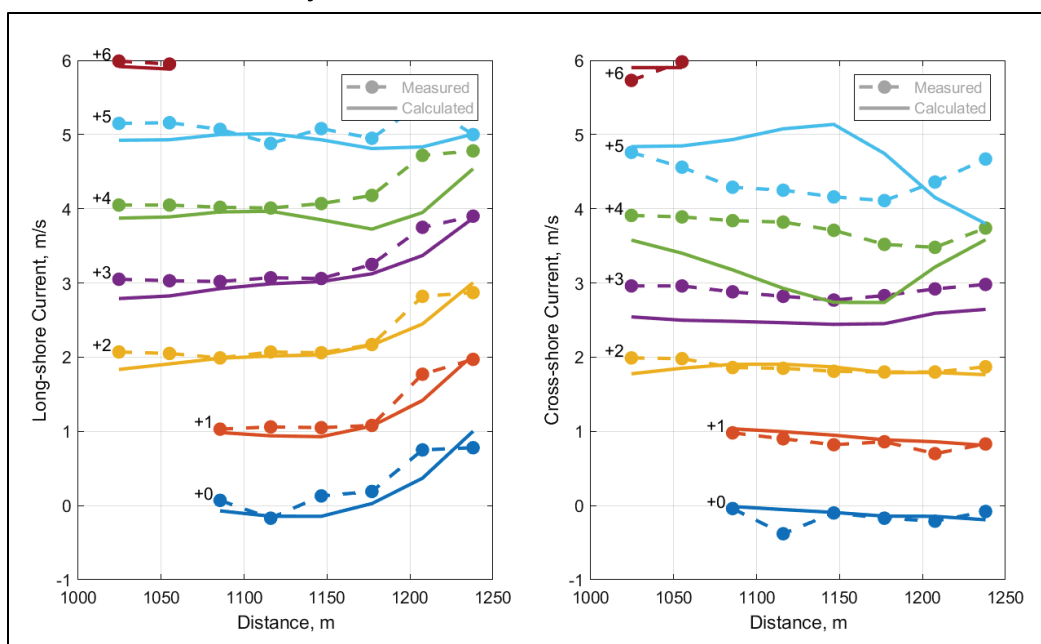


Table 26. Goodness-of-fit statistics* for Case 1 ($H = 1.65$ m, $T = 11$ sec).

Variable	NRMSE, %	NMAE, %	R ²	Bias
Longshore current	25.95	19.62	0.485	-0.070 m/sec
Cross-shore current	27.80	18.26	0.713	-0.149 m/sec
Wave height	15.23	12.71	0.825	-0.076 m

*defined in the appendix

3.5.4.2 Case 2 ($H = 2.0$ m, $T = 11$ sec)

The calculated wave height magnitudes and directions of Case 2 agree well with the measurements, especially far away from the jetty (Figure 39 and Figure 40). The wave height NRMSE, NMAE, and R^2 are 19.09%, 15.53%, and 0.841, respectively (Table 27). Closer to the jetty, the differences are larger, possibly due to reflected wave energy from the jetty. Even though the jetties are made of small stones and absorb most of the wave energy, a small portion of the wave energy is reflected. CMS-Wave has the capability to simulate reflecting waves. However, for this study it is assumed that the jetty reflectance is negligible. Additional tests will be conducted in the future to test this hypothesis. The breaker is located at approximately the fourth cross-shore measurement station from the shoreline and is predicted well by the model.

Figure 39. Measured and calculated wave height (left) and mean current (right) vectors for Case 2. Background colors indicate the local water depth corresponding the right color bar.

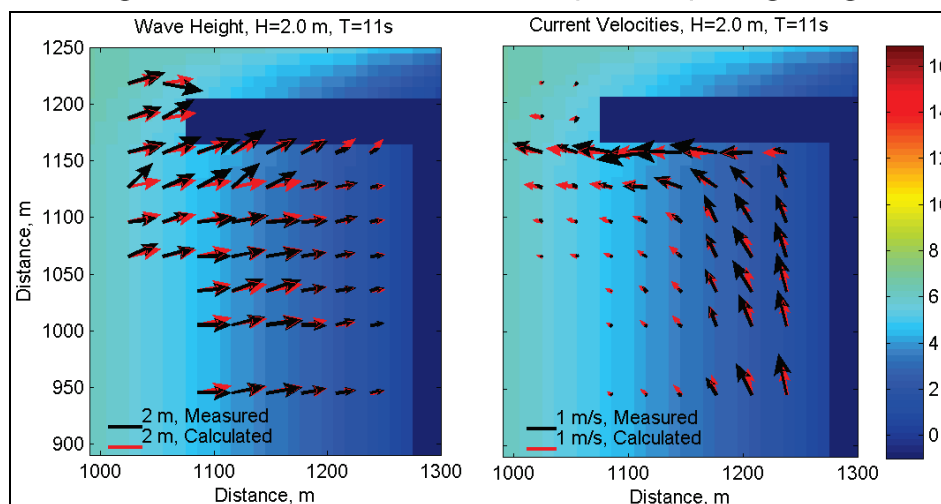
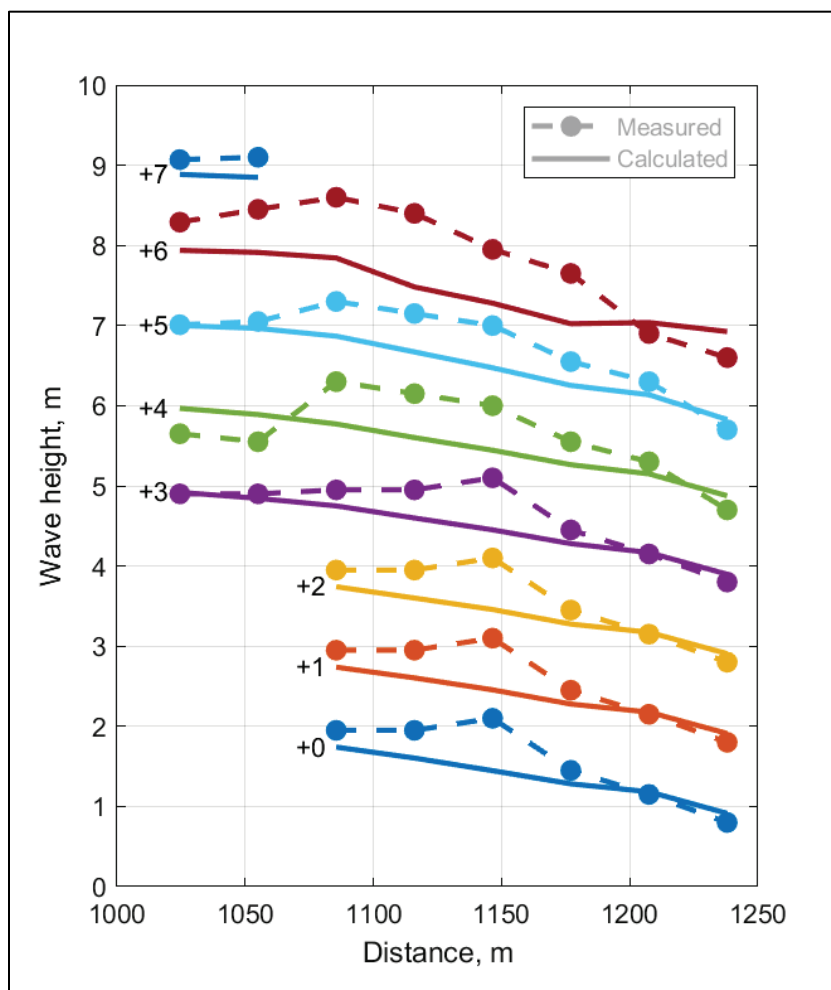


Figure 40. Cross-shore transects of measured and calculated wave heights for Case 2 ($H = 2.0$ m, $T = 11$ sec). For display purposes, wave heights are shifted by the number indicated on the left-hand side of each transect.



Measured and calculated current velocities for Case 2 along cross-sectional transects, shown in Figure 41, have NRMSE and NMAE values less than 31% and 23%, respectively. Although the breaker zone for Case 2 is wider than in Case 1, most of the longshore current is still located within the first three measurement stations from the shoreline. The calculated cross-shore currents tend to be underestimated near the shoreline and slightly overestimated outside of the breaker for all cross-shore transects except the one adjacent to the jetty.

Figure 41. Cross-shore transects of measured and calculated long-shore (left) and cross-shore (right) currents for Case 2 ($H = 2.0$ m, $T = 11$ sec). For display purposes, current velocities are shifted by the number indicated on the left-hand side of each transect.

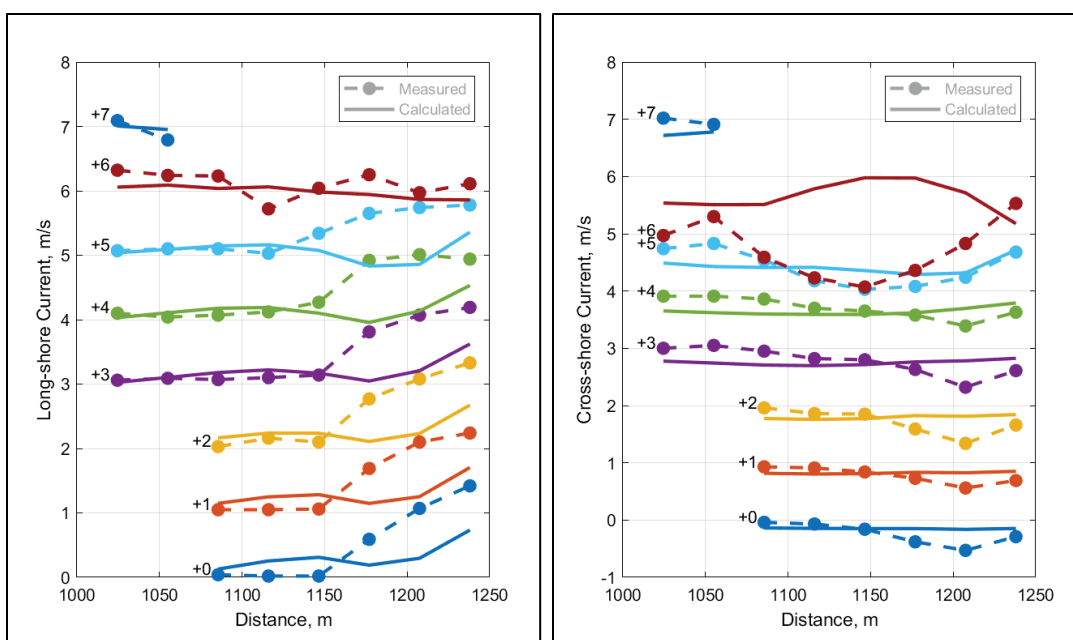


Table 27. Goodness-of-fit statistics* for Case 2 ($H = 2.0$ m, $T = 11$ sec).

Variable	NRMSE, %	NMAE, %	R ²	Bias
Longshore current	25.38	15.94	0.114	0.137 m/sec
Cross-shore current	30.56	22.46	0.570	-0.218 m/sec
Wave Height	19.09	15.53	0.841	-0.238 m

*defined in the appendix.

3.5.4.3 Case 3 ($H = 3.25$ m, $T = 8$ sec)

Figure 42 shows plan-view vector plots of the measured and computed wave heights and current velocities for Case 3. Cross-shore profiles of measured and computed wave heights, and current velocities are plotted

in Figure 43 and Figure 44. For this case, the calculated wave heights are slightly overestimated for most of the measurement locations, indicating that the wave breaker coefficient is slightly overestimated for this steep wave condition. Note that wave breaker coefficient calculation is intended for irregular waves and has not been calibrated for regular waves. Wave directions agree well with the measurements except for a few measurement stations where significant differences are observed in the incident wave angles. From the measurements it appears that the location of the breaker is outside of the measurement stations. The calculated longshore current velocities show the smallest NRMSE and NMAE of all three cases with values of 18% and 14%, respectively. Conversely, the cross-shore velocities show the largest NRMSE and NMAE values of all three cases with values of 33% and 26%, respectively (Table 28). Measured and computed current velocities for Case 3 agree reasonably well. However, the long-shore current speed tends to be overestimated near the breaker.

Figure 42. Measured and calculated wave height (left) and mean current (right) vectors for Case 3. Background colors indicate the local water depth corresponding the right color bar.

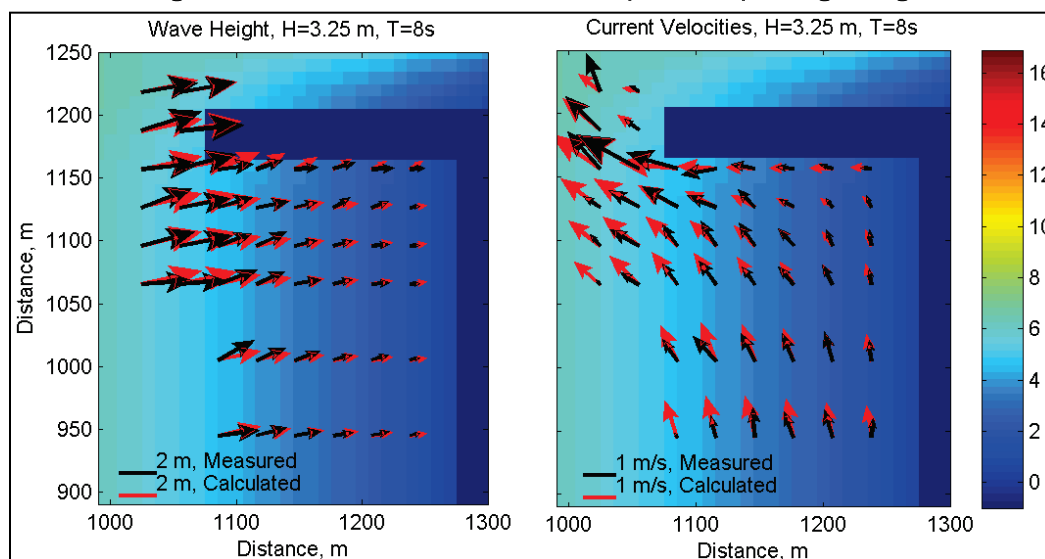


Figure 43. Cross-shore transects of measured and calculated wave heights for Case 3 ($H = 3.25$ m, $T = 8$ sec). For display purposes, wave heights are shifted by the number indicated on the left-hand side of each transect.

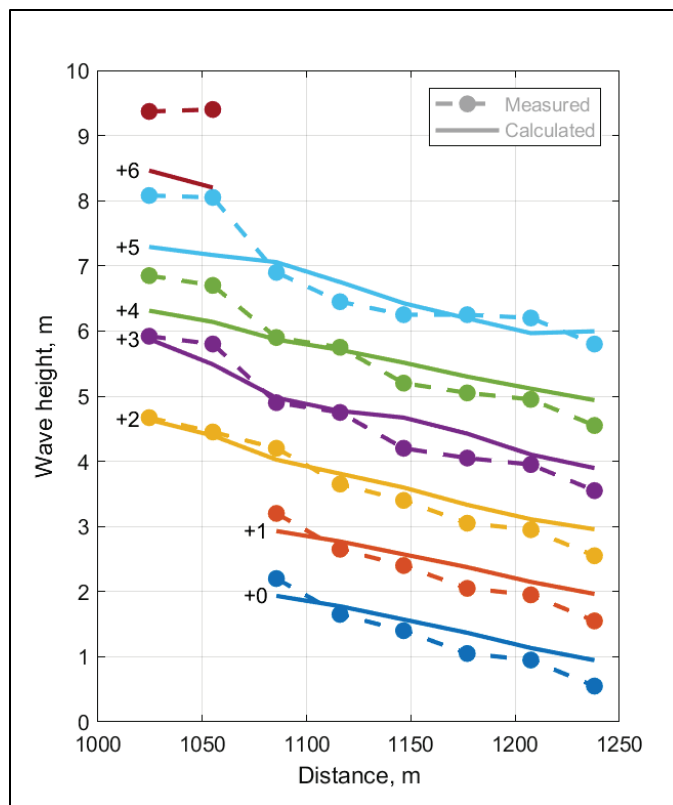


Figure 44. Cross-shore transects of measured and calculated long-shore (left) and cross-shore (right) currents for Case 3 ($H = 3.25$ m, $T = 8$ sec). For display purposes, current velocities are shifted by the number indicated on the left-hand side of each transect.

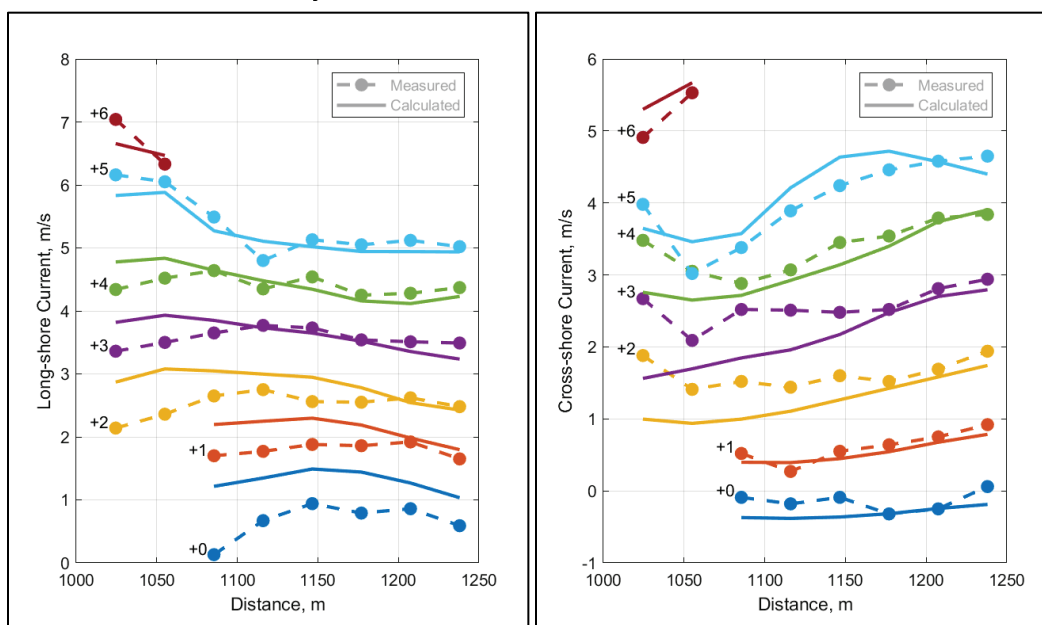


Table 28. Goodness-of-fit statistics* for Case 3 (H = 3.25 m, T = 8 sec)

Variable	NRMSE, %	RMAE, %	R ²	Bias
Longshore current	17.85	13.76	0.698	-0.158 m/sec
Cross-shore current	33.09	26.19	0.581	0.151 m/sec
Wave height	13.8	10.7	0.937	-0.012 m

*defined in the appendix.

3.5.5 Conclusions and recommendations

Laboratory experiments are used to validate the CMS for cross-shore and longshore currents and waves near an idealized inlet with two fully absorbing jetties. Measurements of regular waves and wave-induced currents are compared with CMS simulations at the prototype scale. The CMS is run using mostly default settings, except for the Manning's coefficient ($n = 0.025$) and roller dissipation coefficient ($\beta_D = 0.05$). Both parameters are held constant for all three cases. The value of the roller dissipation coefficient is the recommended value for regular waves. Model performance and behavior vary case by case, but in general, the calculated wave heights and wave-induced current velocities agree reasonably well with measurements as indicated by the goodness-of-fit statistics. Calculated nearshore wave heights and currents upstream of a jetty are found to be within approximately 11% – 16% and 14% – 33% of the measurements, respectively. CMS-Wave is able to accurately predict the location of the wave breaker. However, tests are conducted in a physical model without tidal currents, winds, and with well-known bathymetry and wave conditions, all of which represent additional potential sources of error in field applications.

These results indicate that once the model is calibrated for a specific site, using mainly the bottom roughness, the model may be applied at the same site for different wave conditions without having to recalibrate the model. Using the wave- and depth-averaged hydrodynamic equations for depth-uniform currents as derived by Svendsen (2006) significantly improves the nearshore currents, most noticeably by producing an offshore directed flow or undertow. Including the surface roller improves the longshore currents by moving the peak longshore current closer to the shoreline.

3.6 Test C2-Ex5: Wave breaking experiments on a planar beach

3.6.1 Description

To generate a longshore current with monochromatic incident waves breaking on a planar beach, Visser (1991) conducted eight laboratory experiments, labeled as Run 1 to Run 8. Wave, current, and water level data were collected for a number of incident wave conditions tested on two beach slopes (1:10 and 1:20) and for two different bottom roughness numbers. Although these experiments use monochromatic waves (e.g., no irregular wave tests were performed), the data provided are fundamental in checking wave and flow models for wave refraction, shoaling and breaking, and wave-induced currents. Lin et al. (2008, 2011a) provided additional analysis of these experiments with wave and flow models. Here, only Runs 4 to 7 are selected for model validation because these tests have the same bottom composite slopes and the most complete set of measurements. The beach has a 1:10 slope for the first seaward 1 m distance, 1:20 slope for the next 5 m distance, and a flat bottom for the next 5.9 m to the wave generator. Runs 4 through 6 are conducted on a concrete bed, where the bottom friction is expected to be small and therefore neglected in the numerical wave simulation. For Run 7, the 1:20 slope bottom is roughened by a thin layer (0.5 – 1.0 cm) of gravel grouted on the concrete floor. Table 29 presents the incident wave conditions.

Table 29. Incident wave conditions.

Run	$H_s(\text{cm})^*$	$T_p(\text{sec})^*$	$f_p(\text{Hz})^{**}$	$\theta(\text{deg})^{***}$
4	7.8	1.02	0.98	15.4
5	7.1	1.85	0.54	15.4
6	5.9	0.70	1.43	15.4
7	7.8	1.02	0.98	15.4

* Monochromatic wave.

** $f_p = 1/T_p$.

*** Wave direction relative to shore normal.

3.6.2 Model setup

CMS-Wave is run at laboratory scale, and unless otherwise noted, simulations are made with the default parameters. The model grid consists of 90 cross-shore and 243 alongshore square cells, each 10 cm × 10 cm, to cover the entire basin in these experiments. The spectral transformation is computed in CMS-Wave on 11 frequency bins (covering the range of +/- 0.05 Hz of the incident monochromatic wave frequency at 0.01 Hz

increment) and 35 direction bins (covering a half-plane with 5° spacing). The incident monochromatic, unidirectional wave spectrum is specified in a single frequency and direction bin at the seaward boundary. The input current field is interpolated cross-shore and averaged alongshore from the data. For Run 7 with the gravel floor, a constant Darcy-Weisbach type bottom friction coefficient of 0.01 is specified in CMS-Wave (Lin et al. 2008). This value is determined by a trial-and-error run and is used in all other runs. The model is not calibrated with data.

3.6.3 Results and discussion

Figure 45 shows an example of input current and water level fields derived from the measurements used in the CMS-Wave simulation for Run 4. “Exp” on these figures refers to the “Run” number. Figure 46 shows a comparison of the measured and calculated cross-shore wave heights for Runs 4 through 7. The calculated wave height agrees well with the measurements for four different depth-limiting breaking formulas implemented in CMS-Wave. Table 30 presents the statistics calculated for Runs 4 to 7. Comparatively smaller Mean Absolute Error (MAE) values occur for Runs 4 and 6 using the Extended Goda (Sakai et al. 1989) as well as the Battjes and Jassen (1978) wave-breaking formula; these error values are nearly the same for both formulas. The largest RMSE error in the calculated wave height (i.e., NRMSE), which occurred in Run 5 by the Goda, is 15.8% of the incident wave height along a planar beach. The smallest error is 4.9%, generated in Run 7 by the Goda as well.

Figure 45. Input current and wave setup fields with save stations (dot) for Run 4.

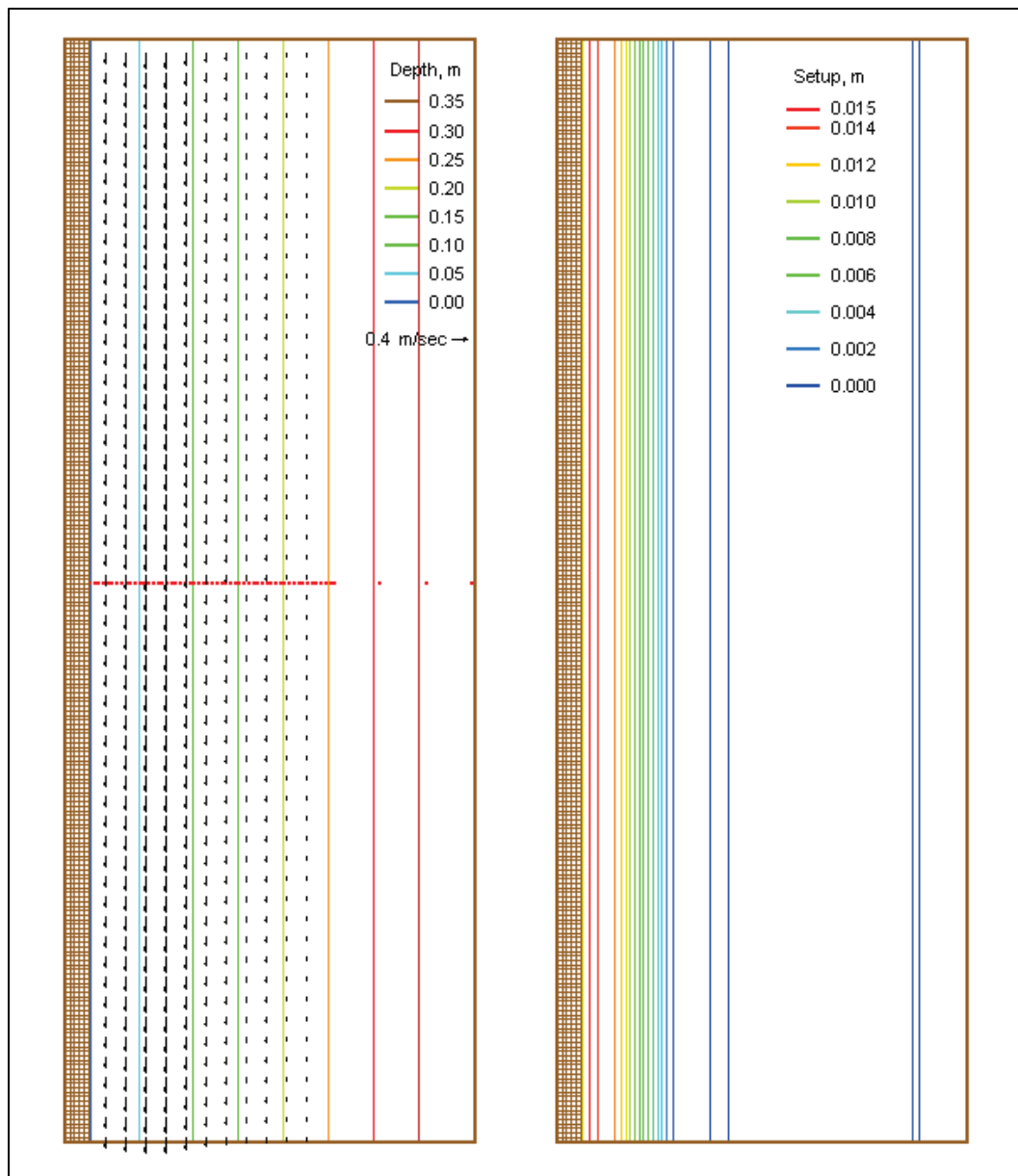


Figure 46. Measured and calculated wave heights, Runs 4 – 7, by Goda , Miche (Miche 1951.), B&J (Battjes and Jassen 1978), and C&K (Chawla and Kirby 2002) breaking formulas.

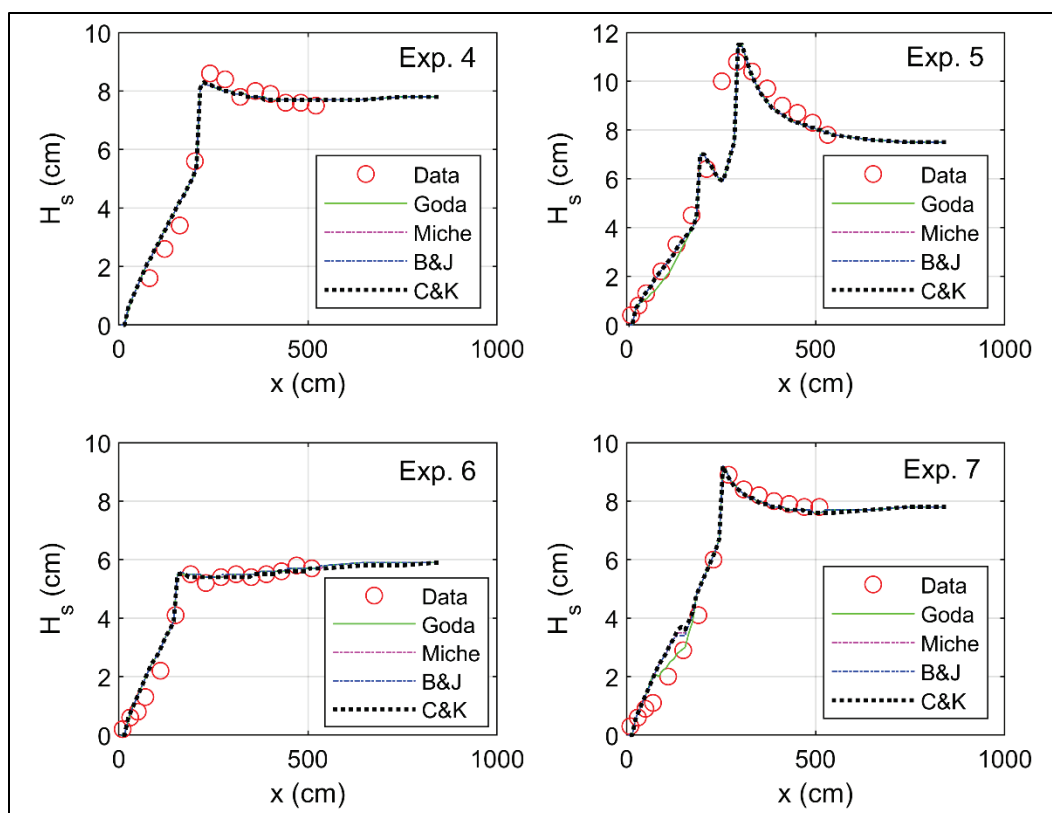


Table 30. Wave height statistics for planar beach, Runs 4 – 7, by Goda , Miche (Miche 1951.), B&J (Battjes and Jassen 1978), and C&K (Chawla and Kirby 2002) breaking formulas.

Case ID	Breaking formula	RMSE (cm)	NRMSE (%)*	R ²
Run 4	Goda	0.412	5.29	0.99
	Miche	0.412	5.29	0.99
	B&J	0.412	5.29	0.99
	C&K	0.412	5.29	0.99
Run 5	Goda	1.118	15.8	0.93
	Miche	1.100	15.5	0.93
	B&J	1.100	15.5	0.93
	C&K	1.100	15.5	0.93

Table 30. (cont.) Wave height statistics for planar beach, Runs 4 – 7, by Goda , Miche (Miche 1951.), B&J (Battjes and Jassen 1978), and C&K (Chawla and Kirby 2002) breaking formulas.

Case ID	Breaking formula	RMSE (cm)	NRMSE (%)*	R ²
Run 6	Goda	0.343	5.82	0.98
	Miche	0.345	5.84	0.98
	B&J	0.345	5.84	0.98
	C&K	0.348	5.89	0.98
Run 7	Goda	0.385	4.94	0.99
	Miche	0.465	5.96	0.99
	B&J	0.459	5.86	0.99
	C&K	0.482	6.18	0.99

*Normalized by model incident wave height.

3.6.4 Conclusions and recommendations

For oblique monochromatic waves breaking on a planar beach and interacting with the longshore current, in general, the calculated wave heights by CSM-Wave agree with all the four experimental data. The model with all the four wave breaking formulas have produced the results of wave heights with a high-order accuracy. In all four cases, the extended Goda breaking formula is slightly better than the other three breakers. Although these results are for monochromatic waves, other test cases provided under Categories 2 (Chapter 2) and 3 (Chapter 3) include applications with irregular waves. Therefore, this test case is necessary for validation of the wave model, to show that CMS-Wave represents wave refraction, shoaling and breaking, and wave-induced currents accurately.

3.7 Test C2-Ex6: Wave runup on impermeable uniform slope

3.7.1 Description

The purpose of this test case is to validate the wave runup on uniform slopes calculated by CMS-Wave with two laboratory experimental

datasets* (Mase and Iwagaki 1984). Detailed information on measurements, including dimensions of flume, gauge types and data analyses performed, are available from these references. Random incident waves are generated in both laboratory wave flumes. Each experiment in an individual flume consists of a flat bottom offshore of a sloping beach. The experiments by Ahrens and Titus include 275 wave conditions (Ahrens and Heimbaugh 1988), with significant wave heights ranging from 4 to 20 cm, spectral peak periods from 1.1 to 4.5 sec, and six uniform slopes (1:1, 2:3, 1:2, 2:5, 1:3, 1:4). The experiments by Mase and Iwagaki use 120 wave conditions (Mase 1989), with significant wave heights ranging from 2.7 to 11 cm, spectral peak periods from 0.8 to 2.5 sec, and four uniform slopes (1:5, 1:10, 1:20, 1:30).

3.7.2 Model setup

For these laboratory applications, CMS-Wave is set up for the laboratory scale, and unless noted otherwise, all numerical simulations are performed with the default parameters listed in the appendix. The model grid consists of 1,750 cross-shore and 100 alongshore square cells, each 2 cm × 2 cm, covering a 10 m long flat bottom and a 25 m long slope sections. The spectral wave transformation is computed with 30 frequency bins (from 0.1 to 1.26 Hz at 0.04 Hz increment for the Ahrens and Titus experiments, and from 0.02 to 2.05 Hz at 0.07 Hz increment for the Mase and Iwagaki experiments), and 35 direction bins (covering a half-plane with 5° spacing). Bottom friction and wave reflection are neglected in these simulations. The default parameters are used including the Extended Goda wave breaking formula. The runup calculation was described in Lin et al. (2011a).

3.7.3 Results and Discussion

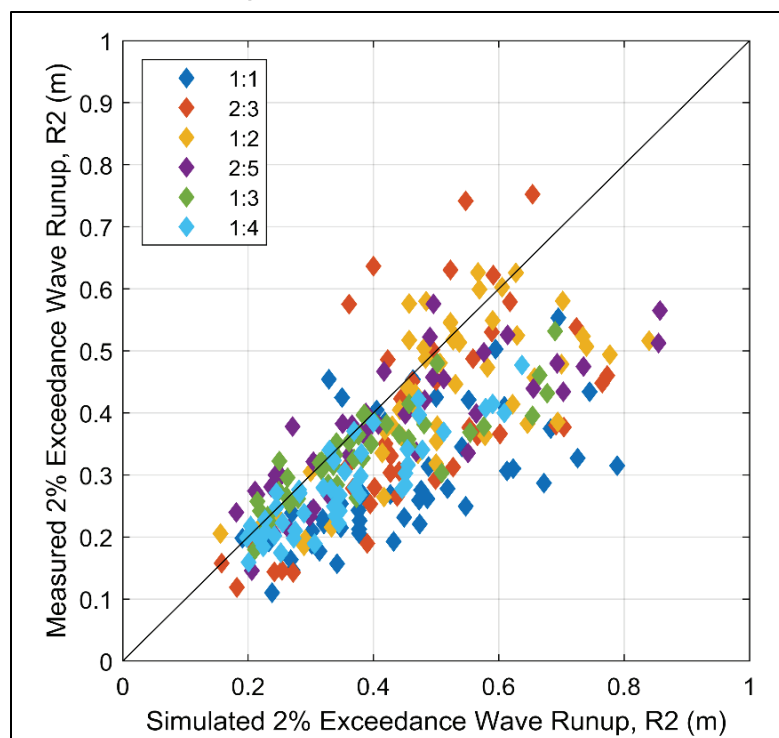
Figure 47 and Figure 48 show the calculated and measured 2% exceedance wave runup ($R_{2\%}$) for all experiments (a total of 395 cases) conducted by Ahrens and Titus[†] and by Mase and Iwagaki (1984). The 45° line in the figures represents a perfect match of calculated and measured values.

* Ahrens, J. P., and M. F. Titus. 1981 (unpublished). *Laboratory Data Report: Irregular Wave Runup on Plane Smooth Slopes*. Coastal Engineering Research Center. Vicksburg, MS: US Army Engineer Waterways Experiment Station.

† Ahrens, J. P., and M. F. Titus. 1981 (unpublished). *Laboratory Data Report: Irregular Wave Runup on Plane Smooth Slopes*. Coastal Engineering Research Center. Vicksburg, MS: US Army Engineer Waterways Experiment Station.

Tables 31 and 32 present the statistics between the calculated and measured R2% values for these two experimental studies. The calculated wave runup values correlate with data differently depending on slope. Overall, the MAE of calculated runup is small for all test cases except for the steepest slope (1:1) (dark blue diamonds in Figure 47), for which CMS-Wave overestimated the runup. A higher correlation, greater than 85% is obtained for gentler slopes (1:5 to 1:30) for the experiments by Mase and Iwagaki (1984) (Figure 48). For steeper slopes, more wave reflection (not calculated in these simulations because reflected waves were removed in data analyses) and less wave energy loss takes place on the slope section, and these affect the wave runup calculations. The CMS-Wave calculations of R2% agree with data (70% to 90% correlation, with RMSE errors less than 20 cm for incident wave heights ranging from 3 cm to 20 cm) for all test slopes, especially for gentler slopes (1:5 to 1:30).

Figure 47. Comparison of 2% exceedance wave runup for slopes of 1:1 to 1:4 with experimental data of Ahrens and Titus*.



* Ahrens, J. P., and M. F. Titus. 1981 (unpublished). *Laboratory Data Report: Irregular Wave Runup on Plane Smooth Slopes*. Coastal Engineering Research Center. Vicksburg, MS: US Army Engineer Waterways Experiment Station.

Figure 48. Comparison of 2% exceedance wave runup for slopes of 1:5 to 1:30 with the experimental data of Mase and Iwagaki (1984).

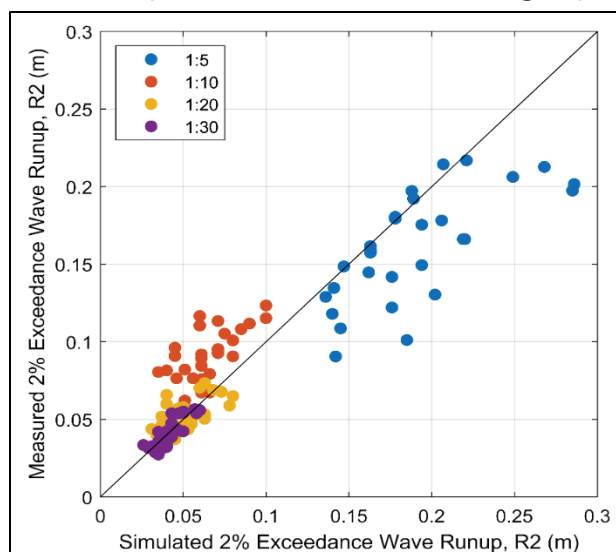


Table 31. Model wave runup accuracy statistics on impermeable uniform slopes of 1:1 to 1:4.

Slope	RMSE (cm)	NRMSE (%)*	R ²
1:1	0.195	34.8	0.37
2:3	0.159	20.9	0.41
1:2	0.133	21.2	0.49
2:5	0.122	20.9	0.65
1:3	0.093	19.0	0.71
1:4	0.093	19.4	0.76

*Normalized by model maximum wave runup on a uniform slope.

Table 32. Model wave runup accuracy statistics on impermeable uniform slopes of 1:5 to 1:30.

Slope	RMSE (cm)	NRMSE (%)*	R ²
1:5	0.041	18.7	0.50
1:10	0.029	23.0	0.40
1:20	0.011	14.1	0.35
1:30	0.005	9.6	0.71

*Normalized by model maximum wave runup on a uniform slope.

3.7.4 Conclusions and recommendations

The calculated 2% exceedance wave runup ($R_2\%$) shows better correlation with data for flatter slopes (1:5 to 1:30) than the steeper slopes (1:5 to 1:1). Meanwhile, the model overestimates the runup for the cases with steeper slopes but underestimates those with gentler slopes. This is expected because the runup is less over flatter slopes. The CMS-Wave runup function is applicable to coastal structures and beaches with the seaward slope less than 1:5. This test case demonstrates that CMS-Wave can be used for preliminary estimates of wave runup in projects. For steep slopes, CMS-Wave tends to over-predict runup, and estimates of wave runup may require using the phase-resolving nonlinear models or physical modeling studies.

3.8 Test C2-Ex7: Cleveland Harbor, Ohio, physical model

3.8.1 Purpose

The purpose of this test is to compare model waves with physical model experiments for Cleveland Harbor, Ohio (Bottin 1983). A 1:100-scale physical model experiment of Cleveland Harbor is conducted at the USACE Waterways Experiment Station to investigate waves, currents, water levels, and river flow on ship maneuverability (Bottin 1983) in the harbor entrance and within the harbor complex. The harbor is protected by two breakwaters with a combined length over 10 km (6 mi). The east breakwater consists of rubble mound stone, and the west breakwater is primarily composed of concrete caisson. The harbor has two entrances: the main entrance is located lakeward of the Cuyahoga River Mouth, and the east entrance is at the eastern end of the east breakwater. Figure 49 and Figure 50 show the harbor main entrance and modeled areas, respectively.

Figure 49. Physical model of Cleveland Harbor main entrance and wave/current data collection locations.

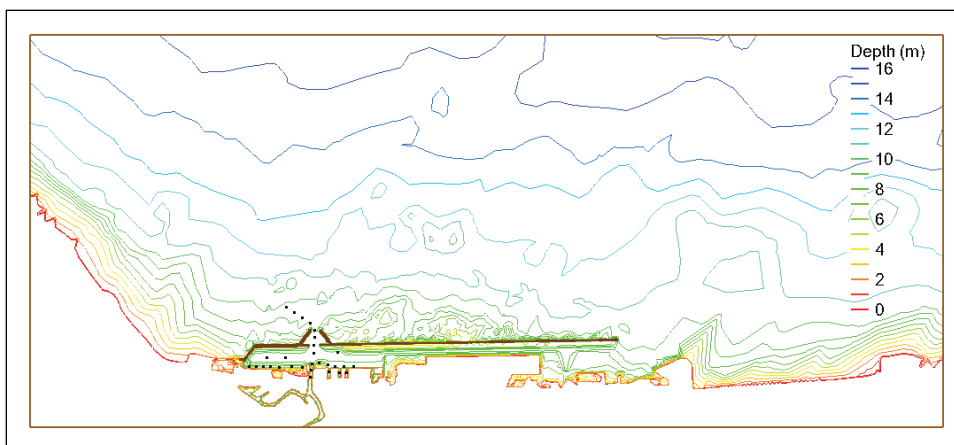
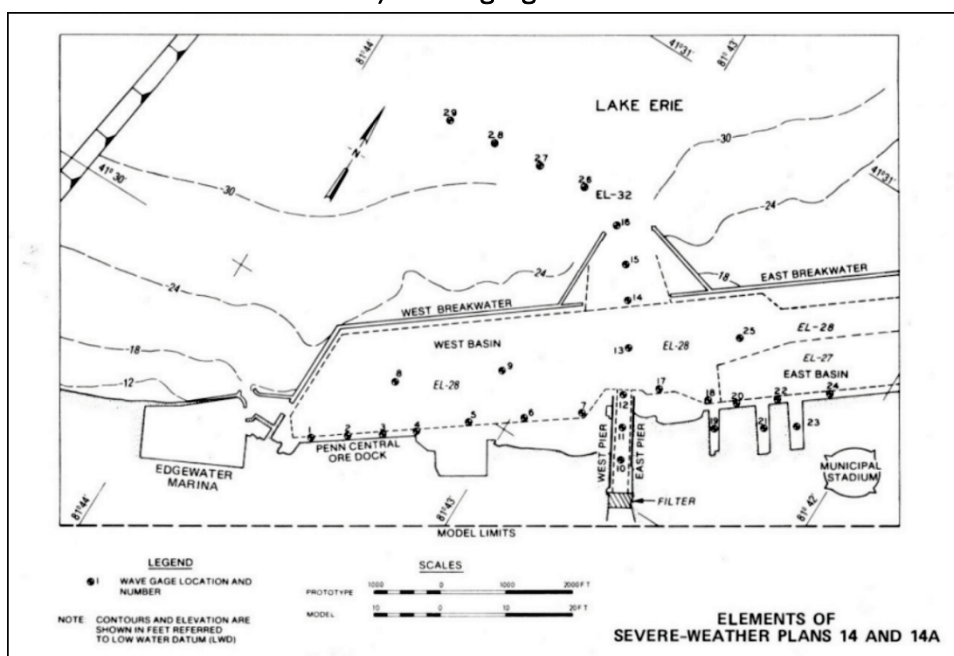


Figure 50. Physical and numerical model area for Cleveland Harbor; black dot denotes wave/current gauge location.



3.8.2 Model setup and parameters

The numerical model grid is oriented east-west with the offshore boundary at the 16 m depth contour and extends from the westward to the eastward ends of the Cleveland Harbor complex. The model simulation is run in the prototype as the laboratory data are made available in the prototype scale. The model domain is 8.6 km $\times$ 20 km, consisting of 860 $\times$ 2000 cells with constant cell spacing of 10 m.

The spectral wave transformation is computed with 30 frequency bins (0.04 to 0.33 Hz with 0.01 Hz increment) and 35 direction bins (covering a half-plane with 5° spacing). A constant Manning $n = 0.025$ is used for bottom friction. Wave-current interaction features, infra-gravity waves, wave transmission, and wave overtopping of breakwaters are used in the model (Lin et al. 2011a). A 50% reflection is specified at breakwater and harbor pier. The experiment tests 126 cases, including 20 incident wave heights (monochromatic wave), 12 wave periods, three wave directions (unidirectional), three lake water levels, and two river discharges. For model V&V, a most probable incident wave condition of 3.14 m and 8 sec at the mean lake level with wave direction nearly perpendicular to long breakwaters and a strong river influx of 227 m³/sec (8,000 cfs) is simulated. The background steady-state current field is supplied by a CMS flow model (Lin et al. 2011b).

3.8.3 Results and discussion

Figure 51 and Figure 52 show the model wave height fields in the harbor main entrance for simulations with and without the diffraction calculation, respectively. There are significant differences between model wave height and spatial variation of wave fields with and without diffraction calculation in the main entrance channel.

Figure 51. Model wave heights and directions without diffraction calculation; black dot denotes wave/current gauge location.

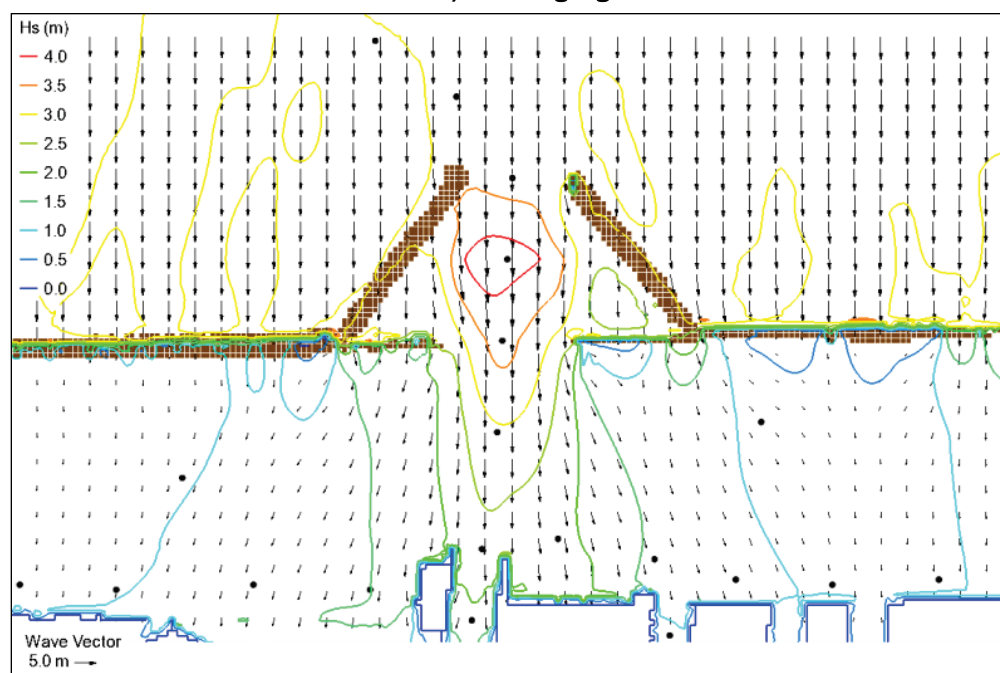


Figure 52. Model wave heights and directions with combined wave D-R; black dot denotes wave/current gauge location.

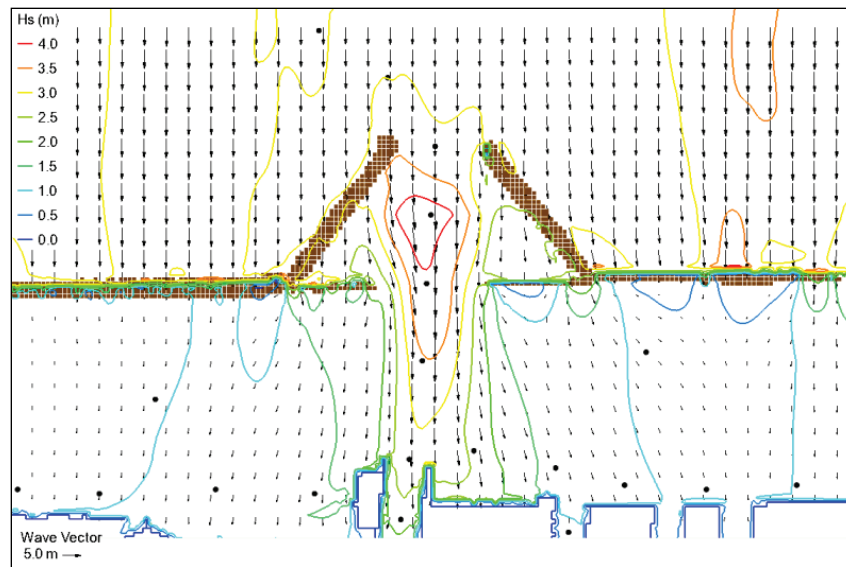


Figure 53 compares the model calculated wave height with data for simulations with and without the diffraction calculation. A 45° line shown in the figure indicates a perfect match between calculated and measured wave heights. The model wave height calculated with combined D-R agrees better with data than the calculation without the diffraction. This is especially true for large waves interacting with strong current from river influx in the main entrance channel.

Figure 53. Cleveland Harbor calculated wave height and data comparison.

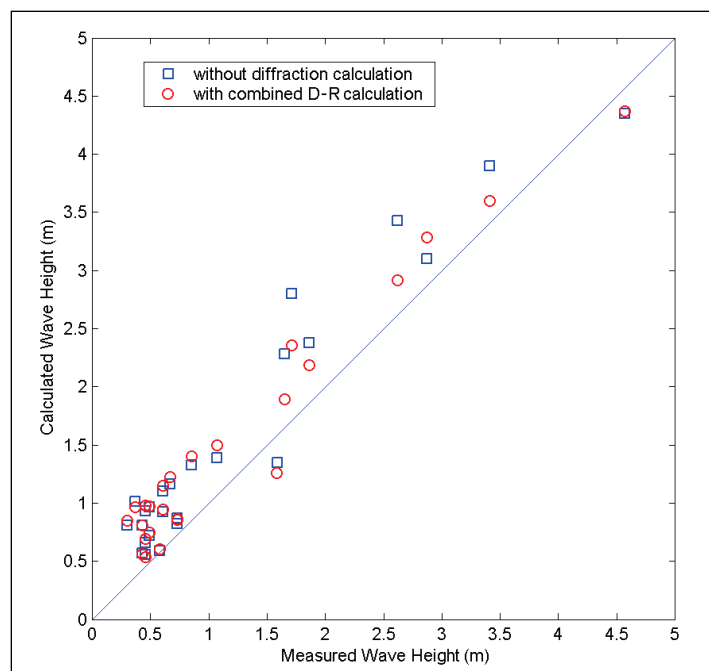


Table 33 lists the model wave height accuracy statistics for Cleveland Harbor experiments.

Table 33. Model wave height accuracy statistics with (1) combined wave refraction and diffraction and (2) with wave refraction only for Cleveland Harbor experiments.

Statistic	(1) with refraction and diffraction	(2) with wave refraction only
RMSE (m)*	0.30	0.38
NRMSE (%)**	7.41	9.38
R ²	0.83	0.72

*Scaled to prototype.

**Normalized by the maximum of incident wave height.

3.8.4 Conclusions and recommendations

CMS-Wave is tested and compared with model waves from a laboratory model experiment for Cleveland Harbor, Ohio (Bottin 1983). The Cleveland Harbor experiment corresponds to the real-world condition with combined coastal wave processes including diffraction, refraction, reflection, bottom friction, wave overtopping rubble-mound and vertical-wall breakwaters, and wave-current interaction at various wave levels. The model simulation includes a storm wave condition at the mean water level with a strong river influx in the back side of the harbor. Model wave heights overall agree well with data, especially for large waves interacting with strong current in the main entrance channel. The test shows CMS-Wave is both practical and reliable in the coastal harbor application.

4 Field Cases

4.1 Overview

As described in the early Category 3 V&V report (Sanchez et al. 2011), 10 field cases are selected to validate CMS-Flow for the real-world applications. With the updated CMS code, seven field cases are included in this report, which are listed as follows:

1. Grays Harbor, Washington
2. Merrimack Inlet, Massachusetts
3. Coos Bay Inlet, Oregon
4. Ogden Dunes, Indiana
5. Duck, North Carolina
6. Waukegan Harbor, Hawaii
7. Ashtabula Harbor, Ohio.

4.2 Test C3-Ex1: Grays Harbor, Washington

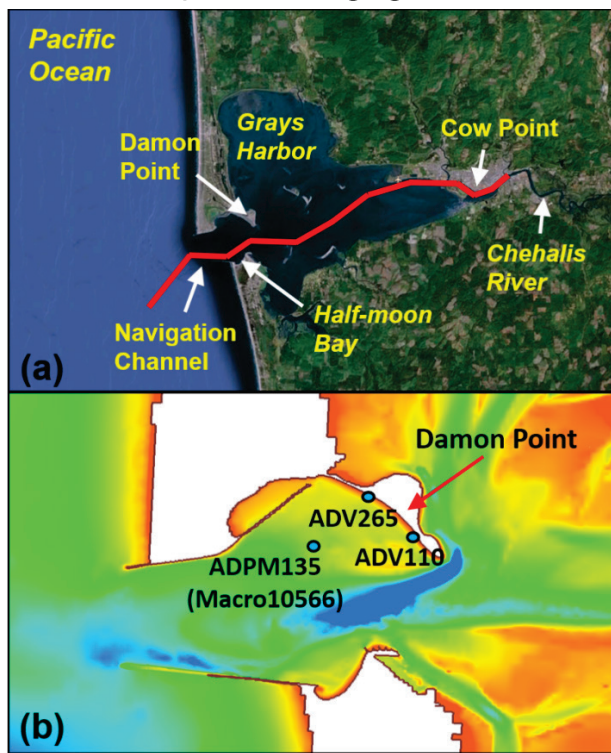
4.2.1 Purpose

The CMS validation at Grays Harbor, Washington, was conducted by Sanchez et al. (2011). In 2010, a data collection program was launched at the entrance to the harbor, which included the deployment of beach pods and bottom mounted acoustic Doppler profiler (ADP) along Damon Point to characterize waves, water levels, and currents. CMS validation of hydrodynamics and wave transformation is performed using the 2010 data.

4.2.2 Field study

Grays Harbor estuary is located on the southern coast of Washington, USA. The Chehalis River flows into the system from the east, and a large natural inlet on the west connects the harbor to the Pacific Ocean. The deep draft federal navigation channel is 39.1 km long and leads from the Pacific Ocean to Cow Point located on the Chehalis River. The channel is 300 m wide outside the inlet and narrows to 100 m near the Port of Grays Harbor at Cow Point (Figure 54).

Figure 54. (a) Grays Harbor estuary. (b) Locations of beach pods and ADP gauges.



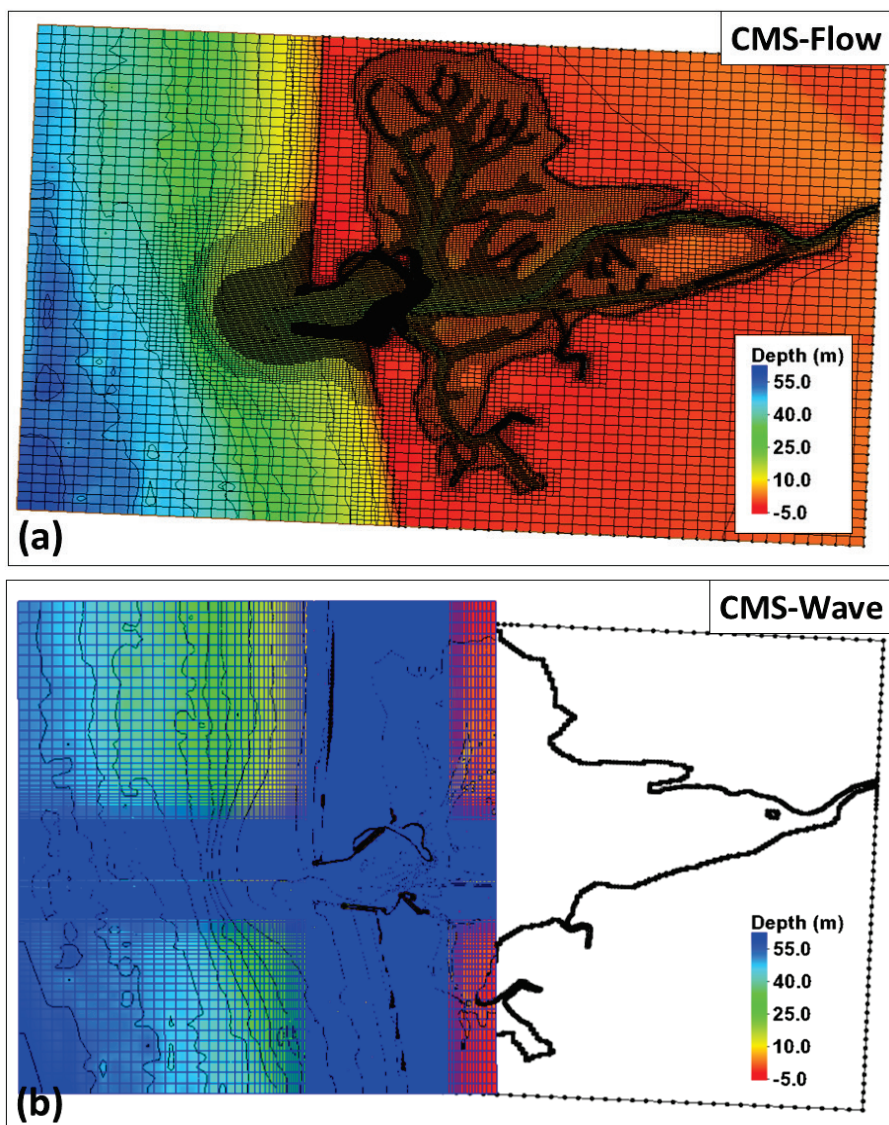
In recent years, Damon Point on the northeast portion of the harbor entrance has experienced continued evolution towards the existing navigation channel. The elongation of Damon Point, a spit located east of the north jetty, has caused the channel thalweg to migrate southeast. Spit encroachment on the navigation channel may lead to increased maintenance requirements.

A field data collection program was conducted in September 2010. Waves, water levels, currents, and sediment data were collected in the region of Damon Point and are used for model calibration and validation. Figure 53 shows the location of the two Beach Pods, ADV265 and ADV110, at the midpoint and near the distal end of the Damon Point spit, respectively. The bottom-mounted ADP, ADPM135, is deployed near the midpoint of Damon Point at approximately 7.5 m depth approximately 1.5 km offshore. This instrument with a pressure sensor measures current, WSE, and wave parameters.

4.2.3 Model setup

A telescoping grid is developed for CMS-Flow. The domain of this grid covers approximately 25 km alongshore and 20 km cross-shore distance from the shoreline to seaward boundary. The domain also covers 30 km into the embayment, as measured from the jetties, for a total coverage area of 1,100 km². The computational grid consists of 57,500 ocean cells ranging from 8 × 11 m to 480 × 720 m. CMS-Wave domain has a horizontal scale of 25 × 26 km. A non-uniform rectangular grid is used for the wave model. Similar to the discretization of the flow model, high-resolution grids of 20 × 20 m are specified for the nearshore area, and grid cell sizes increase to 600 m away from the shoreline (Figure 55).

Figure 55. (a) CMS-Flow telescoping grid and (b) CMS-Wave rectangular grid.



The hydrodynamic time-step is set to 15 min. A spatially constant Manning's n is calibrated as 0.022. The hydrodynamic model was forced with water level measurements at NOAA Westport tidal gauge (#9441102), and the tidal phase is adjusted based on the distance between this gauge and the model open boundary. Wave spectra and wind data are obtained from National Data Buoy Center (NDBC) buoy 46029, located approximately 87 km southwest of Grays Harbor in 135 m water depth. The 30-day simulation from 1 to 30 September 2010 is performed with the CMS. The model setup is summarized in Table 34. The 30-day simulation takes approximately 13.5 hr on a laptop computer using one 2.90 GHz processor.

Table 34. CMS model settings for the Grays Harbor test case.

Parameter	Value
Numerical Scheme	Implicit
Time-step	900 sec
Simulation duration	720 hr
Ramp duration	6 hr
Manning's n	0.022
Steering interval	3 hr

4.2.4 Results and discussion

Figure 56 to Figure 58 compare the computed and measured water levels, current velocities, and wave parameters at three ADP stations. The agreement between calculated and measured water levels is generally good, as demonstrated by the goodness-of-fit statistics presented in Table 29. ADV265 is located on the north western end of Damon Point and has a water depth of less than 1 m. ADPM135 is deployed at approximately 7.5 m MSL depth near the midpoint of and 1.5 km offshore of Damon Point. Model bias in Table 35 shows a slight under prediction of water level at ADV265, which may be associated with bathymetry difference between model and in situ measured depths. Larger model depth at the location results in lower ebb tidal level at the wetting/drying grid cell during low tide.

Figure 56. WSE comparisons between the CMS results and the 2010 field data.

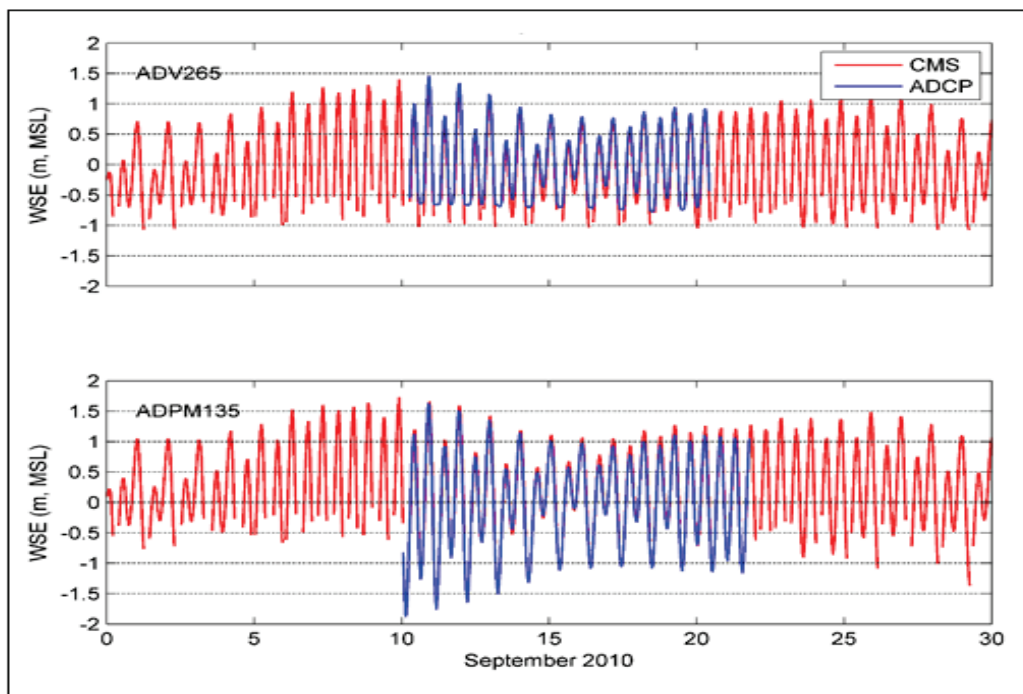


Table 35. Goodness-of-fit statistics of WSE for Grays Harbor, Washington: 2010 field data.

Statistics	ADV265	ADPM135
NRMSE [%]	8.082	2.983
RMSE [m]	0.178	0.104
NMAE [%]	7.195	2.386
R^2	0.981	0.984
Bias [m]	-0.156	0.034

Figure 57. Current comparisons between the CMS results and the 2010 field data.

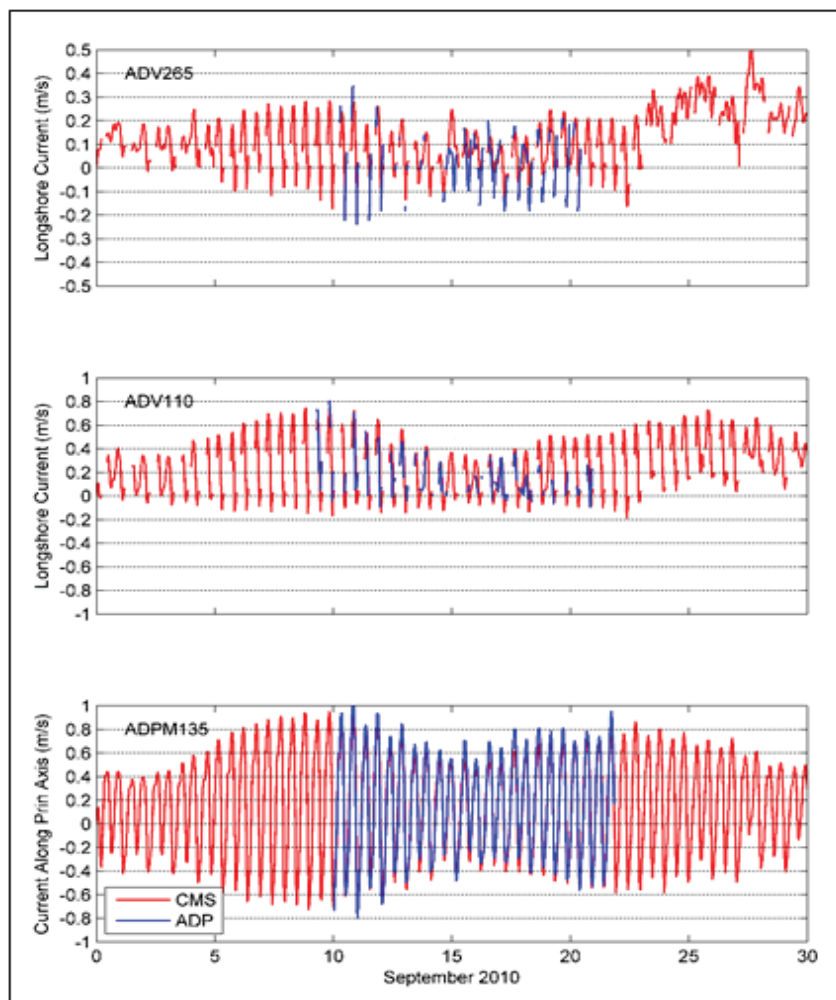
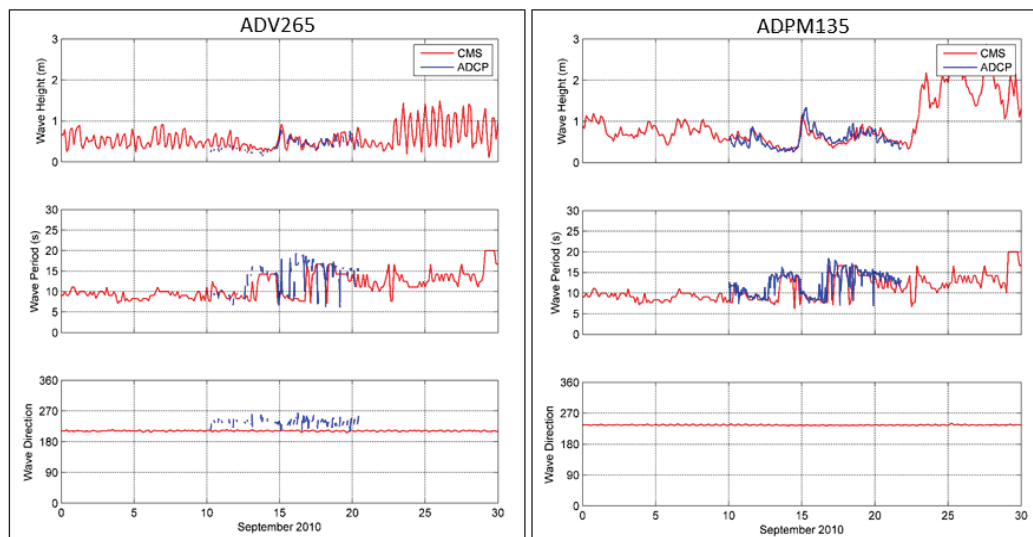


Figure 58. Wave parameter comparisons between the CMS results and the 2010 field data at (a) ADV265 and (b) ADPM135.



Measured (estimated from vertical velocity profile) and calculated depth averaged current velocities are compared along the current principal axis, which represents the longshore component of the current velocity at ADV265 and ADV110, and onshore component at ADPM135. Peak ebb and flood current velocities range from approximately 0.8 to 1 m/sec. Overall, the model predicts the amplitude and phase of the principal current velocities well. The NRMSEs for the principal current velocities are between 10% and 17% at the wetting/drying gauges. The model performed much better at the deeper gauge, ADPM135, with the smallest NRMSE of approximately 5% and the strongest correlation of 0.966 (Table 36). This performance is most likely due to a combination of the model limitations with regards to currents and the locations of ADV265 and ADV110, where the computational grid cells get drying at low tide.

Table 36. Goodness-of-fit statistics of principal current velocity for Grays Harbor, Washington: 2010 field data.

Statistics	ADV265	ADV110	ADPM135
NRMSE [%]	14.483	17.056	5.200
RMSE [m]	0.087	0.154	0.094
NMAE [%]	11.067	13.422	4.150
R ²	0.542	0.531	0.966
Bias [m]	0.029	0.032	-0.013

Wave validation is performed for both ADV265 and the nearshore ADPM135. The results are provided in Figure 57 while goodness-of-fit statistics are detailed for significant wave height and peak wave period in Table 37. Wave direction is not measured for the nearshore ADPM135, but the calculated values are provided in the bottom panel for reference. The goodness-of-fit statistics indicates that the model better reproduces wave parameters at the deeper gauge. In shallow areas, the selection and test of breaking formulas could be the key to improve the simulations of wave propagation and dissipation.

Table 37. Goodness-of-fit statistics of significant wave height and peak wave period for Grays Harbor, Washington: 2010 field data.

Statistics	ADV265		ADPM135	
	Significant Wave Height (m)	Peak Wave Period (sec)	Significant Wave Height (m)	Peak Wave Period (sec)
NRMSE [%]	16.688	22.078	10.169	14.917
RMSE	0.134	4.195	0.132	2.685
NMAE [%]	12.775	14.995	7.815	10.006
R ²	0.424	0.111	0.582	0.412
Bias	0.069	-1.860	0.010	-1.180

4.2.5 Conclusions and recommendations

The CMS performance in simulating tidal hydrodynamics at a coastal inlet in the presence of wave and wind is tested for Grays Harbor, Washington, using field measurements of water levels, current velocities, and waves. Water levels, depth-averaged principal current velocities, and wave parameters (significant wave height, peak wave period, and wave direction) are compared at three gauges, and goodness-of-fit statistics are used to assess the model performance. In general, the model results agree well with measurements in nearshore breaking zone and in offshore deeper area. The model results demonstrate that it is reasonable to use large time-steps, on the order of 15 min, for similar tidal inlet hydrodynamic studies. Using such a large time-step will, however, reduce the accuracy of the wetting and drying. If this is considered to be an important aspect of the study, then a smaller time-step may be more adequate.

4.3 Test C3-Ex2: Merrimack Inlet, Massachusetts

4.3.1 Purpose

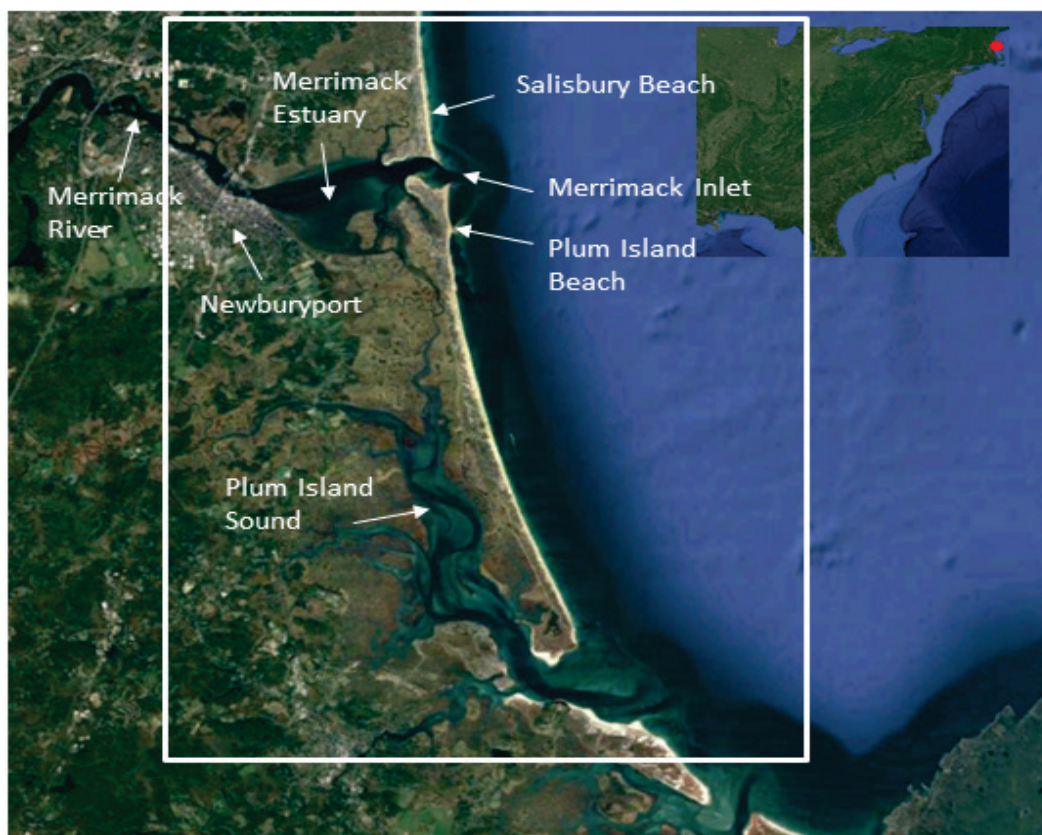
Following the Grays Harbor modeling study, a case on the high-energy West Coast, the Merrimack Inlet case is presented here as an inlet estuarine example on East Coast. The numerical model is developed to evaluate the local sediment management strategy and structure modification design adjacent to Merrimack Inlet, Newburyport, and nearshore in the vicinity of Plum Island, Massachusetts. Hydrodynamic

validation is performed against measurements at the inlet and in Plum Island Sound.

4.3.2 Field study

Merrimack Estuary is an inlet system located in northern Massachusetts near the border with New Hampshire (Figure 59). Merrimack Inlet is adjacent to Salisbury Beach to its north and Plum Island to the south. Newburyport Harbor is located on the southern side of Merrimack Estuary approximately 3.5 mi (5.6 km) from the mouth of the Merrimack River. The Merrimack Embayment, which includes Merrimack Inlet as well as the adjacent Salisbury Beach and Plum Island, is characterized as a mixed-energy and tide-dominated inlet (FitzGerald and Van Heteren 1999).

Figure 59. Site location of Merrimack Inlet and Newburyport.



In Merrimack Estuary and Newburyport Harbor study area, jetty degradation and severe beach erosion have become critical issues for the USACE. As a component of an ongoing Section 204 (beneficial reuse of dredged material) study, numerical modeling is applied to examine

shoaling within the inlet and sediment transport throughout the estuarine and nearshore area. The modeling study includes the development of a coastal wave, hydrodynamic, and sediment transport model for the region, model calibration and validation to field data, and the development and evaluation of alternatives for sediment management and structure modification (Li et al. 2018).

A field data survey is conducted from September through December 2012. Four water level gauges are deployed in Merrimack Estuary and Plum Island Sound, and two acoustic Doppler current profilers (ADCPs) are deployed around the inlet system, one being on the south flank of the channel and one around the south jetty (Figure 60 and Figure 61). WSE and current data are collected from these two ADCPs.

Figure 60. The CMS domain and locations of water level gauges in Plum Island Sound.

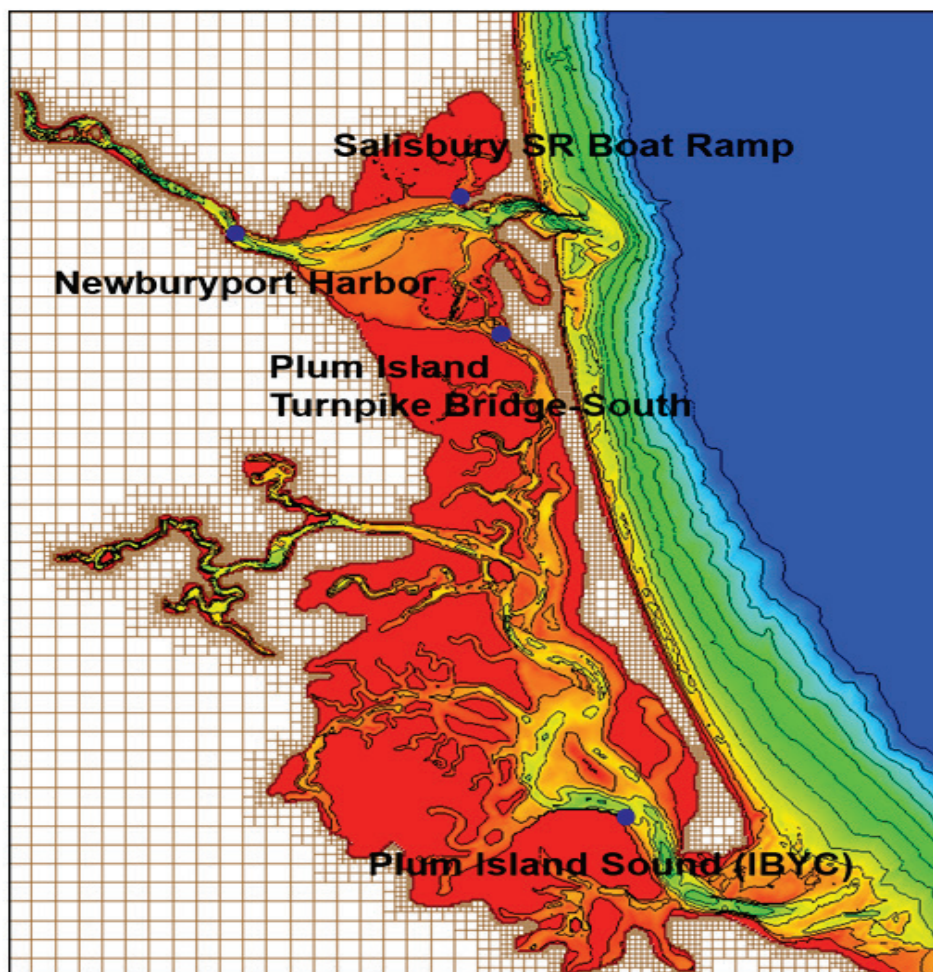
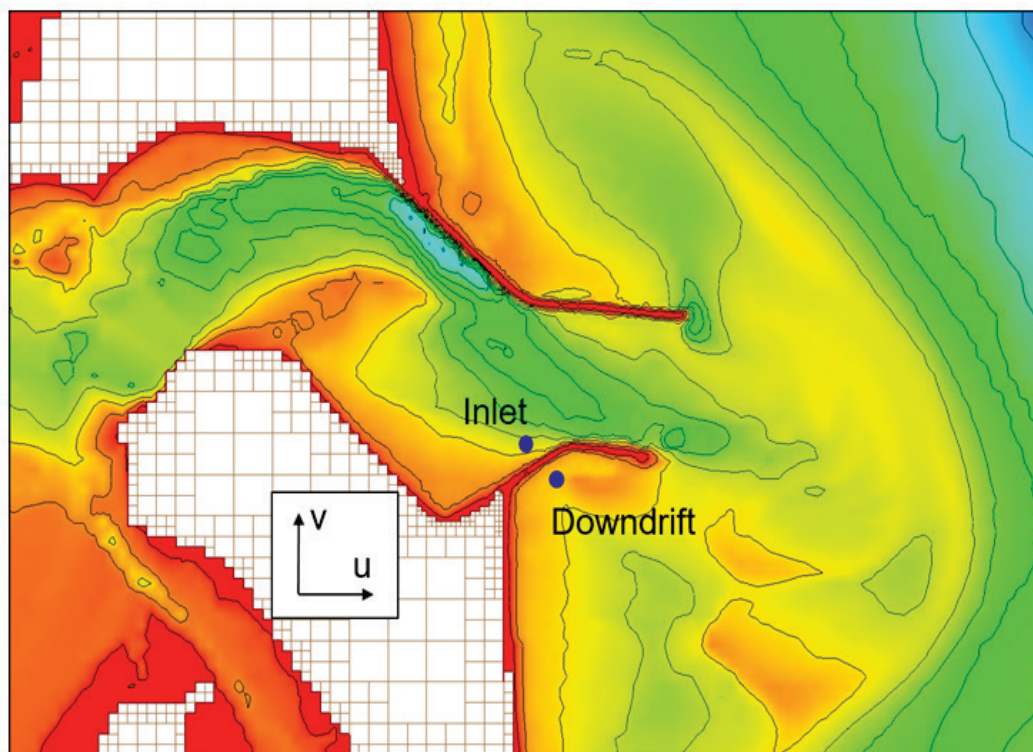


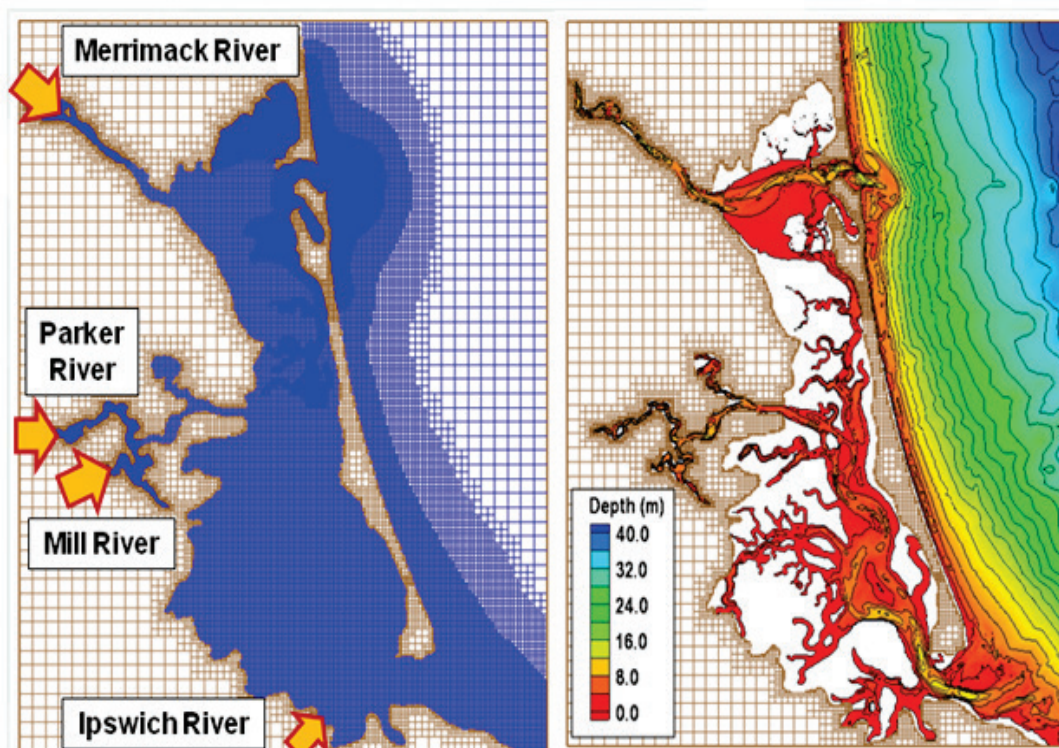
Figure 61. Locations of deployed inlet and downdrift ADCP gauges.



4.3.3 Model setup

A telescoping grid is developed for this coastal inlet and estuarine system. The CMS grid domain consists of 73,000 ocean cells, which covers the entire Plum Island Sound and the open ocean region. It extends approximately 20 km alongshore and 3 – 7 km offshore. The water depth ranges from 1 to 2 m above the mean sea level at tidal marsh areas in the Plum Island Sound to 13 m at the inlet navigation channel, and further increases to 40 m in the offshore boundary of the CMS domain. The telescoping grid system utilized by the CMS permits much finer local grid resolution to resolve hydrodynamic and sediment features well in areas of high interest. For this study the finer cell sizes vary from 6 to 12 m around the Newburyport Inlet, the creeks, and narrow channels linking the north and south Sound, to 400 m in the open ocean (Figure 62).

Figure 62. The CMS telescoping grid and bathymetry, and river inflow locations.



The hydrodynamic time-step is set to 10 min. A spatially varying Manning's n is calibrated, which ranges from 0.021 in open areas to 0.045 around inlet jetties. The hydrodynamic model is forced with water level measurements at NOAA tide gauge #8423898 at Fort Point, New Hampshire. Wave data are available from the NDBC buoy #44098 located about 50 km offshore of the CMS open boundary. Directional wave spectra are retrieved for CMS simulations in a 2 hr interval and transformed to the model seaward boundary. Wind observations are provided by another NDBC buoy IOSN3, which is located approximately 17 km northeast of the CMS domain (Figure 63). The 60-day simulation from 1 September to 30 October 2012 is performed with the CMS. The model setup is summarized in Table 38. The 60-day simulation takes approximately 34 hr on a laptop computer using two 2.90 GHz processors.

Figure 63. NOAA tide gauge and buoy locations. The rectangle indicates the CMS domain.

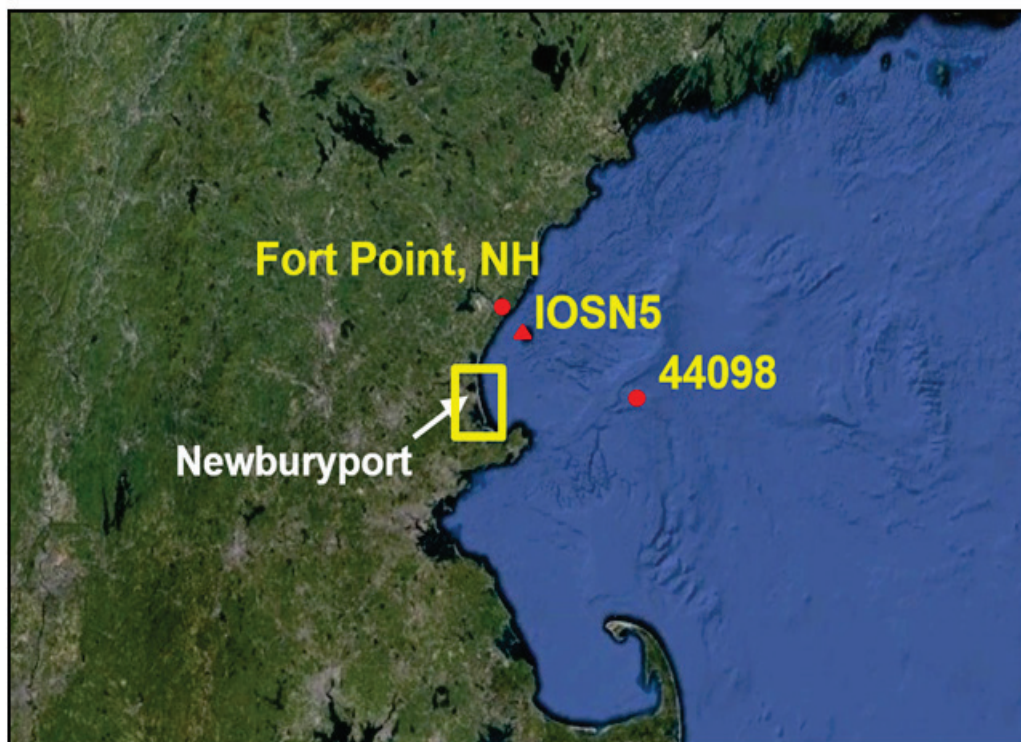


Table 38. CMS model settings for the Merrimack Inlet test case.

Parameter	Value
Numerical Scheme	Implicit
Time-step	600 sec
Simulation duration	1440 hr
Ramp duration	6 hr
Manning's n	0.021 - 0.045
Steering interval	2 hr

4.3.4 Results and discussion

The model is validated with water level and current data from October 2012. The calculated results are compared with the tidal and ADCP gauge data, and the model performance is measured by goodness-of-fit statistics.

Figure 64 shows the calculated and measured WSEs at the four tidal gauges in Merrimack Estuary and Plum Island Sound. Both the measurements and calculations show that the spring tidal amplitude is close to 3.5 – 4 m in the area. A small storm surge occurred near the end of the 30-day simulations as indicated by the increase in WSEs. The CMS results reproduce the amplitude and phase of tide, as well as the

low-frequency storm event displayed in the survey. Table 39 lists the goodness-of-fit statistics of WSEs. The correlation coefficients (R^2) for the water level measurements are extremely good, ranging from 0.965 to 0.996. The RMSEs range from 0.069 to 0.195 m, and the NRMSEs from 1.6% to 5.6% at the four tidal gauges. Note that the model performs better in Plum Island Sound than in Merrimack Estuary. Primary factors that can affect tidal prism and propagation include estuarine geometry, bay bathymetry, bottom friction etc.

Figure 64. Comparison of measured (blue line) and calculated (red line) water levels in Plum Island Sound.

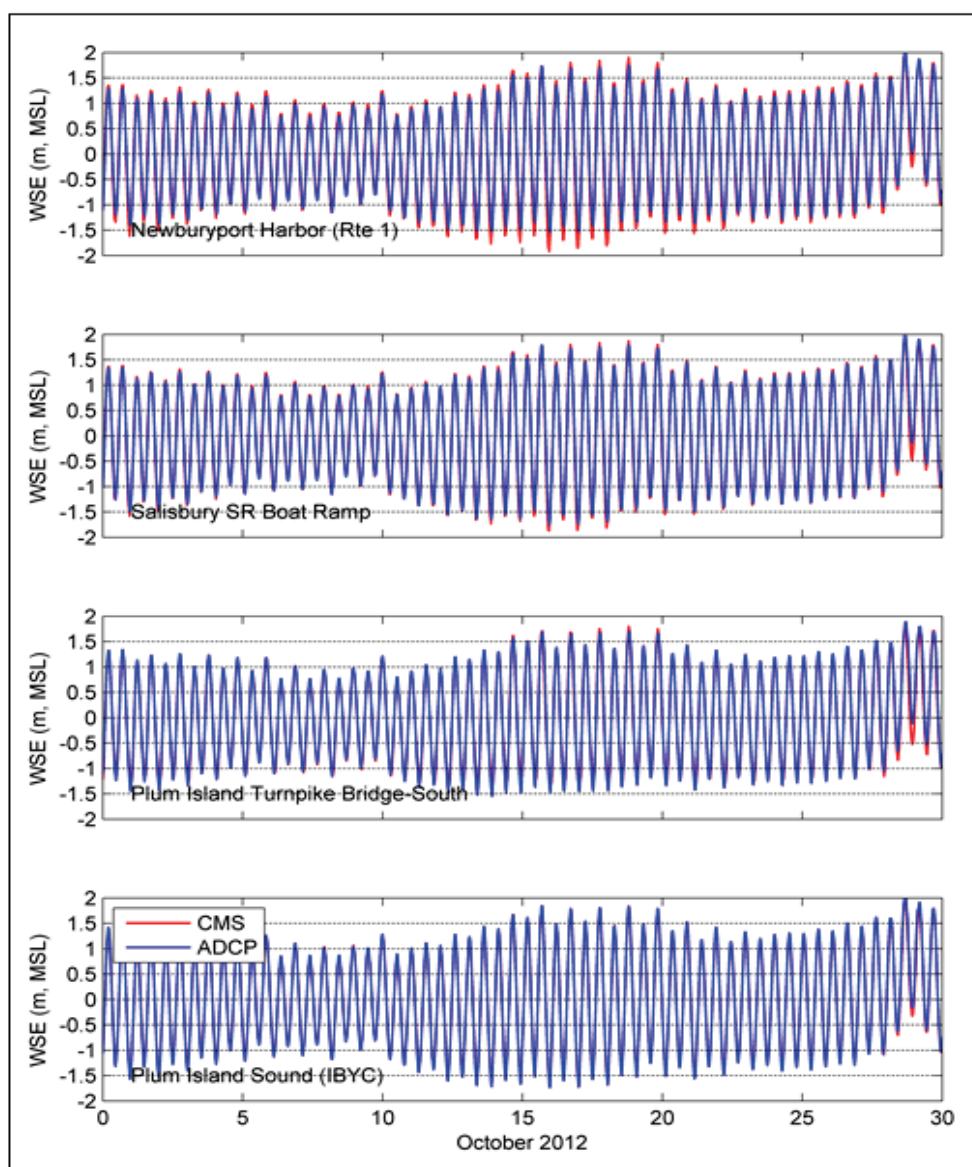


Table 39. Goodness-of-fit statistics of WSE for Merrimack Inlet, MA: 2012 field data.

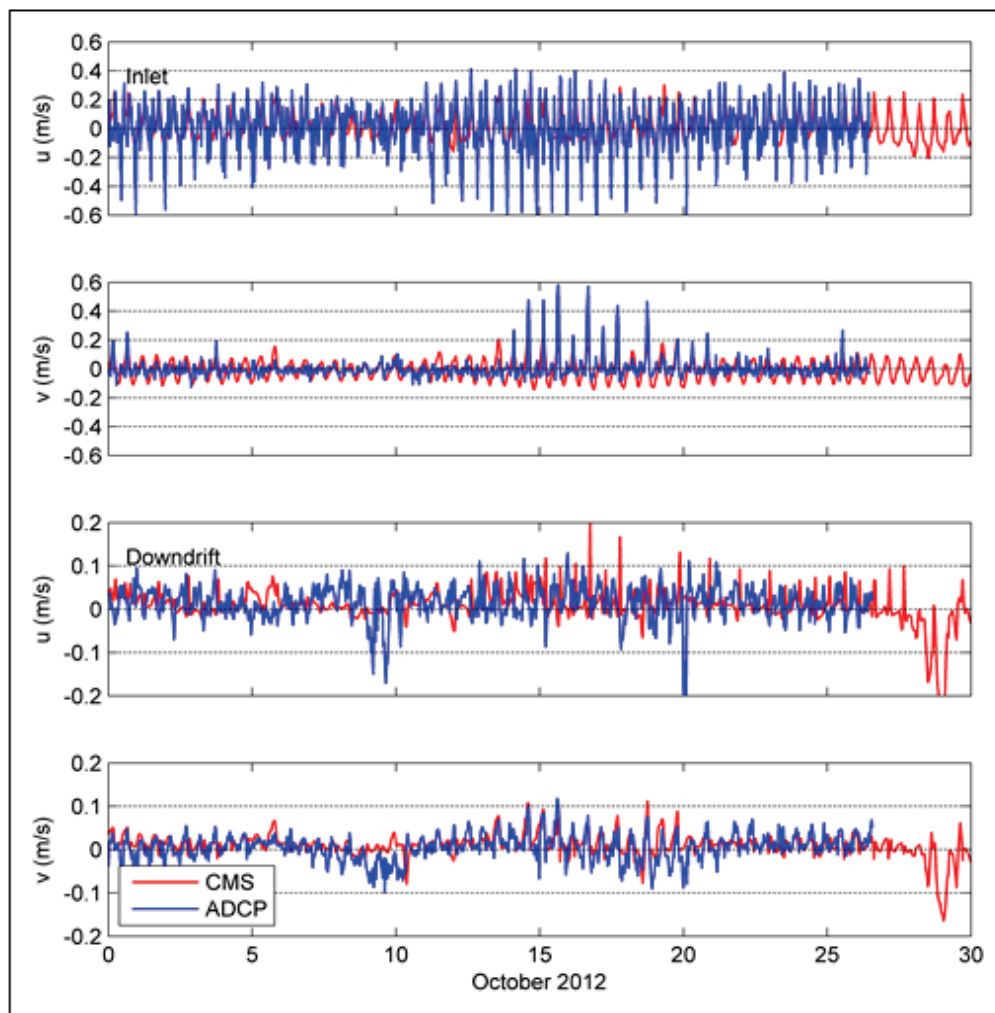
Statistics	Newburyport Harbor	Salisbury SR Boat Ramp	Plum Island Turnpike Bridge-South	Plum Island Sound (IBYC)
NRMSE [%]	5.583	3.305	3.845	1.622
RMSE [m]	0.195	0.122	0.119	0.060
NMAE [%]	4.529	2.697	2.765	1.351
R ²	0.965	0.986	0.982	0.996
Bias [m]	0.008	0.009	-0.003	0.008

The model results are compared to the east-west (u) and north-south (v) components of the current data measured at the inlet and downdrift ADCP gauges (Figure 65). Table 40 lists the goodness-of-fit statistics for the gauges. Peak ebb and flood current velocities range from approximately 0.6 to 0.7 m/sec at the inlet gauge and mostly less than 0.1 m/sec at the downdrift gauge. The RMSEs for the current comparison at the Inlet gauge show a relatively high correlation. At the downdrift gauge, only the v- (east-west) component has a higher correlation; the u-component shows little correlation. Although the model-data correlations are low, the other parameters of the goodness-of-fit statistics are within reasonable ranges.

Table 40. Goodness-of-fit statistics of current velocities for Merrimack Inlet, MA, 2012 field data.

Statistics	Inlet		Downdrift	
	u	v	u	v
NRMSE [%]	14.840	10.857	15.867	14.250
RMSE [m/sec]	0.148	0.076	0.048	0.029
NMAE [%]	11.070	8.457	11.767	10.500
R ²	0.272	0.200	0.019	0.234
Bias [m/sec]	0.014	-0.007	0.000	0.010

Figure 65. Simulated and measured currents in Merrimack Inlet for October 2012.



The current comparisons are not ideal, but the inlet location is off the channel and the downdrift location is protected by the south jetty. Both the measurements and the model results at these sites show relatively weak currents, which presents a challenge for model to properly simulate the process of current-structure interaction. In addition, because of small current speeds, any perturbation caused by instrument instability could result in significant inconsistency within model-data comparisons, such as the measured u-component spike at the downdrift ADCP approximately 21 October 2012.

4.3.5 Conclusions and recommendations

Driven by measured water levels, current velocities, and waves, the CMS is set up to simulate tidal hydrodynamics at an East Coast site, Merrimack

Inlet and Plum Island Sound, Massachusetts. Water levels are compared at four gauges in Merrimack Embayment and depth-averaged current components at two ADCP stations around the inlet South jetty. The calculated water levels reproduce the measurements well in the estuary. The goodness-of-fit statistics shows weak correlation between the calculated and the measured depth-averaged current velocities, but the other statistical parameters present reasonable values. Merrimack Embayment includes Merrimack Estuary and Plum Island Sound. The linkage between two basins is a tidal marsh area, for which determining the bathymetry becomes crucial in correctly simulating tidal prism in the embayment and tidal currents around the inlet.

4.4 Test C3-Ex3: Coos Bay Inlet, Oregon

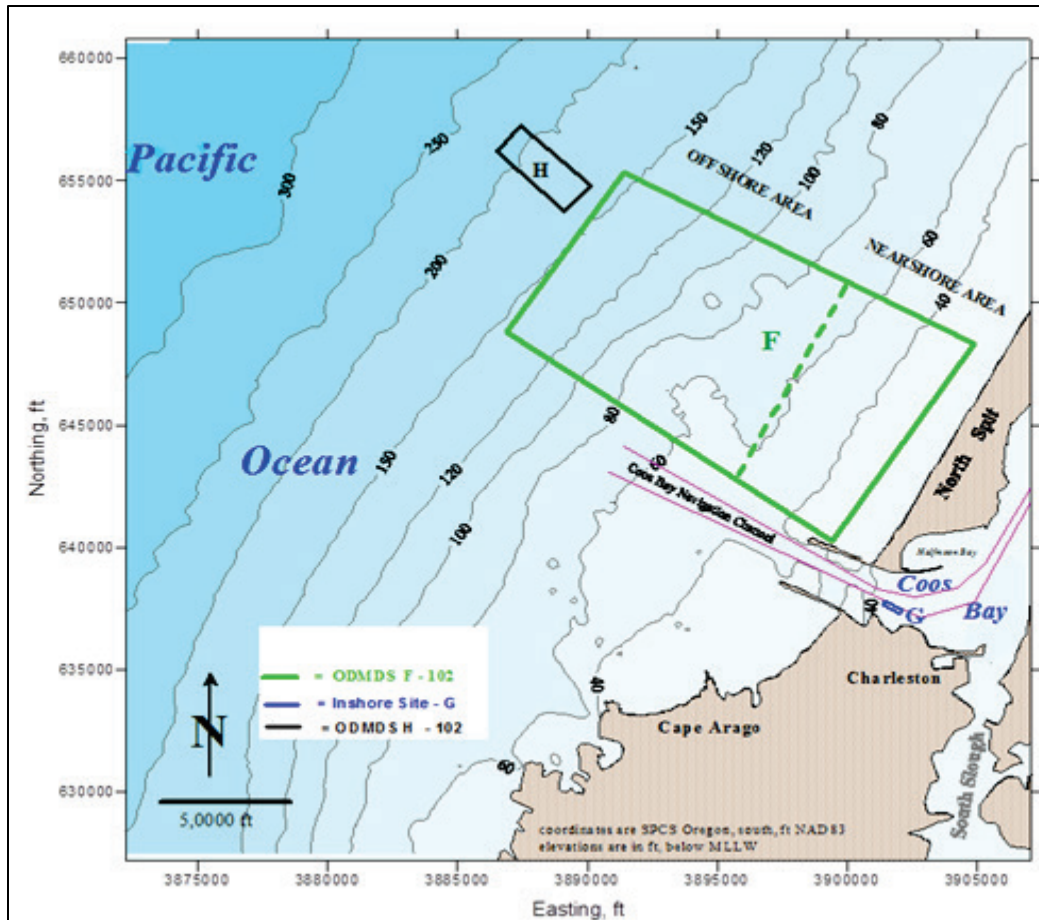
4.4.1 Purpose

A numerical modeling investigation is conducted for dredged material placed in the nearshore and offshore areas of an ocean dredged material disposal site (ODMDS) adjacent to Coos Bay Inlet, Oregon. Historical and newly collected hydrodynamic and wave data in the Coos Bay and the inlet system are assembled and analyzed. The datasets are used to calibrate and validate the CMS. In this document, the model validation is presented using time series of currents, waves, and water levels. The CMS results are also validated against instantaneous measurements on current survey lines.

4.4.2 Field study

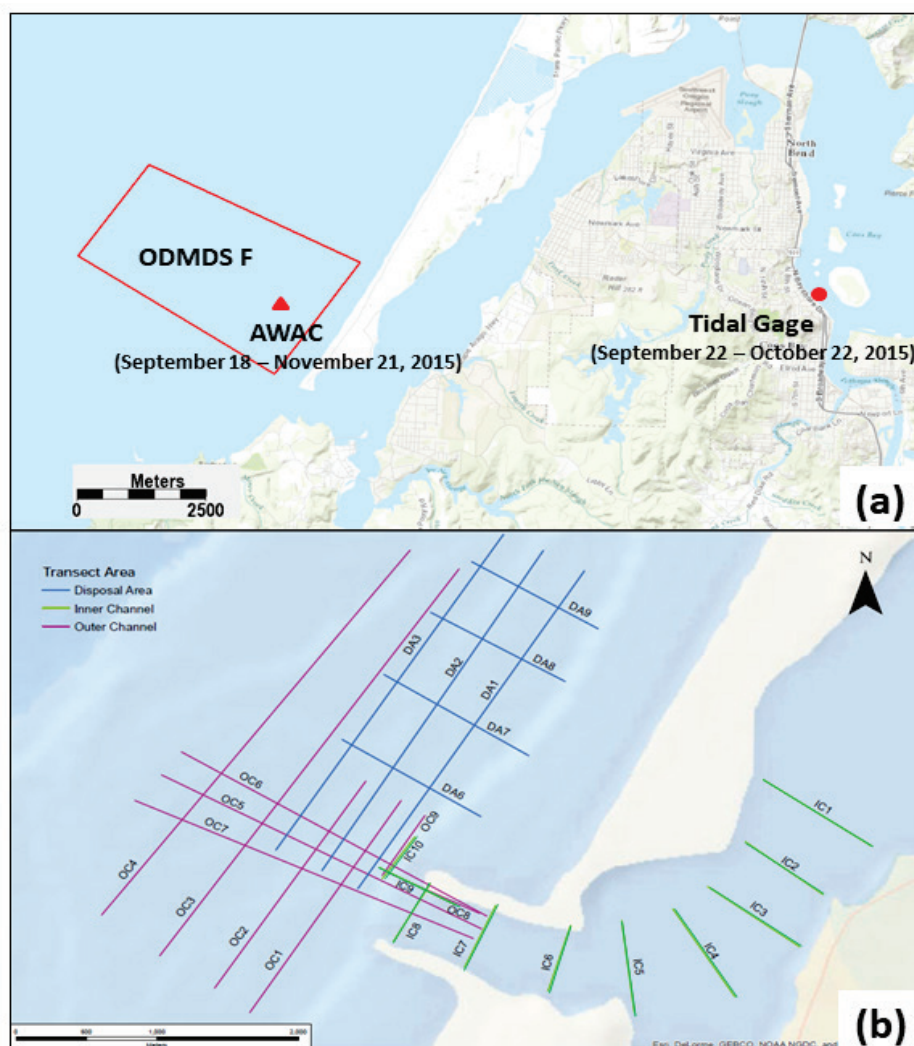
Coos Bay is located on the Oregon coast approximately 200 mi south of the Columbia River in Coos County, Oregon. The USACE maintains a navigation channel from the estuary's mouth at River Mile (RM) 0 to RM 15 and dredges over 1MCY of material from the Coos Bay project each year. This material is placed at a number of in-water disposal sites, including two EPA-designated ODMDSs called Site H and Site F (Figure 66). Up to 500,000 cy of material are placed in the nearshore portion of the site F per year. The USACE preferentially utilizes the F-NS (Site F nearshore area) for the purpose of augmenting the littoral sediment budget on the north side of the north jetty to stabilize the base of the jetty. Coastal engineering analyses are needed to supplement the Portland District knowledge base to meet a future dredging and placement schedule and to understand nearshore sediment transport in the area (Li et al. 2018).

Figure 66. Coos Bay inlet study area, the federal navigation channel (pink outline), and dredged material placement sites ODMDS F and H. The dashed green line inside Site F separates the nearshore area (F-NS) from the offshore area.



In this study, the CMS is applied to calculate sediment movement under the combined influence of waves and current. Field data collection includes time series of currents and waves using a bottom mounted acoustic wave and current profiler (AWAC) within the ODMDS from September 18 to October 30, 2015, and time series of water levels obtained by deploying a pressure sensor in the upper reaches of Coos Bay. A current survey was conducted in the Coos Bay Channel and the nearshore area using a survey vessel with a downward-looking ADCP installed over the side. The survey vessel cruised across sets of survey transect lines over a 4-day period between September 28, 2015, and October 1, 2015 (Figure 67).

Figure 67. (a) The AWAC gauge and the tide gauge in the upper Coos Bay from September to October 2015. (b) Coos Bay transect lines, September 28 – October 1, 2015.

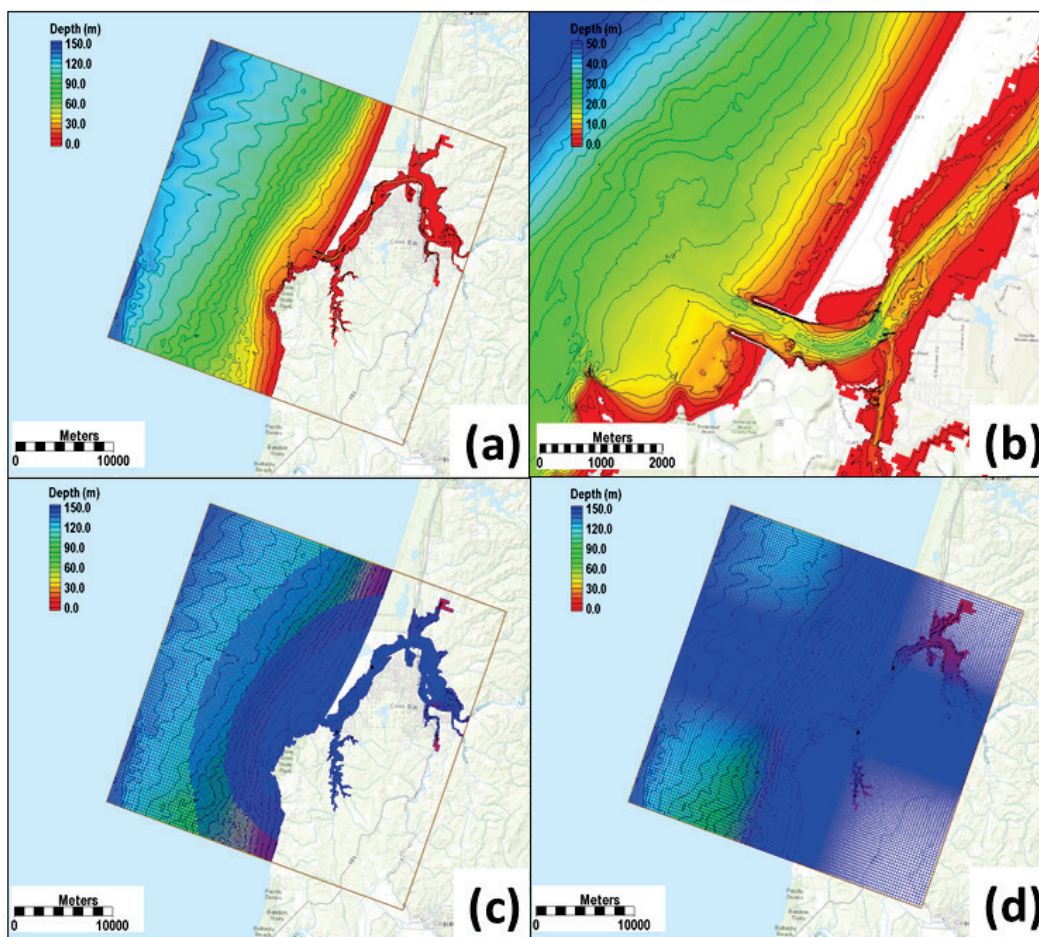


4.4.3 Model setup

The Coos Bay estuarine and inlet system is configured using a telescoping variable-resolution CMS-Flow grid. The areal extent for the modeling domain is approximately 31 km alongshore and 32 km across shore. The CMS grid consists of 132,000 ocean cells. The water depth ranges from 1 to 2 m below mean sea level at tidal marsh areas in the bay area to 16 m at the inlet navigation channel and further increases to 130 m at the offshore boundary. The telescoping grid system permits much finer local grid resolution to resolve hydrodynamic and sediment features in areas of high interest. For this study, the cell sizes vary from 20 m around the Coos Bay Inlet and navigation channel to 320 m in the open ocean. The

CMS-Wave grid with varying cell sizes is generated for wave modeling, covering the same domain and with similar spatial resolution as the CMS-Flow grid (Figure 68).

Figure 68. The Coastal Modeling System (CMS), (a) bathymetry for entire CMS grid domain, (b) bathymetry surrounding Coos Bay Inlet. (c) CMS-Flow telescoping grid, (d) CMS-Wave varying rectangular grid,



The hydrodynamic time-step is set to 10 min. Manning's n is calibrated as 0.015 in the bay and 0.021 in the open ocean. NOAA tide gauge (#9432780) at Charleston, OR, is located on the bayside and provides the tidal forcing for CMS-Flow. Wind speed and direction are obtained from the NOAA offshore buoy #46015 and specified at the air-sea interface. NDBC buoy #46015 measures directional wave spectra, which is located approximately 75 km from the Coos Bay Inlet. Through model tests, the spectrum data at this buoy are used at a 2 hr interval and transformed to the model seaward boundary to drive CMS-Wave. The 81-day simulation from 11 September to 30 November 2015, is performed

with the CMS. The model setup is summarized in Table 41. The simulation takes approximately 4 days and 7 hr on a laptop computer using four 2.90 GHz processors.

Table 41. CMS model settings for the Coos Bay Inlet test case.

Parameter	Value
Numerical Scheme	Implicit
Time-step	600 sec
Simulation duration	1944 hr
Ramp duration	6 hr
Manning's n	0.015 (bay), 0.021 (open ocean)
Steering interval	2 hr

4.4.4 Results and discussion

Figure 69 shows the time series of the measured and calculated current speeds in the longshore direction. The calculations reproduce the tidal currents combined with the low frequency, storm-driven flows fairly well. Larger discrepancies between the calculated results and measured data occurred during a few low-frequency events. The RMSE for the current comparison at the AWAC gauge is 0.08 m/sec. The NRMSE is 15.84%, and the NMAE is 12.42%. The correlation coefficient (R^2) is 0.32 (Table 42). Due to the limited spatial coverage of buoy data, utilizing the offshore wind and wave data as model input might be the cause of the model-data discrepancies at the gauge location in nearshore zones.

Figure 69. Comparison of longshore current between the measurements and the CMS calculations at the AWAC gauge from September to October 2015.

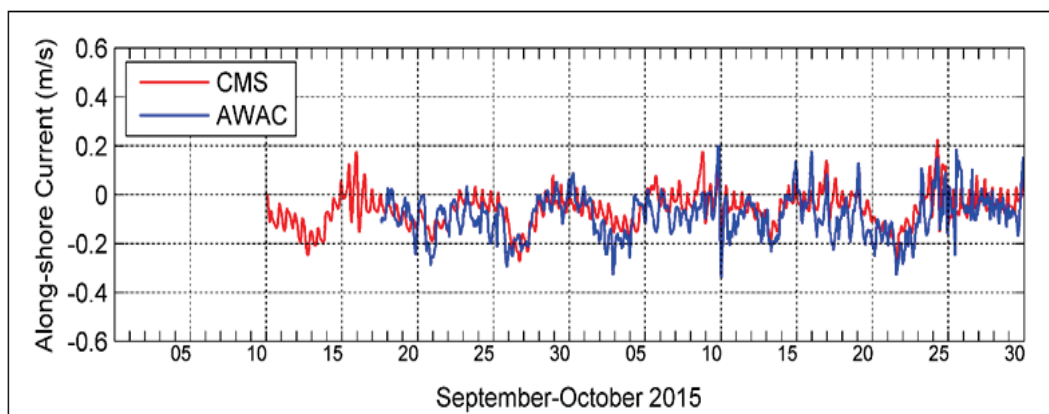


Table 42. Goodness-of-fit statistics of principal current velocity for Coos Bay Inlet, Oregon: 2015 field data.

Statistics	AWAC Gauge
NRMSE [%]	15.840
RMSE [m/sec]	0.079
NMAE [%]	12.420
R^2	0.324
Bias [m/sec]	0.030

The calculated wave parameters are compared with the measurements at the AWAC gauge (Figure 70). The mean significant wave height is 1.7 m. Wave heights exceed 3 m for five time periods during the 50-day simulation. The mean wave period is 10.3 sec, and the predominant wave direction is west-northwest. The calculated wave parameters are correlated with the measurements well. The correlation coefficients (R^2) are 0.83, 0.68, and 0.79 for wave height, wave period, and wave direction, respectively. The RMSE, the NRMSE, and the NMAE are 0.38 m, 8.3%, and 6.4% for wave height, 1.2 sec, 7.2% and 4.6% for wave period, and 6.4°, 2.0%, and 1.6% for wave direction, respectively (Table 43). The sensitivity tests on wave transformation show that the calculated wave parameters are closely associated with the specifications of boundary conditions from the measurements at different buoys.

Figure 70. Comparisons of (a) significant wave height, (b) peak wave period, and (c) mean wave direction between the calculations and the measurements at the AWAC gauge from September to October 2015.

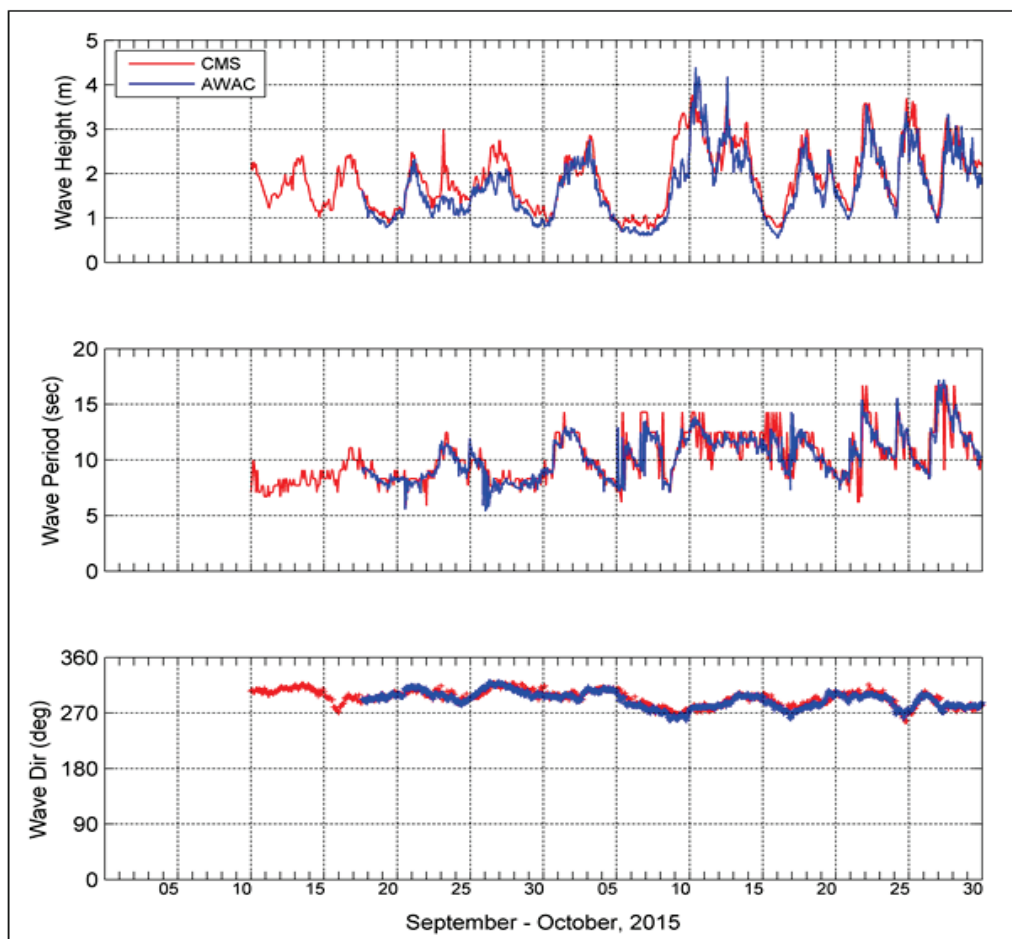


Table 43. Goodness-of-fit statistics of wave parameters for Coos Bay Inlet, Oregon: 2015 field data.

Statistics	AWAC Gauge		
	Significant Wave Height (m)	Peak Wave Period (sec)	Mean Wave Direction (deg)
NRMSE [%]	8.336	7.158	2.006
RMSE	0.375	1.217	6.418
NMAE [%]	6.380	4.586	1.588
R2	0.834	0.682	0.788
Bias	0.222	0.129	2.228

Figure 71 shows the calculated and measured WSEs at the tidal gauge inside Coos Bay (Figure 67). The measurements and calculations show good agreement. The goodness-of-fit statistics for WSEs result in a correlation coefficient (R^2) of 0.95. The RMSE, the NRMSE, and the NMAE are 0.21 m, 6.1%, and 4.9%, respectively (Table 44).

Figure 71. Comparison of WSE between the measurements and the CMS calculations at the tide gauge in the upper Coos Bay from September to October 2015.

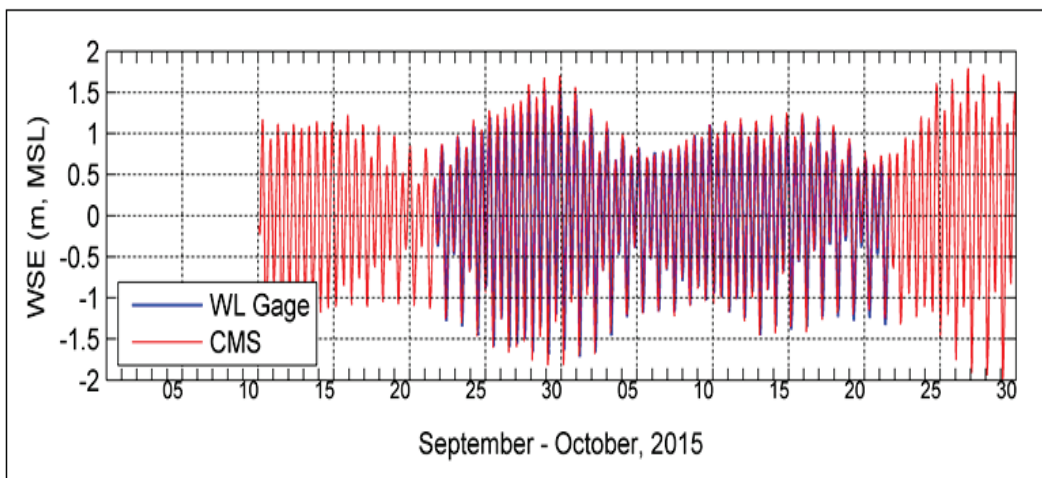


Table 44. Goodness-of-fit statistics of WSE for upper Coos Bay, Oregon: 2015 field data.

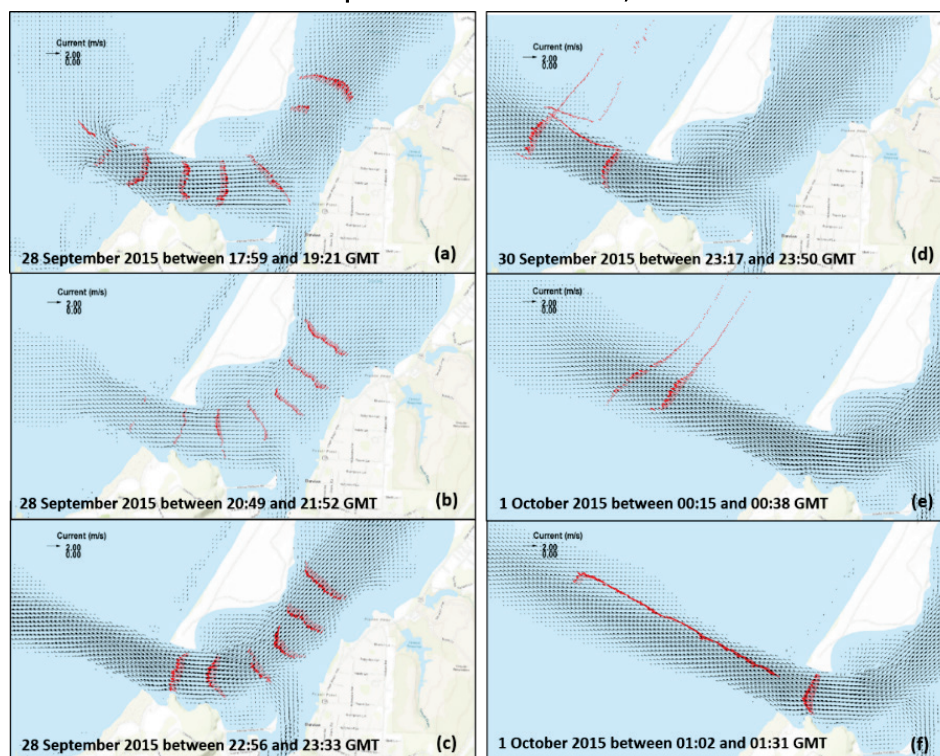
Statistics	WL Gauge
NRMSE [%]	6.068
RMSE [m]	0.206
NMAE [%]	4.938
R^2	0.946
Bias [m]	-0.016

Vessel-mounted ADCP transects provide depth-averaged currents over multiple tidal cycles in and around Coos Bay inlet from 28 September through 1 October 2015. Considering the average time for the vessel to traverse the transects, current output is retrieved at 10 min intervals from the CMS simulation to compare with the measurements on selected transects over the 4-day period.

Six snapshots of the model-data comparisons are shown in Figure 71. The calculated current vectors match the measured flood and ebb current magnitudes and directions well at each transect. Current variations along transects (i.e., strong currents in the center of the navigation channel and

weaker currents on the channel margins) are also reproduced well by the CMS (Figure 72a, Figure 72b, Figure 72c). Two outstanding current features adjacent to an inlet are shown in the figure: (1) the strong ebb jet is captured well beyond the jettied inlet channel (Figure 72f); (2) an eddy formed due to the interactions between the ebb jet and longshore current north of the entrance channel (Figure 72e). Because the measurements along the transects are not conducted simultaneously (typically several minutes per transect), surveyed currents over the longer open ocean transects (10 – 30 min per transect) tend to have more fluctuations and more discrepancies between the calculated and measured currents.

Figure 72. Current comparisons between the vessel-mounted downward-looking ADCP measurements (vectors in red color) and the CMS calculations in transects from September 28 to October 1, 2015.



Goodness-of-fit statistics are calculated for U (east-west) and V (north-south) current components based on the measurements and the model output from ten snapshots. The results are provided in Table 45. The overall comparison indicates a small NRMSE and NMAE approximately 5% and a close correlation above 0.8 between the model results and the measured, depth-averaged ADCP current data.

Table 45. Goodness-of-fit statistics between the vessel-mounted ADCP current measurements and the CMS calculations in transects from 28 September to 1 October 2015.

Statistics	Current Velocity (m/sec)	
	U Component (E-W)	V Component (N-S)
NRMSE [%]	6.393	5.129
RMSE	0.197	0.147
NMAE [%]	4.471	3.669
R ²	0.855	0.840
Bias	0.008	0.003

4.4.5 Conclusions and recommendations

Extensive CMS calibration and validation are performed at Coos Bay and Inlet, Oregon. In addition to anchored gauges measuring current, waves, and WSE, boat-mounted ADCP is deployed to measure instantaneous currents along multiple transects in the bay and inlet system. The goodness-of-fit statistics are computed to assess the model-data comparisons of water levels, depth-averaged principal current velocities, and wave parameters at two gauges, one water level gauge in the bay and the other AWAC gauge in the open littoral zone. The comparisons of instantaneous current components are also examined along transects by evaluating the goodness-of-fit statistics. The model results agree well with measurements in the bay and in open ocean area and demonstrate the model's capability in representing coastal physical processes.

4.5 Test C3-Ex4: Ogden Dunes, Indiana

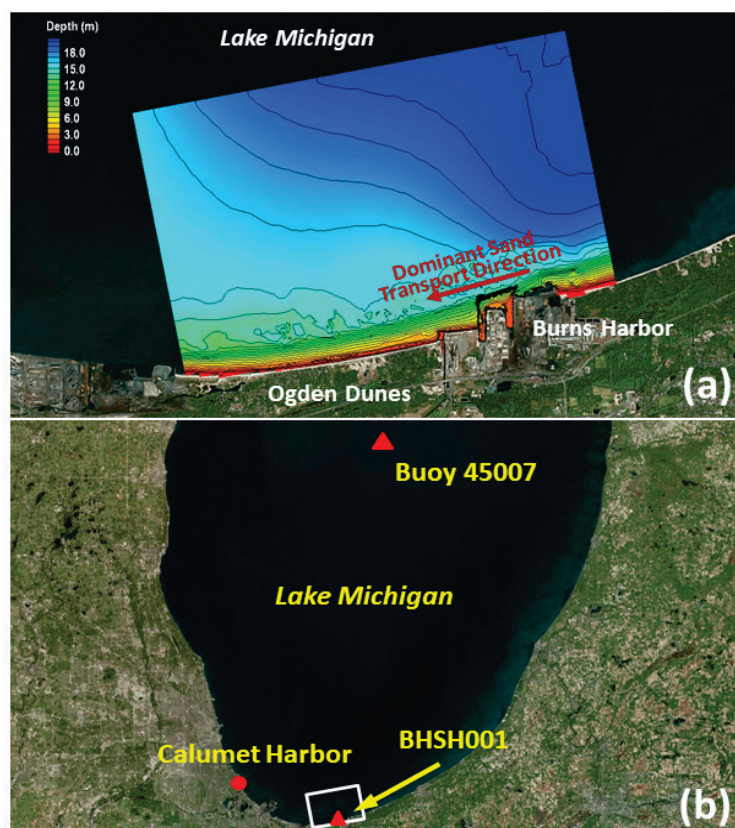
4.5.1 Purpose

To evaluate movement of nearshore placed material and protect eroding shoreline at Ogden Dunes, Indiana, hydrodynamic, wave, and sediment transport modeling simulations are conducted under various physical conditions. The process of the CMS validation is repeated using the updated CMS code to demonstrate the model's capability in calculating currents, WSE, and waves in coastal lake environment.

4.5.2 Field study

The Town of Ogden Dunes and Burns Waterway Harbor are located in southern Lake Michigan, approximately 65 km southeast of the City of Chicago. The presence of harbor structures interrupts the natural littoral movement of sand, mainly from east to west. Sand accretion at the east side of the harbor and erosion at the west side leave the Town of Ogden Dunes, IN, sand starved (Figure 73). Little to no beach situation along the shoreline of Ogden Dunes puts the residents and their properties at greater risk from coastal hazards. To protect the natural habitat at the National Lakeshore and shoreline residences within Ogden Dunes, the USACE has placed over 600,000 cy of dredged sand in the nearshore area as beach nourishment effort since 2006. However, the effectiveness of the nearshore placements for shoreline protection and littoral nourishment has not been established in all cases.

Figure 73. (a) Study area and numerical model domain. (b) Locations of Lake Michigan water level gauge 9087044, NOAA buoy 45007, and the up-looking ADCP (BSH001) site. The white rectangular designates the CMS domain.

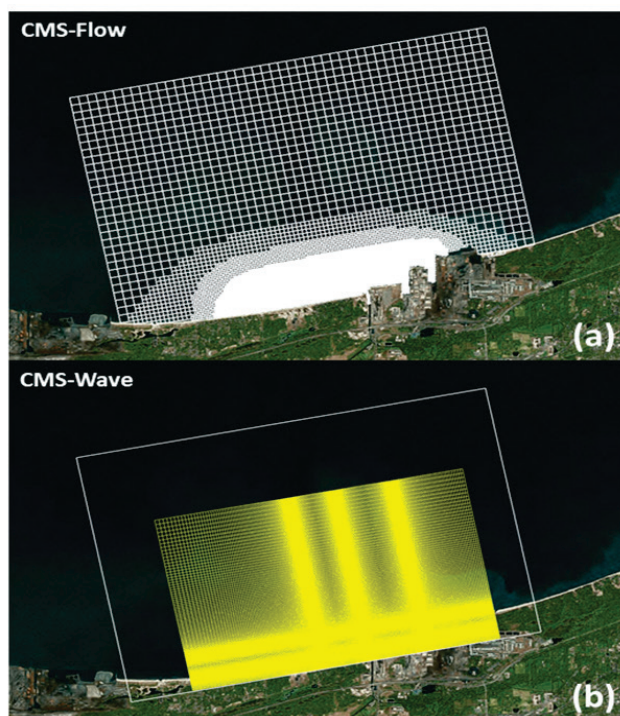


To improve the effectiveness of nearshore placement, recent USACE studies have focused on the evaluation and monitoring of ongoing placement and movement of material into the nearshore region of the beach at Ogden Dunes. This undertaking involves an extensive field program (bathymetry surveys and ADCP deployment) and a CMS modeling of hydrodynamics and sediment transport. The ADCP survey of waves, currents, and WSE was carried out from 2 June to 11 October 2016. The wave and hydrodynamic simulations for model validation are conducted for a period from 20 July to 31 August 2016. Time series values of WSEs, currents, and waves for the July-August simulation are compared against the ADCP measurements at BSH001 (Figure 73) (Li 2021).

4.5.3 Model setup

The CMS-Flow domain is discretized by a telescoping mesh, which is 15.4 km in length along the shore and 10.6 km in width across the shore. The telescoping mesh has approximately 80,000 lake cells. The fine resolution cells with 10 m spacing are specified around the beach front and the nearshore area of Ogden Dunes and the coarse resolution with 320 m spacing in the offshore area. The average water depth is 2 m nearshore and increases to 20 m in the CMS offshore boundary (Figure 74).

Figure 74. (a) CMS-Flow telescoping mesh and (b) CMS-Wave non-uniform Cartesian grid.



CMS-Wave domain is within the CMS-Flow domain with a horizontal scale of 11.5×7.4 km (Figure 74). Non-uniform rectangular grid is used for the wave model. Similar to the discretization of the flow model, high resolution grids of 10.0×10.0 m are specified nearshore, and grid cell sizes increase to 180 m away from the lake shoreline.

The hydrodynamic time-step is set to 10 min. Manning's n was calibrated as 0.022 in the open lake area. CMS-Flow was driven with time-dependent water levels, winds, and waves. Water level data are obtained from NOAA Lake Michigan station (#9087044) at Calumet Harbor, Illinois. Wind and wave data are obtained from the NDBC Buoy #45007, located approximately 120 km north of Ogden Dunes (Figure 73). The directional wave data are transformed to the lakeward boundary of the CMS-Wave grid. CMS-Wave is driven using the spectrum data at a 2 hr interval. The 42-day simulation from 20 July to 30 August 2016 is performed with the CMS. The model setup is summarized in Table 46. The simulation took approximately 22 hr on a laptop computer using four 2.90 GHz processors.

Table 46. CMS model settings for the Ogden Dunes test case.

Parameter	Value
Numerical Scheme	Implicit
Time-step	600 sec
Simulation duration	1008 hr
Ramp duration	2 hr
Manning's n	0.022
Steering interval	2 hr

4.5.4 Results and discussion

The model performance in the lake environment is assessed by comparing time series values of WSEs, currents, and wave parameters against the ADCP measurements at BSH001.

Figure 75 shows the lake water levels during the simulation period. Both measured and calculated results reveal small, high-frequency fluctuations with the maximum water level approximately 0.25 m and the minimum approximately -0.15 m. The standard deviation of the data and the model results are 0.054 and 0.058 m, respectively. As shown in the study by Li (2021), most of the large water level changes correspond to large wind/wave conditions. Goodness-of-fit statistics shows a small RMSE but a relatively large NRMSE due to the small range of variation in data

(Table 47). Although the correlation between the calculation and the measurements is very low, Figure 74 displays that the calculated water level curve reproduces most of elevated lake levels well due to weather events. Therefore, the unfavorable statistical parameters should be associated with those high-frequency oscillations in the lake.

Figure 75. WSE comparisons between the measurements and the CMS calculations from 20 July to 30 August 2016.

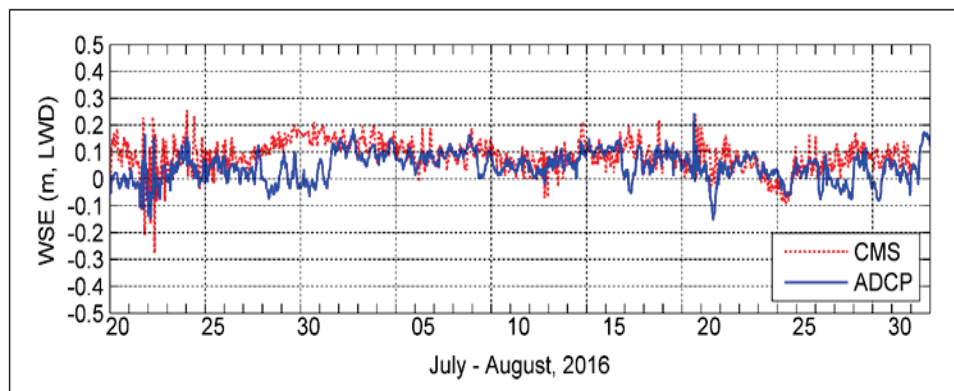
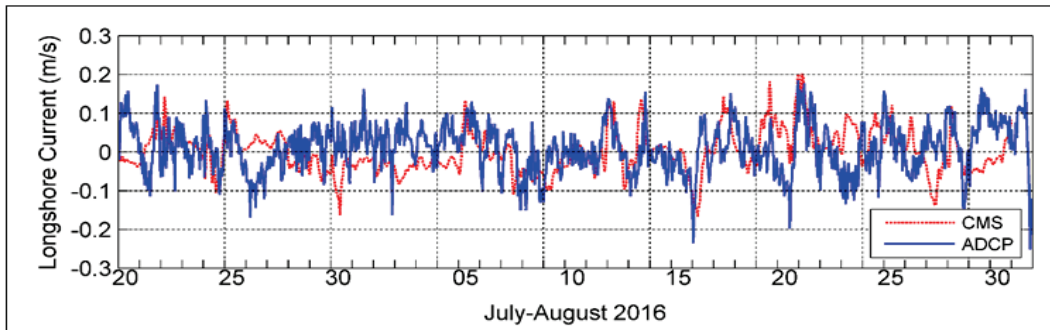


Table 47. Goodness-of-fit statistics of WSE, principal current velocity, and significant wave height for Ogden Dunes, IN: 2016 field data.

Statistics	ADCP		
	Water Surface Elevation (m)	Current (m/sec)	Significant Wave Height (m)
NRMSE [%]	20.350	18.739	27.196
RMSE	0.081	0.079	0.125
NMAE [%]	15.250	15.321	17.500
R ²	0.050	0.014	0.449
Bias	0.041	-0.008	0.046

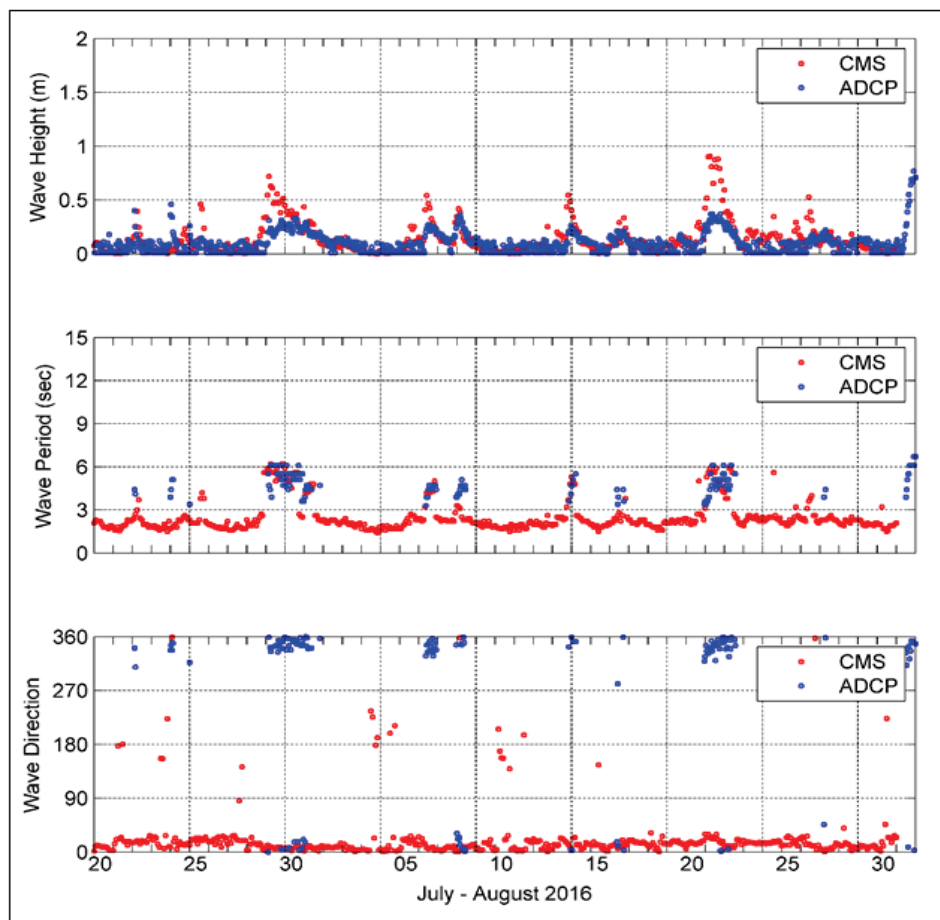
The time series current comparison in the principal direction between the CMS results and the measurements is shown in Figure 76. Like small variations in water levels, the calculated and the measured current speed only varies within ± 0.2 m/sec (east-northeastward and west-southwestward current) in the dominant longshore direction during the simulation period. The measured peak flows are mostly displayed in the calculated results but the model-data discrepancies in comparison do not show any difference in goodness-of-fit statistics due to small amplitudes and high-frequency variations in the lake environment (Table 41).

Figure 76. Longshore current comparisons between the measurements and the CMS calculations at the ADCP station from 20 July to 30 August 2016.



Incident waves that impact the study area are primarily propagating from north and are influenced by wind fields. Figure 77 shows the comparisons of wave parameters between the calculations and the measurements at BHSH001 for the summer period. From the comparison it can be seen that the model overpredicts significant wave heights. The measurements give an averaged wave height of 0.1 m, and the model calculates an average of 0.14 m. The ADCP measures small, long waves propagated from north with peak wave periods of greater than 5 sec, which is reproduced well by the calculated results. Weak wave signals with a significant wave height less than 0.1 m could not be detected well by the instrument, and the corresponding data are not shown in Figure 77. Goodness-of-fit statistics for significant wave heights shown in Table 41 indicate a relatively large RMSE and NRMSE but a much better correlation between the model and the data comparing with the statistical values for water level and current.

Figure 77. Wave parameter comparisons between the measurements and the CMS calculations at the ADCP Station from 20 July to 30 August 2016.



4.5.5 Conclusions and recommendations

With minor and negligible astronomical tide in Lake Michigan, fluctuations and changes in WSE and current are primarily caused by meteorological events. Those fluctuations are small under regular conditions but become significant under storm conditions.

Driven by water level, offshore waves, and wind, the results of the CMS application demonstrate that the changes in WSEs and longshore currents at the nearshore location respond well to weather events. The evaluation of the goodness-of-fit statistics shows relatively large discrepancies between the calculations and the ADCP measurements, which implicates both model's capability and instrument's sensitivity in simulating and measuring small fluctuations in coastal lake environment.

4.6 Test C3-Ex5: Test Bed, Duck, North Carolina

4.6.1 Purpose

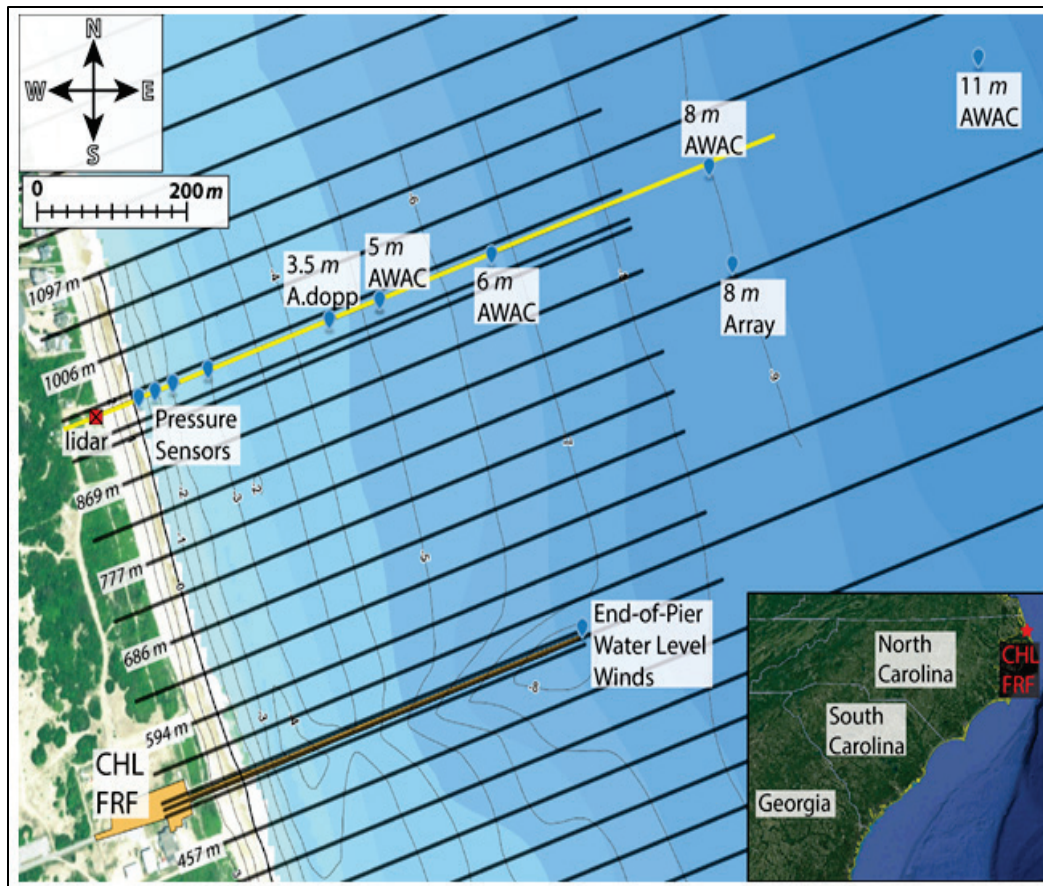
The Coastal Model Test Bed (CMTB) is an initiative to evaluate the performance of coastal numerical models in simulating nearshore physical processes (Bak et al. 2017). The CMS, a coastal hydrodynamic, wave, and sediment transport model, is selected and set up in the CMTB. The model validation is conducted by comparing the model output with high-resolution measurements by deploying an array of instruments at the CMTB.

4.6.2 Field study

Located at the Coastal and Hydraulics Laboratory (CHL), Field Research Facility (FRF), in Duck, NC, the CMTB is designed to provide a near-real-time open ocean environment for assessing the performance of coastal numerical models and understanding nearshore physical processes. In real-world applications, field data are very often used to validate numerical models. However, spatial or temporal coverage of a typical survey program can hardly present a complete picture of field conditions. Over a few decades, extensive and sustained nearshore data have been collected at the CHL FRF, which results in substantial advancements in the science of nearshore processes (Moulton et al. 2014, 2017).

The data collected at the CHL include wave parameters, water level, current, bathymetry, wind, barometric pressure spanning the surf zone to the 26 m bathymetric contour (Figure 78). For model validation in the CMTB, WSE and current are calculated from 20 to 27 October 2015, and the results are compared with the measurements at two onshore gauges, 3.5 m Aquadopp (the Aquadopp Profiler is a highly versatile ADCP) and 6.0 m AWAC.

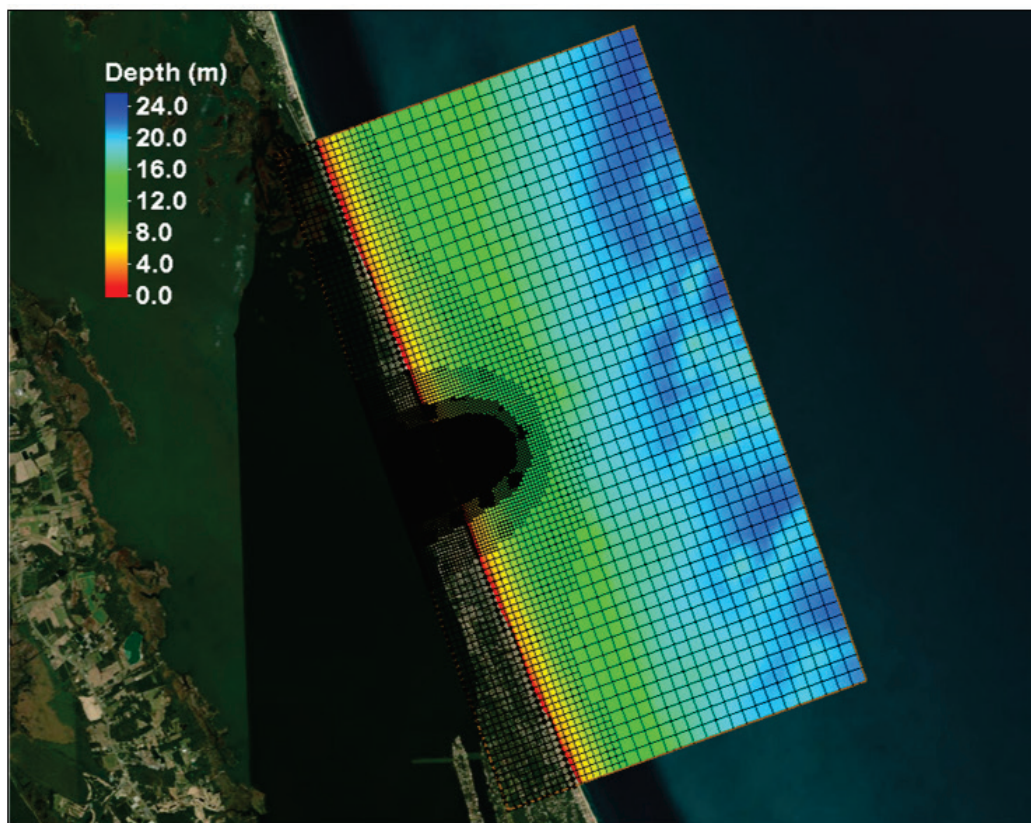
Figure 78. Map of CHL FRF and gauge locations.



4.6.3 Model setup

The bathymetry for the CMS model is obtained from the integrated bathymetry product, an archive of digital elevation models with the update of the most recent bathymetric measurements (Young et al. 2018). The CMS telescoping grid is configured with 72,600 ocean cells in the CMTB (Figure 79). Variable cell sizes are designed with large cell of 320×320 m near the open boundary and small cell of 5×5 m in the vicinity of the CHL FRF pier.

Figure 79. CMS-Flow telescoping grid at FRF CMTB.



For the CMTB setup, CMS-Flow is run with the hydrodynamic time-step of 10 min. Manning's n is calibrated as 0.011 for the entire domain. The tidal and wind data are downloaded from the NOAA gauge (#8651370) at the end-of-pier as the CMS driving force. The 8-day simulation is set up, and the general framework of the CMTB CMS model run is outlined in Table 48. The simulation takes approximately 2.5 hr to complete on a laptop computer using one 2.90 GHz processor.

Table 48. CMS model settings for the CMTB, Duck, NC.

Parameter	Value
Numerical scheme	Implicit
Time-step	600 sec
Simulation duration	192 hr
Ramp duration	2 hr
Manning's n	0.011

4.6.4 Results and discussion

Figure 80 and Table 49 show the comparisons of the water level measured at the 3.5 m Aquadopp and 6.0 m AWAC with the calculated CMS water level at the same locations. During this evaluation period, the calculated water level reproduces the measured water level well. The correlation coefficient (R^2) is greater than 0.97 and 0.99, and the NRMSE is less than 7.9% and 4.7% at the 3.5 m Aquadopp and 6.0 m AWAC, respectively.

Figure 80. Comparisons of the water level measured at the 3.5 m Aquadopp and 6.0 m AWAC to the CMS-Flow calculated water level.

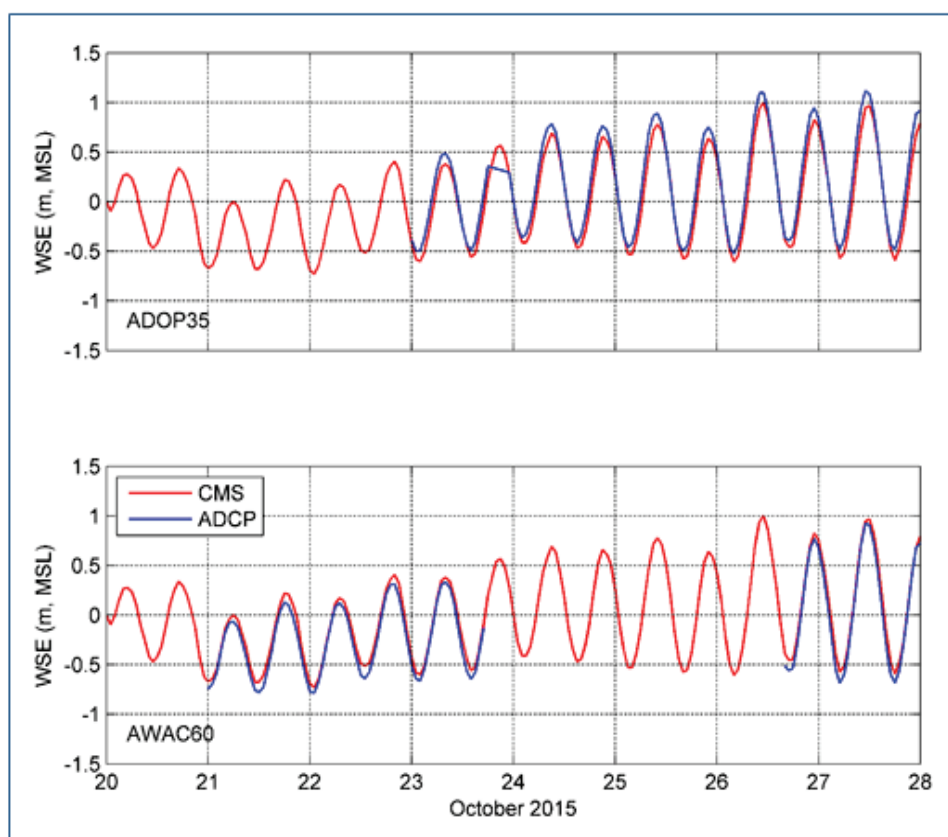


Table 49. Goodness-of-fit statistics of WSE for the CMTB, Duck, NC: 2015 data.

Statistics	ADopp 3.5	AWAC 6.0
NRMSE [%]	7.894	4.663
RMSE [m]	0.126	0.082
NMAE [%]	6.569	4.074
R^2	0.968	0.992
Bias [m]	-0.093	0.071

Figure 81 and Table 50 show the comparison of cross-shore and longshore currents measured at the 3.5 m Aquadopp and 6.0 m AWAC with the calculations at the same location. The cross-shore current components have a much smaller magnitude compared to the longshore currents and are close to zero at the 6.0 m AWAC. The analysis here focuses on the longshore currents. As shown in Figure 81 and Figure 82, there are two occasions of changes in longshore current direction during the 8-day simulation period. The CMS calculated longshore currents predict the changes in dominant longshore current direction well and are correlated with those measured at the two locations. The correlation coefficient (R^2) is close to 0.6 for the longshore current and the NRMSE approximately 20%. However, during the simulation period, CMS-Flow predicts close to zero cross-shore currents relative to those measured at the 3.5 m Aquadopp, which could be attributed to lack of nearshore processes due to wave action.

Figure 81. Model cross-shore and longshore currents versus data.

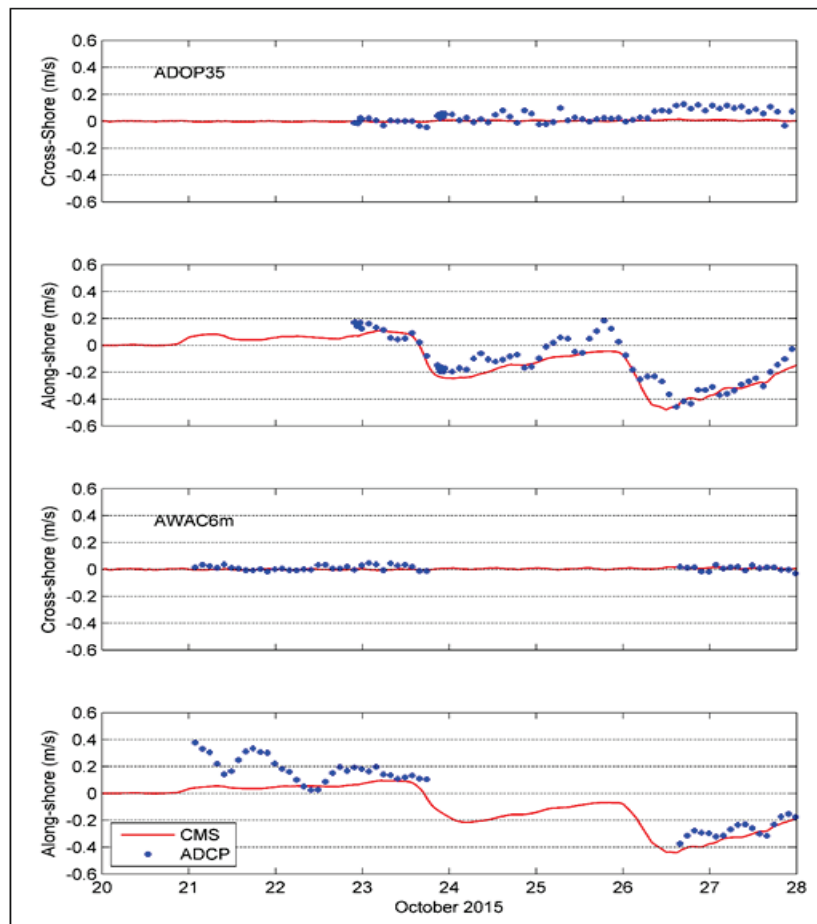


Figure 82. Wind speed and direction measured at FRF pier from 20 to 27 October 2015.

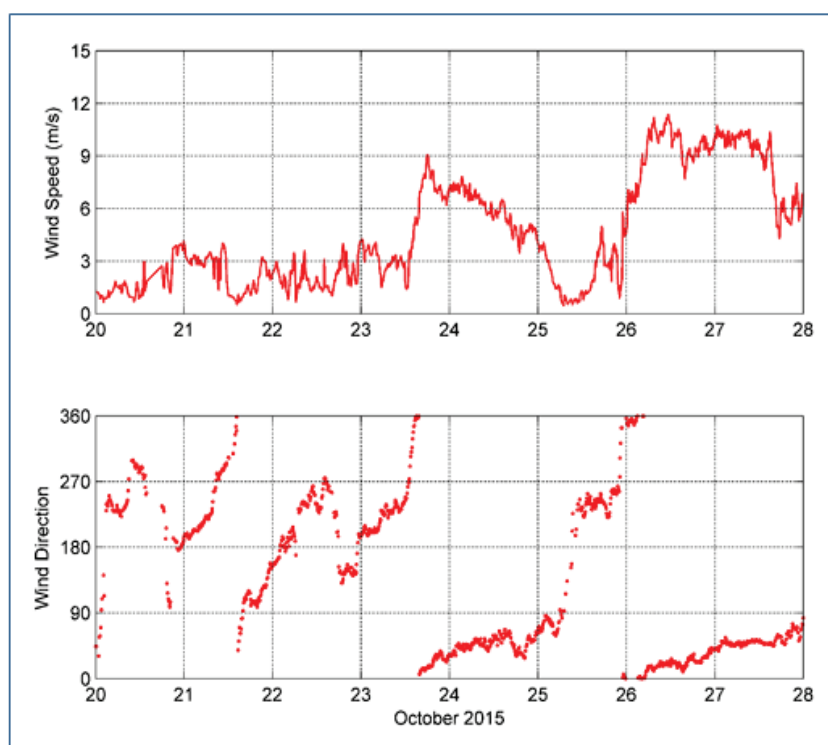


Table 50. Goodness-of-fit statistics of longshore current velocity for CMTB, Duck, NC: 2015 data.

Statistics	ADopp 3.5	AWAC 6.0
NRMSE [%]	20.062	20.963
RMSE [m]	0.130	0.168
NMAE [%]	16.800	18.663
R^2	0.577	0.572
Bias [m]	0.085	-0.035

4.6.5 Conclusions and recommendations

CMS-Flow has been incorporated into the FRF CMTB site to generate forecast/hindcast model products. An 8-day simulation is conducted to demonstrate the model performance against the water level and current measurements in the nearshore area. The model evaluation by the goodness-of-fit statistics illustrates good correlation and relatively small errors between the calculated and the measured WSE and dominant longshore current. At this open coastal site, the longshore current also responds to coastal wind forcing well, demonstrating that the CMS could reproduce wind-driven current well in shallow water areas.

4.7 Test C3-Ex6: Waukegan Harbor, Illinois

4.7.1 Purpose

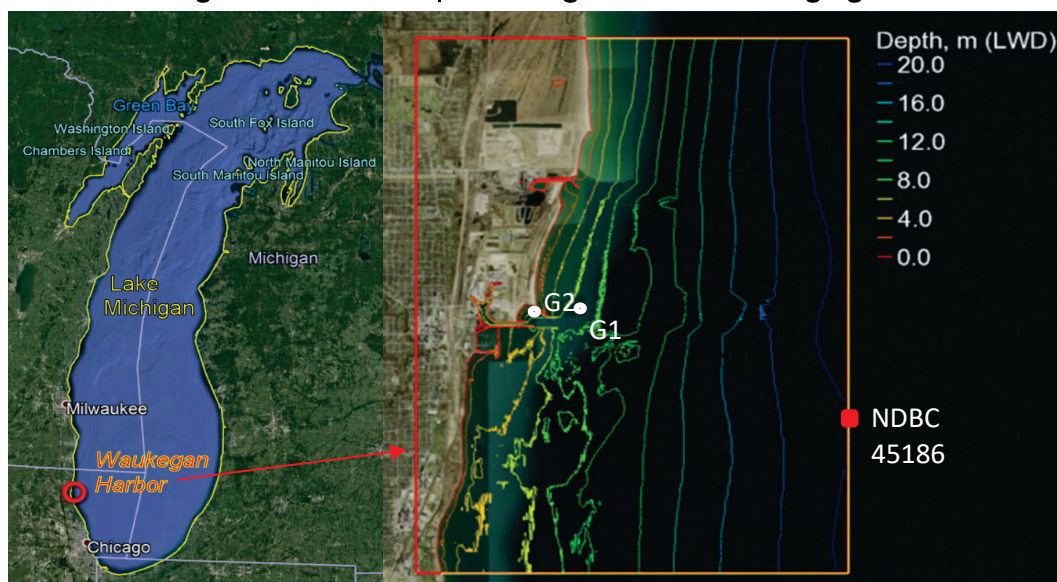
The purpose is to conduct the numerical modeling to evaluate the existing harbor and a variety of structural solutions to reduce sediment shoaling in the navigation channel (Demirbilek et al. 2019) of Waukegan Harbor, located in the lower west coastline of Lake Michigan. The CMS is used in the study, and field data are collected for model calibration, and V&V.

4.7.2 Field data collection

The Illinois State Geological Survey (ISGS) at the University of Illinois at Urbana-Champaign conducted nearshore bathymetry surveys outside the harbor in 2017 and 2018. NOAA provides lidar data for the study area in 2007, 2008, and 2012. Nearby coastal long-term water level data (<https://tidesandcurrents.noaa.gov>) are available from NOAA Station 9087057 at Milwaukee, WI (43° 0.1' N, 87° 53.2' W). Local long-term wind data (<https://www.ndbc.noaa.gov>) are available from NOAA Station WHRI2 at the South Jetty, Waukegan Harbor (42° 21.63' N, 87° 48.8' W). NDBC has collected long-term wave data at Buoy 45186 (42° 22.1' N, 87° 47.68' W) offshore Waukegan Harbor.

The ISGS installed two nearshore wave gauges, G1 and G2, for short-term data collection at Waukegan Harbor for various durations from November 2017 to October 2018. Gauge G1 (42° 21.81' N, 87° 48.28' W) is located outside the harbor, approximately 500 m east of the breakwater while G2 (42° 21.76' N, 87° 48.86' W) is located in the embayment by the Recreational Beach between the breakwater and North Jetty. Figure 83 shows the locations of the ISGS gauges.

Figure 83. Location map of Waukegan Harbor and ISGS gauges.



4.7.3 Model setup

The CMS modeling covers a rectangular area of 6.8 km (easting) $\times$ 9.1 km (northing) with higher cell resolution of 5 m around the harbor channel and lower resolution of 150 m away from the harbor (Figure 83). The wave diffraction intensity is set to 4 (default) for the maximum diffraction allowed in CMS-Wave. A constant Darcy-Weisbach coefficient of 0.005 and a constant reflection coefficient of 0.3 (default) are applied for bottom friction and forward reflection calculations, respectively. Incident wave spectra are specified with 40 frequency bins (covering the range of 0.06 to 0.45 Hz at 0.01 Hz increment) and 35 direction bins (covering a half-plane for incoming wave direction in the range of 0 to 180° azimuth with 5° increment). Wave runup on breakwaters and infra-gravity wave options are activated in simulations.

4.7.4 Results and discussion

The V&V of wave modeling are conducted for August to October 2018. Model forcing includes offshore incident waves from NOAA Buoy 45186 (offshore Waukegan Harbor), local water levels from NOAA Station 9087057 (Milwaukee, WI), and surface wind data from WHRI2 (Waukegan Harbor). Model wave results, including wave heights and periods, are compared with data collected from ISGS Gauges G1 and G2.

Figure 84 and Figure 85 show model significant wave heights and spectral peak periods versus data at Gauges G1 and G2, respectively, for August 2018. For wave data collected at G1 and G2, apparent strong surface atmospheric (barometric) pressure disturbance has caused high noise in wave measurements (Demirbilek et al. 2020). Accordingly, more than 60% in G1 and 30% of wave data G2 were affected and therefore removed for quality control and comparison with model waves. The local wind data from NOAA Station WHRI2 used for wind forcing may also affect model results, as the data may not represent the over-water winds well for modeling. Figure 86 and Figure 87 show model wave heights and periods versus data at Gauges G1 and G2, respectively, for September 2018. Figure 88 and Figure 89 show model wave heights and periods versus data at Gauges G1 and G2, respectively, for October 2018.

Figure 84. Comparison of model wave heights and periods versus data at G1 for August 2018.

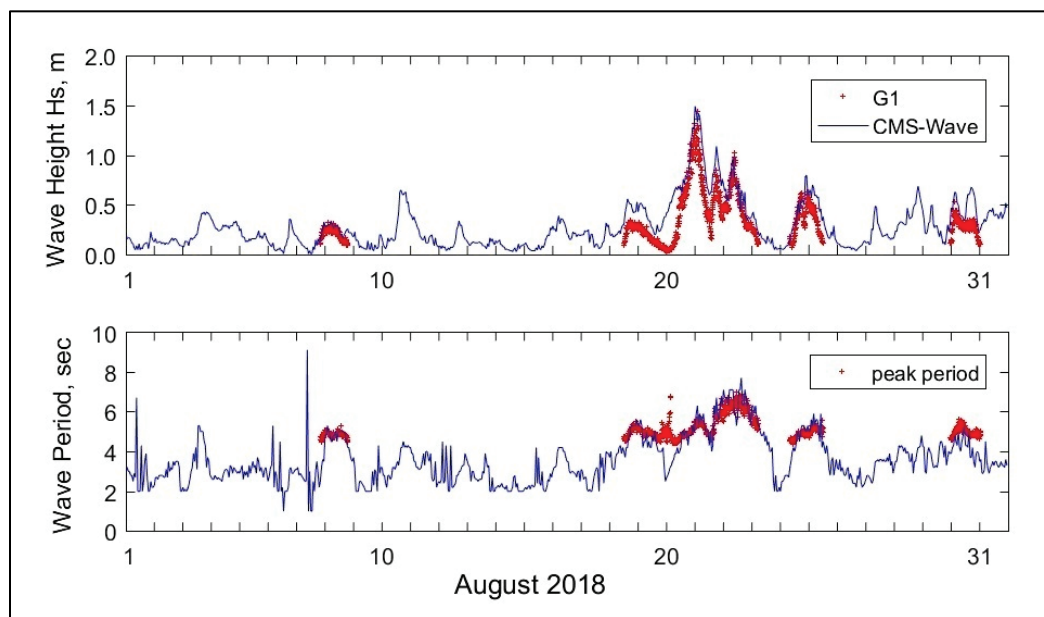


Figure 85. Comparison of model wave heights and periods versus data at G2 for August 2018.

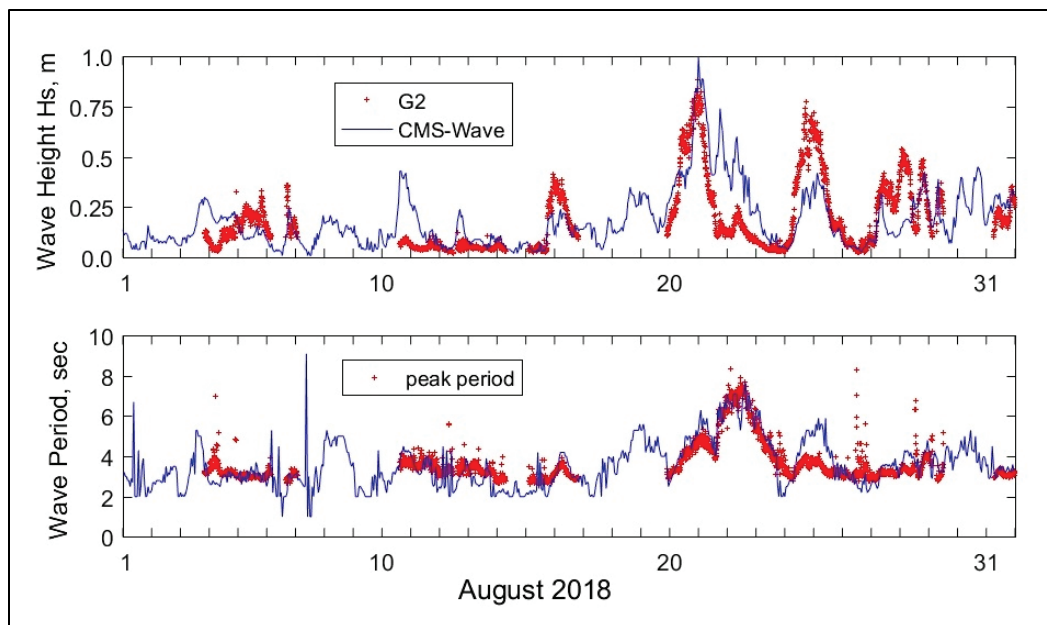


Figure 86. Comparison of model wave heights and periods versus data at G1 for September 2018.

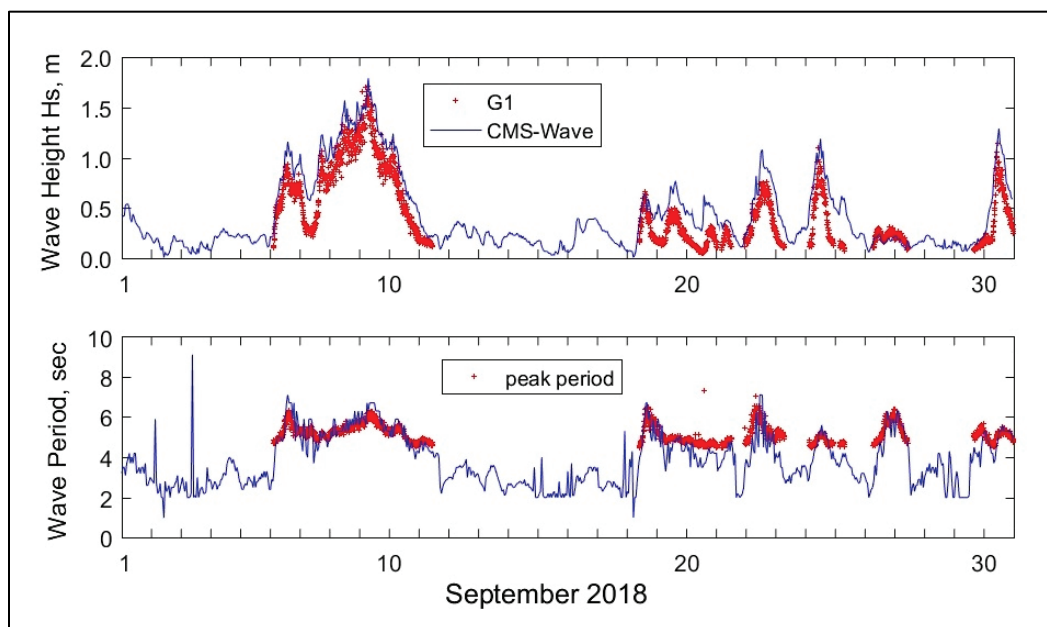


Figure 87. Comparison of model wave heights and periods versus data at G2 for September 2018.

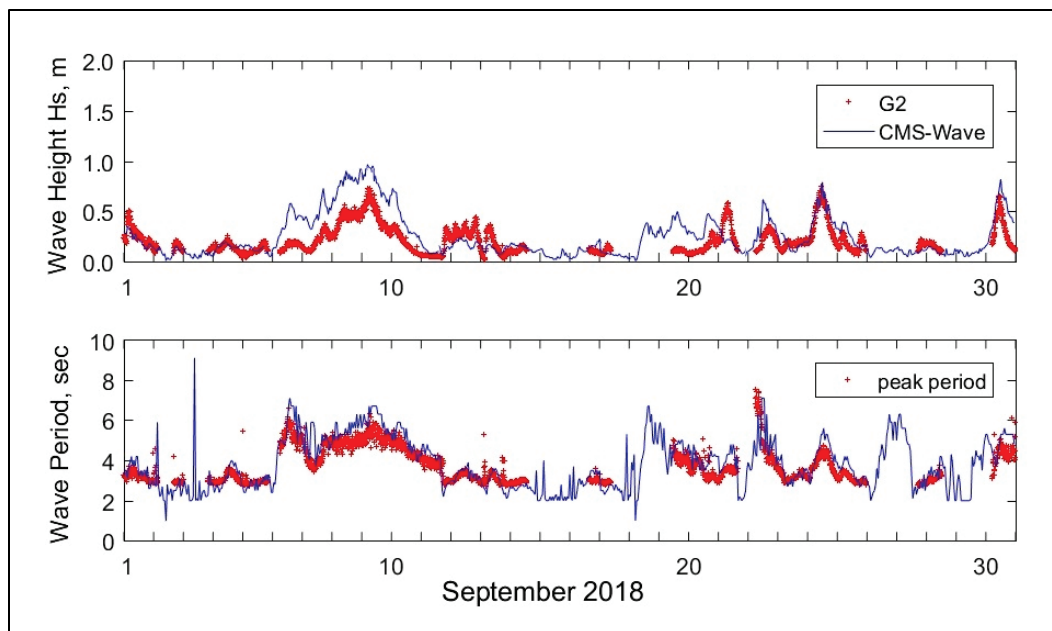


Figure 88. Comparison of model wave heights and periods versus data at G1 for October 2018.

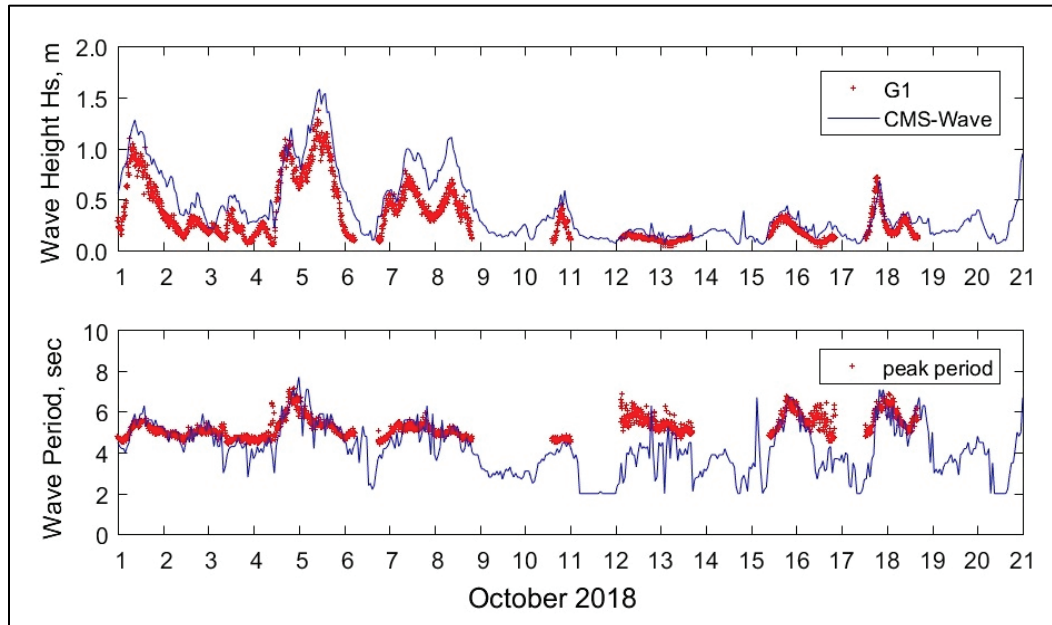
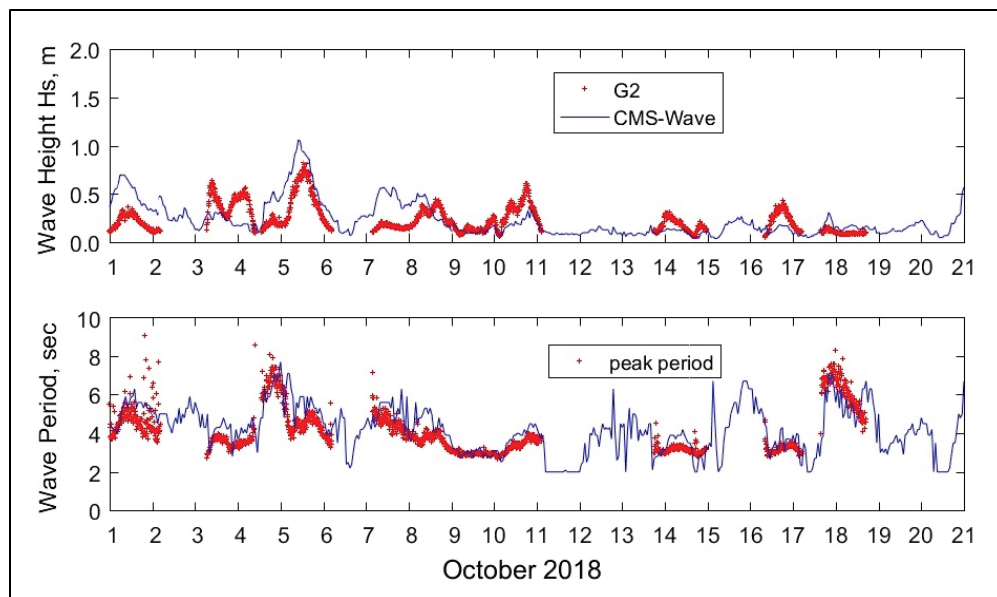


Figure 89. Comparison of model wave heights and periods versus data at G2 for October 2018.



Tables 51 and 52 list the model wave height accuracy statistics at Gauges G1 and G2, respectively, for August to October 2018. Tables 53 and 54 list the model wave period accuracy statistics at G1 and G2, respectively, for August to October 2018.

Table 51. Model wave height accuracy statistics at G1 for August to October 2018.

Month (2018)	RMSE (m)	NRMSE (%)*	R ²
August	0.121	8.07	0.69
September	0.145	8.19	0.59
October	0.156	10.1	0.50

*Normalized by model maximum wave height in a month.

Table 52. Model wave height accuracy statistics at G2 for August to October 2018.

Month (2018)	RMSE (m)	NRMSE (%)*	R ²
August	0.163	16.3	0.55
September	0.185	18.5	0.55
October	0.252	23.6	0.42

*Normalized by model maximum wave height in a month.

Table 53. Model wave period accuracy statistics at G1 for August to October 2018.

Month (2018)	RMSE (sec)	NRMSE (%)*	R ²
August	0.47	8.10	0.74
September	0.36	6.55	0.77
October	0.48	8.28	0.45

*Normalized by model mean wave period in a month.

Table 54. Model wave period accuracy statistics at G2 for August to October 2018.

Month (2018)	RMSE (sec)	NRMSE (%)*	R ²
August	0.78	14.7	0.79
September	0.31	7.05	0.85
October	0.28	4.83	0.59

*Normalized by model mean wave period in a month.

4.7.5 Conclusions and recommendations

CMS-Wave is tested and compared with field wave measurements from Waukegan Harbor, located in the lower west coastline of Lake Michigan. Directional wave data are collected from wave gauges located inside and outside the harbor breakwater which shelters the entrance channel from predominant waves from the northeast. Both model wave height and period agree well with data inside and outside the breakwater. Model results show that CMS-Wave is reliable to simulate wave conditions for the coastal harbor application.

4.8 Test C3-Ex7: Ashtabula Harbor, Ohio

4.8.1 Purpose

The purpose is to evaluate the stability of dredged material used to construct a coastal wetland in the lee side of East Breakwater of Ashtabula Harbor, located at the southeast central coastline of Lake Erie. The CMS is used in the study, and field data are collected for model calibration and V&V.

4.8.2 Field data

The bathymetry and topography for the modeling study are extracted from 2006 and 2011 lidar data (<https://www.coast.noaa.gov/dataviewer/#/lidar/search/>). The harbor channels and breakwater elevations are primarily based on 2019 and 2020 surveys conducted by the USACE Buffalo District. Long-term water level data near Ashtabula Harbor are available from two NOAA coastal stations at Erie Harbor, Pennsylvania (Station 9063038 or EREP1; 42° 9.2' N, 80° 5.6' W) and Fairport Harbor, Ohio (Station 9063053; 41° 45' N, 81° 16' W). Field wave data are available from coastal buoys at Erie Harbor (Buoy 45167; 42° 11.08' N, 80° 0.08' W) and Lakewood, OH (Buoy 45169; 41° 36.88' N, 81° 49.27' W). Local wind data are also available at NOAA station at Erie Harbor.

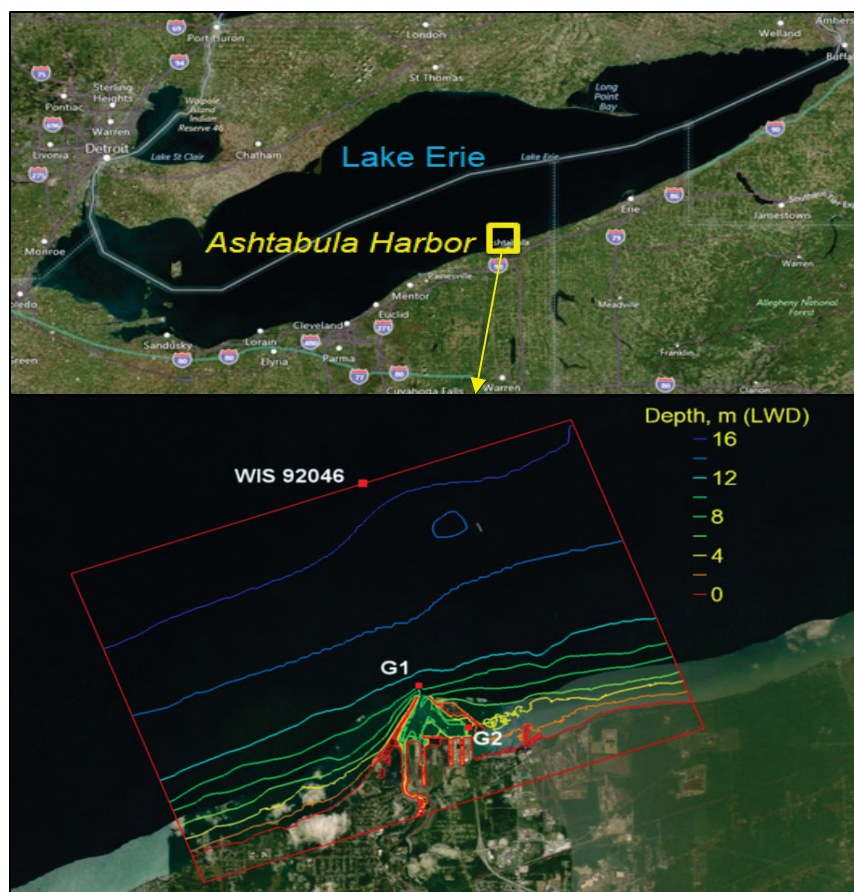
Wind wave hindcasting information at Lake Erie is available from two sources: (1) USACE Wave Information Studies (WIS) (<http://wis.usace.army.mil/>) and the Great Lakes Coastal Forecasting System (GLCFS) (<https://www.glerl.noaa.gov/res/glcfs/>). For Lake Erie, the WIS wind wave database covers the years from 1960 to 2014 while GLCFS provides 10 yr data (2011 – 2020).

ERDC deploys two AWAC gauges at the harbor entrance and interior for collecting short-term wave data. Figure 89 shows the location of two AWAC gauges; G1 (41° 55.24' N, 80° 47.67' W) is outside the harbor near the harbor entrance, and G2 (41° 54.83' N, 80° 47.01' W) is in the interior of the harbor just north of the northeast navigation channel. The AWAC meters collected wave data during two installation periods of 23 May – 7 August 2019 and 8 August – 17 November 2019.

4.8.3 Model setup

The wave grid covers a rectangular area of 7.5 km (shore normal) × 10 km (shore parallel) and has a finer resolution (5 m grid cells) around the entrance and a coarser resolution (200 m grid cells) away from the harbor complex. The offshore boundary of model grid aligns with the WIS Station 92046 (Figure 90).

Figure 90. Location of Ashtabula Harbor and nearshore wave gauges G1 and G2.



The wave diffraction intensity is set to 4 (default) for the maximum diffraction allowed in CMS-Wave. A constant Darcy-Weisbach coefficient of 0.005 and a constant reflection coefficient of 0.3 (default) are applied for bottom friction and forward reflection calculations, respectively. Incident wave spectra are specified with 40 frequency bins (covering the range of 0.06 to 0.45 Hz at 0.01 Hz increment) and 35 direction bins (covering a half-plane for incoming wave direction in the range of 0° to 180° azimuth with 5° increment). Wave runup on breakwaters and infra-gravity wave options are activated in simulations.

4.8.4 Results and discussion

The V&V of wave modeling are conducted to compare nearshore wave data collected at G1 and G2. The offshore water level boundary condition used the data from NOAA Station 9063038 at Erie Harbor, Pennsylvania. Incident waves and wind inputs to the model are based on NOAA Buoy 45169 at Lakewood, OH. Figure 91 and Figure 92 show the comparison of

model wave heights and periods to field data collected from 23 May to 30 June 2019 at AWAC Gauges G1 and G2, respectively. Note that wave period data from G1 and G2 were contaminated by noise as both gauges were partially buried in the silty bed after the installation. As a result, the model wave and data comparison are more meaningful for wave heights than wave periods. Table 55 presents the corresponding statistics for model wave height and data comparison at G1 and G2.

Figure 91. Model wave height and period versus data at G1 for 23 May to 30 June 2019.

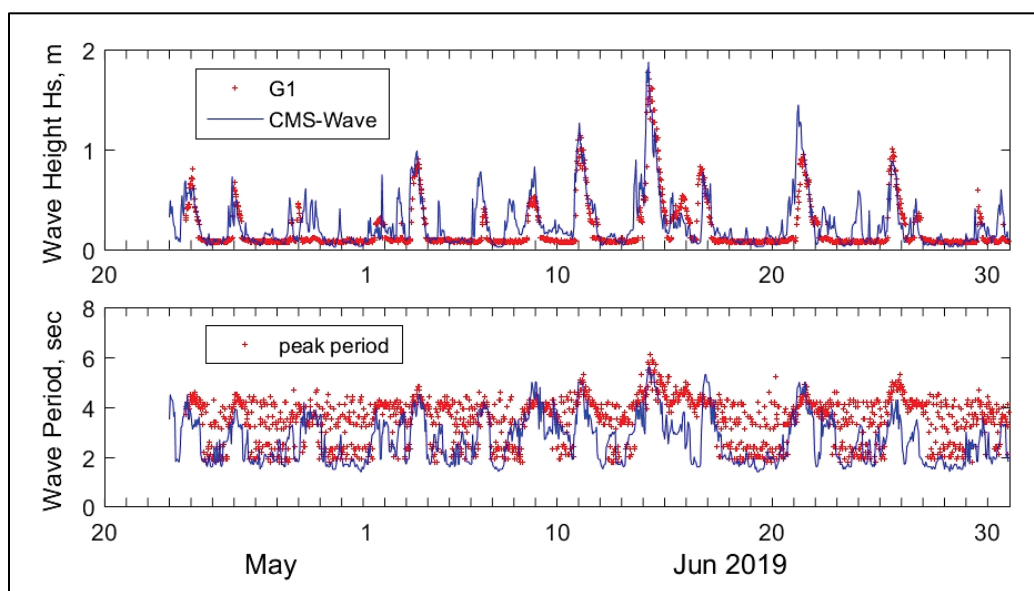


Figure 92. Model wave height and period versus data at G2 for 23 May to 30 June 2019.

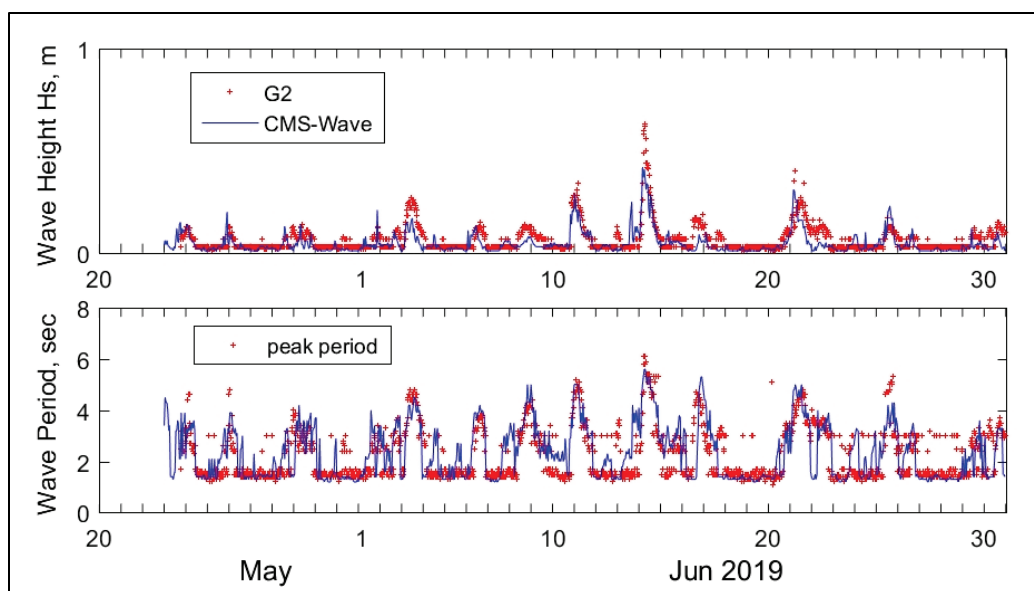


Table 55. Statistics of model wave heights versus data at G1 and G2 for 23 May to 30 June 2019.

Statistics	G1	G2
Bias [m]	0.06	-0.01
RMSE [m]	0.16	0.07
R^2	0.84	0.33

Figure 93 and Figure 94 show model wave heights and periods versus data from 1 July to 7 August 2019 at Gauges G1 and G2, respectively. Table 56 presents the corresponding statistics for model wave height and data comparison at G1 and G2.

Figure 93. Model wave height and period versus data at G1 for 1 July to 7 August 2019.

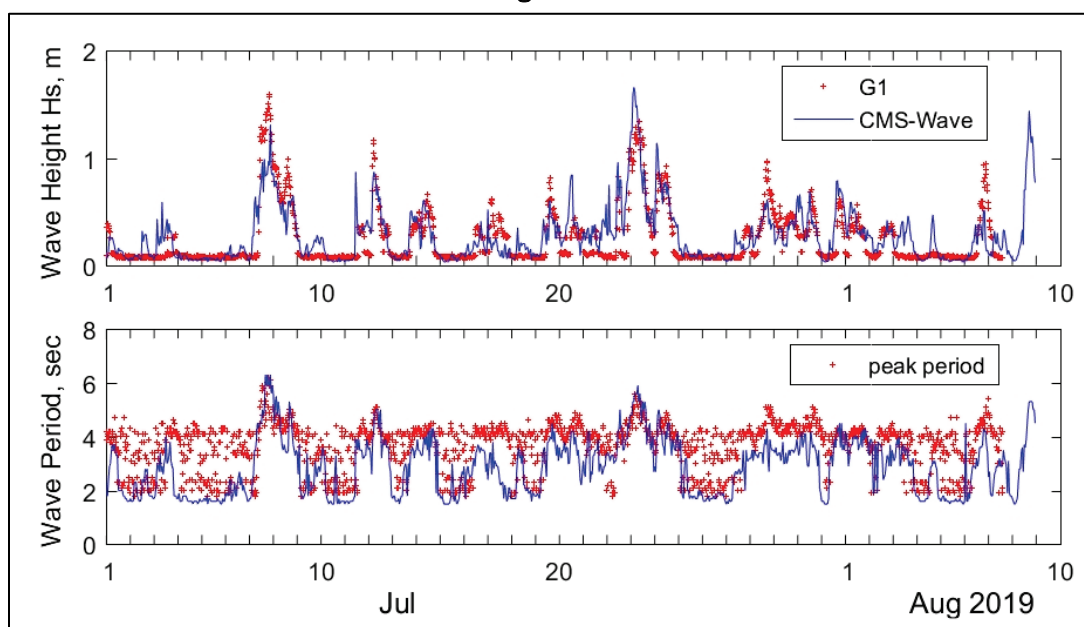


Figure 94. Model wave height and period versus data at G2 for 1 July to 7 August 2019.

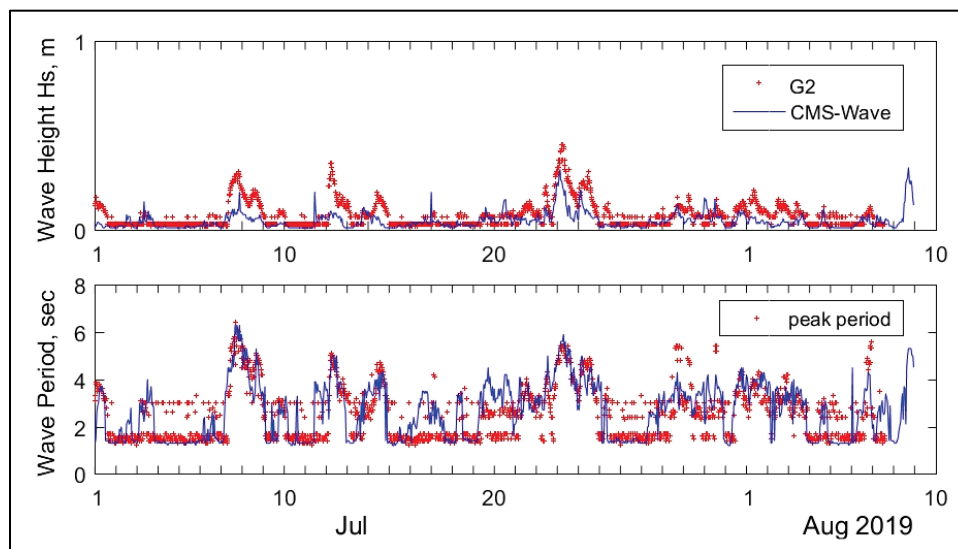


Table 56. Statistics of model wave heights versus data at G1 and G2 for 1 July to 7 August 2019.

Statistics	G1	G2
Bias [m]	0.03	-0.02
RMSE [m]	0.14	0.05
R ²	0.85	0.71

Tables 57 and 58 list the model wave height accuracy statistics at Gauges G1 and G2, respectively, for May to August 2019.

Table 57. Model wave height accuracy statistics at G1 for May to August 2019.

Month (2019)	RMSE (m)	NRMSE (%)*	R ²
May - June	0.16	8.16	0.84
July - August	0.14	8.05	0.85

*Normalized by model maximum wave height in a month.

Table 58. Model wave height accuracy statistics at G2 for May to August 2019.

Month (2019)	RMSE (m)	NRMSE (%)*	R ²
May - June	0.07	15.6	0.33
July - August	0.05	13.2	0.71

*Normalized by model maximum wave height in a month.

Generally, a slightly better agreement was observed between model-data comparison for wave height at the outer gauge G1 than inner gauge G2. The agreement that is less satisfactory at G2 could be due to entrapped debris buildup inside the harbor that contaminated the wave data. The model wind forcing using NDBC Buoy 45169 may also affect model results, as the wind input may not represent the over-water winds well in the inner and outer harbor areas for modeling.

4.8.5 Conclusions and recommendations

CMS-Wave is tested and compared with field wave measurements from Ashtabula Harbor, located in the southeast central coast of Lake Erie. Directional wave data were collected from wave gauges located outside and inside the harbor entrance. The inside wave gauge is behind the east breakwater, which shelters the entrance channel for predominant waves from the north and east. Both model wave height and period agree well with data inside and outside the harbor. Model results show that CMS-Wave is practical and reliable to simulate wave conditions for the coastal harbor application.

5 Summary and Recommendations

The comprehensive V&V study of hydrodynamic and wave modeling for the updated CMS and the GUI-SMS is conducted and the model evaluated with analytical, laboratory, and field applications across a wide variety of conditions. Field cases in this study cover coastal inlets, navigation channels, estuaries and bays, adjacent beaches, and coastal structures. For each test case, the CMS model setup is described, default values of the computational parameters are provided, and features and limitations of the model are discussed.

Goodness-of-fit statistics are calculated for different test cases, and Table 59 summarizes the NRMSEs for the comparisons between the analytical solutions, laboratory experiments, and field measurements, and the CMS calculations.

Table 59. Averages and ranges of NRMSEs for the comparisons between all analytical solutions, laboratory experiments, and field measurements, and calculated water surface elevations, currents, and significant wave heights.

NRMSE [%]	Analytical Solution		Laboratory Experiment		Field Case	
	Average	Range	Average	Range	Average	Range
Water Surface Elevation	2.3	0.1-5.4	*	*	6.5	1.6-20.4
Current	11.7	10.9-12.4	16.0	5.0-33.1	13.8	5.1-20.1
Significant Wave Height	3.2	1.2-5.2	17.3	4.9-74.8	13.4	8.3-27.2

*No water level comparisons for laboratory experiments.

For the comparisons with all analytical solutions, wherein CMS results are compared with analytical solutions to simplified versions of the equations governing open-channel and free-surface flow, the lower average NRMSE values prove that CMS accurately and robustly reproduces the fundamental processes of coastal hydrodynamics. All the field cases present an average NRMSE value of 6.5% for water level comparisons and approximate 13% for current and significant wave height comparisons, which demonstrates the CMS applicability in real world.

Through the V&V process, a summary and user recommendations are given as follows.

- Analytical cases verify the CMS in simulating wind-induced flow, tidal propagation, transcritical flow, and long-wave runup. The model accuracy in representing ocean dynamic and coastal processes is demonstrated.
- When setting up a CMS simulation, it is recommended that open boundaries should be specified along straight boundaries, which are simpler to implement and may avoid errors induced by the stair-case representation of curved open boundaries. Depending on the geometry of an application, either the non-uniform or telescoping Cartesian grid can be designed. The telescoping grid provides more flexibility and efficiency and is recommended for most practical applications.
- The implicit solver is set as the default solver for a CMS simulation. If areas with great depth gradients or extensive wetting and drying spots exist in the model domain, smaller time-steps or large grid cells covering those areas are recommended to resolve physics and ensure model stability when applying this solver. If a small time-step greatly reduces the model efficiency, use of the explicit solver is recommended.
- The inline flow and wave coupling (steering) is tested using laboratory and field cases in which the CMS-Flow and CMS-Wave models either share the same grid or use different grid (non-uniform Cartesian grid and telescoping grid). The CMS performs well while the coupling simulations proceed.
- Field cases cover inlet systems, estuaries, open coasts with the focus on nearshore zone, and rivers. Water level and land-water boundary conditions are tested, and the CMS performance is examined with wind, wave, river, and tidal forcing.
- In most applications, the CMS results are not sensitive to the selections of the horizontal eddy viscosity and wave-current bottom friction coefficient. The main calibration parameter is usually the bottom roughness (e.g., Manning's roughness coefficient). Also, the model results can be sensitive to wall friction when simulating channel flows. The default turbulence settings (turbulence models and parameters) seem to work well for most V&V cases. However, other embedded turbulence models may need further investigation for different cases.
- For these laboratory cases with regular monochromatic waves, the best results are obtained with the wave surface roller turned on. Test cases also show that the surface roller enhances wave-induced nearshore

longshore and cross-shore currents. It is recommended that the roller be included in nearshore simulations for best representation of the longshore and cross-shore currents.

- In most coastal applications the domain scales usually range from a few kilometers to tens of kilometers. The CMS results of the field cases show that it is reasonable to apply spatially constant wind forcing over the study domains. However, spatially varying winds might be needed for simulations of tropical storm passage.
- For the implicit solver, a spin-up period of 1 to 6 hr is recommended for the system to reach dynamic equilibrium. Comparing with that, the explicit solver requires a longer spin-up period, usually from a few hours to a few days.

All the data sets discussed in this report can be used for V&V of other numerical models and are available from the CIRP website (<https://cirp.usace.army.mil>).

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Appendix: Goodness-of-Fit Statistics

In this report, the Root Mean Square Error (RMSE) is defined as

$$RMSE = \sqrt{\langle (x_c - x_m)^2 \rangle} \quad (A-1)$$

The RMSE has the same units as the measured data. Lower values of RMSE indicate a better match between measured (x_m) and computed (x_c) values.

The Normalized Root Mean Squared Error (NRMSE) is defined as

$$NRMSE = \frac{RMSE}{range(x_m)} \quad (A-2)$$

The NRMSE is often expressed as percent ($NRMSE \times 100\%$). The measured data range, $range(x_m)$, can be estimated as $\max(x_m) - \min(x_m)$. Lower values of NRMSE indicate a better agreement between measured and computed values.

The Mean Absolute Error (MAE) is defined as

$$MAE = \langle |x_c - x_m| \rangle \quad (A-3)$$

Similarly, the Normalized Mean Absolute Error (NMAE) is given by

$$NMAE = \frac{MAE}{range(x_m)} \quad (A-4)$$

The NMAE is also expressed as percent quantity ($NMAE \times 100\%$). Smaller values of NMAE indicate a better agreement between measured and calculated values.

Correlation is a measure of the strength and direction of a linear relationship between two variables. The correlation coefficient R is defined as

$$R = \frac{\langle x_m x_c \rangle - \langle x_m \rangle \langle x_c \rangle}{\sqrt{\langle x_m^2 \rangle - \langle x_m \rangle^2} \sqrt{\langle x_c^2 \rangle - \langle x_c \rangle^2}} \quad (A-5)$$

A correlation of 1 indicates a perfect one-to-one linear relationship, and -1 indicates a negative relationship. The square of the correlation coefficient is the variance between two variables as described by a linear fit. The interpretation of the correlation coefficient depends on the context and purposes for which this statistical measure is used. In the present work, the following interpretations apply to model-to-data comparisons: $0.7 < R^2 < 1$ for a strong correlation, $0.4 < R^2 < 0.7$ for a medium correlation, $0.2 < R^2 < 0.4$ for a weak or small correlation, and $R^2 < 0.2$ for no correlation.

The model Bias is defined as

$$Bias = \langle x_c - x_m \rangle \quad (A-6)$$

Positive values indicate overprediction and negative values indicate underprediction.

Abbreviations

ADCP	Acoustic Doppler current profilers
ADP	Acoustic Doppler profiler
AWAC	Acoustic Wave and Current Profiler
CHL	Coastal and Hydraulics Laboratory
CIRP	Coastal Inlets Research Program
CMS	Coastal Modeling System
CMTB	Coastal Model Test Bed
D-R	Diffraction-refraction
FRF	Field Research Facility
GLCFS	Great Lakes Coastal Forecasting System
ISGS	Illinois State Geological Survey
MAE	Mean absolute error
NDBC	National Data Buoy Center
NMAE	Normalized mean absolute error
NRMSE	Normalized root mean square error

ODMDS Ocean dredged material disposal site

RM River Mile

RMSE Root mean square error

SMB Sverdrup-Munk-Brettschneider

SMS Surface-water Modeling System

SPM Shore Protection Manual

USACE US Army Corps of Engineers

V&V Verification and validation

WIS Wave Information Studies

WSE Water surface elevation

REPORT DOCUMENTATION PAGE					Form Approved OMB No. 0704-0188	
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1. REPORT DATE September 2022		2. REPORT TYPE Final Report		3. DATES COVERED (From - To) FY21–FY22		
4. TITLE AND SUBTITLE A Revisit and Update on the Verification and Validation of the Coastal Modeling System (CMS): Report 1–Hydrodynamics and Waves				5a. CONTRACT NUMBER		
				5b. GRANT NUMBER		
				5c. PROGRAM ELEMENT NUMBER		
6. AUTHOR(S) Honghai Li, Lihwa Lin, Cody L. Johnson, Yan Ding, Mitchell E. Brown, Tanya M. Beck, Alejandro Sánchez, and Weiming Wu				5d. PROJECT NUMBER		
				5e. TASK NUMBER		
				5f. WORK UNIT NUMBER		
7. PERFORMING ORGANIZATION NAME(S) AND ADDRESS(ES) (See reverse)				8. PERFORMING ORGANIZATION REPORT NUMBER ERDC/CHL TR-22-18		
9. SPONSORING/MONITORING AGENCY NAME(S) AND ADDRESS(ES) Coastal and Hydraulics Laboratory US Army Engineer Research and Development Center Vicksburg, MS 39180-2199				10. SPONSOR/MONITOR'S ACRONYM(S) ERDC CHL		
				11. SPONSOR/MONITOR'S REPORT NUMBER(S)		
12. DISTRIBUTION/AVAILABILITY STATEMENT Approved for public release; distribution is unlimited.						
13. SUPPLEMENTARY NOTES Funding Account Code U430420; AMSCO Code 060000						
14. ABSTRACT This is the first part of a two-part report that revisits and updates the verification and validation (V&V) of the Coastal Modeling System (CMS). The V&V study in this part of the report focuses on hydrodynamic and wave modeling. With the updated CMS code (Version 5) and its latest graphical user interface, the Surface-water Modeling System (Version 13), the goal of this study is to revisit some early CMS V&V cases and assess some new cases on model performance in coastal applications. The V&V process includes the comparison and evaluation of the CMS output against analytical solutions, laboratory experiments in prototype cases, and field cases in and around coastal inlets and navigation projects. The V&V results prove that the basic physics incorporated are represented well, the computational algorithms implemented are accurate, and the coastal processes are reproduced well. This report provides the detailed descriptions of those test simulations, which include the model configuration, the selection of model parameters, the determination of model forcing, and the quantitative assessment of the model and data comparisons. It is to be hoped that, through the V&V process, the CMS users will better understand the model's capability and limitation as a tool to solve real-world problems.						
15. SUBJECT TERMS Coastal engineering, Hydrodynamics--Computer simulation, Model-integrated computing, Water waves—Computer simulation						
16. SECURITY CLASSIFICATION OF:			17. LIMITATION OF ABSTRACT	18. NUMBER OF PAGES	19a. NAME OF RESPONSIBLE PERSON	
a. REPORT	b. ABSTRACT	c. THIS PAGE			Honghai Li	
Unclassified	Unclassified	Unclassified	SAR	154	19b. TELEPHONE NUMBER (Include area code) 601-634-2840	

7. PERFORMING ORGANIZATION NAME(S) AND ADDRESS(ES) (continued)

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