US Army Corps of Engineers®
New York District

Integrated Hurricane Sandy General Reevaluation Report and Environmental Impact Statement

Atlantic Coast of New York

East Rockaway Inlet to Rockaway Inlet and Jamaica Bay

Sub Appendix A1-A:

ENGINEERING MODELING APPENDIX

US Army Corps of Engineers

A. SUB-APPENDIX A1.A: ENGINEERING MODELING APPENDIX

EAST ROCKAWAY TO ROCKAWAY INLET, NY

REFORMULATION STUDY

ENGINEERING MODELING APPENDIX

U.S. Army Corps of Engineers

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1.0 INTRODUCTION

1.1 Overview of Numerical Modeling

The numerical modeling strategy for Rockaway addresses a comprehensive list of physical processes by utilizing a range of hydrodynamic, wave, sediment transport, and shoreline change models. The following numerical models were applied in the study:

- ADCIRC – storm surge propagation;
- WISWAVE – regional wave transformation;
- STWAVE – nearshore wave transformation;
- SWAN – nearshore wave transformation;
- GENESIS – long-term shoreline evolution;
- SBEACH – storm induced profile change;
- XBEACH – cross-island flooding.

1.2 Reach Delineation

The Project Area has been divided into smaller segments known as reaches for the purpose of engineering and economic analysis. As shown in Figure 1-1, the Project Area is an 11 mile long narrow peninsula with Rockaway Inlet at the western project limit near Breezy Point and East Rockaway Inlet at the eastern limit near Beach 19th Street. The purpose of reach delineation is to facilitate the engineering and economics analyses. The engineering analyses include historical erosion rates, sediment budget, historic shoreline changes and alternatives design considerations. The economic analyses include damages, cost, and benefit estimates. The reaches are developed based on physical, economic, and institutional differences including sediment transport rate boundaries, shoreline orientation, coastal structures, topographic elevations, and existing economic developments. The engineering reaches do not have to coincide with the economic reaches; however, they must be interrelated. The Project Area has been divided into six major reaches based on both engineering and economic considerations as follows:

- Reach 1: Rockaway Point to Beach 193rd Street
- Reach 2: Beach 193rd Street to Beach 149th Street
- Reach 3: Beach 149th Street to Beach 109th Street
- Reach 4: Beach 109th Street to Beach 86th Street
- Reach 5: Beach 86th Street to Beach 42nd Street
- Reach 6a: Beach 42nd Street to Beach 28th Street



- Reach 6b: Beach 28th Street to Beach 19th Street

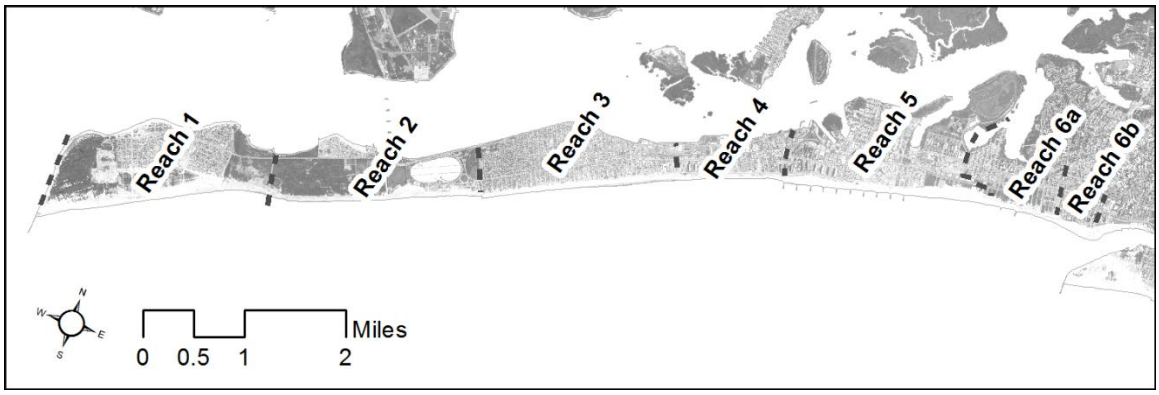


Figure 1-1: Rockaway Engineering Reaches

2.0 HISTORICAL STORM SUITE

2.1 Storm Selection

The historical storm suite for this study used in the Beach-fx model is based on a training set of 38 historical tropical and extratropical storms. The training set of storms was developed originally for the Fire Island to Montauk Point (FIMP) Reformulation Study (USACE-NAN, 2005). Two of the 38 storms, Hurricane Irene (2011) and Hurricane Sandy, are recent events and were added specifically for the Rockaway Reformulation Study. The original 36 storms were selected for FIMP by evaluating the National Hurricane Center's (NHC) hurricane tracks database and NOAA water level measurements at Sandy Hook, NJ, Montauk, NY, and Newport, RI. All tropical storms from 1930 through 2001 whose track came within 500 nautical miles of Long Island were considered. Of these storms those with peak surges greater than 2.23 ft, at any of the three tidal stations were selected for storm surge modeling. The September 1954 hurricane was added because Harris (1963) reports a peak surge over 3.3 ft at Montauk.

All historical extratropical events from 1950 to 1998 with a peak surge greater than 3.3 ft at any of the three tidal stations was included in the extratropical storm training set. The storm suite includes 16 tropical events and 22 extratropical events. A complete list of the storm training set is shown in Table 2-1.

Table 2-1: Historical Storm Training Set

Storm	Hs (ft)	Tp (s)	Dir (deg TN)	Peak Surge (ft)	Tropical (T) or Extratropical (E)
1938-09-20	15.9	14	98	4.3	T
1944-09-12	13.7	14	97	4.9	T
1950-11-22	18.9	14	112	4.3	E
1953-11-04	14.6	14	106	3.2	E
1954-08-27	7.1	11	98	2.7	T
1954-09-09	7.9	14	97	1.6	T
1954-10-14	19.6	11	134	3.7	T
1955-08-08	16.3	11	120	1.8	T
1955-10-11	12.8	11	108	2.9	E
1956-09-25	7.4	14	97	1.7	E
1960-09-11	11.5	14	97	4.4	T
1961-09-19	9.6	14	96	1.4	T
1962-03-03	12.2	14	96	3	E
1971-08-27	16.1	11	140	3	T

1972-06-19	12.9	11	115	2.7	T
1976-08-08	12.7	11	103	3.3	T
1977-11-05	10.2	11	107	2.1	E
1978-01-17	7.1	11	97	2.4	E
1978-02-04	7.2	14	96	2.2	E
1979-01-22	13.6	11	128	2.3	E
1980-10-22	15.4	11	141	3.2	E
1984-03-26	9.7	14	96	3.5	E
1985-02-09	12.3	11	120	2.1	E
1985-09-26	13.9	11	135	6.6	T
1991-08-17	8.6	11	97	1.6	T
1991-10-27	8.4	14	96	2	E
1992-01-01	7.1	11	97	2	E
1992-12-08	13.8	14	98	3.9	E
1993-03-02	8.5	11	99	2.1	E
1993-03-09	12.7	11	151	3.2	E
1994-02-28	8.8	11	99	2.7	E
1994-12-21	3.9	11	96	1.8	E
1996-01-05	7.6	14	96	2.6	E
1996-10-07	4.9	9	97	1.5	E
1998-02-02	7.4	14	96	1.9	E
1999-09-15	10.7	9	112	2.7	T
2011-08-28	18.9	13.7	144	4.6	T
2012-10-29	22.8	14.6	137	9.8	T

2.2 Average Rate of Storm Occurrence

The original storm training set was developed based on tropical events from 1930 through 2001 and extratropical events from 1950 through 1998. Tide measurements at Sandy Hook, Montauk, and Newport were analyzed from 2000 through 2013 in order to adjust the average rate of storm occurrence. The storm surge at each station was calculated from the measured and predicted tide. All storm events that fall within the hurricane season (June 1 to Nov 30th) with a storm surge greater than 2.23 ft were identified as possible tropical events. Any storm events from Dec 1 to May 31 with a storm surge greater than 3.3 ft were identified as new extratropical events.



Potential tropical events were cross-referenced against records of past hurricane seasons to determine if the surge was the result of a tropical or extratropical disturbance. All but three of the possible tropical storm surge events were actually extratropical storms. In total three tropical (Table 2-2) and nine extratropical (Table 2-3) events were identified from 2000 to 2013. Bringing the total number of tropical events from 1930 to 2013 (84 years) to 17, and the total number of extratropical events from 1950 to 2013 (64 years) to 31. The updated average rate of storm occurrence is 0.20 for tropical events, and 0.48 for extratropical events. It is noted that only Hurricane Irene and Sandy were included in the revised storm training set. However, the purpose of this analysis was to adjust the average rate of storm occurrence. The storm training set includes sufficient range of tropical storm and extratropical storm events. It was imperative to include Hurricane Sandy since it is the storm of record and may have a significant impact on the profile changes and storm damages calculated in the study.

Table 2-2: Tropical Events (2000 to 2013)

Date	Storm	Measured Peak Surge at Sandy Hook (feet)
2012-10-29	Hurricane Sandy	7.79[1]
2011-08-28	Hurricane Irene	4.50
2005-10-25	Hurricane Wilma	3.96

[1]Sensor failed, peak surge is estimated to be 9.8 ft

Table 2-3: Extratropical Events (2000 to 2013)

Date	Measured Peak Surge at Sandy Hook (feet)
2012-12-27	5.29
2010-03-14	4.40
2012-12-21	3.96
2009-12-09	3.88
2010-12-27	3.54[1]
2003-12-15	3.53
2007-02-14	3.36
2002-12-25	3.36[1]
2007-04-16	3.34

[1]Peak Surge measured at Montauk

2.3 Storm Suite Expansion for Beach-fx

The historical storm training set was expanded to the Beach-fx storm suite by assuming that the historical storm could have occurred at one of 12 possible tidal phase/range combinations. The storm surge for each of the storms in the historical training set was combined with three tidal



ranges (spring, mean, and neap) and four different phases of the tidal cycle by aligning the peak surge with high tide, mean falling tide, low tide, or mean rising tide. As a result 12 different hydrographs were generated for each storm in the training set, increasing the total number of storms in the Beach-fx storm suite to 456 (38 x 12).

An overview of the peak water level for all 456 events in the Beach-fx storm suite is provided in Figure 2-1. Each dot in Figure 2-1 represents one of the 456 storms in the storm suite and the solid black line represents the overall FEMA stage frequency curve offshore of Rockaway. It is apparent from the top and bottom panels that tropical storm events represent the most severe storm events, form a storm tide perspective. The largest extratropical storm only has a return period of less than 20 years. The majority of severe storm tide events are derived from Hurricane Sandy (blue dots). The largest of the storm events has a peak water level of 12.7 ft NAVD and a return period of 217 years.

It is important to recognize that factors other than the just the storm tide affect the magnitude of coastal erosion and potential damages. The wave height, wave period, and duration of the storm all also have a large impact on coastal erosion.

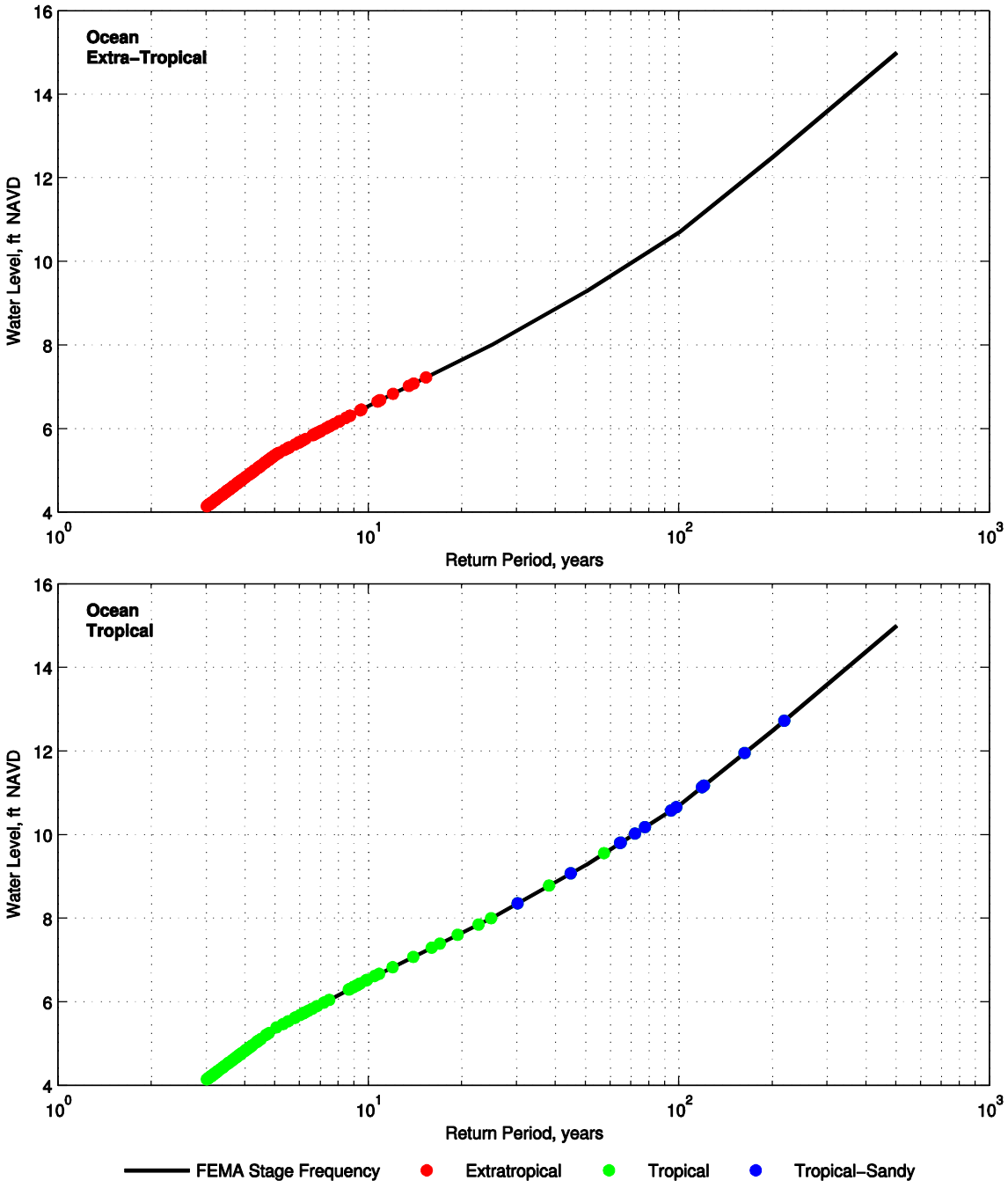


Figure 2-1: Historical Storm Suite – Ocean Stages

2.4 Determination of Equivalent Back Bay Water Levels

Back bay flooding may be incorporated in Beach-fx based on a single peak back bay water level during each storm event. Back bay water levels were estimated in this study by assuming that the return period of the peak water levels on the ocean-side and bay-side are the same for each storm.

In reality the bay-side and ocean-side return periods may differ for individual storm events due to timing of the storm and local wind patterns.

An example of how the equivalent back bay water level is obtained from two storm events is presented in Figure 2-2. In this example the peak ocean water level of the two storms is 8 and 12 ft NAVD. The return period associated with these peak ocean water levels is 24 and 164 years. The equivalent peak bay water level, 7.8 and 10.5 ft NAVD, is obtained from the back bay stage frequency curve for these two return periods.

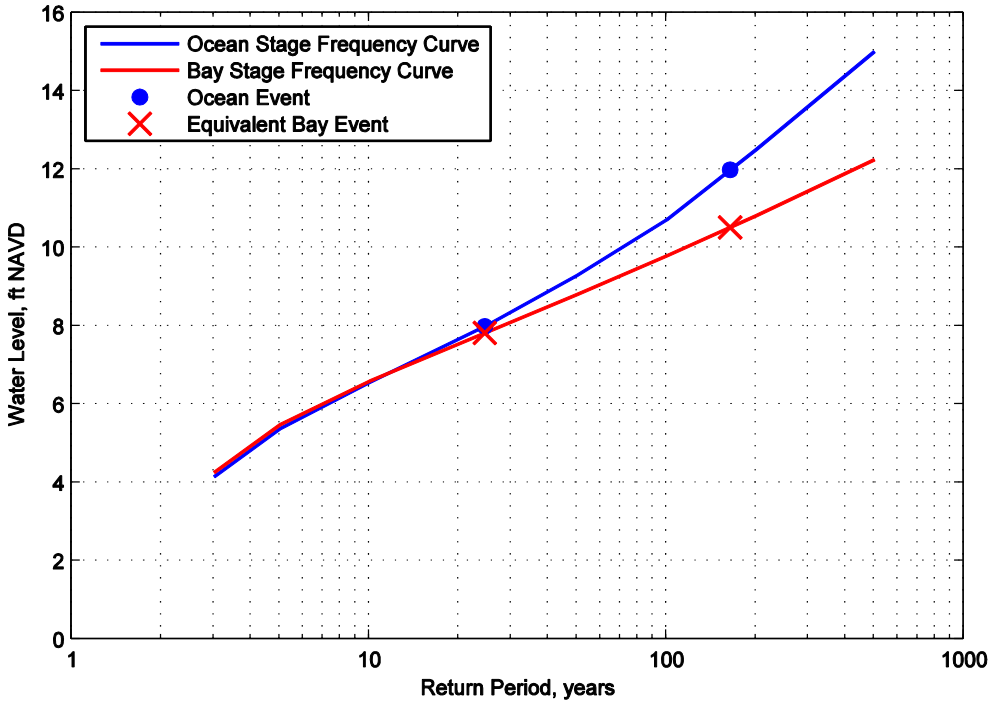


Figure 2-2: Example Showing How Peak Bay Water Levels are Determined

3.0 REGIONAL STORM SURGE AND WAVE MODELING

A comprehensive regional storm surge and wave modeling study was completed for the entire south shore of Long Island as part of the Fire Island to Montauk Point (FIMP) Reformulation study. The FIMP study provides the offshore storm tide and wave conditions for 36 of 38 storm events in the historical storm suite. The remaining two storms, Hurricanes Irene and Sandy, were based on observed storm tide and wave conditions. The regional storm surge and wave modeling results from FIMP are used here for Rockaway to define the conditions at Rockaway during the 36 historical storm events.

An overview of the FIMP modeling study is provided here, a complete description of the FIMP storm surge and wave modeling study is provided in the FIMP Baseline Conditions Report (USACE-NAN, 2005). Detailed wind and barometric pressure fields were developed by OCTI/Ocean Weather Inc. for each of the 36 historical storm events using either a Planetary Boundary Layer (PBL) model for tropical storm events or by Ocean Weather Inc.'s Interactive Kinematic Objective Analysis of publicly available wind fields from NOAA/NCEP Global Reanalysis Project (NRA). These wind and pressure fields were used to force the regional hydrodynamic and wave models. The FIMP study used the Advanced Circulation Model (ADCIRC) to model storm surge and the Wave Information Studies Wave Model (WISWAVE) to model wave conditions.

The ADCIRC and WISWAVE model domains encompassed nearly the entire eastern Atlantic Ocean and the entire east cost of the United States. Nesting or variable grid resolution was utilized by the models with finer resolution along the South Shore of Long Island. Modeling results from ADCIRC and WISWAVE provide time series of the wave conditions and storm tides offshore of Rockaway for the 36 historical storm events.

3.1 ADCIRC

Ocean and nearshore, outside the surf zone, storm water levels are simulated with ADCIRC Version 43.02 (ADvanced CIRculation model; Luettich et al., 1992). ADCIRC is a long-wave hydrodynamic numerical model that simulates water surface elevations and currents from astronomic tides, wind, and barometric pressure. ADCIRC solves the two-dimensional, depth-integrated momentum and continuity equations on a finite element grid in spherical coordinates.

3.2 Computational Grid

For the FIMP study, the numerical grid covers a large computational domain, spanning the northeastern Atlantic Ocean, to fully capture large-scale wind and pressure effects during storm events (Figure 3-1). In addition, the numerical grid has high resolution at inshore areas, such as the tidal inlets and bays, to fully capture the complexity of the hydrodynamics in these areas. The subaerial portions of the barrier islands are not included in the computational domain. Instead the shorelines are represented with shoreline boundary conditions. In total, the computational domain



includes 44329 nodes. Model bathymetry is based on several data sources. At the three FIMP tidal inlets and in Shinnecock Bay, recent high-resolution SHOALS LIDAR or multibeam survey data was used. In Moriches and Great South Bays and in the nearshore areas of FIMP, data from the GEODAS (GEOphysical DAta System) database, supplemented with data provided on NOAA charts, was employed. In all other areas, the bathymetry is based on that compiled for developing the East Coast 2001 Tidal Constituents Database (Mukai et al., 2002). This data set was developed using the ETOPO5, Digital Nautical Charts, and National Ocean Service raw sounding databases.

3.3 Model Forcing

The ADCIRC model is forced:

- On its offshore boundaries with astronomic tidal constituents from the ADCIRC East Coast 2001 Tidal Constituent Database for seven main tidal constituents (Mukai et al., 2002; Table 3-1).
- Throughout the computational domain with simulated wind and barometric pressure fields (see USACE-NAN 2005).

3.3.1 Model Setup

For each storm simulation, time series of simulated water level are output from the model every 6 minutes (real-time) at station 52 (Figure 3-2). Station 52 is used to force subsequent SBEACH and STWAVE simulations.

Table 3-1: ADCIRC Astronomic Tidal Constituents.

Tidal Constituent	NOAA Measured Amplitude (ft (m))	
	Sandy Hook, NJ (west of FIMP)	Montauk Fort Pond, NY (eastern FIMP)
M2	2.258 (0.688)	0.992 (0.302)
N2	0.518 (0.158)	0.260 (0.079)
S2	0.438 (0.134)	0.213 (0.065)
K2	0.126 (0.038)	0.061 (0.019)
K1	0.338 (0.103)	0.244 (0.074)
O1	0.176 (0.054)	0.176 (0.054)
Q1	0.037 (0.011)	0.049 (0.015)



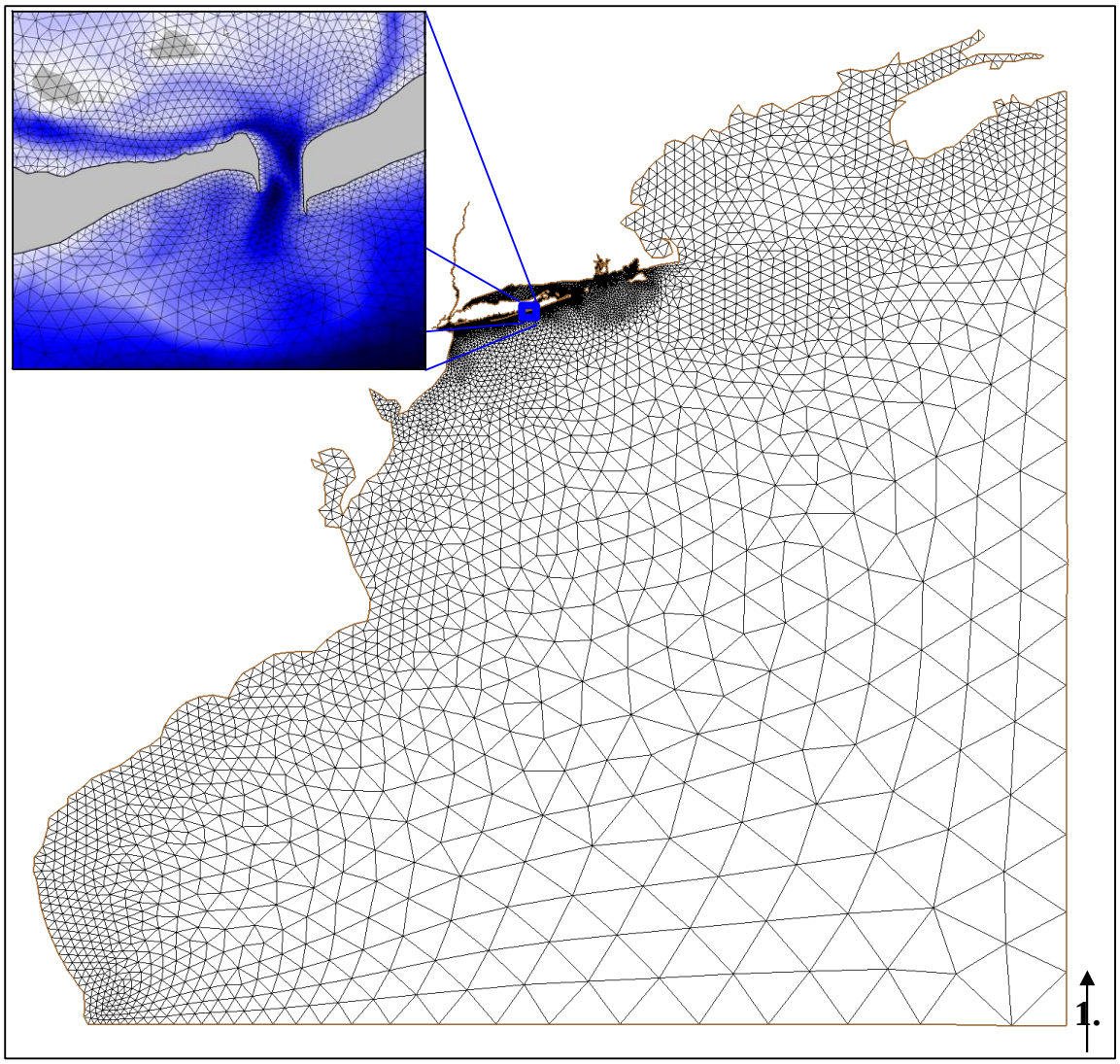


Figure 3-1: ADCIRC computational domain

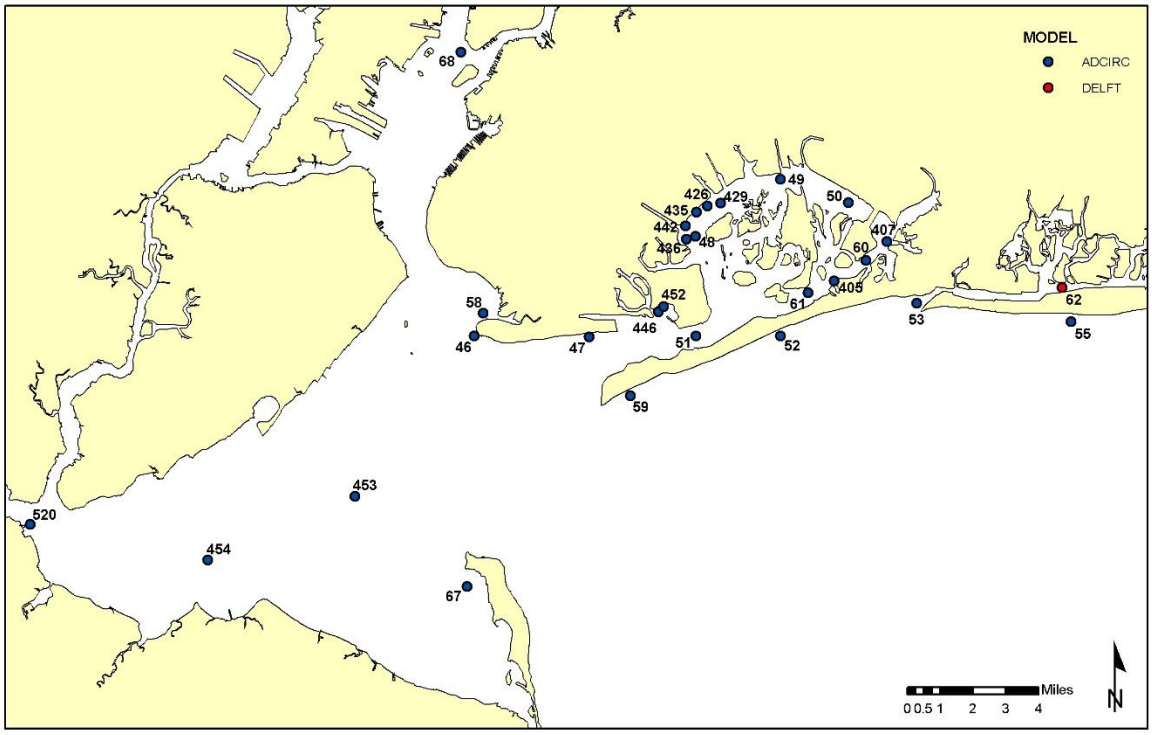


Figure 3-2: ADCIRC Storm water level output stations

3.4 WISWAVE

In an effort to determine extreme storm wave conditions during the training set of 36 storms (14 tropical and 22 extratropical), a directional spectral, temporally sensitive wave model WAVAD (also known as WISWAVE) was applied.

3.4.1 Model Input and Output Parameters

The wave model was driven by the wind fields developed for the wind hindcast, described in Chapter 2.5 of USACE-NAN (2005). The wind fields were interpolated onto the wave grid domains for each level of nesting. Input parameters for the model included the model time step, (30 seconds for the finest grid), the 15 wave frequencies over which the wave spectrum was computed (0.03 to 0.31 Hz with an increment of 0.02 Hz), and the discretization increment for the directional spectrum (22.5 degrees). Output included bulk parameters (zero-moment wave height, peak wave period, peak wave direction) and full two-dimensional wave spectra. Storm simulations were reported at output locations on an hourly basis.

3.4.2 Computational Grid

Open ocean bathymetry was obtained from NOAA nautical charts, with the New York Bight area resolved from chart 12300, "Approaches to New York" and from chart 12352, "Shinnecock Bay to East Rockaway Inlet." Five levels of nesting were used to generate the wave data for FIMP:

- A 1.0-degree grid extending from 50 degrees to 80 degrees west longitude and from 20 to 45 degrees north latitude.
- A 0.25-degree grid extending from 67.75 degrees to 74.25 degrees west longitude and from 36.75 to 42.25 degrees north latitude.
- A 0.08333-degree grid (5-min) extending from 69.50 degrees to 74.083 degrees west longitude and from 40.1666 to 41.3333 degrees north latitude (purple box in Figure 3-3).
- A 0.01667-degree (1-min) (approximately 1 nautical mile) grid extending from 70.750 degrees to 74.083 degrees west longitude and from 40.417 degrees to 41.000 degrees north latitude (red box in Figure 3-3).
- A 0.008333-degree (0.5-min) (approximately 0.5 nautical mile) grid extending from 72.08333 degrees to 73.5000 degrees west longitude and from 40.500 degrees to 40.833 degrees north latitude (green box in Figure 3-3).

The offshore wave forcing for STWAVE model simulations was extracted from station I07J04 in the 5 min resolution grid (purple box in Figure 3-3).

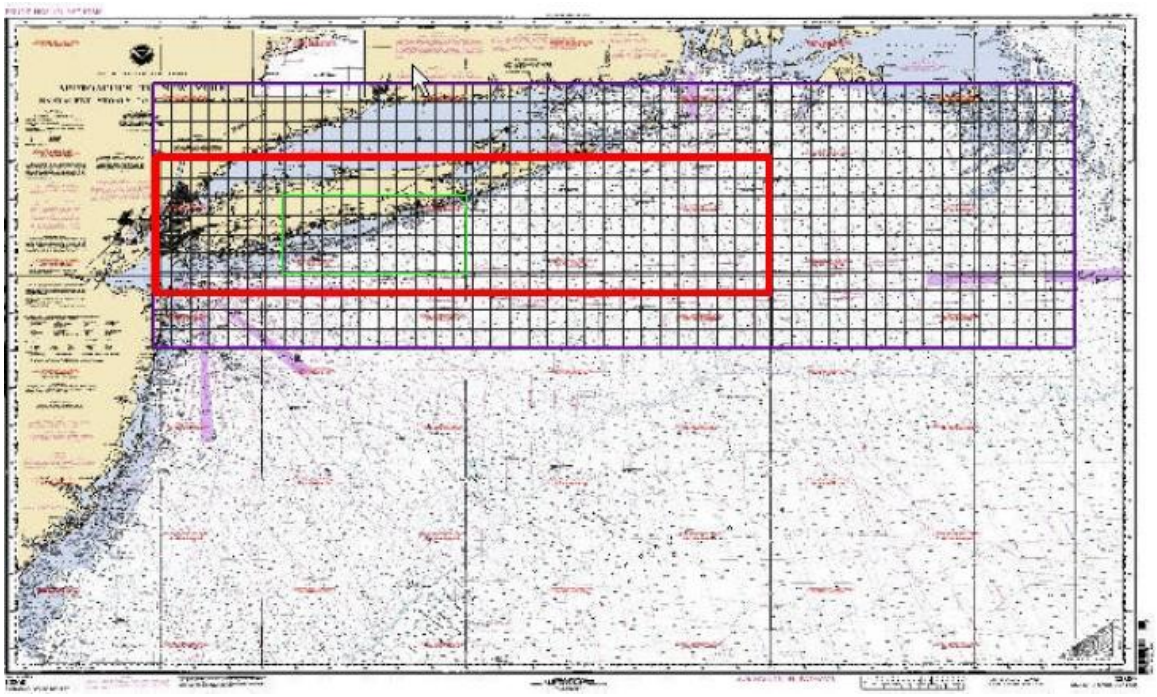


Figure 3-3: WISWAVE 5-min, 1-min, and 0.5 min model grid.

4.0 NEARSHORE WAVE MODELING

Two nearshore wave modeling studies were conducted for Rockaway. Prior to Hurricane Sandy STWAVE was used to determine the nearshore wave conditions at Rockaway during 36 of the 38 storms in the Historical Storm Suite. After Hurricane Sandy SWAN was used to determine the nearshore wave conditions during Hurricane Irene and Hurricane Sandy.

4.1 STWAVE

STWAVE modeling is required in this study to determine the nearshore wave conditions for each historical storm in the FIMP training set and to evaluate the impact excavating proposed borrow areas offshore of Rockaway have on nearshore wave conditions. STWAVE (Steady State spectral WAVE) is a phase-averaged spectral wave model appropriate for modeling wave transformation and wind wave generation in the nearshore (Smith, 2001). Here the full plane version, STWAVE-FP (Smith, 2007), is applied to simulate wave transformation and generation on the full 360-degree plane. The model accounts for the following physics: Linear wave refraction and shoaling over a bottom of variable depth; dissipation due to both depth and steepness induced wave breaking; dissipation due to bottom friction; wind-wave growth; non-linear wave-wave interactions and whitecapping. Wave-current interactions have not yet been implemented for STWAVE-FP. Since the model does not account for pure diffraction effects or wave-current interactions the wave field computed in the immediate vicinity of obstacles, harbors, and inlets will not be accurate.

STWAVE-FP is a finite difference model based on the wave action balance equation. STWAVE-FP computations are carried out on a rectangular grid. In STWAVE-FP the wave conditions must be homogenous along the offshore boundary. Wave conditions on the lateral boundaries can either be specified in the same way as the offshore boundary or determined from one-dimensional wave transformation along the boundary. Non-stationary conditions are simulated with STWAVE-FP by performing a steady state simulation at each time step. Such stationary wave computations are usually considered to be acceptable since the travel time of the waves from the seaward boundary to the coast is relatively small relative to the time scale of variations in the incoming wave field, the wind or tidal induced variations in depth and currents.

4.1.1 STWAVE Model Grid and Bathymetry

The STWAVE model grid extends from the western end of Coney Island to Jones Beach. From east to west it includes Coney Island, East Rockaway, Long Beach, and a portion of Jones Beach. The distance between the ocean shoreline and the offshore model boundary varies between 9 and 14 miles. However, the western lateral boundary is only 2.2 miles offshore of New Jersey. A Cartesian model grid is built with a constant grid resolution throughout the domain. The cross-shore resolution is (100 m) and the alongshore resolution is (200 m). Figure 4-1 shows the extent of the model grid and Figure 4-2 the resolution of the model grid. The model bathymetry was developed from a combination of offshore surveys, navigation charts, inlet surveys, and nearshore



beach profiles. Table 4-1 shows a list of the data sources used to create the model bathymetry. Figure 4-3 shows the model bathymetry constructed from these data sources. The depth at the offshore model boundary varies from (20.5 m) to (43.1 m). The datum of the model is NAVD88.

Table 4-1: STWAVE Bathymetric Data Sources

Data Source	Applied Region of Model
ACNYMP Beach Profiles (1996-2005)	Coney Island, Rockaway, Long Beach
NOAA Navigational Charts, Chart # 12327, 12350, 12352a, (2006-2009)	NY Harbor, Rockaway Inlet, Jones Inlet, Jones Beach
USGS Trackline Bathymetry from single beam echo sounder (1995-1998)	NY Bight Continental Shelf
USACE Single Beam Survey (2004)	East Rockaway Inlet

4.1.2 STWAVE Model Boundary Conditions

Boundary conditions for the historical storm training set simulations are developed from previous FIMP WISWAVE and ADCIRC modeling work (USACE-NAN, 2005). The WISWAVE model was driven by wind fields developed from a wind hindcast study. Bulk spectral parameters (zero-moment wave height, peak wave period, peak wave direction) are available from the WISWAVE model for the either the full 360° spectra or for a 160° banded spectra. The 160° banded spectra only contains wave energy propagating plus or minus 80° away from shore-normal. Since the offshore wave data will be transformed to nearshore, the WISWAVE model output derived from the 160° banded spectra was used to determine the nearshore wave conditions.

As noted earlier a limitation of STWAVE-FP is that the wave conditions along the offshore boundary must be homogenous. Comparison of the WISWAVE results along the offshore boundary suggests that the variation in the wave conditions along the offshore boundary is generally small. Therefore, WISWAVE results at I07J04 were used as input along the entire offshore boundary. The water depth at I07J04 is 23.3 m (NAVD88). The zero-moment wave height (Hs), peak wave period (Tp), and peak wave direction (θ) at I07J04 was used to create a TMA spectrum with a cosine power directional spreading distribution. The spectral grid consists of 30 wave frequencies (.04 Hz to .33 Hz with an increment of 0.01 HZ) and 72 directions (0° to 360° with an increment of 5°). The lateral boundaries are determined internally from one-dimensional (1-D) wave transformation along the boundary.

The water level (tide + storm surge) for the training set of storms is available from previous FIMP ADCIRC modeling (USACE-NAN, 2005). The water level for each STWAVE-FP steady state simulation can either be specified as a spatially varying or homogenous across the entire grid. Local wind wave generation is not included in the STWAVE-FP model, since the focus of this study is on wave conditions along Rockaway and locally generated waves are limited due to the



fetch. Bottom friction is also not included in the model since the dissipation from bottom frictions is negligible at this site. The offshore wave and water level conditions at the peak of the storm are shown in Table 2-1.

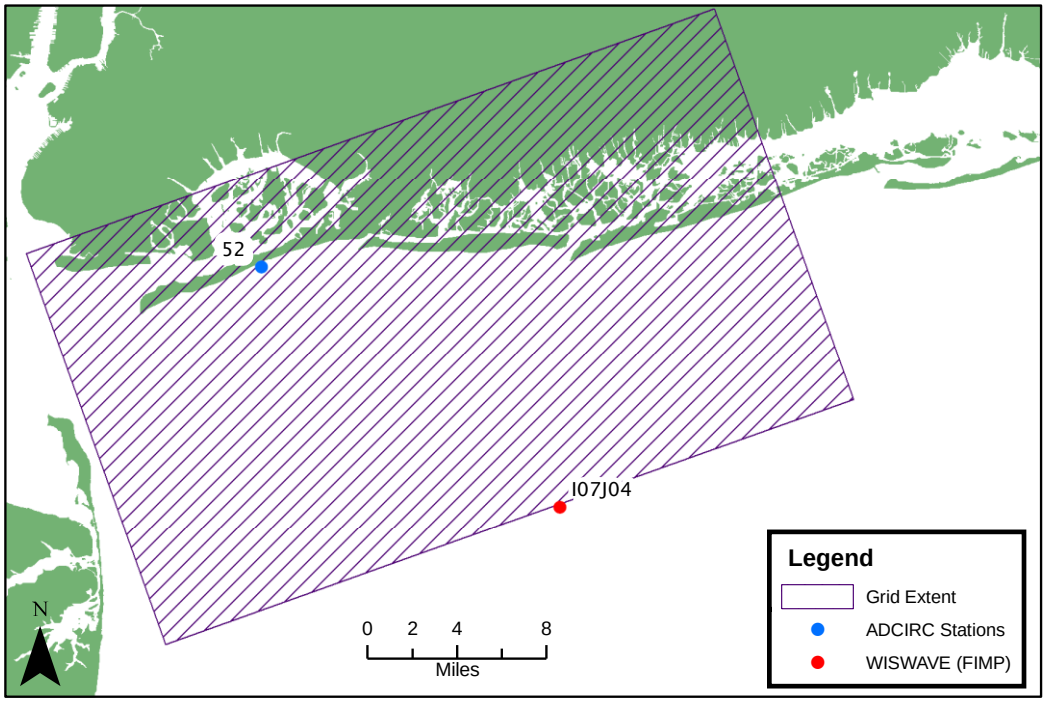


Figure 4-1: STWAVE Model Grid Extent

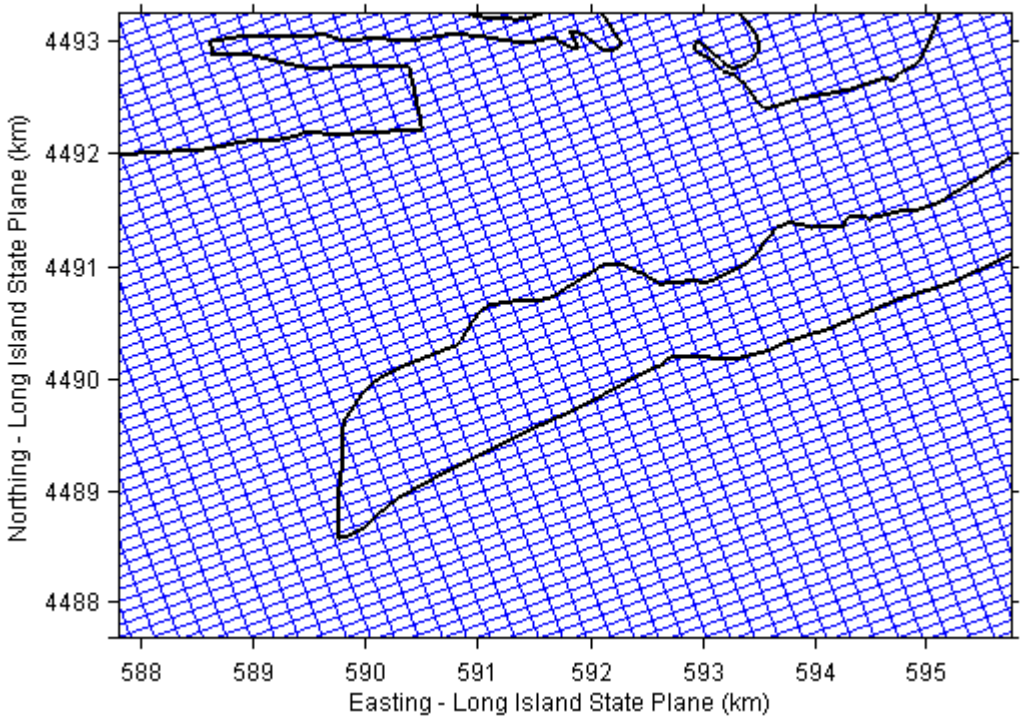


Figure 4-2: STWAVE Model Grid Resolution

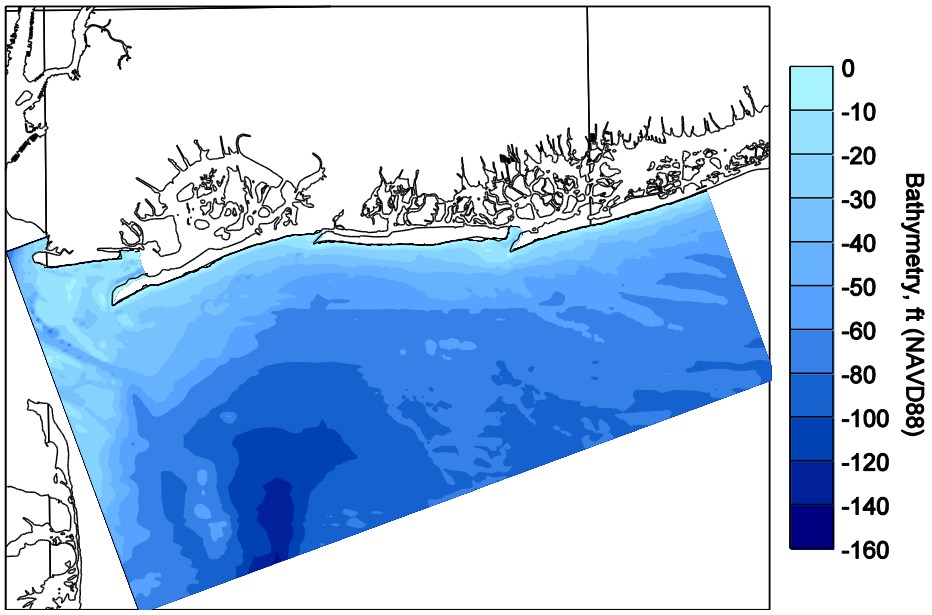


Figure 4-3: STWAVE Model Bathymetry

4.2 SWAN

Additional wave hindcast simulations were performed for Hurricanes Irene and Sandy using the spectral wave model SWAN. These model simulations were carried out several years after the 36 STWAVE hindcast simulations. SWAN, developed at Delft University of Technology in Netherlands, is a 3rd generation fully spectral (in all directions and frequencies) wave model based on the discrete spectral action balance equation. SWAN computes the evolution of random, short-crested waves in coastal regions with deep, intermediate and shallow water and ambient currents. SWAN includes wave generation (exponential wave growth by wind), wave propagation (shoaling and refraction), wave transformation (quadruplet and triad non-linear wave-wave interactions), and wave dissipation (wave breaking, whitecapping, and bottom friction). The SWAN model has successfully been validated and verified in several laboratory and complex field cases (Ris et al., 1999).

4.2.1 SWAN Model Grid

Four nested SWAN grids were used to simulate wave conditions offshore at NOAA NDBC Buoy 44025 to nearshore. The "Rockaway" model grid (shown in gold) is identical to the STWAVE model grid in extent, resolution, and bathymetry. The model bathymetry sources for the larger model domains are based on the NOAA Coastal Relief Model.

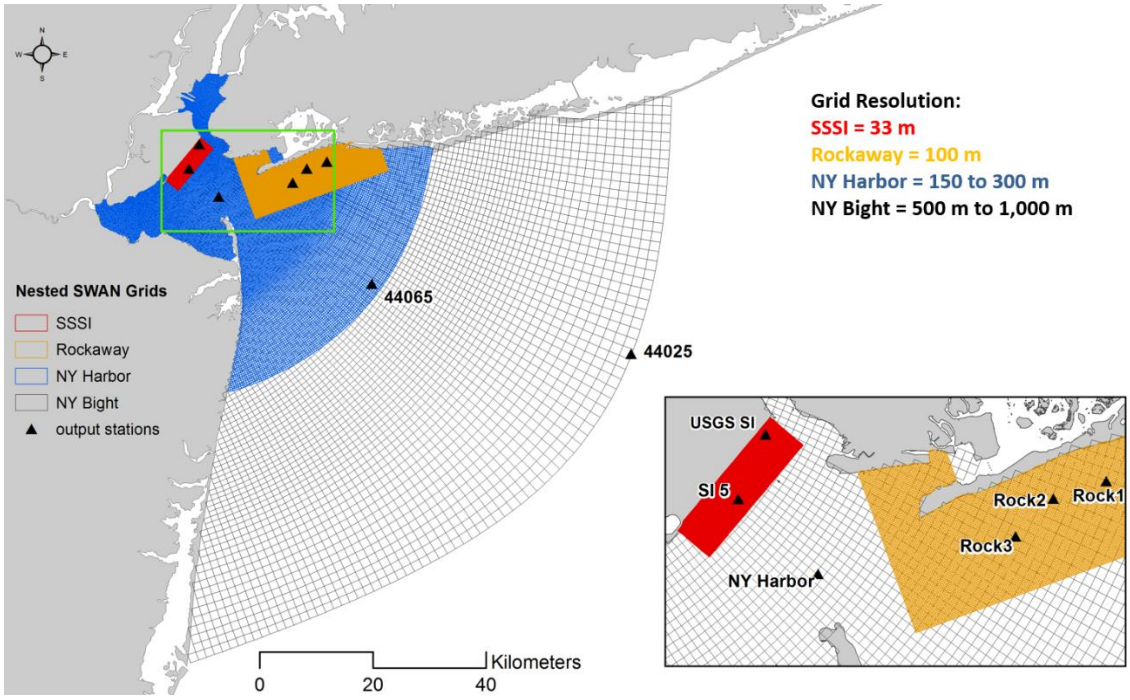


Figure 4-4: SWAN Model Grids

4.2.2 SWAN Model Boundary Conditions

SWAN boundary conditions for Hurricane's Sandy and Irene were obtained from NOAA NDBC Buoy 44025. Initially the model was force with the bulk wave parameters (H_s, T_p, MWD), however it became apparent that the bulk parameters were insufficient to achieve an acceptable model calibration (Figure 4-6). Instead, the model was forced with a directional frequency spectrum (1 hour time steps) from NDBC buoy. The frequency spectrum (energy in each frequency bin) is provided directly by NOAA (Figure 4-5), however the directional spectrum must be calculated based on the provided Longuet-Higgins Fourrier coefficients (r_1, r_2, α_1, α_2). Earle & Steele (1999) provide method to convert Longuet-Higgins Fourier coefficients to cross-spectral parameters (C11, C22, C33, C23, Q12, Q13). The cross-spectral parameters converted to a directional spectrum using the Maximum Likelihood Method (MLM) (Oltman-Shay and Guza, 1984).

During model calibration a shadow zone was also observed near Rockaway, which is attributed to the substantial wave energy being transported east, away from the boundary. This is a limitation of forcing the model with wave conditions at a single location. It is likely that the mean wave direction varies along the offshore boundary. To reduce the wave shadow zone the mean wave direction was set greater than or equal to 110 degrees for all frequency bins. Figure 4-7 shows a comparison of the model calibration using the raw direction spectrum vs. the adjusted direction spectrum.

A uniform wind field based on the observed data at NBDC buoy 40025 and uniform water levels based on a reconstructed water level time series at Sandy Hook are also applied in the SWAN simulations.

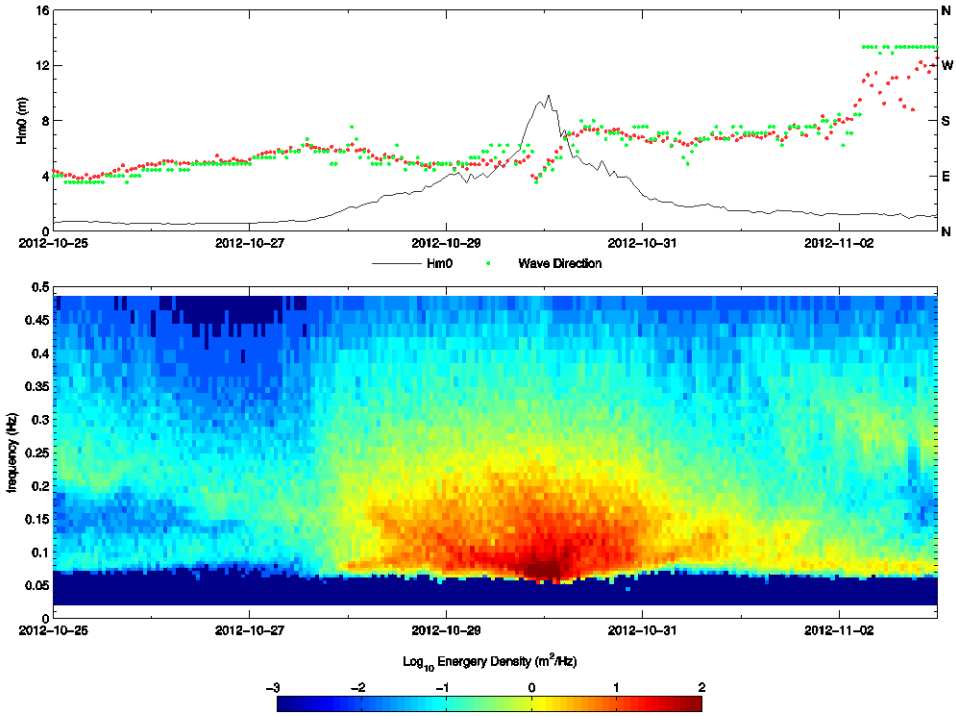


Figure 4-5: Spectrograph at NBDC 40025 during Hurricane Sandy

2013-10-29 01:00 UTC **2013-10-30 00:00 UTC**

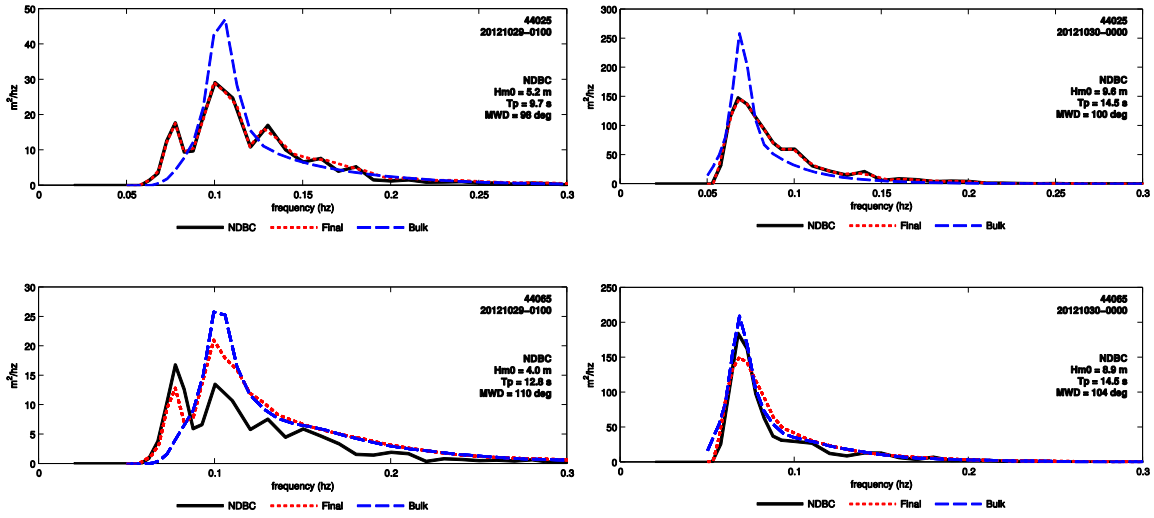


Figure 4-6: SWAN Model Sensitivity to Bulk vs. Spectral Boundaries

2013-10-29 20:00 UTC **2013-10-30 00:00 UTC**

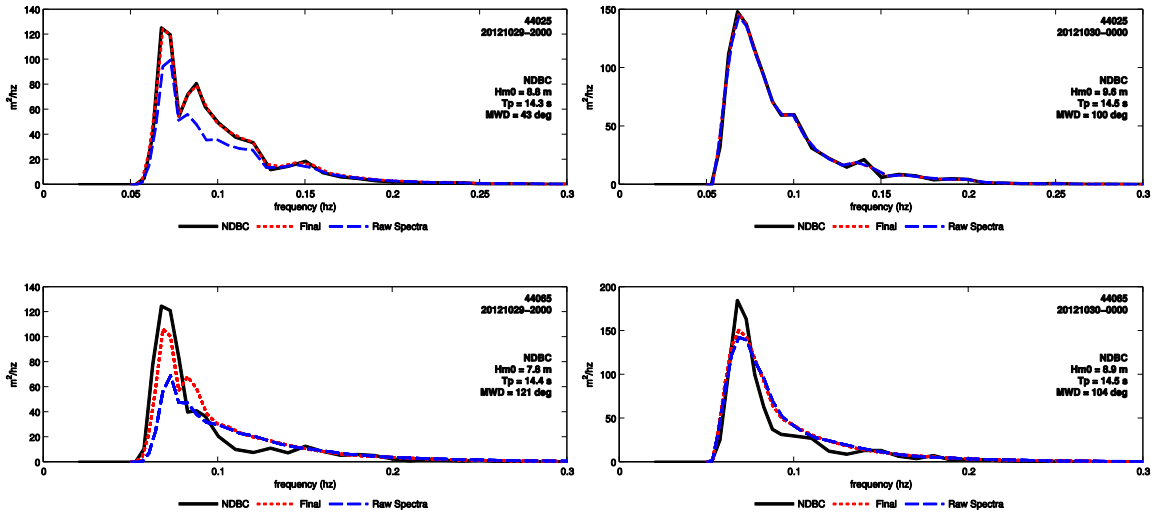


Figure 4-7: SWAN Model Sensitivity to Raw vs. Adjusted Spectrum Boundaries

4.2.3 SWAN Model Calibration

Site specific model calibration was performed to observations during Hurricane Sandy. NOAA NDBC buoys 44025 and 40065 and temporary USGS wave gauge SSI-NY-RIC-001WV. Model calibration at 44065 is excellent with good agreement in the wave height, peak wave period, and mean wave direction (Figure 4-8). A comparison of observed and modeled wave conditions at Staten Island (Figure 4-9) also indicate that the model is able to accurate predict the complex wave transformation processes all the way into New York Harbor. It is encouraging that the shape of the frequency spectrum at Staten Island is generally reproduced by the model, showing a bi-modal frequency spectrum with a combination of energy from sea (high-frequency) and swell (low-frequency) at the site.

An overview of the bulk wave parameters (significant wave height, peak wave period, and mean wave period) are shown in Figure 4-10. It is apparent that the waves lose energy, decrease in height, upon entering New York Harbor. Depth-induced wave breaking and energy dissipation from bottom friction are responsible for the decrease in energy, which is primarily associated with energy in the low-frequency portion of the spectrum. As a result, the mean wave period drops significantly upon entering the harbor. However, in much of the Harbor the peak wave period is still associated with the ocean swell since the energy in the low-frequency bins is more concentrated than in the higher-frequency bins.

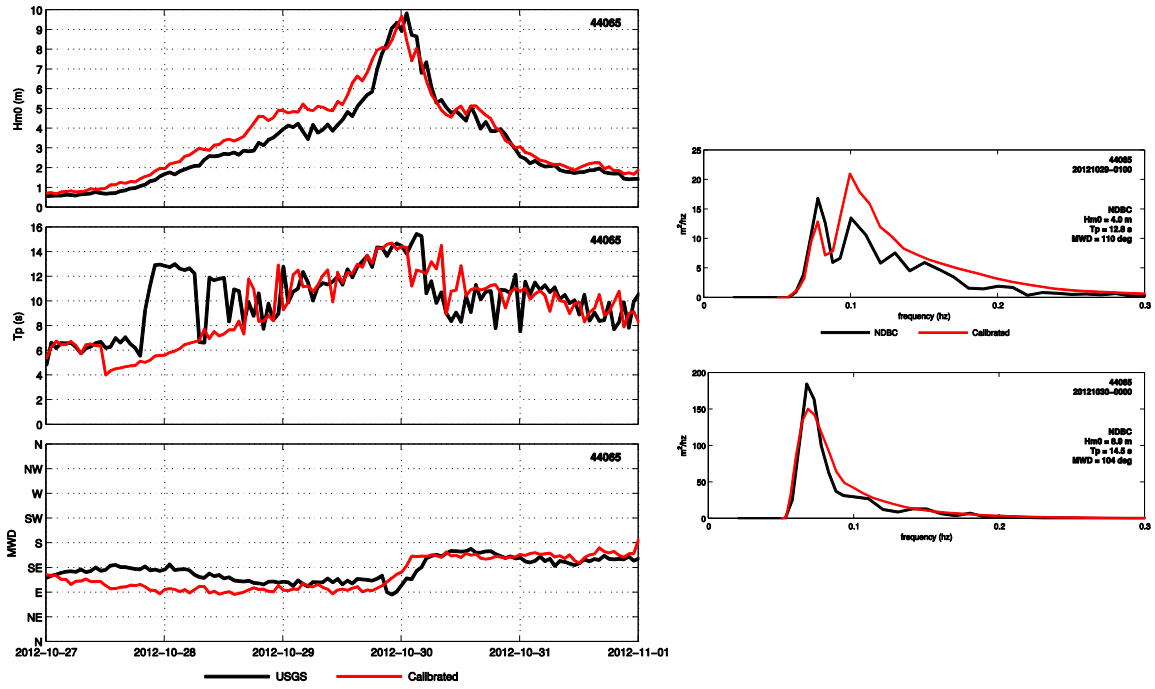


Figure 4-8: SWAN Calibration at 44065

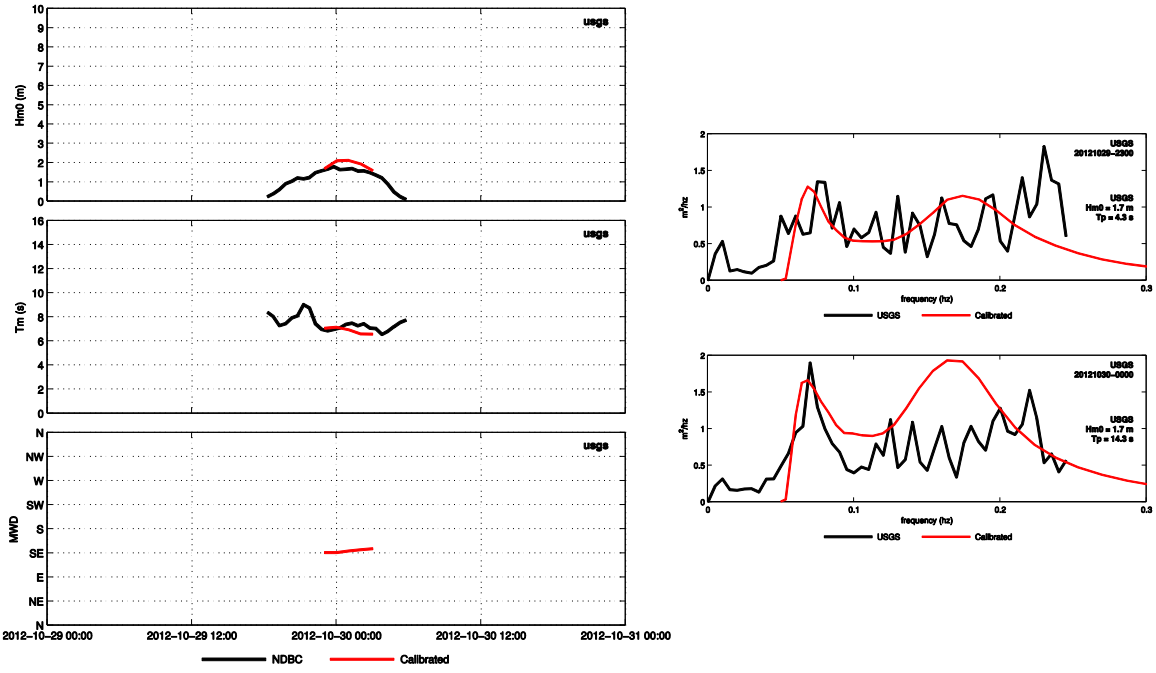


Figure 4-9: SWAN Calibration at SSI-NY-RIC-001WV

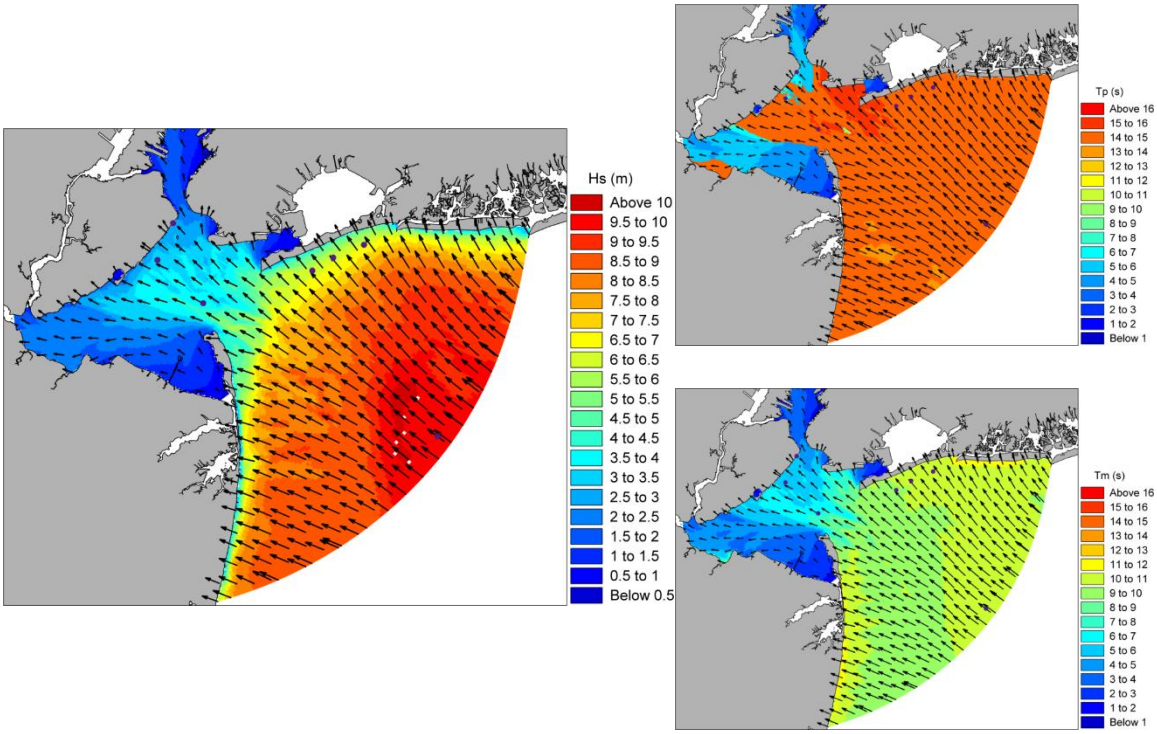


Figure 4-10: SWAN Model Results for Hurricane Sandy (2013-10-30 00:00 UTC)

5.0 STORM-INDUCED MODELING (SBEACH)

5.1 SBEACH Overview

Storm induced profile changes were investigated for the project area using the Storm-Induced Beach Chance Model (SBEACH). SBEACH is a one-dimensional model, developed by the United States Army Corps of Engineers (USACE), which simulates cross-shore erosion of beaches, berms, and dunes under storm water levels and waves. A basic assumption of SBEACH is that all profile change is produced by cross-shore processes, with no net gain or loss of sediment. This is only true if longshore sediment transport processes are uniform, which is typical considered a reasonable assumption during storm events on open coasts away from inlets and structures. Long-term morphologic changes (e.g. shoreline erosion) are typically controlled by longshore sediment processes, which are not simulated by SBEACH. The SBEACH model calculates beach profile change using an empirical morphologic approach with emphasis on beach and dune erosion. In model simulations, the beach profile progresses to an equilibrium state based on the initial profile, median grain size, and storm conditions (wave height, wave period, wave condition, wind speed and direction, and water level). The model also simulates overwash and dune lowering.

SBEACH is primarily used in this study to build the Beach-fx Storm Response Database (SRD). The SRD is a lookup table that stores the morphological profile responses (i.e. change in berm width and dune width/height) and damage driving parameters (i.e. wave height, water level, and vertical erosion). The SRD is based on approximately half a million SBEACH simulations for a range or possible beach profile configurations and storm conditions.

SBEACH modeling results are also used in combination with the multivariate Empirical Simulation Technique (EST) to screen potential beach and seawall design alternatives and characterize their performance.

5.2 SBEACH Model Calibration

Calibration of the SBEACH model was performed for East Rockaway using available profile storm response data. Model calibration was performed to determine the optimal model settings for the project region and evaluate the models ability to reproduce observed profile changes following a storm event. Model settings obtained through the calibration will be used in both existing and project conditions model runs. Since the intended purpose of the SBEACH modeling is to create a storm response database for Beach-fx, which simulates the morphological evolution of the dry beach, emphasis during the calibration was placed on simulating observed changes in the dry beach.

The only available set of beach profiles on East Rockaway measured closely before and after a major storm event is the Hurricane Belle and Hurricane Emmy set, obtained in June 1976 and September 1-4, 1976. In-between these surveys two major Hurricanes occurred causing significant



beach erosion. Therefore, calibration and verification of the SBEACH model parameters were made to this data set.

5.2.1 Available Data

Required input data for calibration of SBEACH include pre- and post-storm beach profiles, time histories of wave height, wave period, wave direction, and water levels. The only data set that fulfills these requirements is the Hurricane Belle and Emmy data set. At its peak strength Hurricane Belle was a category 3 hurricane with 120 mph winds. On August 10th, 1976 Hurricane Belle made landfall near Jones Beach, NY as a category 1 hurricane with winds of 75 mph. Figure 5-1 shows the path and storm history of Hurricane Belle. Storm surges along Rockaway Beach have been modeled to be in excess of 3 ft NGVD29 with nearshore wave heights peaking at 15 ft (USACE-NAN, 2005). In contrast to Hurricane Belle, Hurricane Emmy never came to the Atlantic shoreline, and as a result did not produce any significant storm surge at Rockaway. However, very large long period waves persisted at Rockaway from August 29th through August 31st. WISWAVE hindcast indicate wave heights in excess of 15 ft with wave periods of 17 s occurred offshore of Rockaway during Hurricane Emmy (Jensen, 1983).

Previous modeling efforts completed as part of FIMP were used to provide the storm inputs during Hurricane Belle. However, the FIMP model simulations were only carried out for the duration of the Hurricane Belle and had to be supplemented with other modeling studies to create the storm inputs for Hurricane Emmy. Since beach profile response is controlled by storm events, SBEACH was calibrated to the two storm events instead of simulating the entire time span between surveys.

Storm inputs at Rockaway Beach during Hurricane Belle were obtained from previous modeling efforts completed as part of FIMP study (USACE-NAN, 2005). Water level time series (including storm surge) were obtained from the ADCIRC modeling results. Time series of the water level at station 52, located just offshore of the center of Rockaway Beach, were extracted from the ADCIRC modeling results. Time series of the offshore significant wave height, wave period, and wave direction were obtained from WISWAVE model results at station I05J04.

Storm inputs for Hurricane Emmy were obtained from historical WISWAVE model simulations carried out by the USACE (Jensen, 1983) and a combination of tidal harmonics obtained from the FIMP ADCIRC modeling results at station 52 and the observed water surface elevations at NOAA tidal Station, Sandy Hook, NJ. Figure 5-2 shows the location of the ADCIRC station 52, WISWAVE station I05J04, WISWAVE station 73, and the NOAA tidal station at Sandy Hook which were used to generate the storm time series at Rockaway Beach.

The offshore wave height and wave direction for both the FIMP and historical WISWAVE model results were transformed from offshore to a nearshore water depth of 30 ft using linear wave theory and assuming straight and parallel contours. Figure 5-3 shows the wave conditions (i.e. significant wave height, wave period, and water level) at -30 ft (NGVD29) contour offshore of Rockaway



during the time between surveys. Figure 5-4 shows the storm inputs for Hurricane Belle and Hurricane Emmy used to calibrate SBEACH.

Pre and post storm beach profiles were measured along the center of Rockaway Beach in June of 1976 and August 31st to September 4th of 1976 respectively. Each of the pre and post profiles were surveyed at the same base station allowing easy comparison. Figure 5-5 shows the profile origins for the pre and post storm surveys. The comparison shows that there was a net increase in volume (above -25 ft NGVD29 contour) in all of the profiles. It is probable that the accretion between surveys is related to the large Federal Nourishment project that placed over 3.5 million cubic yards (MCY) of sand east of the calibration profiles (USACE-NAN records). During Hurricane Belle and Hurricane Emmy the waves were propagating from the E-SE resulting in a westward current and westward sediment transport along Rockaway Beach. Therefore, it is possible that a large portion of the beach fill placed east of the calibration profiles was transported westward into the calibration profiles during the storm. Figure 5-6 and Figure 5-7 show the pre and post profiles at a location exhibiting a large increase in volume (Profile 24) and a relatively small increase in volume (Profile 28) between pre and post surveys. The total volumetric increase, above -25 ft contour (NGVD29), between each pre and post profile is shown in Table 5-1. Volumetric increases ranged from 3.4% to 17% and in general the western profiles exhibited more accretion than the eastern profiles. Note that an increase in volume was observed at all the profiles.

The large volumetric increases between pre and post surveys indicate that longshore sediment transport processes played an important role in-between survey dates. Therefore, caution must be taken when attempting to calibrate and validate SBEACH which only simulates cross-shore sediment transport processes. Emphasis is placed here on simulating observed changes in the dry beach profile where cross-shore processes are dominant.

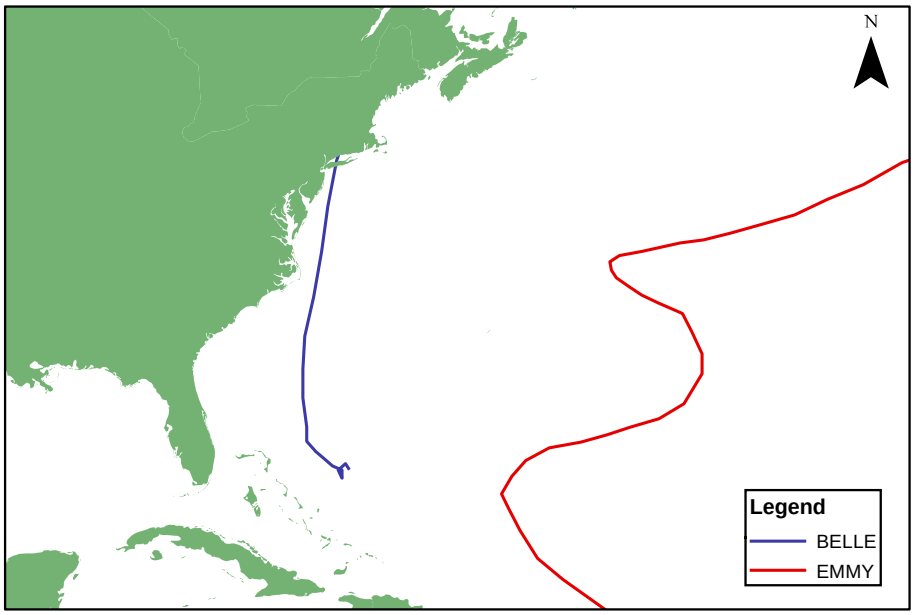


Figure 5-1: Hurricane Belle and Emmy Storm Tracks

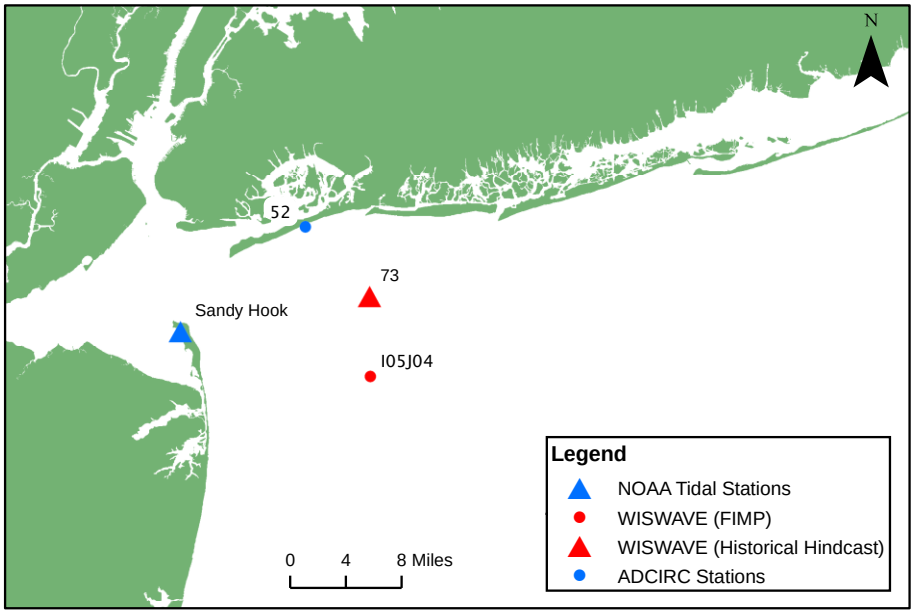


Figure 5-2: ADCIRC station 52, FIMP WISWAVE station I05J04, WISWAVE (Historical Hindcast) station 73, and the NOAA tidal station at Sandy Hook were used to generate the storm time series for Calibration at Rockaway Beach

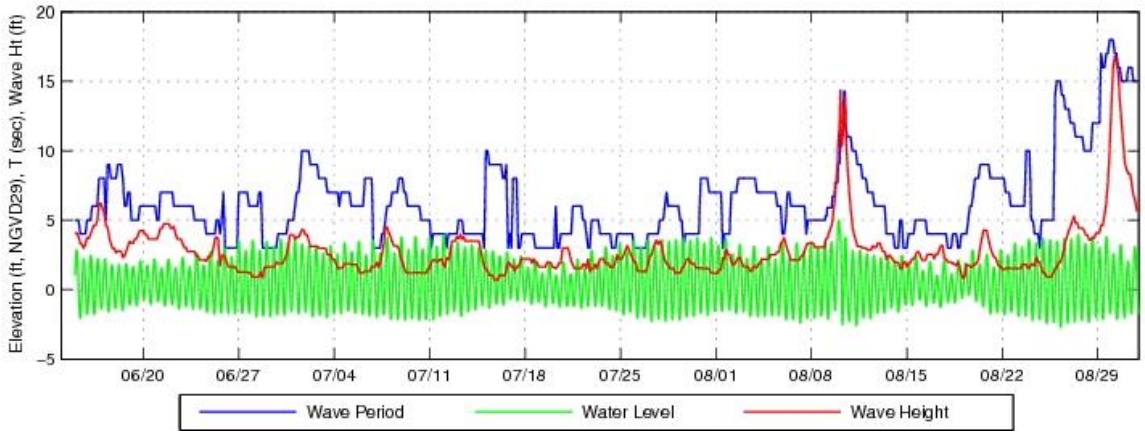


Figure 5-3: Observed Wave Conditions between Profile Surveys

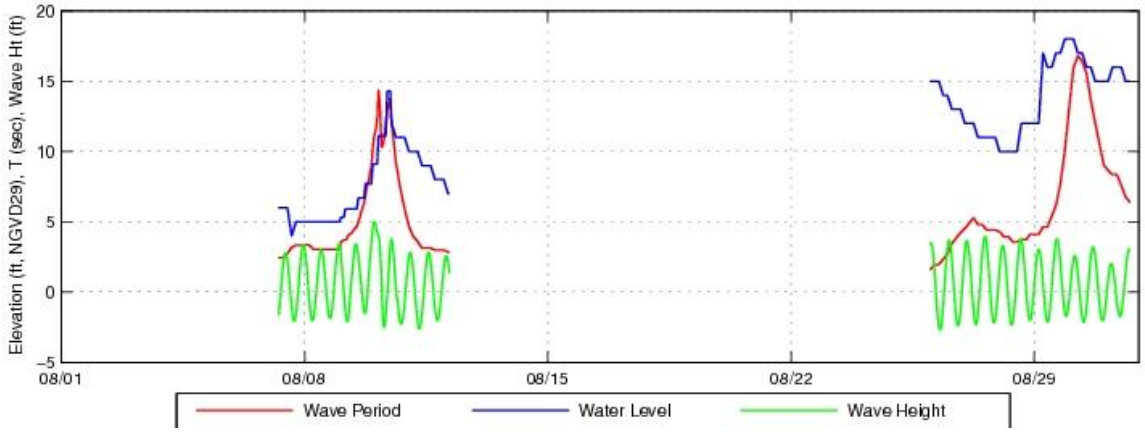


Figure 5-4: Modeled Wave Conditions for Calibration



Figure 5-5: Profile Origins for SBEACH Model Calibration

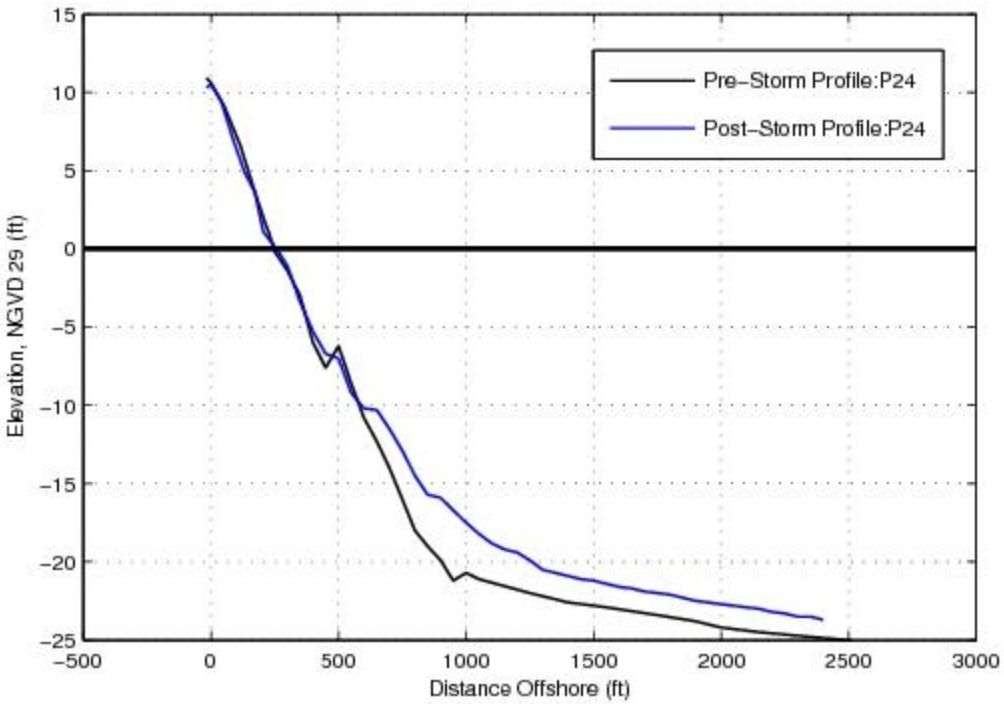


Figure 5-6: Pre and Post Storm Profile 24

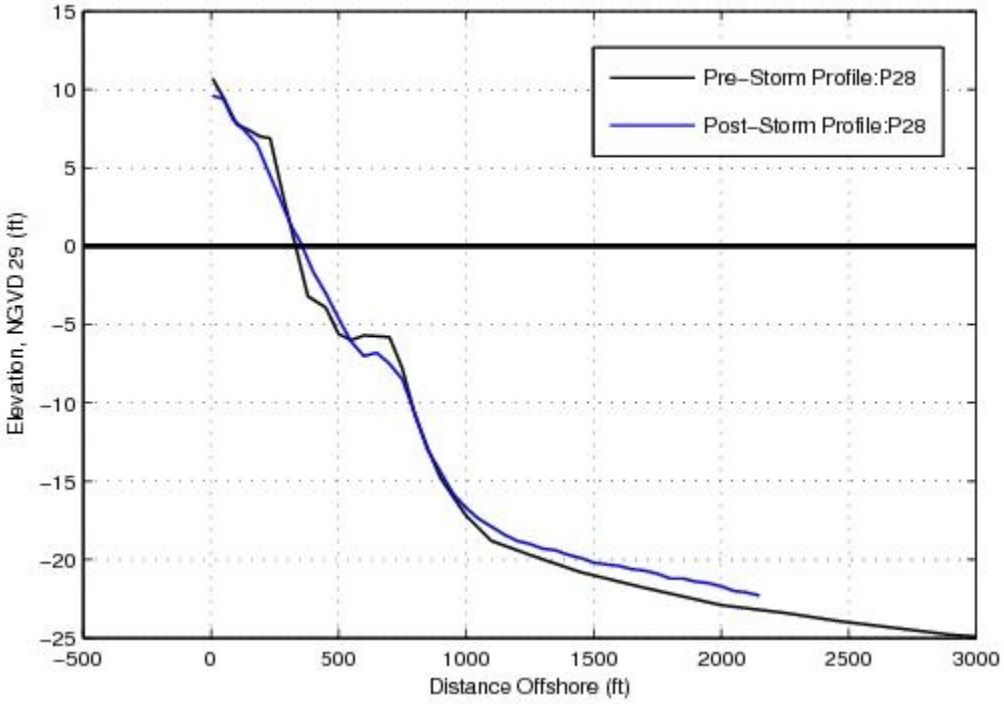


Figure 5-7: Pre and Post Storm Profile 28

Table 5-1: Differences between Pre and Post Storm Profiles

Profile	Pre Volume (CY/ft)	Post Volume (CY/ft)	Difference (CY/ft)	% Difference
P-19	876	973	97	11.1%
P-20-21	810	900	90	11.2%
P-21-22	846	905	58	6.9%
P-23	828	879	51	6.2%
P-24	762	891	129	17.0%
P-25	835	961	126	15.2%
P-26	800	889	89	11.2%
P-27	854	909	56	6.5%
P-28	946	978	32	3.4%
P-29	985	1078	94	9.5%
P-30	1045	1123	78	7.5%
P-31	826	879	53	6.4%

Note: All Post profiles experienced an increase in Volume. All volumes were measured from the (-25 ft NGVD29 contour

5.2.2 Calibration Procedure and Results

SBEACH calibration parameters include the sediment transport rate coefficient (K), coefficient for slope-dependent transport term (EPS) and depth of foreshore (DFS). The primary calibration parameter is K which controls the rate of erosion. The slope-dependent transport term controls the impact local gradients in the profile have on the transport rate. Higher values of EPS will result in a more peaked bar and a flatter inner surf zone. The depth of foreshore defines the transition between the surf zone and swash zone and influences the magnitude of transport occurring in the swash zone.

Other parameters that are site dependent and need to be specified in SBEACH are the mean sediment grain size (μ), maximum slope prior to avalanching (BMAX), and water temperature (T). The mean sediment grain size was determined to be 0.29 mm from sediment analysis performed by USACE-NAN. The typical water temperature in the summer is 22ºC, and the maximum slope prior to avalanching is set to 30º.

For each SBEACH calibration simulation a uniform grid with a grid spacing of 5 ft was setup across the profile and simulation time step of 1 minute is used. Simulations were performed initially for both the default SBEACH parameters and the previously determined calibration parameters for FIMP. An additional 2 calibration scenarios were run to try and achieve a better calibration. Table 5-2 shows the parameters used in the calibration scenarios. The Default and FIMP calibration scenarios are nearly the same, except a higher value of EPS in the FIMP



parameters. In the remaining calibration scenarios EPS and K were reduced to try and obtain better agreement between observed and modeled morphologic response. The calibration results at 5 profiles, Profile 20-21, Profile 24, Profile 27, Profile 28, and Profile 31 are analyzed here. The 5 profiles exhibited both large and small amounts of volumetric accretion between surveys.

Quantification of the model results were obtained by comparing three erosion parameters calculated for the dry profile. The three parameters selected are: volume change above 0 NGVD29, recession of the Mean-High-Water (MHW) contour, and storm intrusion defined by the distance from the profile origin to the landward most occurrence of a given amount of vertical profile change (1 ft). These are the same parameters used by Gravens et al. (1999) to evaluate the calibration results for FIMP. Table 5-3 presents the results from the quantitative analysis for all 5 profiles. Figure 5-8 to Figure 5-12 show the modeling results for the calibration scenarios.

It is apparent in Profile 20-21 (Figure 5-8) that SBEACH reproduces erosion of the dry profile and redistribution of the eroded sediment over the submerged profile. Table 5-3 shows that each calibration scenario for Profile 20-21 over predicts the volumetric erosion of the dry beach, landward storm intrusion, and recession of the MHW line. It is also apparent in Table 5-3 that reducing K and/or EPS result in less dry beach erosion. Comparison of the submerged portion of Profile 20-21 shows that the model is not reproducing the gentle slope in surfzone and the significant accretion observed across the entire profile. Since SBEACH only simulates cross shore processes and is incapable of simulating volumetric increases resulting for longshore processes. The observed gentler slope in the surfzone is characteristic of all the profiles and could be caused by the introduction of finer sediment into the submerged profile from erosion of the dry beach or longshore transport of the nourished sediment. However, the general shape and slope of the modeled post-storm submerged profile is similar to the observed post-storm profile and in both instances the sediment in the bar has been redistributed offshore. The results at Profile 20-21 are characteristic of the calibration, and it is possible that SBEACH is over predicting dry beach erosion because the model does not include longshore processes. Volumetric accretion, as observed in all the profiles causes more dissipation of wave energy offshore and leads to less erosion of the dry beach. Therefore, a slight over prediction in dry beach erosion is anticipated in the model.

In contrast to Profile 20-21, Profile 24 experienced very little volumetric dry beach erosion (Figure 5-9). The model over predicts the volumetric dry beach erosion, and recession of the MHW. However, it accurately predicts the limit of landward storm intrusion.

The model results for Profiles 27, 28, and 31 are similar to Profile 20-21 and Profile 24, where the volumetric dry beach erosion and recession of the MHW are over predicted (Figure 5-10 to Figure 5-12). In all 3 profiles SBEACH accurately simulates erosion of the dry beach and redistributes the sediment in the bar offshore.

Generally the model produces a steeper dry beach, and greater erosion. The over estimation of dry beach erosion in the model is attributed to longshore processes not modeled in SBEACH. Of the four calibration scenarios, scenario 4 produced the best results. Scenario 4 still over estimated dry beach erosion but by reducing K and EPS the gap between the observed and modeled erosion is reduced. Therefore, the model parameters in Calibration 4 will be used in both existing and project conditions model runs. The level of calibration obtained in this study using is comparable to the FIMP calibration (Gravens et al., 1999).

Table 5-2: SBEACH Calibration Scenarios

Scenario	K (m^4/N)	EPS (m^2/s)	DFS (ft)	Grain Size(mm)	T (°C)	BMAX (°)
Cal-1	1.75E-06	0.003	0.91	0.29	22	30
Cal-2	1.75E-06	0.002	0.91	0.29	22	30
Cal-3	1.75E-06	0.0015	0.91	0.29	22	30
Cal-4	1.00E-06	0.0015	0.91	0.29	22	30

Notes: Cal-1 uses the FIMP calibration values for K and EPS. Cal-2 uses the default SBEACH calibration values for K and EPS

Table 5-3: SBEACH Erosion Parameters for Profile 20-21

Erosion Parameter	Observed	Cal-1	Cal-2	Cal-3	Cal-4
Profile 20-21					
Volume Change above 0 NGVD29 (CY/ft)	-266	-557	-481	-437	-352
Recession of MHW (ft)	-13	-56	-48	-42	-34
Limit of Landward Storm Intrusion (ft)	47	54	60	65	75
Profile 24					
Volume Change above 0 NGVD29 (CY/ft)	-12	-52	-43	-38	-29
Recession of MHW (ft)	-3	-53	-45	-40	-32
Limit of Landward Storm Intrusion (ft)	79	59	67	72	81
Profile 27					
Volume Change above 0 NGVD29 (CY/ft)	-16	-36	-29	-26	-23
Recession of MHW (ft)	-14	-52	-45	-41	-38
Limit of Landward Storm Intrusion (ft)	1	15	1	1	1
Profile 28					
Volume Change above 0 NGVD29 (CY/ft)	-23	-40	-34	-30	-27
Recession of MHW (ft)	-17	-46	-39	-35	-31
Limit of Landward Storm Intrusion (ft)	168	149	161	168	172
Profile 31					

Volume Change above 0 NGVD29 (CY/ft)	-22	-50	-42	-38	-30
Recession of MHW (ft)	-12	-42	-34	-30	-24
Limit of Landward Storm Intrusion (ft)	131	137	142	145	151

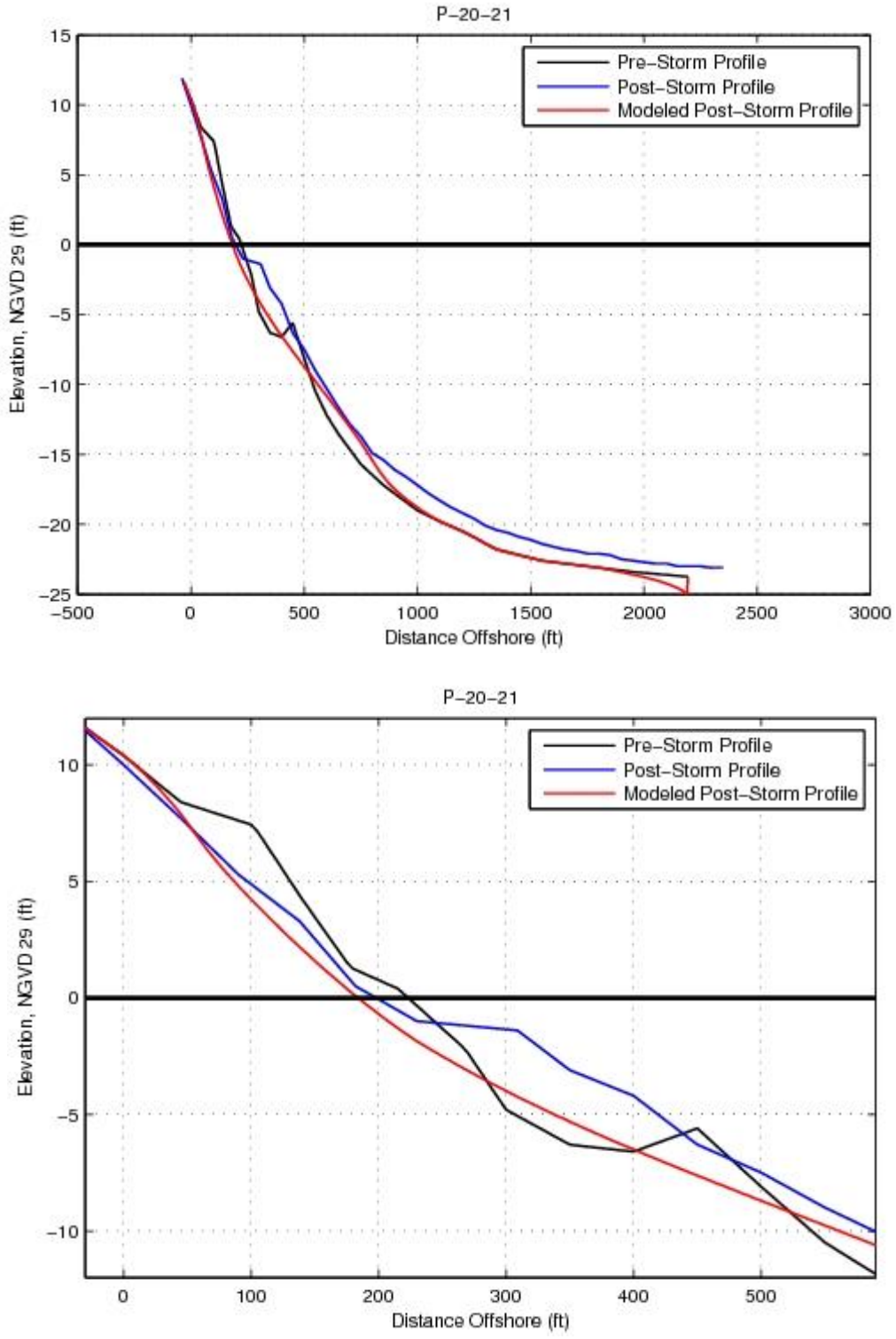


Figure 5-8: SBEACH Calibration results at Profile 20-21

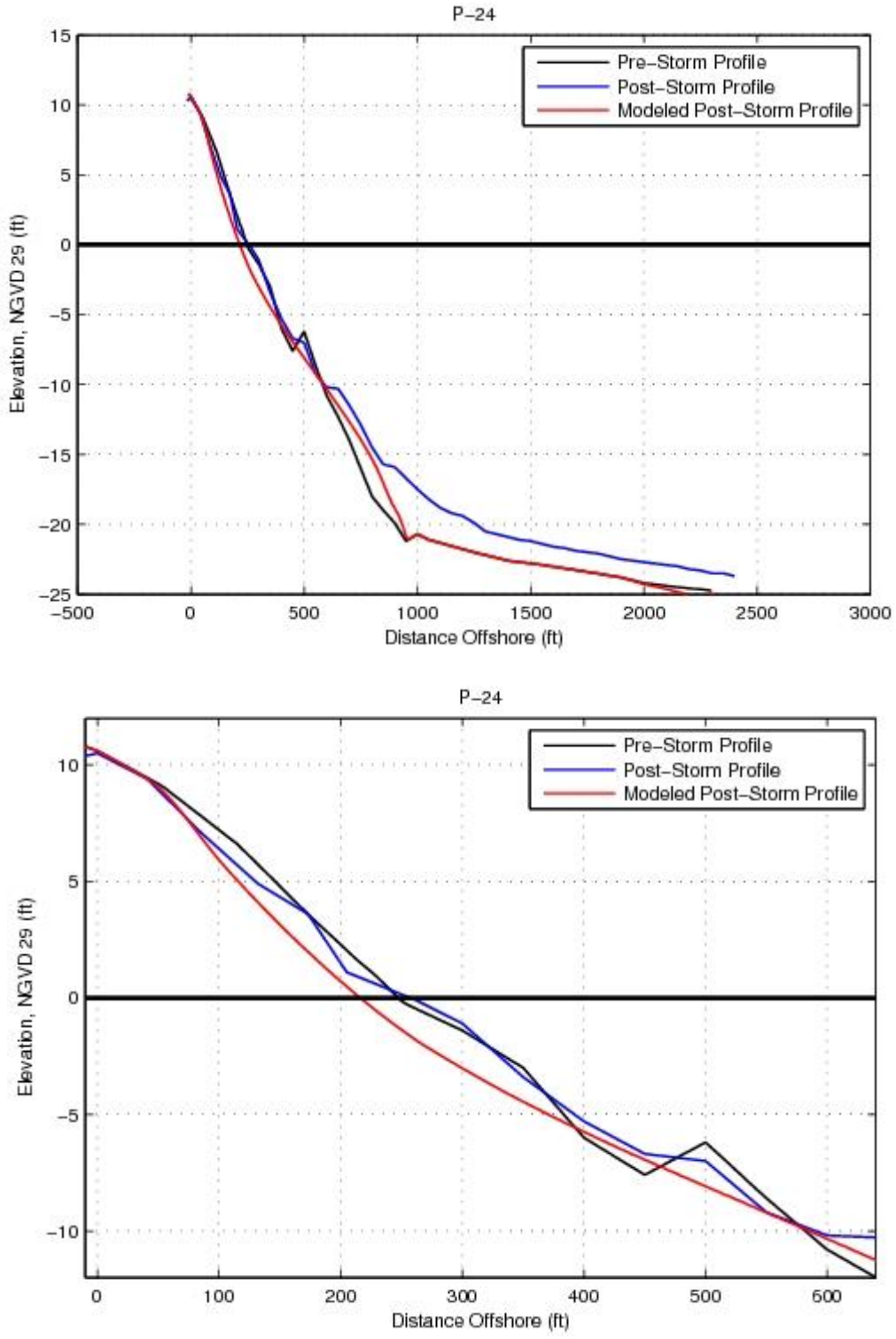


Figure 5-9: SBEACH Calibration results at Profile 24

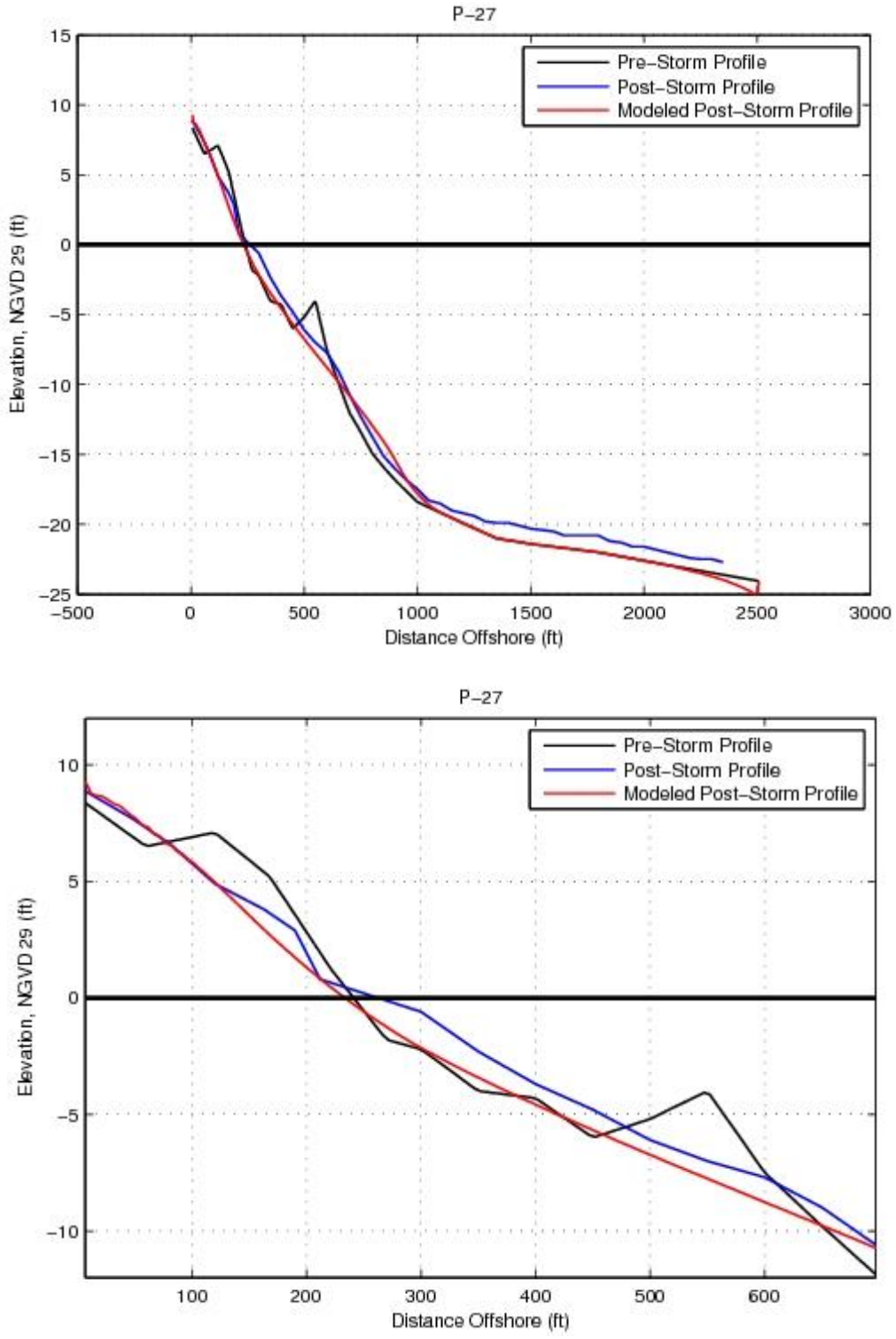


Figure 5-10: SBEACH Calibration results at Profile 27

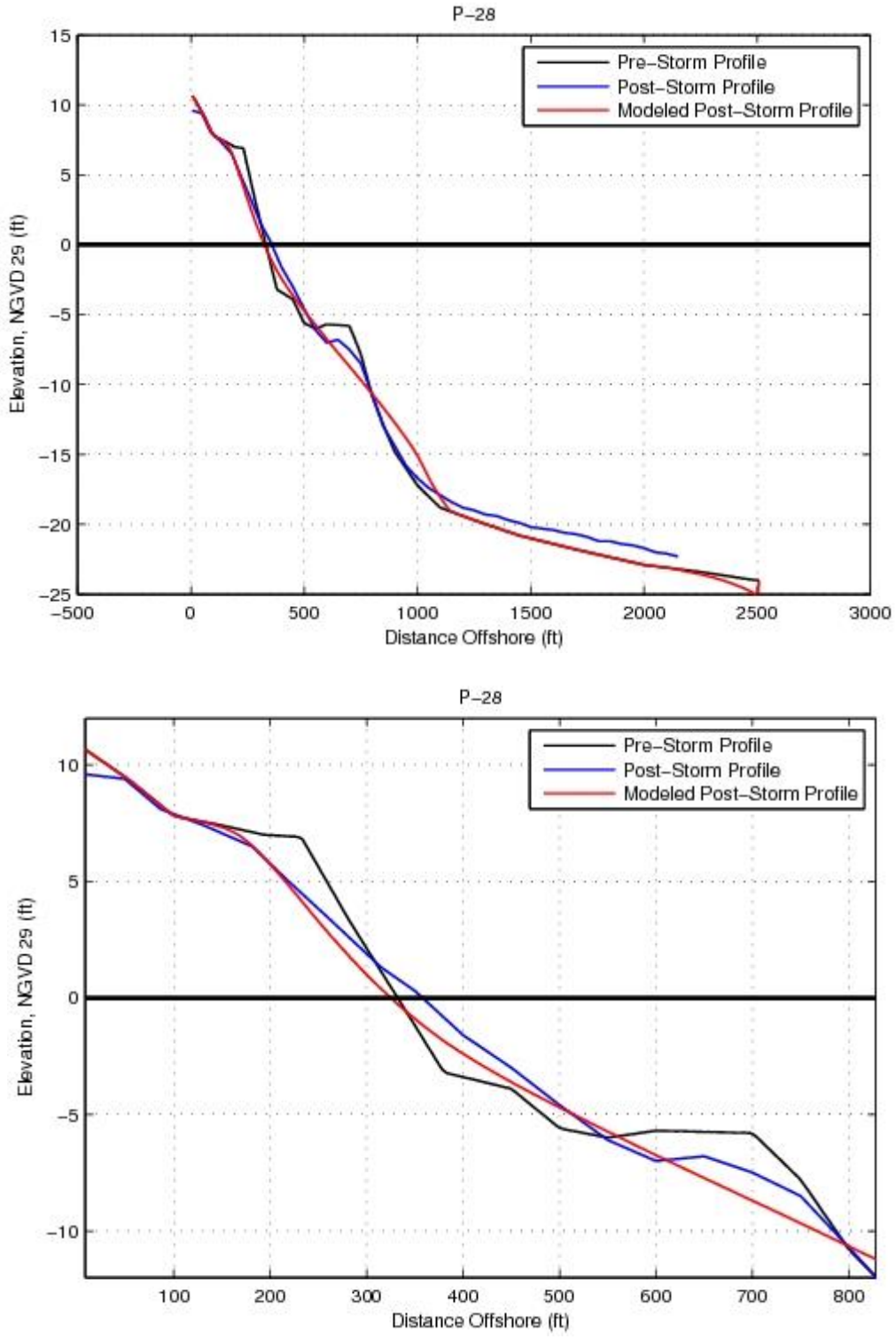


Figure 5-11: SBEACH Calibration Results at Profile 28

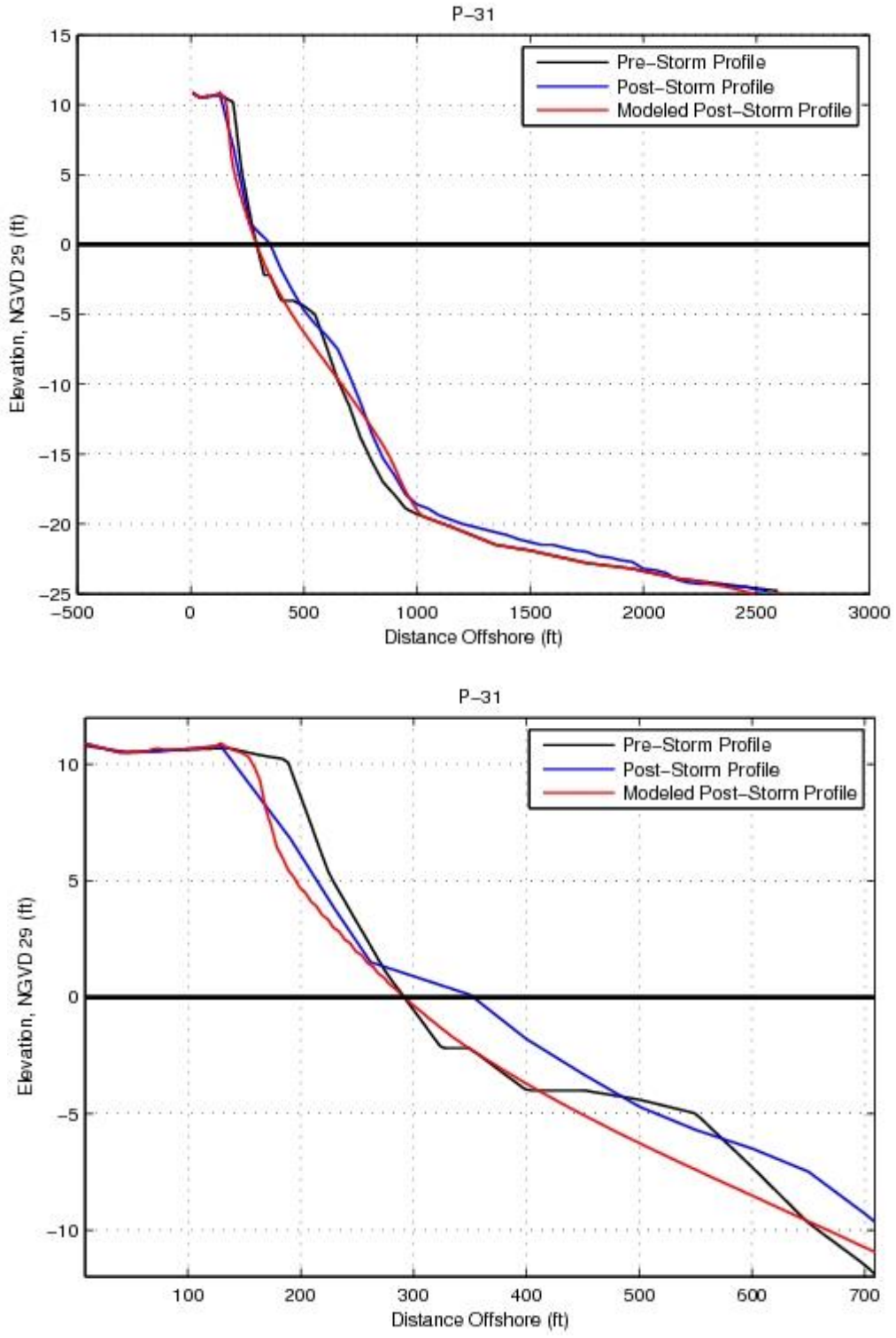


Figure 5-12: SBEACH Calibration results at Profile 31

5.2.3 SBEACH Model Sensitivity

5.2.3.1. Sensitivity to Calibration and Site Specific Parameters

A sensitivity analysis of SBEACH to the main calibration parameters and site specific parameters was performed to gain a better understanding of the influence of each parameter in the model. Specifically, the sensitivity of SBEACH results to the sediment transport rate (K), coefficient for slope-dependent transport (EPS), sediment grain size, and wave height was tested. For each of the sensitivity runs, the FIMP calibration parameters were used and four variations of sensitivity parameter were tested. Table 5-4 shows the values for each of varied parameters in the sensitivity tests. All sensitivity results are shown for Profile 28 because it displayed good agreement between the measured and SBEACH results.

Table 5-4: SBEACH Sensitivity Analyses

Sensitivity Parameter	Value 1	Value 2	Value 3	Value 4
K (m^4/N)	1.75E-06	2.50E-06	1.00E-06	2.50E-07
EPS (m^2/s)	0.001	0.002	0.003	0.004
μ (mm)	0.44	0.36	0.28	0.2
Wave Height Multiplier	0.9	1	1.1	2
Water Level Multiplier	0.9	1	1.1	2

The sensitivity of SBEACH to the sediment transport rate coefficient is shown in Figure 5-13. As discussed earlier, the impact of K is to either speed up or slow down the rate of erosion. In the instance where K is too low (pink line in Figure 5-13) SBEACH under predicts the dry beach erosion. It is also noted that the model is much more sensitive to K than any of the other calibration parameters. The sensitivity of SBEACH to the coefficient for slope-dependent transport is shown in Figure 5-14. Lowering EPS clearly decreases the amount of erosion along the steep dry beach. This is expected as EPS increases the rate of erosion along steep parts of the profile.

The sensitivity of SBEACH to the mean grain size is shown in Figure 5-15. As expected a larger grain size results in less erosion of the dry beach and a steeper profile. Figure 5-15 also indicates that SBEACH is relatively sensitive to the grain size as a 0.08 mm difference in the grain size results in a dry beach retreat difference of 10 ft. The sensitivity of SBEACH to the input wave height is shown in Figure 5-16. Relative to the impact of the sediment grain size, the sensitivity of SBEACH to a 10% increase in the wave height is very small. The sensitivity of the profile response to the input wave height is largest at the offshore bar were the waves are breaking. The sensitivity of SBEACH to the input water level is shown in Figure 5-17. Relative to the impact of the sediment grain size, the sensitivity of SBEACH to a 10% increase in the water level is very small. A noticeable difference exists between the sensitivity of the dry beach to a large increase in wave



height versus a large increase in the water level. Significantly more dry beach erosion occurs if the water level is doubled than if the wave height is doubled. However, more erosion of the submerged profile occurs when the wave height is doubled. This sensitivity exercise suggests that dry beach erosion is more sensitive to an increase in water level, whereas submerged profile erosion is more sensitive to an increase in the wave height.

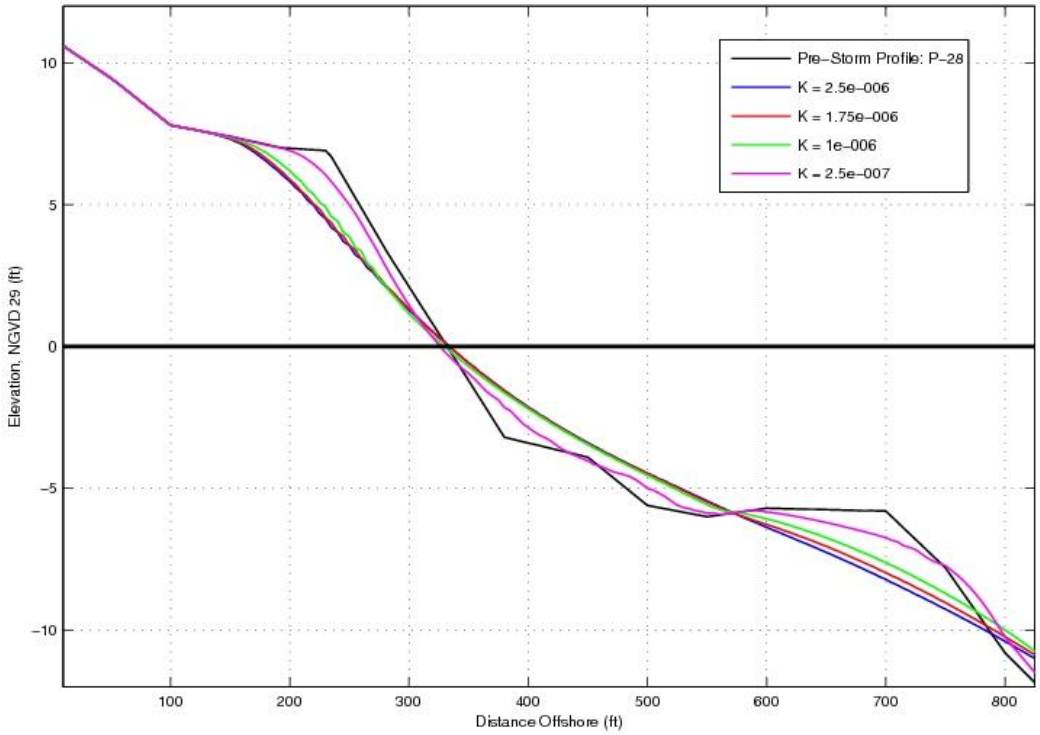


Figure 5-13: SBEACH sensitivity to sediment transport rate coefficient (K)

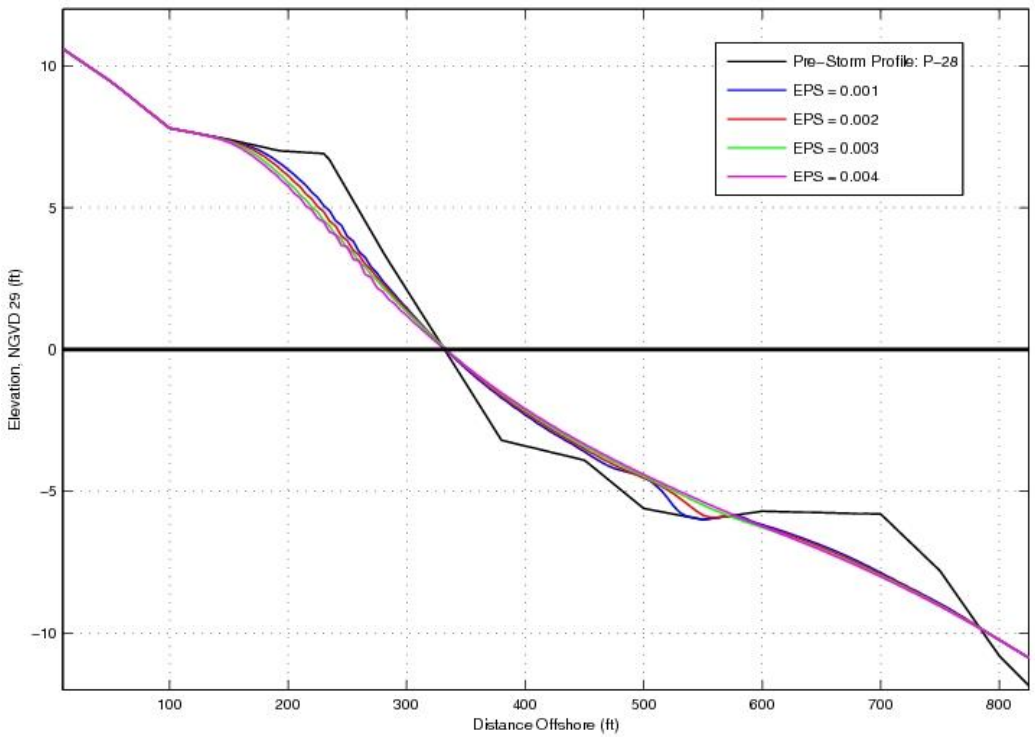


Figure 5-14: SBEACH sensitivity to coefficient for slope-dependent transport (EPS)

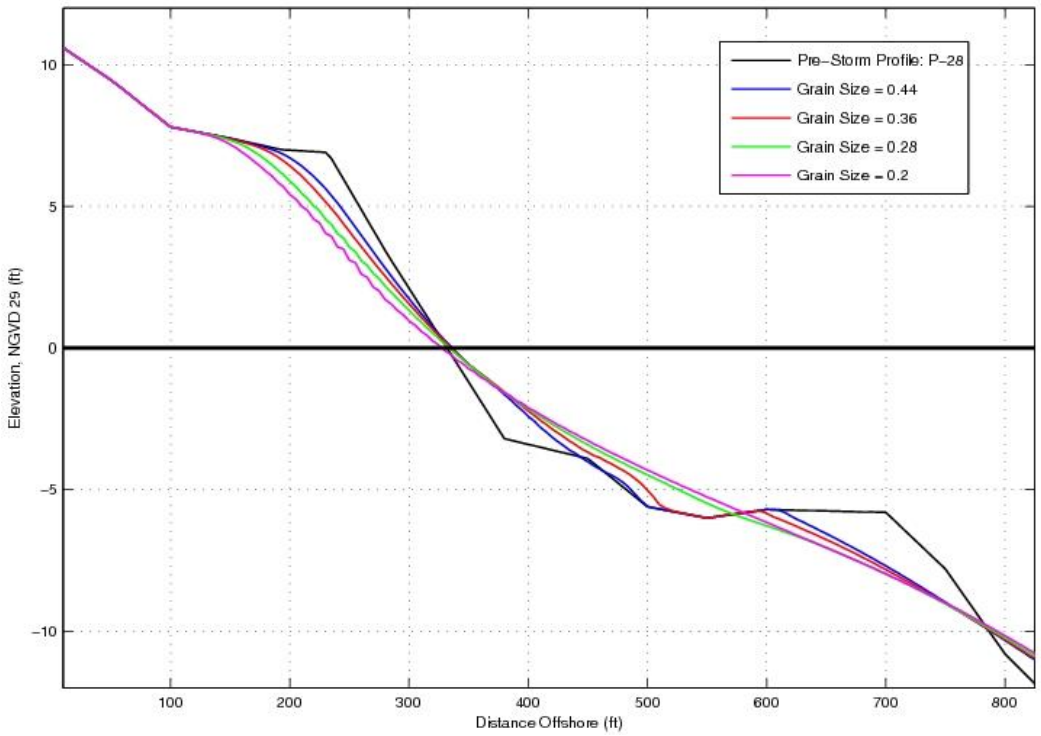


Figure 5-15: SBEACH sensitivity to mean sediment grain size

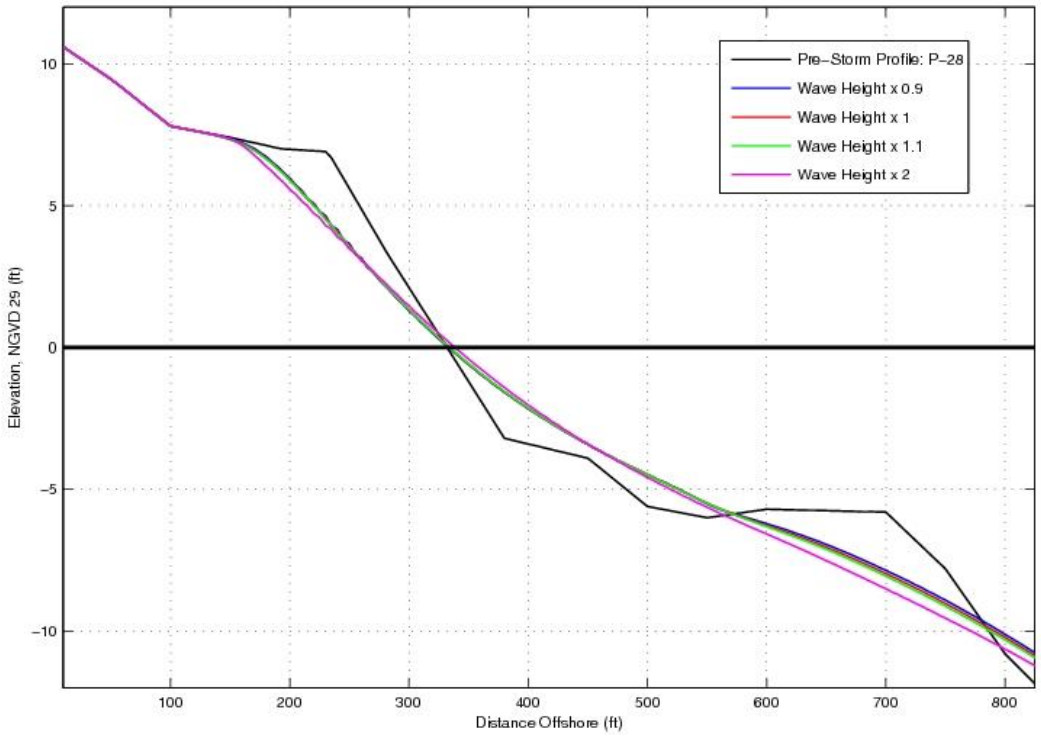


Figure 5-16: SBEACH sensitivity to magnitude of the significant wave height (Hs)

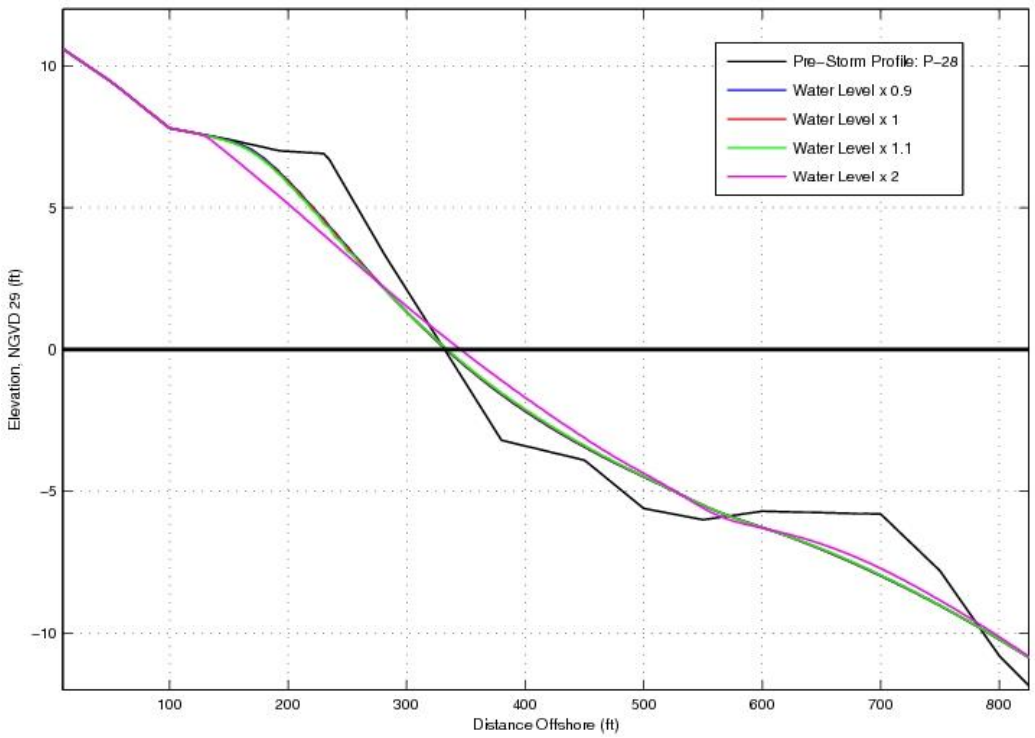


Figure 5-17: SBEACH sensitivity to magnitude of the water level

5.2.3.2. Sensitivity to Model Grid Size and Time Step

During existing conditions model simulations it was observed that dune response to large storms was sensitive to the model grid size and time step. In instances where dune overtopping and erosion occurs the profile response can be very unstable and appear jagged if long time steps and/or small grid cells are used. However, if shorter time steps and/or larger grid cells (in the dry beach) are used the dune response is smoother and more stable. The response of the submerged profile is insensitive to small changes in the grid resolution and time step. Additionally, an increase in the grid size in the submerged profile has minimal impact on the response of the upper profile.

A small sensitivity test was conducted to highlight the observations discussed above. The sensitivity test simulated one profile using 3 different grid cell sizes (5 ft, 10 ft, and 15 ft), and a single storm (October 1954 Hurricane) using 3 different time steps (1 min, 2 min, and 3 min). Table 5-5 shows the 3 profile configurations, 3 storm configurations, and whether the profile response visually appears to be stable or unstable. Figure 5-18 presents the results of profile with 10 ft grid spacing and all the 3 time steps. It is apparent in Figure 5-18 that both the 2 min and 3 min time steps results in jagged and unstable model results. Figure 5-19 presents the results of all 3 grid cell sizes and the most stable time step (1 min). Figure 5-19 indicates that larger grid cells in the dry beach are more stable.

The results of the sensitivity analysis confirm the observations from the existing conditions simulations; small grid sizes and large time steps can cause unstable model results. This analysis was used to determine appropriate grid cell sizes and time steps for the existing conditions simulations.

Table 5-5: SBEACH Sensitivity to Time Step and Grid Spacing

Grid Spacing	Time Step		
	3 min	2 min	1 min
5 ft	Unstable	Unstable	Medium
10 ft	Unstable	Unstable	Stable
15 ft	Medium	Stable	Stable

5.2.4 Calibration Conclusions for Hurricanes Bell and Emmy

SBEACH calibration and sensitivity tests were performed for a set of profiles measured before and after Hurricane Belle and Hurricane Emmy. This was the only available data set with measurements taken before and after a major storm event. Unfortunately significant volumetric accretion occurred in the majority of the profiles between measurements. It is believed that accretion was the result of longshore processes and the placement of over 3.5 million cubic yards of sediment in Rockaway Beach, just east of the surveyed profiles. However, calibration to the data set was still possible and the model was able to reproduce erosion of the dry beach. The calibration parameters that provided the best results for Rockaway Beach was scenario Cal-4 (Table 5-2). Comparison between the observed and modeled volumetric dry beach erosion, recession of the MHW, and landward storm intrusion yielded acceptable results and are comparable to the level of calibration obtained previously for FIMP (Gravens et al., 1999).

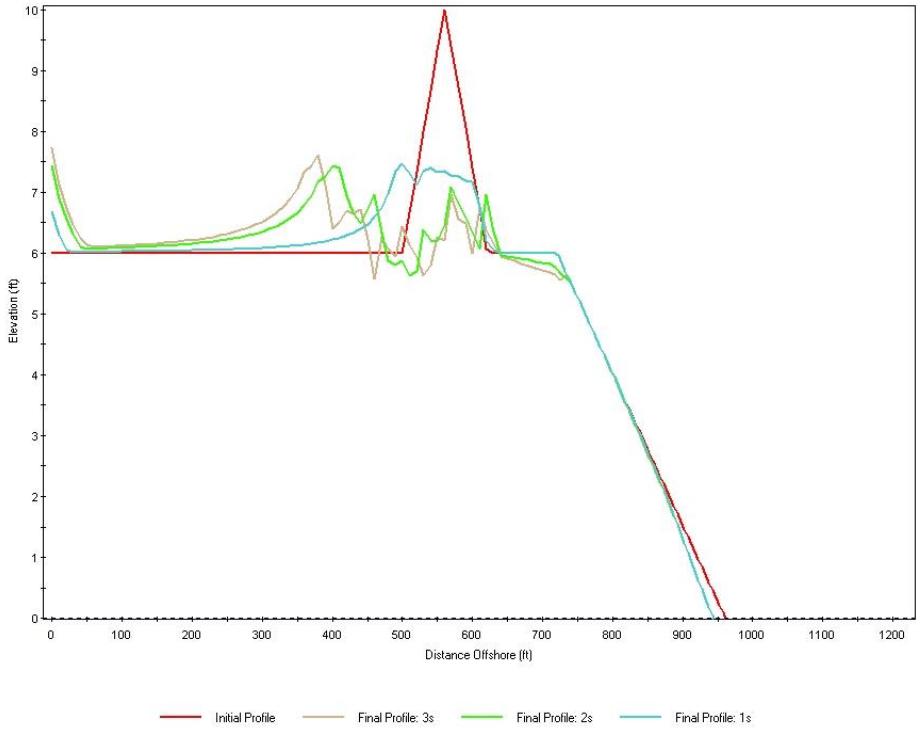


Figure 5-18: SBEACH sensitivity to model time step (grid spacing 10 ft)

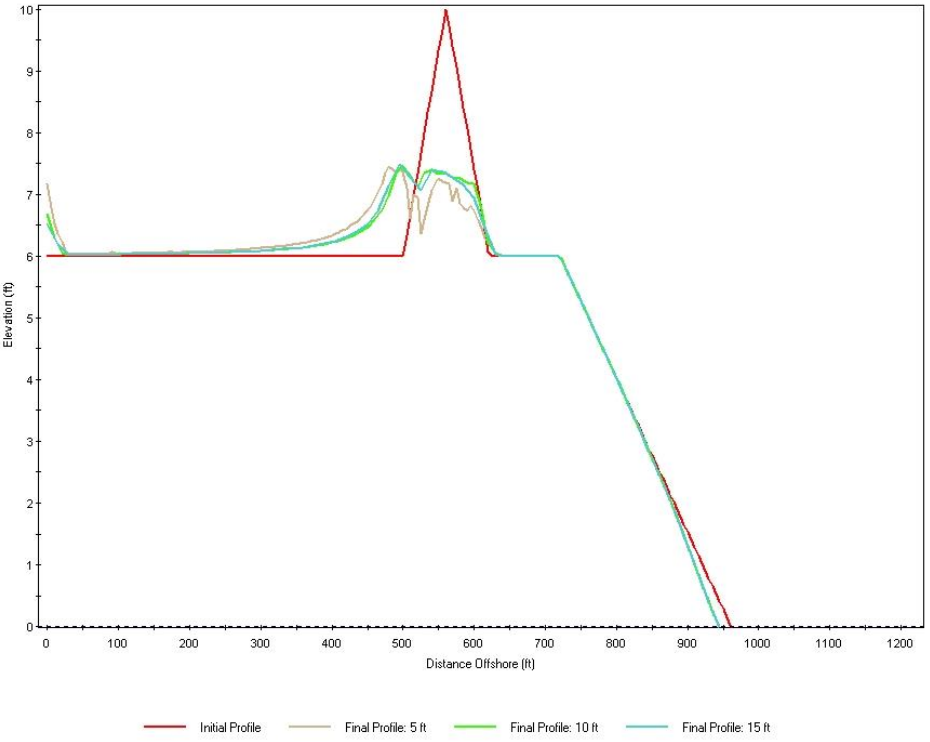


Figure 5-19: SBEACH sensitivity to model grid size (time step of 1 min)

5.2.5 Updated Model Calibration for Overwash Parameter

This appendix documents the calibration of the overwash transport parameter (K_B) in SBEACH. Overwash is defined as "the flow of water over the crest of the beach or dune that does not directly return to the originating water body" (Wamsley and Kraus 2004). Washover is the sediment deposited landward as a result of overwash.

The need to perform this calibration arose from the preliminary "with-project" simulation results where unexpected dune overwash was observed using the default K_B value of $5e^{-3}$. The issue was encountered when simulating profiles with low-crested dunes (relative to the upland elevation) under medium-sized storms that generated run-up induced overwash (i.e., the still water elevation did not overtop the dune). The top plot in Figure 5-20 exemplifies this scenario, showing the formation of a large overwash deposit landward of the pre-storm, 11 ft dune. The bottom plot of this figure shows an example of "inundation overwash" in which the dune becomes completely submerged. The geometry of the overwash deposit under "inundation overwash" is much flatter and broader. In instances where overwash does not occur, then the model results are not affected by K_B.

In view of these results, a reduction in the default value of K_B was deemed appropriate to reduce the excessive overwash. Figure 5-21 shows the results obtained for a range of K_B values equal to and smaller than the default value. The top plot illustrates that, under a medium-sized storm, the reduction in K_B translates into a significant reduction in the amount of dune overwash and dune retreat. Under a storm of Hurricane Sandy's intensity, the bottom plot shows a similar dune response, with the higher K_B values spreading more sediment landward. Note that in both plots the beach erosion is practically the same for all values of K_B.

Similar to Figure 5-21, Figure 5-22 illustrates the results obtained for a range of K_B values on a 20 ft dune under Hurricane Sandy. While consistent with the previous observations, it is evident in this figure that a significant reduction in K_B may lead to severe underestimation of dune erosion, as seen by the remaining dune "trunk" for the lowest tested K_B value. This figure also shows that changes in K_B have very little impact on berm erosion.

Generalizing, the overwash sensitivity analysis indicates that two unfavorable situations may arise if the default value or a very small value for K_B is chosen:

1. If the default value is used, the overwash deposit created by runup-induced overwash may form a dune of significant size that would reduce damages in future storm events during Beach-fx simulations.
2. If K_B is reduced, a flatter overwash deposit is obtained; however, a very small value may severely under-predict dune erosion, offering undesired protection to the upland area in future storm events during Beach-fx simulations.



In this light, a K_B value of $1e^{-3}$ was deemed to be a reasonable middle ground between these constraints. Personal communication with Mark Gravens (USACE CHL) suggested that this value is within a suitable range.

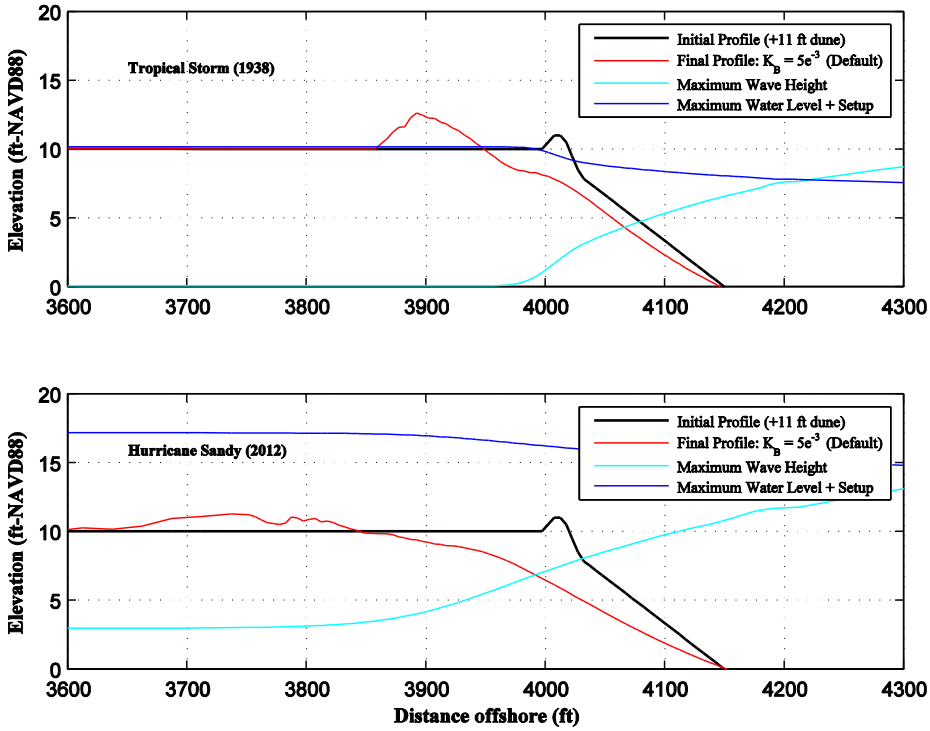


Figure 5-20: SBEACH dune overwash with default overwash transport parameter

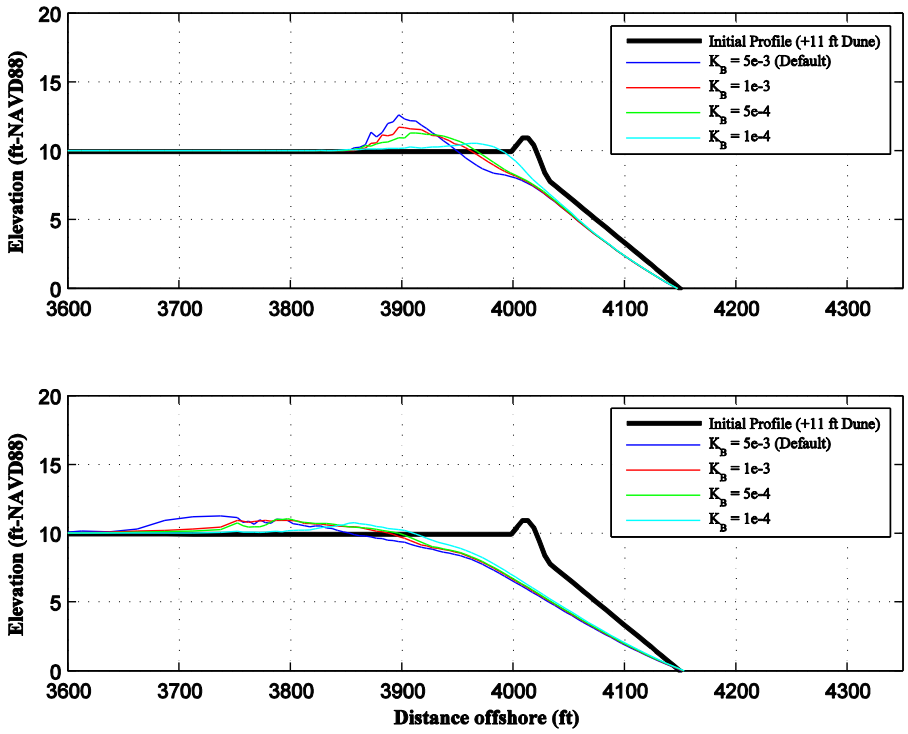


Figure 5-21: SBEACH dune overwash sensitivity to overwash transport parameter (K_B)

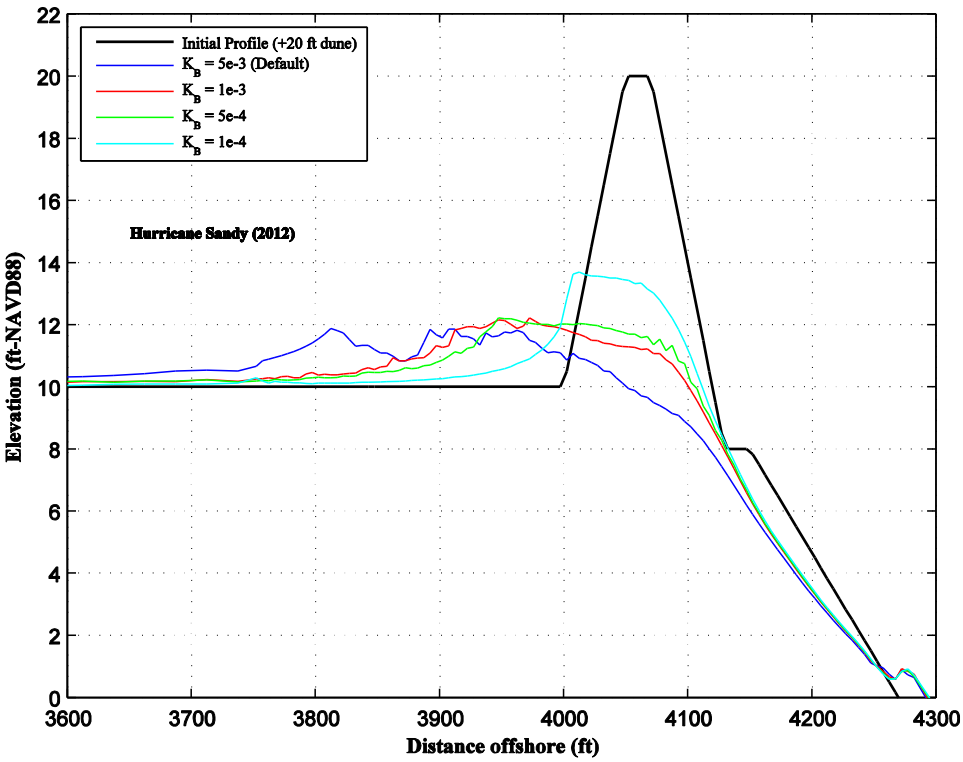


Figure 5-22: SBEACH dune erosion sensitivity to overwash transport parameter (KB)

In a previous Task Order, SBEACH was calibrated to observed morphological changes at Long Beach, NY, following Hurricane Belle and Hurricane Emmy (1976). This calibration defined the SBEACH sediment transport and beach parameters used in the present Task Order. In order to ensure that changes in K_B would not affect the other parameters, the original calibration was revisited. The results indicated that K_B was independent of the other parameters and no further adjustments were required.

Subsequently, the Hurricane Sandy validation was revisited, with emphasis on Reach 1 where most of the dunes in Rockaway are present. Some limitations were present from the start as this reach is absent of post-Irene profile data and the available (2010) profile data is, in general, considerably less eroded than the post-Sandy profile data. Figure 5-23 show the results for profiles 3 (Reach 1) and 73 (Reach 5) for the default and proposed K_B values. The latter profile is the only profile with a dune outside Reach 1.

As observed in these figures, it is difficult to determine whether the proposed K_B value can be justified based on the Hurricane Sandy validation results. Nonetheless, the results are consistent with the previous sensitivity analysis, i.e.: the reduction in the K_B value tends to underestimate dune erosion and reduce the landward spreading of sediment.

Based on the sensitivity analysis and calibration runs, M&N proposes to use a K_B value of $1e^{-3}$. Table 5-6 provides a summary of the parameters that have been adopted for the SBEACH production runs.

Table 5-6: SBEACH Final Model Calibration Parameters

SBEACH Parameter	Value
Landward surf zone depth (ft)	0.91
Effective grain size (mm)	0.29
Maximum slope prior to avalanching (deg)	30
Transport rate coefficient (m^4/N)	$1e^{-6}$
Overwash transport parameter (K_B)	$1e^{-3}$
Coefficient for slope-dependent term (m^2/S)	$1.5e^{-3}$
Transport rate decay coefficient multiplier	0.5
Water temperature (°C)	13

The sensitivity analysis of K_B leads to the following conclusions:

- For profiles with low-crested dunes relative to the upland elevation, the default K_B value can induce the formation of an unrealistically large washover deposit landward of the pre-storm dune as a result of runup-induced overwash.
- The reduction of K_B from its default value limits the excessive landward washover. However, if K_B is significantly reduced, the model tends to underestimate dune erosion.
- A good compromise between avoiding excessive landward washover and underestimating dune erosion is to use a K_B value of 1e-3. This value has been used by other SBEACH experts (Mark Gravens, personal communication).
- Berm and beach erosion were found to be unaffected by changes in K_B.
- K_B is independent of the other sediment transport and beach parameters; therefore, reducing its value does not affect the original calibration.

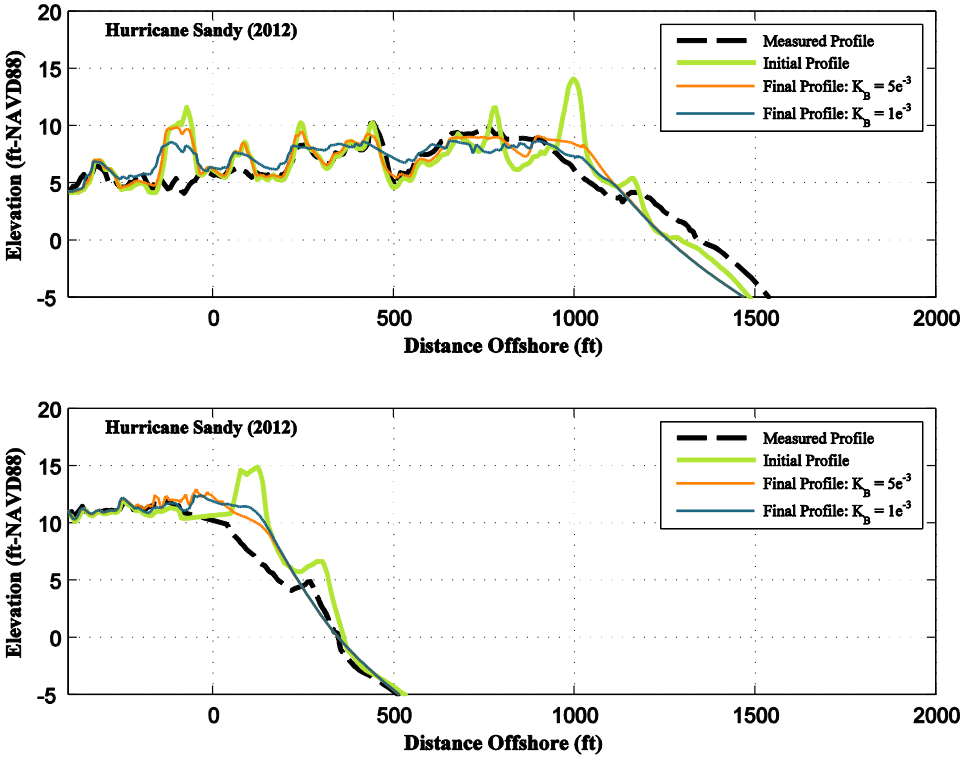


Figure 5-23: SBEACH Hurricane Sandy validation for Profile 03 and Profile 73

5.2.6 Hurricane Sandy Model Validation

SBEACH model validation was performed to observed subaerial beach profile changes during Hurricane Sandy. Unfortunately long-profile surveys with data below MHW are sparse and the majority of the observations are limited to subaerial beach changes. In addition, the time period between the pre-storm profiles and actually occurrence of Hurricane Sandy is 1 to 2 years. Therefore, the observed profile changes represent a much longer period of time than just Hurricane Sandy. Nonetheless, the model validation provides an useful and reassuring comparison of the predicted profile changes by SBEACH and observed post-storm conditions.

Pre-storm beach profiles were constructed using 2011 surveyed profiles and 2010 LIDAR data at locations without a 2011 survey. The post-sandy subaerial beach conditions are based on a 2012 LIDAR survey. The submerged profile for all pre- and post-storm profiles is based on the representative submerged profile (Section 5.3.1). The final SBEACH calibration parameters shown in Table 5-6 are used in the model validation. Figure 5-24 shows the profile locations (in yellow) used in the Hurricane Sandy SBEACH model validation.

Model boundary conditions for Hurricane Sandy are based on the conditions in the Historical Storm Suite (Section 2.1).

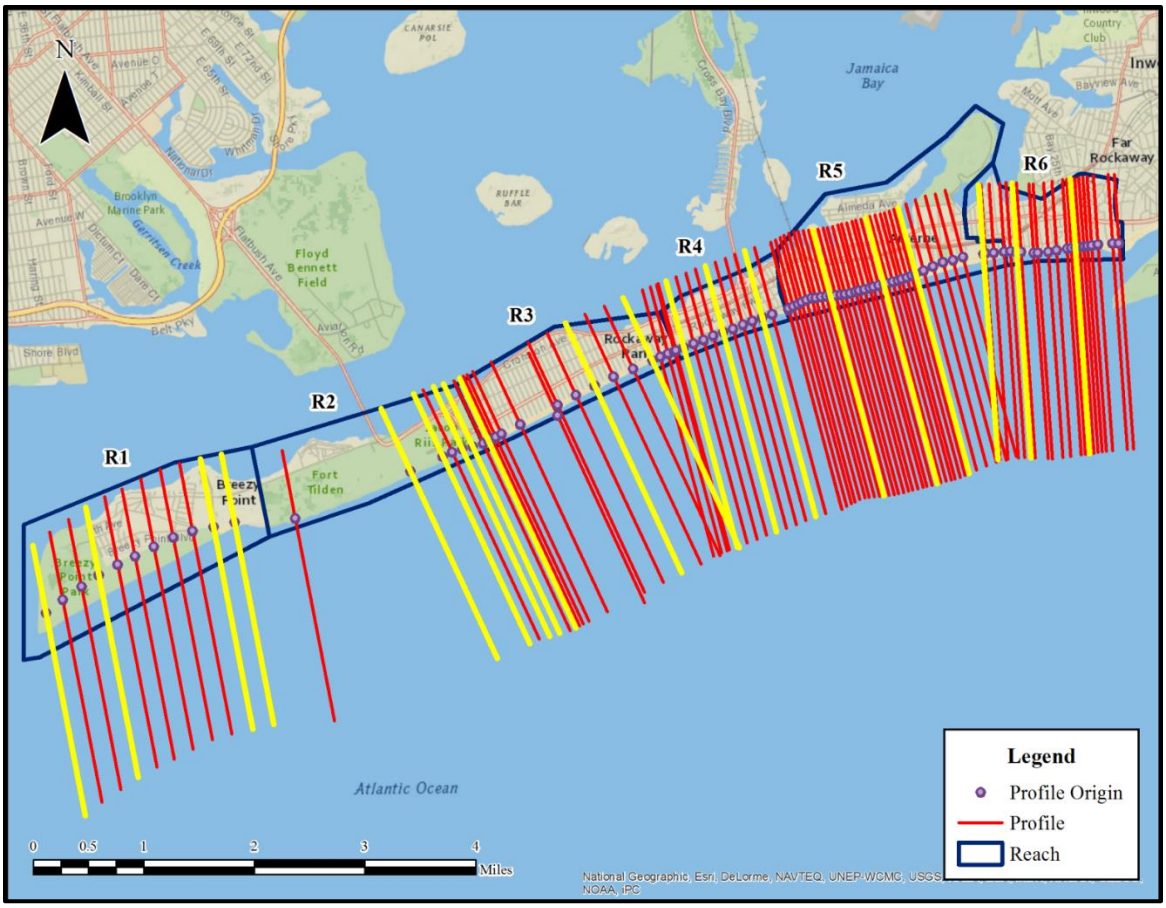
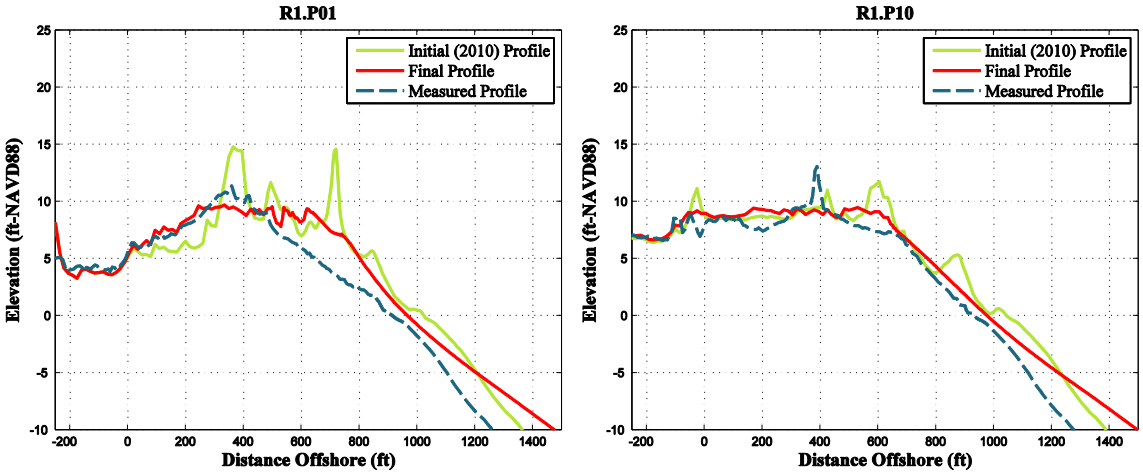


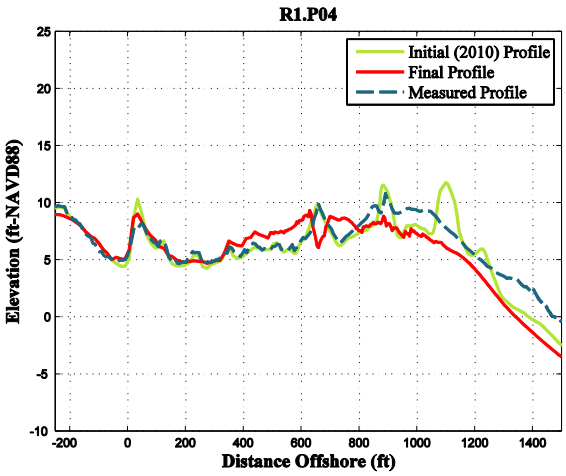
Figure 5-24: SBEACH Hurricane Sandy Validation Profile Locations

Overall the SBEACH validation results for Hurricane Sandy, Figure 5-25 to Figure 5-29, show that the dry beach erosion predicted by SBEACH was in satisfactory agreement with the observed changes, especially in Reaches 3 and 5. The validation analysis found that dunes at Breezy Point were effectively flattened by Hurricane Sandy. Reach 2 experienced very little erosion between 2012 and 2012, and the model overestimated erosion in Reach 2. However, the 2010 data may not be representative of the pre-Sandy conditions in this Reach. Validation comparison at Reach 6 is limited by the post-Sandy nourishment that is already present in the post-Sandy LIDAR data. Potential sources of error in the model calibration are attributed to alongshore sediment not capture by SBEACH and beach profiles changes occurring between the pre- and post-Sandy data as a result of wave conditions and storm events other than Hurricane Sandy.


R1
R2
R3
R4
R5
R6
1
2
3
4
5
6
7
8
9
10
11
N
Legend
Profile Origin
Profile
Reach


R1.P01
Initial (2010) Profile
Final Profile
Measured Profile
Elevation (ft-NAVD88)
Distance Offshore (ft)

R1.P10
Initial (2010) Profile
Final Profile
Measured Profile
Elevation (ft-NAVD88)
Distance Offshore (ft)


R1.P04
Initial (2010) Profile
Final Profile
Measured Profile
Elevation (ft-NAVD88)
Distance Offshore (ft)

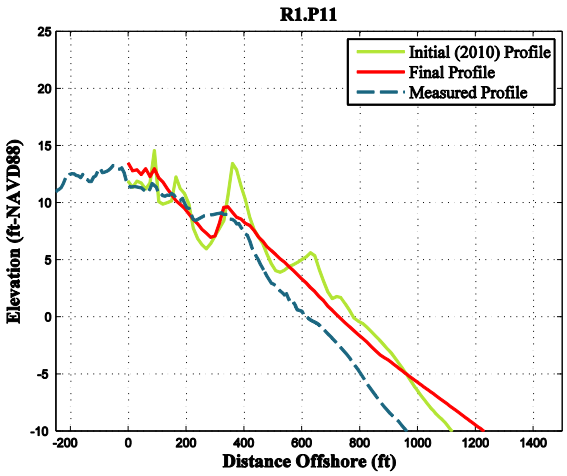
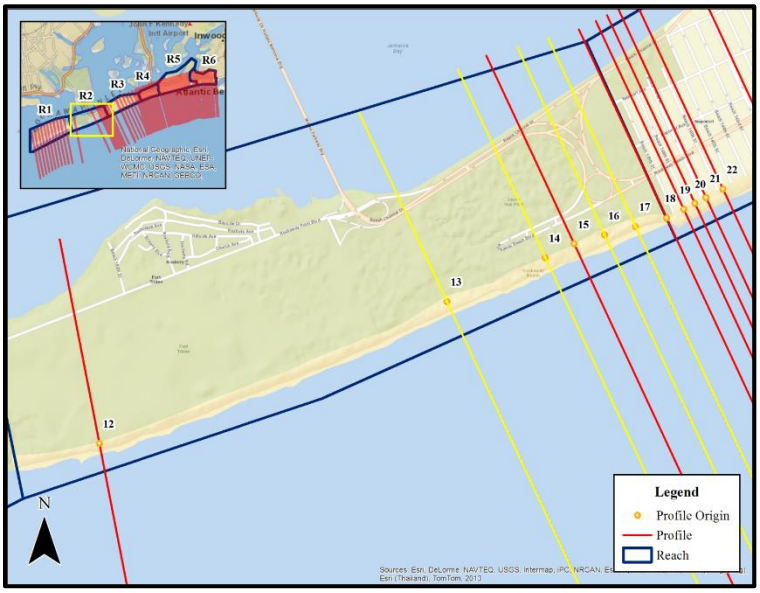
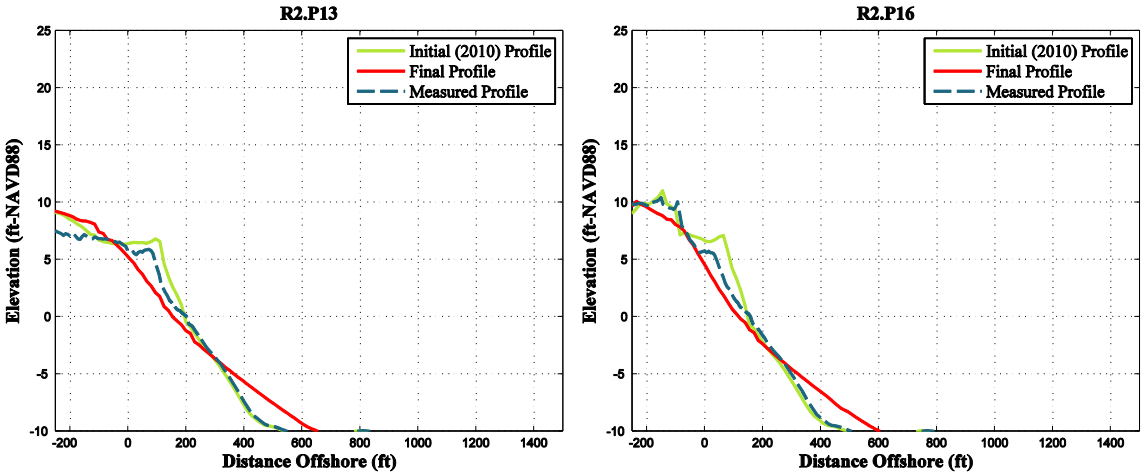

R1.P11
Initial (2010) Profile
Final Profile
Measured Profile
Elevation (ft-NAVD88)
Distance Offshore (ft)

Figure 5-25: SBEACH Hurricane Sandy Validation Results – Reach 1





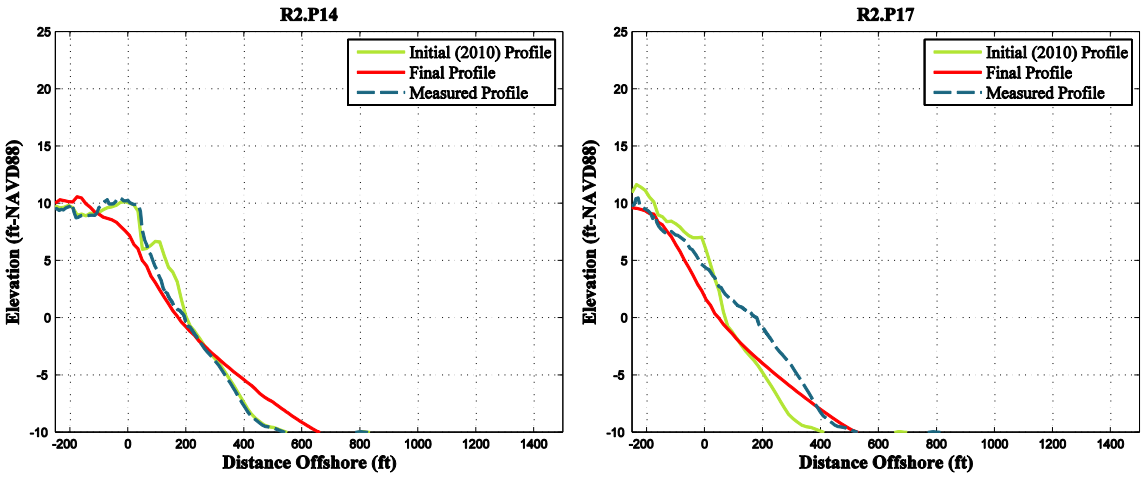
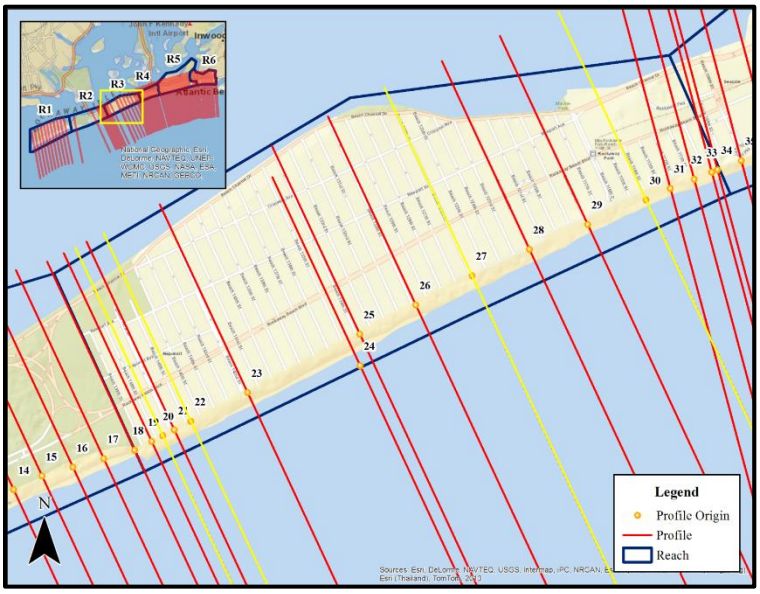


Figure 5-26: SBEACH Hurricane Sandy Validation Results – Reach 2



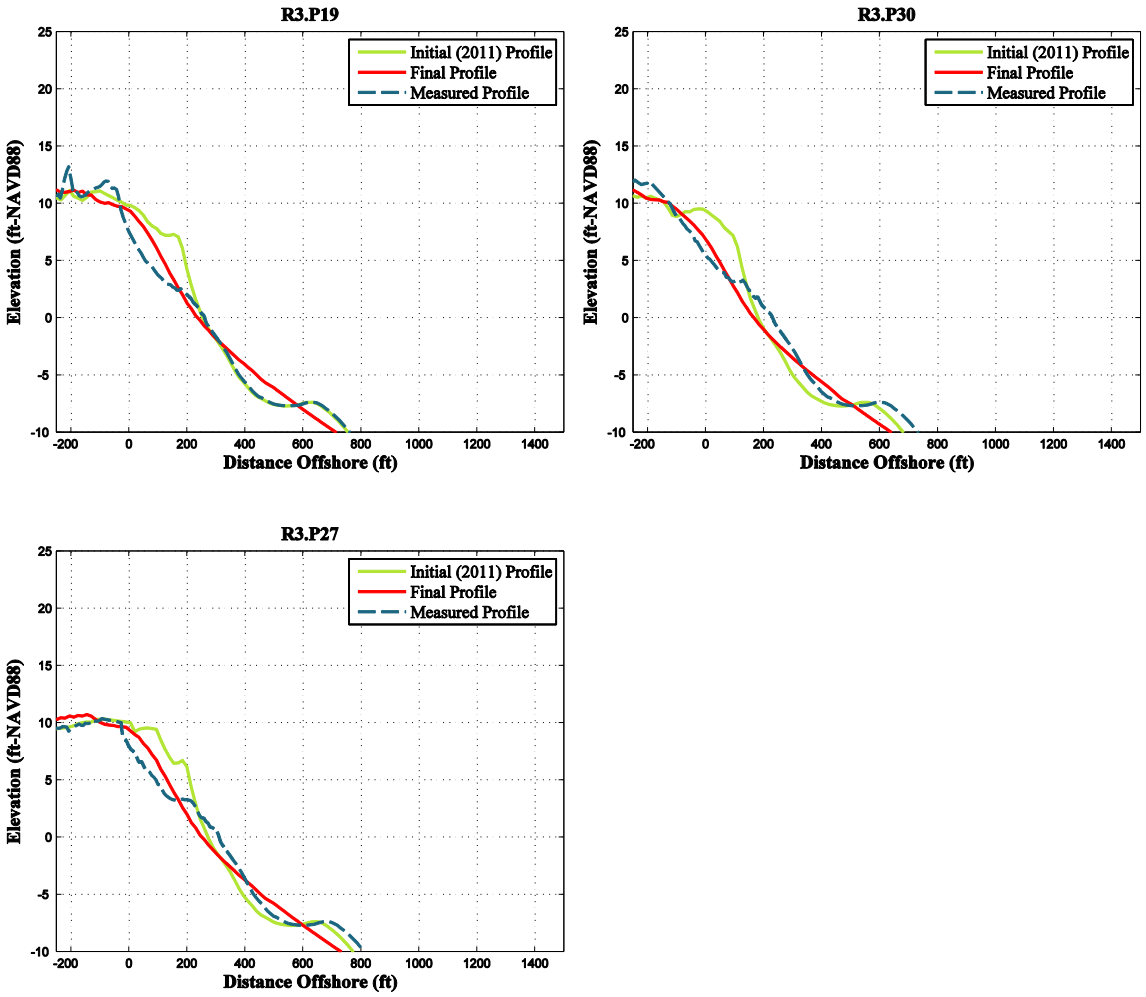
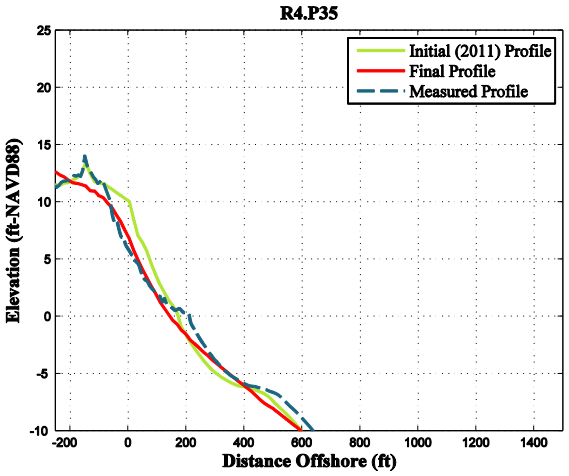
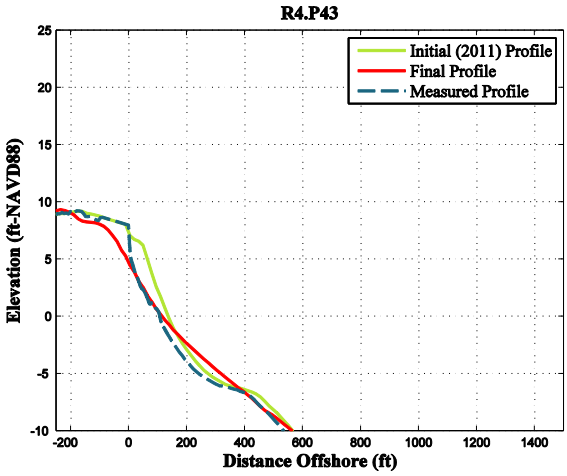


Figure 5-27: SBEACH Hurricane Sandy Validation Results – Reach 3


R1
R2
R3
R4
R5
R6
Legend
Profile Origin
Profile
Reach
N


R4.P35
Initial (2011) Profile
Final Profile
Measured Profile
Elevation (ft-NAVD88)
Distance Offshore (ft)


R4.P43
Initial (2011) Profile
Final Profile
Measured Profile
Elevation (ft-NAVD88)
Distance Offshore (ft)

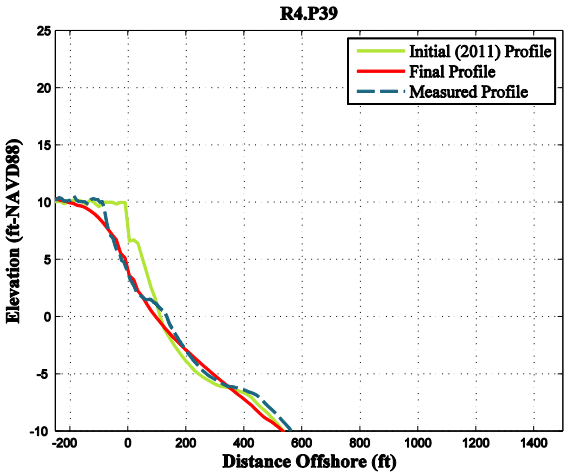
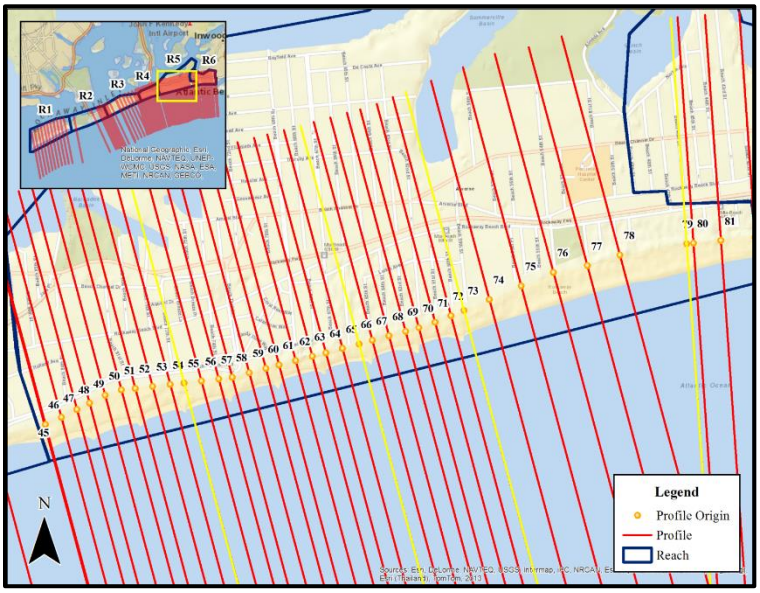
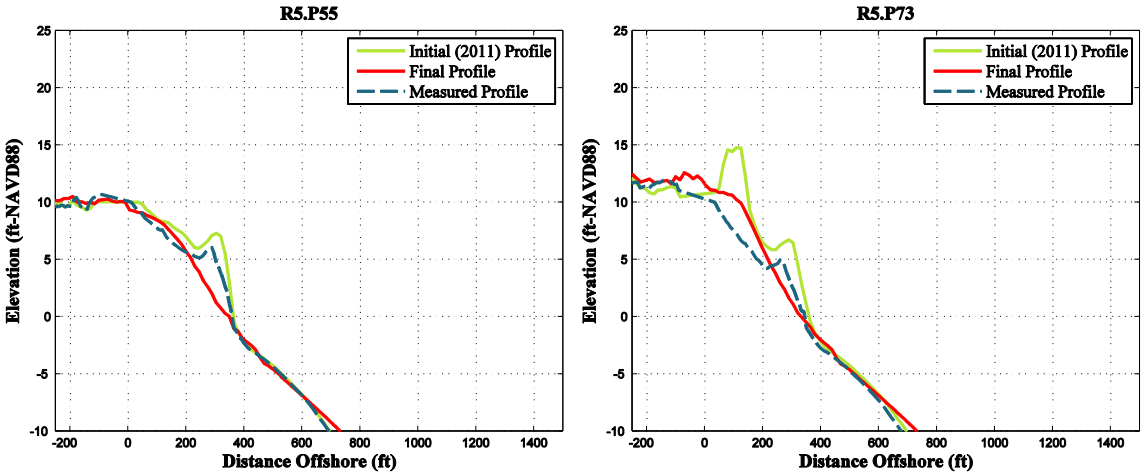

R4.P39
Initial (2011) Profile
Final Profile
Measured Profile
Elevation (ft-NAVD88)
Distance Offshore (ft)

Figure 5-28: SBEACH Hurricane Sandy Validation Results – Reach 4





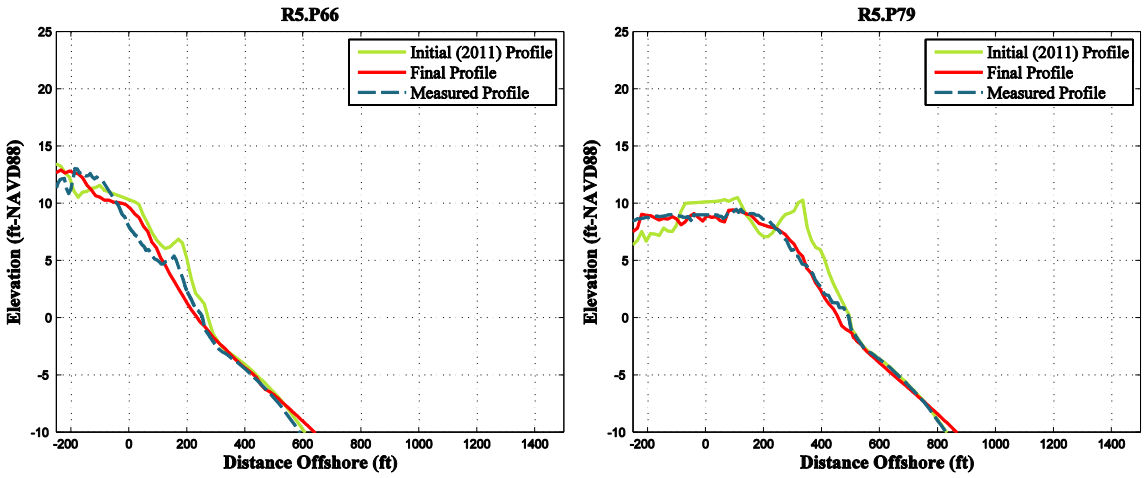


Figure 5-29: SBEACH Hurricane Sandy Validation Results – Reach 5

5.3 SBEACH Without- and With-Project Simulations

The engine behind the Beach-fx Monte Carlo simulations is the Storm Response Database (SRD). The SRD is a lookup table that stores the morphological profile responses (i.e. change in berm width and dune) and damage driving parameters (i.e. wave height, water level, and vertical erosion). The SRD must include SBEACH results for all storm and profile combinations. The storm suite is based on a training set of 38 historical storms, which expands to 456 total events when all twelve possible tidal combinations are added. Approximately 900 unique profiles are required to capture the range of possible existing and with-project profile combinations for R1T1 and R2T1. As a result 410,400 (900 x 456) SBEACH simulations are required to construct the SRD.

5.3.1 Typical Beach Profiles

Twelve (12) simplified profiles, two for each reach were developed based on surveyed profile data sets measured from 1996 through 2003 by the Atlantic Coast of New York Erosion Monitoring Program (ACNYMP). Each survey repeated measurements from an established profile origin. Only measurements taken during the spring of each year were included for seasonal consistency and to capture the post-winter beach conditions. The delineated reaches for Rockaway Beach are shown in Figure 4-1. In order to develop simplified profiles for each reach to be used in Beach-fx model, representative profiles from temporally and spatially averaging profiles within each reach were created. The following steps were used to create the simplified profiles:

1. Compile all existing long range profile data for the project area and import into RMAP (Regional Morphology Analysis Package). Long range profiles extend from the upland origin point to a minimum of 2500 ft offshore;

2. At each profile station, align measured profiles at Mean High Water (Figure 4-2). Only spring measurements are used here;
3. At each profile station, average aligned profiles to determine the representative station profile (Figure 4-3). Temporal averaging at each individual profile station is performed prior to determining reach representative profiles to eliminate unfair weighting due to variations in the total number of surveys available at each profile station;
4. Separate shoreline into reaches and sub-reaches based on general profile morphology and shoreline characteristics (orientation etc.);
5. Align all representative station profiles within each reach at Mean High Water;
6. Compute the representative profile for each reach by averaging all the aligned representative station profiles (Figure 4-4). Spatial averaging of all stations within the sub-reach is used to produce the 'reach representative profile'.

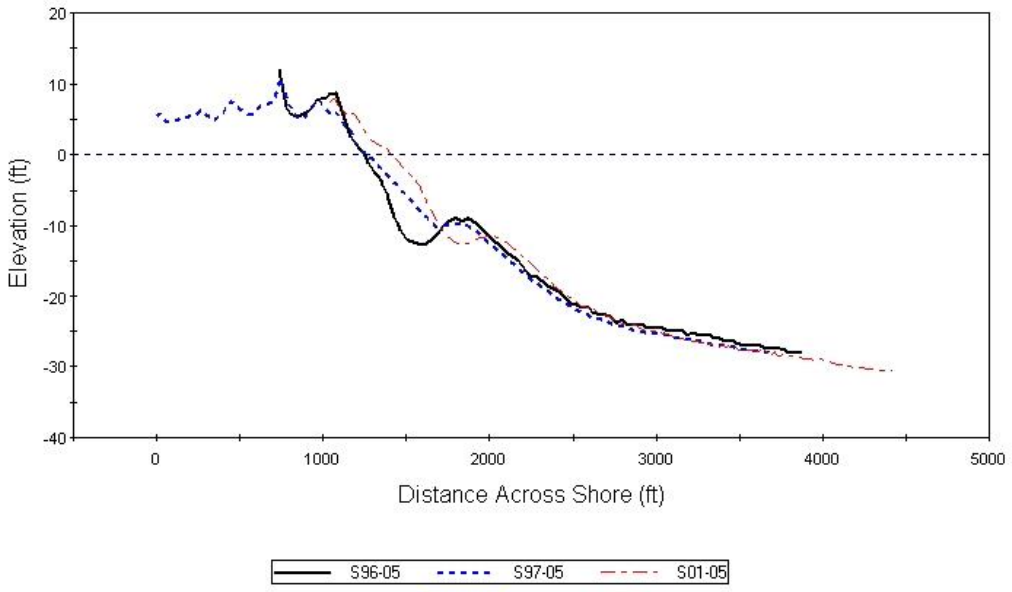


Figure 5-30: Measured profiles at Station 5. S96-05 stands for Profile 5, surveyed in the spring of 1996

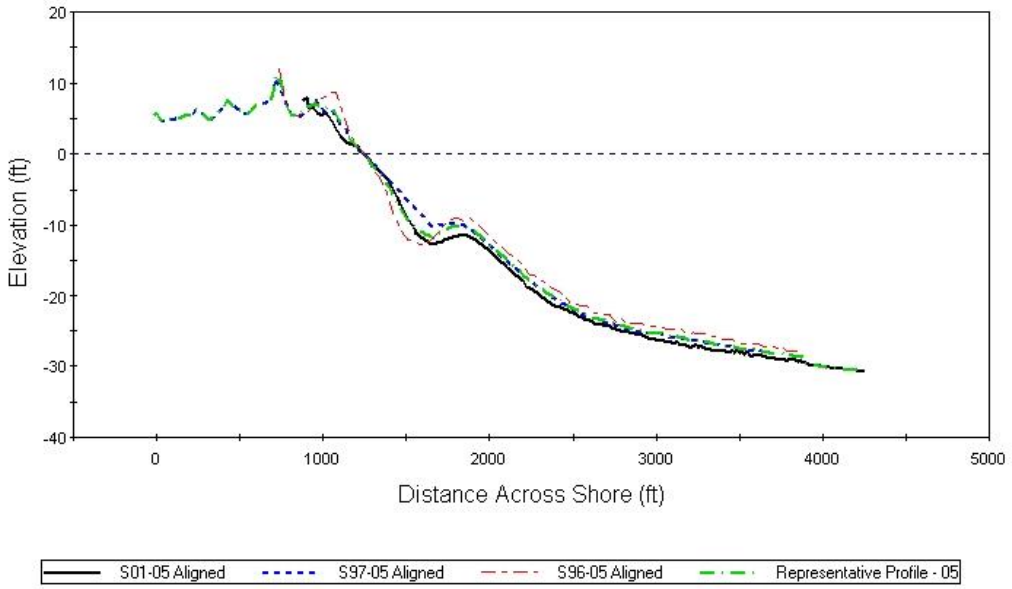


Figure 5-31: Aligned profiles at Station 5. The Representative Station Profile is the average of the aligned (at MHW) profiles

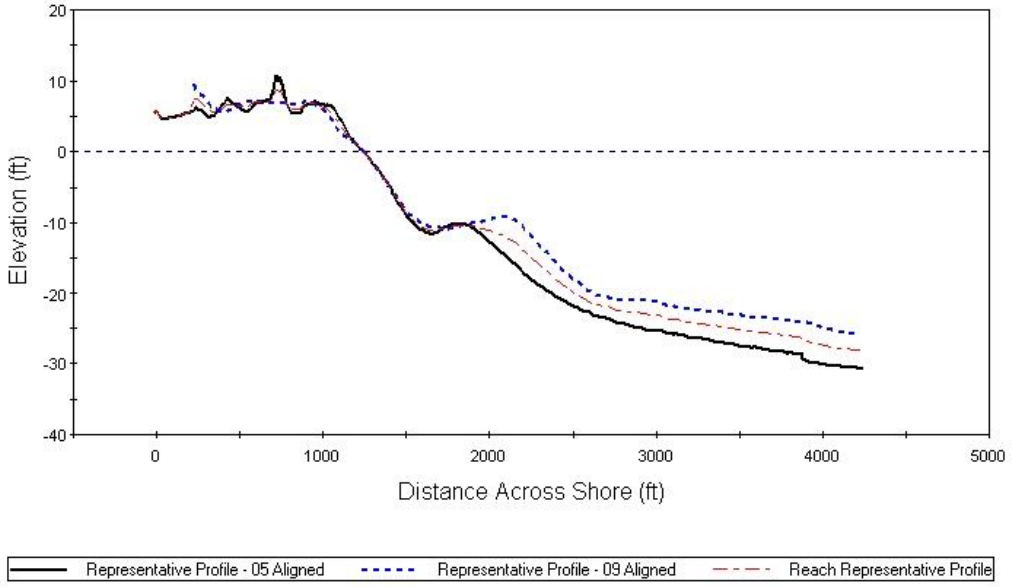


Figure 5-32: Both Representative Station Profiles in this Reach have been aligned at MHW. The Reach Representative Profile is the average of the aligned representative profiles

5.3.2 Beach-fx Simplified Profiles

The representative profile for each reach was further simplified to develop the simplified profiles required for Beach-fx. Each simplified profile in Beach-fx is represented by a single trapezoidal dune (constant dune slope), horizontal berm, constant foreshore slope, and a detailed submerged profile. Figure 4-5 shows the key features in a simplified Beach-fx profile. The existing conditions simplified beach profile configurations are shown in Table 4-2. A single berm height of 6 ft (NAVD88) is used throughout, and all the profiles, except R1T1, do not contain a "physical" dune feature. Specifically, even though all the reaches have a defined dune elevation, and width, only profile R1T1 contains a dune elevation above the upland elevation. All the other dunes are really just an extension of the upland elevation.

The submerged portion (below 0 NAVD88) of each "reach representative profile", developed in step 6, was used as the detailed submerged profile for each reach. In addition to defining the "existing conditions" simplified profile for each sub-reach, a range of pre-defined profiles for each sub-reach, expressed by variations in berm width, dune width, and dune elevation are required for the SRD. It is necessary that the profile variations span all the possible profile configurations during the lifetime of the project (50 years). In order to reduce the number of required profiles in the SRD, it is assumed in Beach-fx that some of the characteristic features of the profile are constant throughout the lifetime of the project (i.e. they do not vary with the storm response). The constant profile features are the upland elevation, dune slope, berm, foreshore slope, and shape of the submerged profile. The profile characteristics that change in response to storms are the dune width, dune elevation, and berm width. Therefore, the pre-defined profiles for each sub-reach only contain changes to the profile characteristics that change in response to storms.

Even though upland erosion may occur in response to a storm in Beach-fx, it is not necessary to run SBEACH simulations for profile variations with reduced upland widths. This is because all such profiles have identical storm responses except translated landward/seaward. Within Beach-fx a translation of the SBEACH results is performed, eliminating the need for profile configurations with reduced upland widths. Furthermore, if the dune elevation and upland elevation are the same it makes no difference in Beach-fx or SBEACH if the high upland morphology is defined including a dune width. Consequently, in cases where the dune elevation and upland elevation are the same, possible profile configurations do not include reduced dune widths either.

The predefined profile configurations were generated using a program, "SBEACHGenerator.exe", provided by USACE-CHL. The program is capable of generating all possible profile permutations at specified dune width, berm width, and dune height increments. The dune and berm widths were modified in 5 ft and 10 ft increments respectively and dune heights were modified in 1 ft increments. A sample of profile configurations for R1 is shown in Figure 28-30. The naming convention used for the profiles is based on previous Beach-fx projects and insures a smooth transition from SBEACH to Beach-fx. An example of a profile name generate by



SBEACHGenerator.exe is R1T1_10.0_001_260, where R1T1 is the profile name (this must match the profile description in Beach-fx), 10.0 is the dune height, 001 is the dune width, and 260 is the berm width.

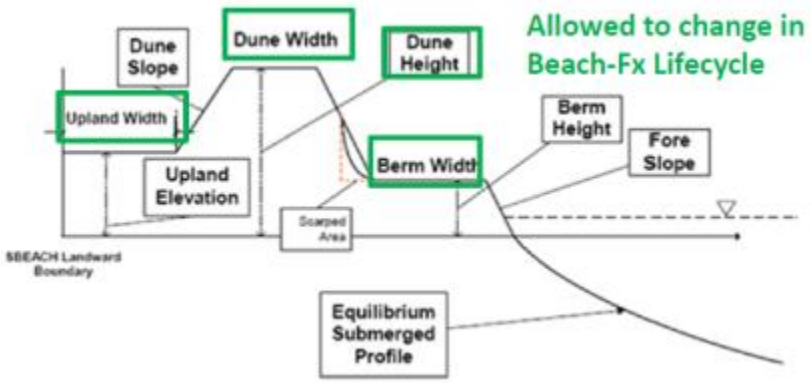


Figure 5-33: Beach-fx Simplified Profile

All possible dune and berm configurations anticipated in the existing and with-project Beach-fx simulations were identified and included in the Storm Response Database (SRD). A Sensitivity analysis indicated that the profile responses for berm widths greater than or equal to 210 feet were nearly identical, and therefore it is not necessary to include berm widths greater than 210 feet in the SRD. An overview of the profile combinations identified for inclusion in the SRD is presented below for R1T1 and R2T1.

R1T1 – Reach 1 (Breezy Point)

Fixed Parameters

Upland Elevation = +6 ft NAVD

Berm Elevation = +6 ft NAVD

Dune Slope = 1:5

Fore Slope = 1:25

Submerged Profile = R1

Variable Parameters

Dune Elevations (ft, NAVD) = 6, 7, 8, 9, 10

Dune Widths (ft) = 5, 10, 15, 20, 30, 40

Berm Widths (ft) = 0, 10, 20, 30, 40, 50, 60, 80, 100, 120, 150, 180, 210

R2T1 - Reach 2 to Reach 6

Fixed Parameters

Upland Elevation = +10 ft NAVD

Berm Elevation = +8 ft NAVD

Dune Slope = 1:5

Fore Slope = 1:15

Submerged Profile = R4

Variable Parameters

Dune Elevations (ft, NAVD) = 10, 11, 12, 13, 14, 15, 16, 18, 20, 22

Dune Widths (ft) = 5, 10, 15, 20, 25

Berm Widths (ft) = 0, 10, 20, 30, 40, 50, 60, 80, 100, 120, 150, 180, 210

5.3.3 Reinforced Dunes

Two reinforced dune alternatives have been proposed for Rockaway Beach. The first alternative is a buried seawall designed to protect inland areas from erosion and wave damages during severe storm events such as Hurricane Sandy. The second alternative is a composite seawall designed to limit storm surge inundation and cross-island flooding in addition to erosion and wave damages during severe storm events. This appendix provides an overview of the approach applied to the East Rockaway to Rockaway Inlet and Jamaica Bay Reformulation Study to integrate the reinforced dune alternatives in the economic analyses (i.e. Beach-fx).

Presently, Beach-fx is capable of simulating seawalls that prevent landward erosion, but not seawalls that reduce upland wave heights and upland flood elevations. SBEACH is used in this study to generate the Beach-fx Storm Response Database (SRD) also has limitations in how it treats seawalls. Due to the limitations of these two models a new approach is applied in this study to evaluate the reinforced dune alternatives.

In general, the approach involves “offline” wave transmission and wave overtopping calculations, which are then used to estimate wave heights and flood elevations landward of the reinforce dunes. The offline calculations use the SBEACH model results as inputs to the empirical formulas (e.g. water level, wave heights, etc.). The wave transmission coefficients and wave overtopping rates are used to adjust the SBEACH results landward of the structures and write new modified SBEACH output files. The modified SBEACH output files are identical to the original SBEACH output files except landward of the reinforced dunes where wave heights and flood depths have been altered.



The following sections briefly describe the approach for the two reinforced dunes and compare the original and revised SBEACH output. A comparison of the SBEACH output for the three design beachfill profiles and two reinforced dunes is also provided at the end.

5.3.3.1. SBEACH Standard Treatment of Seawalls

This section describes how seawalls are treated in SBEACH. Seawalls may be incorporated in SBEACH by specifying the seawall location in the profile and a set of failure triggers (the seawall can also be set to never fail). The seawall crest elevation cannot be specified in SBEACH, which treats the seawall as an infinitely high wall; thus, wave transmission, wave overtopping, or landward flooding is simulated in SBEACH (unless the seawall fails). Seawall failure may be incorporated based on the following three failures triggers:

- Maximum allowable vertical erosion;
- Maximum allowable total water elevation (including wave setup);
- Maximum allowable wave height.

If any one of these conditions is exceeded at the seawall during the simulation, the seawall "fails" (e.g. instantly disappears). The failure triggers must be applied at the same location as the seawall. When the seawall fails, SBEACH completely removes the structure from the simulation, allowing for waves and water to freely penetrate. Conversely, when the seawall does not fail, the structure completely prevents water, waves, and erosion landward of the seawall. As a result seawall protection and failure can be quiet unrealistic, selecting too low or too high failure triggers may significantly over-predict or under-predict storm-induced damages landward of the seawall.

In this study SBEACH simulations with seawalls (no failure allowed) are performed to capture the morphological changes with a seawall. Offline wave transmission and wave overtopping calculations are performed to estimate the wave and water level conditions landward of the seawalls.

5.3.3.2. Buried Seawall Approach

Buried seawalls are dunes with a reinforced rubble mound core. Buried seawalls were originally developed as an alternative to larger standalone seawalls / massive dunes and are designed to function in conjunction with beach restoration projects. The primary advantage of buried seawalls over traditional dunes is the additional protection against erosion and wave attack provided by the stone core. The buried seawall has a permeable core and will therefore not prevent inland flooding.

The approach applied to incorporating Buried Seawalls in SBEACH is as follows:

7. Perform SBEACH simulations for regular dune conditions (D-Runs)
8. Perform SBEACH simulations for regular dune with seawall (S-Runs)

9. Calculate wave transmission coefficient based on local water level (including wave setup) and wave height from S-runs.
10. Replace water level profiles from S-Runs with D-Runs (e.g. structure is permeable).
11. Replace wave height profiles from S-Runs with D-Runs, apply wave transmission coefficient to waves landward of structure.
12. Re-write SBEACH output file from the S-runs with the changes described above.

The wave transmission coefficient is calculated based on the simplified prediction method from CIRIA (2001) which relates the relative crest height of the structure to the wave transmission coefficient is used here to estimate the wave transmission. The CIRIA wave transmission relationship indicates that roughly a 50% decrease in the transmitted wave height occurs if the structure freeboard (Rc) is zero. Therefore, the crest elevation of the structure must be roughly equal to the local water level at the toe of the structure (including wave setup) to achieve a 50% reduction in the wave transmitted wave height. The primary inputs to the wave transmission formula are shown in Table 5-7.

Table 5-7: Wave Transmission Inputs

Parameter	Value/Definition	Comments
Crest elevation	16 ft NAVD	Fixed value
Seawall toe location	100 ft seaward from seawall	Defined by ~1/2 wavelength in front of seawall
Local Water Level	Still water level plus wave setup	Extracted from SBEACH results
Crest Freeboard, R_c	Crest elevation minus local water level	Calculated from SBEACH results
Wave height at toe	Maximum significant wave height during storm, at the toe position	Extracted from SBEACH results

5.3.3.3. Composite Seawall Approach

The second reinforced dune concept is a composite seawall with an impermeable core (i.e. steel sheet pile). The composite seawall is designed to limit storm surge inundation and cross-island flooding as well as erosion and wave attack. The composite seawall may also be compatible with a comprehensive storm surge barrier for Jamaica Bay. In contrast to the buried seawall, it is assumed that the wave transmission coefficient is zero due to the impermeable core. Wave overtopping and subsequent inland flooding is calculated as described below.

The approach applied to incorporating Composite Seawalls in SBEACH is as follows:

1. Perform SBEACH simulations for regular dune with seawall (S-Runs)
2. Calculate wave overtopping rate based on structure freeboard and wave height from S-runs.



3. Calculate corresponding upland flood depth based on wave overtopping rate and uniform open channel flow theory (Manning's formula).
4. Modify water levels in S-Runs landward of structure to reflect upland flood depth.
5. Re-write SBEACH output file from the S-runs with the changes described above.

The composite seawall is treated as a rubble mound structure from a wave overtopping perspective and the methods in the EurOtop Overtopping Manual (2007) were applied. The primary inputs to the wave overtopping formula are shown in Table 5-8.

Table 5-8: Wave Overtopping Inputs

Parameter	Value/Definition	Comments
Crest elevation	17 ft NAVD	Fixed value
Seawall toe location	100 ft seaward from seawall	Defined by ~1/2 wavelength in front of seawall
Still Water Level	Maximum water level at 40 ft depth	Extracted from SBEACH boundary conditions
Crest Freeboard, Rc	Crest elevation minus still water level	Calculated from SBEACH results
Wave height at toe	Maximum significant wave height during storm, at the toe position	Extracted from SBEACH results
Peak wave period	Maximum peak wave period	Extracted from SBEACH boundary conditions
Roughness factor, γ_f	0.55	Rocks (2 layers, impermeable core)

The upland flood depth is calculated based on the wave overtopping rate and uniform open channel flow theory (Manning's formula). A composite roughness coefficient of 0.065 was estimated based on the weighted average of the surfaces landward of the reinforced dune (e.g. asphalt, bare earth, short grass, light brush and trees, medium brush and trees, and buildings). A surface slope of 1V:666H is applied in the open channel flow calculations.

Since SBEACH does not simulate wave overtopping, there may be some instances in the sand-only alternative simulations where minor wave overtopping is not captured by the model. In order to provide a "apples to apples" comparison of the sand-only and composite seawall alternative, upland flooding is only incorporated for storms that result in a wave overtopping rate greater than 20 l/s/m, corresponding to a flood depth of approximately 0.5 feet.

Upland Flood Depth

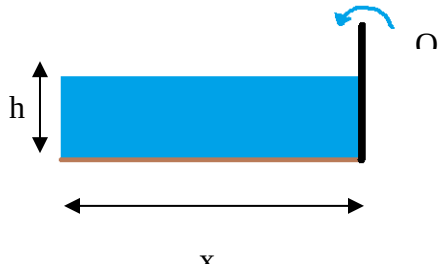
The flood depth on the leeside of the structure is estimated by applying uniform open channel flow theory (Manning's formula). Although wave overtopping during a storm is typically not uniform,

this approach provides a simple way to estimate the flooding landward of the seawall caused by high rates of wave overtopping.

Manning's formula is:

$Q = AV = (1.49/n)R_h^{2/3}S_e^{1/2}$ (U.S. Customary Units) (1)

where:



Q = discharge

A = area = hx

V = velocity

h = water (flood) depth

R_h = hydraulic radius = area/wetted perimeter = A/x

S_e = bed slope (landward of seawall)

n = Manning (roughness) number

The design wave overtopping rate (q) calculated with the EurOtop Manual is a discharge per unit width *x* (i.e. q = Q/x). Substituting into Equation 1 and solving for the flood depth *h* results in (Bedient and Huber, 1992):

$h = (nq/1.49S_e^{1/2})^{3/5}$ (U.S. Customary units) (2)

Hence, for this application, the flood depth is a function of the wave overtopping discharge, bed slope, and Manning's number. As explained previously, the wave overtopping rate was calculated at the toe of the seawall, determined to be 100 ft seaward from the structure. The bed slope on the leeside of the wall was defined as 1/666, thus giving a decrement of 3 ft over the upland width of about 2000 ft. The Manning's number, however, had to be estimated based on the land cover across Reaches 3 to 6 where the seawall would be located.

Estimation of Manning's Number

Aerial imagery of Rockaway suggests that the area to be protected by the seawall is in general highly developed with buildings, streets, and parking lots. However, the variations in building density as well as the presence of vegetation and undeveloped areas make it difficult to identify an appropriate Manning number for the entire area. The selection of a representative Manning's number is important, as the sensitivity of the flood depth to the Manning's number is high (Figure 5-34).

A composite Manning's number was estimated for each reach by examining the alongshore land cover characteristics within 1000 ft landward of the proposed seawall location. The composite value was calculated in the following manner (USGS 1984):

$n_{composite} = (n_1d_1 + n_2d_2 + \ldots n_nd_n)/(d_1 + d_2 + \ldots d_n)$ (3)

where *d* is the distance covered by a surface of Manning's number *n*. The type of surfaces that were found to primarily cover the area of interest and their corresponding Manning's number are listed in Table 5-9. With the exception of the building category, the source of these values is Chow (1959). A value of 0.12 was adopted for buildings based on guidance from URS, and this was found to be within other ranges found for buildings in the literature.

Table 5-9: Manning's number for main surfaces covering Reaches 3 to 6

Surface Type	Manning's Number
Asphalt (rough) / Concrete	0.016
Bare earth	0.025
Short grass	0.03
Light brush and trees	0.06
Medium brush and trees	0.08
Building	0.12

Figure 5-35 illustrates the application of this approach to a segment in Reach 3 and Table 5-10 shows the composite *n* values estimated for each reach. These values appear to be consistent with the building and vegetation densities observed in each reach. Given the small difference between these values and based on the sensitivity analysis shown in Figure 8, a Manning's number of 0.065 is proposed to be applied to all reaches.

Table 5-10: Composite Manning's number for each reach

Reach	Composite Manning's Number
3	0.076
4	0.057
5	0.056
6	0.064
Overall (apply to all reaches)	0.065

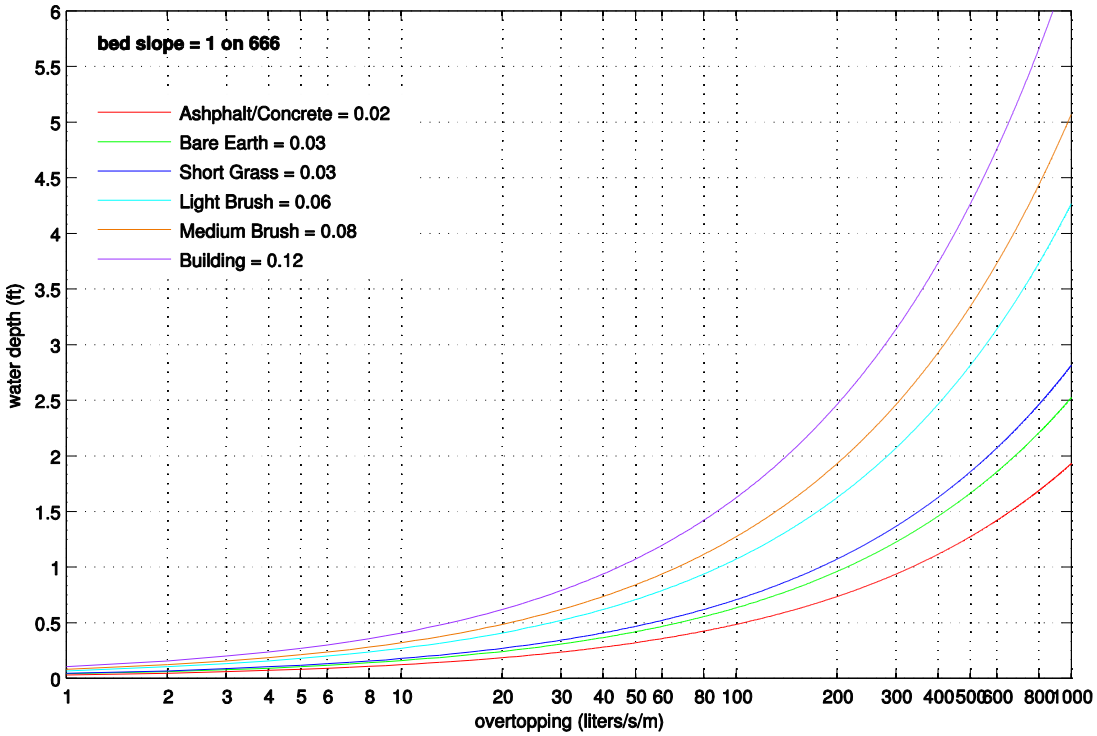


Figure 5-34: Sensitivity of flood depth (h) to Manning's number (n)



Figure 5-35: Illustration of composite Manning's number approach for Reach 3

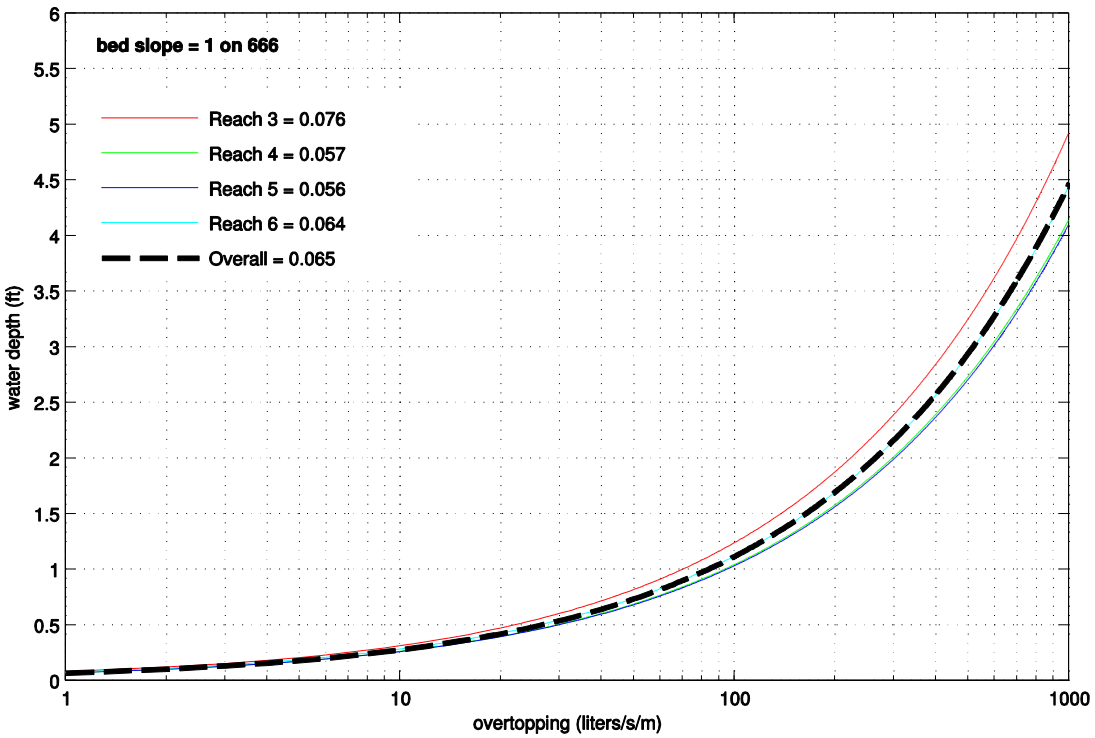


Figure 5-36: Sensitivity of flood depth to the various composite Manning's numbers

Incorporation of Flood Depth

Once calculated, the flood depth was incorporated into the original SBEACH output file from the location of the seawall to the extent of the upland elevation (~2000 ft). Figure 5-37 depicts the final product. The flood depth was sloped down using a 1/666 slope so its maximum is on leeside of the seawall and then decreases away from the structure.

Since SBEACH does not simulate wave overtopping, there may be some instances in the sand-only alternative simulations where minor wave overtopping is not captured by the model. In order to provide a fair evaluation of the sand-only and seawall alternatives wave overtopping and flood depths will only be incorporated for storms that result in a wave overtopping rate exceeding a threshold of 20 l/s/m, which yields a flood depth of approximately 0.5 feet. Figure 5-38 shows the number of storm events that exceed the wave overtopping threshold for each dune crest scenario.

In some cases where the seawall was not fronted by a dune and the overtopping threshold was exceeded, the flood depth on the leeside of the seawall was calculated to be higher than the ocean side water level. In these instances, the flood depth was set equal to the water level at the seawall on the ocean side and sloped down as described before. None of the storms were observed to generate a water level (including setup) greater than the seawall crest elevation (+18 ft NAVD).

However, if lower seawalls are considered than the local water level (including setup) for some storms will exceed the structures crest elevation.

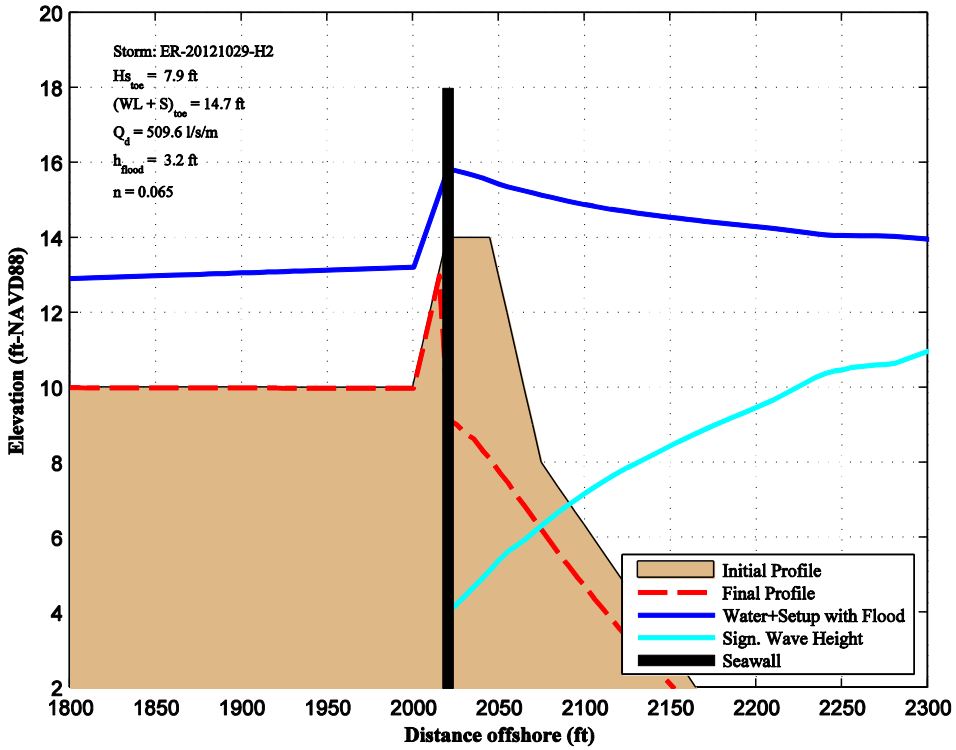


Figure 5-37: SBEACH output with flooding behind seawall

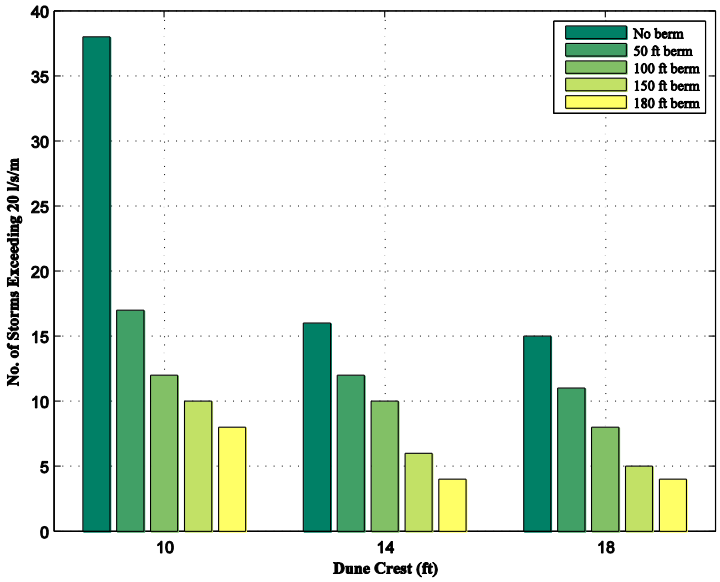


Figure 5-38: Number of storms to be modeled based on overtopping threshold

5.3.4 SBEACH Model Results

Over 410,400 (900 x 456) SBEACH simulations were required to construct the SRD for Beach-fx. A sample set of the SBEACH results for the following profiles and all 456 storms are provided in Enclosure A "SBEACH Model Results":

- R1T1_06.0_000_000
- R1T1_10.0_040_180
- R2T1_10.0_000_060
- R2T1_13.0_025_060
- R2T1_16.0_025_060
- R2T1_18.0_025_080
- R2T1_20.0_025_100
- R2T1_18.0_025_060 – Buried Seawall
- R2T1_18.0_025_060 – Composite Seawall

The naming convention "R2T1_20.0_025_180" indicates the profile (R2T1), dune height (20.0), dune width (025), and berm width (180).

6.0 EMPIRICAL SIMULATION TECHNIQUE (EST)

The Empirical Simulation Technique (EST) is used here to define frequency of occurrence relationships for important design parameters such as nearshore wave height, wave setup, wave overtopping, and beach and dune erosion.

6.1 Important Probability Concepts

To develop design parameter vs. frequency relationships, probability methods must be applied based on peak design parameter records (historical or simulated). The probability that an event, X, will occur and be equal to some prescribed value, x, is given by $P(X=x)$. Probability of exceedance, $P(X \geq x)$, is the probability that an event, X, will occur, in some prescribed time interval, and exceed x. Probability of exceedance, $P(X \geq x)$, is equal to the sum of all probabilities, $P(X=xi)$, where $xi \geq x$. For example, if a peak wave height of 10 ft has a probability of exceedance of 0.9, there is a 90% chance that peak wave height will exceed 10 ft. Conversely, the probability of nonexceedance, $P(X \leq x)$, is equal to $1-P(X \geq x)$ and is the probability that no event, X, will occur and exceed some prescribed value, x. For this study, probability of exceedance is analogous to frequency. Typically, engineers present design parameter vs. frequency relationships in terms of return period. Return period, Tr, is the reciprocal of the probability of exceedance ($Tr(x)=1/P(X \geq x)$).

To associate probability with a given event, it is necessary to assume the population of events follows some defined probability distribution. A cumulative distribution function (CDF), F(x), is a function that describes the probability of nonexceedance, such that $F(x)=P(X \leq x)$. The probability density function (PDF), f(x), is defined as the derivative of the cumulative distribution function, $f(x)=d/dx(F(x))$. The most well-known PDF is the normal (Gaussian) distribution function, or bell curve.

In this study univariate EST and multivariate EST are applied to determine frequency-of-occurrence relationships. Furthermore, the population of storms was divided into two distinctly different sets: extratropical and tropical.

6.1.1.1. Peak-Over-Threshold Analyses

Peak-over-threshold methods involve analyses of historical records to extract a subset of peak values, N values, exceeding some prescribed cutoff criteria, such as a maximum expected annual tide or similar. This subset is then ranked, with rank n, in order of magnitude and assigned a probability, P. In this study a training set of storms has already been developed and is assumed to represent the peak wave height and wave setup events for Rockaway.

For peak-over-threshold analyses, the probability of a wave event occurring during a particular interval along with the probability of that event exceeding a certain value must be determined. The EST is used to determine the cumulative distribution, F(x), values that correspond to the

probability of an event causing a wave height at or below a specified value, given that an event occurs. The EST method is detailed in Section 6.2. In the EST, the number of storms in a time interval, t, is modeled with the Poisson distribution. The Poisson distribution is:

$$F(M = m) = \frac{e^{-\lambda t}(\lambda t)m}{m!} \qquad \text{(Equation 6-1)}$$

where:

λ is the average rate of occurrence (ex. 14 storms in 72 years gives λ=14/72=0.194), P(M = m) is the probability of M=m occurrences in the interval t.

Then, the return period for a particular event is:

$$T_r(x) = \frac{1}{\lambda(1 - F(x))} \qquad \text{(Equation 6-2)}$$

The storm training set contains 22 extratropical events over 49 years and 14 tropical events over 72 years. This corresponds to $\lambda_{tropical}$=0.194 and $\lambda_{extratropical}$=0.449.

6.1.1.2. Combined Storm Populations

Two distinct storm populations were considered for this study: extratropical events, characterized by extratropical weather patterns and durations on the order of days, and tropical events, characterized by cyclonic weather patterns and durations on the order of hours. The combined return period, including both extratropical and tropical events, is defined as:

$$T_{combined}(x) = \frac{1}{\left(\frac{1}{T_{tropical}(x)} + \frac{1}{T_{extratropical}(x)}\right)} \qquad \text{(Equation 6-3)}$$

6.2 Empirical Simulation Technique (EST)

EST is a nonparametric method for determining frequency-of-occurrence relationships. Parametric methods assume that the storm population follows some prescribed probability distribution, for example a normal (Gaussian) distribution. In contrast, nonparametric methods do not presume a distribution; instead the distribution is computed from the available data (i.e. it is assumed the future events will be statistically similar in magnitude and frequency to past events). When selecting a method for use with a particular data set, it is important to realize that nonparametric methods are more appropriate when the population is distribution is unknown, while parametric methods are more appropriate when the distribution is known beforehand. As such, nonparametric methods are more appropriate for determining the extreme wave heights and wave setup at Rockaway.

EST begins with an analysis of historical events that have impacted a specific location. In this study these events are the storm training set. The selected database of events is then parameterized based on event characteristics and the impacts of each event. Parameters that define the storm are referred to as input vectors (e.g. wave period, storm surge, etc.). Response vectors define storm related-impacts such as wave setup and shoreline erosion. These input vectors are then used as a basis for generating life-cycle simulations of storm-event activity.

The probability of a storm event occurring in a given year in the life-cycle simulation is modeled by the Poisson distribution (Equation 6-1). When a storm event(s) occurs during a life-cycle simulation a realistic storm event is generated from the historical events using the nearest neighbor interpolation, bootstrap, resampling technique developed by Borgman et al. (1992). The response vectors for each simulated storm event are obtained from 1-D (Univariate EST) or multi-dimensional (Multivariate EST) interpolation between generated storm event and input vectors. Events that are output by the EST are similar to those in the training set with some degree of variability from the historic events.

Each life cycle simulation represents a realistic prediction, albeit with a degree of variability, of the frequency and magnitude of storm events the might occur over a real life cycle. Simulating many life-cycle simulations, as in EST, allows statistical quantification of response parameter vs. frequency-of-occurrence relationships with definable confidence limits. For theoretical details associated with the EST life-cycle simulation procedure the reader is referred to Scheffner et al. (1999).

6.2.1 Univariate EST

Univariate (1-D) EST is any EST simulation with one input vector. In 1-D EST simulations the response parameters are obtained from 1-D interpolation between the input vector and response vector(s). Generally the input and response parameter are the same (e.g. wave height). This approach is used for determining the frequency-of-occurrence relationships for the offshore wave and nearshore wave heights. In this study 500 500-year life-cycle simulations for tropical and extratropical storms to determine the offshore and nearshore wave height vs. frequency-of-occurrence relationships.

6.2.2 Multivariate EST Methods for Determining Probability Distributions

Multivariate EST is a statistical procedure used to develop frequency relationships of occurrence relationships for design parameters as a function of the input parameters (i.e. wave, tide, and surge) that are descriptive of the storm event but have unknown joint probabilities.. The multivariate EST approach used here is the same as used for FIMP (Gravens et al., 1999). Details of the approach, as applied to wave setup along Rockaway, are provided below from Gravens et al. (1999).

Multivariate EST requires specifying a set of parameters that describe the dynamics of some physical system – nearshore wave setup in response to tropical and extratropical storms in this



case. The response parameter, wave setup, must be descriptive of both the process being modeled and the effects of that process. The parameters that describe only the physical attributes of the process are referred as input vectors.

Input vectors are specific parameters describing those characteristics of the storm which have a first-order effect on the magnitude of the modeled response. Nine input parameters, the same as in FIMP, are defined:

6.2.2.1. Storm Surge Parameters

- Peak surge
- Duration of storm surge (length of time surge exceeds 0.3 m)
- Average value of surge over the storms duration

6.2.2.2. Tide Parameters

- Tide amplitude (spring, average, or neap)
- Tide phase at peak surge (high, mid, or low tide)
- Slope of tide at peak surge (needed to distinguish between high tide occurring before and after peak surge)

6.2.2.3. Storm Wave Parameters

- Peak wave height.
- Duration of storm waves (length of time wave height exceeds 1 m).
- Average value of wave height over the storm's duration.

The second class of vectors involves some selected response resulting from the input-vector-parameterized storm. Originally Graves et al. (1999) evaluated 19 specific response vectors, three new response vectors (#20, #21, and #22) have been added to help characterize the performance of the dune and seawall alternatives. The 22 responses are as follows:

1. Erosion Distance – Distance from 0 NAVD on initial beach profile to the landward-most point of 0.3 m (1 ft) vertical erosion/accretion.
2. Eroded Volume above 0 NAVD.
3. Eroded Volume above +3 m (10 ft) NAVD.
4. Vertical Erosion of Dune Crest.
5. Landward Translation of Dune Crest.
6. Maximum Runup Elevation – relative to NAVD.
7. Maximum Runup Height – magnitude of runup relative to instantaneous still water level.
8. Maximum Wave Height – highest wave at offshore boundary of model domain (~10 m depth).



9. Wave Period – wave period associated with maximum wave height.
10. Maximum Tide and Surge – relative to NAVD.
11. Maximum Wave Setup Elevation – relative to NAVD.
12. Maximum Wave Setup Magnitude – relative to instantaneous still water level.
13. Maximum Nearshore Wave Crest Elevation – elevation of wave crest at the shoreline, relative to NAVD.
14. Maximum Nearshore Wave Height – wave height at the landward edge of the inner surf zone.
15. Active Profile Distance – distance from 0 NAVD on initial profile to landward limit of profile change.
16. Recession of 0 NAVD contour.
17. Recession of 1.5-m (5-ft) NAVD contour.
18. Recession of 3.0-m (10-ft) NAVD contour.
19. Recession of 4.6-m (15-ft) NAVD contour.
20. Maximum Upland Flood Depth – height of water above ground on leeward side of dune/seawall
21. Maximum Upland Wave Height – wave height on leeward side of dune/seawall
22. Maximum Wave Overtopping Rate – peak (mean) wave overtopping rate in liters per second per meter during storm event. Calculated outside of SBEACH.

The results for response vector #20 (Upland Flood Depth) and vector #21 (Upland Wave Height) are shown in Figure 6-1.

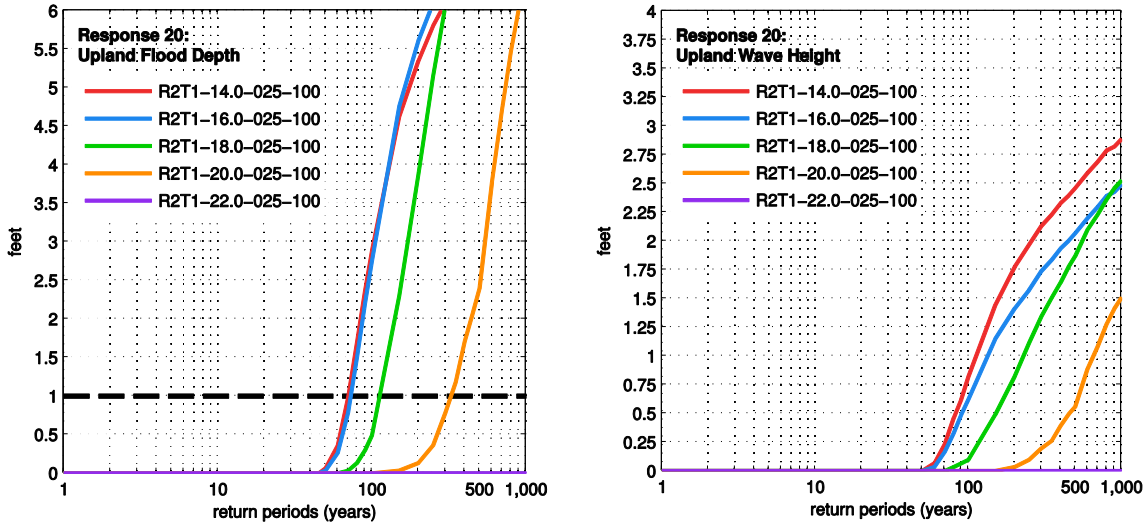


Figure 6-1: EST Results for Upland Flood Depth and Wave Height

7.0 CROSS-ISLAND FLOODING MODELING (XBEACH)

7.1 XBEACH Overview

The purpose of the XBeach modeling effort is to evaluate cross-island flooding during severe coastal storms at Rockaway Beach, NY. XBeach resolves long wave motions (e.g. infragravity waves) and is well suited to predict surfbeat, wave breaking, wave setup, surf and swash zone processes, overwash, inundation, and cross-island flooding during storm events. The primary difference between XBeach and other hydrodynamic models is that XBeach includes wave group driven motions. Studies have shown that nearshore hydrodynamics and morphodynamics are often dominated by time scales associated with wave groups during storm conditions (Roelvink et al. 2009).

XBeach modeling results are used to characterize cross-island flooding during a range of extreme storm conditions (10, 25, 100, and 500-year return period) in Without Project Future Conditions (WOPFC). The modeled water levels across the peninsula are being used as inputs to a HEC-FDA economics model to calculate WOPFC damages to non-shorefront infrastructure.

Unlike many coastal shorelines, Rockaway Beach does not have a natural dune system. The high point along most of the shoreline is a broad coastal ridge with elevations between +10 ft and +12 ft NAVD. Urban development has occurred along the ridge, which is located at the seaward edge of the developed neighborhoods. Landward of the ridge, the ground slopes gradually down towards Jamaica Bay. Figure 7-1 shows pre- and post-Sandy photos along a representative section of Rockaway Beach (Neoponsit). Debris and sand deposits provide evidence of the strong waves and currents capable of transporting large quantities of sand across the peninsula.

The photos also show that the storm did not cause any substantial changes in the bed elevations apart from lowering of the beach berm. If a dune had been present prior to the storm, the post-storm photo would likely have shown substantial dune erosion and depending on the size of the dune, complete flattening of the dune. Not surprisingly, in the absence of a dune sediment transport and morphological processes appear to only play a minor role in cross-island flooding at Rockaway Beach.

It is acknowledged that the Army Corps of Engineers is in the process of placing 3.5 million cubic yards of sand and building a dune with a crest elevation of +16 feet NAVD88. However, in the absence of a continued renourishment it is unclear how long the constructed beach and dune will last. Historical records indicate that substantial quantities of sand have been required (approximately 350,000 to 400,000 cubic yards) to maintain the beach at Rockaway. Therefore, it is likely that over the 50-year project evaluation period that WOPFC will best be described by beach conditions with a narrower beach and no dune. The XBEACH simulations assumed that the post-Sandy bed elevations (e.g. no dune / narrow beach) are representative of the WOPFC.



XBeach is also well suited to calculate sediment transport and morphology changes during storm events. However, in this study sediment transport and morphology processes were not included in the model simulations to simplify and streamline the study. It is also believed that sediment transport and morphology processes play a minor role in evaluating cross-island flooding in WOFPC (e.g. no dune).

It is beyond the scope of the present study to complete a full model calibration for the XBeach model. Instead a desk top validation analysis was performed to compare the model response to Hurricane Sandy conditions.



Figure 7-1: Pre- and Post-Hurricane Sandy Comparison at Rockaway Beach (USGS)

7.2 XBEACH Model Setup

7.2.1 XBeach Description

XBeach (eXtreme Beach behavior) is a two-dimensional model for short wave envelope propagation, nonstationary shallow water equations, sediment transport and bed update. XBEACH is a public-domain model that has been developed with funding and support by the US Army Corps of Engineers, by a consortium of UNESCO-IHE, Deltares, Delft University of Technology, and the University of Miami.

XBeach uses an innovative time-dependent wave action balance solver, which solves the wave refraction (bottom and current) and allows variation of wave action in x, y, time, and over the direction space (Roelvink et. al, 2010). The wave action balance solver is used to simulate the propagation and dissipation of wave groups (infragravity waves). The directional distribution of the action density is taken into account, whereas the frequency spectrum is represented by a single representative frequency. Several wave dissipations models are available for use in the nonstationary wave action balance.

XBeach uses the Generalized Lagrangean Mean (GLM) approach to solve the nonstationary shallow water equations. This approach was implemented to represent the depth-averaged undertow and its effect on bed shear stresses and sediment transport.

XBeach also includes sediment transport, morphology, and many other advanced features (i.e. groundwater flow, non-hydrostatic waves, etc.) that are not included in this study. A complete description of the XBeach is available in *"Modelling storm impacts on beaches, dunes and barrier islands"* (Roelvink et. al, 2009) or the "*XBeach Manual"* (Roelvink et. al, 2010).

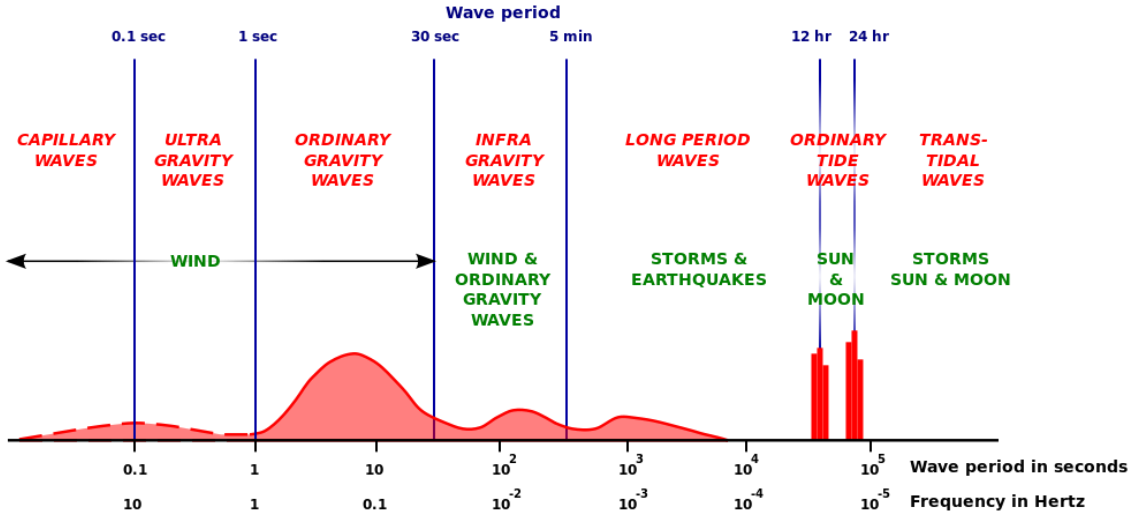


Figure 7-2: Ocean Wave Spectrum Classification (Munk, 1950)

7.2.2 XBeach Model Grid & Bathymetry

A rectilinear model grid encompassing the entire Rockaway peninsula and part of Long Beach was developed for the study. The model extends offshore beyond the seaward edge of the surf zone and includes a portion of Jamaica Bay and Hempstead Bay.

The modeling domain does not include Rockaway Inlet and therefore does not include the complex processes at this inlet. The selected modeling approach simplifies the boundary conditions and allows greater flexibility and control over the back bay water elevations. One of the limitations of this approach is that the back bay water elevations are uniform along the boundary.

Preliminary modeling simulations were performed with a separate and larger modeling grid that included Rockaway Inlet, however such a modeling domain would need to be calibrated to ensure that the tide and surge propagation into Jamaica Bay were correctly predicted by the model. A larger modeling domain would have been inefficient with too much burden on computational resources

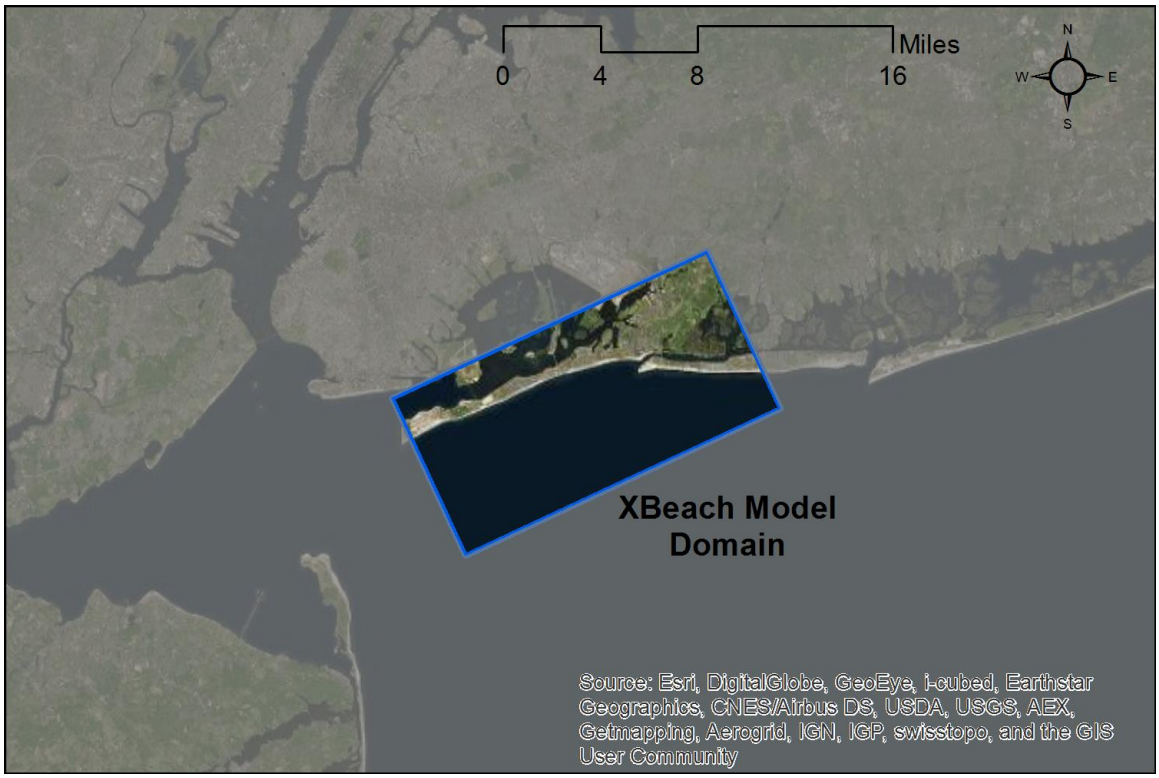


Figure 7-3: XBeach Model Domain

The model grid resolution ranges between 48 meters offshore to up to 12 meters along the shoreline and upland areas. Several data sources were used to create the model bathymetry:

- LIDAR data collected by the USACE on 2012-11-16 and New York State over several weeks in the winter of 2012;

- ACNYMP Beach Profiles;
- Multibeam survey of East Rockaway Inlet performed by Rogers Surveying in 2005.
- FEMA ADCIRC Modeling Grid.

All the bathy data was converted to a consistent spatial coordinate system (UTM 18N) and vertical datum (meters NAVD) prior to use. In addition, the "bare earth" LIDAR data was modified to remove debris piles in areas such as Jacob Riis Park, boardwalk debris, and locations where the LIDAR data picked up the elevation of the overpasses.

All the bed elevations in Jamaica Bay were set to -15 ft NAVD effectively removing the marshes and small islands in the bay. This simplification of the bed elevations in the bay allows the application of a simple boundary condition along the bay. The model bathymetry along the offshore boundary was also adjusted to ensure a uniform depth along the boundary. These adjustments do not significantly impact the solution in the area of interest, Rockaway Beach.

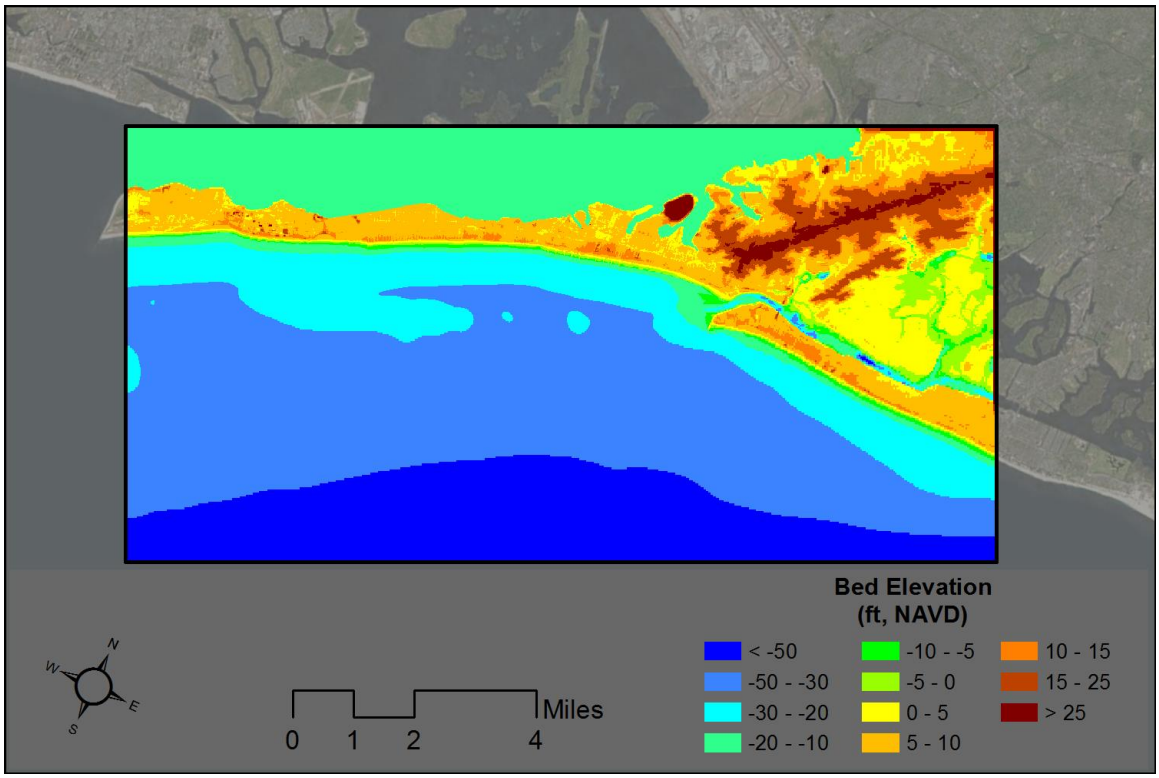


Figure 7-4: XBeach Model Bathymetry

7.2.3 XBeach Boundary Conditions

7.2.3.1. Flow

The flow boundaries for the model are the offshore, bay, and lateral boundaries. Weakly-reflective boundaries (absorbing boundary) are applied on the offshore and bay boundaries with a separate

time series of water elevations applied uniformly along each boundary. The lateral flow boundaries are prescribed as Neumann or "no-gradient" boundaries. The Neumann boundary preserves the alongshore gradient in the water level and longshore currents.

7.2.3.2. Waves

Two types of offshore wave boundaries are applied in the model simulations. In the Hurricane Sandy validation simulations time-varying directional spectrum output from SWAN simulations at single location near the center of the offshore boundary are prescribed. Details of the SWAN model are provided in a Memorandum for Record titled "Task 2 – Incorporate Sand Scale Storm in the Evaluation" prepared by Hampson & Jimenez and dated December 4, 2013. In the production simulations of synthetic storm events the wave conditions are specified by the bulk parameters (significant wave height, peak wave period, mean wave angle, peak enhancement factor for the JONSWAP spectrum, and direction spreading coefficient). In both instances the wave conditions are specified in 1 hour blocks with a slight taper (100 seconds) between the blocks.

7.2.4 XBeach Model Settings

With the exception of the bed roughness, the default settings for the XBeach model are applied. A spatially varying map of the bed roughness (Chezy coefficient) was used in the model to capture the change in the roughness over land due to vegetation and urban infrastructure. Over water a Chezy coefficient of 55 $m^{1/2}/s$ is used and over land a value of 15 $m^{1/2}/s$. The over land Chezy value was selected based on previous work for Reformulation Study which calculated a composite Manning Number of 0.065 (unitless) based on the roughness from vegetation and urban development (*"Seawall Overtopping and Flooding"*). A Manning Number of 0.065 is equivalent to Chezy value of 15.4 $m^{1/2}/s$ in 1-meter of water depth.

7.3 XBEACH Validation

7.3.1 XBeach Validation Boundary Conditions

The flow boundary conditions (water level) for Hurricane Sandy are obtained from two temporary USGS storm tide stations deployed prior to the storm: Sea Gate (SSS-NY-KIN-001WL) and Broad Channel (SSS-NY-QUE-005WL). The Sea Gate storm tide sensor is used to define the ocean water levels and the Broad Channel sensor is used to define the bay water levels. Figure 7-5 shows the locations of the two stations and Figure 7-6 shows the recorded water levels during the storm event. A 1 hour running average filter was applied to the water elevations to remove oscillations caused by waves.

The wave boundary conditions (directional frequency spectrum) are obtained from SWAN spectral wave model output at a location approximately 2 miles offshore of Rockaway. Figure 7-7 presents



the bulk parameters (H_{m0}, T_p, wave direction) obtained from the XBEACH boundary conditions (direction spectrum) for Hurricane Sandy.

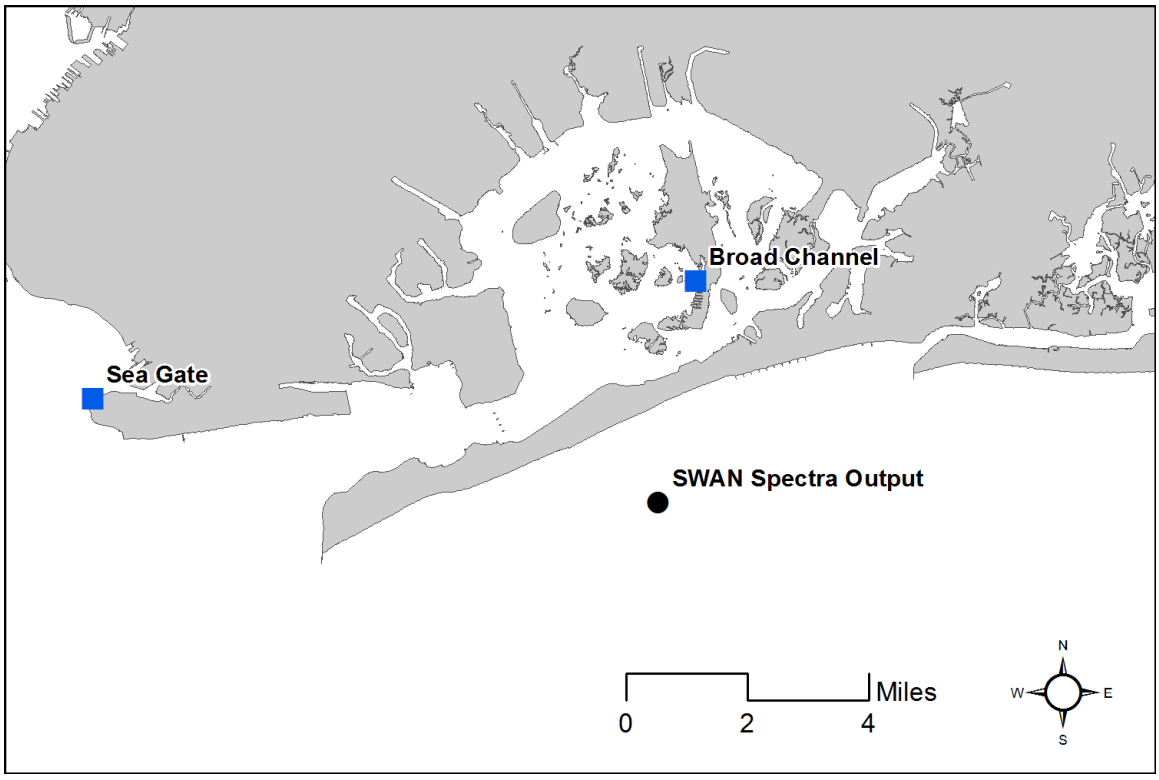


Figure 7-5: Hurricane Sandy Tide and Wave Stations

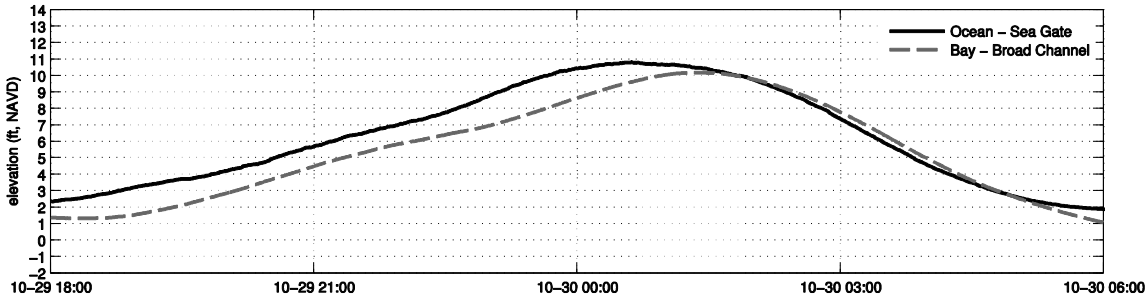


Figure 7-6: XBEACH Storm Tide Boundary Conditions – Hurricane Sandy

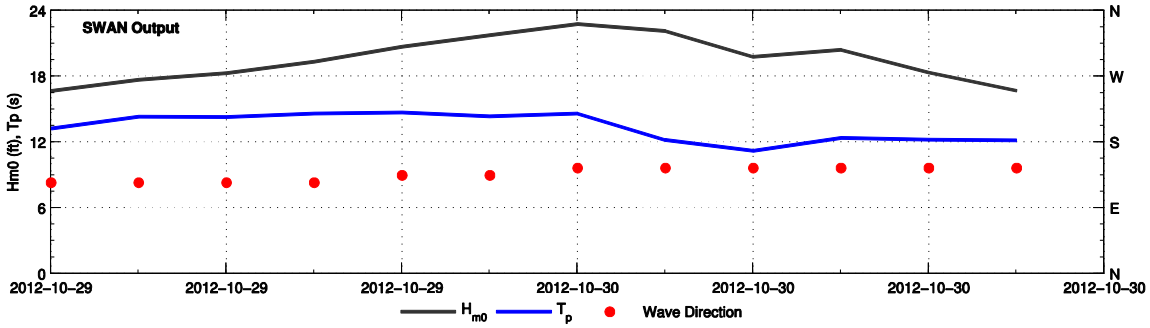


Figure 7-7: XBEACH Wave Boundary Conditions – Hurricane Sandy

7.3.2 XBeach Model Validation Results

Instantaneous snapshots of the water surface and bed elevation during the Hurricane Sandy simulation are shown in Figure 7-8. Only a small portion of the modeling domain, near Jacob Riis Park, is shown in the snapshot. The top panel shows the conditions about 4 hours before the peak of storm, before any noticeable overwash or cross-island flooding has occurred. The bottom panel shows the conditions 1 hour before the peak of the storm, at which time inundation and cross-island flooding has already begun to occur.

The ocean water levels are several feet higher than the bay water levels due to: 1) higher ocean side storm surge, 2) nearshore wave setup. The island is also characterized by a high coastal ridge (+10 ft to +12 ft NAVD) along the shoreline. Landward of this coastal ridge, the ground slopes gradually back towards the bay. Therefore, once the high coastal ridge is overtopped, water flows across the island towards the bay. In instances when nearly the whole peninsula is inundated (e.g. Hurricane Sandy) a gradient in the water level develops between the ocean and bay. Figure 7-9 presents a map of the maximum modeled water elevations during Sandy and Figure 7-10 shows the maximum inundation depths (height above ground) along Rockaway. These maps capture the extent of flooding during Sandy as well as the gradient in the water levels across the peninsula.

The map of maximum water elevations, Figure 7-9, also shows a comparison of the observed and modeled High Water Marks (HWM). Most of the HWM are located near the bay, and thus controlled by the bay water levels. However, there are two HWM located closer to the ocean, both of which had much higher flood elevations (12.7 ft NAVD). The model is able to reproduce the water elevations at these two locations (12.9 ft and 13.4 ft NAVD). This provides confidence that the model is able to reproduce the gradient in water elevations across the peninsula.

A time series of the modeled water elevation at two of the HWM (QUE-729 and QUE-730) is presented in Figure 7-11. One of the HWM (QUE-729) is located in an area exposed to ocean waves and the other (QUE-730) is not. The light grey line is the instantaneous water elevation at 1 minute intervals and the black line is the filtered water elevation (1-hour running average).

Oscillations from long waves are visible in the time series of the water level at the HWM exposed to ocean waves. This variation is not present in the bay side HWM.

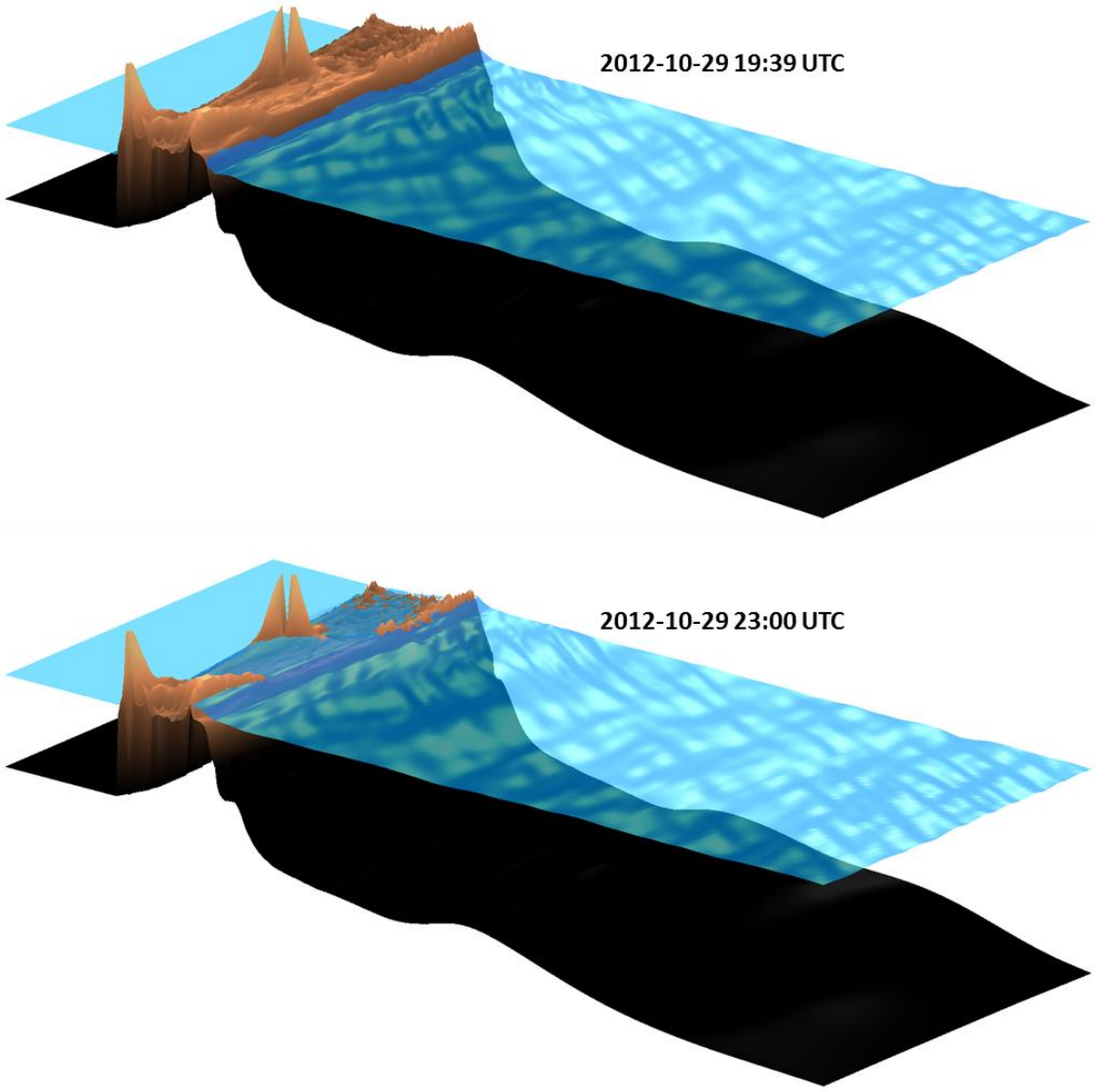


Figure 7-8: XBEACH Snapshot of Water Level near Jacob Riis Park

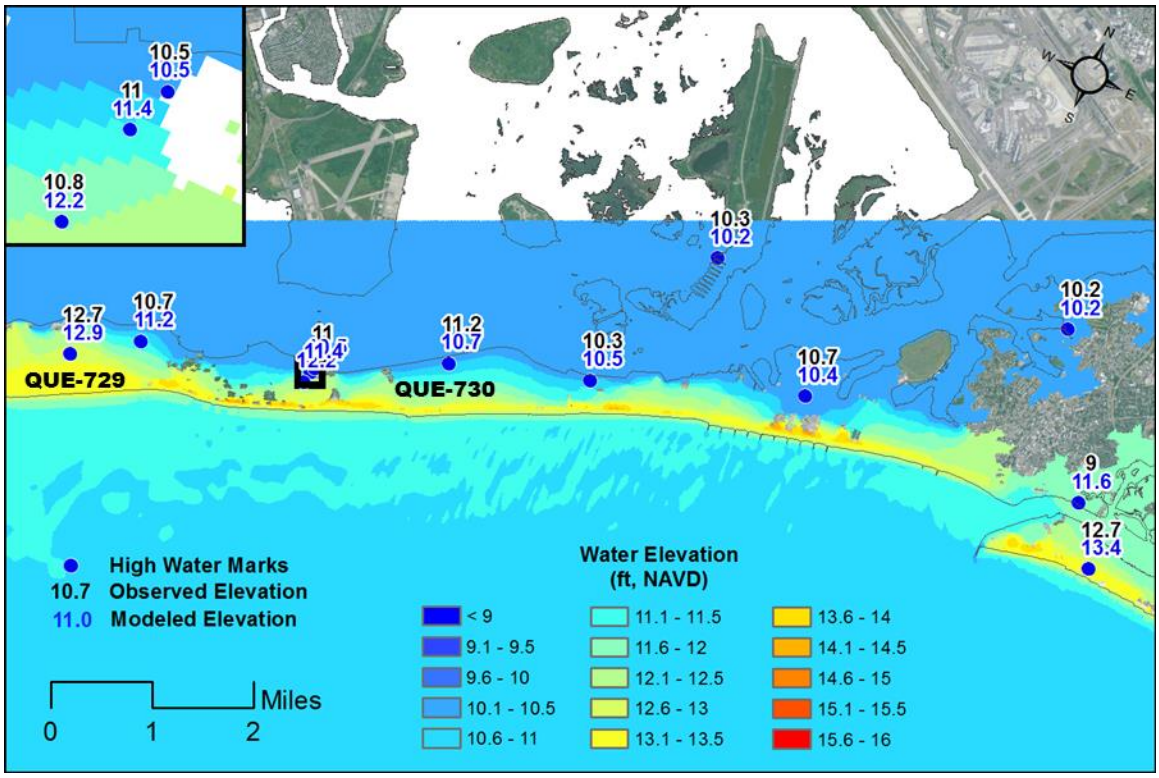


Figure 7-9: XBEACH Modeled Maximum Water Elevations – Hurricane Sandy

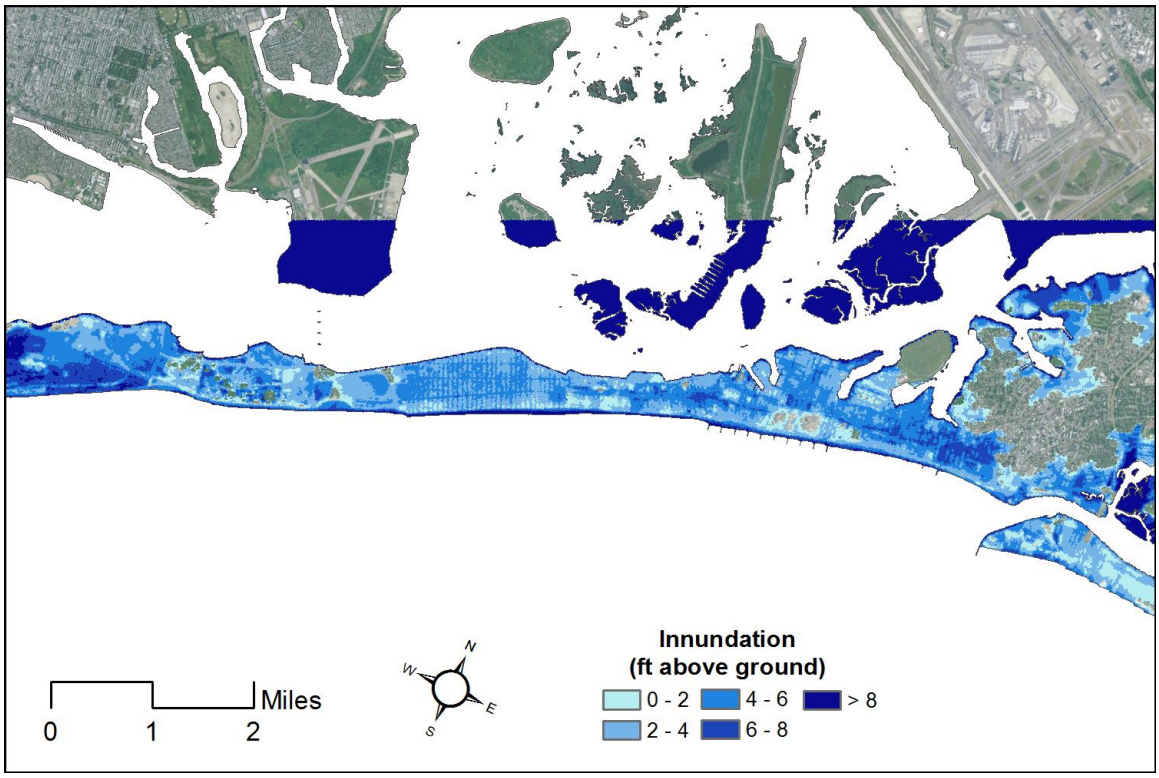


Figure 7-10: XBEACH Modeled Maximum Inundation – Hurricane Sandy

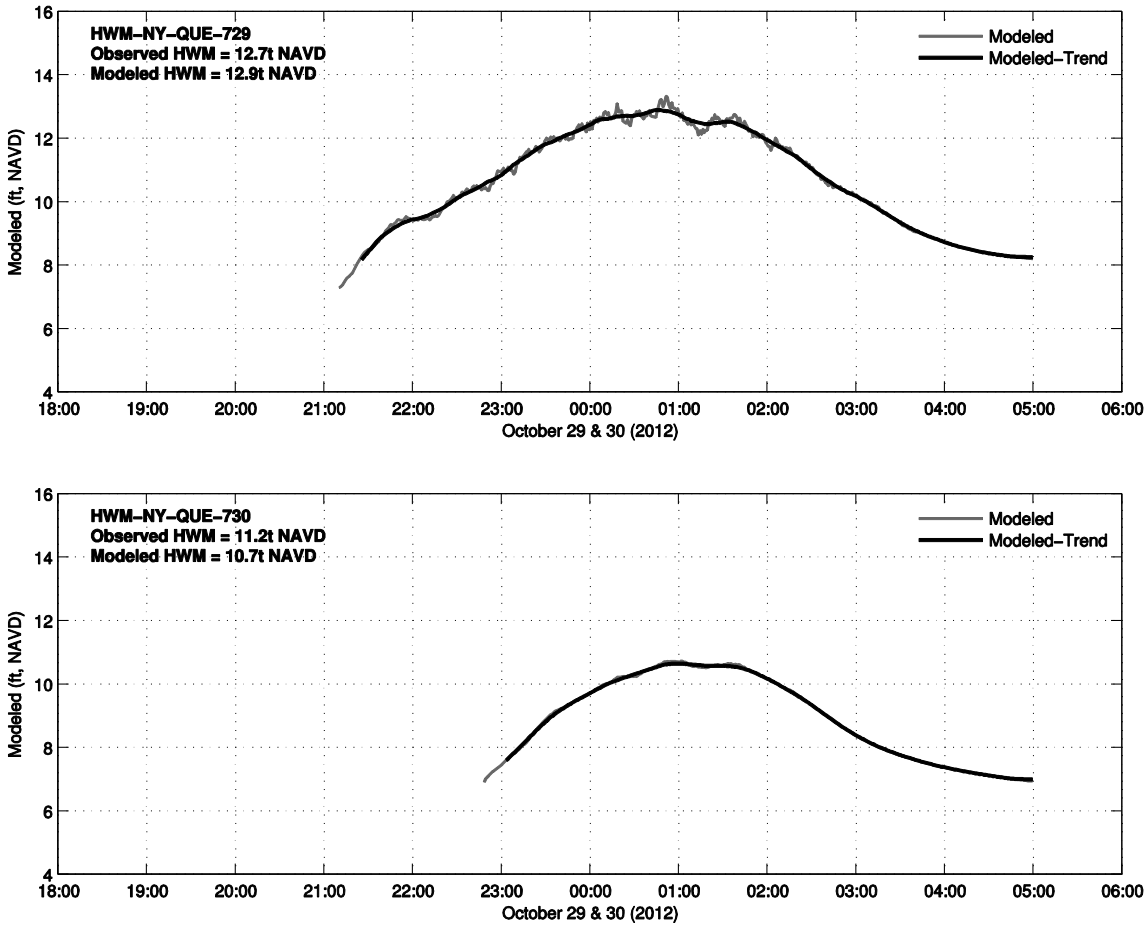


Figure 7-11: XBEACH Time Series of Modeled Water Elevations – Hurricane Sandy

7.4 XBEACH Without Project Simulations

7.4.1 XBeach Without Project Boundary Conditions

XBEACH simulations were performed for four synthetic storm events representing the 10, 25, 100, and 500 year return periods. The shape of the synthetic storm hydrographs is developed following the same approach applied by Larson & Kraus (1989) and utilized mathematical functions to represent the evolution of the surge during hurricanes and nor'easters. The basic premise is that Hurricanes have relatively short storm surge durations when compared to nor'easters. The 10 and 25 year events were simulated as Nor'easters and the 100 and 500 year events were simulated as Hurricanes. Synthetic surge and wave boundary conditions were developed such that the maximum still water level and maximum significant wave height and wave period corresponded to the design conditions as described above and shown in Table 7-1. The development of the design water levels and wave conditions has been documented in earlier reports. Figure 7-12 presents an example of the boundary conditions for a Nor'easter and

Hurricane. Since only the peak 12 hours of the storm are simulated in XBEACH, the significant wave height and peak wave period were held constant during the 12-hour simulations. Only the ocean and bay water levels varied during the simulations.

Table 7-1: XBEACH Boundary Conditions – Synthetic Storms

Return Period (years)	Ocean SWL (feet, NAVD)	Bay SWL (feet, NAVD)	T_p (s)	H_{m0} (feet)
10	6.5	6.6	14	15.3
25	7.9	7.9	14	18.6
100	10.5	9.8	14	23.9
500	14.7	12.3	14	29.3

Ocean and Bay Still Water Levels obtained from FEMA (2013)

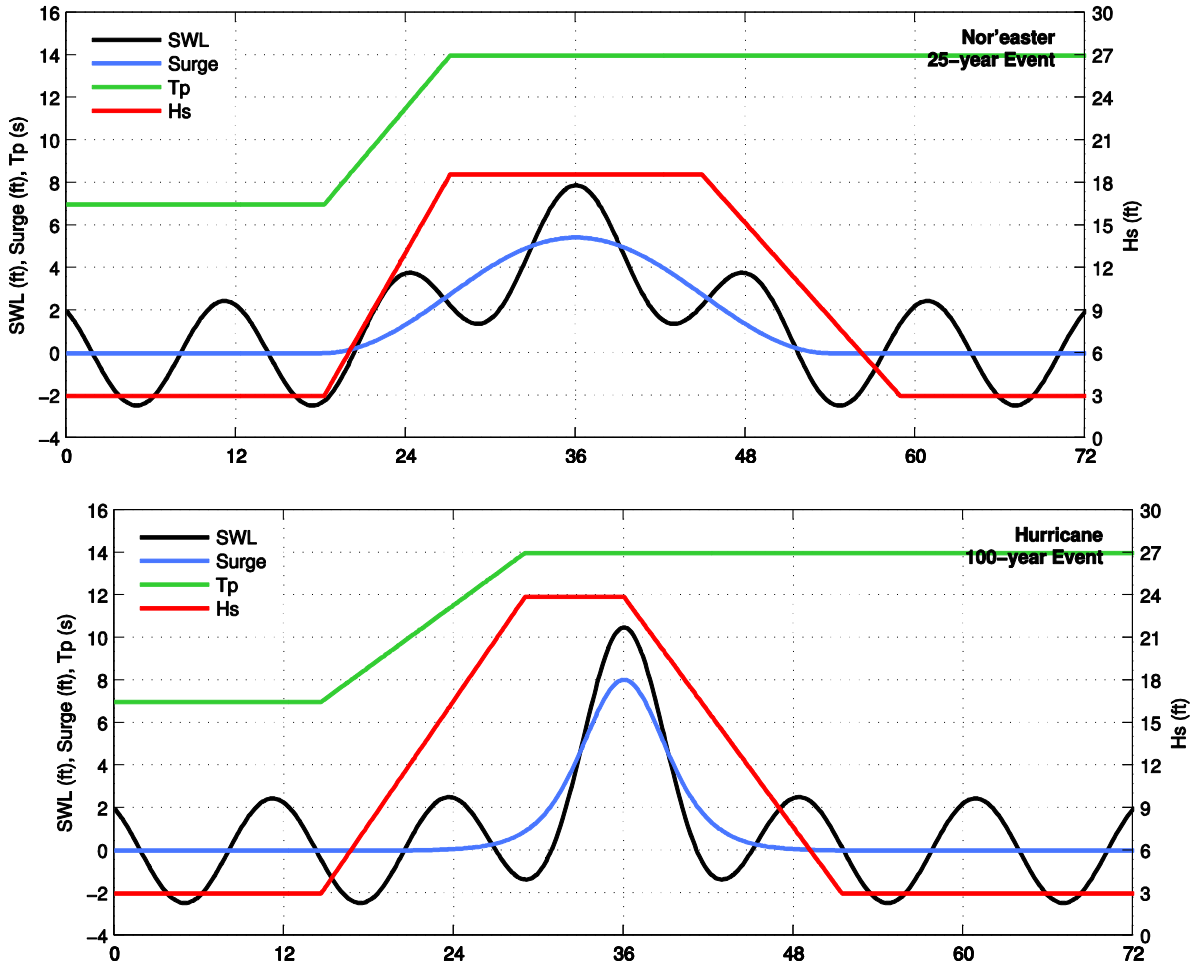


Figure 7-12: XBEACH Synthetic Nor'easter and Hurricane Boundary Conditions

7.4.2 XBeach Without Project Model Results

Results for the four synthetic storm simulations are presented in Figure 7-13. Each panel shows the maximum modeled water elevation during the simulation. The four storm events (10, 25, 100, and 500 year return period) capture four very different levels of flooding. The model results show what areas are most vulnerable as well whether the flooding is caused by the ocean or bay.

During the 10-year event, only a few areas of Rockaway are flooded, most notably Breezy Point, Roxbury, the center of the peninsula, and the eastern end of the peninsula. It appears that both Breezy Point and the center of the peninsula are flooded from the ocean side. However, Roxbury and the eastern end of Rockaway appear to be flooded from the bay. These are the most vulnerable areas along Rockaway and are most likely to flood first during storm events.

During the 25-year event flooding is much more prevalent along the peninsula. Nearly all of Breezy Point is inundated as well as large portions of the heavily developed neighborhoods along the center of the peninsula.

During the 100-year and 500-year events nearly the entire peninsula is inundated, with distinct gradients in the water elevations between the ocean and bay.

The model results, maximum modeled water surface elevation, have been saved as rasters (GeoTiff file) compatible with ArcGIS. These results will be used as inputs to the Non-Shorefront economics model.

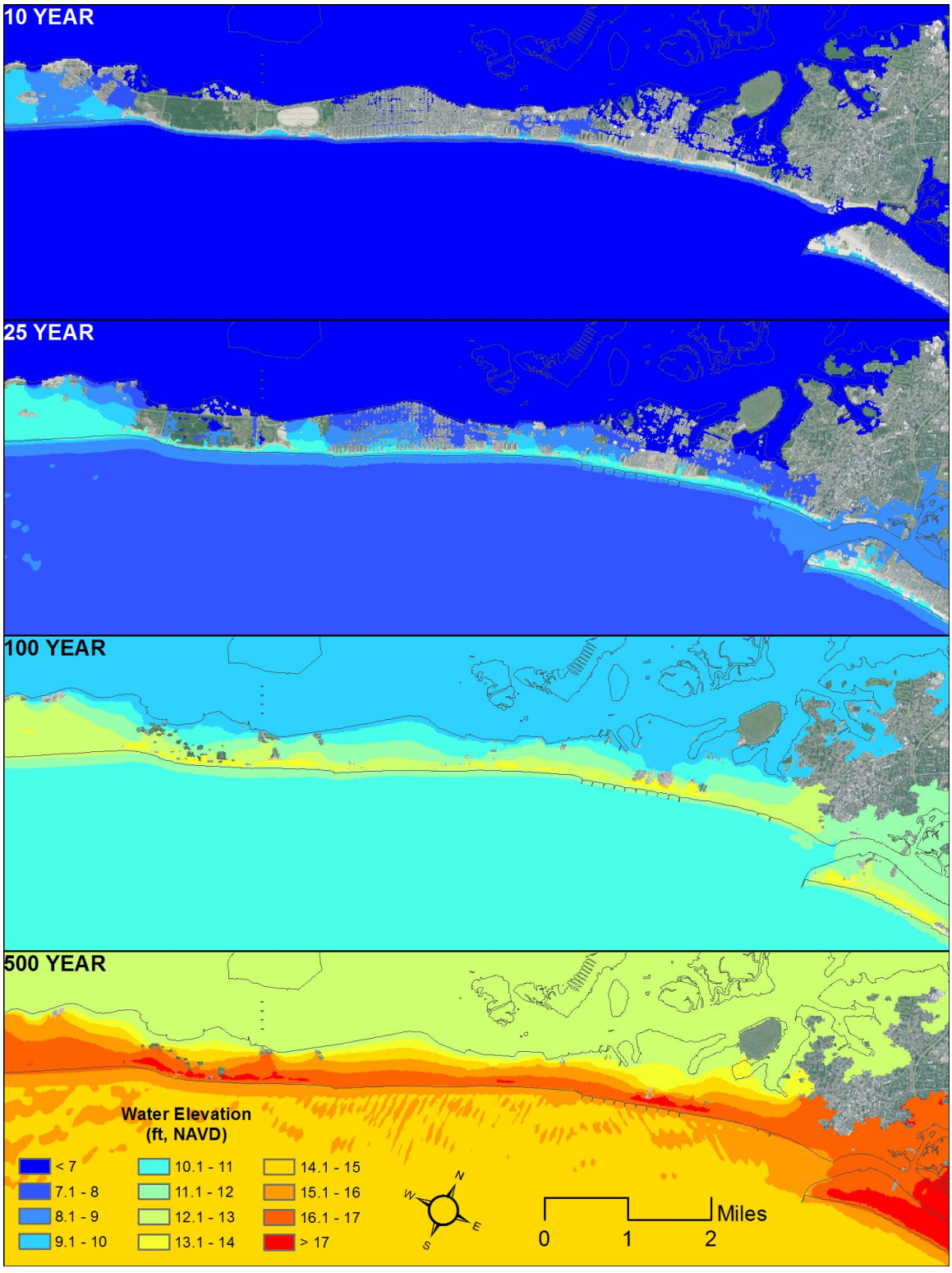


Figure 7-13: XBEACH Modeled Maximum Water Elevations – Synthetic Storms

7.5 XBEACH With Project (Seawall) Simulations

XBeach model simulations were also performed to better understand the potential flooding behind the seawall if the dunes or seawalls are not tied in to high ground. The Hurricane Sandy validation simulation was repeated with an impermeable barrier along shore shoreline from Reaches 3 to Reach 6. However, flood waters are able to flank the impermeable barrier and flood the areas around the seawall. The model results provide insight into the increase in flood elevations, above the back bay flood elevations, that could be attributed to the ocean flooding flanking the dunes or seawall. Figure 7-14 shows the flood elevations during Hurricane Sandy if an impermeable barrier is present along the shoreline. Figure 7-15 shows a comparison of the flood elevations above the back bay flood elevation under the without-project and with-project conditions.

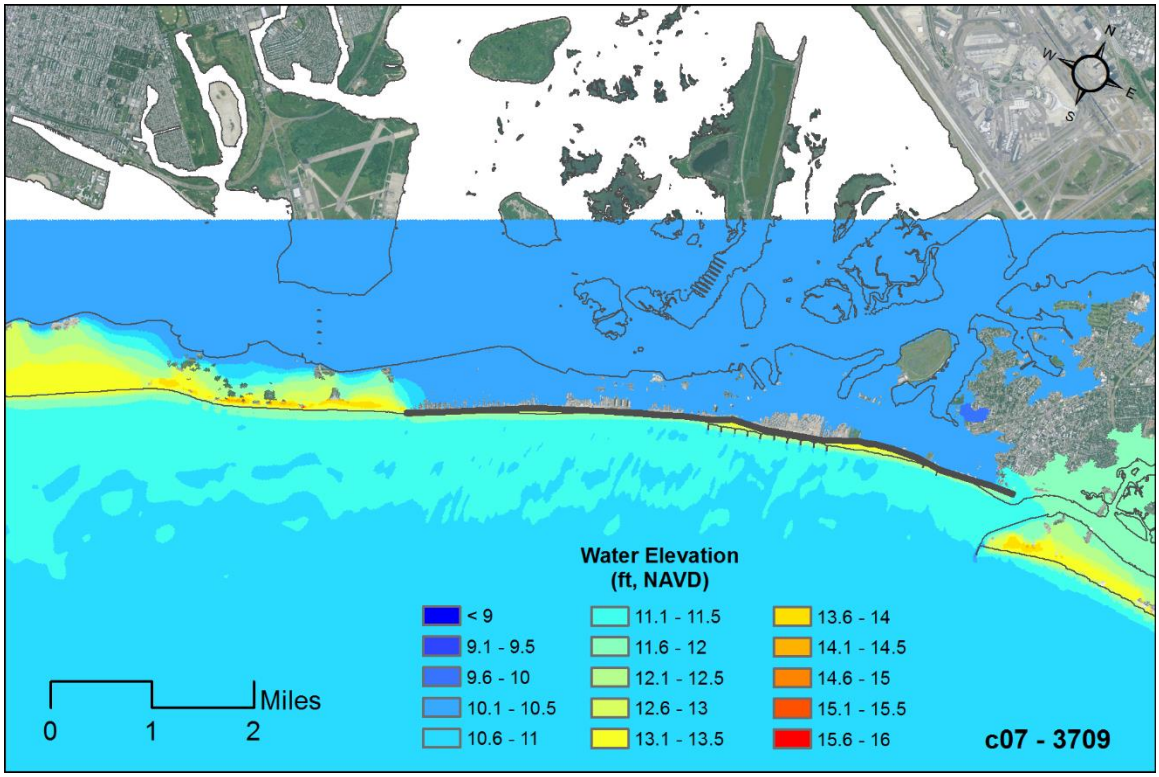


Figure 7-14: XBEACH Water Levels during Hurricane Sandy with Seawall

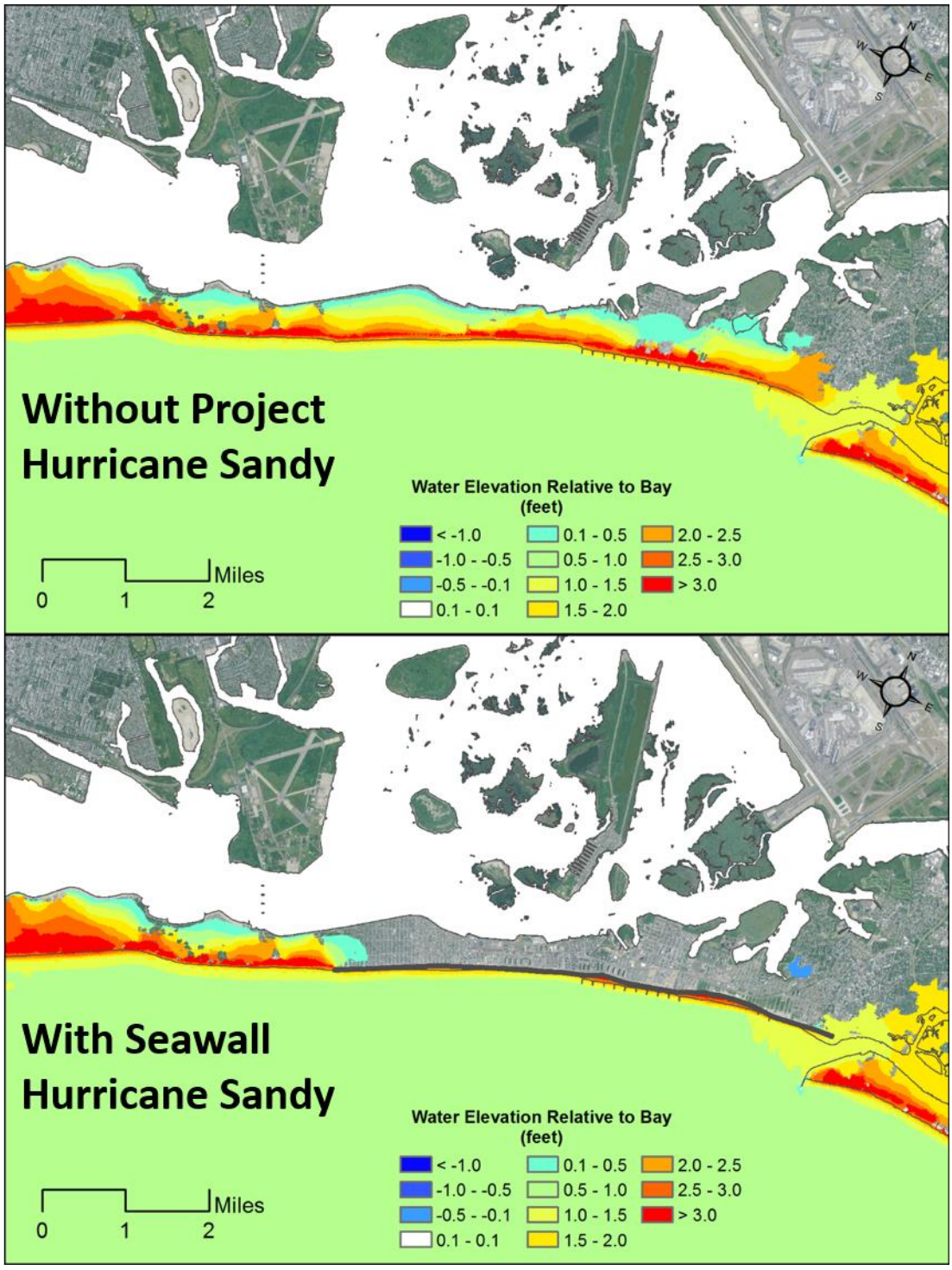


Figure 7-15: XBEACH Water Elevation Relative to Bay during Hurricane Sandy

8.0 SHORELINE CHANGE MODELING (GENESIS)

The GENESIS-T model was applied to better understand the historical longshore sediment transport and erosional patterns along the Rockaway Study Area and to evaluate numerous shoreline stabilization and restoration alternatives.

Shoreline, survey, sediment, and engineering activities data was compiled and studied to understand the history of the Study Area and develop inputs for the GENESIS-T model. Wave data from multiple sources was also compiled to develop a long-term wave climate for use in GENESIS-T.

To establish the appropriate model parameters for the existing system, the GENESIS-T model was calibrated for an April 1996 to August 2010 time period using historical shoreline positions and coinciding wave data. Model parameters established during the calibration were then used to determine the future without project conditions as well as the impact of various shoreline stabilization and restoration alternatives (with project conditions), helping to determine the best solution to the erosion problems along the Rockaway shoreline.

The GENESIS-T model coverage extends from the terminal groin at Rockaway Inlet east to East Rockaway Inlet, covering approximately 10.5 miles along the Rockaway shoreline. This area has typically been divided into 6 reaches based on shoreline change patterns and engineering structures. Figure 8-1 shows the GENESIS-T model extent and reaches along the study area.

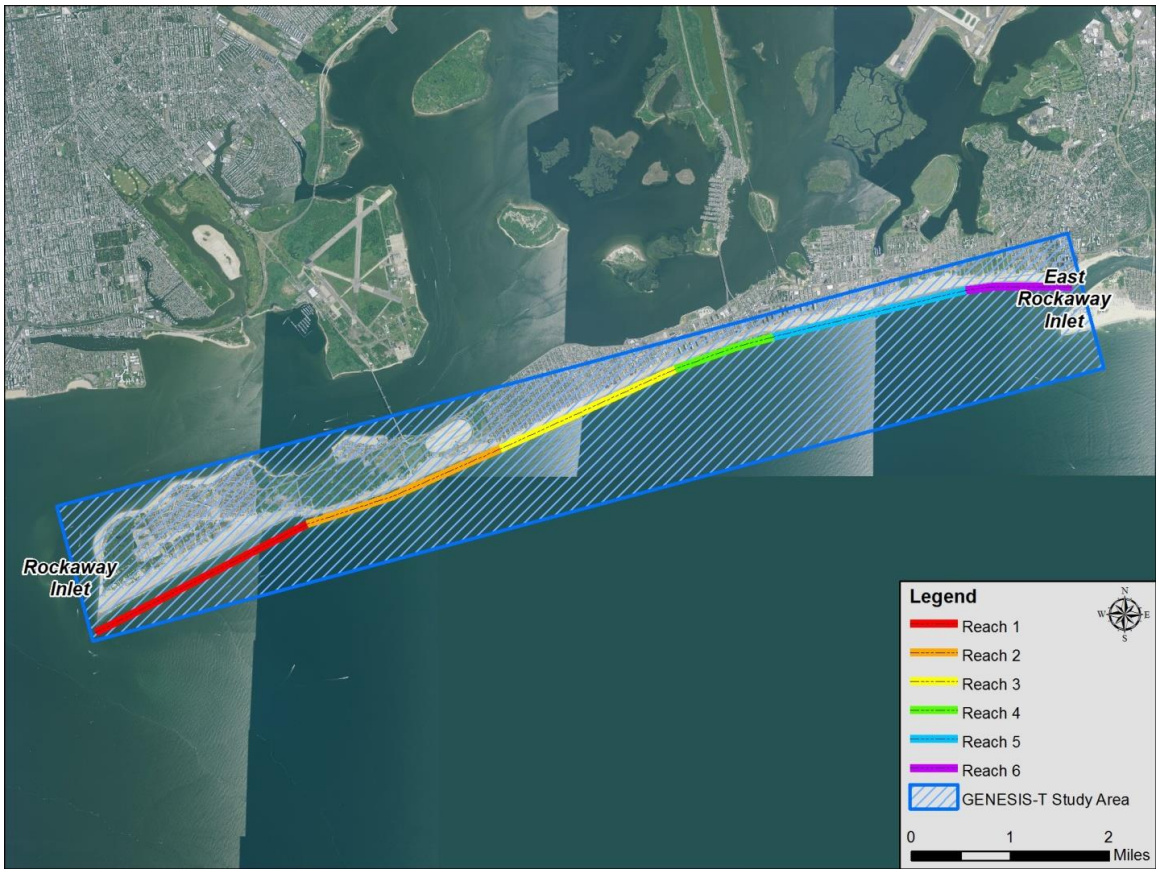


Figure 8-1: GENESIS-T Model Extent

8.1 GENESIS Overview

GENESIS (Generalized Model for Simulating Shoreline Change) is designed to simulate long-term shoreline change based on spatial and temporal differences in longshore sediment transport induced primarily by wave action. The GENESIS modeling system allows for a number of user-specified inputs including wave inputs, initial shoreline positions, coastal structures and their characteristics, and beach fills; all of which aid in the calculation of sediment transport and shoreline change. This model was developed at the U.S. Army Corps of Engineers (USACE), Engineer Research and Development Center (ERDC), Coastal & Hydraulics Laboratory (CHL). For a more detailed description of the GENESIS model, the reader is referred to the User's Manual and Technical Reference published on the model (Hanson and Krauss, 1989, Gravens et al, 1991).

GENESIS-T is a more recent release that expands on the modeling capabilities of GENESIS, allowing for the formation of tombolos at detached breakwaters and/or T-groins. After a comparison of GENESIS and GENESIS-T, it was decided to use GENESIS-T for this study due to the possible alternatives to be studied. It was determined in previous studies by M&N that GENESIS and GENESIS-T results yield almost identical predicted shorelines in the absence of tombolo formation.

GENESIS-T operates within the Coastal Engineering Design and Analysis System (CEDAS), a suite of tools developed by Veri-Tech, based on various numerical models and codes developed at CHL. GENESIS and GENESIS-T run through the Nearshore Evolution Modeling System (NEMOS), which is designed to ease in the preparation of data inputs, analysis, and manipulation for a number of related coastal models.

The GENESIS-T model has potential for many applications in the coastal environment, including evaluation of longshore sediment transport, analysis of beach fill performance, or the analysis of the impact of coastal structures on shoreline change.

The main inputs to the GENESIS-T model include:

- Shoreline Position Data – one-dimensional description of the shoreline position relative to a straight baseline position,
- Wave Data – long-term time dependent description of wave heights, periods, and directions applicable to the Study Area,
- Coastal Structures – position and characteristics of coastal structures (breakwaters, groins, jetties, or seawalls) acting along the Study Area,
- Beach Fill – starting and ending dates and location of beach fill defined by an added berm width,
- Bypassing – starting and ending dates and location of bypassing defined by the quantity in cy/hr
- Sediment and Beach Characteristics – effective grain size, average berm height, and closure depth for the Study Area,
- Sediment Transport Parameters – used to characterize longshore sediment transport and calibrate the model,
- Boundary Conditions – seaward boundary conditions for the input wave data and lateral boundary conditions for the shoreline (left and right), and
- Regional Contour – an offshore contour to account for bathymetry which may affect wave direction/energy

While GENESIS-T can be a very useful tool for predicting future shoreline change due to the impacts of engineering activities (i.e. beach nourishment, structures, etc.), there are some limitations to the model, making it best to follow up with additional modeling or analytical techniques. The main limitations include:

- Longshore Transport Only – The one dimensional model does not account for cross shore transport when predicting shoreline change and shoreline position, forcing the user to assume all change is based on longshore transport, and

- Performance around Inlets – Coastal processes which occur around inlets are very complex and not able to be modeled with a one dimensional model. In addition to the lack of cross shore transport in the model, there is also no "sink" component whereby sediment that may deposit in the inlet shoals is removed from the system.

8.2 GENESIS Calibration

The GENESIS-T model was calibrated to reflect the historical trends of longshore sediment transport and the resulting shoreline change over the Study Area. The overall calibration time period was based on the availability of shoreline data, wave data, and knowledge of engineering activities (i.e. beach nourishment, structure construction, etc.) in the area. The calibration period was chosen, based on the coincidence of the above mentioned data, from April 1, 1996 to August 13, 2010. The model was setup with a 60 ft grid spacing over 55,260 ft of shoreline (921 grid cells).

For this study, the general calibration procedure involved:

- Establishing known model inputs including shoreline position, waves (height, period, and direction), locations of structures, sediment and beach characteristics (grain size, berm height, depth of closure), beach fills, bypassing, and boundary conditions,
- Establishing initial sediment transport parameters and adjusting these parameters until the relative shoreline response (erosion/accretion) and sediment transport rate matched historical trends,
- Adjusting the groin permeability until the shoreline response better matched historical trends around the structures, and
- Adjusting the regional contour to account for bathymetric influences on sediment transport and coastal processes near the inlets

Over 100 model runs were completed during the model calibration process. The input data for the calibration and calibrated model settings will be presented in the following sections, along with a discussion of the results and issues encountered during the calibration process.

8.2.1 Shoreline Position Data

For shoreline input, GENESIS-T requires the shoreline be specified in a station-offset formulation whereby the station represents a position along a landward baseline and the offset is the perpendicular distance from this baseline to the shoreline. The initial shoreline used in the GENESIS-T model was the Spring 1996 wet/dry shoreline, digitized from aerial photography. An exact date was not known for the aerial photography so it was assumed to be April 1, 1996. The final reference shoreline to which the model was calibrated was the August 13, 2010 shoreline, based upon a MHW contour derived from LiDAR data. It should be noted that the two shorelines



were derived from different sources, thereby introducing some uncertainty to the calibration process. The initial and reference shoreline positions used in the GENESIS-T calibration model can be seen in the calibration results.

8.2.2 Wave Data

GENESIS-T requires a continuous record of wave conditions (wave height, wave period, and wave direction) at one or more stations along the modeling domain to drive the long-term shoreline changes. The wave conditions may be defined in relatively deepwater and transformed to nearshore using the GENESIS-T internal wave model or transformed to multiple nearshore stations using an external wave model. The latter approach (external wave model) was applied in this study to capture the alongshore variation in the wave conditions.

The first step in defining the wave conditions, is constructing a continuous record of the offshore wave conditions. Wave data in the study area was available at two NDBC wave buoys in addition to the USACE Wave Information Studies (WIS) wave hindcast:

- WIS 123 (1980-01-01 to 1999-12-31)
- NDBC 44025 (1991-04-29 to 2013-10-24)
- NDBC 44065 (2008-10-30 to 2013-10-05)

All three stations contained information about the wave height, wave period, and wave direction at hourly (WIS) or 3-hour (NDBC) intervals. As shown in Figure 8-2 the locations of the NDBC wave buoys and WIS station differ. The wave conditions at NDBC 44025 buoy were transformed to the closer buoy location, 44065, with a spectral wave model (SWAN). The WIS data was combined with the transformed wave data at the 44065 station to create a continuous record of the wave conditions from 1980-2013.

The next step was to transform the offshore wave conditions to 58 nearshore wave stations along the Rockaway shoreline, typically around the depth of closure (~25 ft). Figure 8-3 shows an example of the nearshore wave stations used along the eastern end of the GENESIS-T modeling domain. A nearshore wave model (SWAN) was used to build a look-up table that establishes relationships between nearshore wave conditions and offshore wave conditions.

The purpose of using multiple wave stations along the Rockaway shoreline was to capture the impact of changing wave conditions along the study area. Figure 8-4 shows annual wave roses at the offshore wave station and two nearshore stations near East Rockaway Inlet. It is apparent that the waves near East Rockaway Inlet (GENESIS 002) are of a different direction and magnitude than those away further away from the inlet (GENESIS 008). Capturing the alongshore variability in the wave conditions in this area was critical to reproducing a nodal point in the sediment transport that exists within Reach 6 of the Study Area.



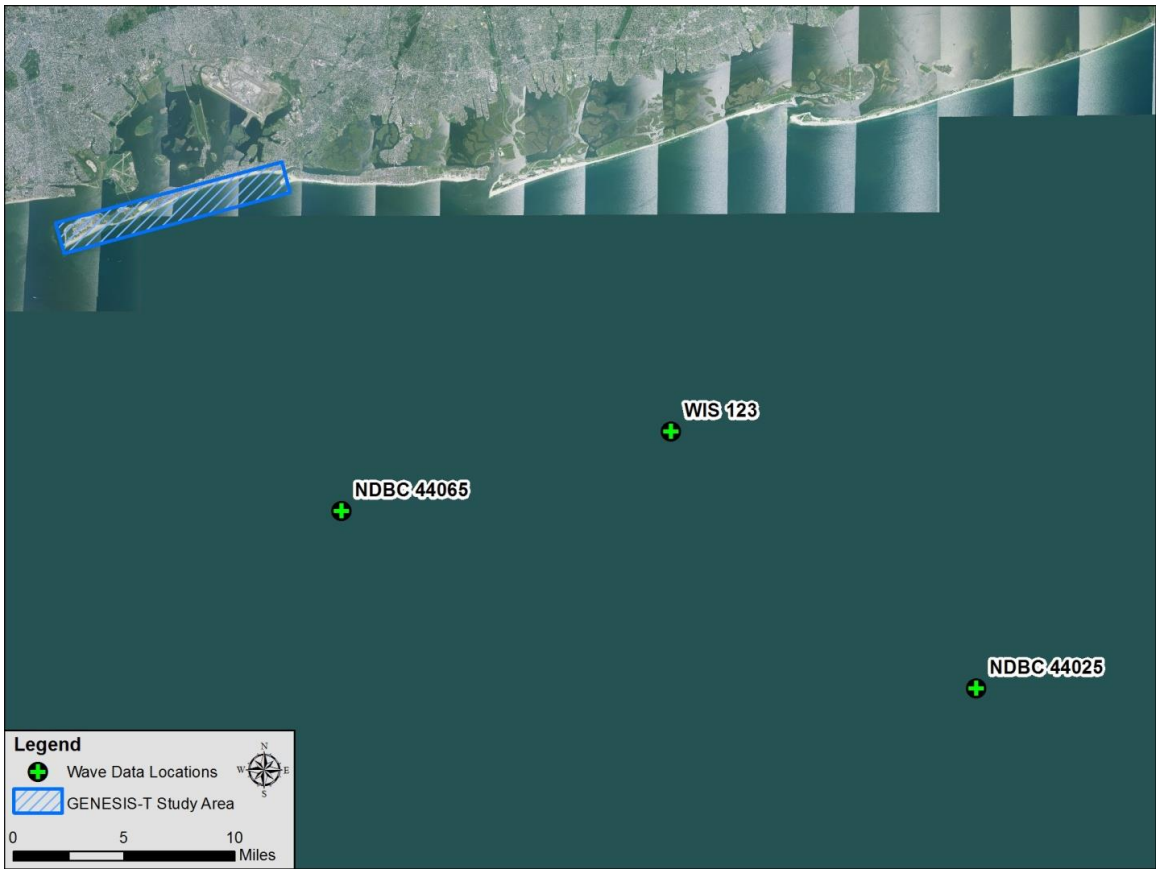


Figure 8-2: GENESIS-T Offshore Wave Data Locations

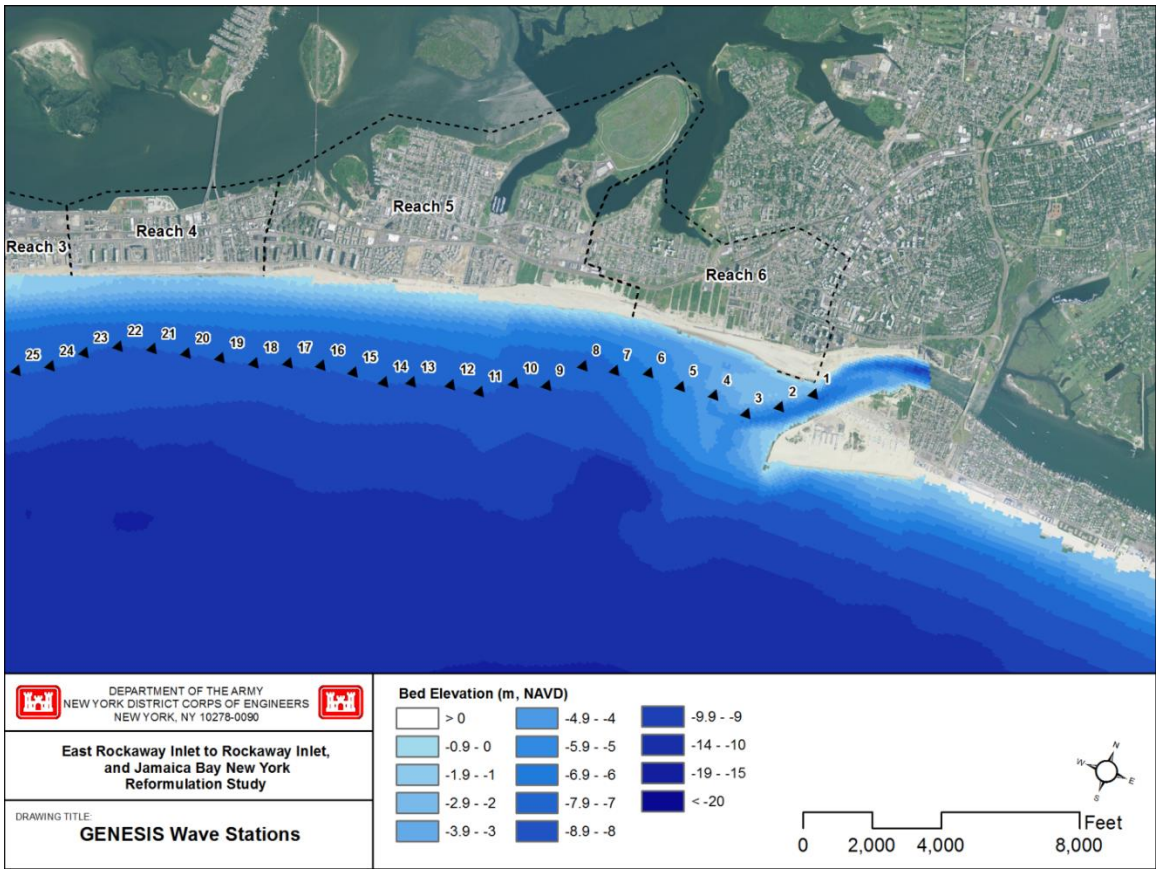


Figure 8-3: GENESIS-T Nearshore Wave Stations

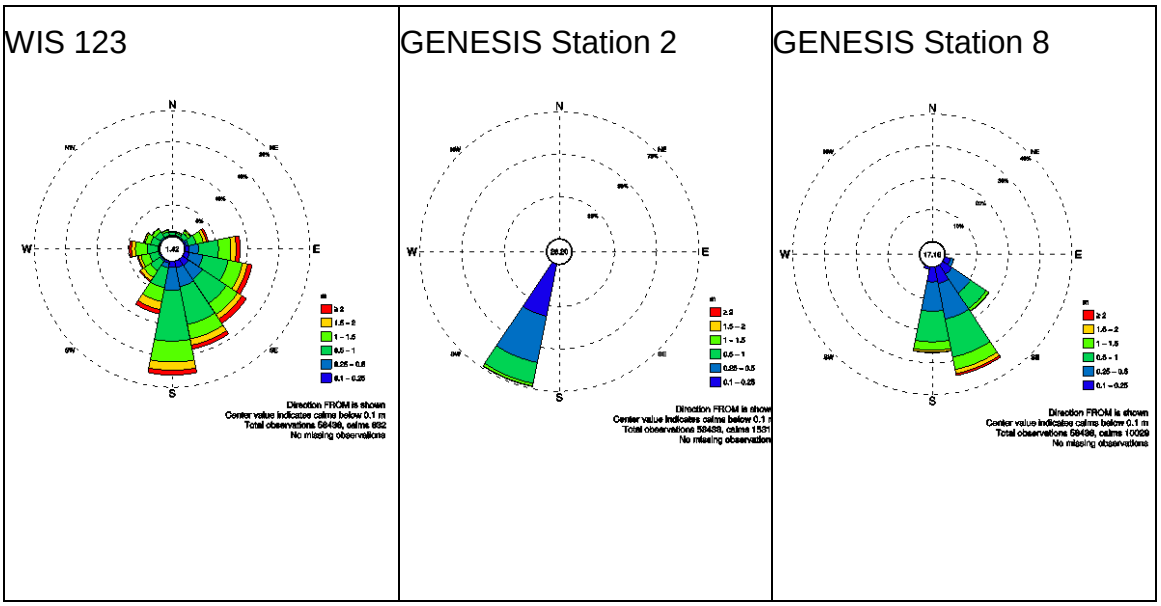


FIGURE 8-4: GENESIS-T REPRESENTATIVE WAVE ROSES

8.2.3 Coastal Structures

GENESIS-T requires the locations and characteristics of nearshore structures as input. The coastal structures are incorporated in the GENESIS-T model by a station-offset formulation, similar to the shoreline position. Allowable structures include non-diffracting groins/jetties, diffracting groins/jetties, seawalls, and/or detached breakwaters. Each structure is modeled uniquely with respect to longshore transport and shoreline change. In general, structures exert two direct effects on the shoreline change modeling:

- Structures extending into the surf zone block a portion, or all, of the longshore transport from their updrift sides and may reduce the transport of sand towards the downdrift side. This effect may be induced by a groin or jetty.
- Structures which have seaward ends extending well beyond the surf zone, including jetties or detached breakwaters, induce wave diffraction which causes the local wave height and direction to change.

Wave transmission through and over breakwaters is controlled by the user-specified transmission coefficient (K_t). The transmission coefficient is defined as the ratio of wave heights on the shoreward side of the breakwater to the incident wave heights on the seaward side of the breakwater and may range from 0 (no transmission) to 1 (complete transmission).

Similar to detached breakwaters, a non-diffracting or diffracting groin implemented in GENESIS-T must have a defined permeability coefficient which controls the transmission of sand over and through the structure. The permeability can range from 0, implying an impermeable structure to 1, implying a completely transparent structure. The literature indicates that the range of permeability coefficients for a groin that has gaps or is overtopped during parts of the tidal cycle is often between 0.1 and 0.5.

The Rockaway calibration model includes 37 existing rock groins within the Study Area (see Figure 8-5). Each groin was assigned a permeability coefficient. Both structure condition and relative permeability to other structures are taken into account but the final number is based on observed shoreline change near each structure during calibration. There are also several dilapidated timber groins that were not included in the model as they are not believed to have an impact on the shoreline. Table 8-1 shows the permeability coefficient determined for each of the 37 groins included in the calibration model. As can be seen, most of the structures have permeability coefficients ranging from 0.2 – 0.5. The rock groins are in fairly good condition, so this matches well with the chosen permeability coefficients and ranges suggested in the literature. The only structure with a differing value was the terminal groin at Rockaway Inlet (0.8). This is the result of trying to match shoreline change processes near an inlet in GENESIS-T and not an indication that the groin is not functional.



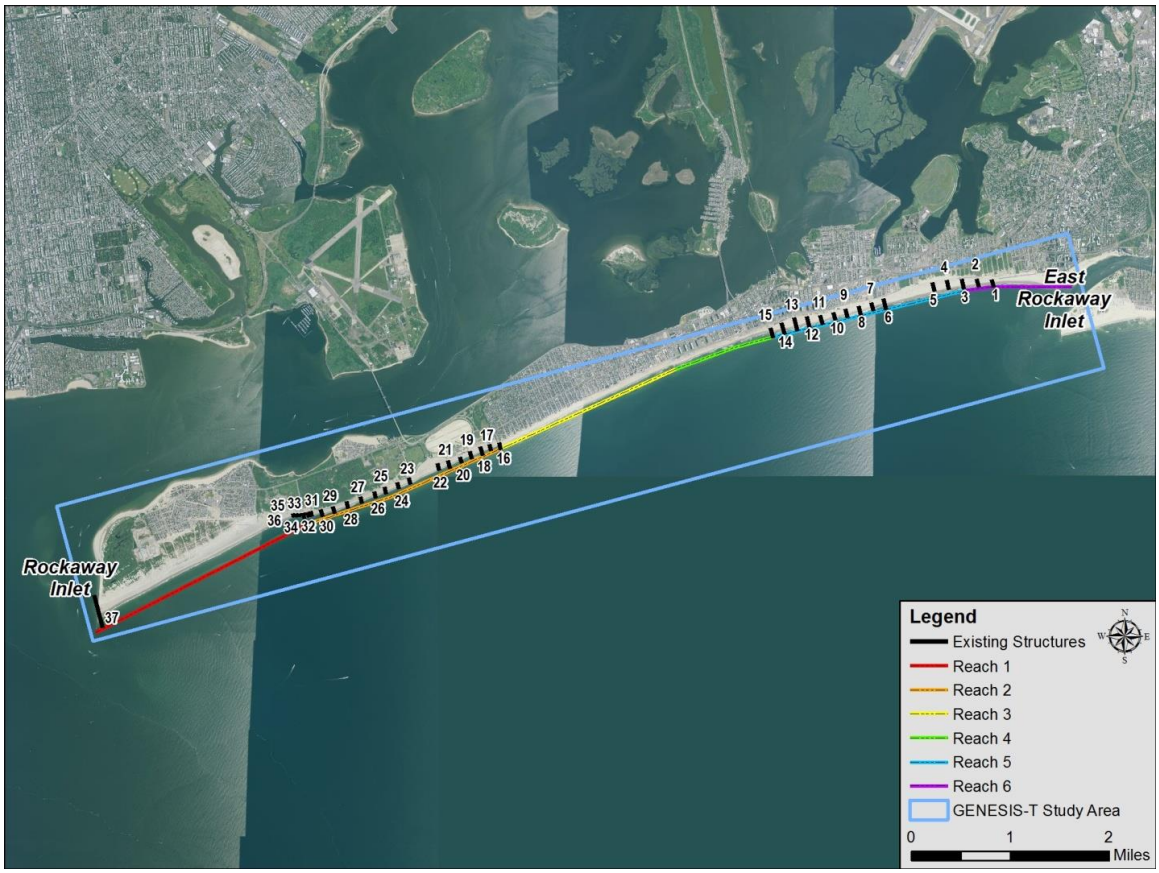


Figure 8-5: GENESIS-T Calibration (Existing) Structures

Table 8-1: GENESIS-T Groin Permeability Coefficients

Groin	K_t	Groin	K_t	Groin	K_t	Groin	K_t
1	0.2	**11**	0.2	**21**	0.3	**31**	0.5
2	0.2	**12**	0.2	**22**	0.3	**32**	0.5
3	0.2	**13**	0.2	**23**	0.3	**33**	0.5
4	0.2	**14**	0.2	**24**	0.3	**34**	0.5
5	0.2	**15**	0.2	**25**	0.3	**35**	0.5
6	0.2	**16**	0.25	**26**	0.3	**36**	0.5
7	0.2	**17**	0.3	**27**	0.3	**37**	0.8
8	0.2	**18**	0.3	**28**	0.3		
9	0.2	**19**	0.3	**29**	0.3		
10	0.2	**20**	0.3	**30**	0.3		

Notes: K_t – groin permeability, 0 (no transmission) to 1 (complete transmission)

8.2.4 Beach Fill Projects

Multiple beach nourishment projects took place during the calibration period from April 1996 to August 2010, including inlet maintenance dredging from East Rockaway Inlet in addition to multiple renourishment projects. GENESIS-T requires, as input, the lateral extents of the nourishment as well as an added berm width. The added berm width was determined based on the active profile height (depth of closure to berm elevation), project length, and volume of nourishment. Table 8-2 shows the beach fill activities that took place during the calibration time period and were included in the calibration simulation. Figure 8-6 displays the location and quantities of each of the nourishment projects.

Table 8-2: GENESIS-T Calibration Beach Fill Data

Year	Authorization	Project Name	Source of Fill	Area of Placement	Total Fill Placed (cy)	Length (ft)	Added Berm Widths (ft)
1996	Section 934 of WRDA 1986	Renourishment #6	Borrow area 1B	B110th St. to B46th St.	1,485,800	15,720	82
1996	Section 934 of WRDA 1986	Renourishment #6	Borrow area 1B	B149th St. to B110th St.	592,800	9,720	53
1998	Rivers and Harbors Act of 1930	Navigation/Operations & Maintenance Dredging	Dredged Material From East Rockaway Inlet	B27th St. to B39th St.	218,000	3,050	62
2000	Rivers and Harbors Act of 1930	Navigation/Operations & Maintenance Dredging	Dredged Material From East Rockaway Inlet	B27th St. to B39th St.	241,000	3,050	69
2000	Section 934 of WRDA 1986	Renourishment #7	Borrow Area 2	B59th St to B74th St	333,075	3,750	76
2000	Section 934 of WRDA 1986	Renourishment #7	Borrow Area 2	B90th St. to B118th St.	676,260	7,800	75
2002	Rivers and Harbors Act of 1930	Navigation/Operations & Maintenance Dredging	Dredged Material From East Rockaway Inlet	B27th St. to B39th St.	140,000	3,050	40
2004-2005	Rivers and Harbors Act of 1930	Navigation/Operations & Maintenance Dredging	Dredged Material From East Rockaway Inlet	B28th St. to B36th St.	221,002	2,100	92
2004	Section 934 of WRDA 1986	Renourishment #8	Borrow Area 2	B118th St. to B110th St.	94,968	2,050	40
2004	Section 934 of WRDA 1986	Renourishment #8	Borrow Area 2	B110th St. to B46th St.	634,557	15,550	36
2004	Section 934 of WRDA 1986	Renourishment #8	Borrow Area 2	B46th St. to B24th St.	271,953	5,400	44
2007	Rivers and Harbors Act of 1930	Navigation/Operations & Maintenance Dredging	Dredged Material From East Rockaway Inlet	B28th St. to B38th St.	260,000	2,600	87
2009	Rivers and Harbors Act of 1930	Navigation/Operations & Maintenance Dredging	Dredged Material From East Rockaway Inlet	B28th St. to B39th St.	285,000	2,850	87
2010	Rivers and Harbors Act of 1930	Navigation/Operations & Maintenance Dredging	Dredged Material From East Rockaway Inlet	B92nd St. to B103th St.	137,000	3,150	38



Figure 8-6: GENESIS-T Beach Fill Project Map

8.2.5 Inlet Bypassing

It is believed that natural sediment bypassing occurs at East Rockaway Inlet, however the quantity and path of the bypassing is not well understood. Previous studies have estimated that between 70,000 cubic yards (OCTI, 2011) and 170,000 cubic yards (USACE Draft Basis of Design) of bypassing occurs. A range of bypassing amounts from 0 cy/yr to 200,000 cy/yr were tested during model calibration. Based on historical references, sediment budget calculations, and modeled shoreline positions, a bypassing rate of 100,000 cy/year was determined to provide the best model calibration. The attachment point, or location in which the bypassing enters the littoral system, was also determined during model calibration. Bypassing is defined in GENESIS-T as a constant sediment flux (cy/hr) over a defined extent. Figure 8-7 shows the alongshore extent of bypassing determined through model calibration.

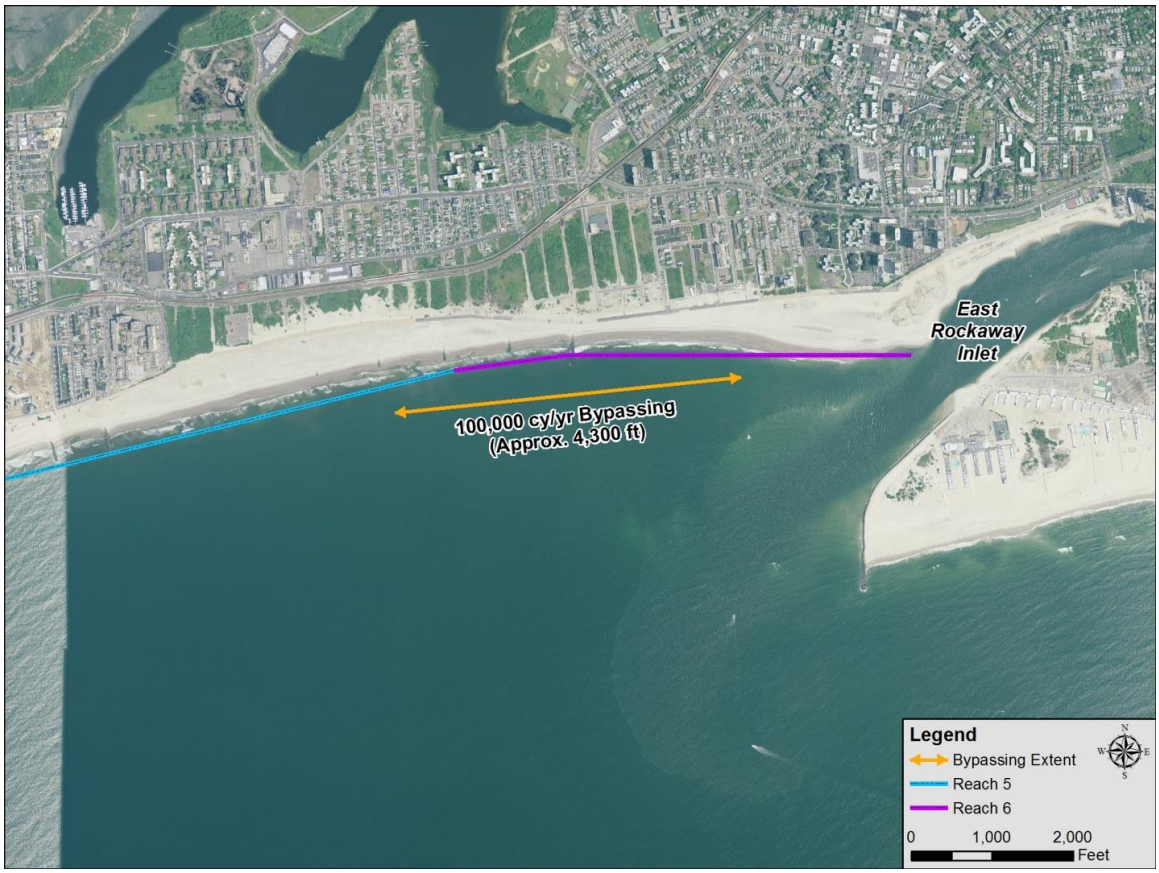


Figure 8-7: GENESIS-T Bypassing at East Rockaway Inlet

8.2.6 Sediment and Beach Characteristics

GENESIS-T requires, as input, the effective grain size (mm), average berm height (ft), and closure depth (ft). The effective grain size assumed was 0.29 mm based on beach sediment samples collected in 1974 (USACE, 2011). The average berm height was set to +6 ft NAVD88 and the closure depth was set to -25 ft NAVD88. These values were determined based on beach profile surveys collected between 1996 and 2003 as part of the Atlantic Coast of New York Monitoring Program (ACNMYP) (USACE, 2011).

8.2.7 Sediment Transport Parameters

An initial calibration run indicated that the nearshore wave stations were properly defining a sediment transport nodal point in Reach 6, moving sediment east of that point into East Rockaway Inlet and west of that point towards Rockaway Inlet. With sediment transport going in the correct direction, the magnitude of the transport was adjusted based on the transport parameters K1 and K2 in GENESIS-T. The transport rate coefficient, K1, is used to control the time-scale and magnitude of the simulated shoreline change, while K2 is used to control shoreline change and longshore sand transport in the vicinity of structures. Although the values of K1 and K2 have been

empirically estimated, these coefficients are treated as calibration parameters in GENESIS-T and range in value from 0 to 1.0 with higher K values producing higher transport rates in the model.

The final calibration transport coefficient values were chosen to be K1 = 0.2 and K2 = 0.2 after applying a range of values from 0.1 to 0.3. These K values allowed for the correct magnitude of sediment transport along the Rockaway shoreline based on historical sediment transport rate studies, sediment budget calculations, and modeled shoreline position. Figure 8-8 shows the resulting transport rates from the GENESIS-T calibration model, with the nodal point occurring midway through Reach 6.

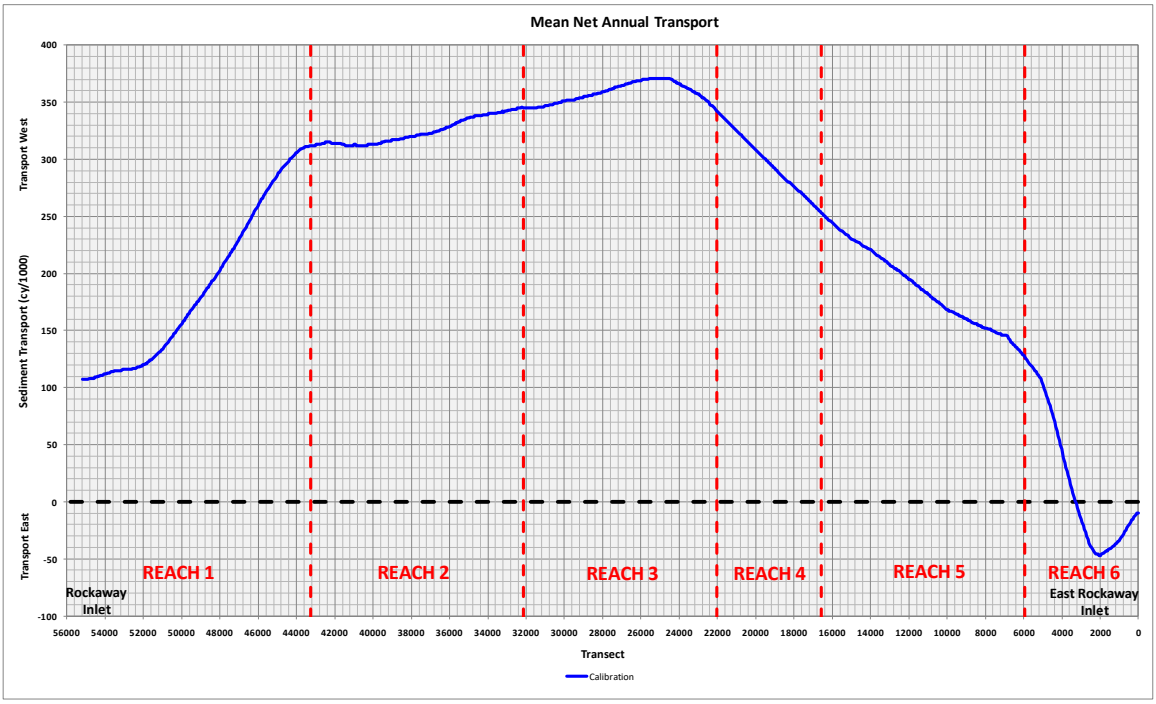


Figure 8-8: GENESIS-T Calibration - Sediment Transport Rate

8.2.8 Boundary Conditions

The required boundary condition inputs for GENESIS-T include the seaward wave data boundary conditions and the lateral boundary conditions at the left (west) and right (east) ends of the shoreline, as described in the following section.

8.2.8.1. Seaward Boundary Conditions

Within the seaward boundary conditions, the user may modify the input wave conditions (wave height and direction) to analyze the impact changes in modeled wave conditions have on the resulting shoreline response. The user may also define an offshore regional contour to account for variations in nearshore bathymetry (landward of the depth of closure) which may cause a change

in wave energy/direction, altering sediment transport patterns along the shoreline from what they might be if these features didn't exist.

During calibration it was determined that the input wave height and direction was representative of nearshore conditions and were not modified by any factors. It was determined that inlet influences near the east and west boundaries of the model were causing shoreline change responses that were not being captured correctly in GENESIS-T. Therefore, the regional contour was used to match the observed shoreline change behavior at the inlets. The regional contour was determined by starting with an actual offshore contour near the depth of closure. Slight adjustments were made to capture any bathymetry features landward of the depth of closure that may impact the shape of the shoreline. Larger adjustments were made near the inlets to help capture the inlet effects on shoreline change processes. Figure 8-9 shows the final regional contour used in the model. As can be seen, the regional contour is parallel to the shoreline along most of the study area, indicating that it does not play a large part in manipulation of the shoreline in the model calibration. The areas nearest the inlets that are not shore parallel are where the regional contour has the greatest impact on the model calibration, helping to better reflect historical shoreline change patterns in these areas.



Figure 8-9: GENESIS-T Regional Contour Location

8.2.8.2. Lateral Boundary Conditions

The west boundary of the model was located at the terminal groin structure adjacent to Rockaway Inlet. A gated (groin) boundary was used at this location. The east boundary of the model was established at the mouth of East Rockaway Inlet where a pinned boundary was established based on the absence of difference between the initial (1996) and reference (2010) shoreline position data at that location.

8.2.9 Calibration Model Results

Figure 8-10 shows a plot of the final shoreline resulting from the GENESIS-T calibration modeling against the initial shoreline position (April 1996) and the comparable measured shoreline position (August 2010). Enclosure B shows the results from the GENESIS-T model calibration overlaid on aerial photography.

As shown in Figure 8-10, the model output matched the measured reference shoreline position fairly well in most areas. It appears to slightly over predict the erosion in Reaches 4, 5, and 6 (eastern portion of the study area) and slightly over predict accretion in Reaches 1, 2, and 3 (western portion of the study area). These differences will be taken into account when assessing the performance of various alternatives. Figure 8-11 shows the measured and modeled shoreline change for comparison. It is apparent that the modeled data closely follows the trends of the measured data even though the modeled shoreline position does not exactly match the measured shoreline position.

Figure 8-12 shows the difference in position between the 2010 measured shoreline and 2010 modeled shoreline. This figure shows the modeled shoreline being landward of the measured shoreline in Reaches 4-6 and seaward of the measured shoreline in Reaches 1-3. Again, these differences will be considered when assessing the performance of various alternatives. Table 8-3 contains some statistics per reach related to the difference in measured and modeled shoreline position results. It is important to note that the amount of change seen in the shoreline from year to year is very considerable. The maximum measured change from 1996 to 2010 was 381 ft (26 ft/yr) and the average measured change was 69 ft (5 ft/yr). Therefore, a calibration model error on the order of 1 to 4 ft/yr, depending on the reach, is considered very reasonable.

Table 8-3: GENESIS-T Measured vs. Modeled Shoreline Error Analysis

Reach	Avg. Error (ft)	Avg. Error (ft/yr)
Reach 1	+64	+4.4
Reach 2	+47	+3.3
Reach 3	+16	+1.1
Reach 4	-22	-1.5
Reach 5	-31	-2.2
Reach 6	-25	-1.7
Overall	+15	+1.1

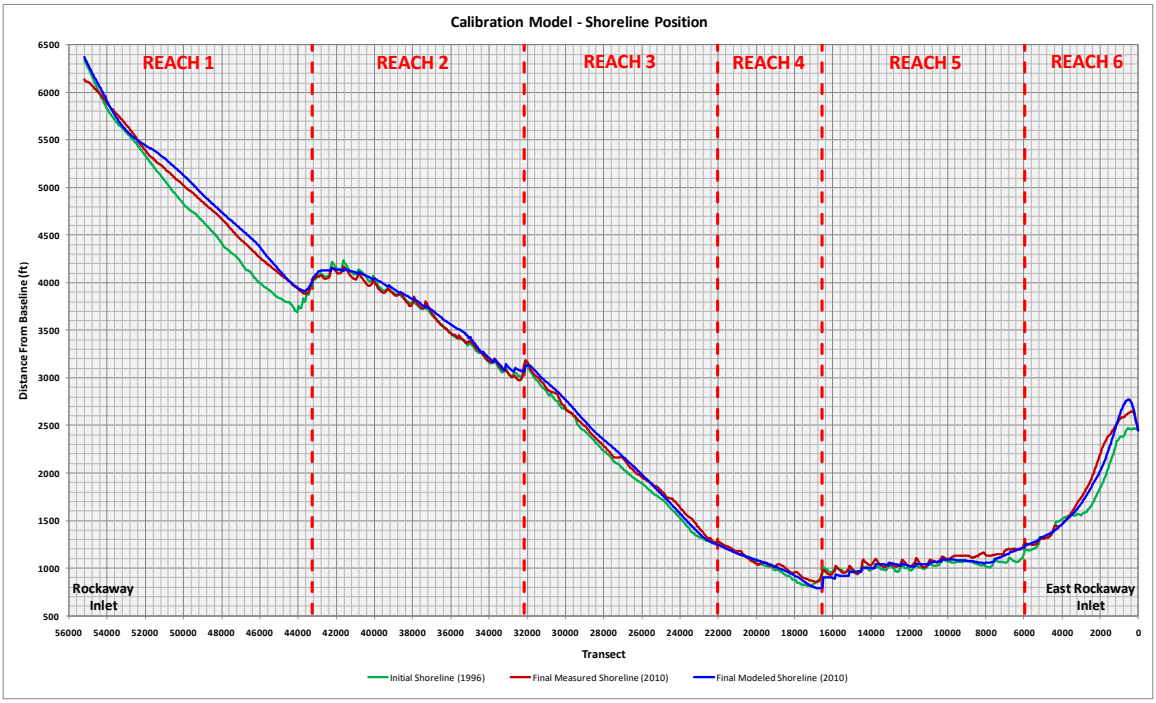


Figure 8-10: GENESIS-T Calibration Results – Shoreline Position Overview

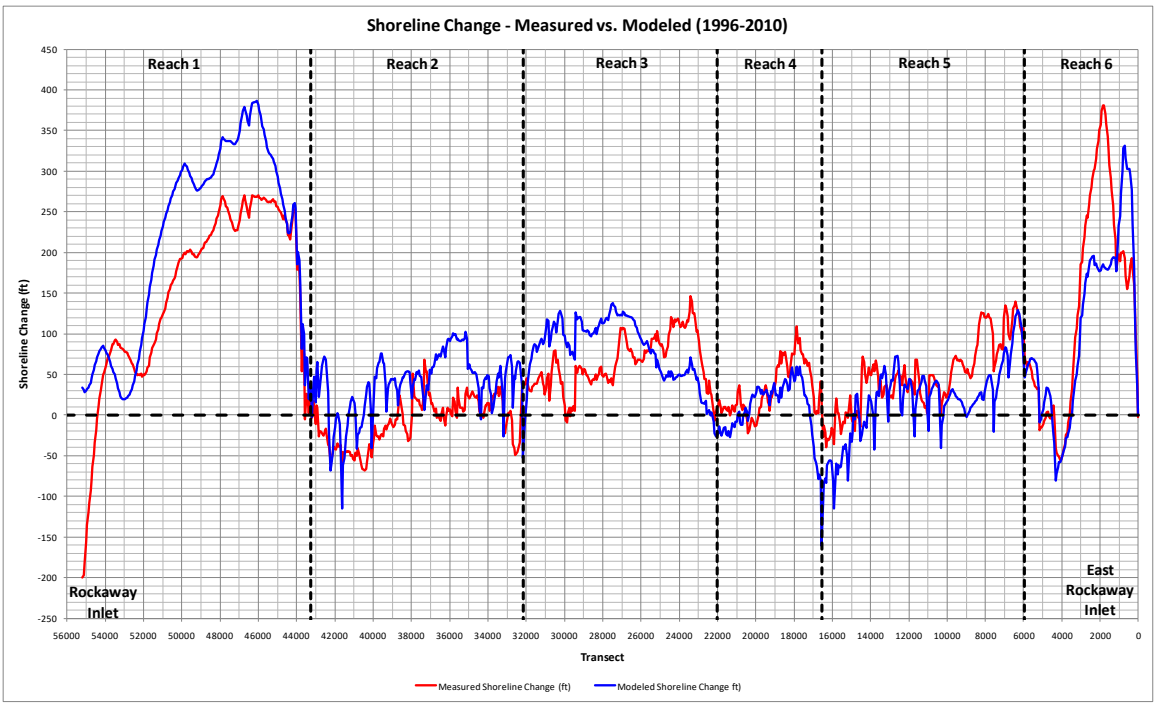


Figure 8-11: GENESIS-T Measured vs. Modeled Shoreline Change

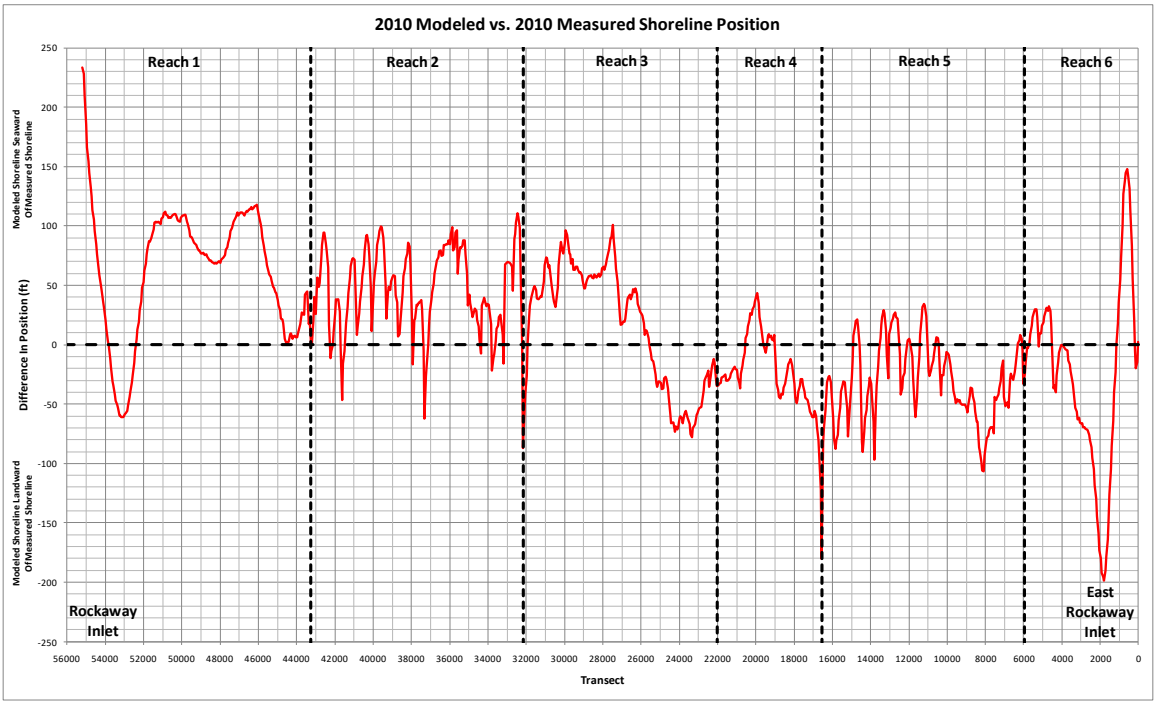


Figure 8-12: GENESIS-T Measured vs. Modeled Shoreline Position Difference

8.3 GENESIS Without Project Simulation

The Without Project Future Condition (WOPFC) is by definition the projection of the most-likely future conditions in the project area if no actions are taken as a result of this study. The WOPFC must be representative of what is most likely to occur in the absence of a federal project, and as such must be based upon historic practice and trends. The WOPFC serves as the base conditions for all the alternative analyses.

Identifying the WOPFC at Rockaway is particularly important and difficult because approximately 18,500,000 cubic yards of beach fill have been place along the shoreline since 1974 under the federal Beach Erosion Control Project. Therefore, historical data may not be used to describe the shoreline and beach conditions if no actions are taken in the project area. Instead, the calibrated GENESIS-T model is used to describe the shoreline changes that are likely to occur in the WOPFC.

In defining the WOPFC, the following assumptions are made to establish the framework of what is likely to occur.

- As defined by existing Federal/State navigation authorities, the existing inlets (Rockaway Inlet and East Rockaway Inlet) and their corresponding approach and back-bay navigation channels will be maintained near the present widths depths, and locations. Approximately 230,000 cubic yards of material will be dredged from East Rockaway Inlet every 2 years and placed in Reach 6.
- No other major beach fill projects will occur in the project area.

The 50-year study period for the WOPFC and alternative analyses is from June 2018 to June 2068. A 3.5 year GENESIS-T simulation was performed from January 2015 to June 2018 to create a realistic starting shoreline that would serve as consistent starting point for the WOPFC and alternative With Project Future Condition (WPFC) simulations. After developing the initial shoreline a 16-year simulation from June 2018 to June 2034 was performed in GENESIS-T to define the WOPFC. The same wave conditions from the calibration time frame (plus 1 extra year) were applied in the 16-year simulation.

The January 2015 shoreline was developed based on the latest available aerial imagery (Spring 2013) and accounts for the expected shoreline changes due to FCCA beach nourishment project currently under construction. Wave data for the 3.5 year GENESIS-T simulation was taken from years 2004-2007, which were considered to be a representative of the average wave climate. It was assumed that no inlet maintenance dredging would occur over the 3.5 year simulation since a substantial amount of material was removed for the FCCA beach nourishment project.

As stated above, the WOPFC does not include any beach fill except for inlet maintenance beach fill as part of the Federal Navigation Project (dredging) at East Rockaway Inlet. Based on the beach nourishment history compiled for the model calibration, it was determined that approximately 230,000 cubic yards of material is dredged from East Rockaway Inlet every 2 years



and placed in Reach 6 (Inlet Maintenance Nourishment). This equated to 66 ft of added berm width over 3,700 feet of shoreline.

Figure 8-13 show the results of the WOPFC simulations. It is important to acknowledge that the GENESIS-T simulation only covers 16 years of the 50-year study period. It is assumed that shoreline change trends would likely continue at similar rates for another 34 years. Starting in Reach 6 it is apparent the inlet maintenance beach fill is not enough to prevent continued shoreline erosion at the hot spot (near the eastern most groin). Although, the inlet maintenance beach fill does significantly reduce the erosion that would likely occur in its absence.

Moving west along the shoreline, significant erosion is expected to occur within the groin field in Reach 5. Erosion increases throughout the groin field and peaks at close to 200 feet in the western most compartment. The erosion in Reach 5 masks the effectiveness of the groin field. As the shoreline recedes, the effective length of the groins increases and the groin fields trapping efficiency also increases. The impact of the groin field is visible downdrift in Reach 4, where the most severe erosion is predicted to occur. Immediately downdrift of the groins shoreline changes on the order of 400 feet are predicted to occur. Severe erosion continuous throughout Reach 4 and extends into the beginning of Reach 3. The shoreline stabilizes in Reach 3 and begins to show accretion starting in Reach 2. Significant accretion is predicted in Reach 1 and Reach 2.

Based on the trends seen in the calibration results, some inferences can be made with respect to the without project conditions results. First, the model may slightly overestimate erosion in Reach 6 and Reach 4. Conversely, the accretion seen in Reaches 1 and 2 may also be overestimated. However, the errors in the model calibration were on the order of 3 feet/year, which is relatively small in comparison to some of the predicted shoreline change rates (-20 feet/year in Reach 4).

Even with these uncertainties, it is important to recognize the overall poor condition of the beach in the WOPFC. The WOPFC provides a baseline to compare the impact of each alternative on the future condition of the beach.

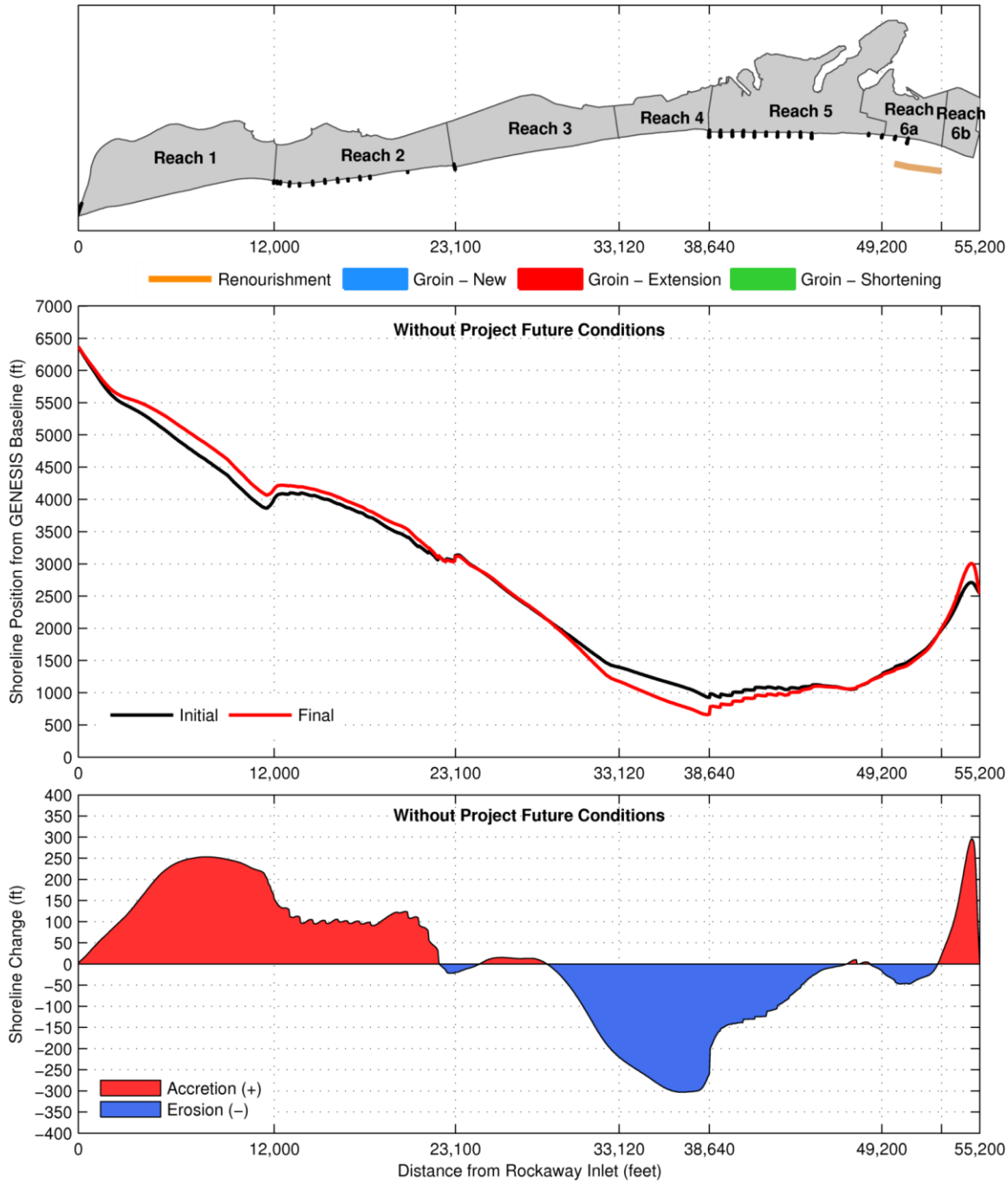


Figure 8-13: GENESIS-T WOPFC Shoreline Changes

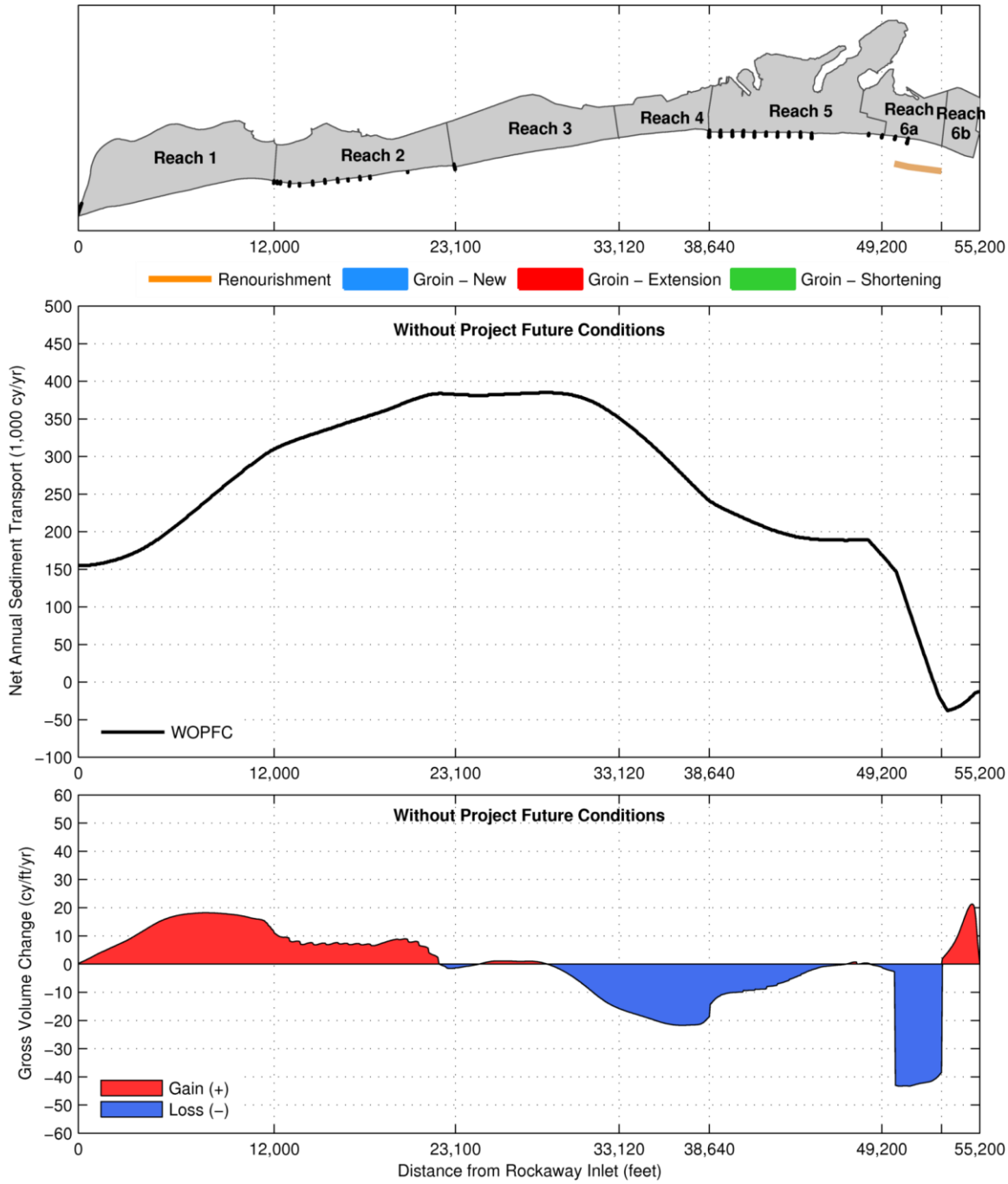


Figure 8-14: GENESIS-T WOPFC Sediment Transport & Gross Volume Changes

8.4 GENESIS Alternative Simulations

This section documents the evaluation of several beach restoration and erosion control alternatives for Rockaway Beach. The alternative evaluation focuses on ways to reduce future renourishment requirements and life-cycle costs. The alternative analysis does not consider storm damage reduction benefits; all of the alternatives provide roughly the same level of protection since the

alternatives have the same design profile (e.g. berm and dune dimensions). Different measures, such as new groins, shortening / lengthening of existing groins, and boardwalk relocation, are evaluated based on their ability to reduce future renourishment requirements and life-cycle costs. The three alternatives evaluated are (Figure 8-15):

- Alternative 1: Beach Restoration
- Alternative 2: Beach Restoration + Reduced Erosion Control
- Alternative 3: Beach Restoration + Increased Erosion Control

The objective of all three alternatives is the same: maintain the design beach profile over the life of the project at the lowest possible cost. The focus of Alternatives 2 and 3 are managing the two historical erosional hot spots in Reach 4 and 6 with either reduced erosion control (groin shortening and boardwalk relocation) or increased erosion control (new groins and extension of existing groins).

The calibrated GENESIS-T model was used to simulate With Project Future Conditions (WPFC) for each short-listed alternative. The same 16-year WOPFC time period (wave conditions from 1996 to 2012) is used to evaluate the alternatives. The GENESIS-T model simulates the impact of beach fill project and coastal structures on shoreline change and longshore sediment transport. The purpose of the GENESIS-T alternative simulations is to 1) confirm that each alternative is able to maintain the design shoreline between renourishment operations, 2) determine the location and quantity of beachfill needed to maintain the design shoreline.

The WPFC modeling was an iterative process requiring several simulations for each short-listed alternative. In particular, Alternatives 2 and 3 required numerous simulations to determine optimal groin configurations (e.g. length and number groins) and extent and density of renourishment fill needed. Each change in the groin configuration and renourishment quantities/extent affected sediment transport rates and shoreline changes. Therefore, it took several iterations to converge to an optimal solution.

In the beginning of the modeling study individual erosion control measures (e.g. new tapered groin field in Reaches 3 and 4) were evaluated separately. After investigating the merit of each individual measure, effective measures were combined to match the original short-listed alternatives. All the WPFC simulations include natural inlet bypassing and inlet maintenance dredging, both of which provide an influx of sediment to Reach 6a which helps reduce shoreline erosion.

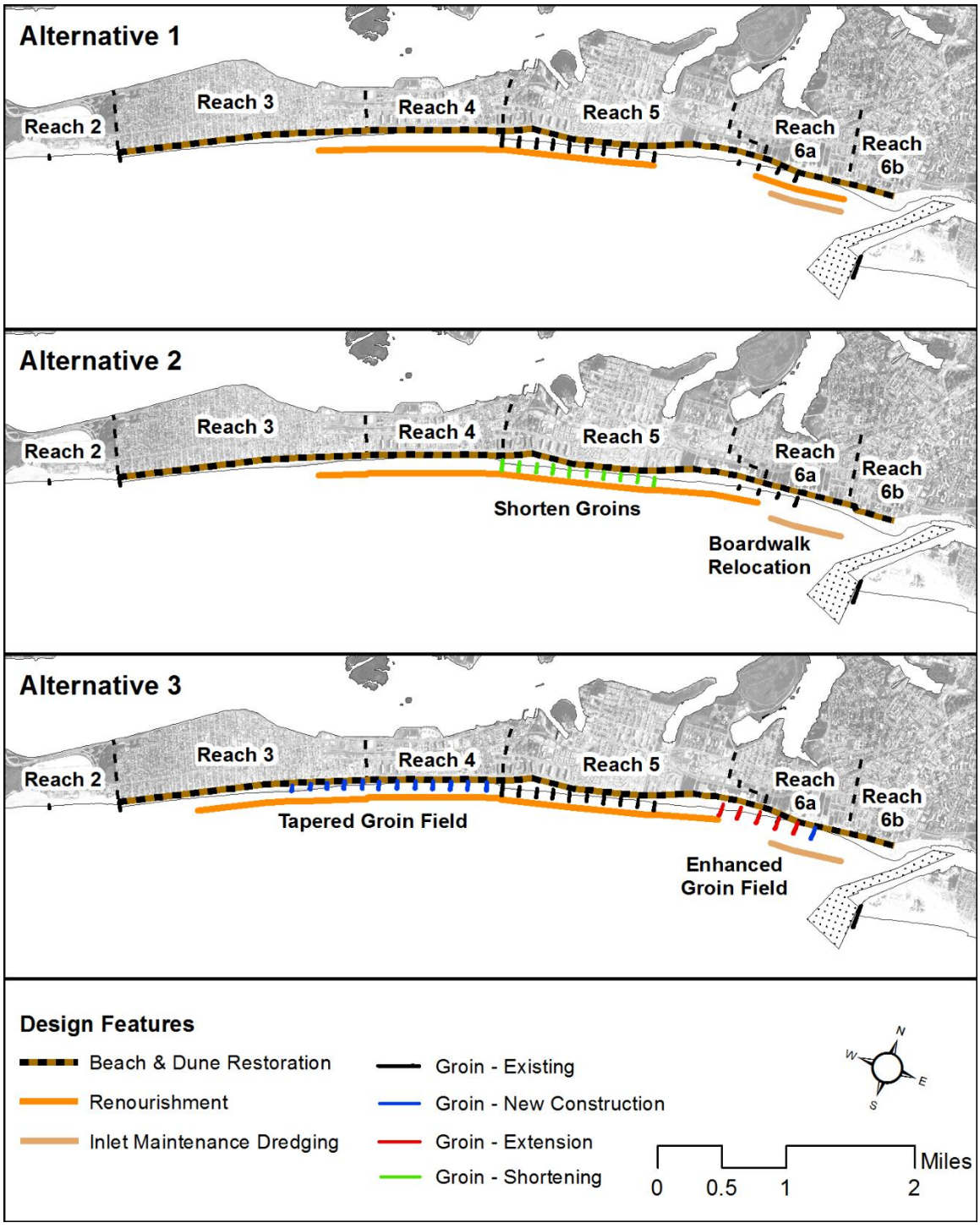


Figure 8-15: GENESIS-T Alternatives

8.4.1 Alternative 1

Alternative 1, Beach Restoration, is similar to the Section 934 Beach Erosion Control Project constructed and maintained from 1996-2004. Alternative 1 includes initial construction, advance

fill, and future renourishment operations. No modification of the existing groins is included in this alternative. Several iterations of Alternative 1 were simulated in GENESIS-T to determine the location and density of renourishment needed to maintain the design shoreline throughout the 16-year simulation. The final renourishment beach fill quantities are shown in Table 8-4.

Table 8-4: GENESIS-T Alternative 1 Renourishment Quantities

Reach	Fill Extent (ft)	Cycle (yr)	Renourishment (cy/yr)	Renourishment (cy per operation)	Fill Density (cy/ft/yr)
Reach 3	1,980	4	30,000	120,000	15.2
Reach 4	5,520	4	120,000	480,000	21.7
Reach 5	6,240	4	65,000	260,000	10.4
Reach 6a	3,660	4	60,000	240,000	16.4
Reach 6a	2,880	2	115,000[1]	230,000	39.9
Total			390,000		

[1]Inlet Maintenance Dredging of East Rockaway Inlet

Two types of figures are used in this document to present the GENESIS-T modeling results. The first type of figure, Figure 8-16, presents modeled net shoreline changes. Net shoreline changes show the difference between the initial shoreline position and final shoreline position. The final shoreline position is at the end of the last renourishment cycle and captures the loss of advance fill. The shoreline change figures illustrate the ability of each alternative to stabilize the shoreline and highlights the areas of the shoreline that require advance fill and periodic renourishment operations. Advance fill is a sacrificial quantity of sand which acts as an erosional buffer against long-term and storm-induced erosion with the goal of preventing erosion of the design profile between renourishment cycles. In general the design shoreline is maintained within the Project Area over the 16-year GENESIS-T simulations. Therefore, shoreline erosion within the Project Area in Figure 8-16 captures the loss of the advance fill volume over the last renourishment cycle. The x-axis, distance from Rockaway Inlet, is the same for all three panels. It is important to note that the shoreline changes are compared to the initial shoreline (i.e. after initial construction of the project with advance fill).

The second type of figure, Figure 8-17, presents modeled net annual longshore sediment transport rates in 1,000 cy/yr (middle panel) and gross sediment volume changes in cy/ft/yr (bottom panel). The bottom panel (gross volume changes) illustrates the total volume of sediment lost or gained along the shoreline. The gross volume changes combines shoreline changes and placed beachfill quantities to capture the true erosive nature of certain areas, highlighting the difficulty in maintaining the design shoreline in these areas.

A third type of figure, which is not included in this appendix, but instead as an enclosure, shows the initial and final shoreline positions from the GENESES-T modeling results in plan-view with



an aerial image of the study in the background. These figures are included in Enclosure B - "GENESIS-T Model Results".

The middle panel of Figure 8-17 shows the net annual longshore sediment transport rates under WOPFC (black) and Alternative 1 (red). The only difference between these two conditions is the initial shoreline positions and placement of beachfill during renourishment operations. It is clear that these two changes result in greater sediment transport rates throughout the project area. The increase in sediment transport rates is attributed to end losses caused by beachfill diffusion and reduced effectiveness of groins (e.g. Reach 5) caused by an increase in the beach width / reduction in effective groin length.

A summary of the performance of Alternative 1 is provided below:

- End losses and background erosion rates make it difficult and costly to maintain the design shoreline in Reach 6a, requiring approximately 175,000 cy/yr of fill.
- A large portion of the beachfill placed in Reach 6a is transported to adjacent shorelines where sediment is accumulated resulting in excess beach widths.
- Periodic renourishment is required in the groin field in Reach 5 and downdrift of the groin field in Reaches 3 and 4 to offset background shoreline erosion.
- A large portion of the beachfill placed in Reach 4 is transported to adjacent shoreline in Reach 3 where sediment is accumulated resulting in excess beach widths.
- The design shoreline in the hot spot in Reach 4 is be maintained by 1) keeping the groin compartments in Reach relatively filled which allows more sediment transported downdrift, 2) providing sufficient advance fill to offset background shoreline erosion.
- The quantity of beachfill required over the life of the project, 390,000 cy/year is consistent with the amount of beachfill placed in the Historical Sediment Budget (398,000 cy/yr).

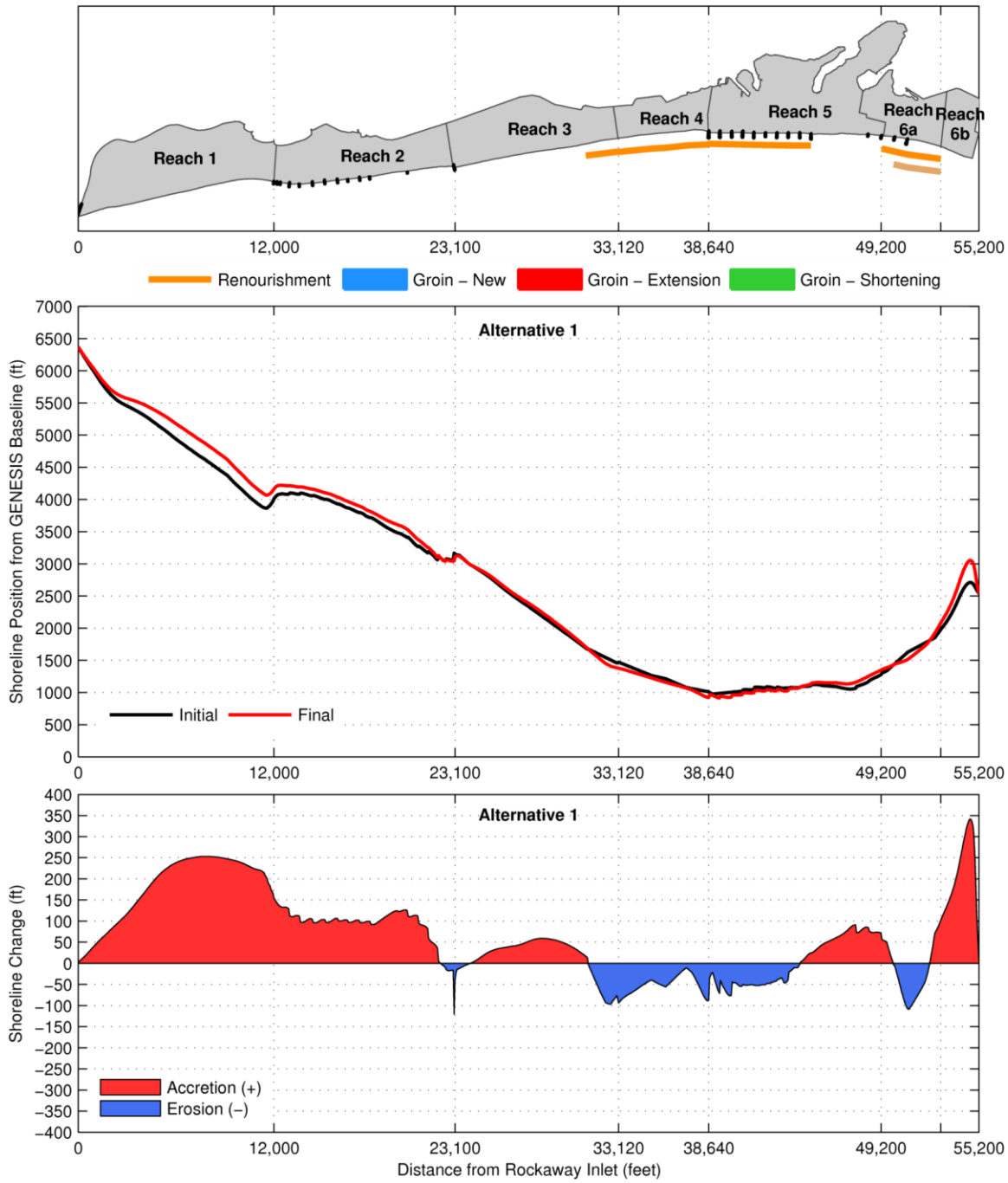


Figure 8-16: GENESIS-T Alternative 1 Shoreline Changes

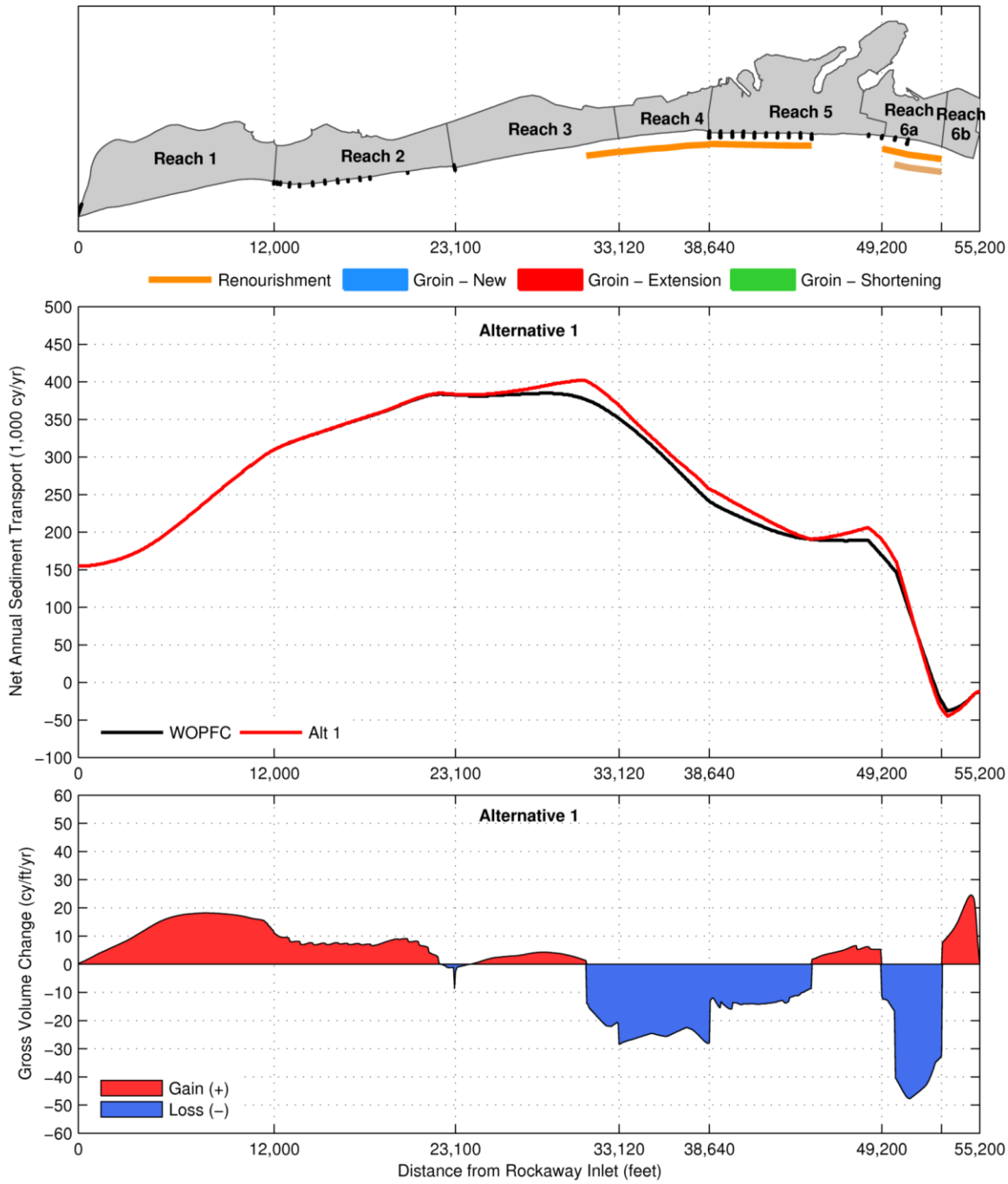


Figure 8-17: GENESIS-T Alternative 1 Sediment Transport & Gross Volume Changes

8.4.2 Alternative 2

Alternative 2, Beach Restoration and Reduced Erosion Control, includes two measures in addition to Alternative 1: 1) boardwalk relocation in Reach 6a with landward relocation of design shoreline,

and 2) groin shortening in Reach 5 to address historic erosional hot spots in Reach 6a and Reach 4. Several iterations of Alternative 2 were simulated in GENESIS-T to determine the location and density of renourishment needed to maintain the design shoreline throughout the 16-year simulation. The final renourishment beach fill quantities are shown in Table 8-5.

Table 8-5: GENESIS-T Alternative 2 Renourishment Quantities

Reach	Fill Extent (ft)	Cycle (yr)	Renourishment (cy/yr)	Renourishment (cy per operation)	Fill Density (cy/ft/yr)
Reach 3	2,000	4	25,000	100,000	12.5
Reach 4	5,520	4	70,000	280,000	12.7
Reach 5	6,240	4	90,000	360,000	14.4
Reach 5	4,320	4	35,000	140,000	8.1
Reach 6a	2,880	2	115,000[1]	230,000	39.9
Total			335,000		

[1]Inlet Maintenance Dredging of East Rockaway Inlet

The modeling results are presented in Figure 8-18 and Figure 8-19. Net shoreline changes are presented in Figure 8-18. Net annual longshore sediment transport rates and gross volume changes are presented in Figure 8-19.

The middle panel of Figure 8-19 shows the net annual longshore sediment transport rates under Alternative 1 (red) and Alternative 2 (blue). The impact of boardwalk relocation is captured by the reduction in sediment transport rates in Reach 6a and the eastern end of Reach 5. Likewise, the effect of shortening the groins in Reach 5 is captured by an increase in the sediment transport rates in this area.

A summary of the performance of Alternative 2 is provided below:

- End losses and background erosion rates in Reach 6a are reduced by relocating the boardwalk in Reach 6a. Inlet maintenance dredging (115,000 cy/yr) is enough to maintain the design shoreline, no additional renourishment is required.
- Due to the boardwalk relocation and associated reduced renourishment quantities there is no longer an excess of sediment accumulating in the adjacent areas of Reach 5.
- Groin shortening in Reach 5 increases sediment transport within the groin field allowing more sediment to be transported downdrift to Reach 4. As a result, renourishment quantities in Reach 4 are reduced. However, this reduction comes at the expense of Reach 5 which now requires higher renourishment volumes.
- Groin shortening evenly redistributes the background shoreline erosion rates between Reach 4 and 5. Groin shortening does not affect the overall renourishment quantities.

- Groin shortening has a negligible effect on sediment transport, shoreline changes, and renourishment quantities in Reach 3 in comparison with Alternative 1.
- 335,000 cy/year of beachfill are required over the life of the project, a reduction of 55,000 cy/yr compared to Alternative 1.The reduction of renourishment quantities is associated with the boardwalk relocation feature.

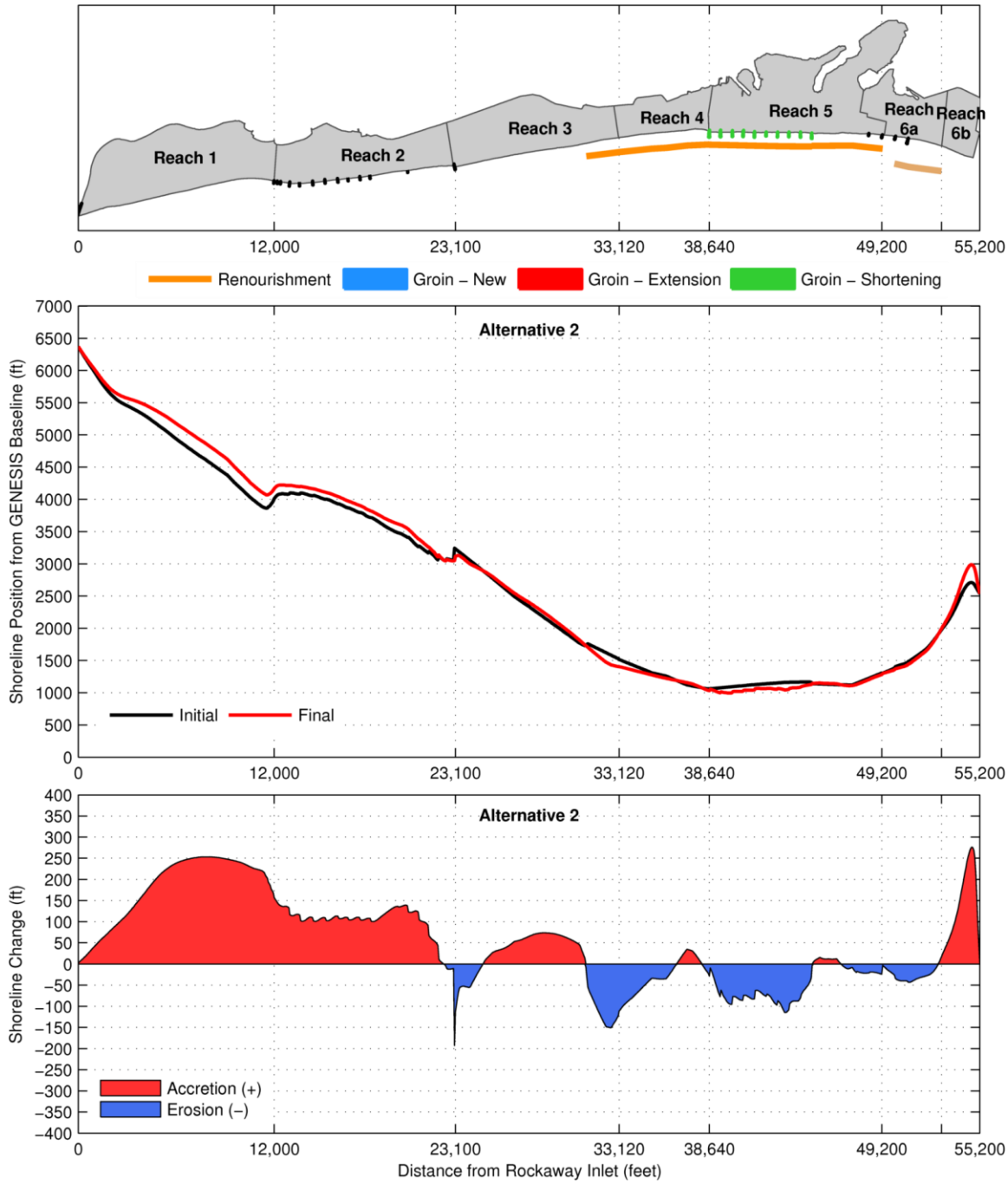


Figure 8-18: GENESIS-T Alternative 2 Shoreline Changes

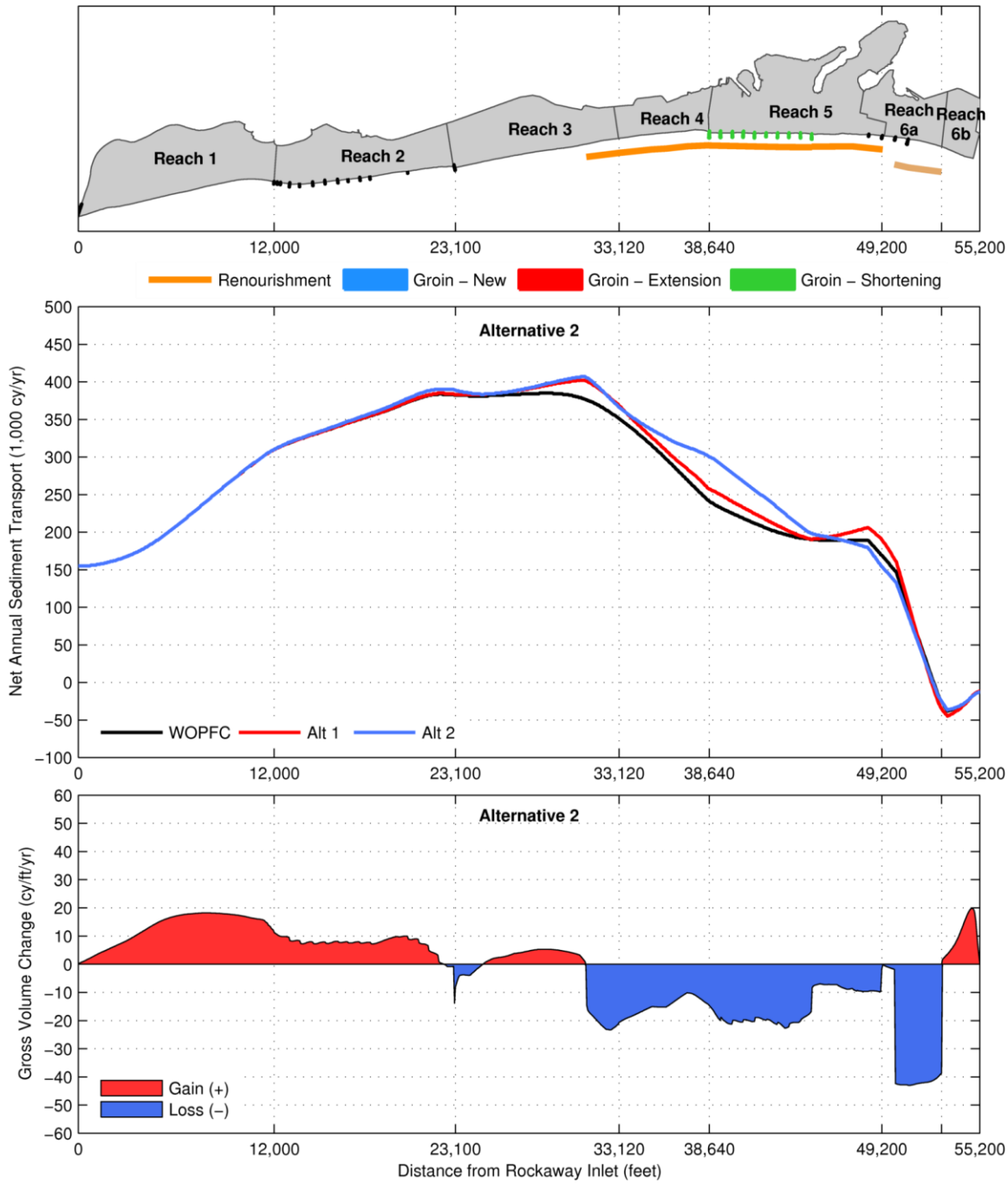


Figure 8-19: GENESIS-T Alternative 2 Sediment Transport & Gross Volume Changes

8.4.3 Alternative 3

Alternative 3, Beach Restoration and Increased Erosion Control, includes two measures in addition to Alternative 1: 1) new groin construction and groin extensions in Reach 6a, and 2) new groin

construction in Reaches 3 and 4 to address historic erosional hot spots in Reach 6a and Reach 4. Several iterations of Alternative 3 were simulated in GENESIS-T to determine the optimal groin configurations as well as the location and density of renourishment needed to maintain the design shoreline throughout the 16-year simulation. The final renourishment beach fill quantities are shown in Table 8-6.

Table 8-6: GENESIS-T Alternative 3 Renourishment Quantities

Reach	Fill Extent (ft)	Cycle (yr)	Renourishment (cy/yr)	Renourishment (cy per operation)	Fill Density (cy/ft/yr)
Reach 3	3,960	4	40,000	160,000	10.1
Reach 3	3,000	4	50,000	200,000	16.7
Reach 4	5,520	4	25,000	100,000	4.5
Reach 5	6,240	4	80,000	320,000	12.8
Reach 5	2,700	4	10,000	40,000	3.7
Reach 6a	2,880	2	115,000[1]	230,000	39.9
Total			335,000		

[1]Inlet Maintenance Dredging of East Rockaway Inlet

The modeling results are presented in Figure 8-20 and Figure 8-21. Net shoreline changes are presented in Figure 8-20. Net annual longshore sediment transport rates and gross volume changes are presented in Figure 8-21.

The middle panel of Figure 8-21 shows the net annual longshore sediment transport rates under Alternative 1 (red) and Alternative 3 (green). The impact of new groin and groin extensions in Reach 6a is captured by the reduction in sediment transport rates in this area. Likewise, the effect of constructing new groins in Reaches 3 and 4 results in a reduction in sediment transport within the groin field.

A summary of the performance of Alternative 3 is provided below:

- End losses and background erosion rates in Reach 6a are reduced by extending the existing groins and adding one new groin. Inlet maintenance dredging (115,000 cy/yr) is enough to maintain the design shoreline, no additional renourishment is required.
- The groin extensions in Reach 6a combined with the decrease in renourishment fill placed, decreases the sediment flux into Reach 5. As a result renourishment quantities in Reach 5 increase.
- Sediment transport rates at the western end of Reach 5 have increased in response to the increase in beachfill (i.e. beachfill diffusion). As a result, the sediment flux into Reach 4 increased.

- The groin field in Reach 4 needed to be extended into Reach 3 to prevent creating a new hot spot.
- The sediment transport rate in within the new groin field in Reaches 3 and 4 is suppressed by the groins.
- The tapered groins lead a smooth transition between the groin field and downdrift shoreline.
- The quantity of beachfill required over the life of the project is 320,000 cy/year, a total reduction of 70,000 cy/yr from Alternative 1 (390,000 cy/yr).
- 35,000 cy/yr of the reduction in renourishment quantities is attributed to the groin features in Reach 6a.
- 35,000 cy/yr of the reduction in renourishment quantities is attributed to the groin features in Reaches 3 and 4.

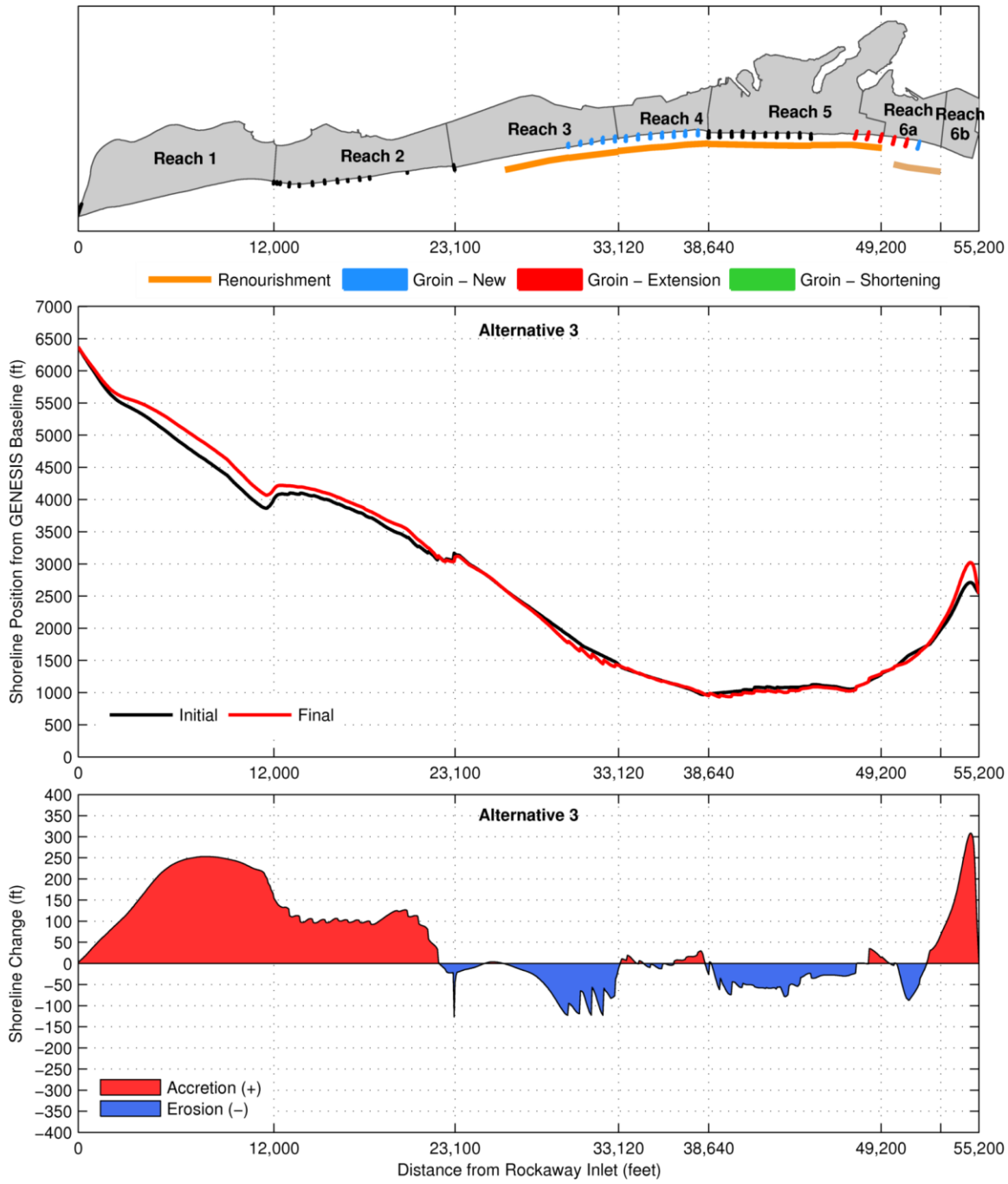


Figure 8-20: GENESIS-T Alternative 3 Shoreline Changes

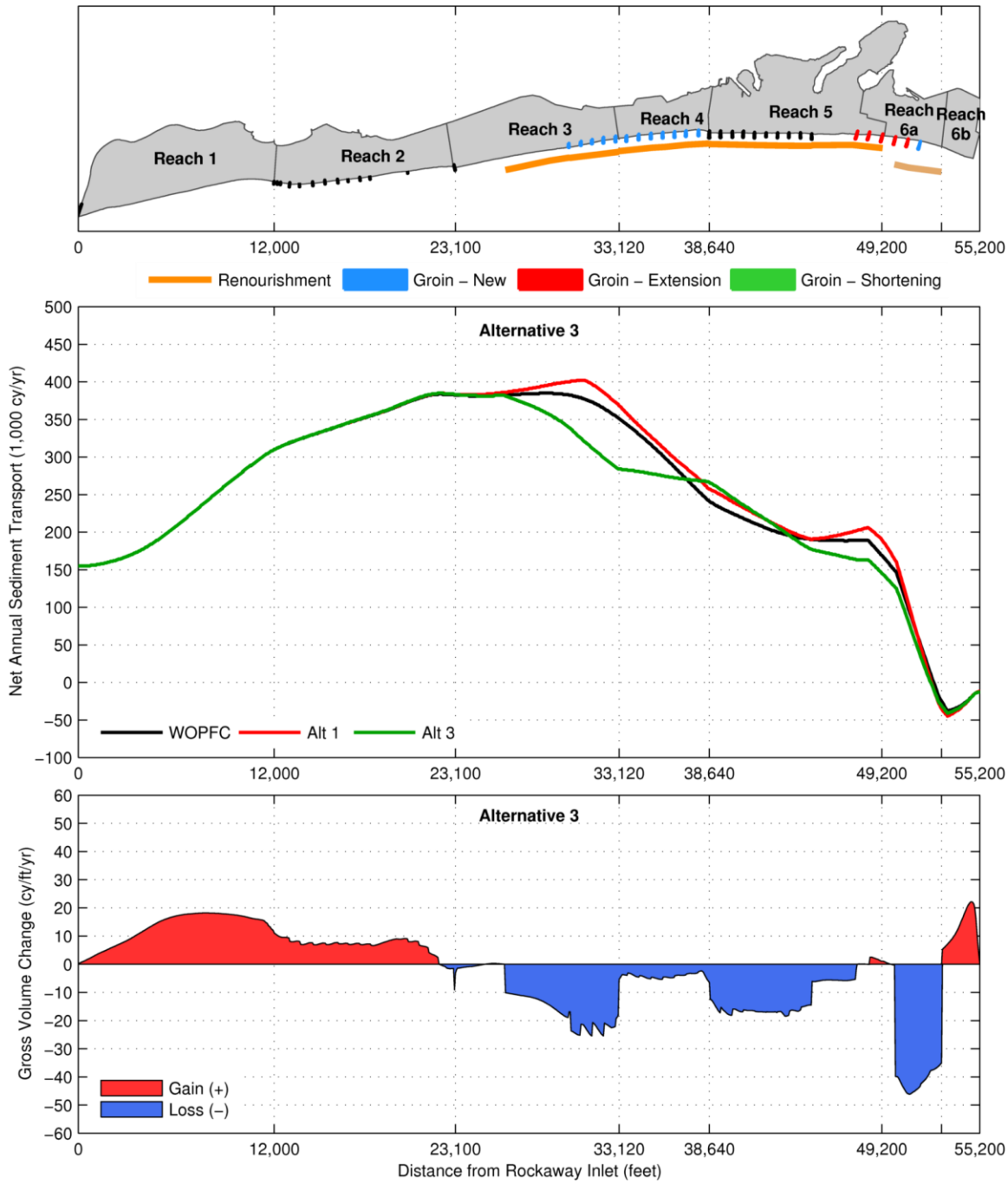


Figure 8-21: GENESIS-T Alternative 3 Sediment Transport & Gross Volume Changes

8.4.4 Other Alternatives

Several other alternatives were evaluated in GENESIS-T before converging to the three alternatives described above. In this case it is just as informative review the GENESIS-T results

for a few of the alternatives that were not carried forward. GENESIS-T results for three additional alternatives are presented here, all of which are variations of Alternative 3:

- Alt A - Alt 3 without any groins in Reach 3 and longer groins in Reach 4;
- Alt B - Alt 3 without any groins in Reaches 3 or 4, but tapering of the last two groins in Reach 5;
- Alt C - Alt 3 with additional groins extending all the way through Reach 2.

The sediment transport results for these 3 additional alternatives are shown in Figure 8-22. The purpose of showing Alt A is to reinforce the point that adding new groins has the potential to just push the "problem" downdrift. A snapshot of the shoreline positions at the end of the 16-year GENESIS simulation for Alternative A is shown in Figure 8-23, which shows that Alternative A has just moved the EHS to Reach 3. Alternative 3 included 5 tapered groins in Reach 3 to avoid creating an EHS equal or worse in this case to the historical EHS in Reach 4.

Alternative B, tapering of the last two groins in Reach 5, does increase the flux of sediment into Reach 4. However, the shoreline in Reach 5 experiences additional erosion (Figure 8-24). This measure could be combined with Alternative 3 with the additional erosion in Reach 5 managed by placing more beachfill in this area. However, the same objective may be achieved by decreasing the effective length of these groins by placing additional sand in the last two groin cell compartments

Alternative C, extending groins from Reach 5 through Reach 2, is included to show that the sediment transport rate could be suppressed over the entire project area if enough groins are constructed. Downdrift impacts in Alternative C would be minimal since Reach 1 is an area that is accreting. Alternative C would reduce renourishment quantities by 50,000 to 100,000 cy per year. However, the cost of building groins along the entire shoreline would likely outweigh the cost savings.

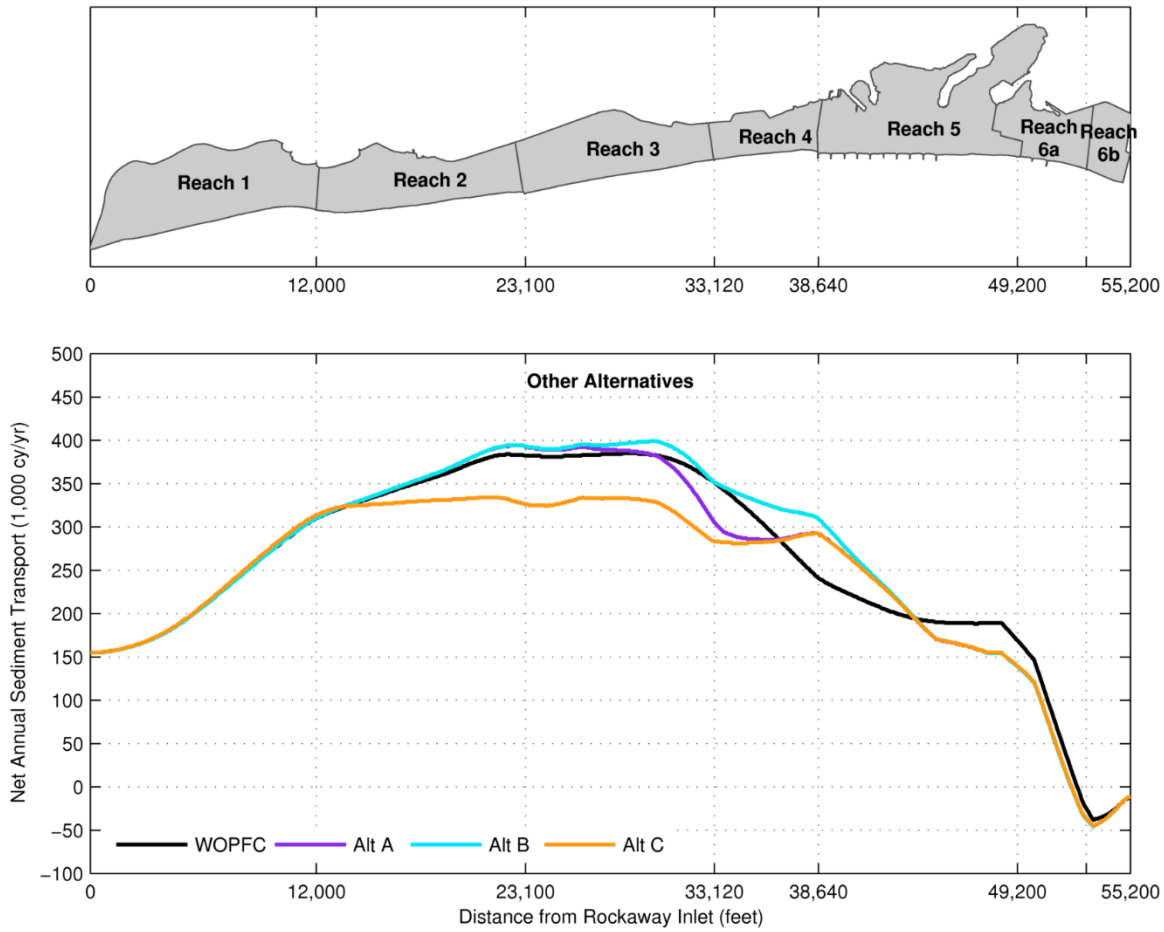


Figure 8-22: GENESIS-T Sediment Transport for Other Alternatives

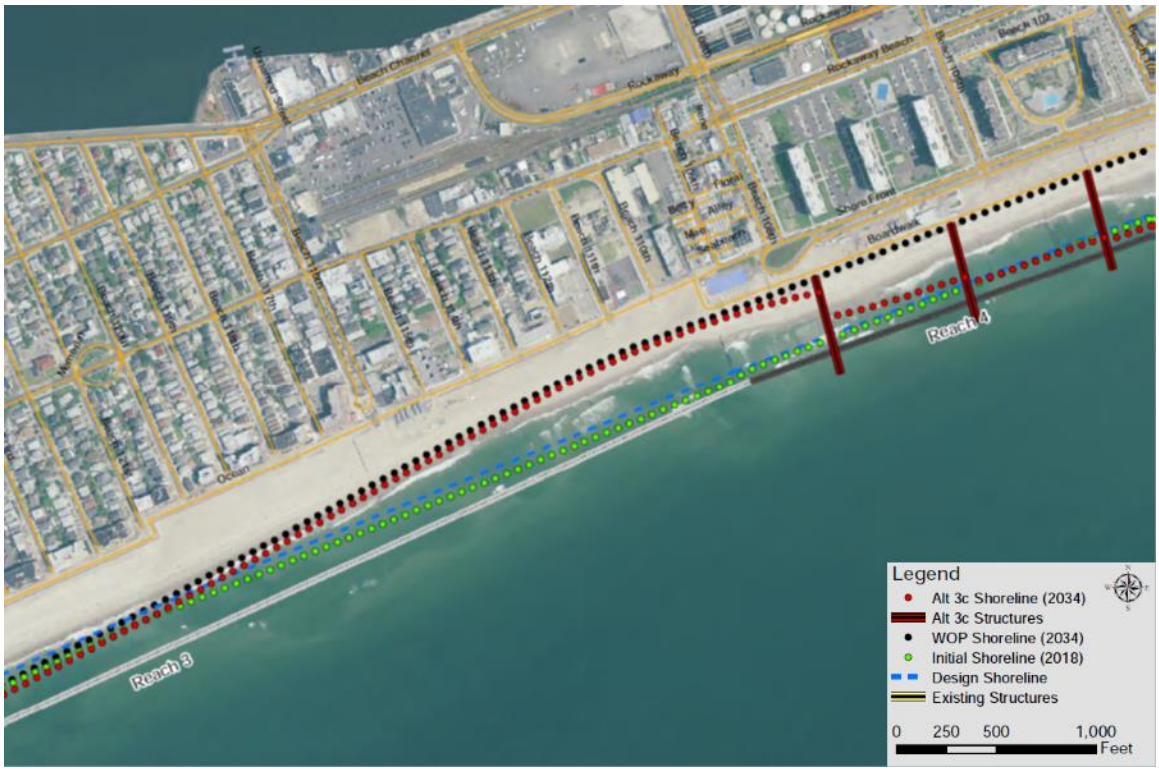


Figure 8-23: GENESIS-T Downdrift Erosion in Alternative A (red dots)



Figure 8-24: GENESIS-T Impact of Tapered Groins in Alternative B (orange dots)

8.4.5 Summary of GENESIS-T Results

GENESIS-T modeling was conducted for the three short-listed alternatives. Several GENESIS-T simulations were performed for each alternative prior to converging to a combination of renourishment fill and erosion control measures that maintained the design shoreline between renourishment operations. A summary of the renourishment quantities is provided in Table 8-7. It is noted that these renourishment quantities do not account for sea level rise or overfill requirements. These factors will be incorporated in the life-cycle cost estimates.

Alternative 1, Beach Restoration, requires 390,000 cy/yr of renourishment to maintain the design shoreline. As a point of comparison, 398,000 cy/yr of beachfill was placed along Rockaway from 1975-2010, and 354,000 cy/yr of beachfill was required during the Section 934 Beach Erosion Control Project.

Alternative 2, Reduced Erosion Control, requires 335,000 cy/yr of renourishment to maintain the design shoreline. A reduction in renourishment quantities of 55,000 cy/yr, all of which is attributed to the boardwalk relocation measure. Groin shortening in Reach 5 does not reduce the overall renourishment quantities. However, groin shortening does address the erosional hot spot in Reach 4 by increasing the flux of sediment into Reach 4, creating a more uniform shoreline erosion rate.

Alternative 3, Increased Erosion Control, requires 320,000 cy/yr of renourishment to maintain the design shoreline. A reduction in renourishment quantities of 35,000 cy/yr is attributed to the new groin and groin extensions in Reach 6a. A reduction of 35,000 cy/yr is attributed to the new tapered groin field in Reaches 3 and 4. The modeling results indicated that the new tapered groin field in Reach 4 needed to be extended into Reach 3 in order to avoid creating a new erosional hot spot downdrift.

Table 8-7: GENESIS-T Summary of Renourishment Quantities

Reach	Renourishment (cy/yr)[1]		
	Alternative 1	Alternative 2	Alternative 3
Reach 3	30,000	25,000	90,000
Reach 4	120,000	70,000	25,000
Reach 5	65,000	125,000	90,000
Reach 6a	175,000[2]	115,000[2]	115,000[2]
Total	390,000	335,000	320,000

Notes [1]Renourishment quantities do not include SLR or overfill

[2]Includes Inlet Maintenance Dredging (115,000 cy/yr)

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ENCLOSURE A

SBEACH MODEL RESULTS

SUB APPENDIX A

SBEACH MODEL RESULTS

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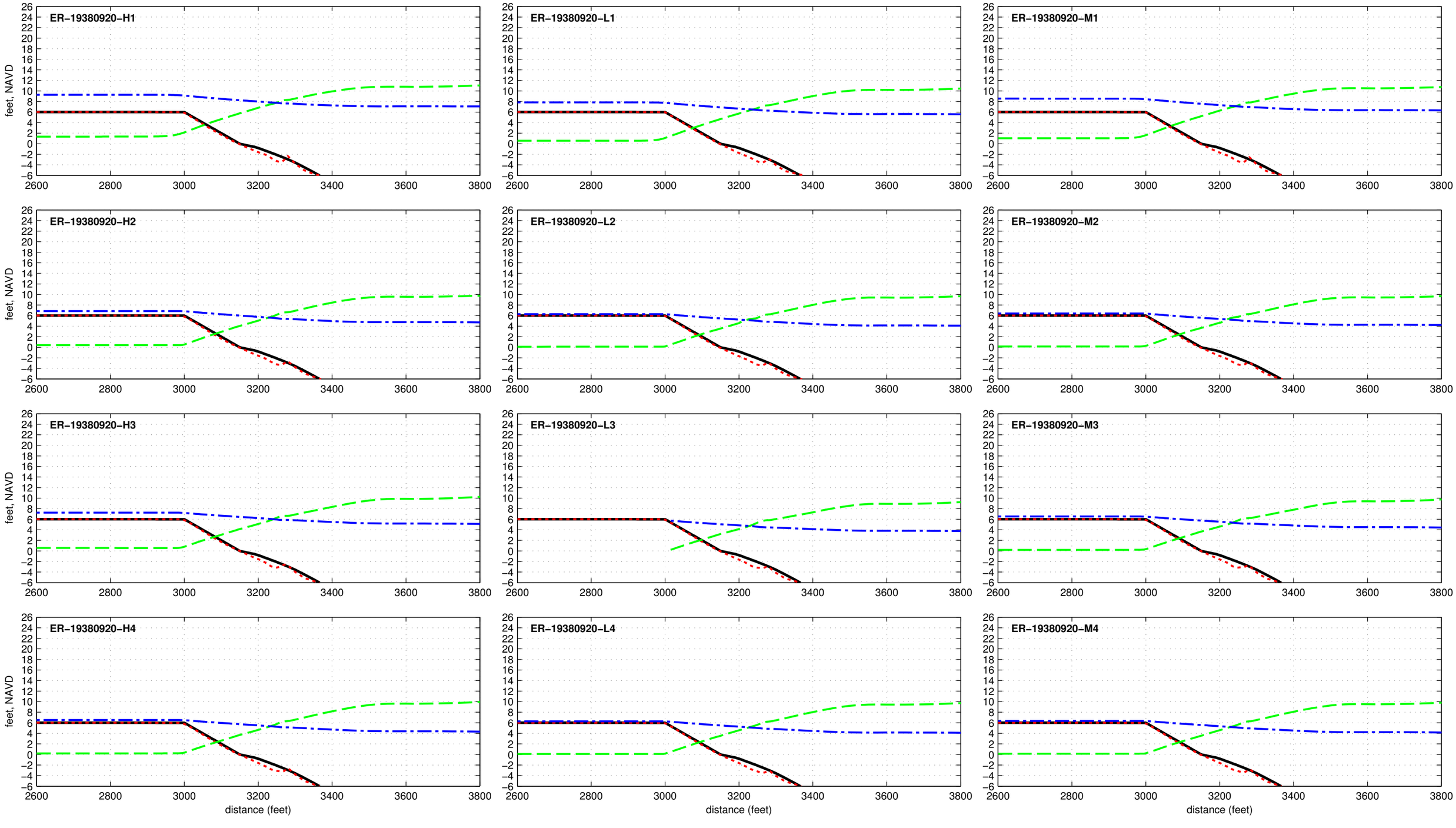

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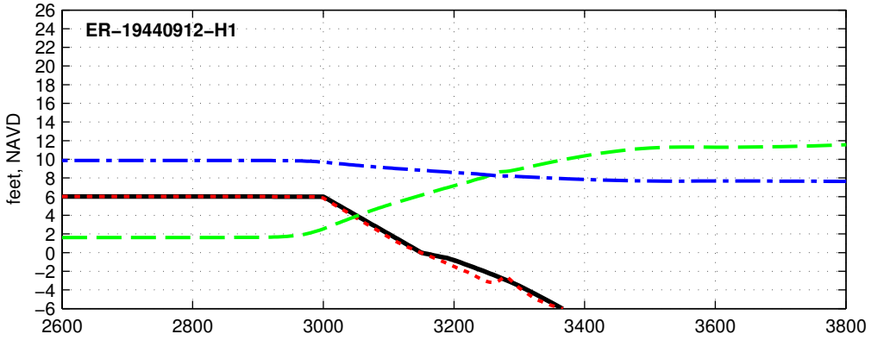
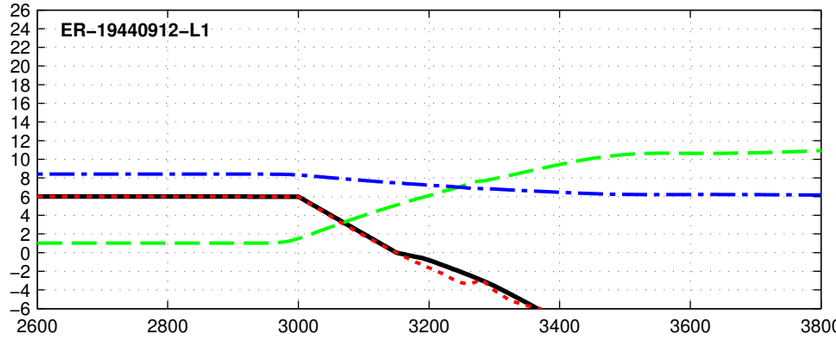
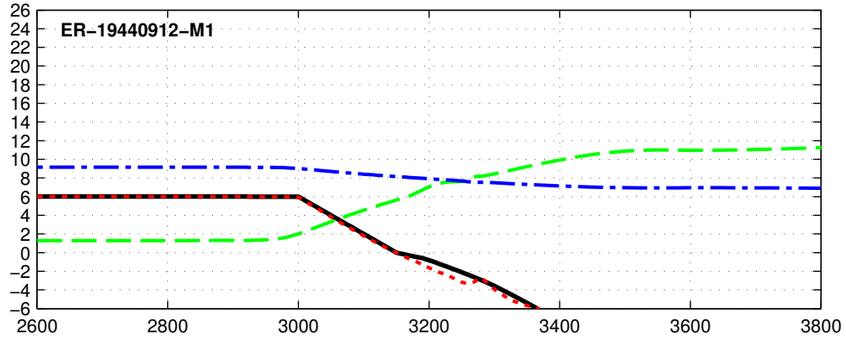
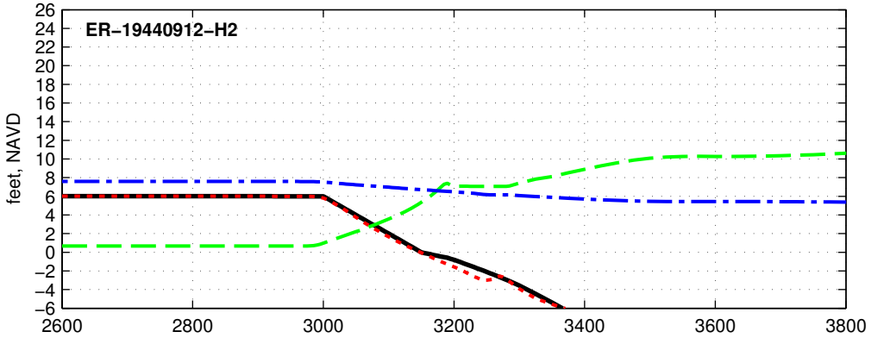
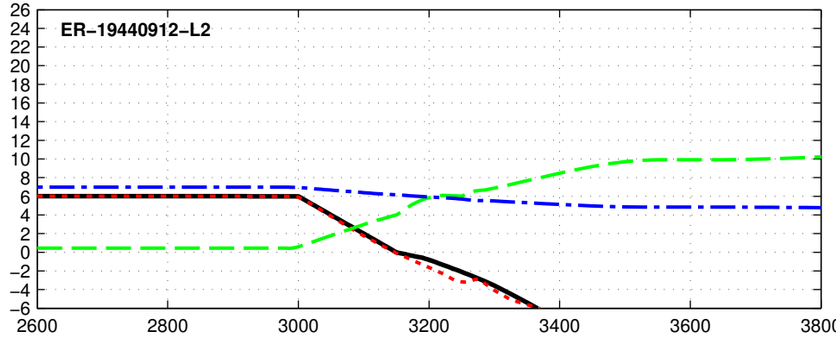
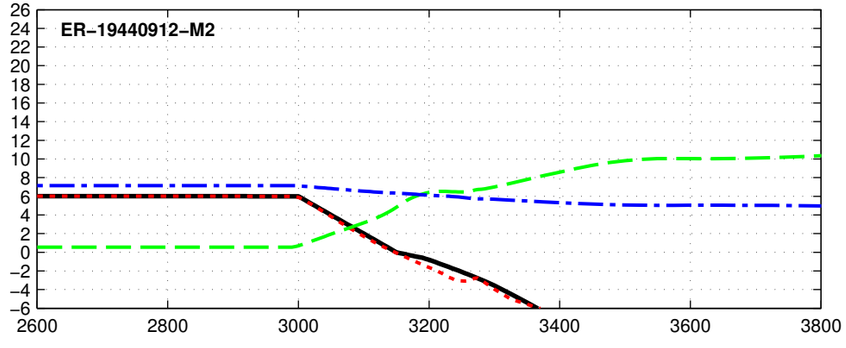


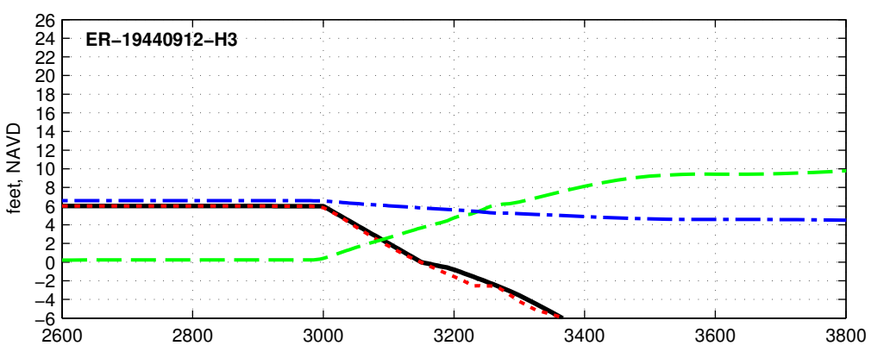


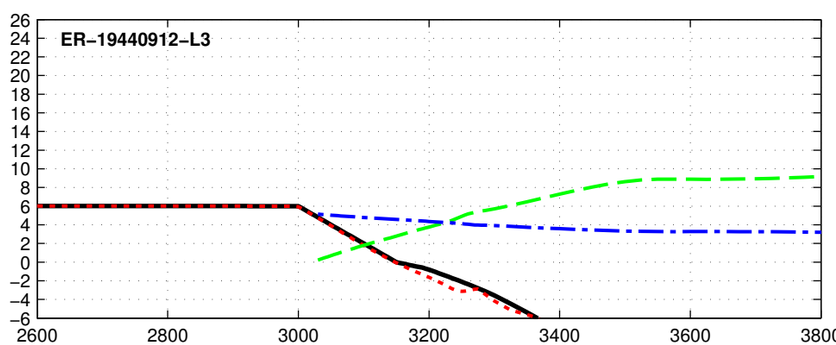


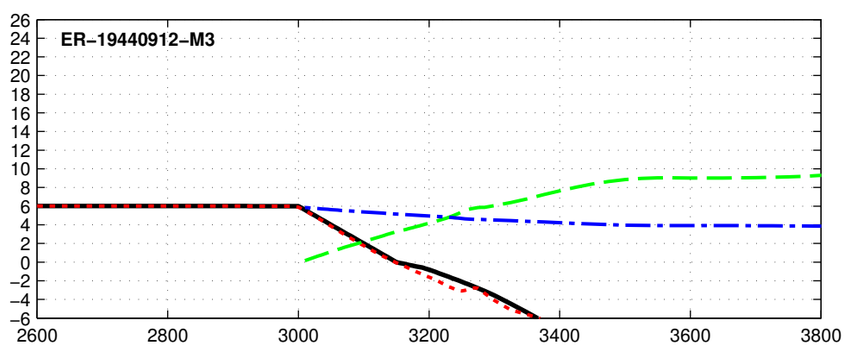


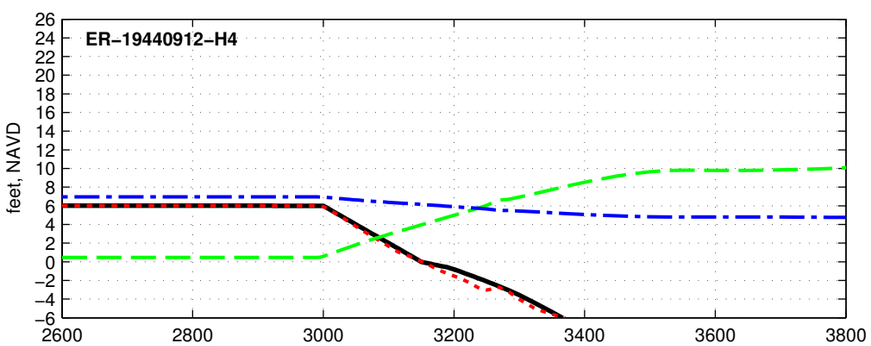


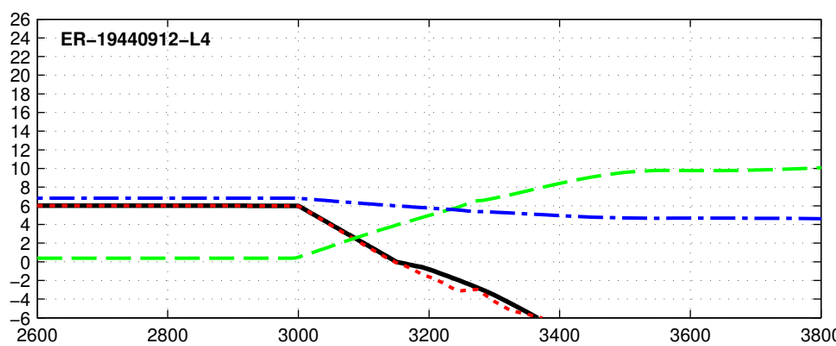


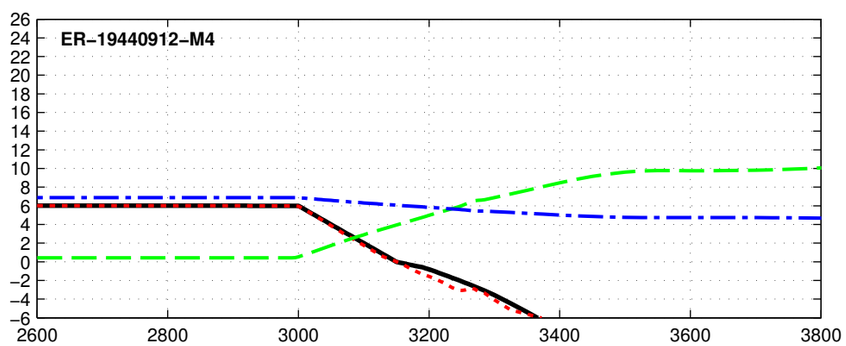






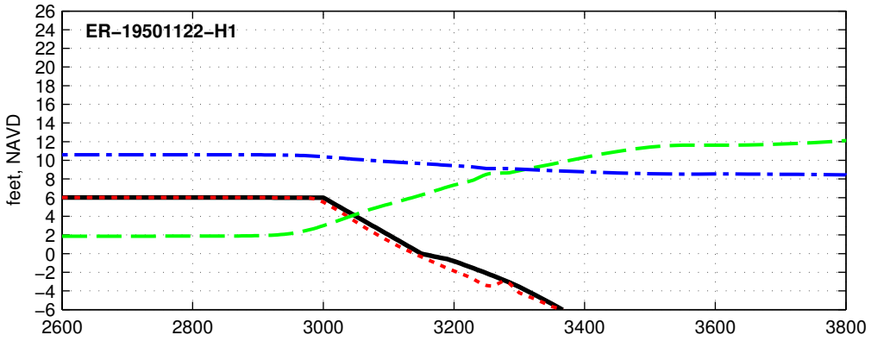


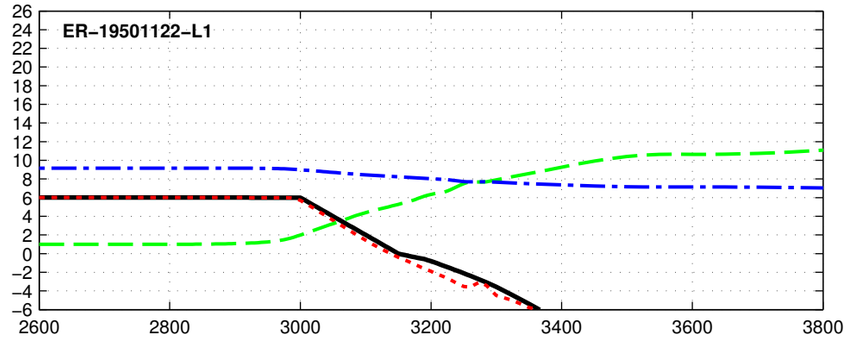


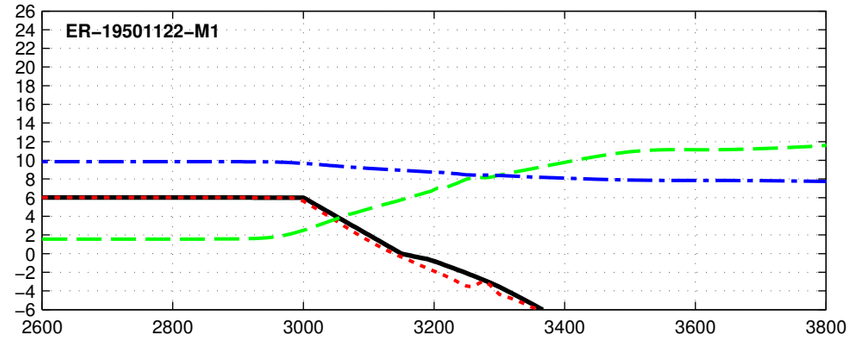


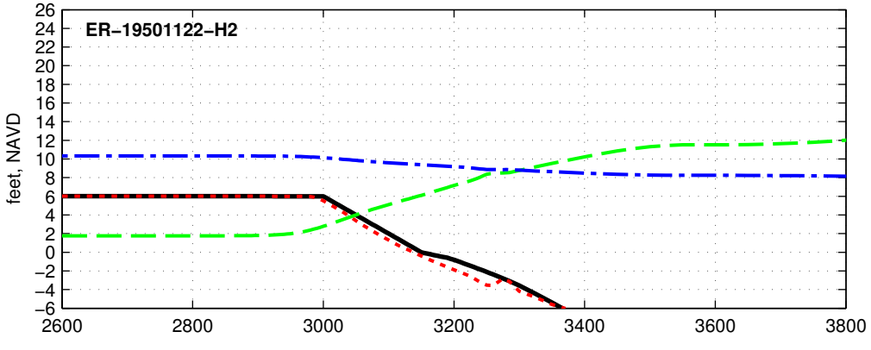
Initial Final Max Hs Max Water Level

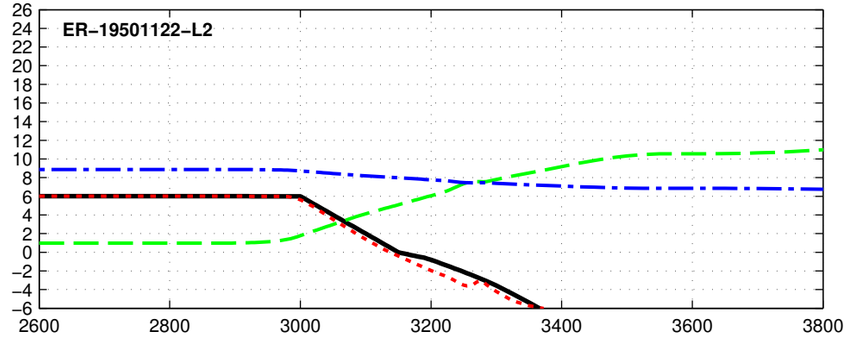
Dune R1T1-6.0-000-000 - ER-19501122

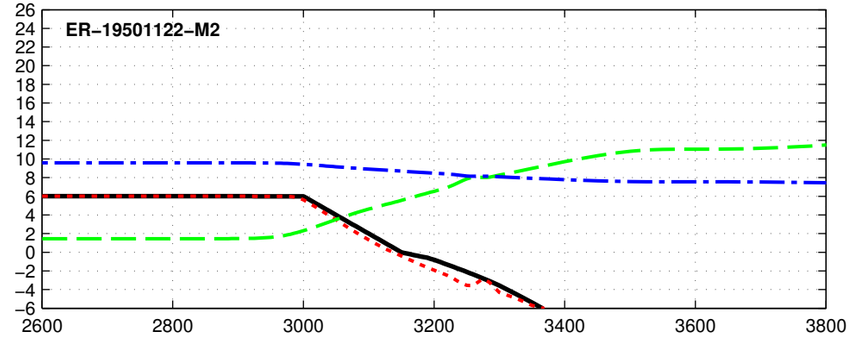


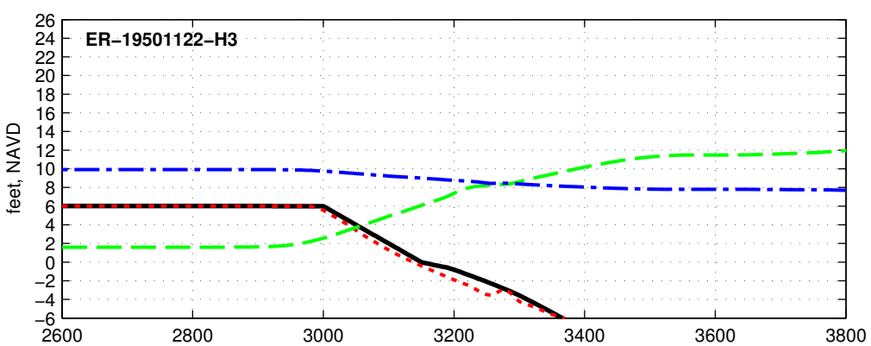


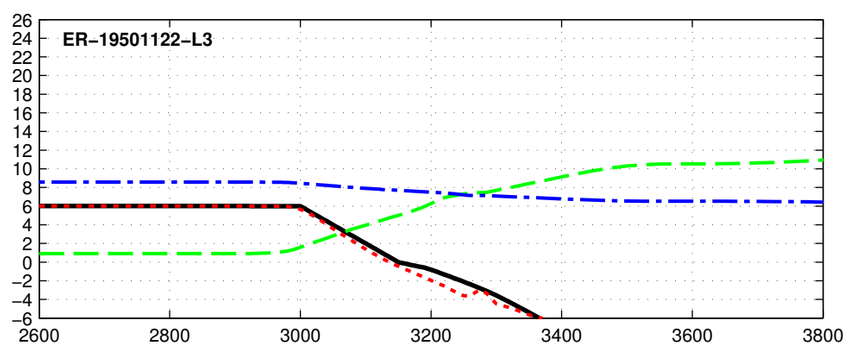


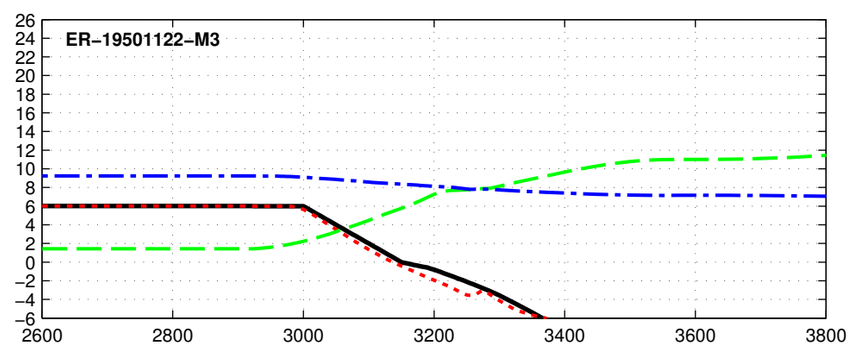


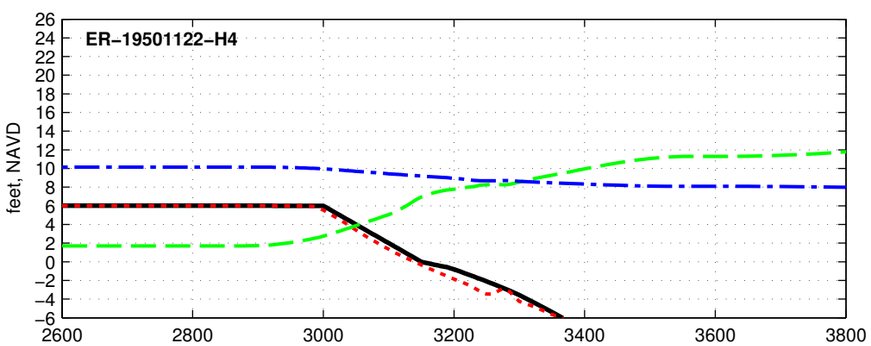


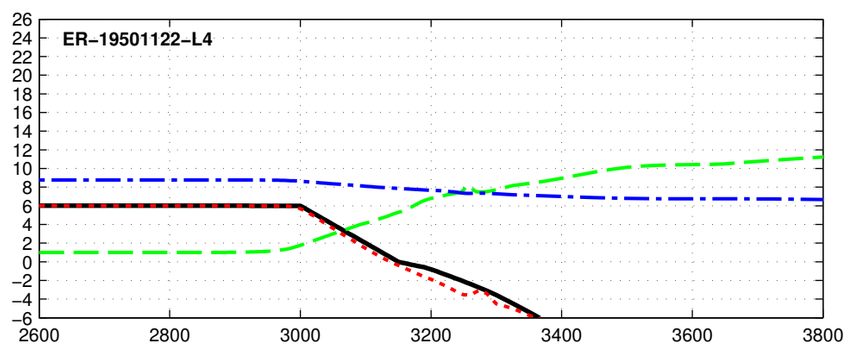


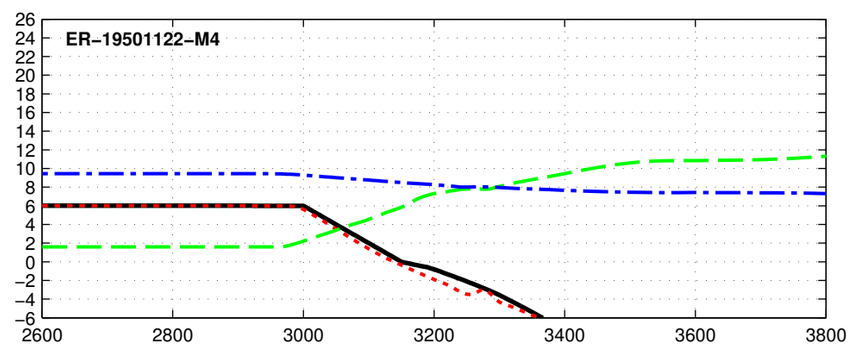












Initial Final Max Hs Max Water Level

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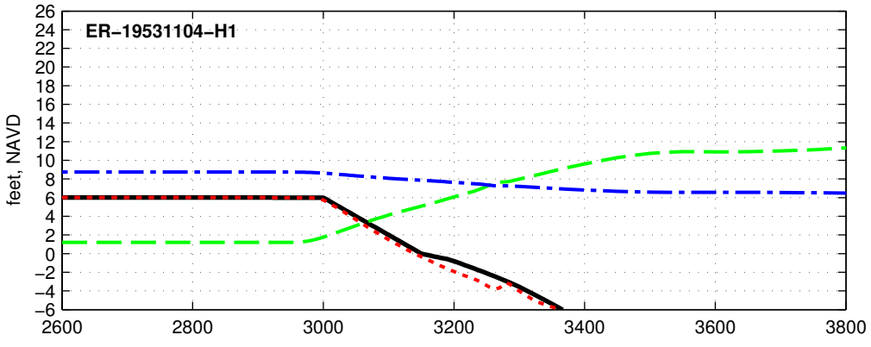

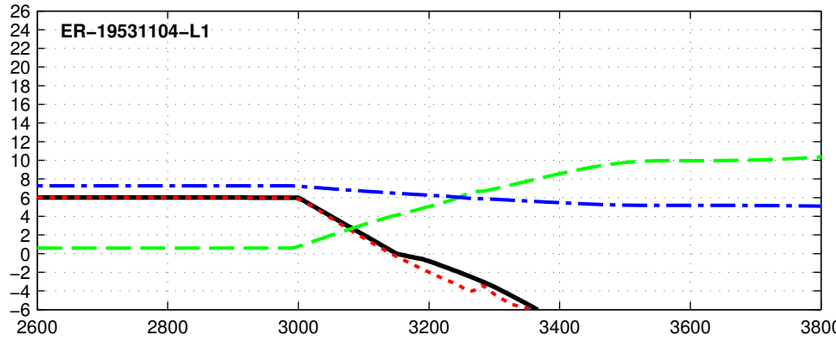

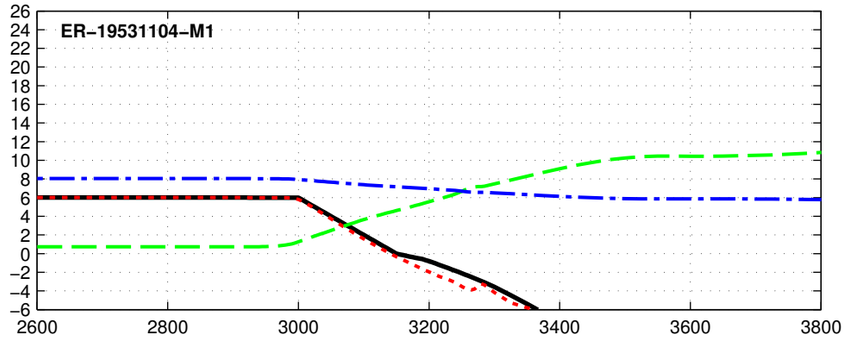

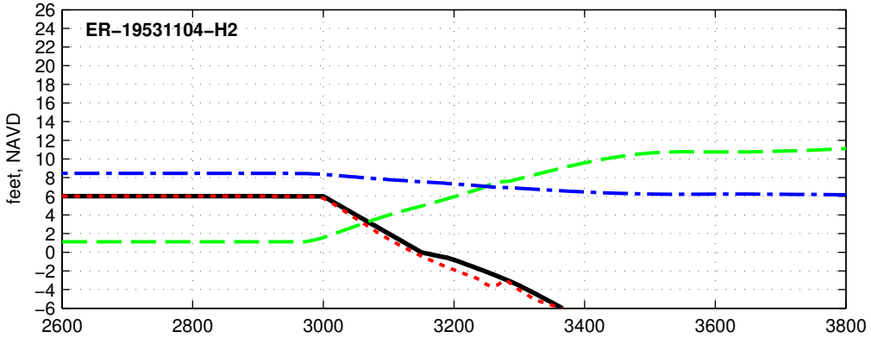

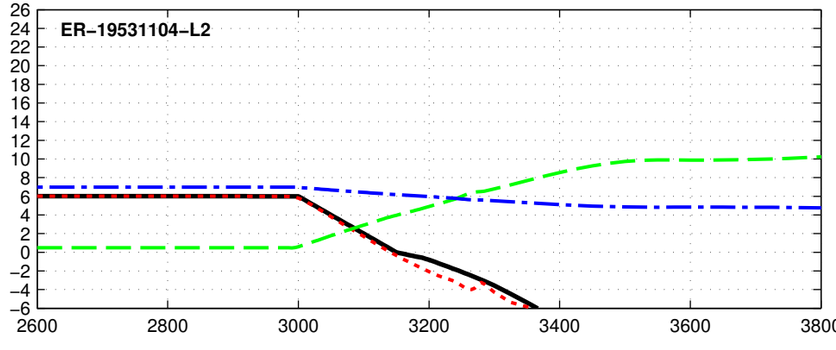

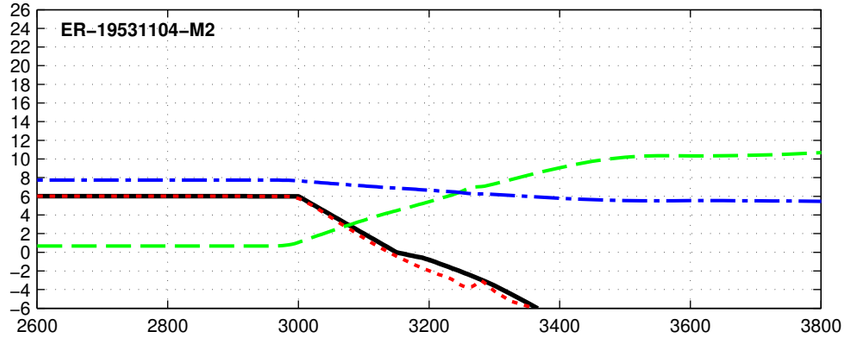

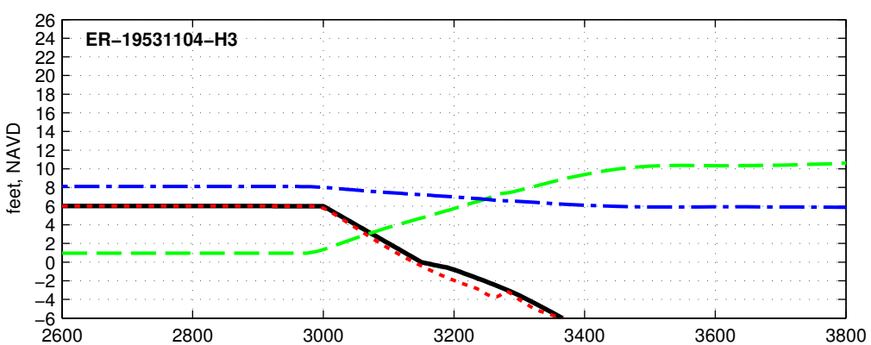

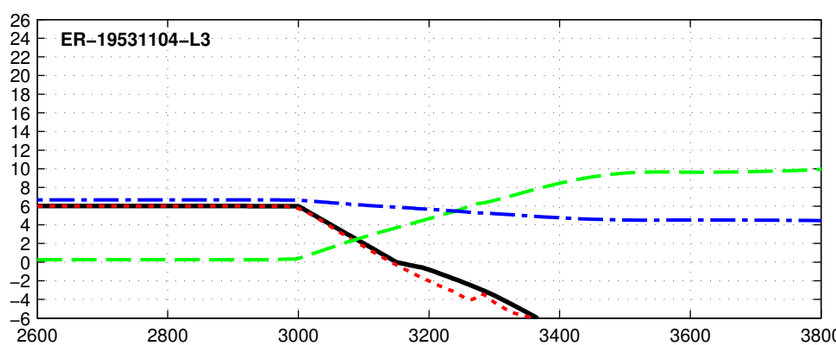

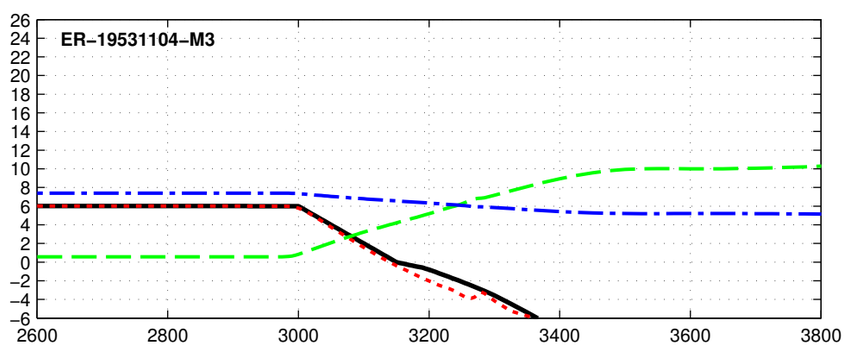

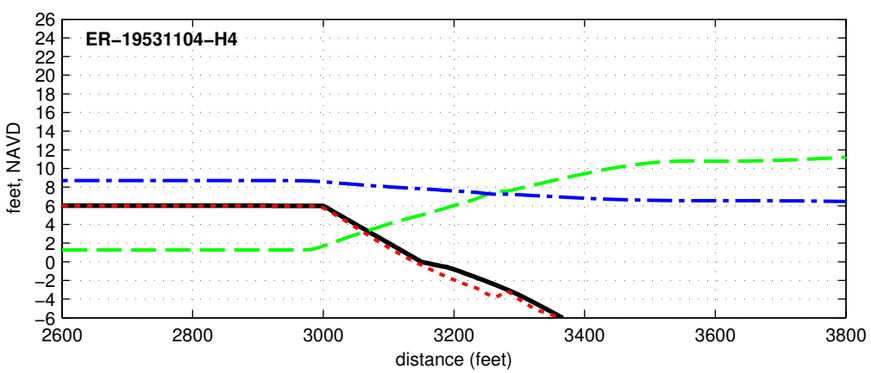

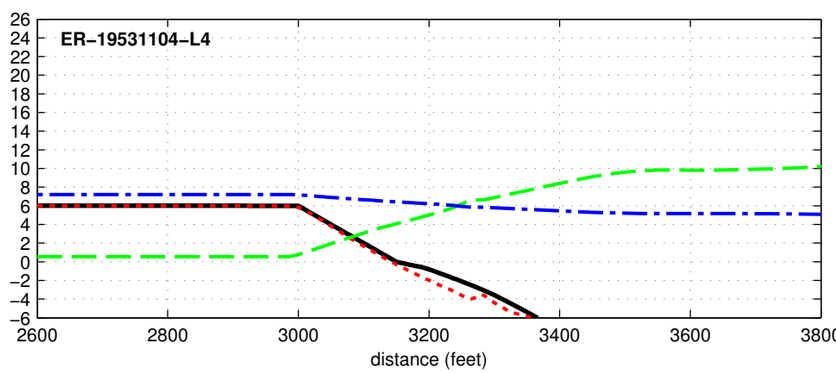

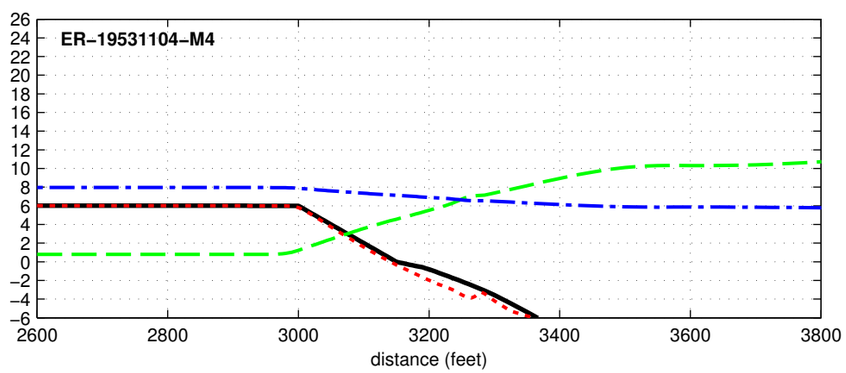

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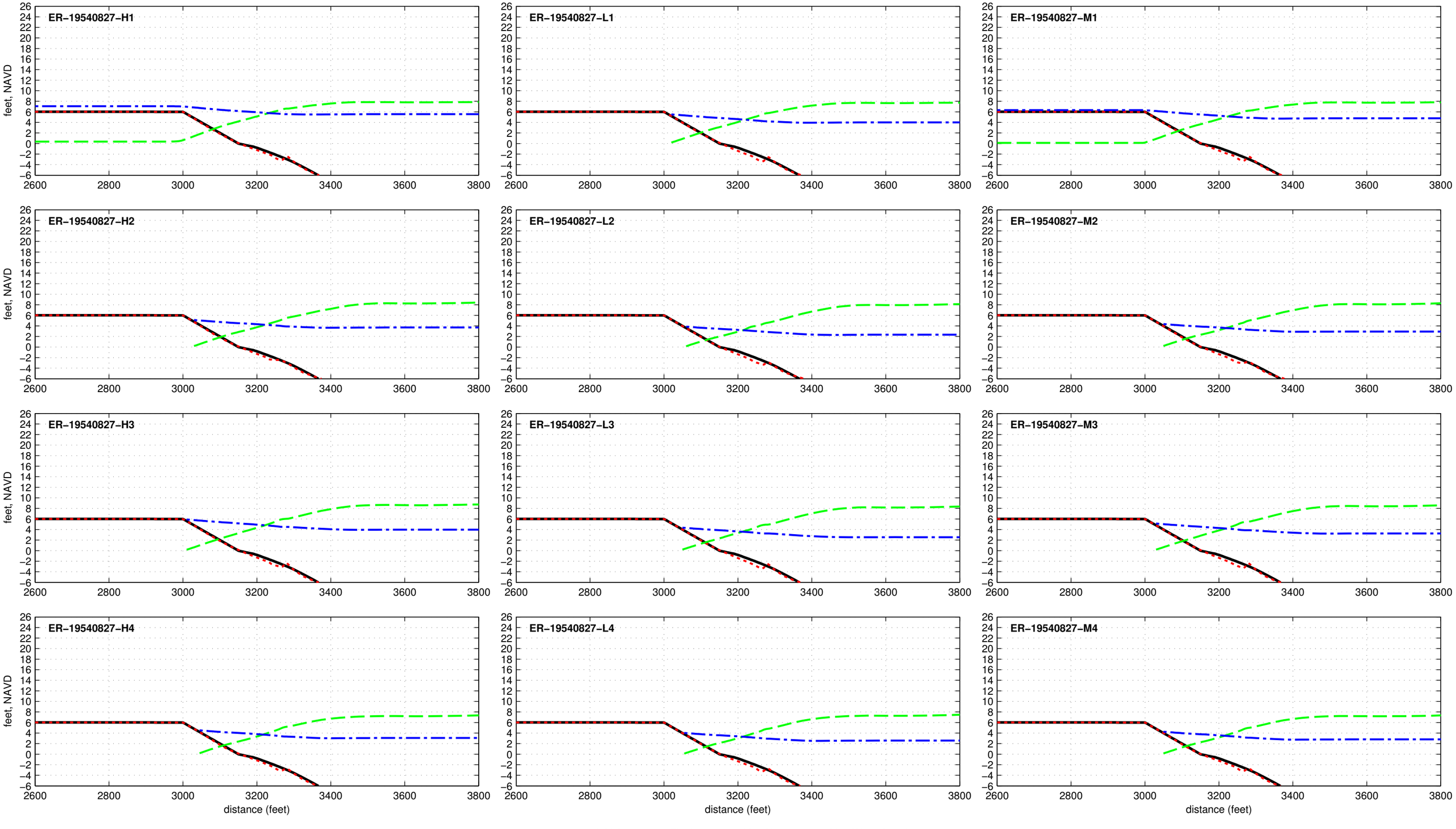

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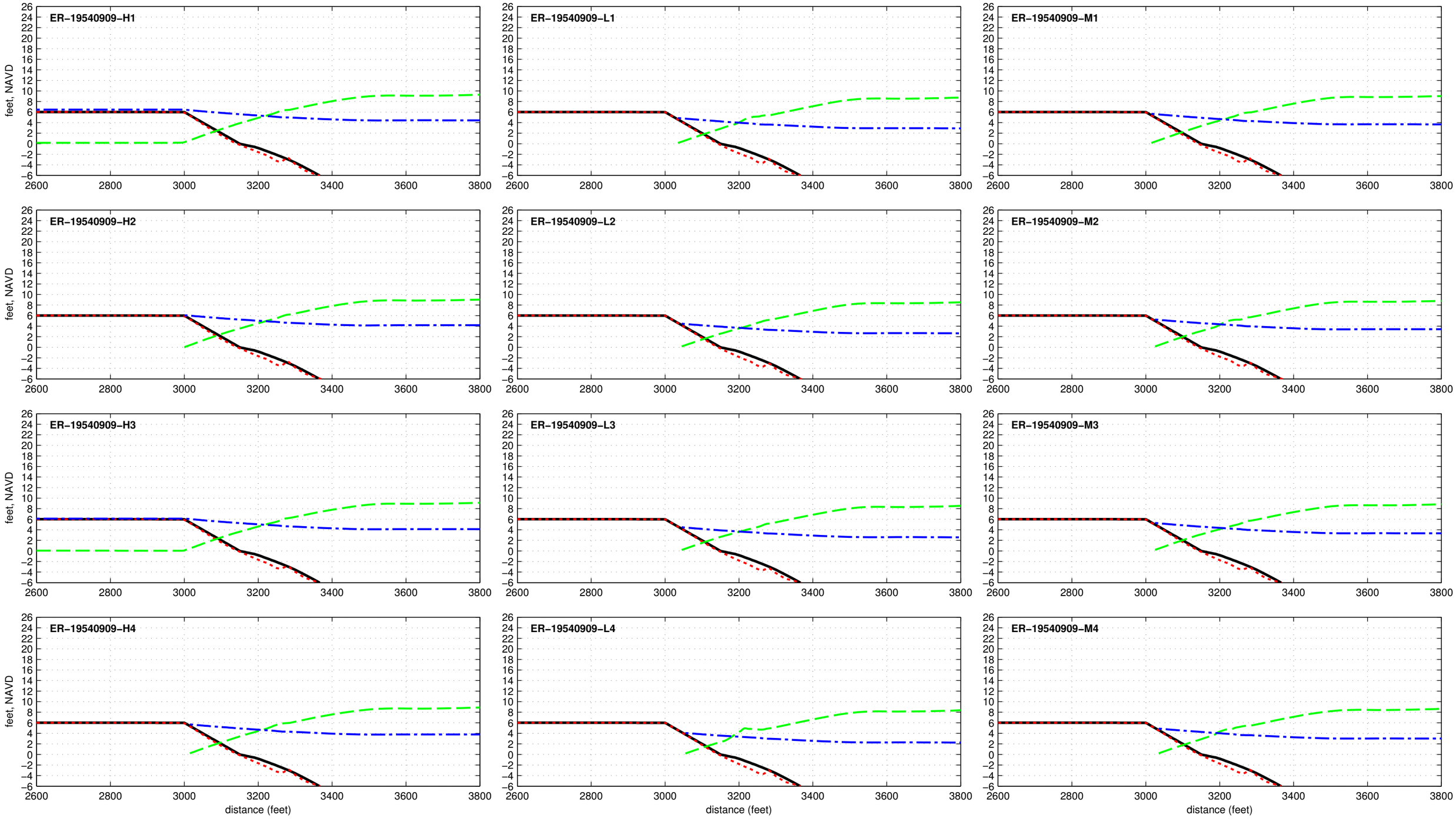

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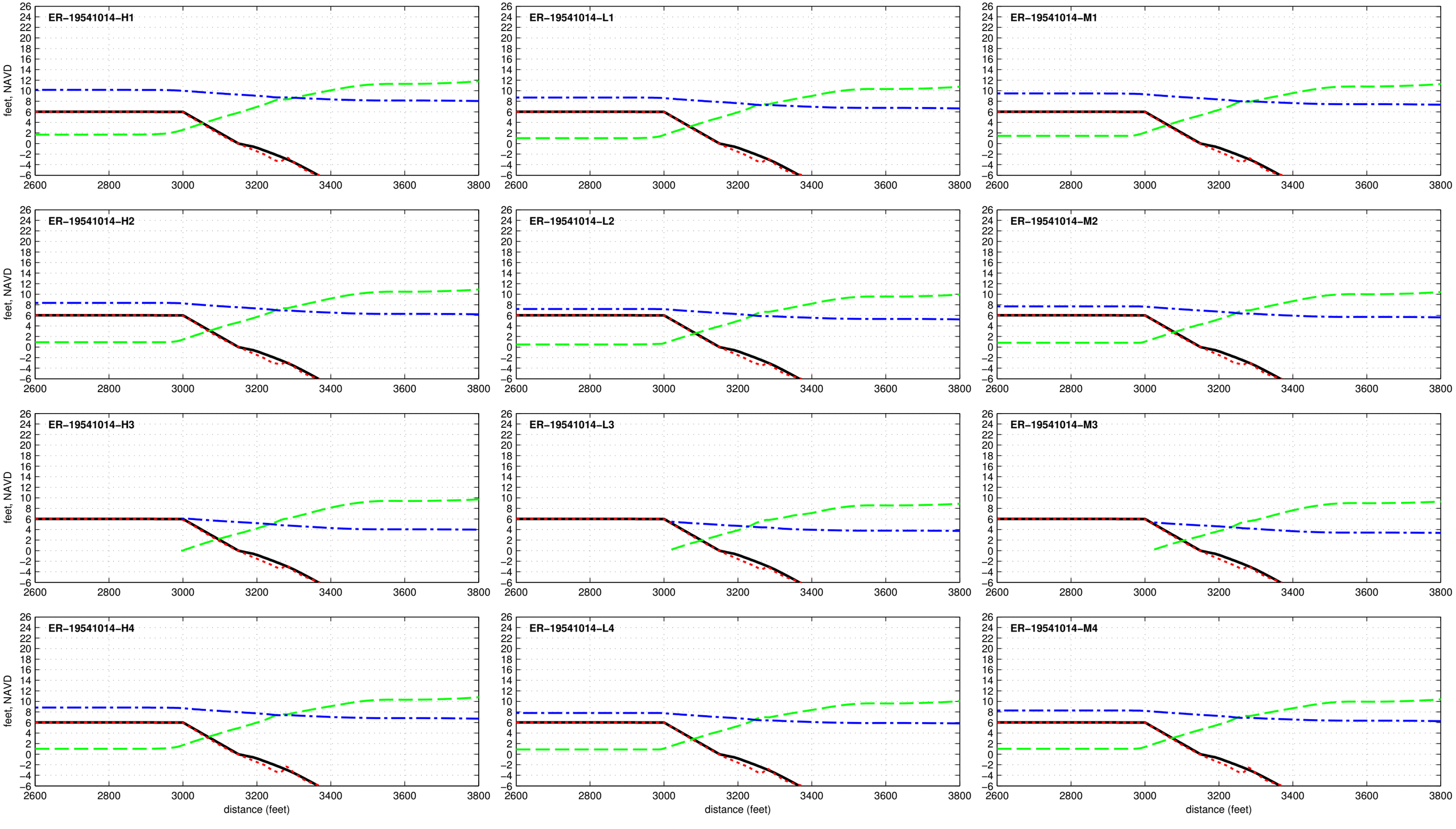

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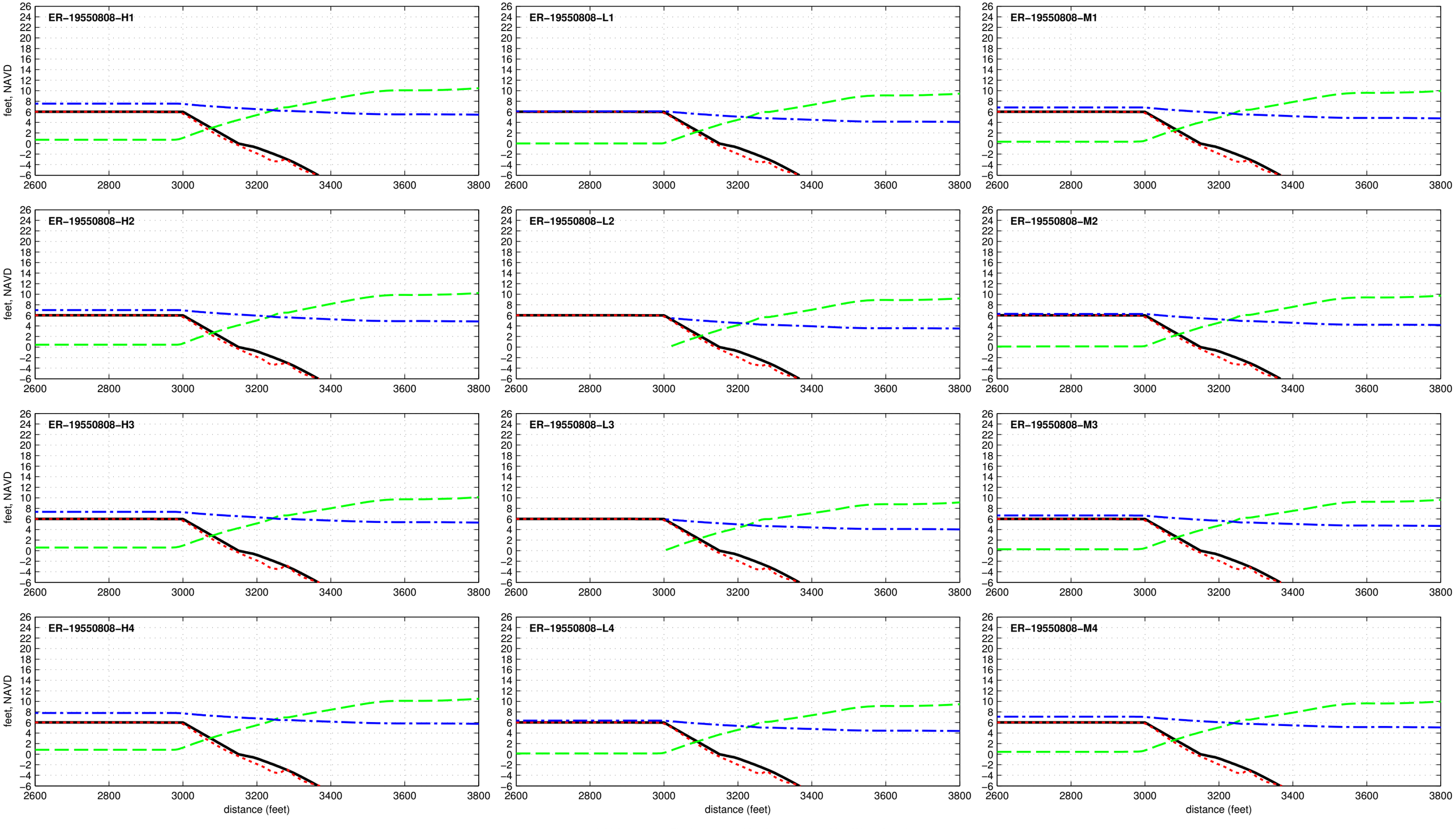

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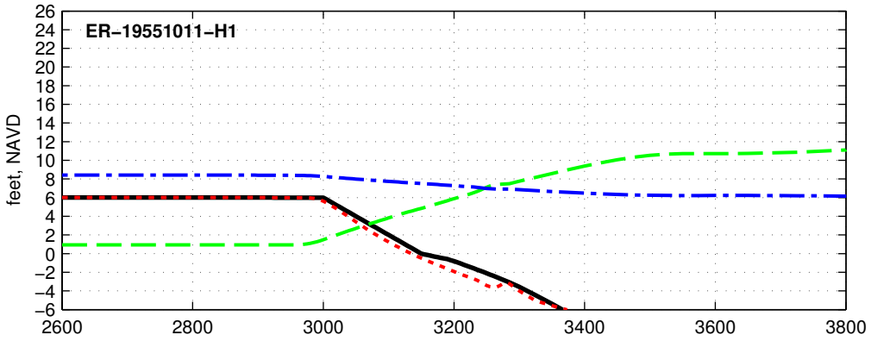
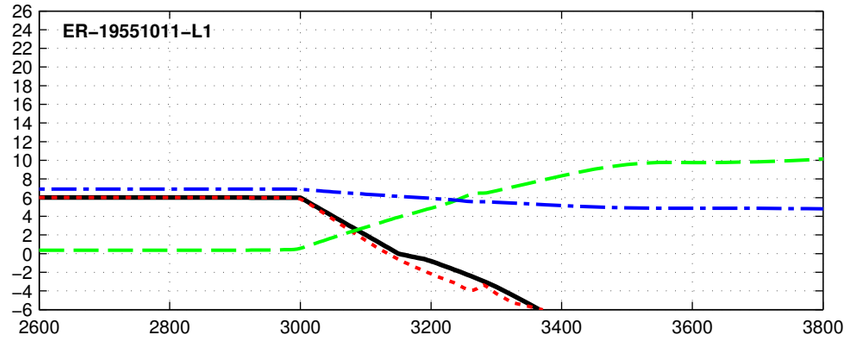
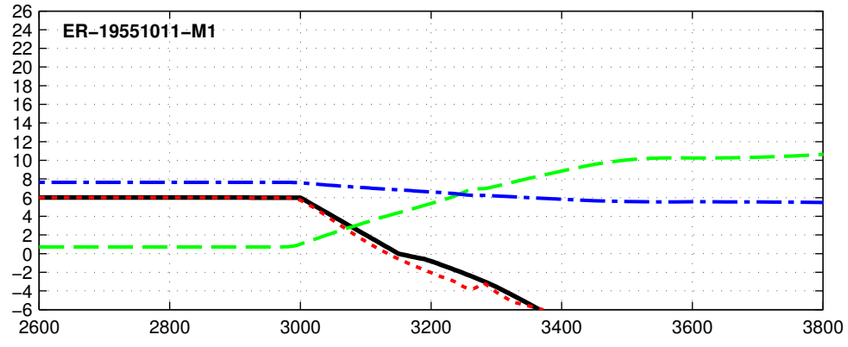
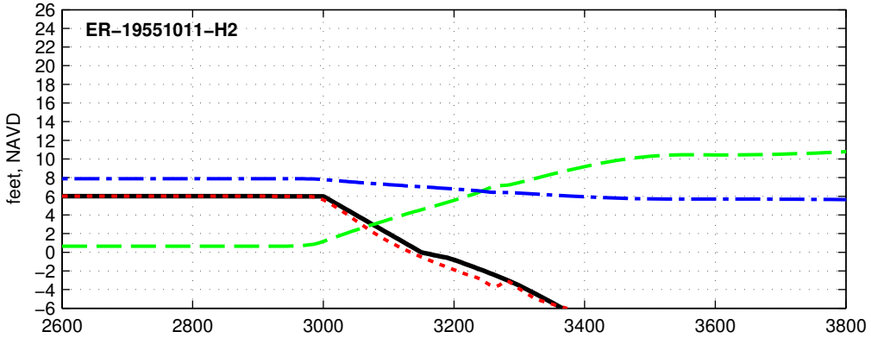
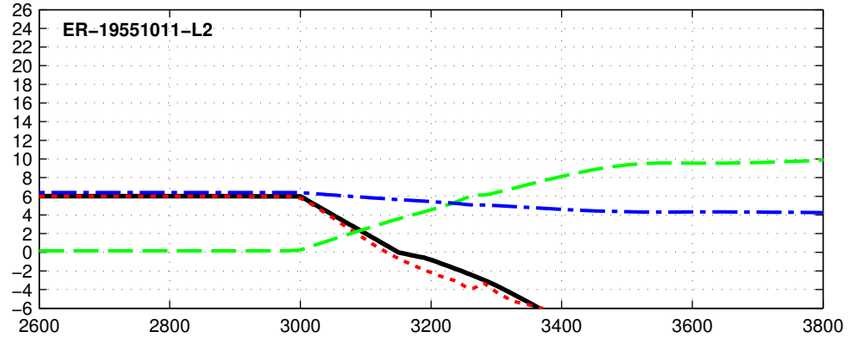
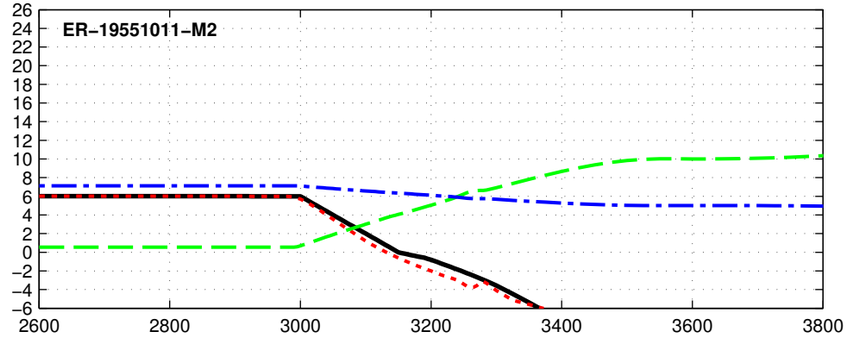


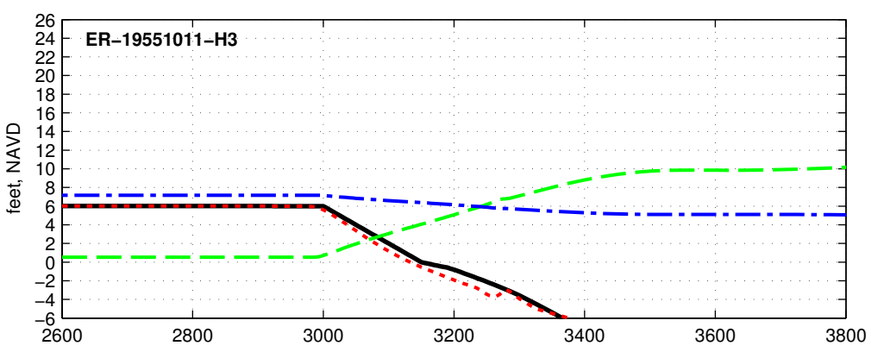


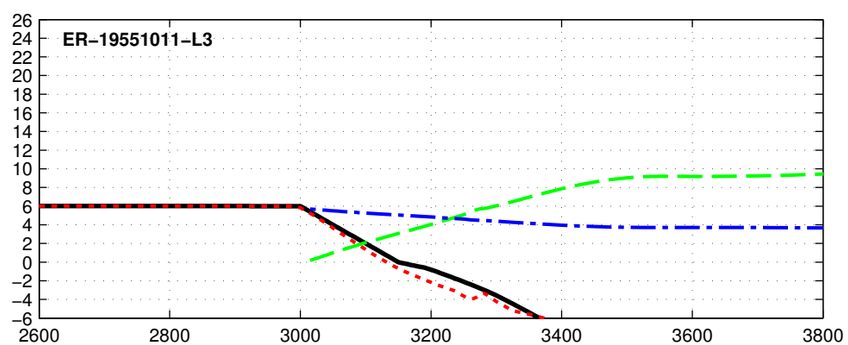


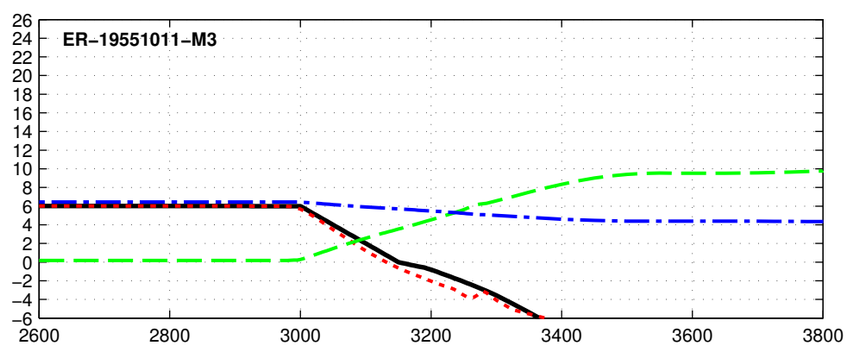


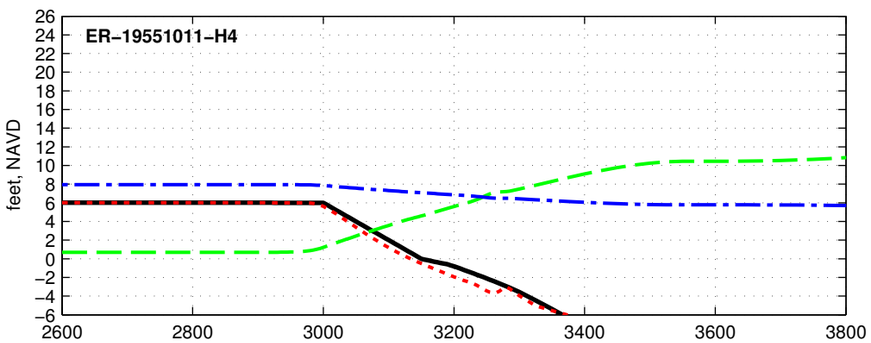


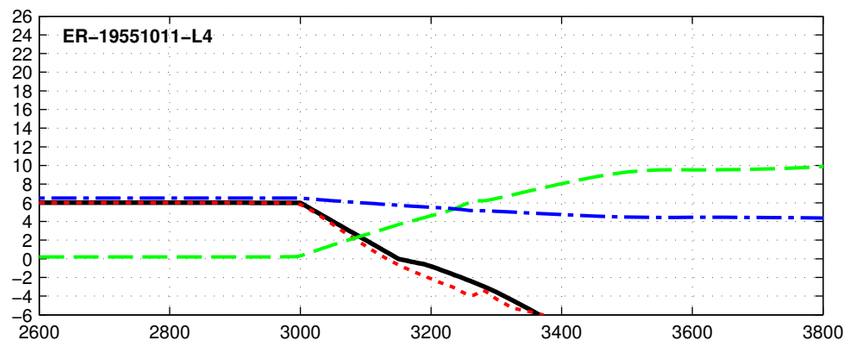


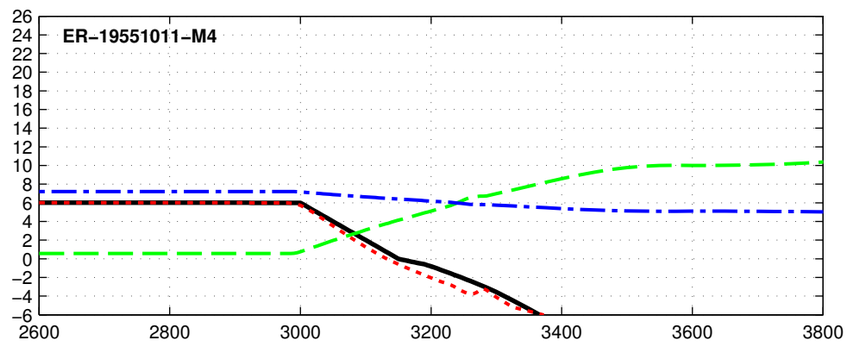






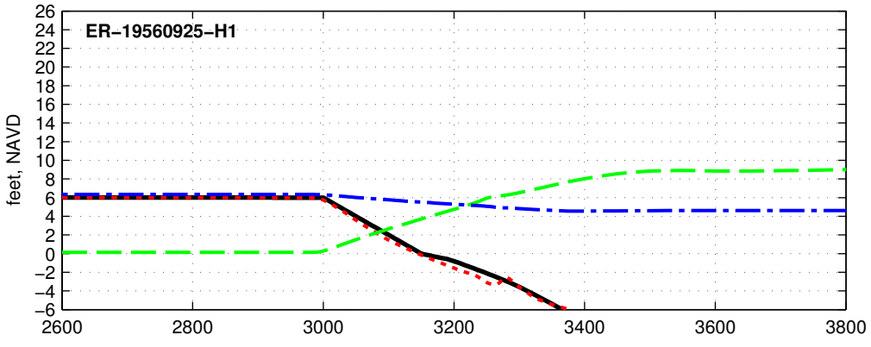


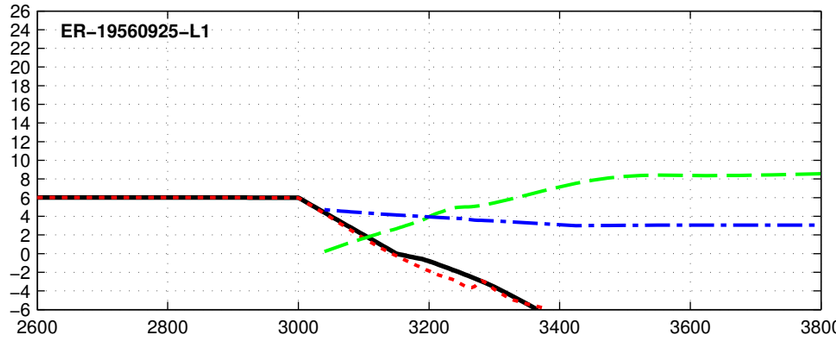


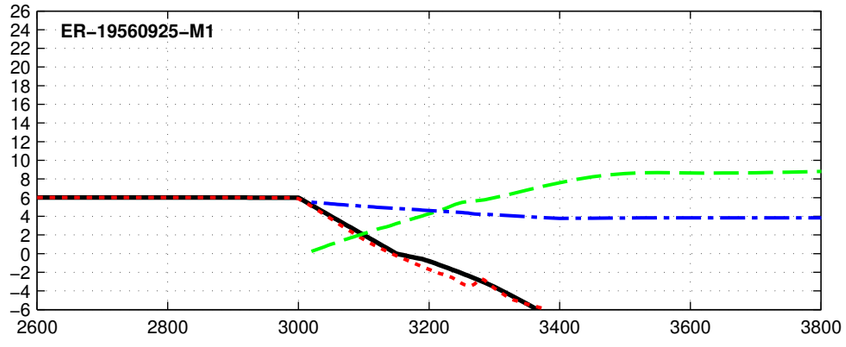


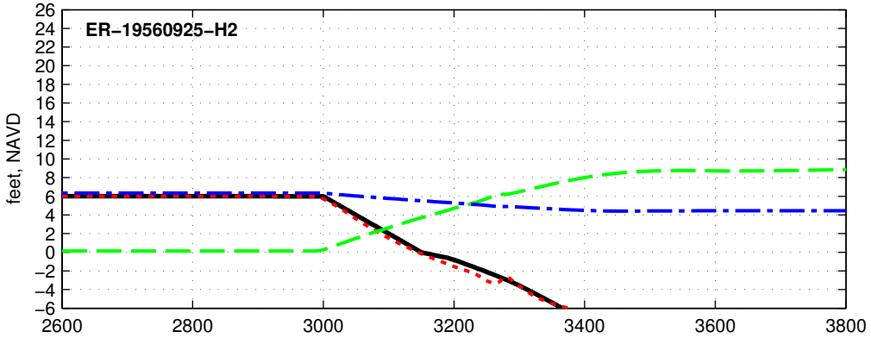
Initial Final Max Hs Max Water Level

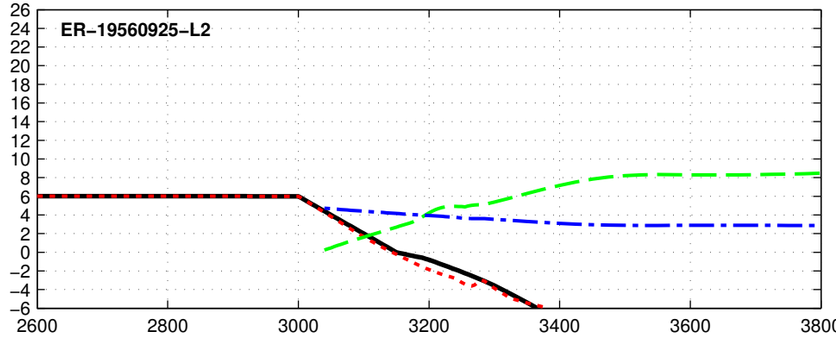
Dune R1T1-6.0-000-000 - ER-19560925

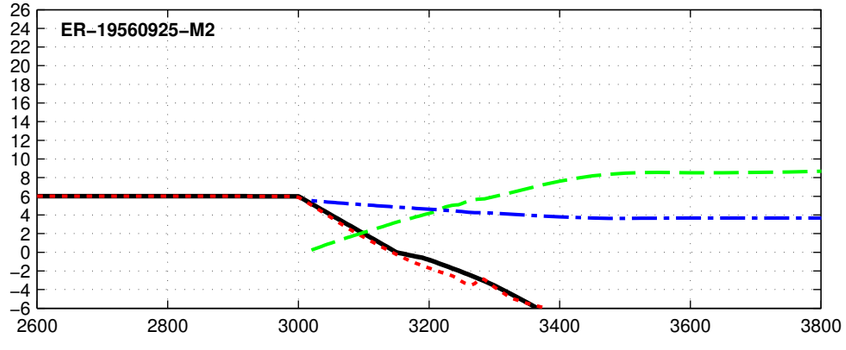


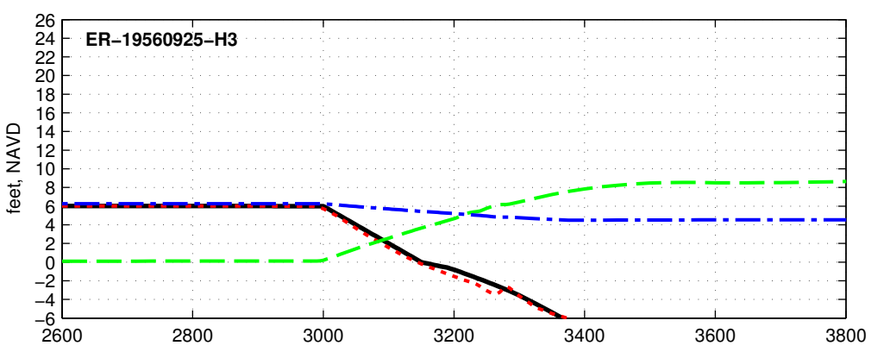


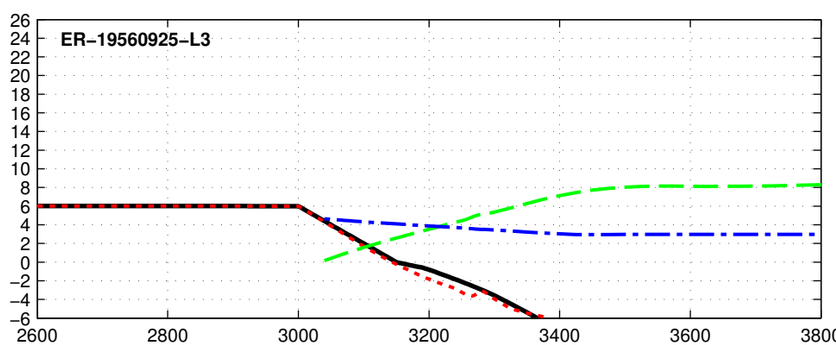


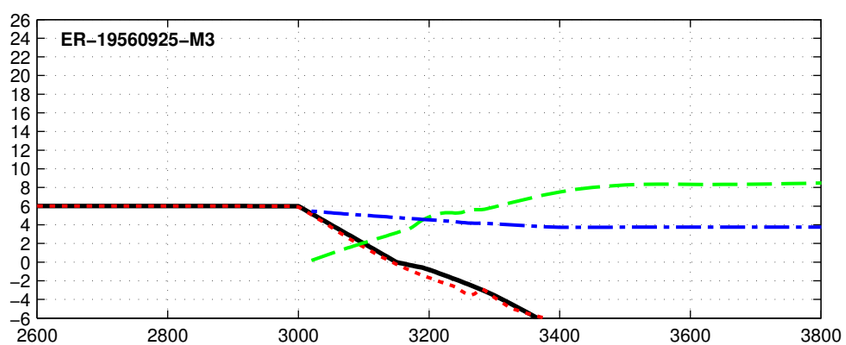


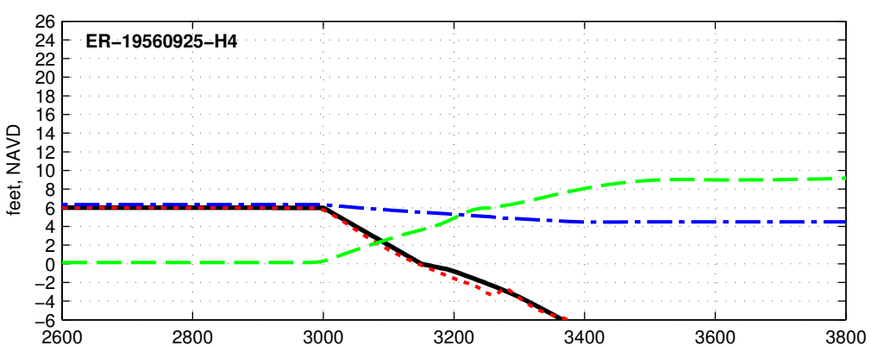


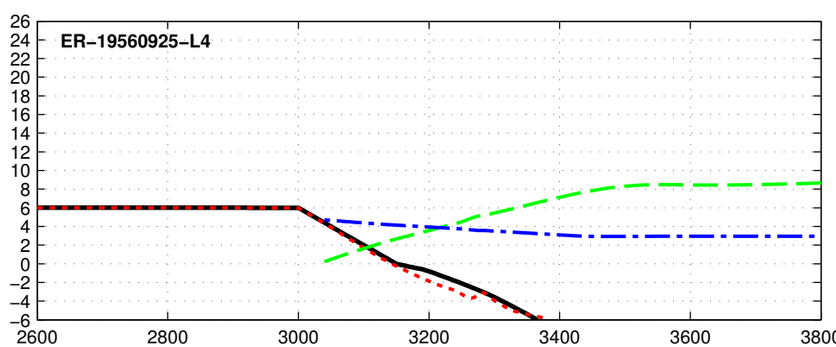


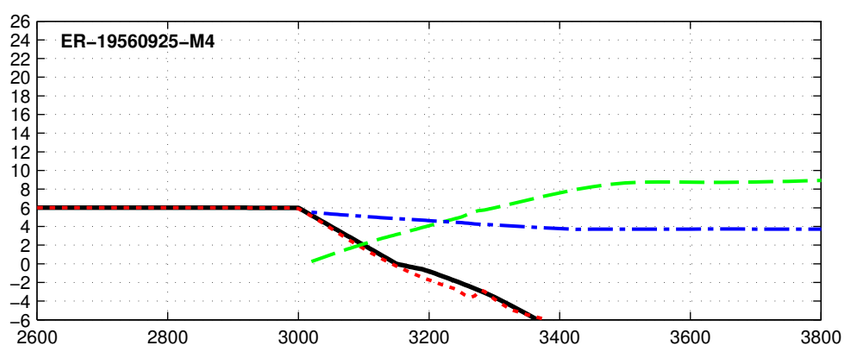










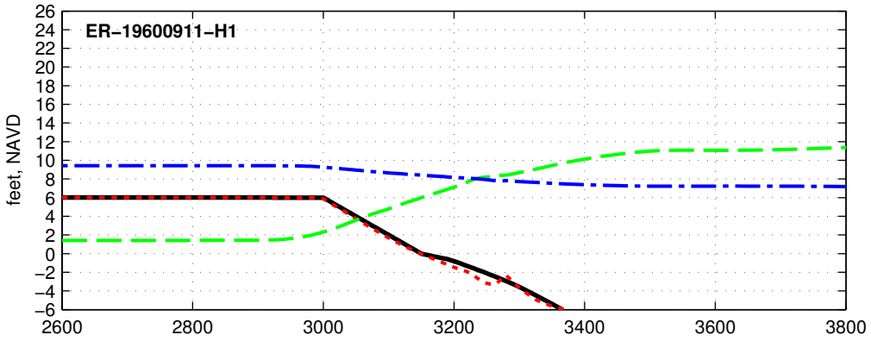


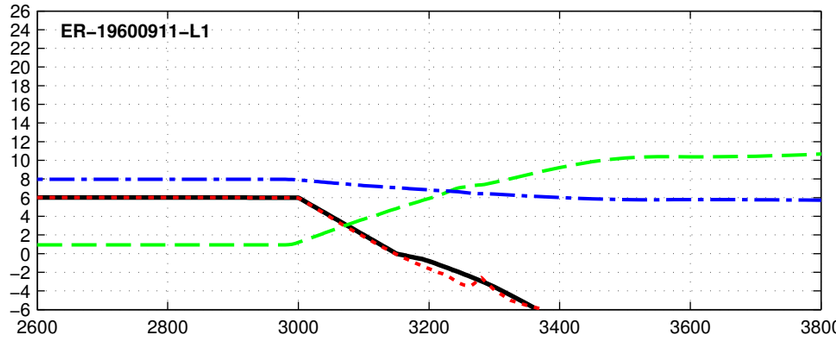
feet, NAVD

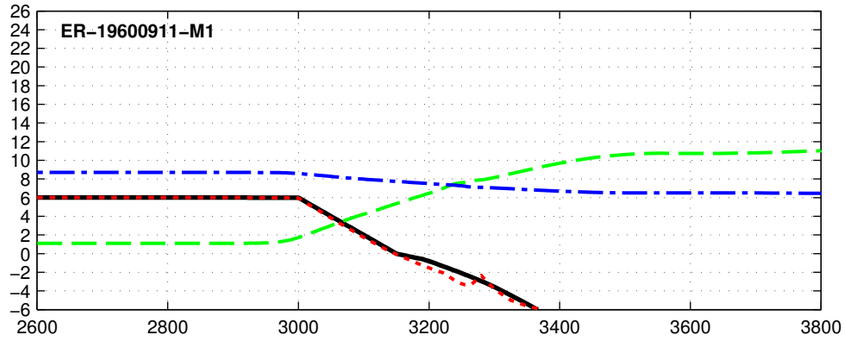
distance (feet)

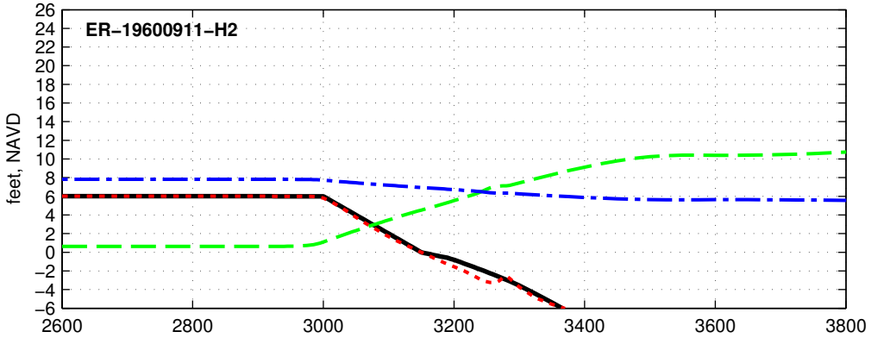
Initial Final Max Hs Max Water Level

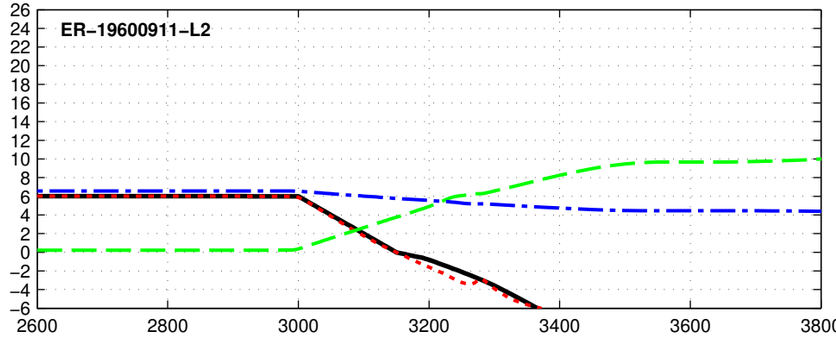
Dune R1T1-6.0-000-000 - ER-19600911

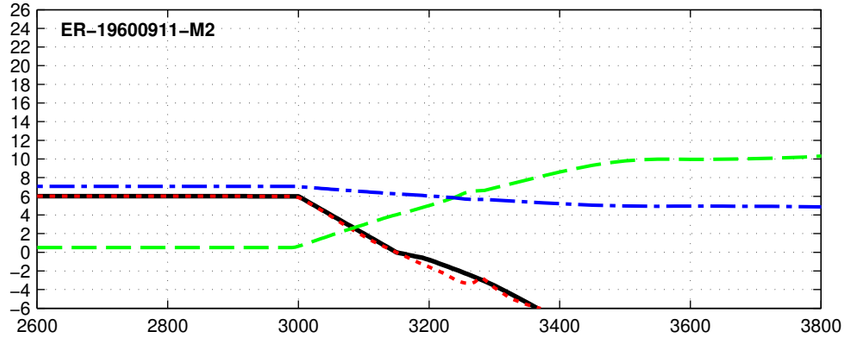


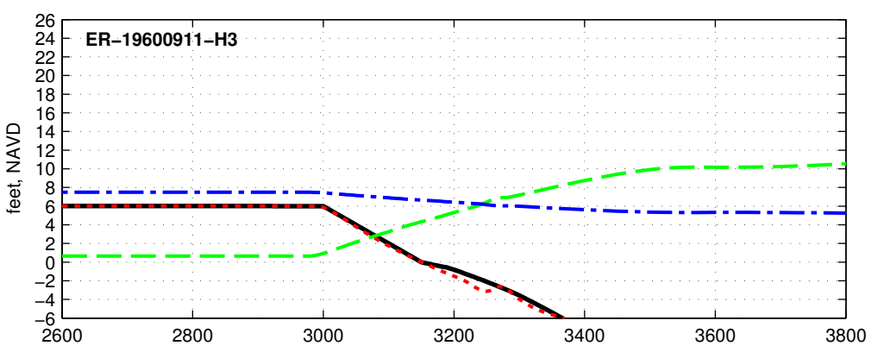


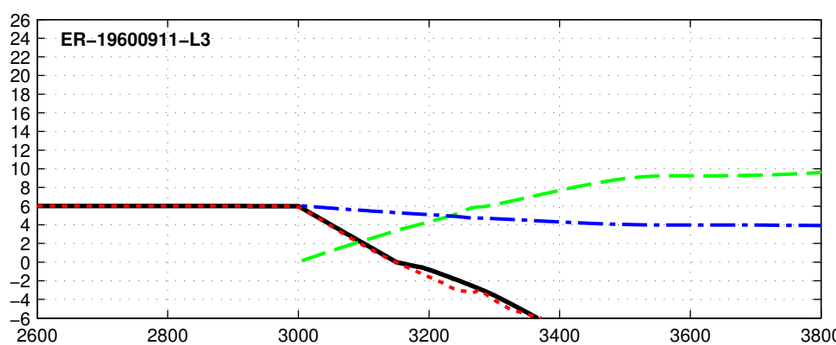


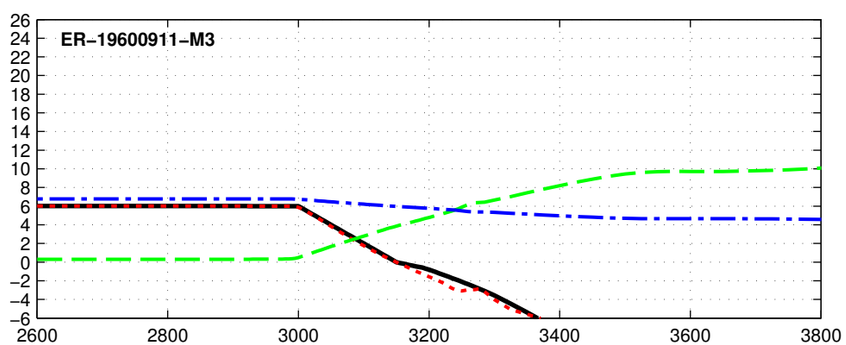


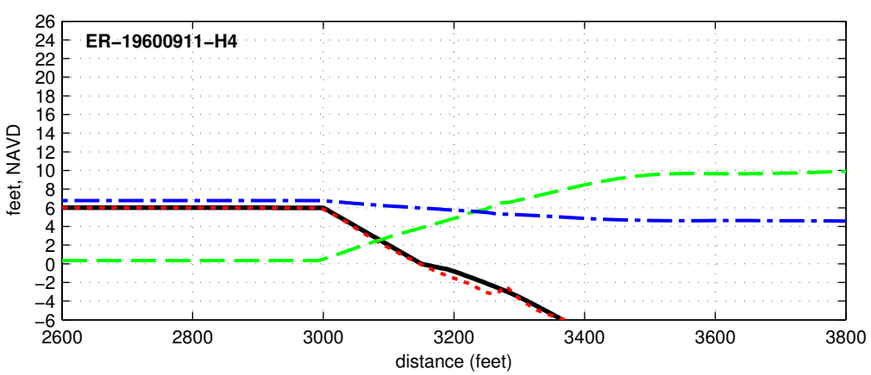


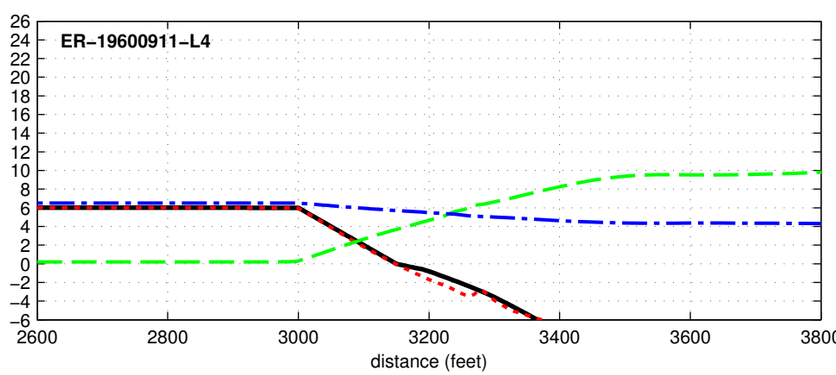


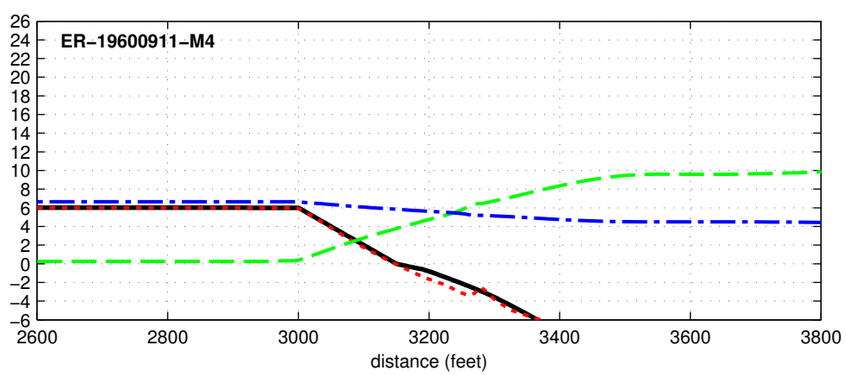










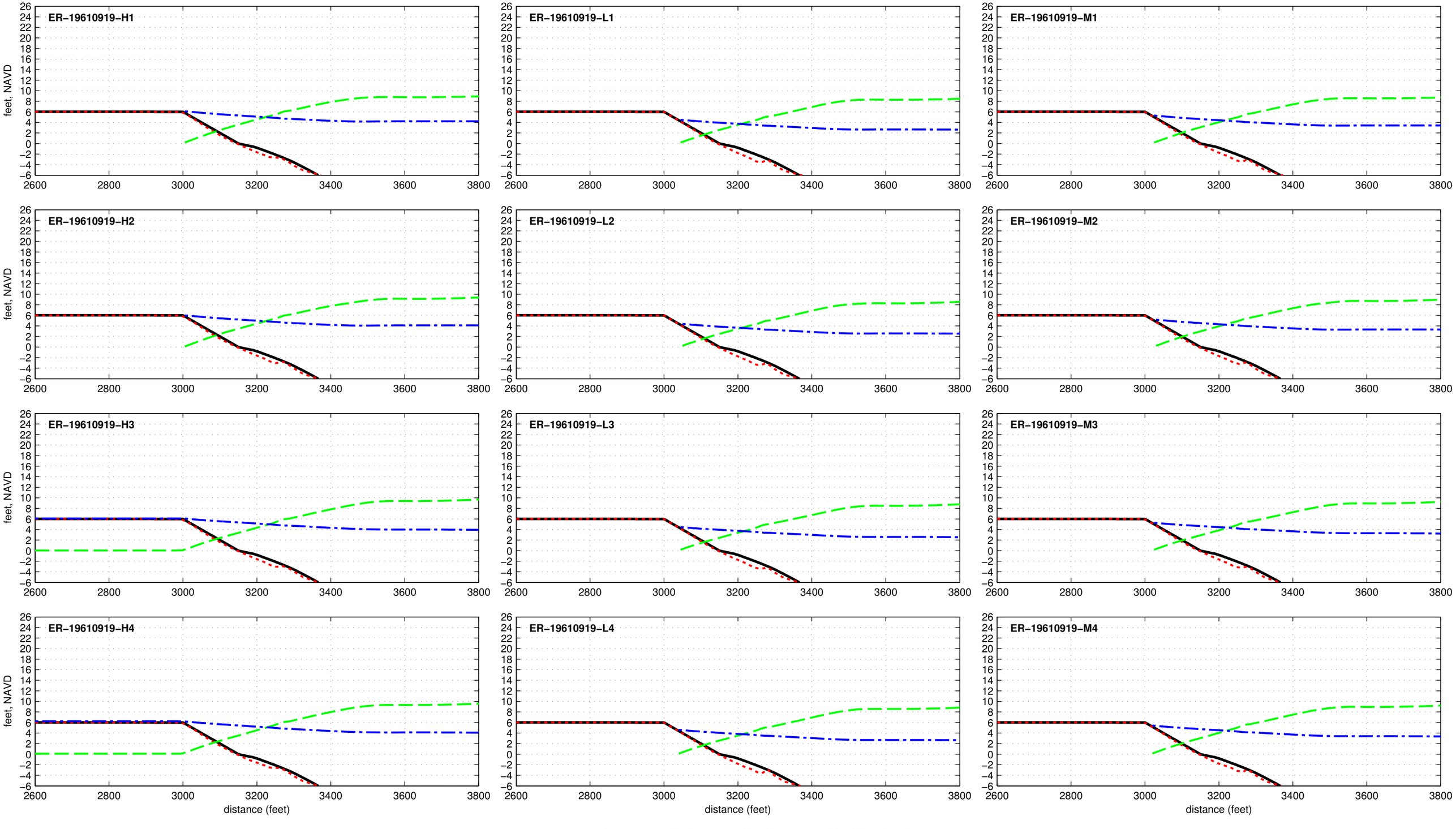


feet, NAVD

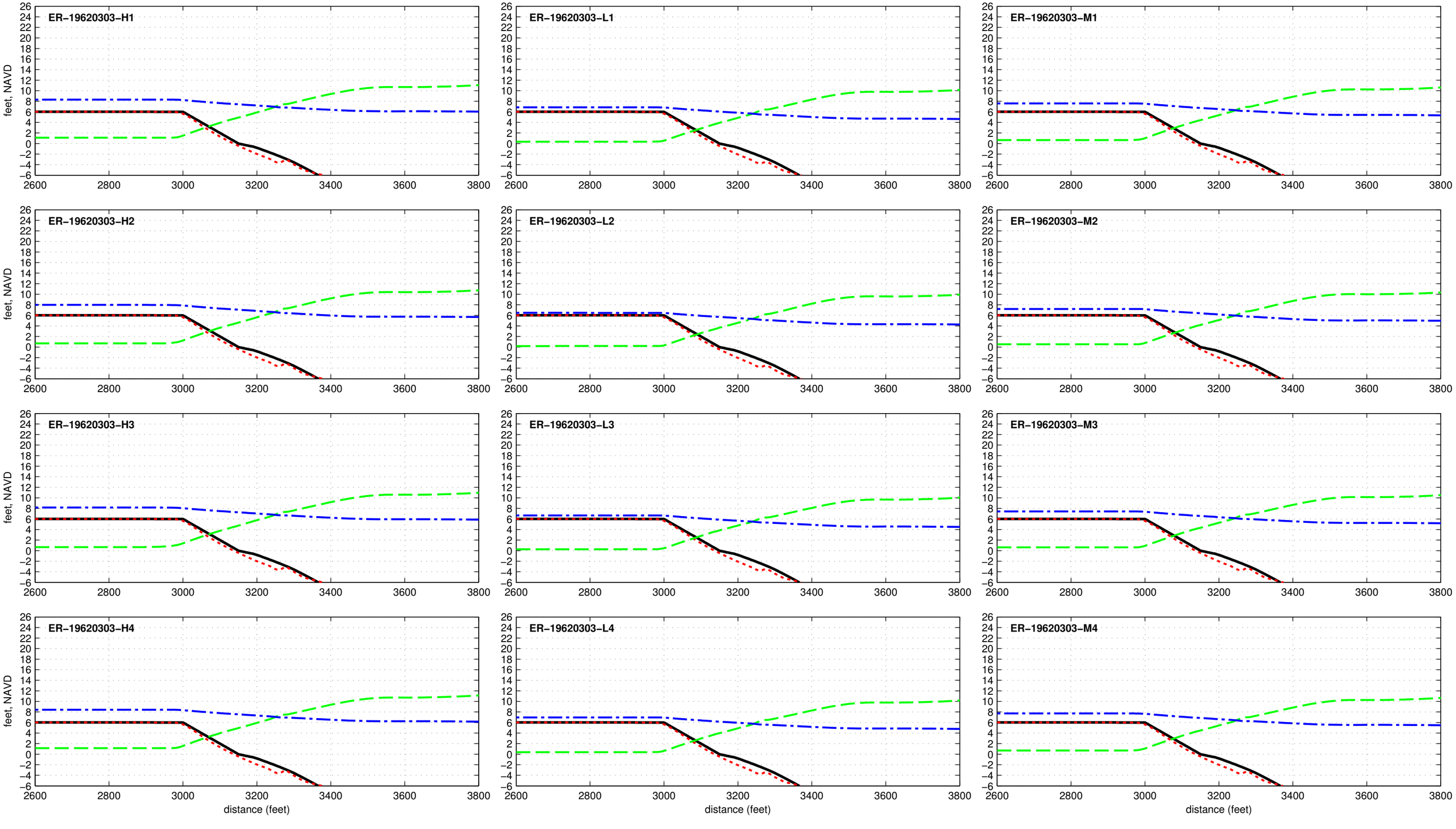
distance (feet)

Initial Final Max Hs Max Water Level

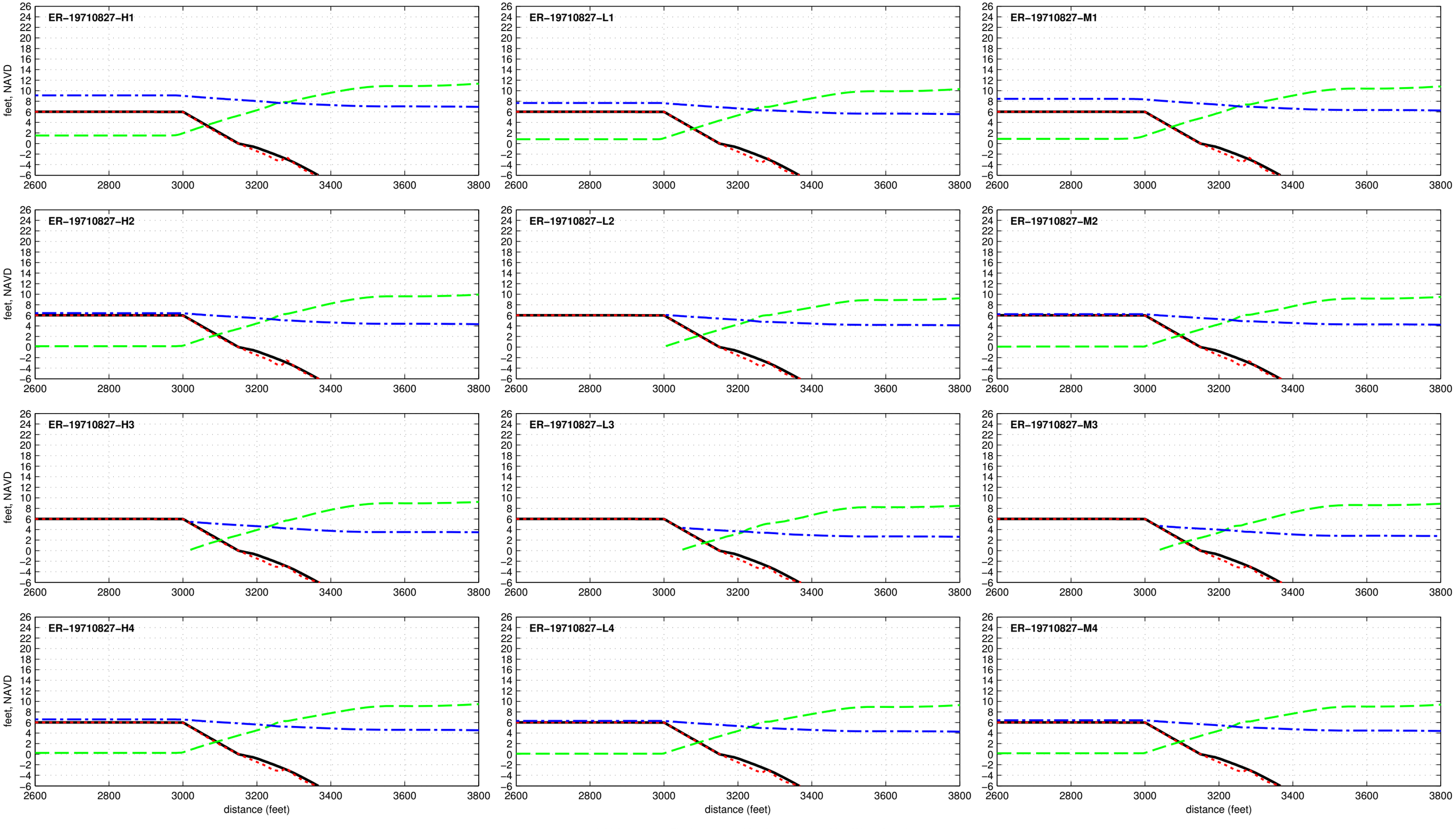
Dune R1T1-6.0-000-000 - ER-19610919



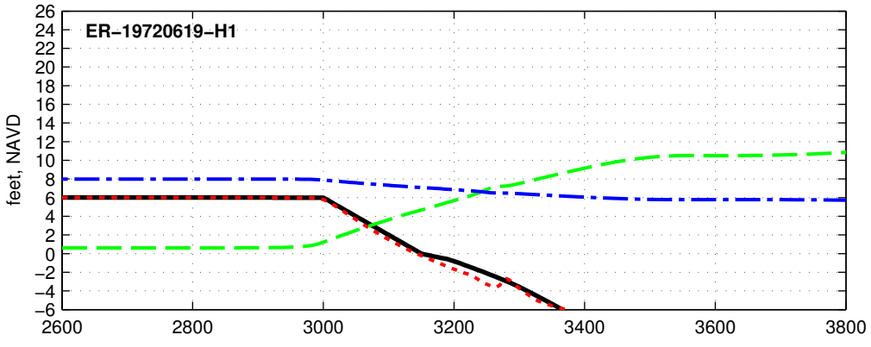
Dune R1T1-6.0-000-000 - ER-19620303

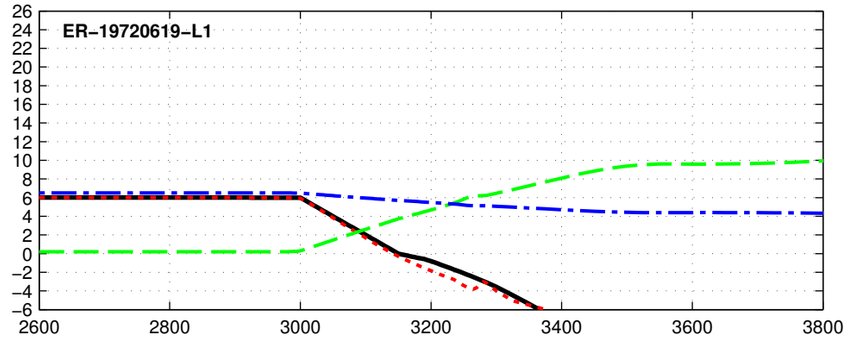


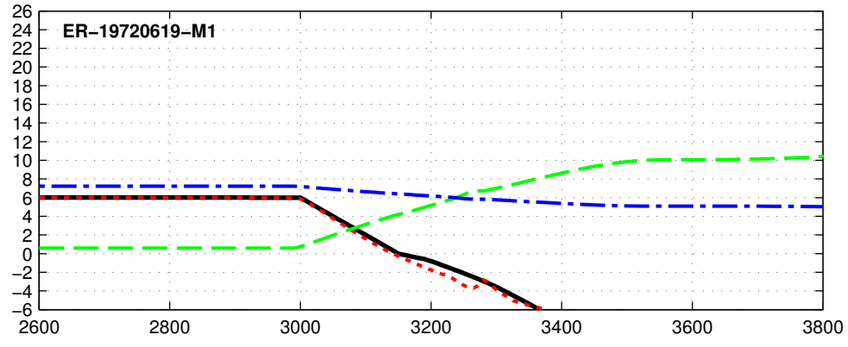
Dune R1T1-6.0-000-000 - ER-19710827

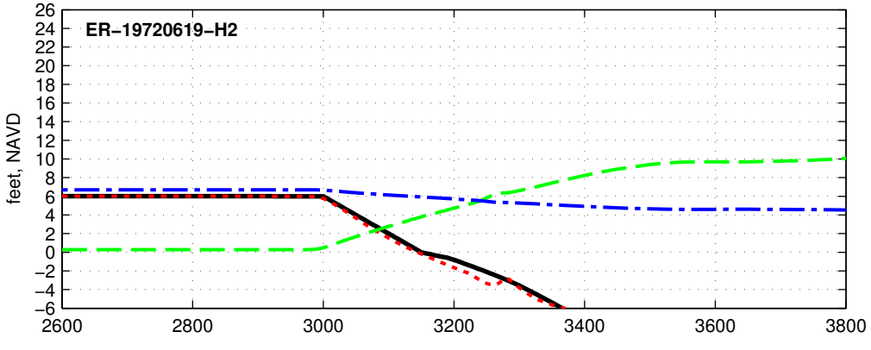


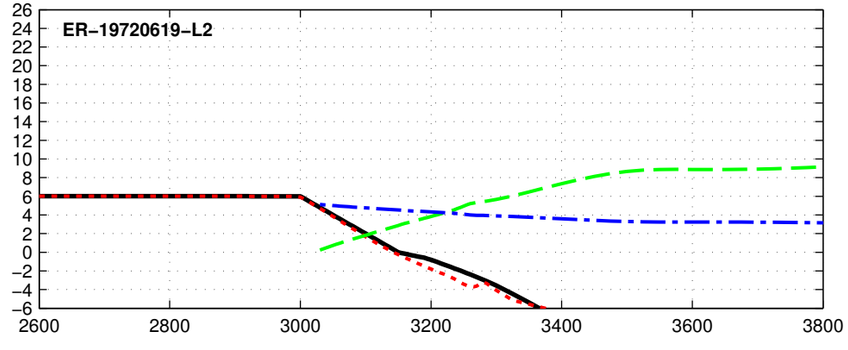
Dune R1T1-6.0-000-000 - ER-19720619

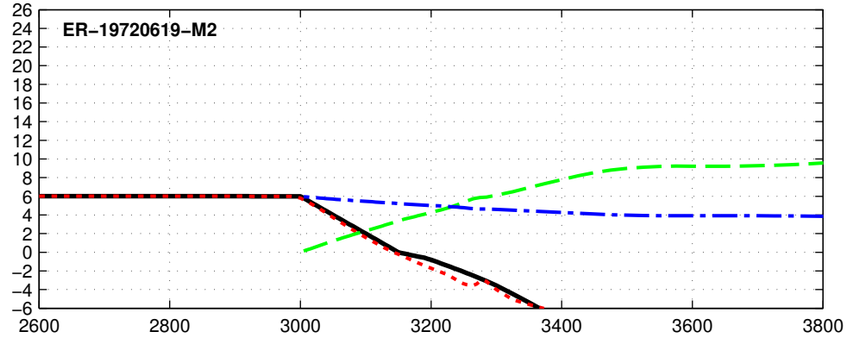


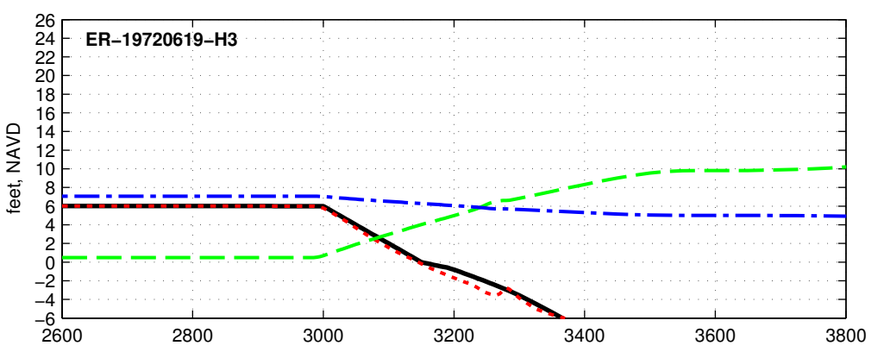


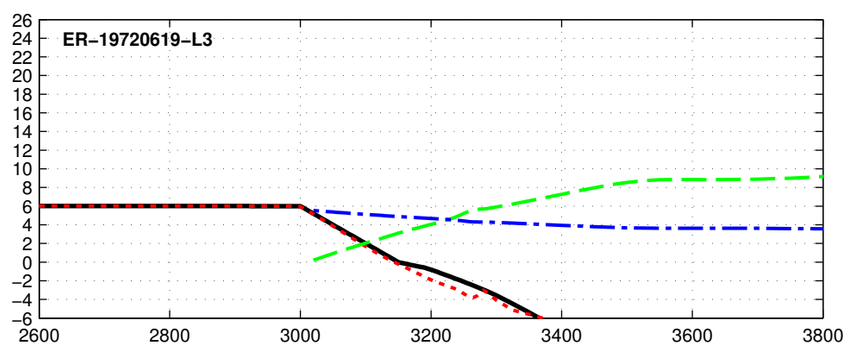


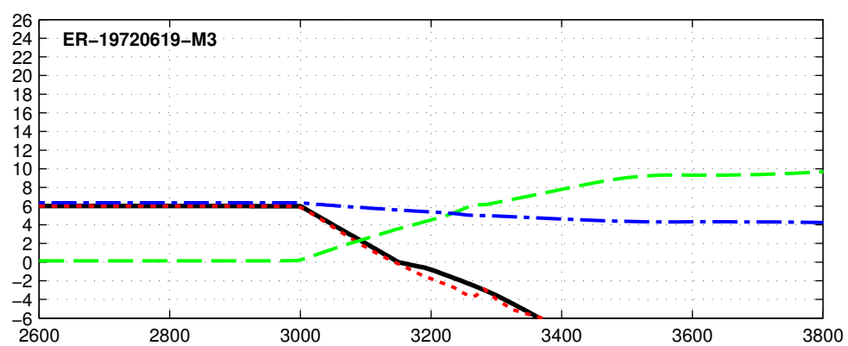


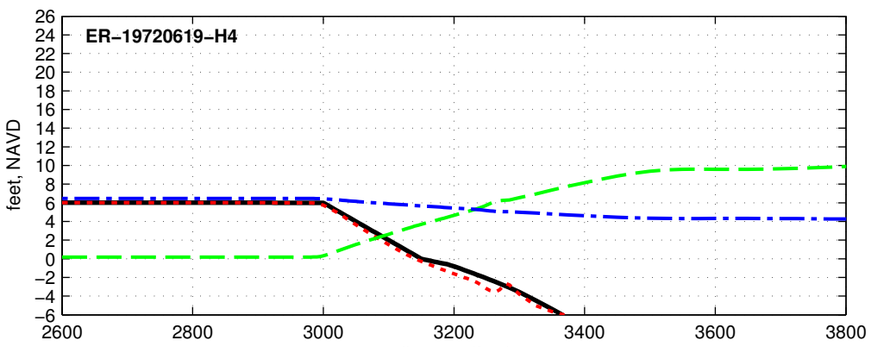


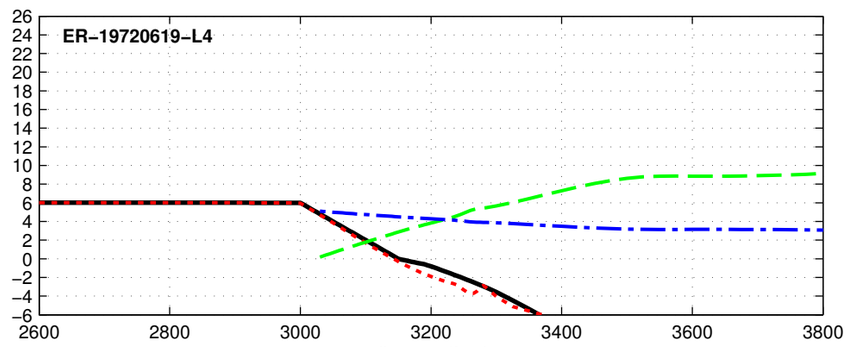


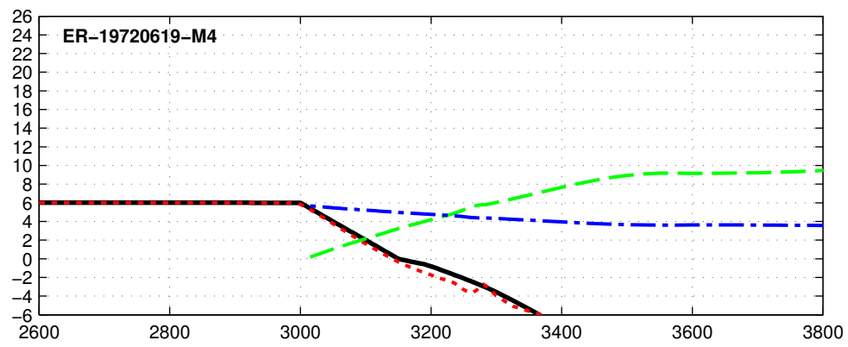






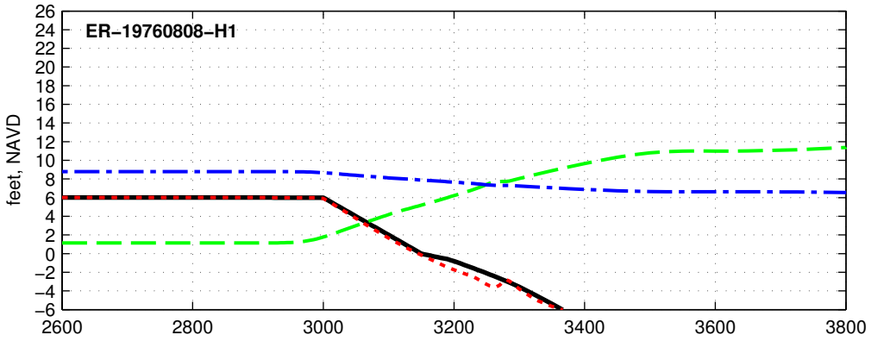


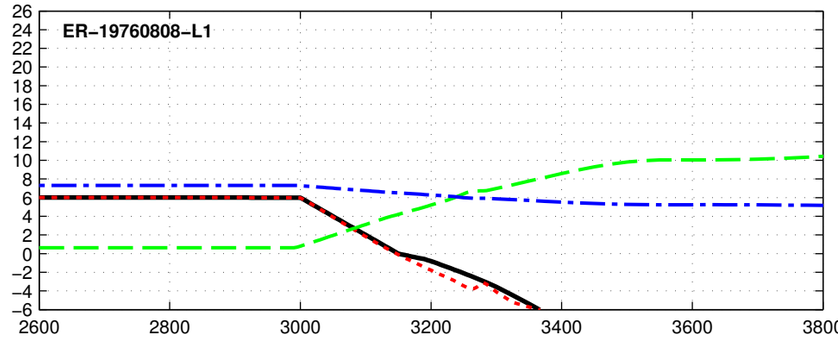


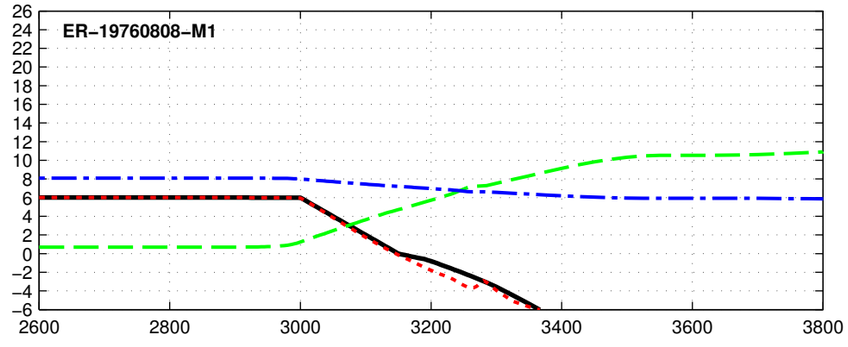


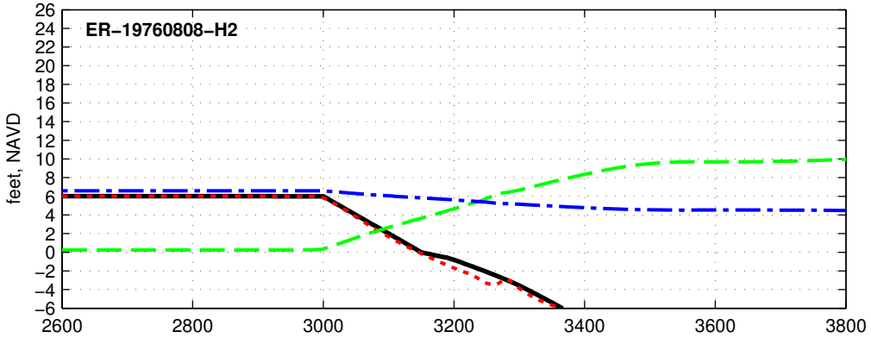
Initial Final Max Hs Max Water Level

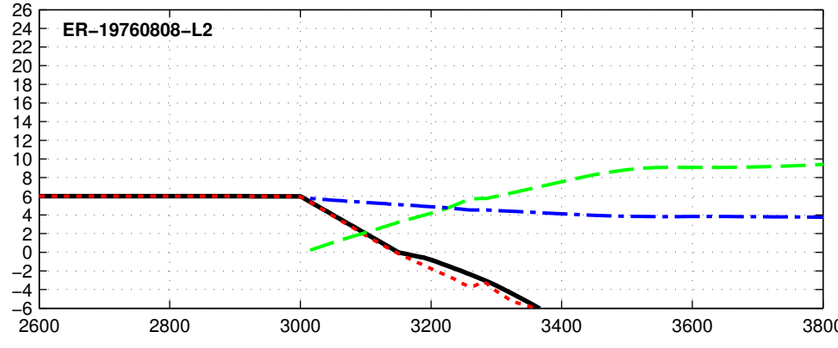
Dune R1T1-6.0-000-000 - ER-19760808

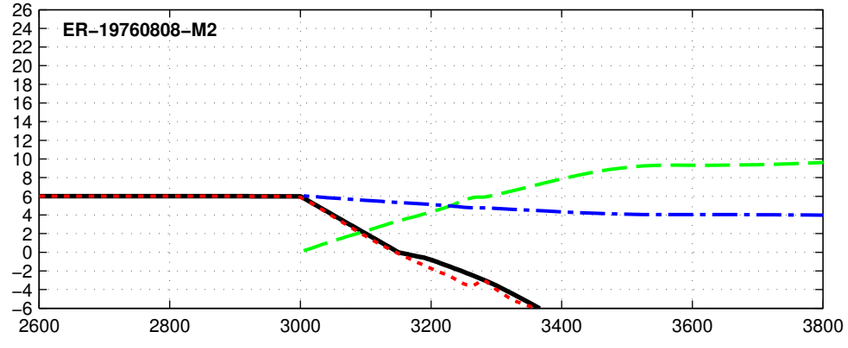


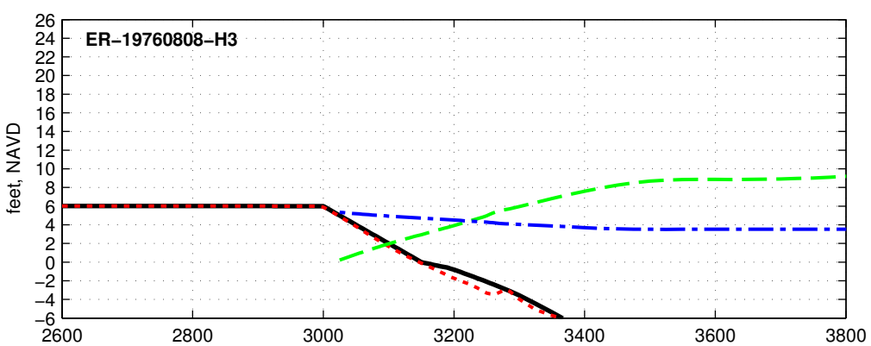


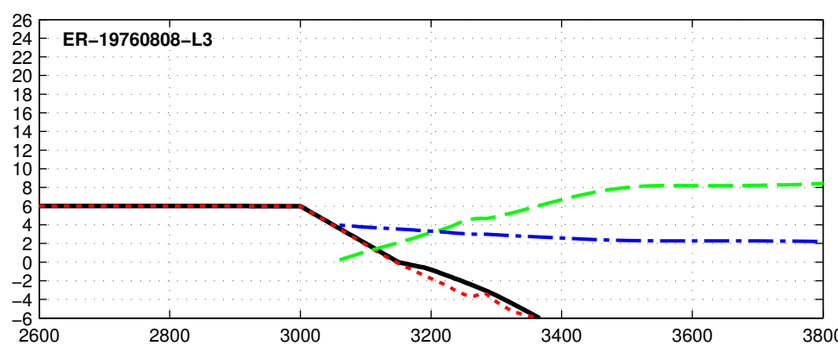


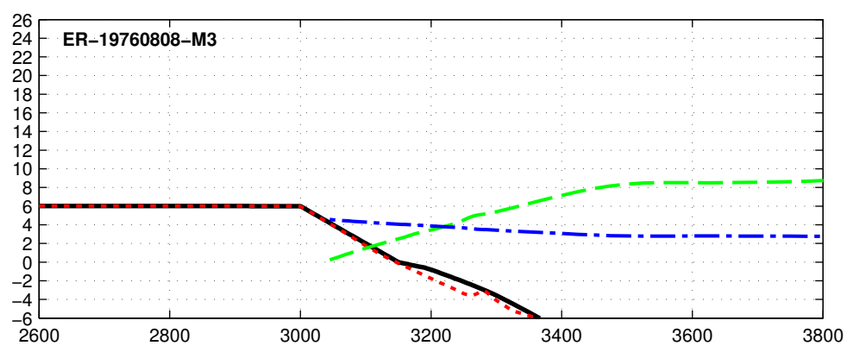


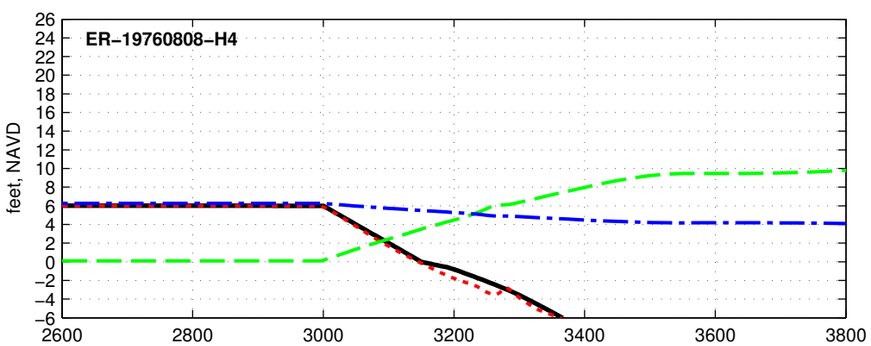


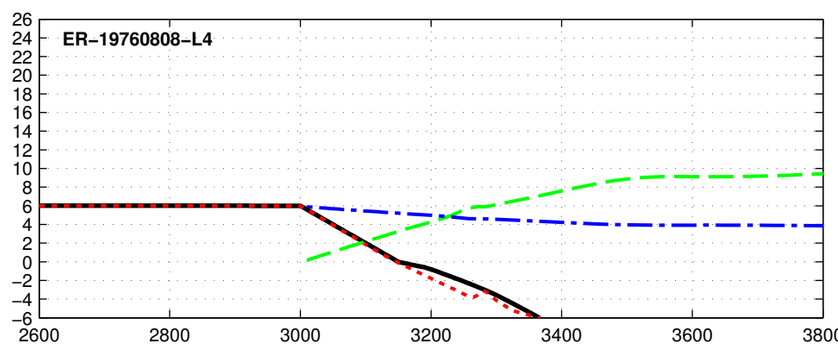


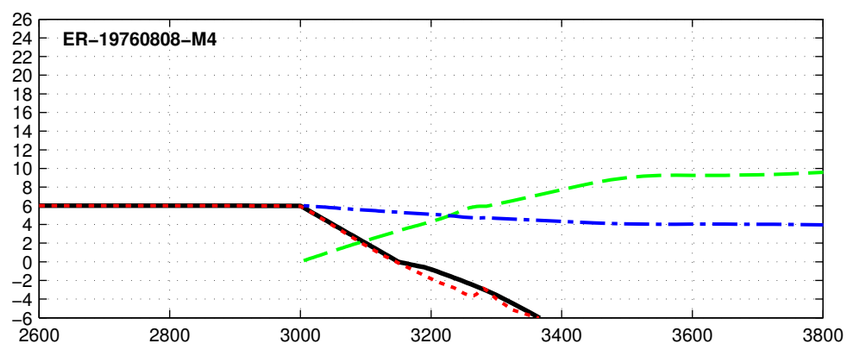






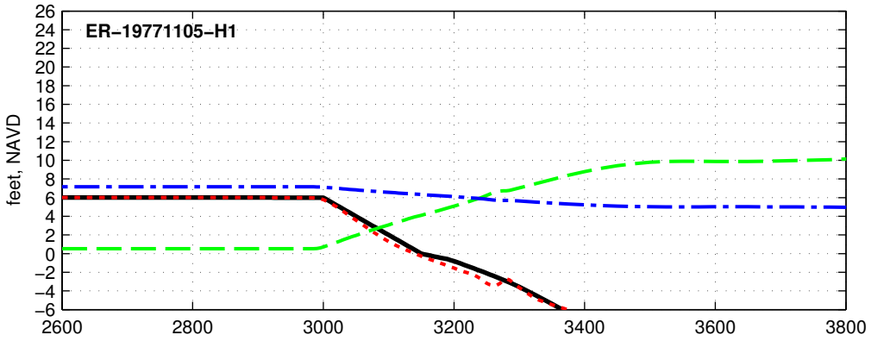


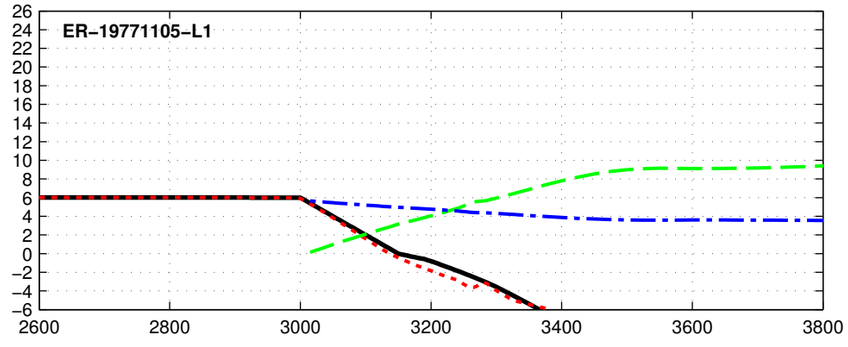


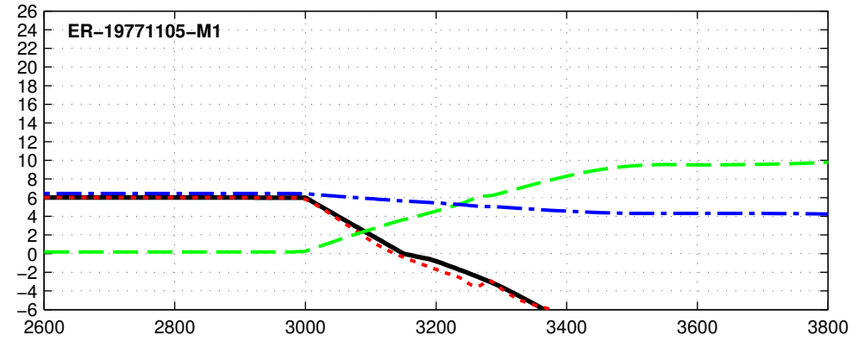


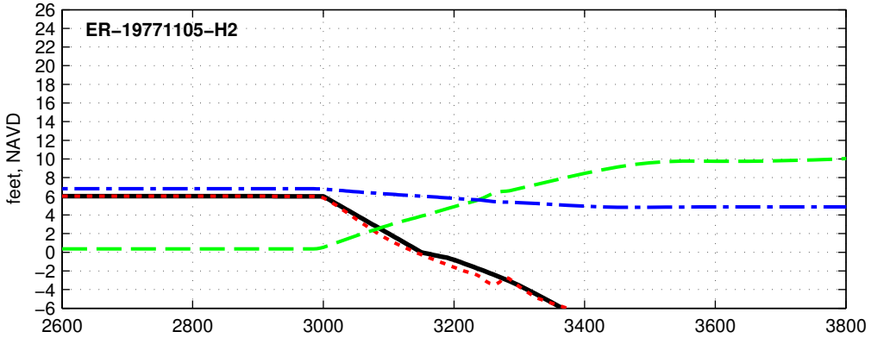
Initial Final Max Hs Max Water Level

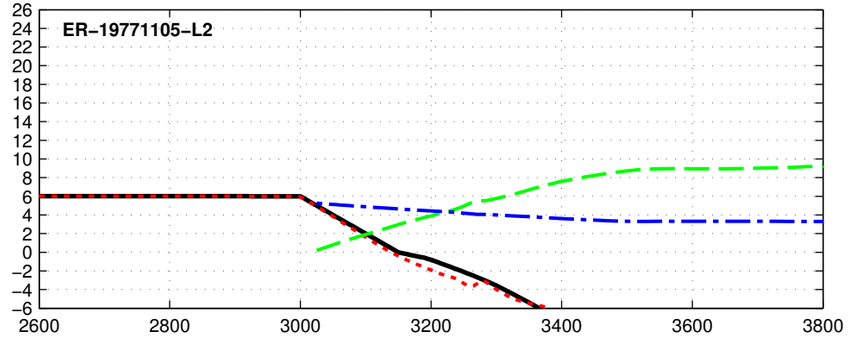
Dune R1T1-6.0-000-000 - ER-19771105

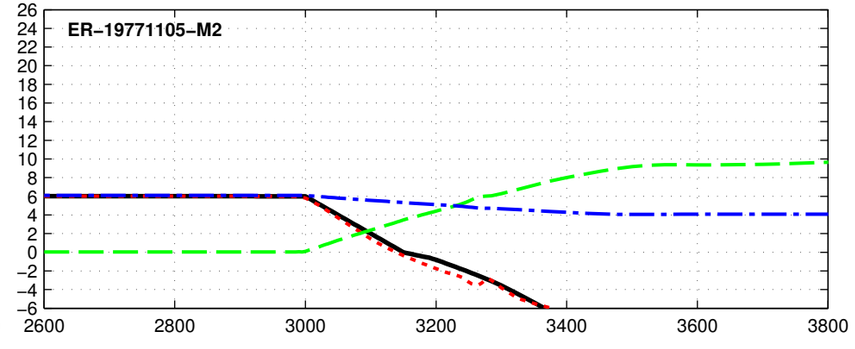


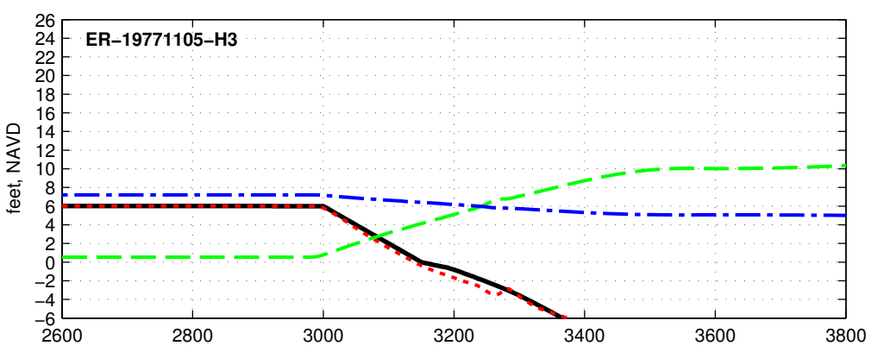


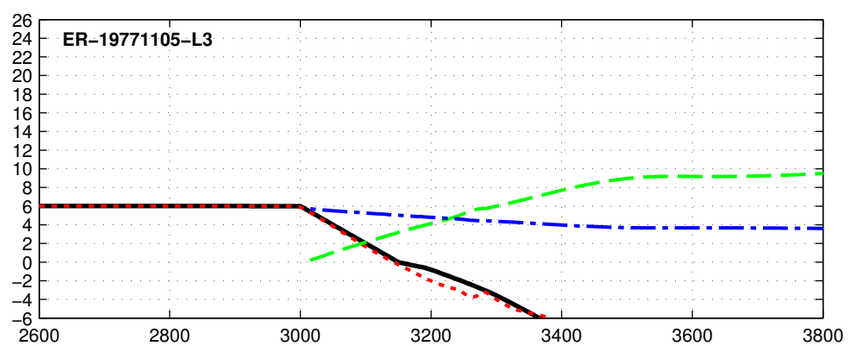


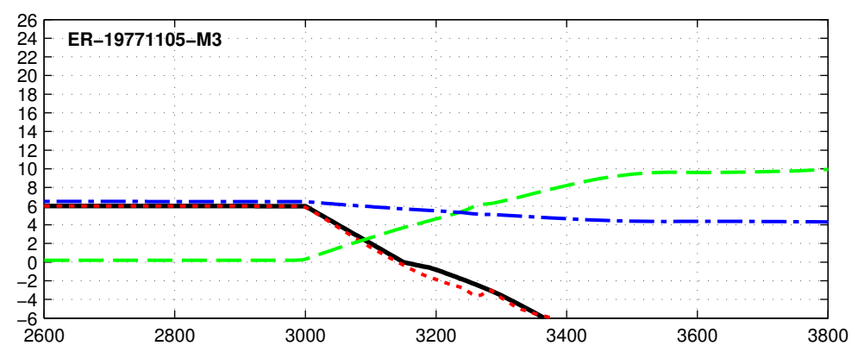


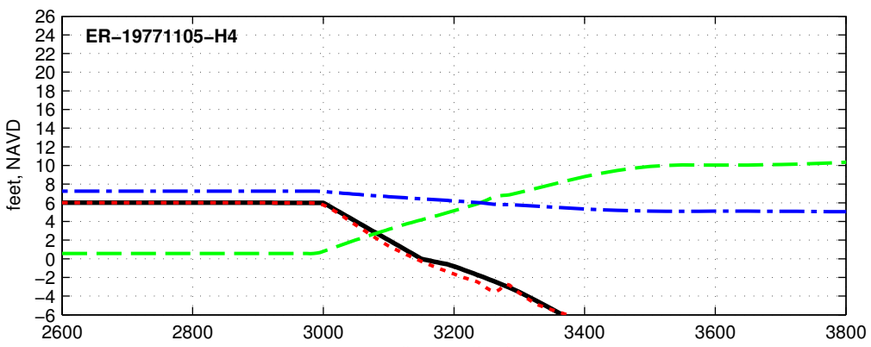


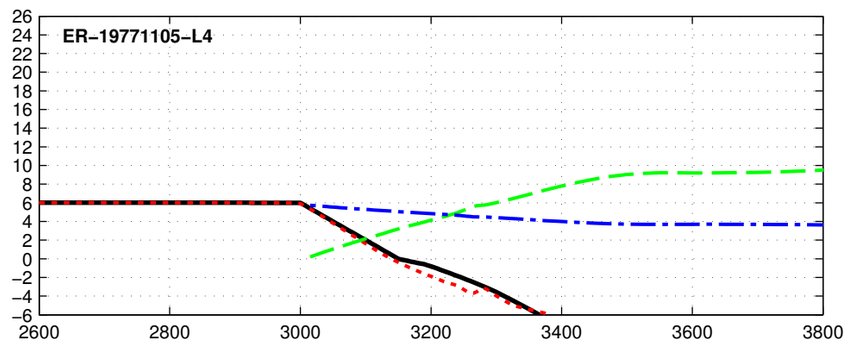


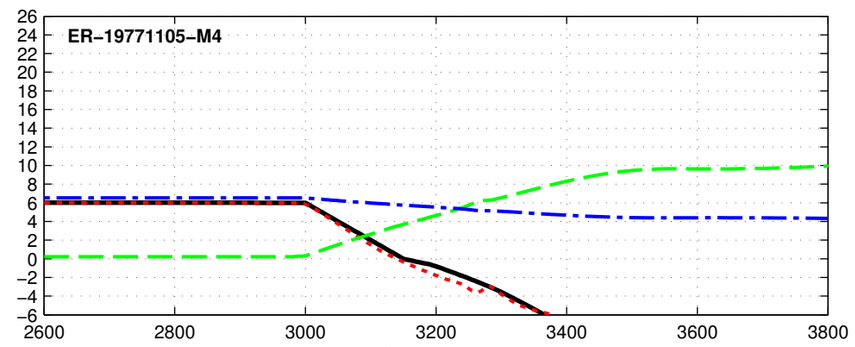






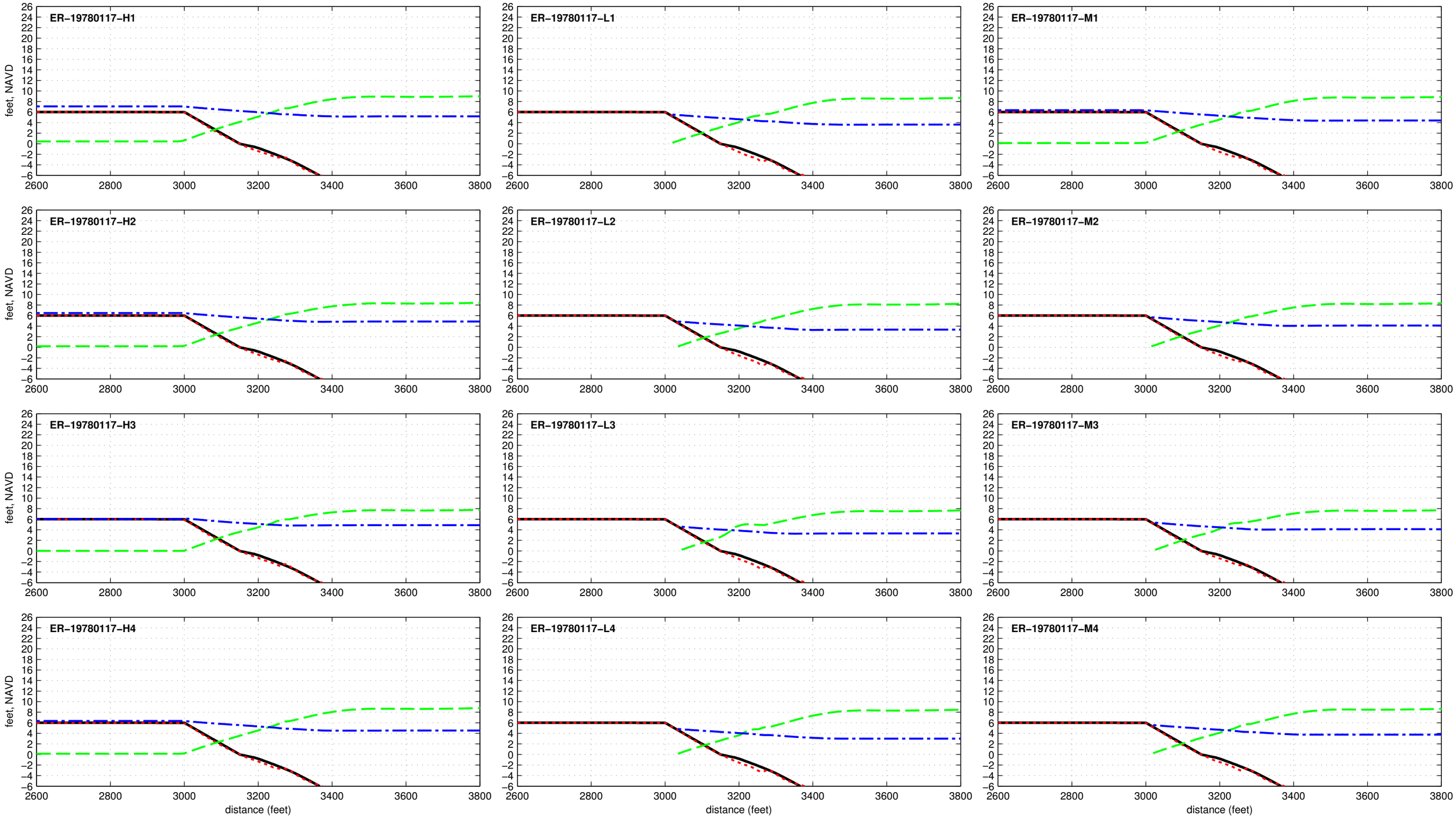




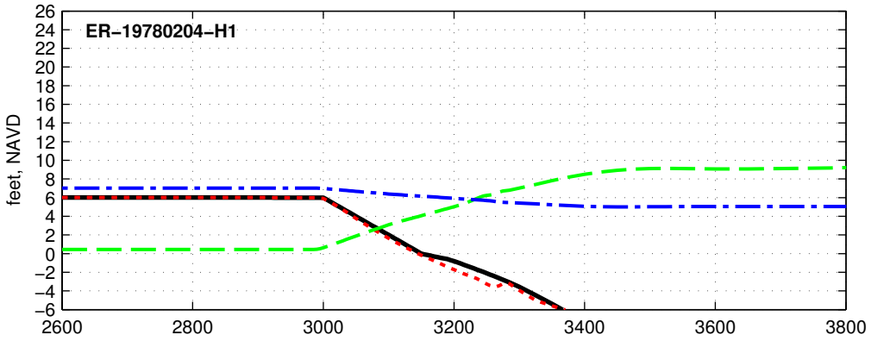


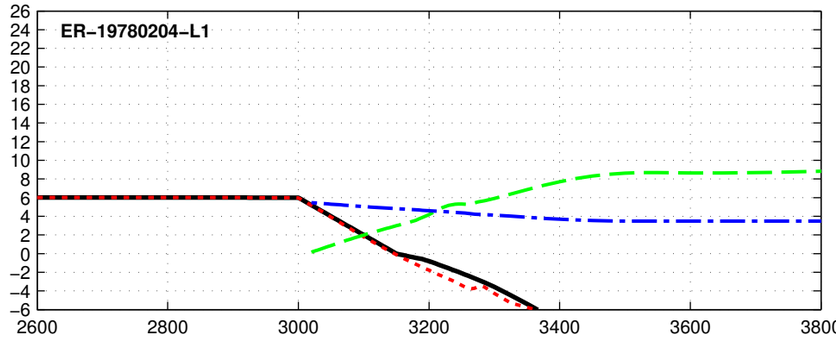
Initial Final Max Hs Max Water Level

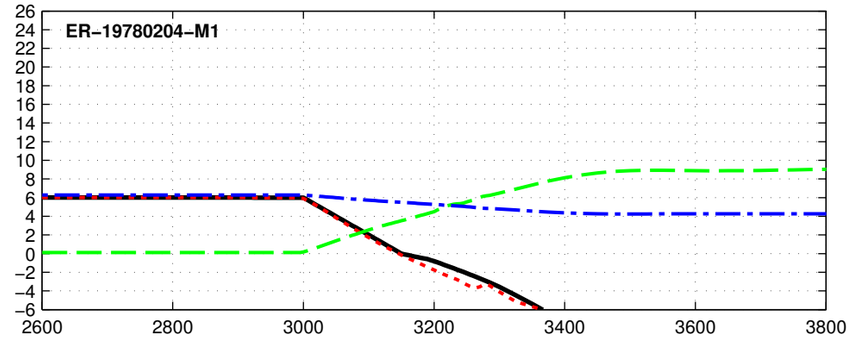
Dune R1T1-6.0-000-000 - ER-19780117

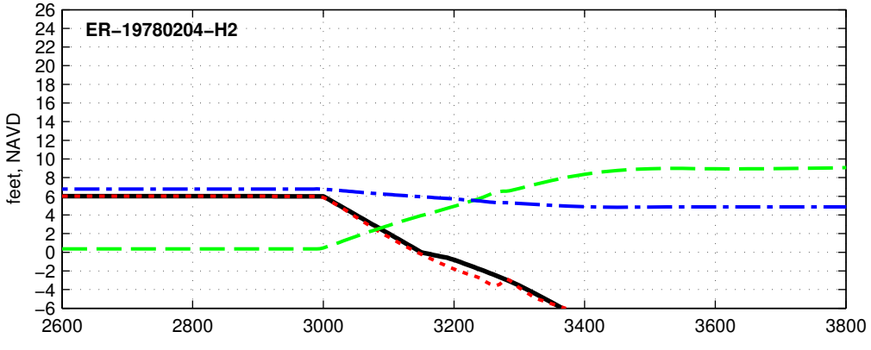


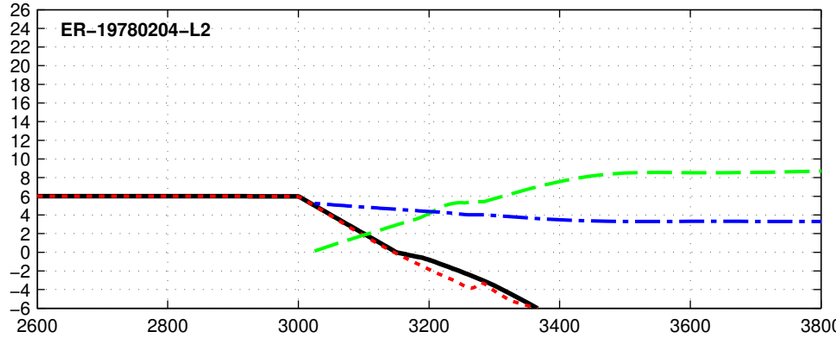
Dune R1T1-6.0-000-000 - ER-19780204

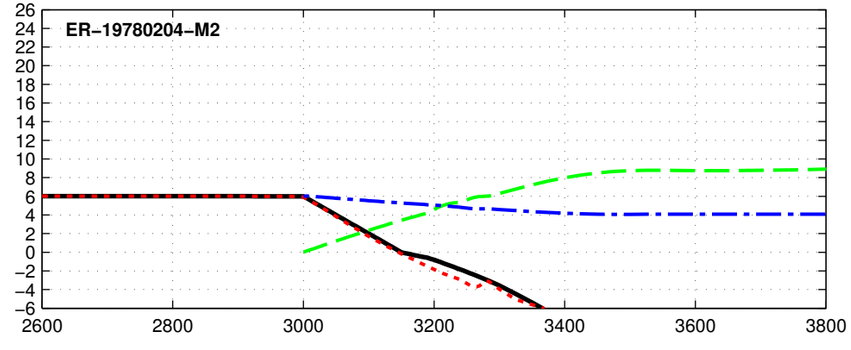


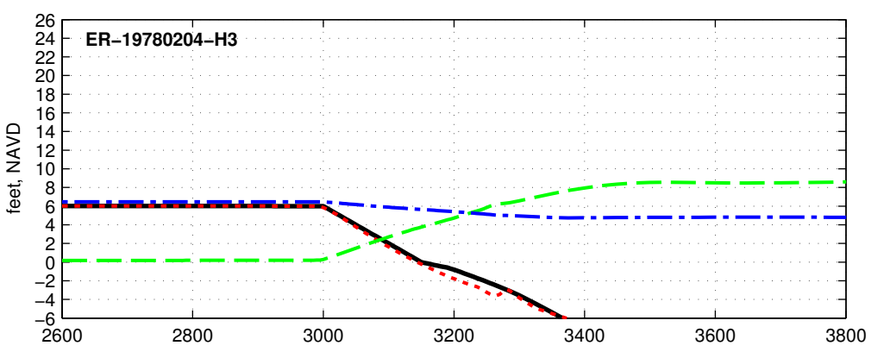


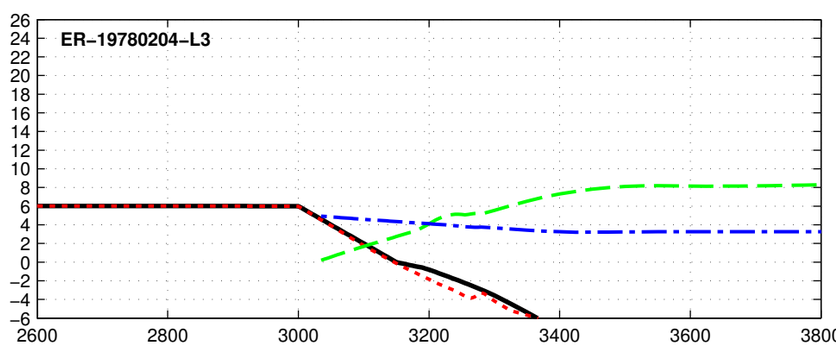


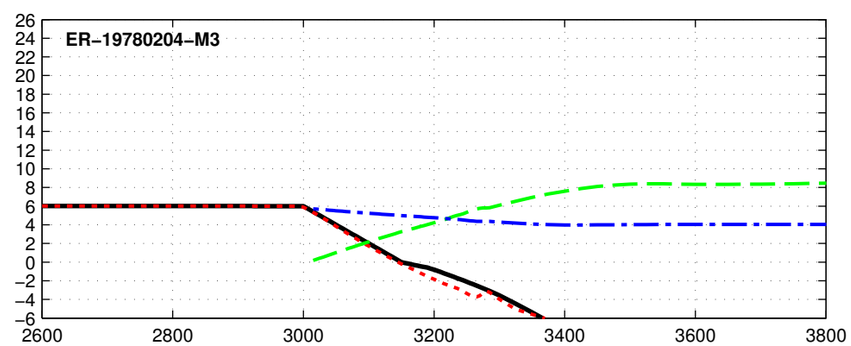


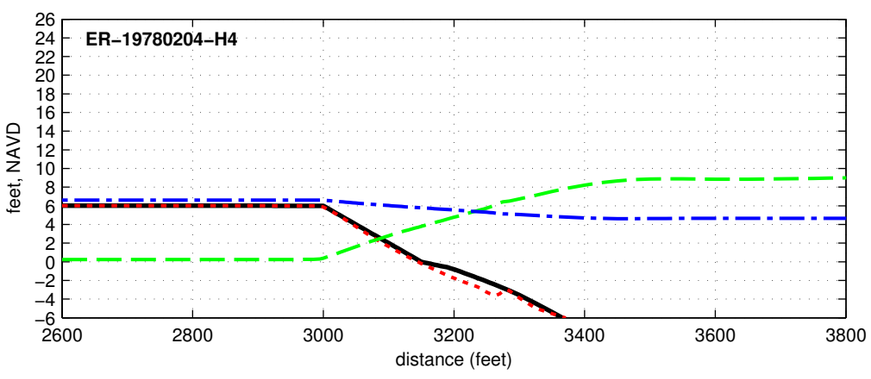


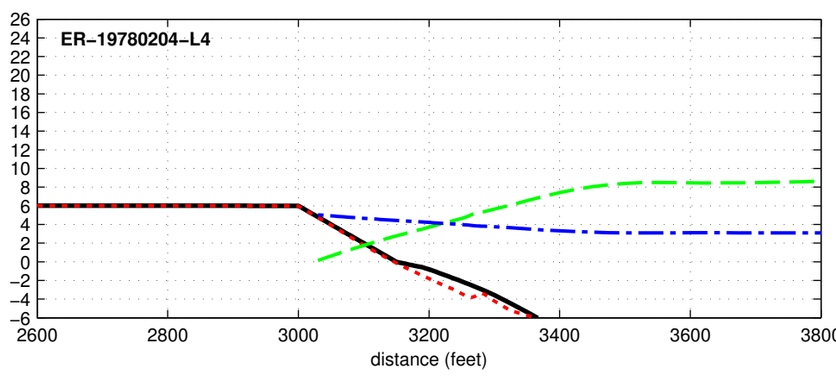


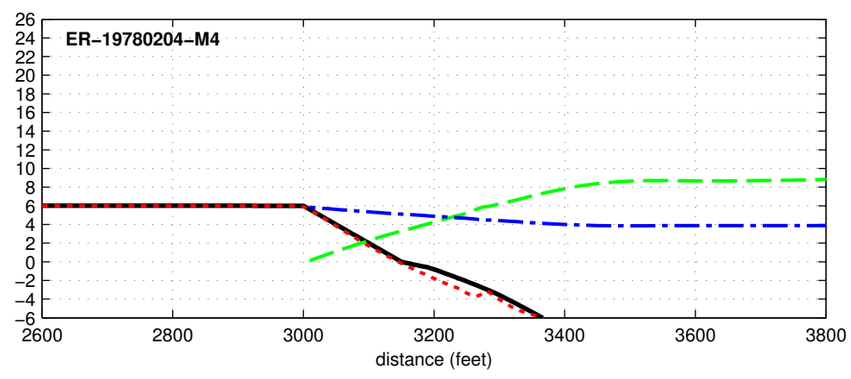






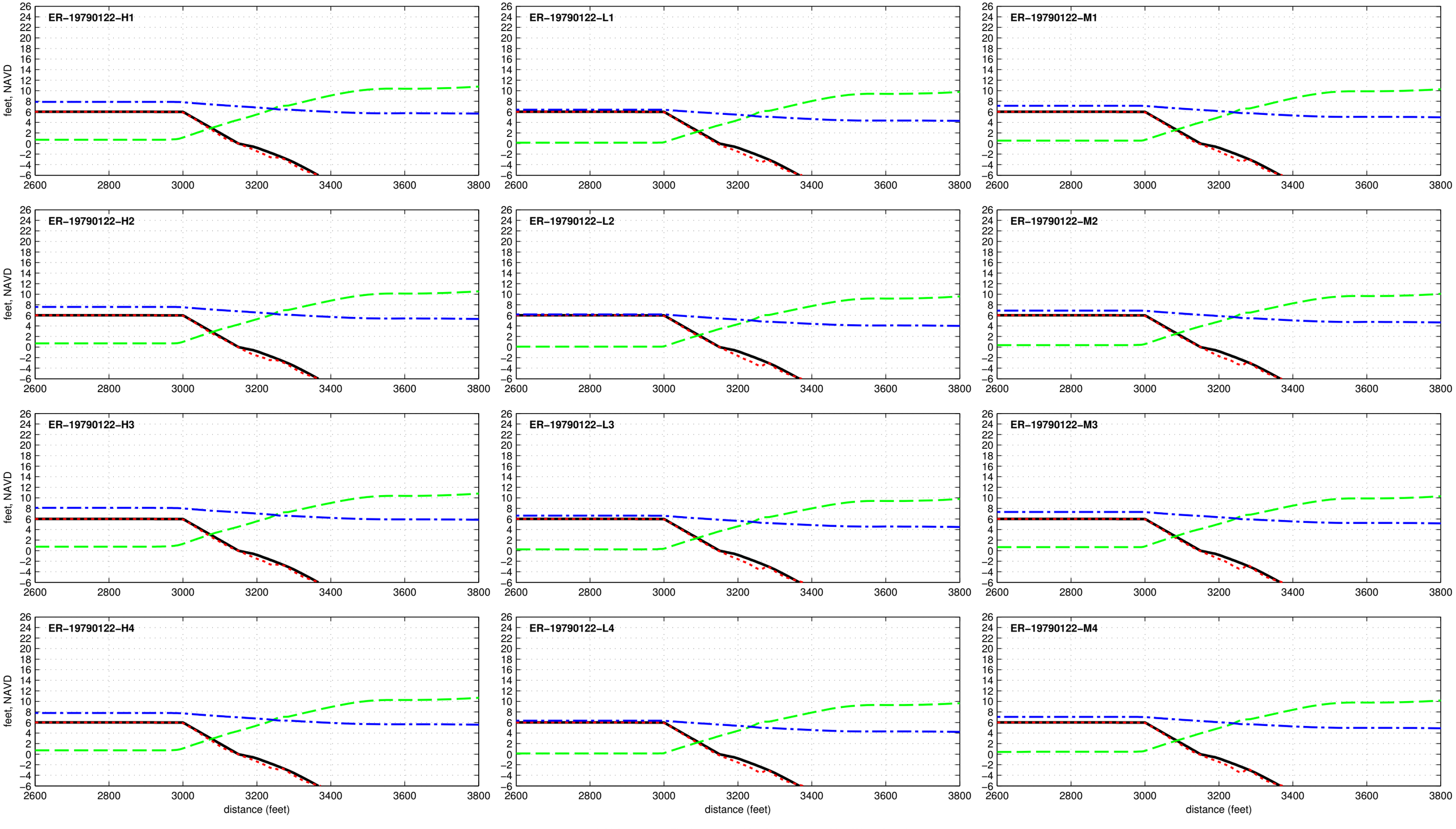




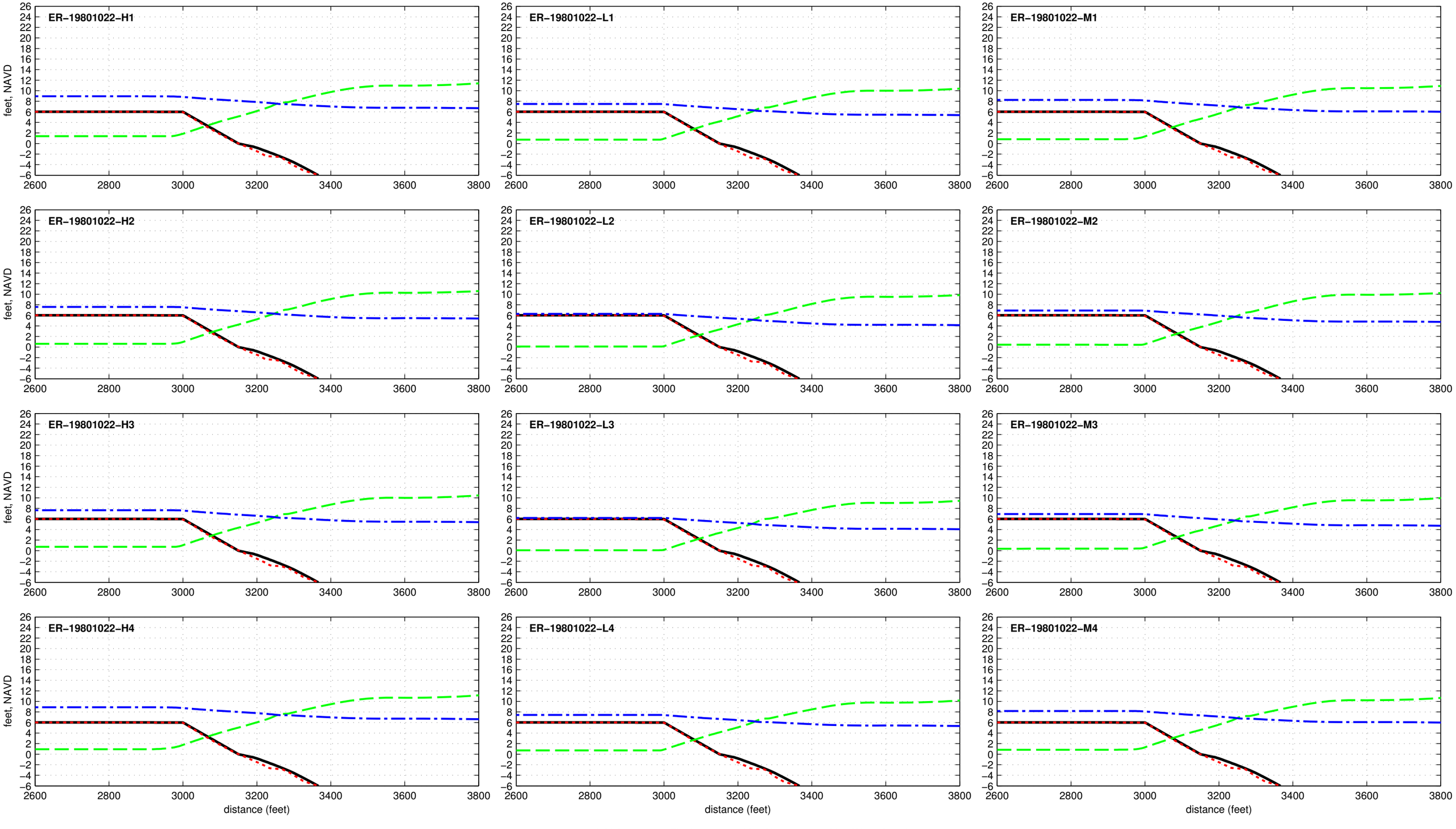


Initial Final Max Hs Max Water Level

Dune R1T1-6.0-000-000 - ER-19790122



Dune R1T1-6.0-000-000 - ER-19801022



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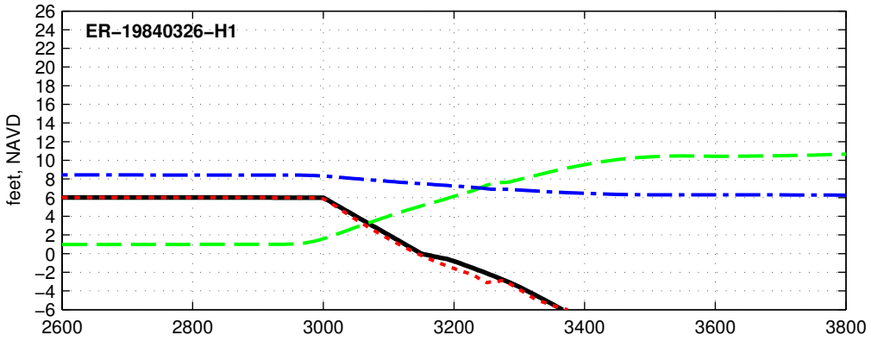
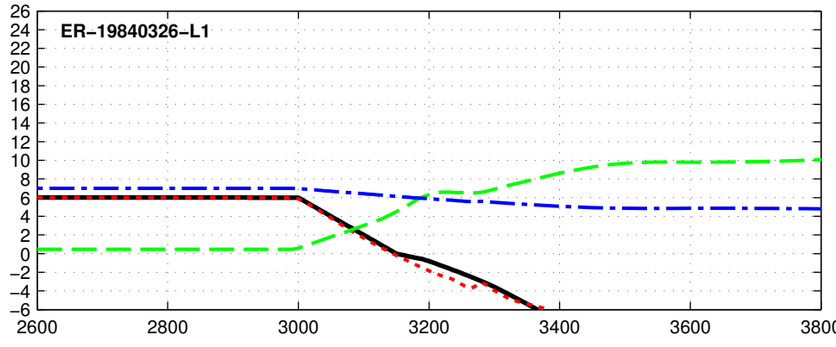
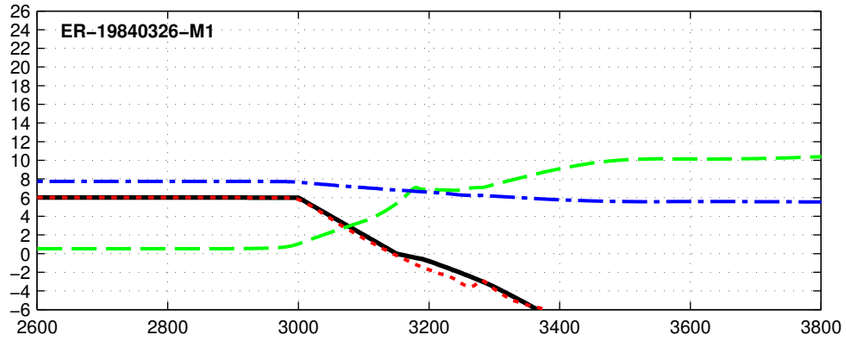
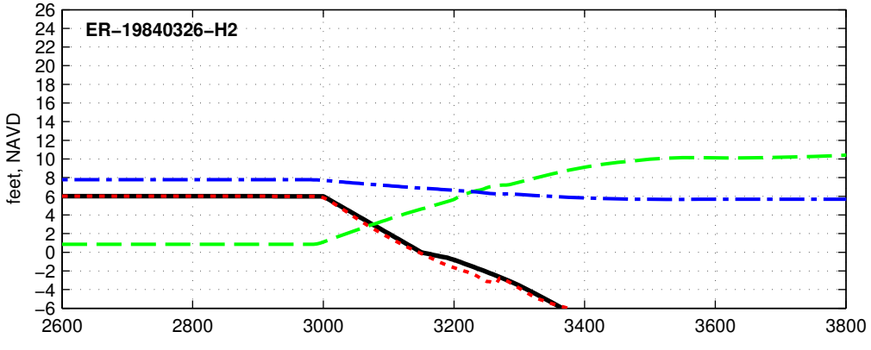
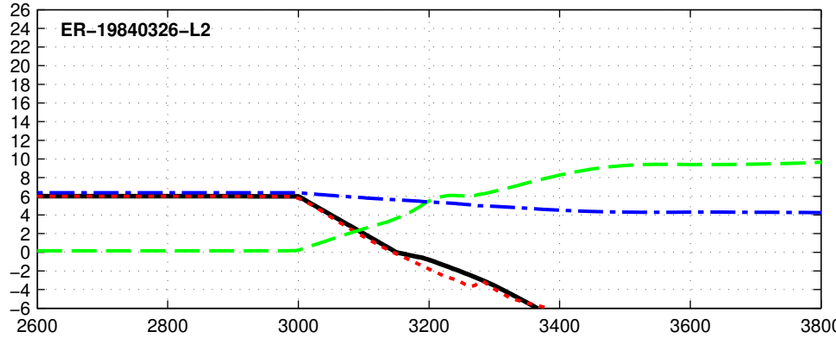
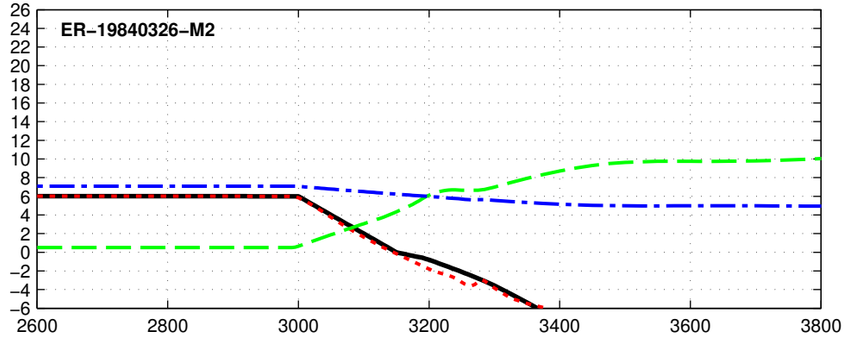


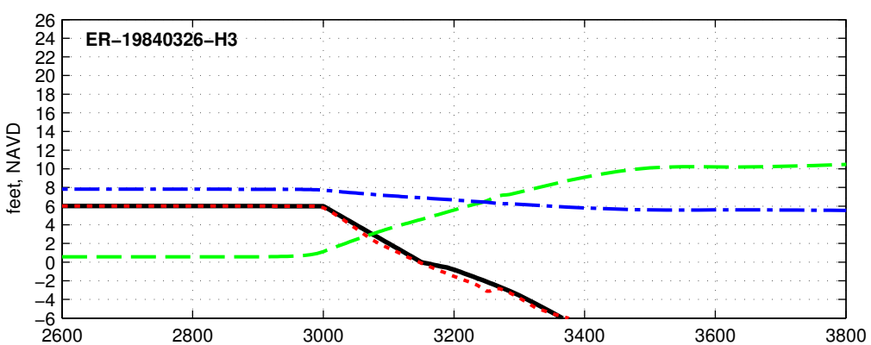


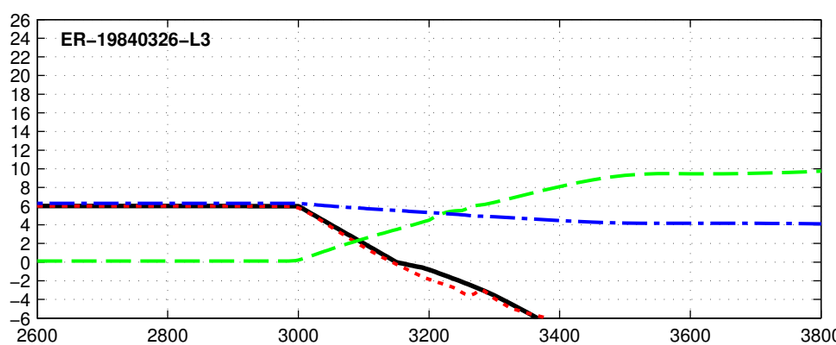


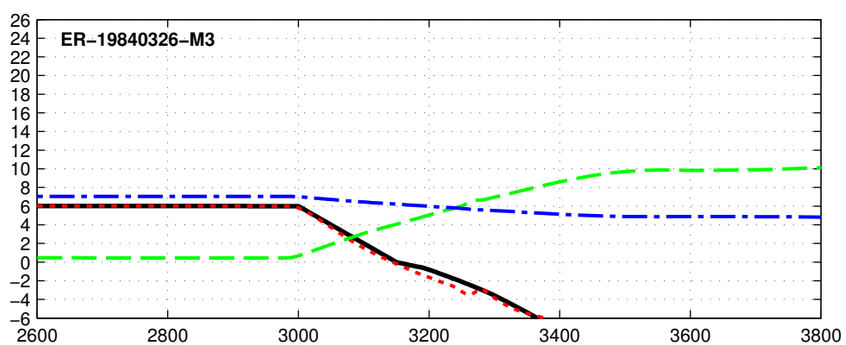


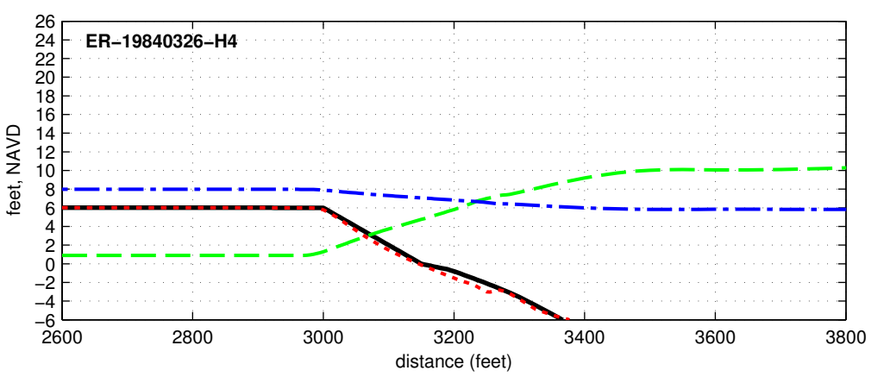


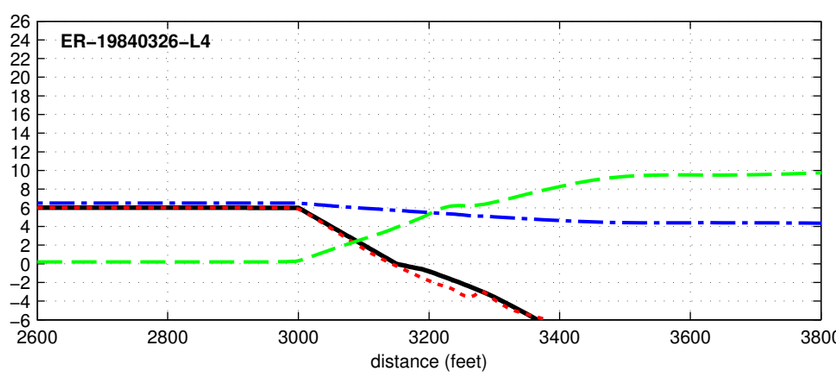


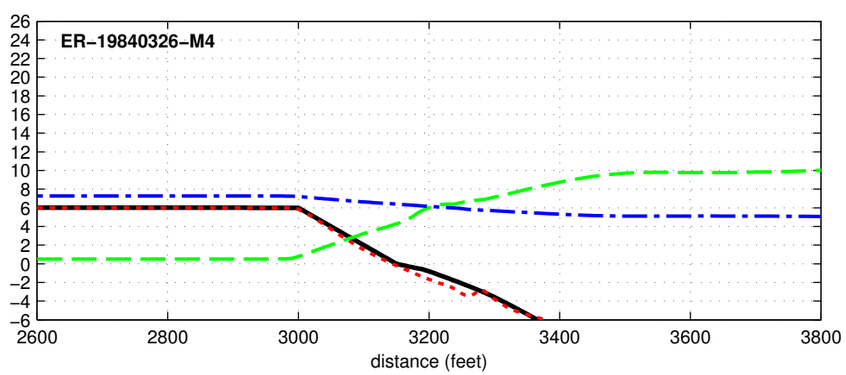






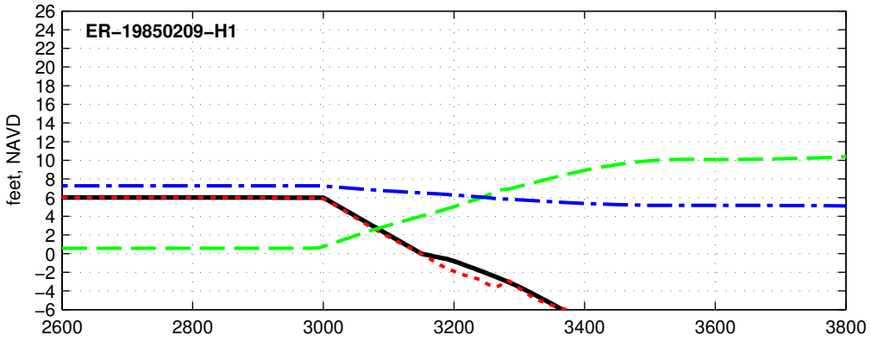


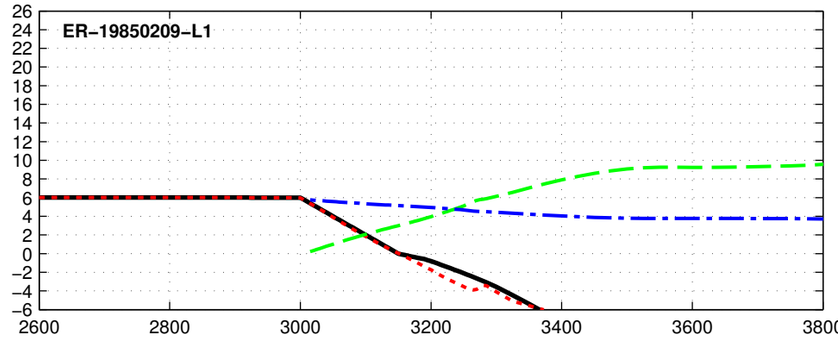


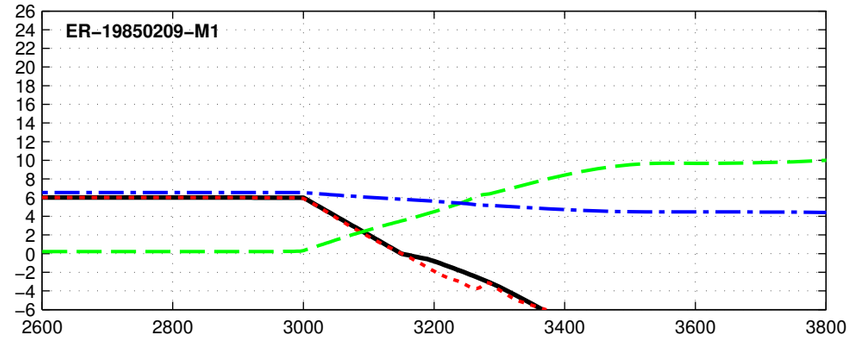


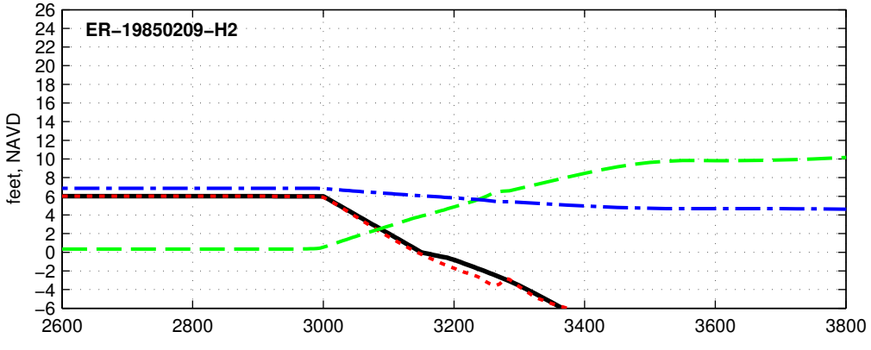
Initial Final Max Hs Max Water Level

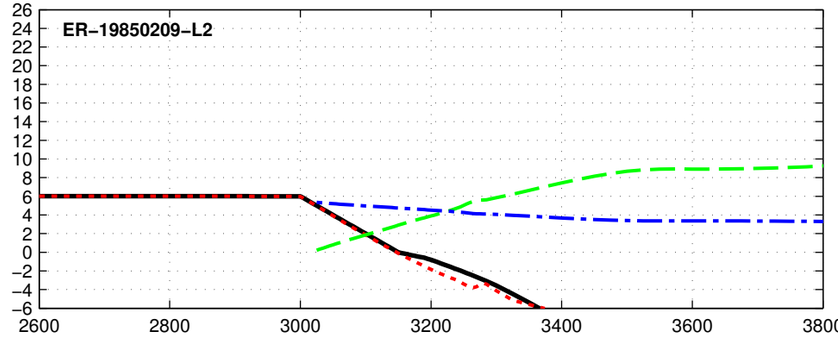
Dune R1T1-6.0-000-000 - ER-19850209

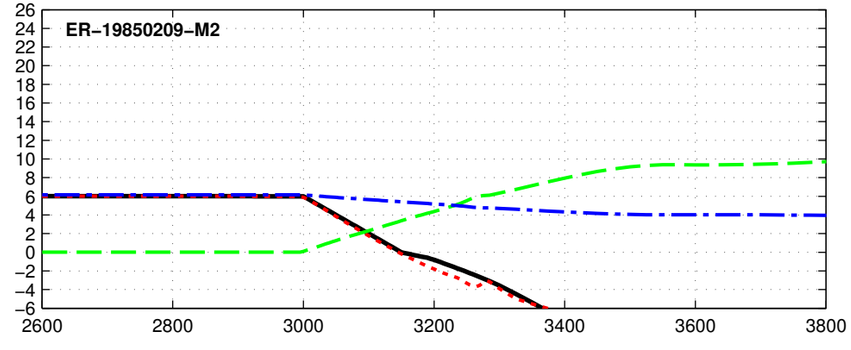


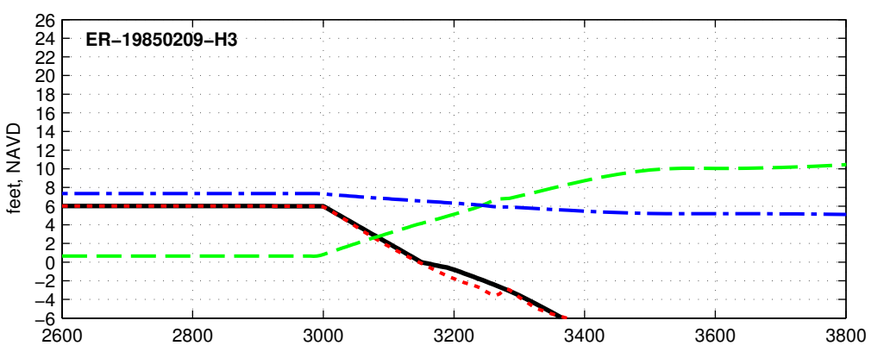


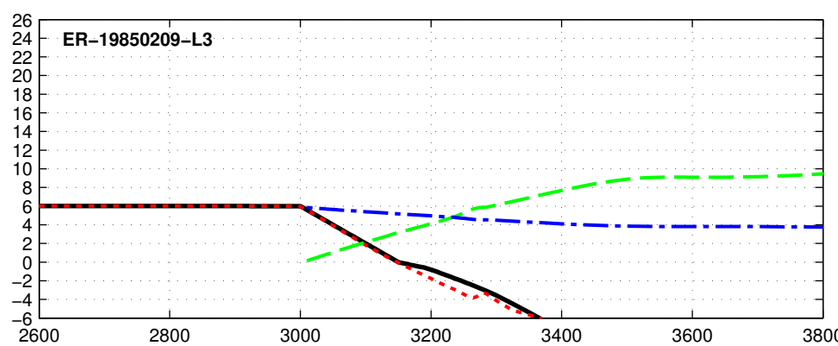


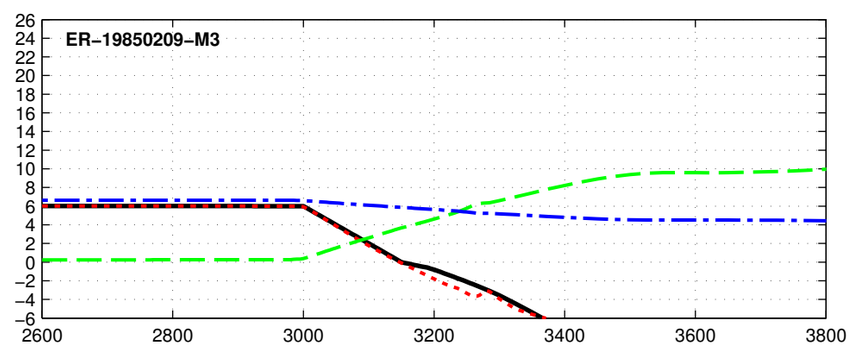


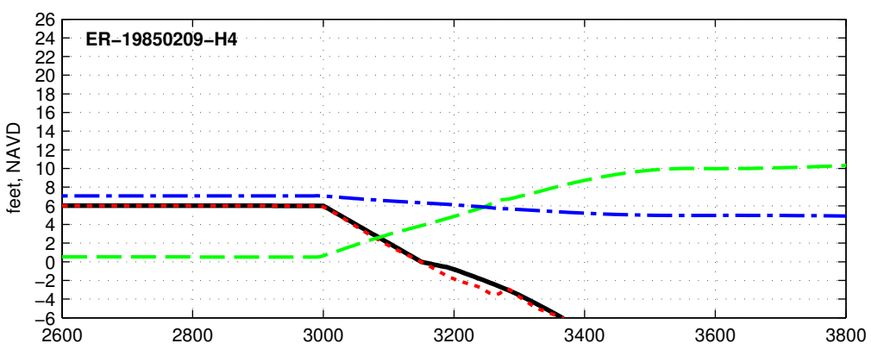


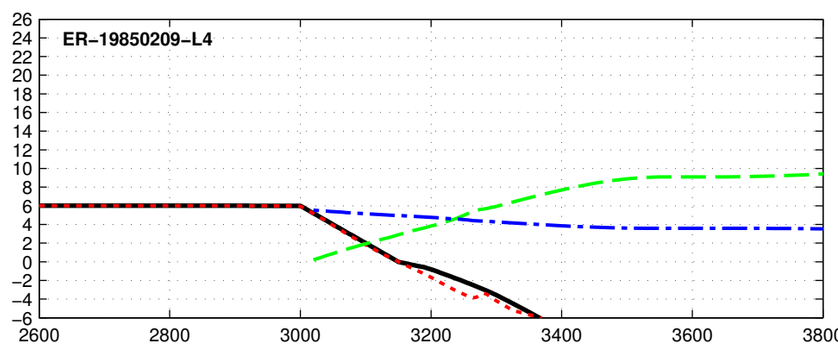


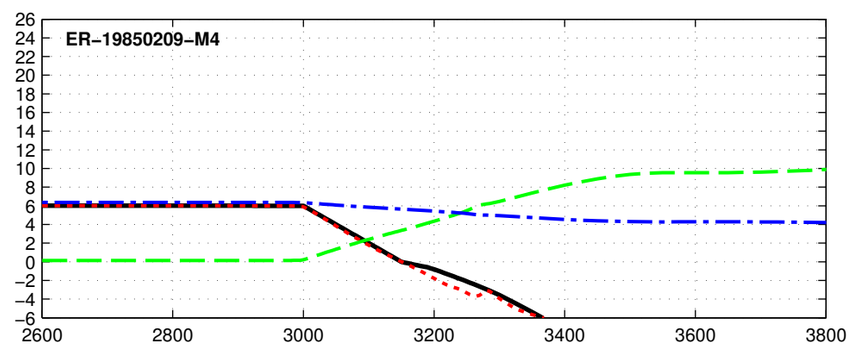






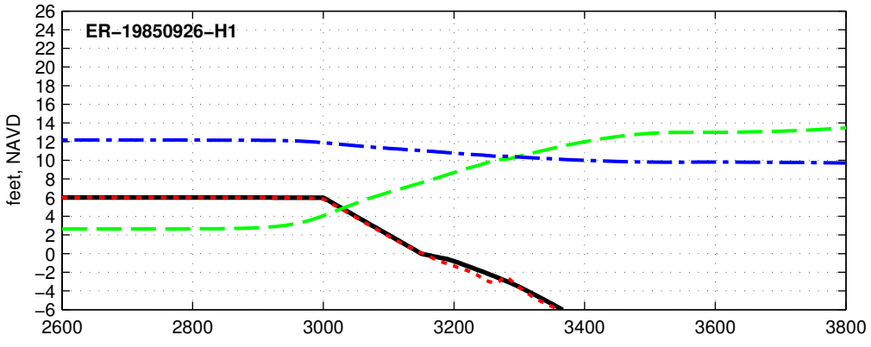


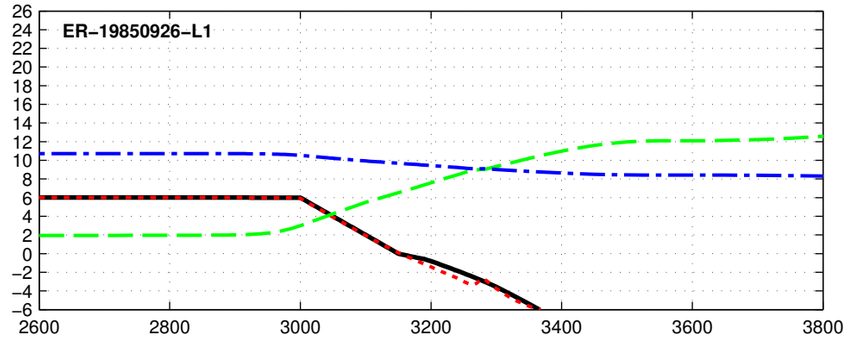


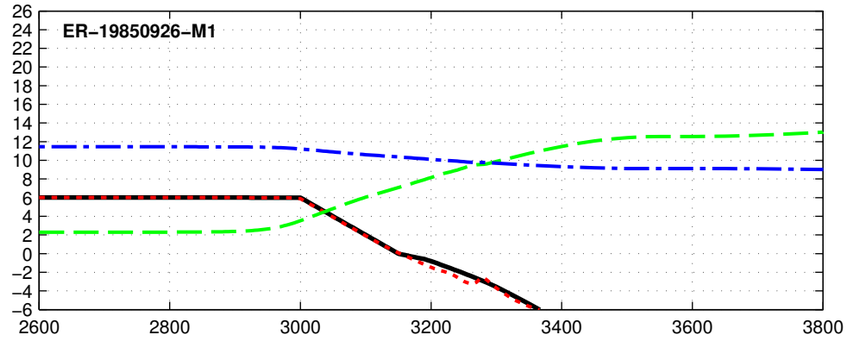


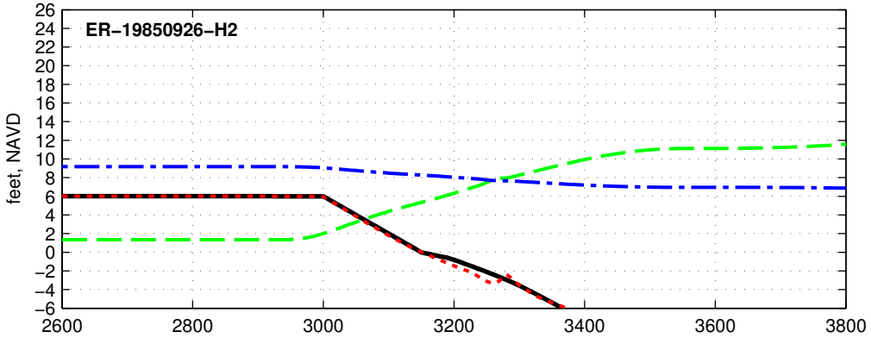
Initial Final Max Hs Max Water Level

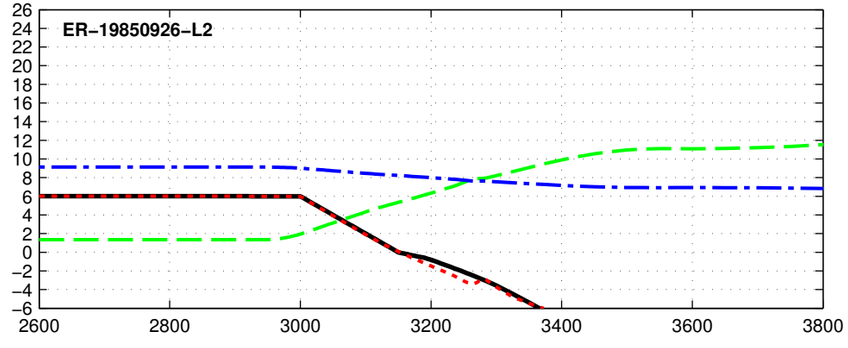
Dune R1T1-6.0-000-000 - ER-19850926

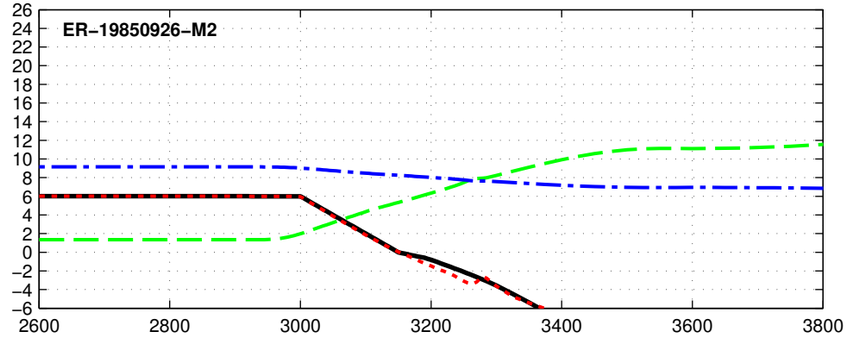


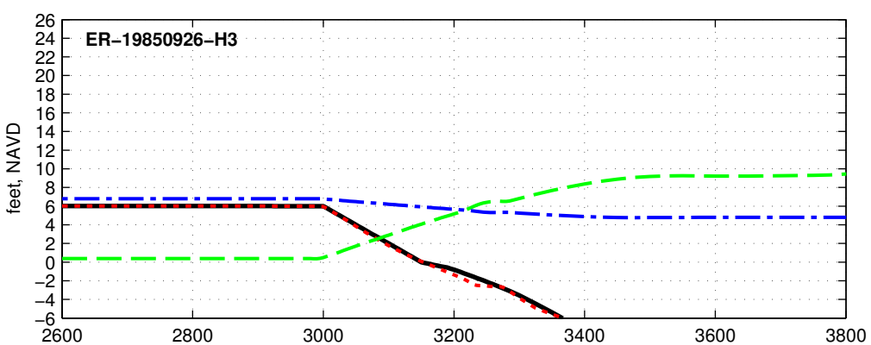


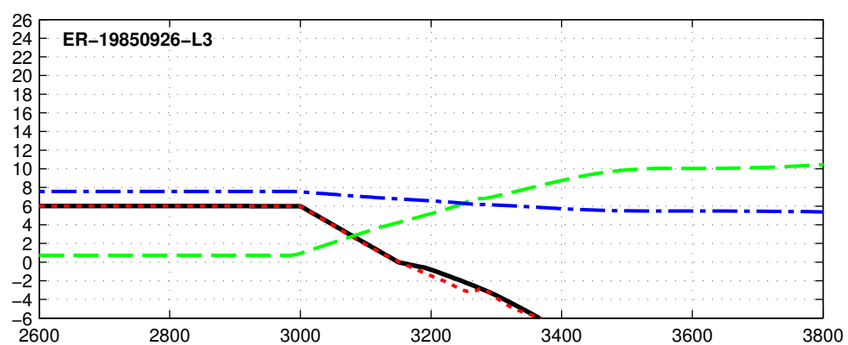


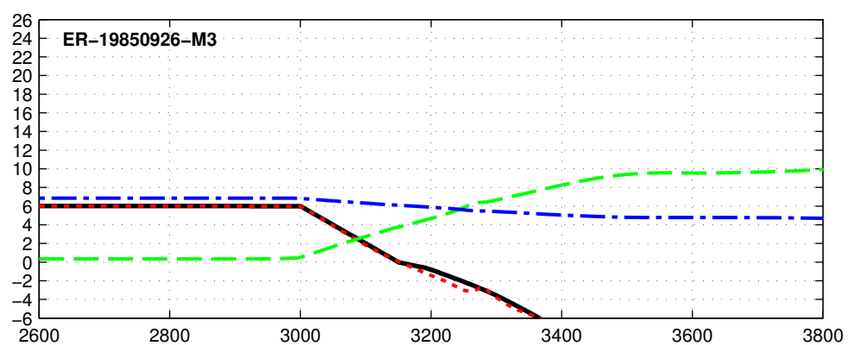


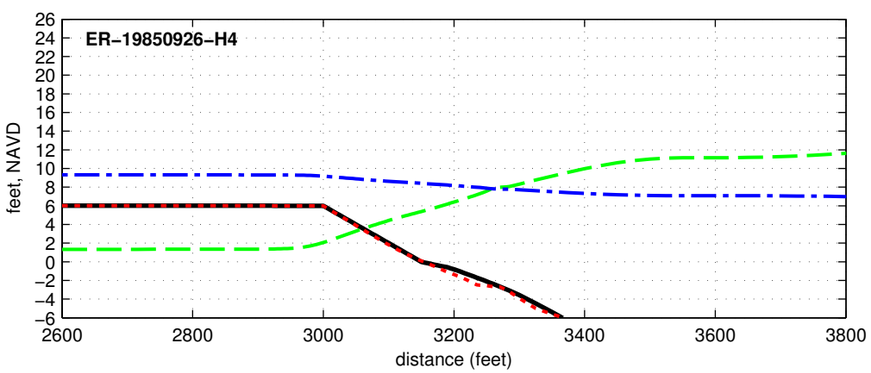


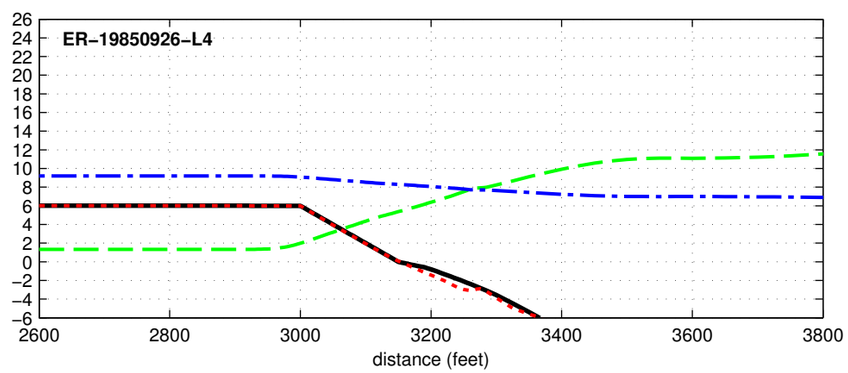


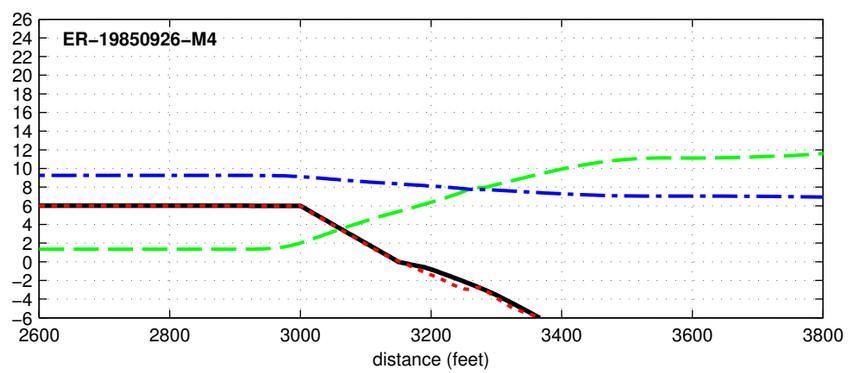






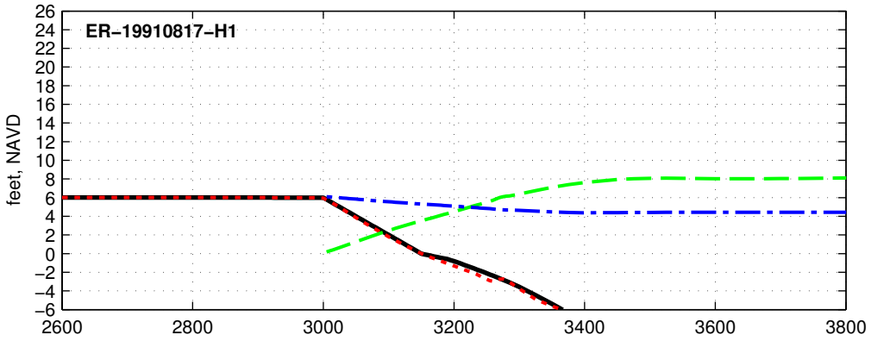


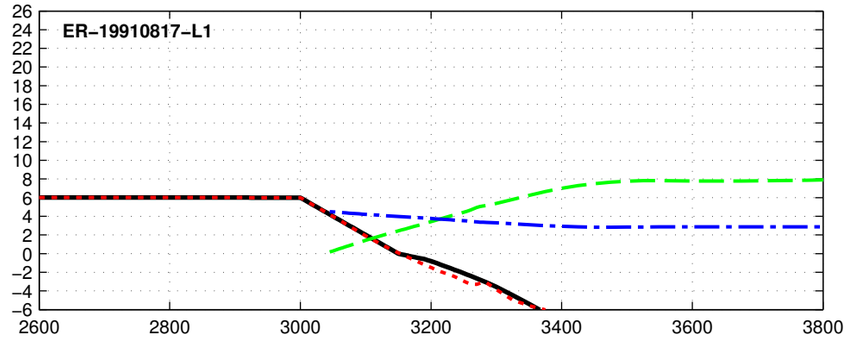


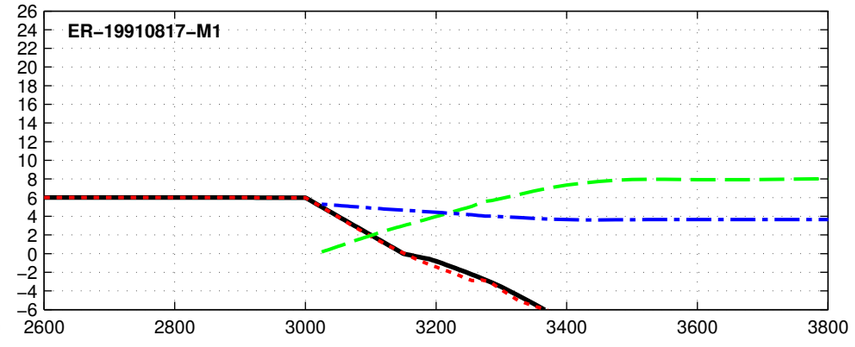


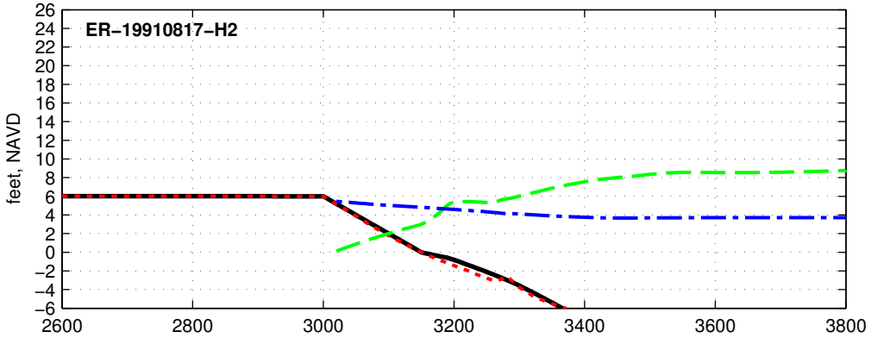
Initial Final Max Hs Max Water Level

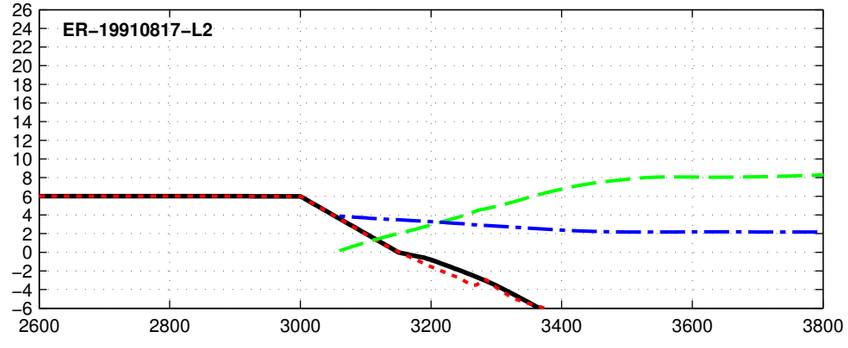
Dune R1T1-6.0-000-000 - ER-19910817

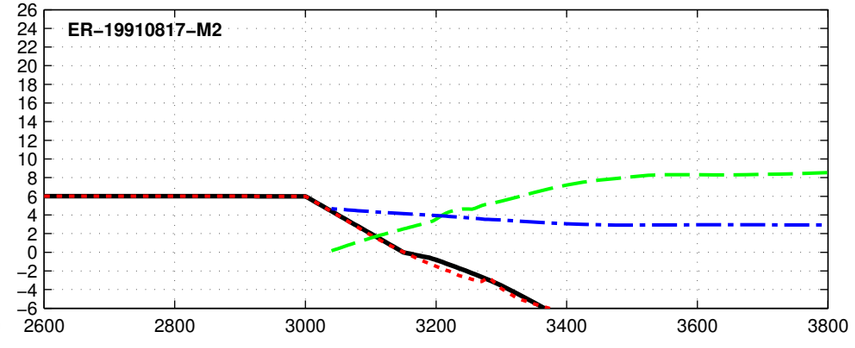


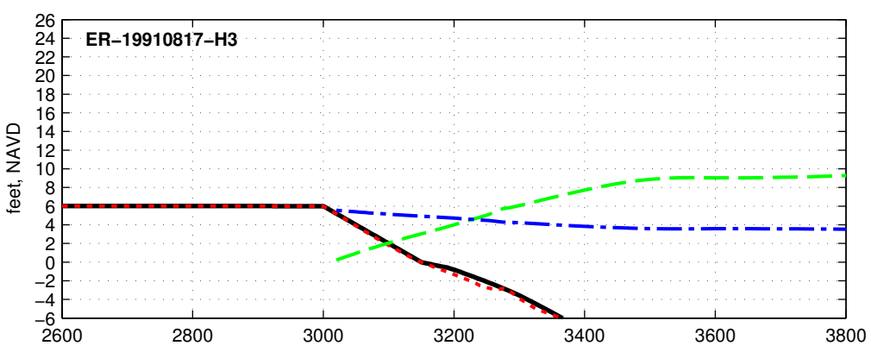


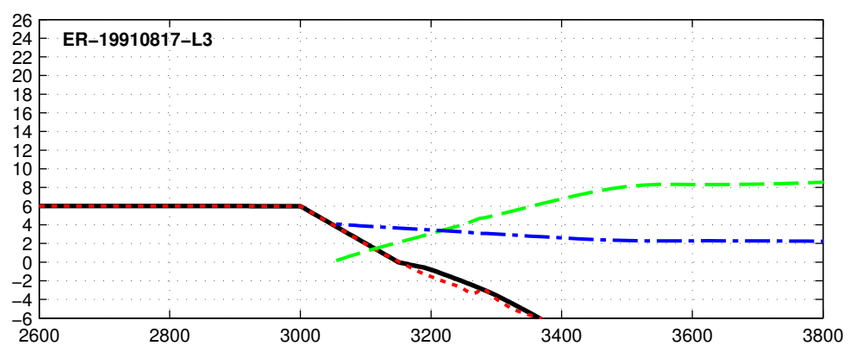


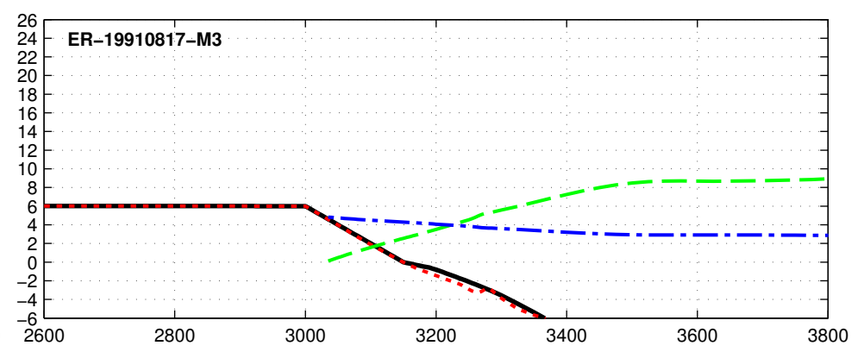


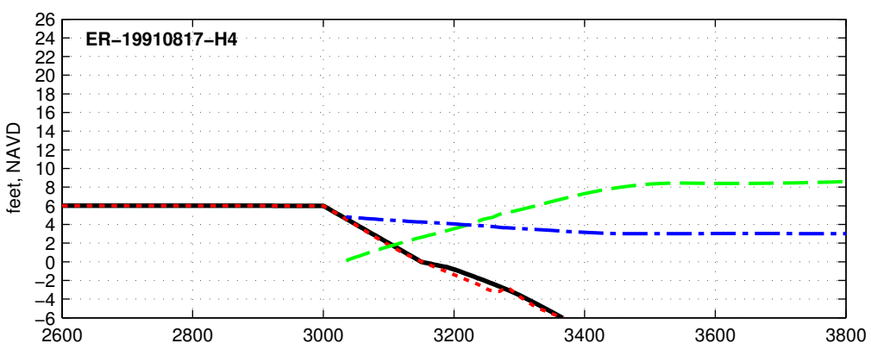


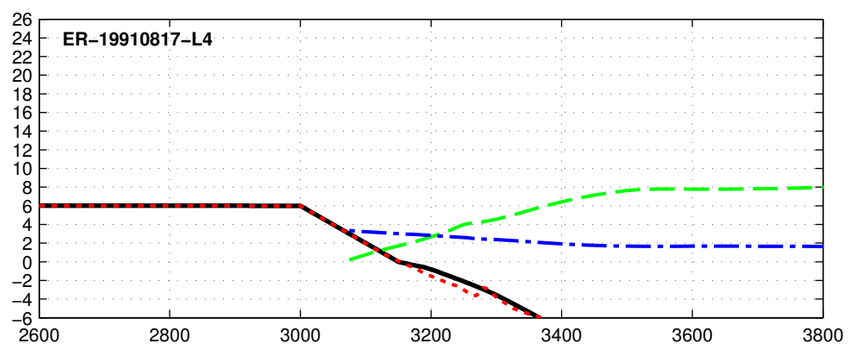


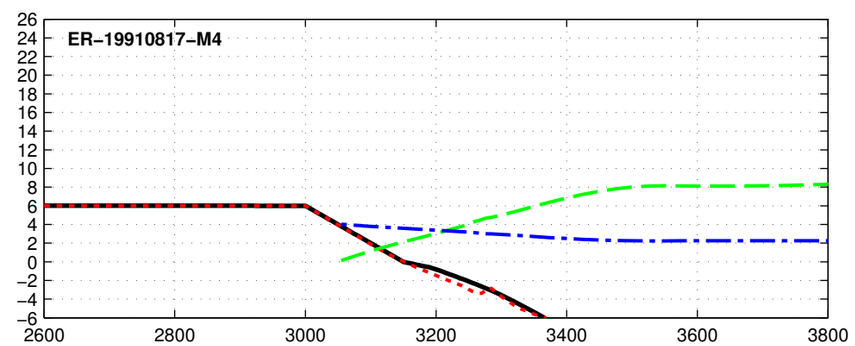






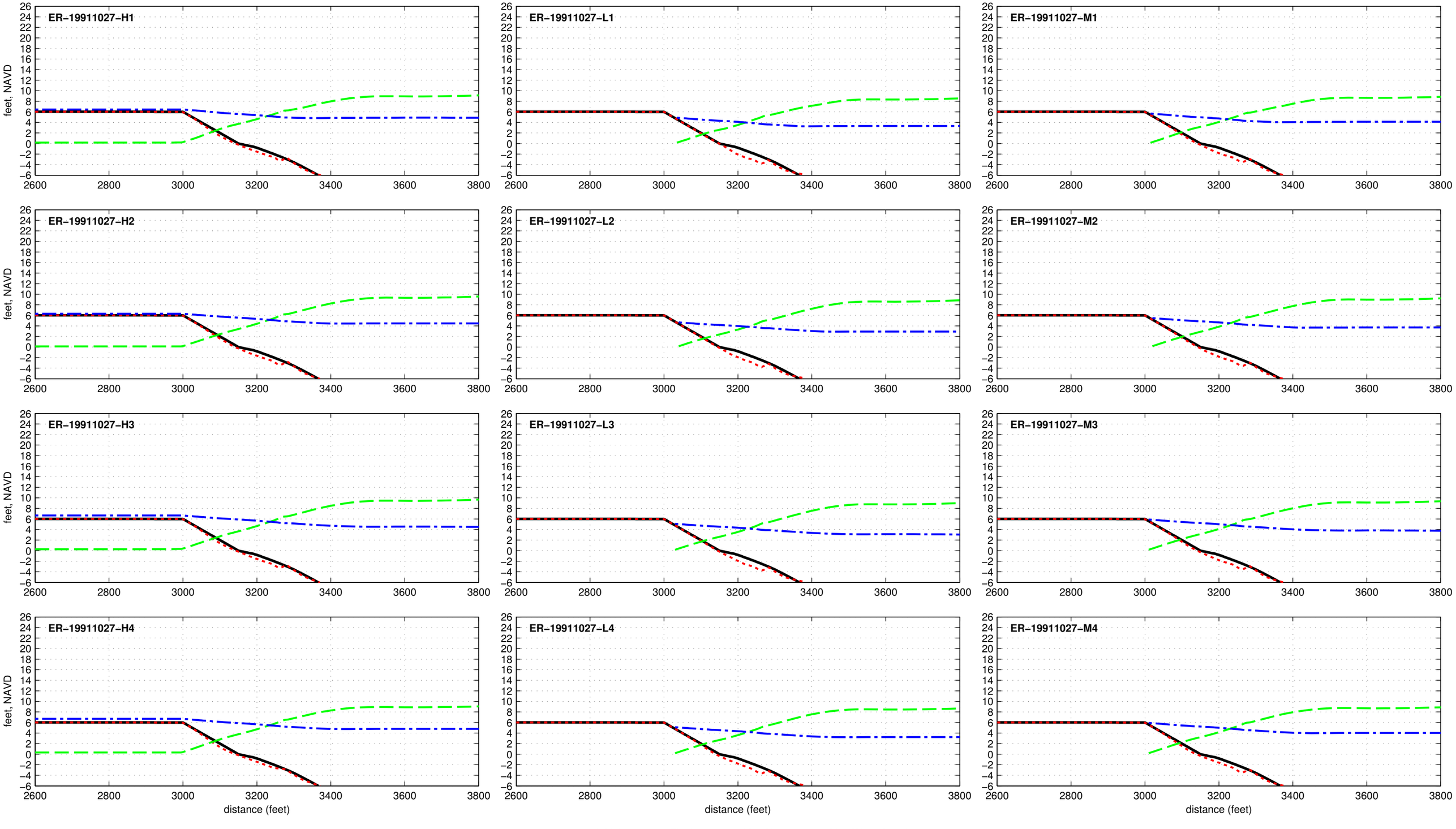




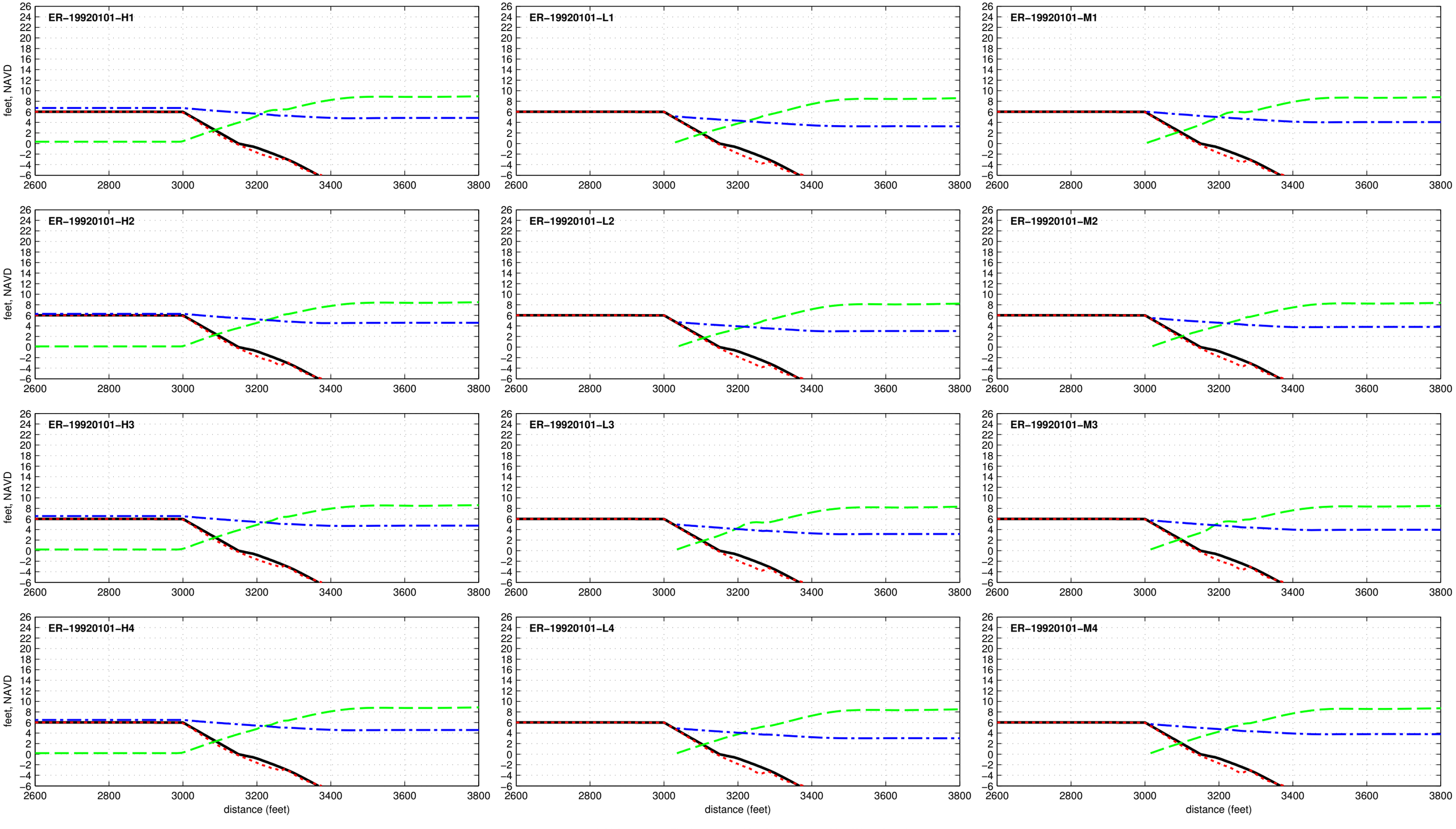


Initial Final Max Hs Max Water Level

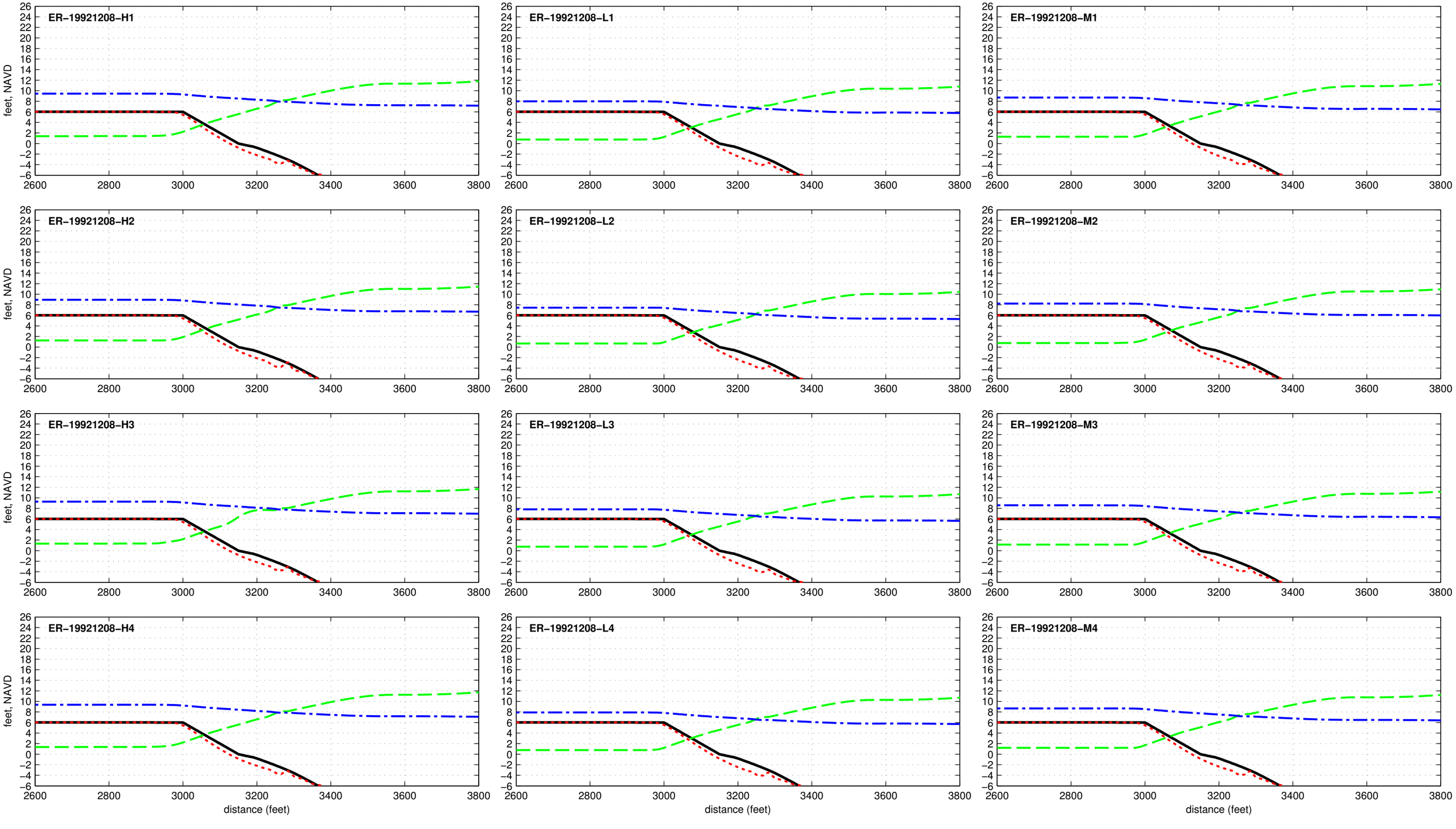
Dune R1T1-6.0-000-000 - ER-19911027



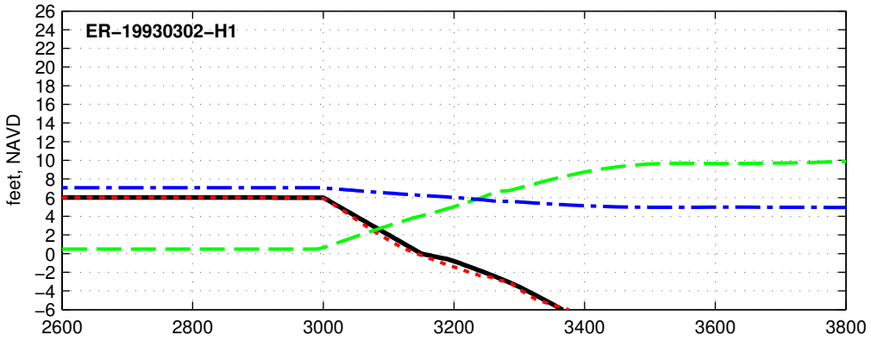
Dune R1T1-6.0-000-000 - ER-19920101

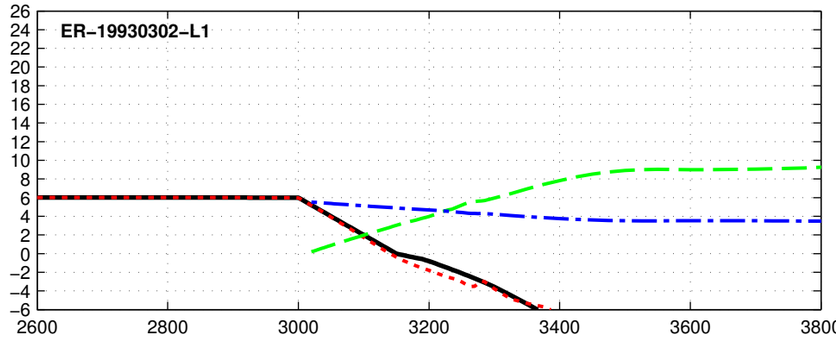


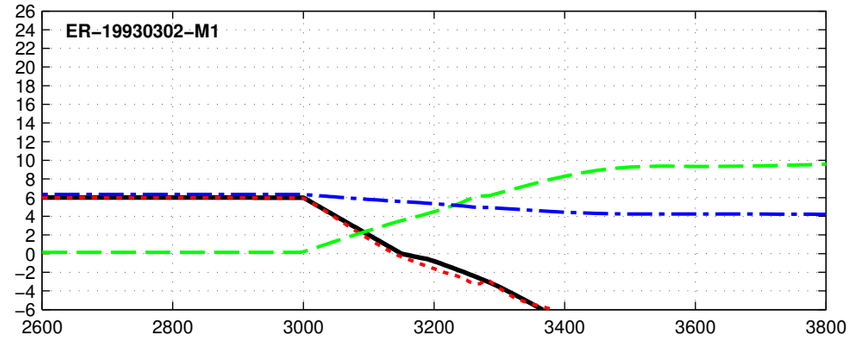
Dune R1T1-6.0-000-000 - ER-19921208

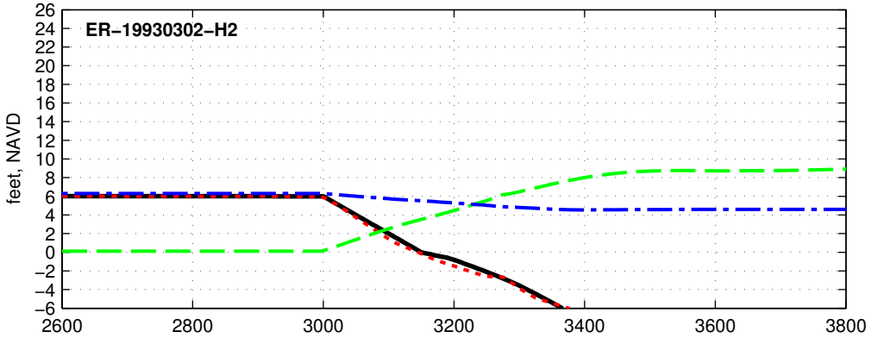


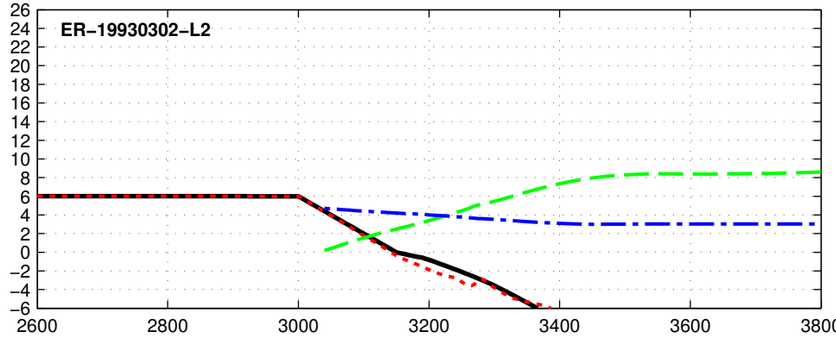
Dune R1T1-6.0-000-000 - ER-19930302

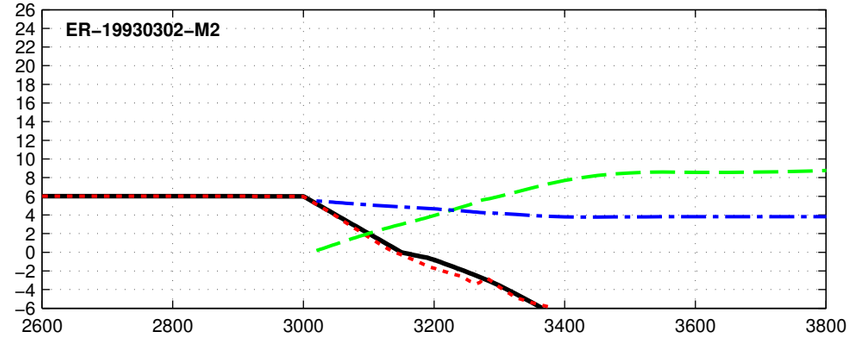


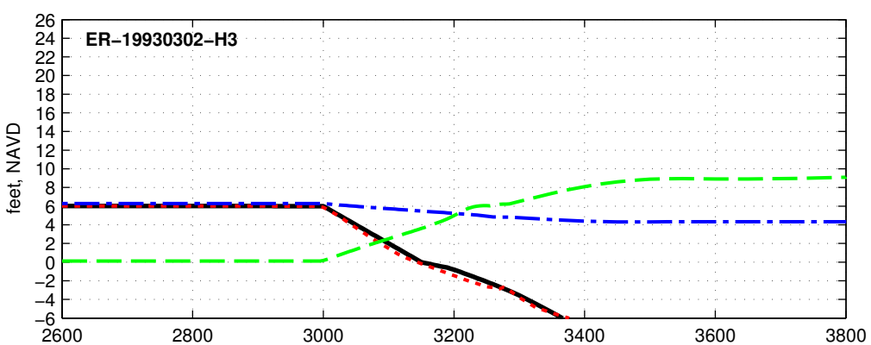


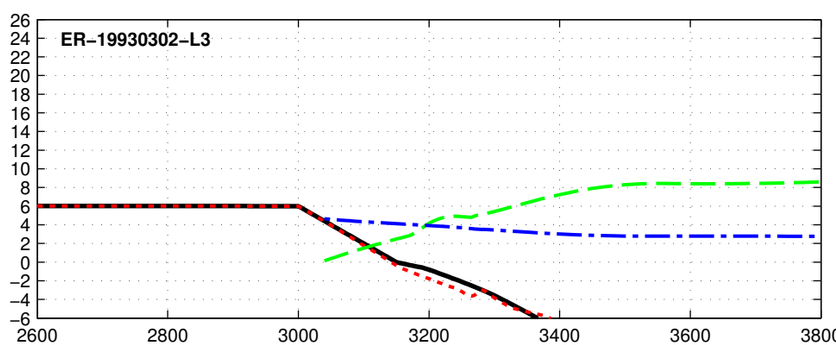


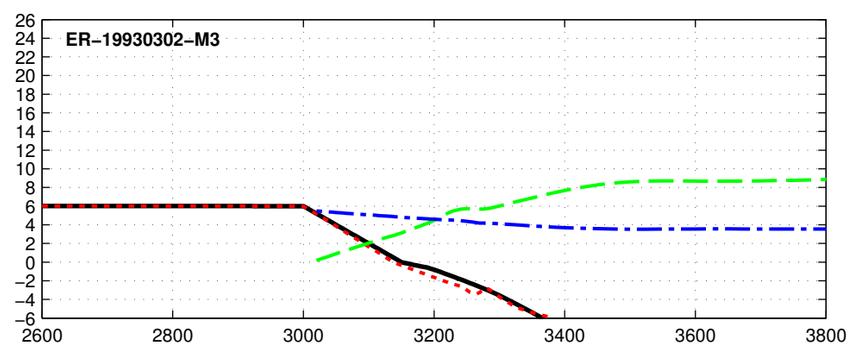


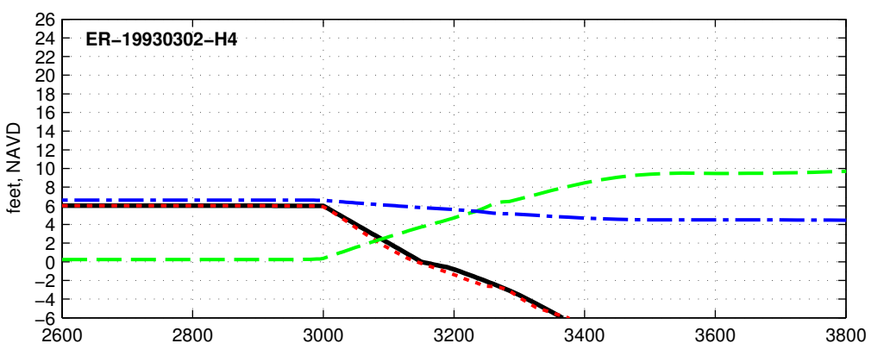


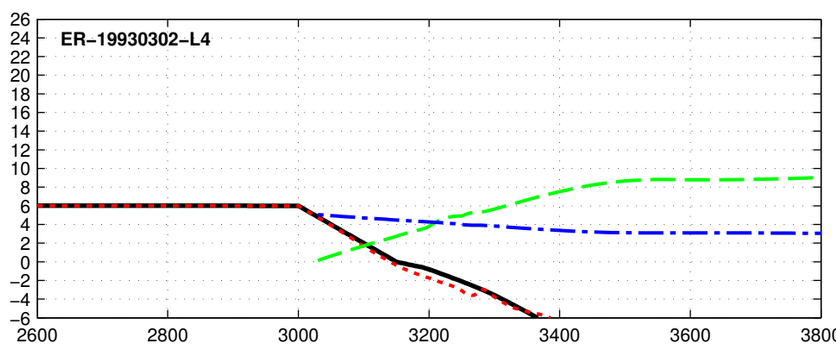


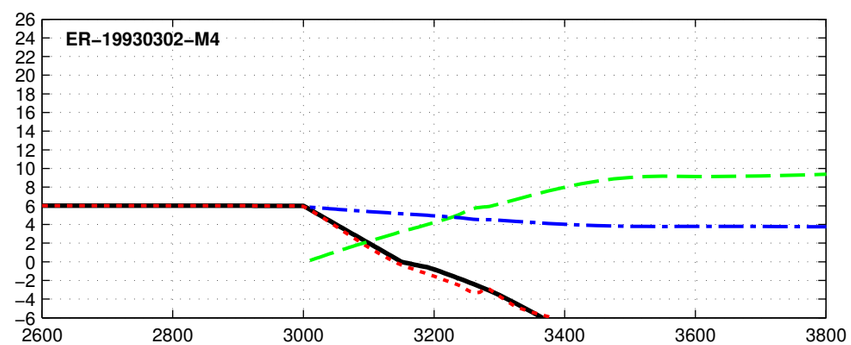










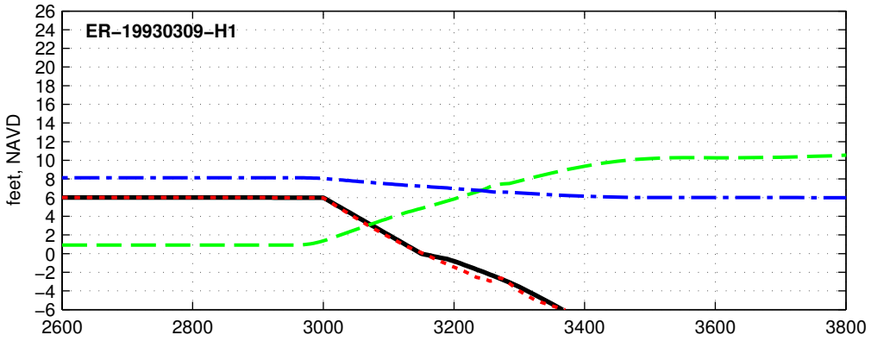


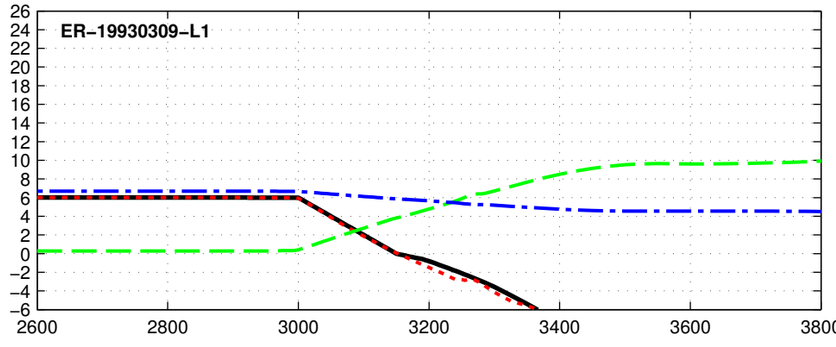
feet, NAVD

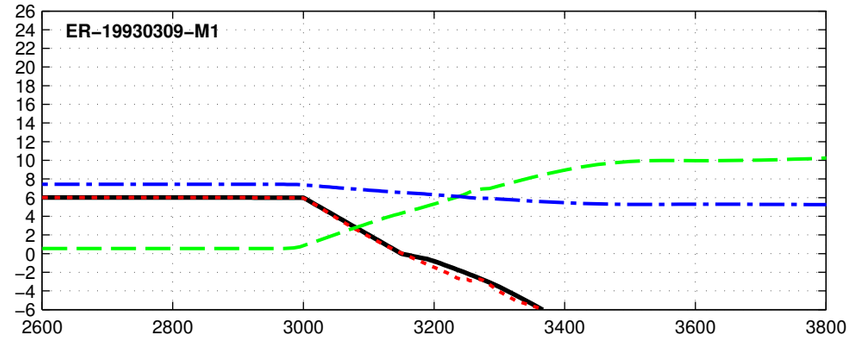
distance (feet)

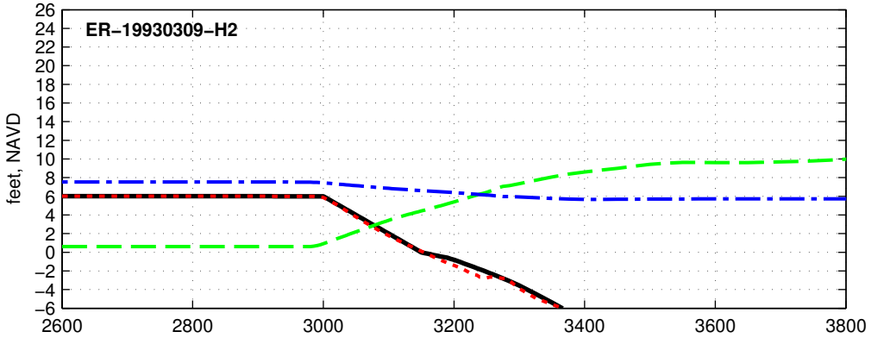
Initial Final Max Hs Max Water Level

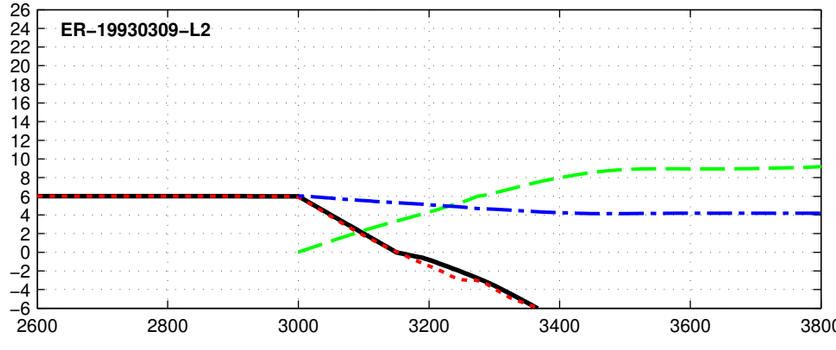
Dune R1T1-6.0-000-000 - ER-19930309

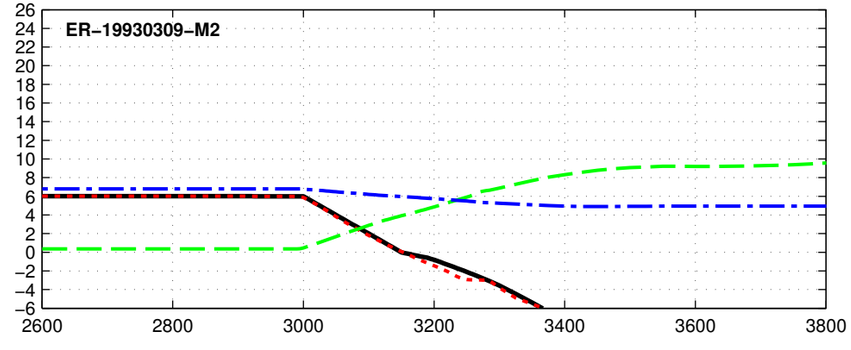


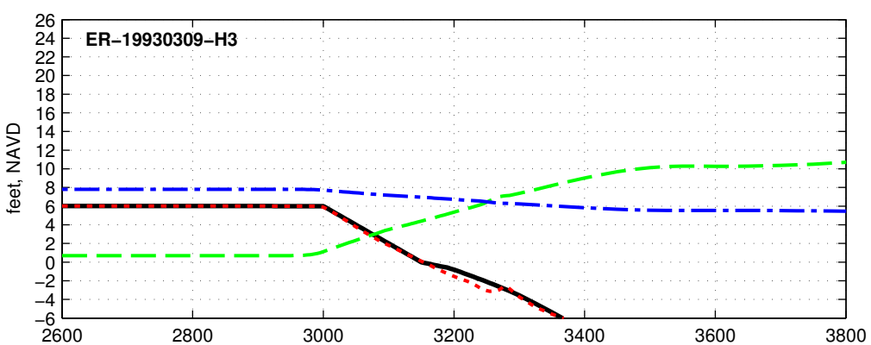


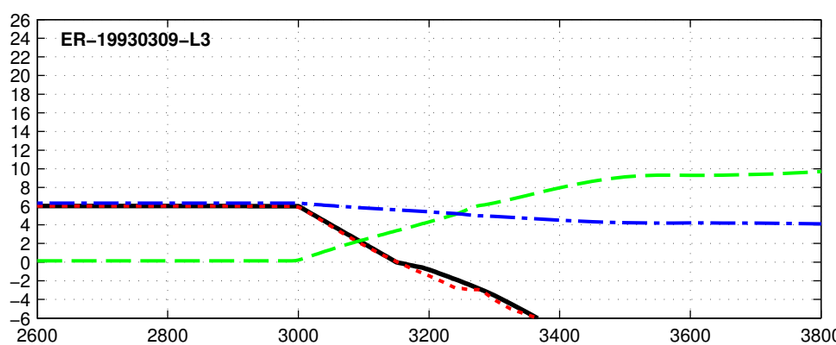


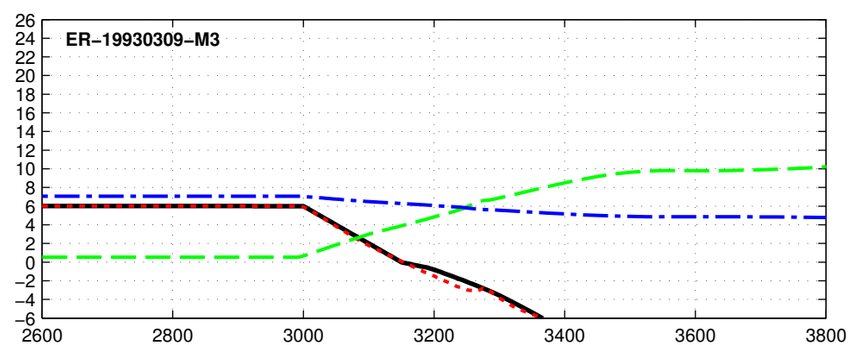


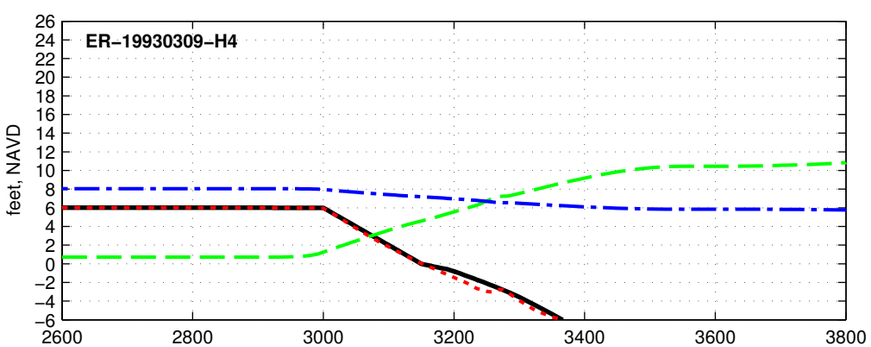


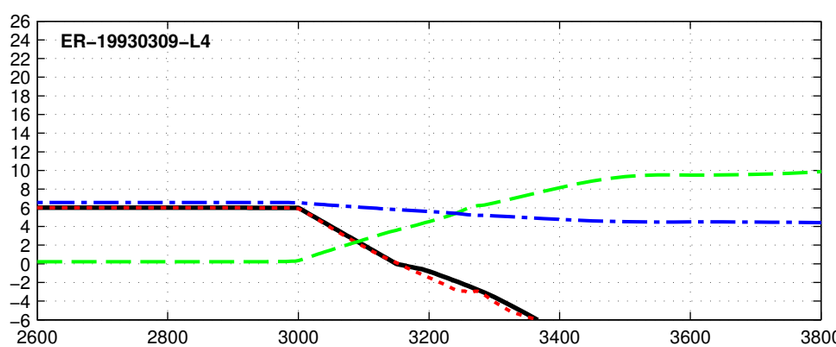


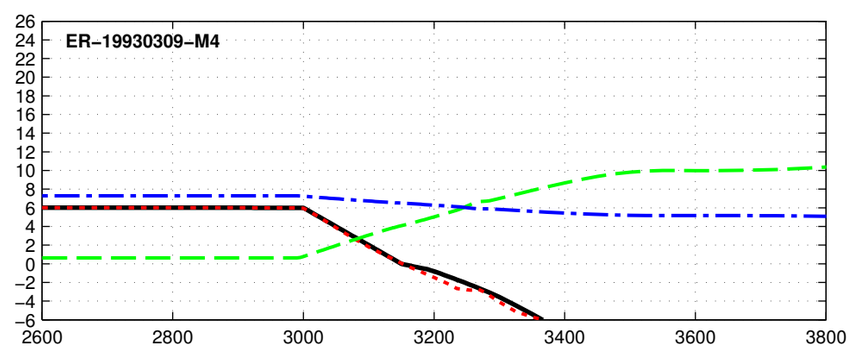










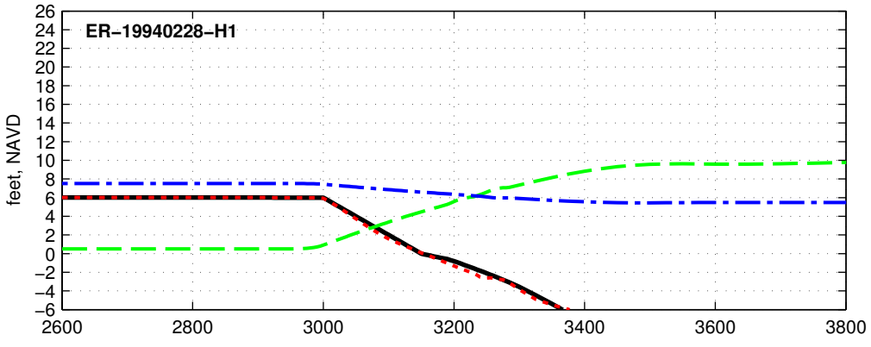


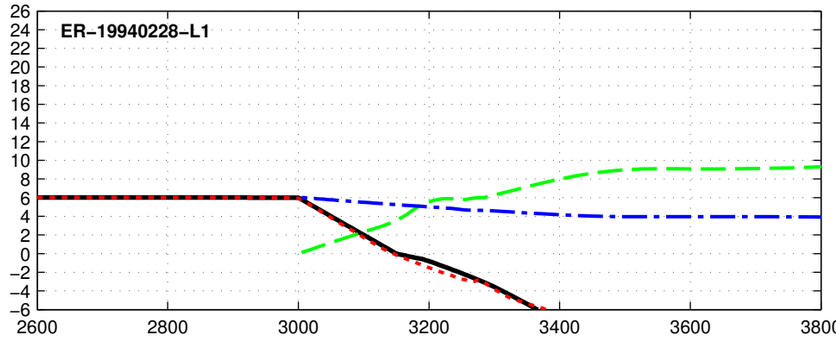
feet, NAVD

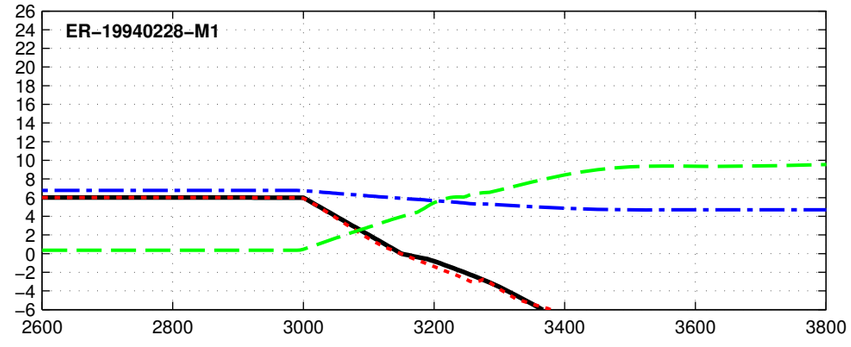
distance (feet)

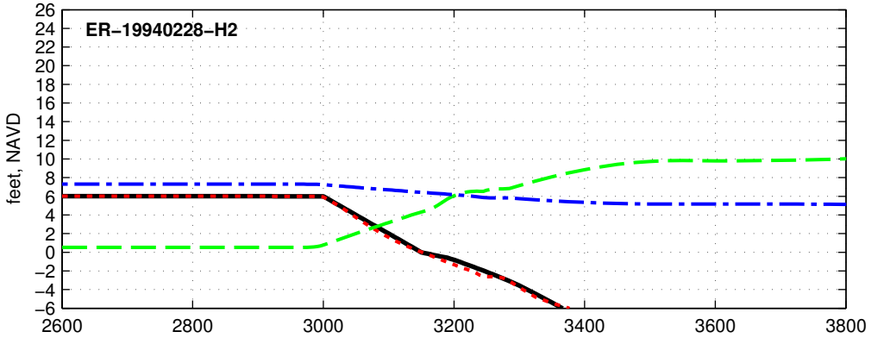
Initial Final Max Hs Max Water Level

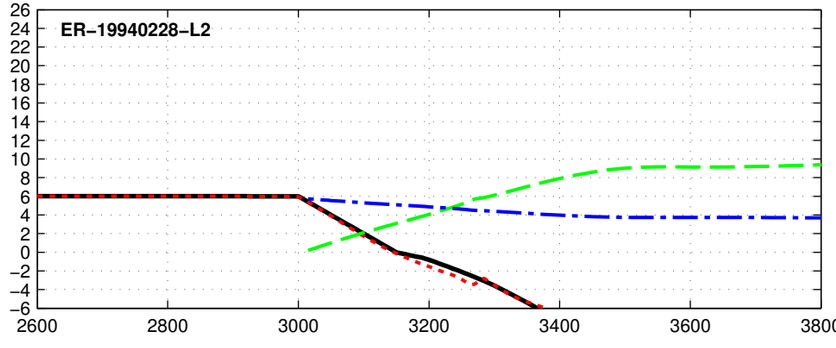
Dune R1T1-6.0-000-000 - ER-19940228

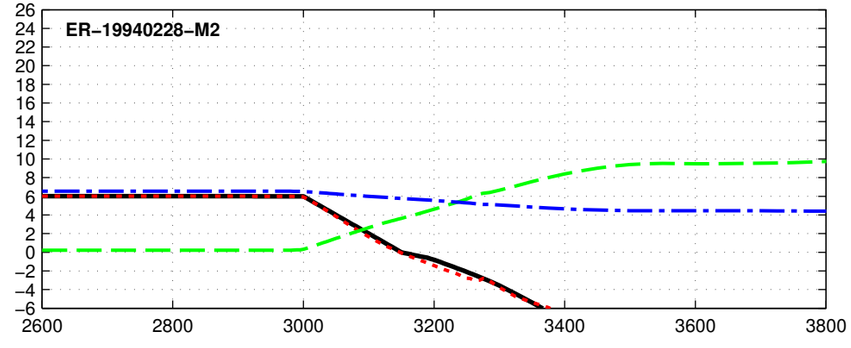


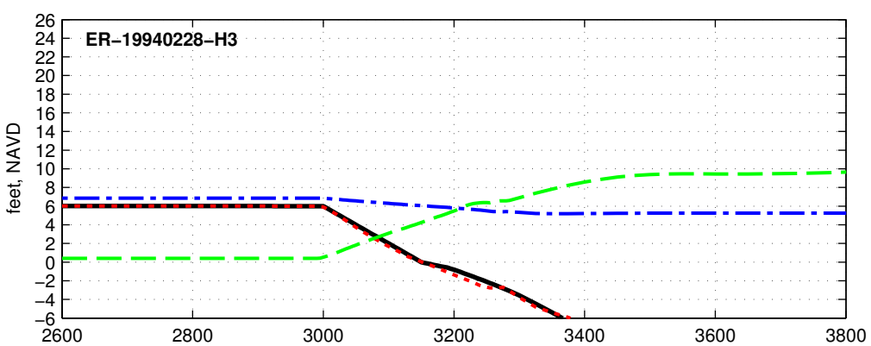


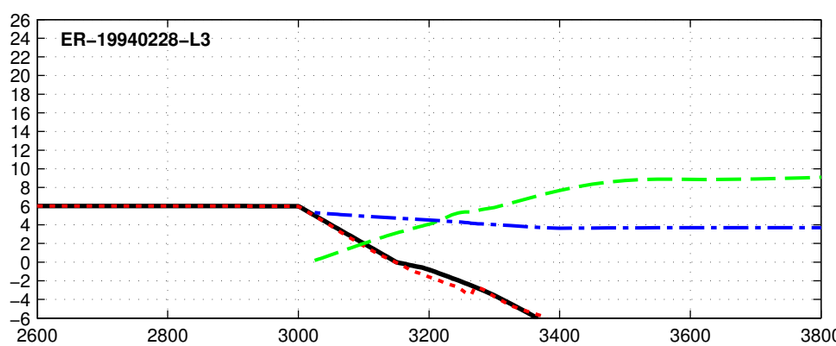


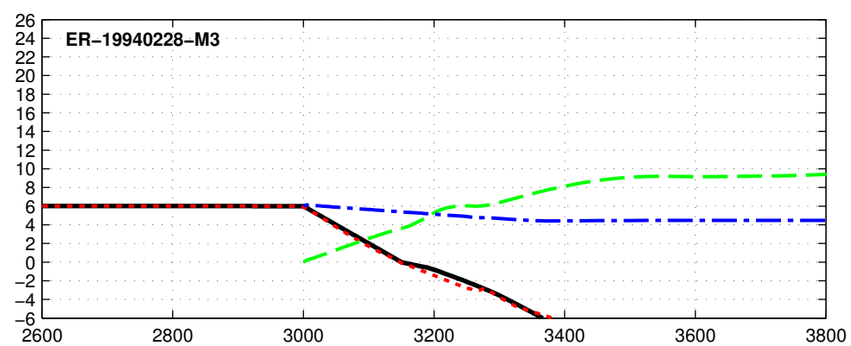


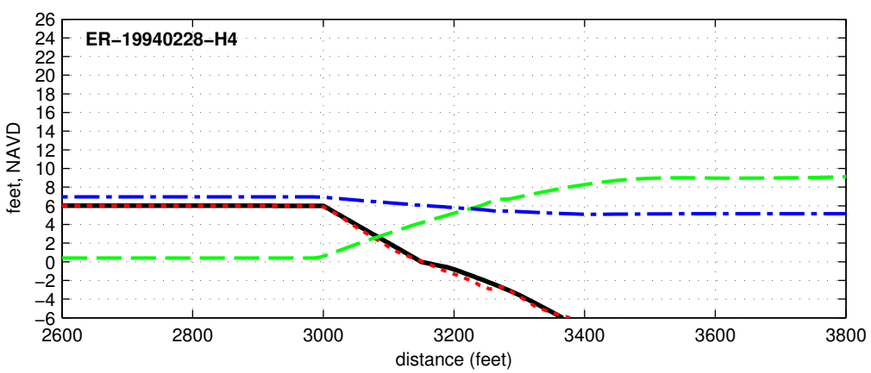


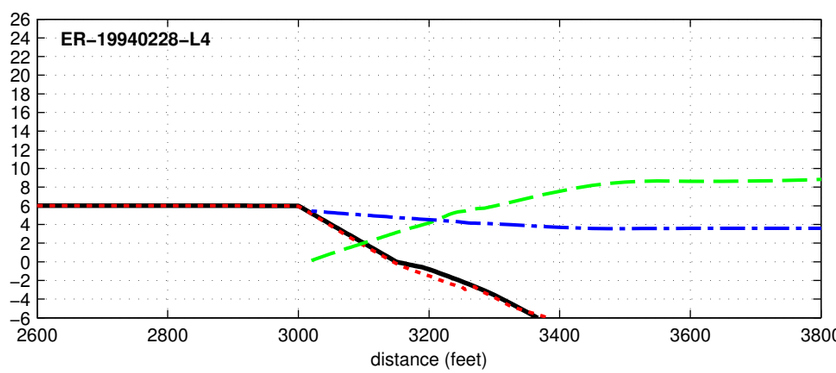


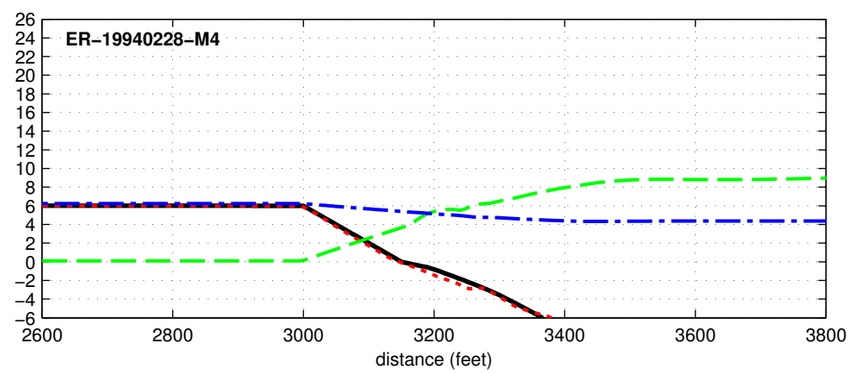






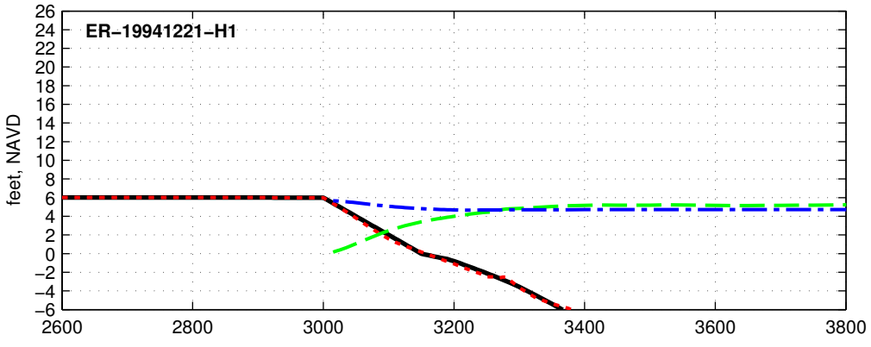


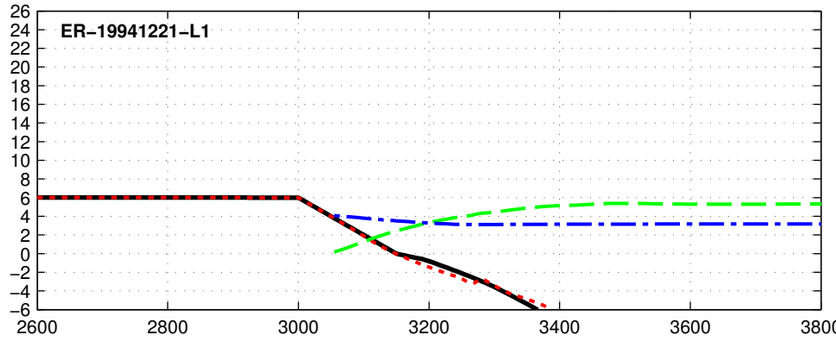


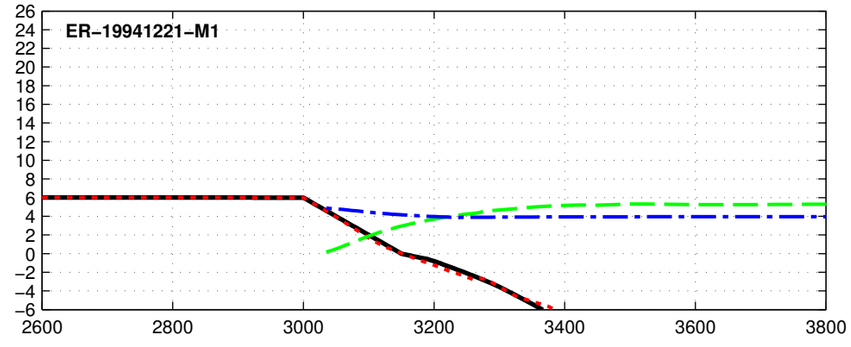


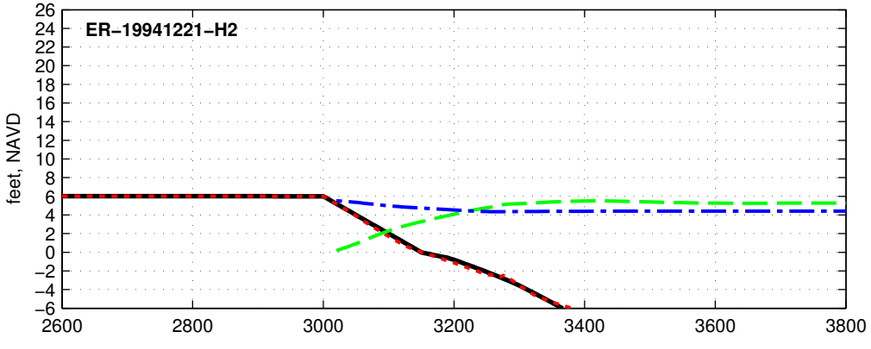
Initial Final Max Hs Max Water Level

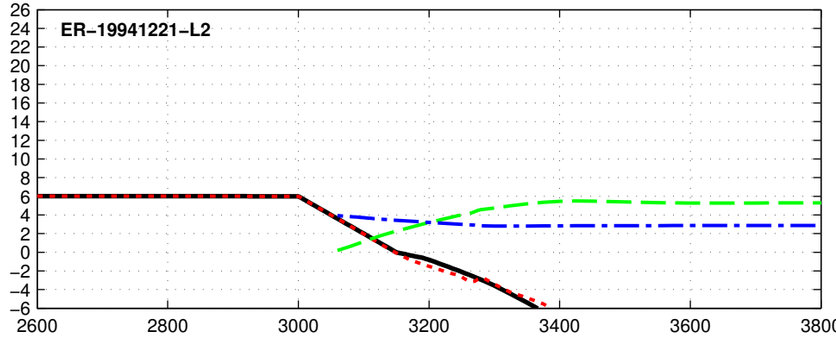
Dune R1T1-6.0-000-000 - ER-19941221

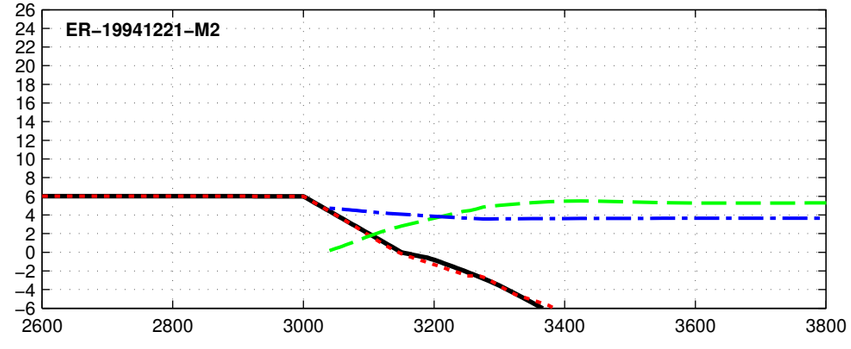


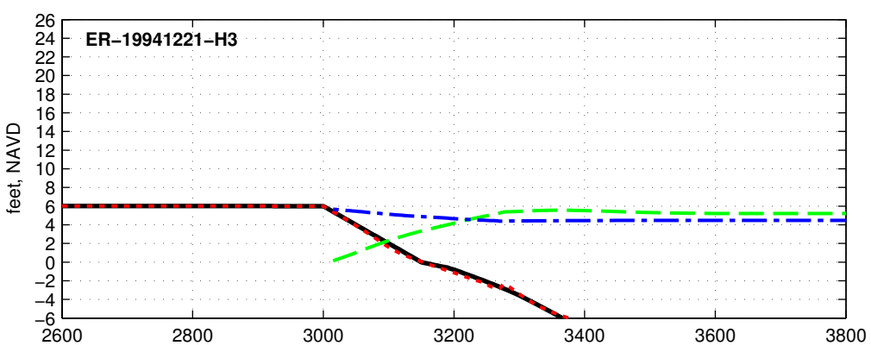


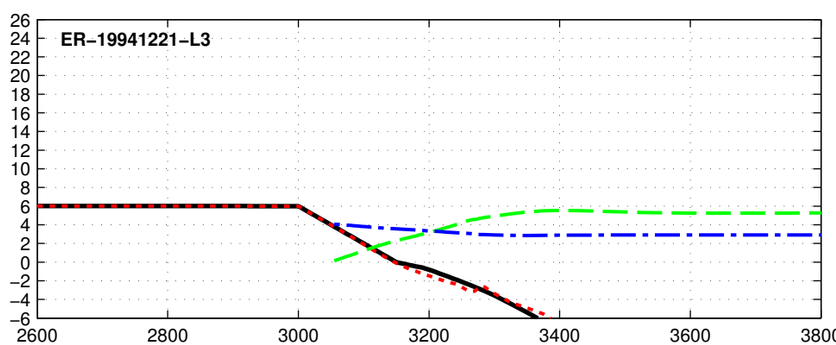


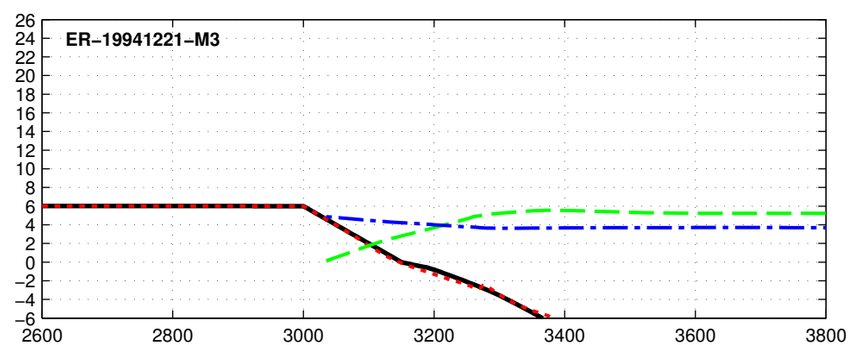


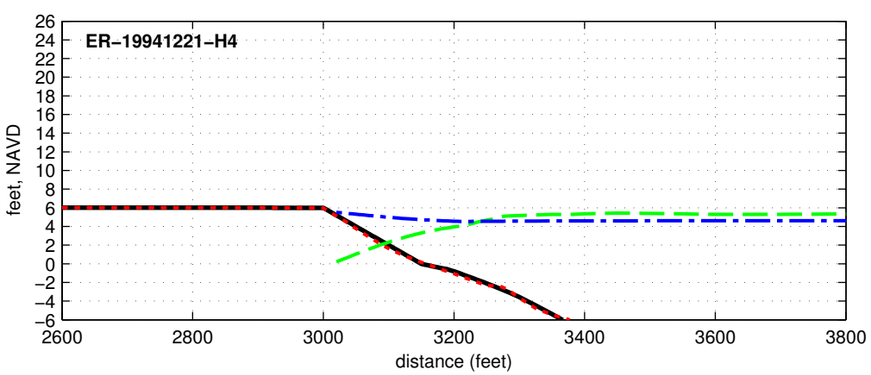


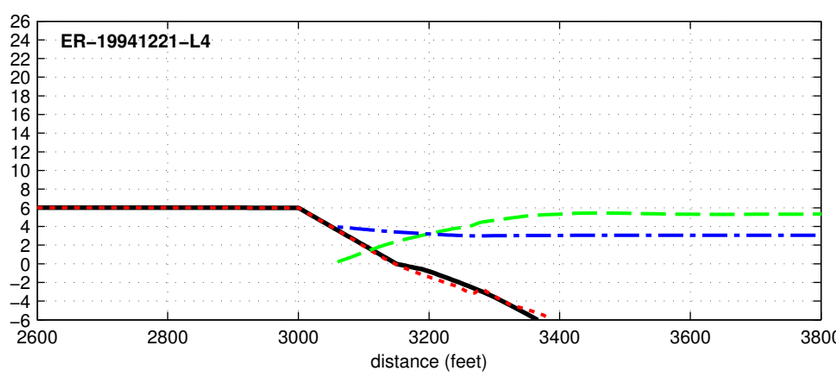


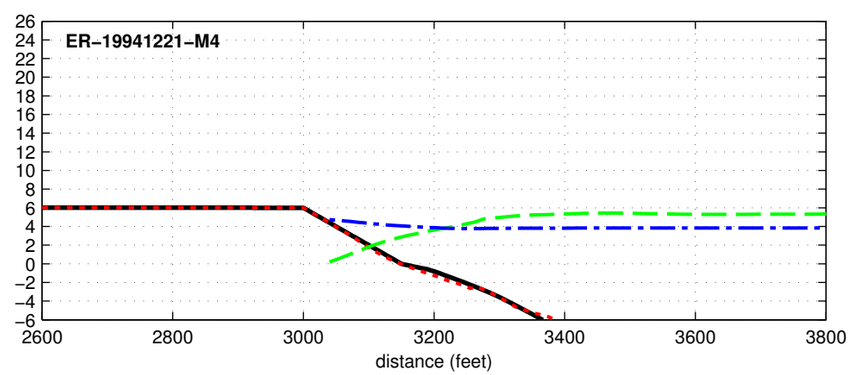






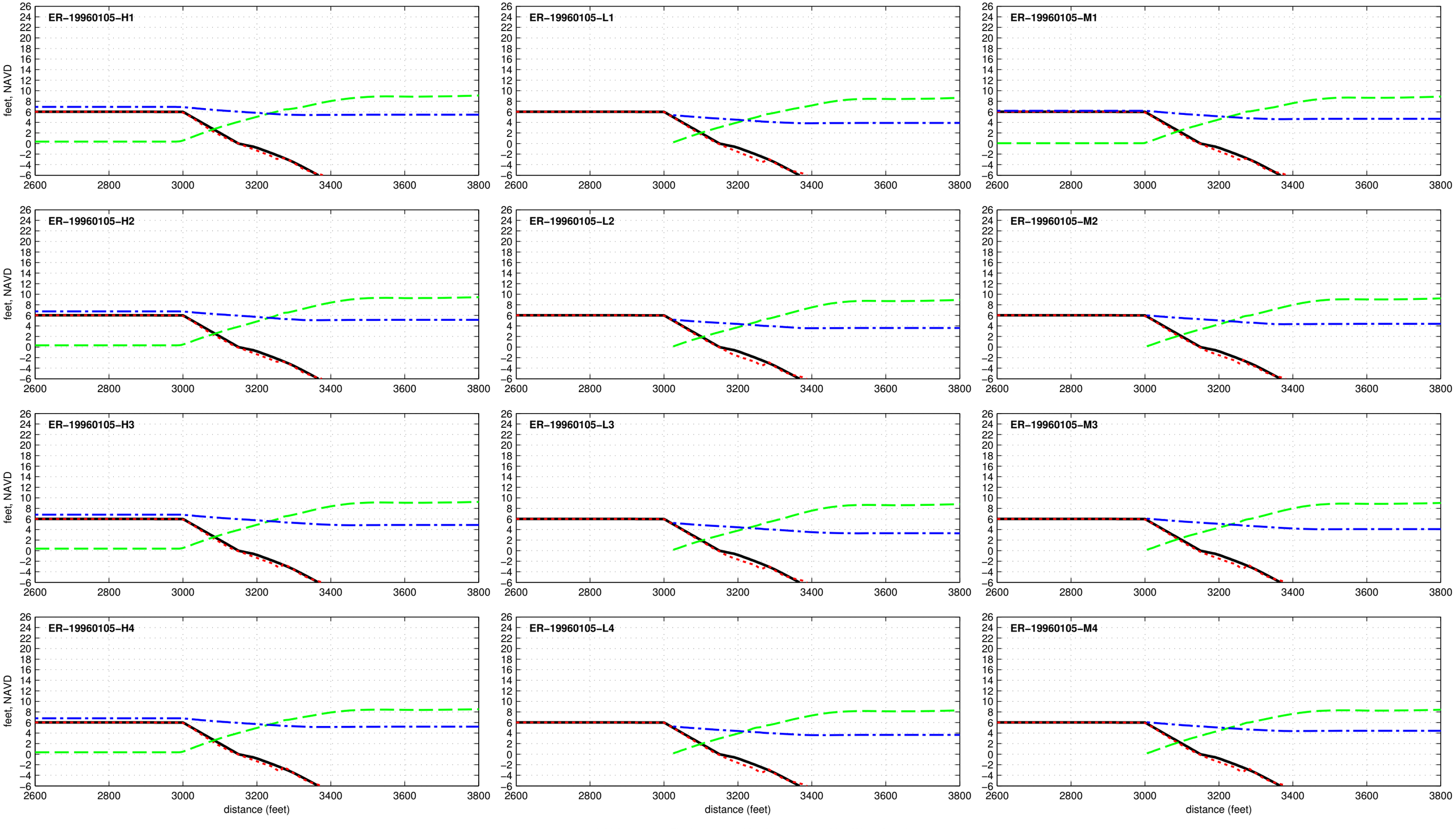






Initial Final Max Hs Max Water Level

Dune R1T1-6.0-000-000 - ER-19960105



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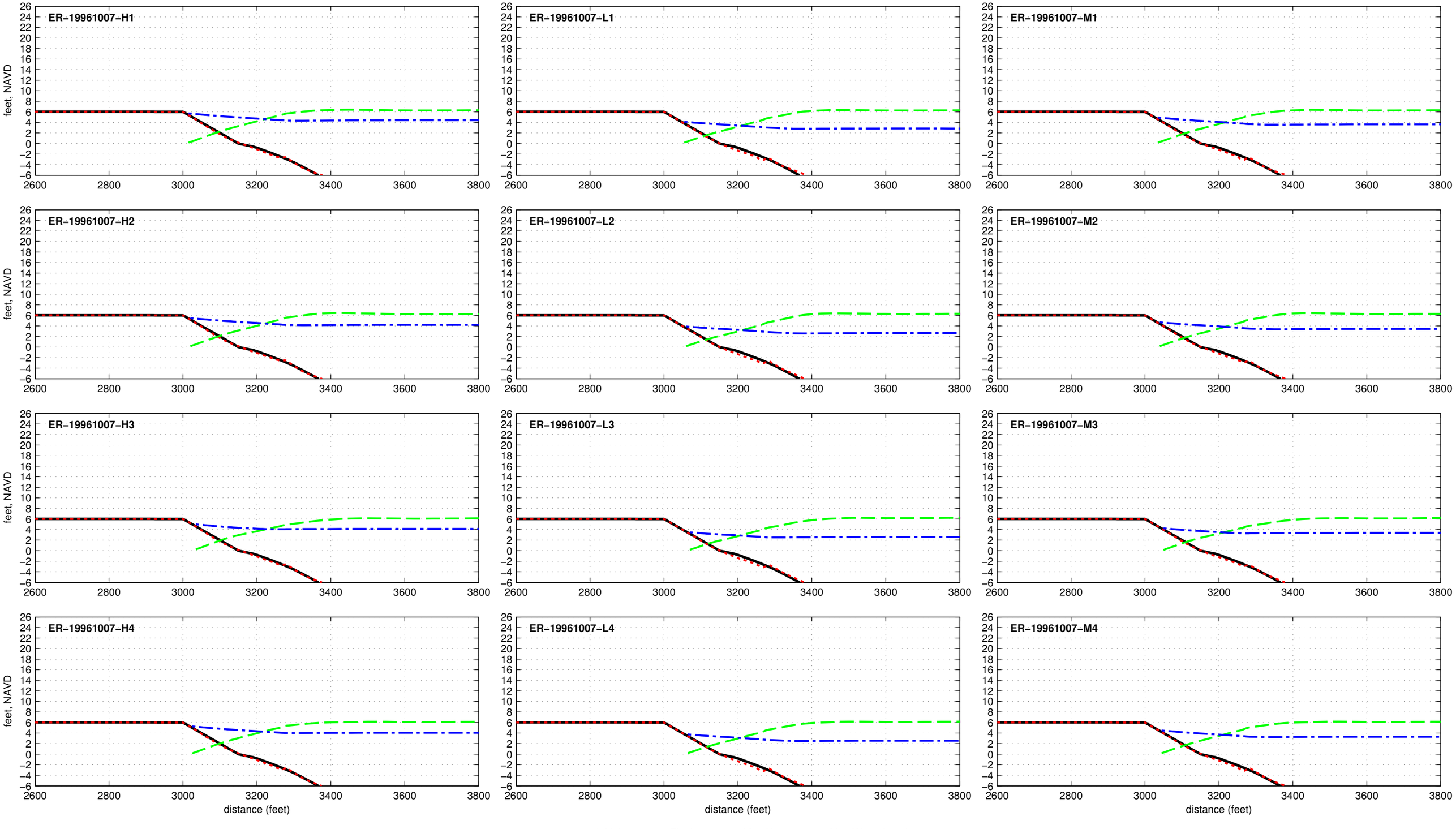

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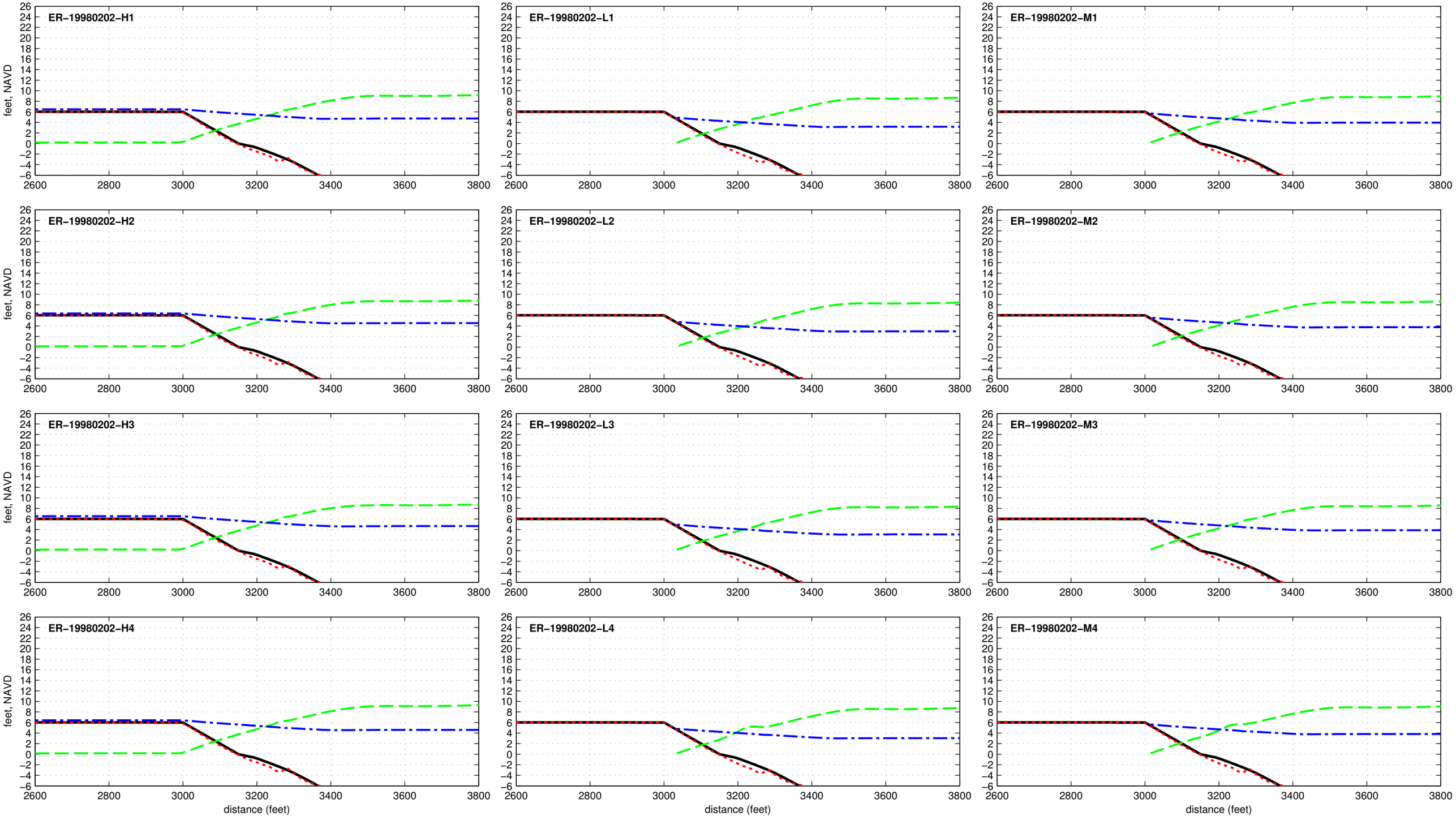

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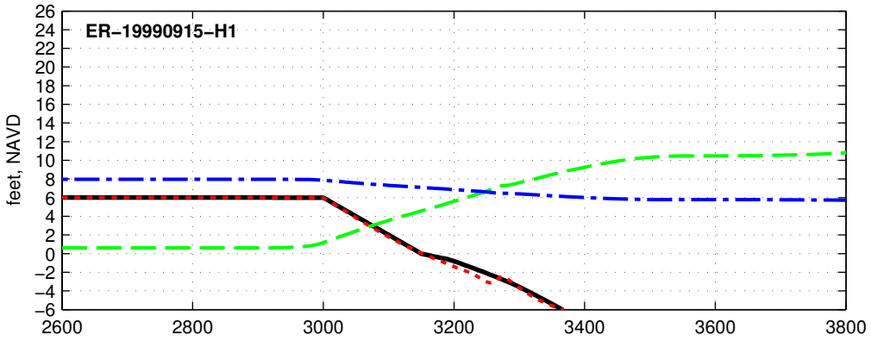
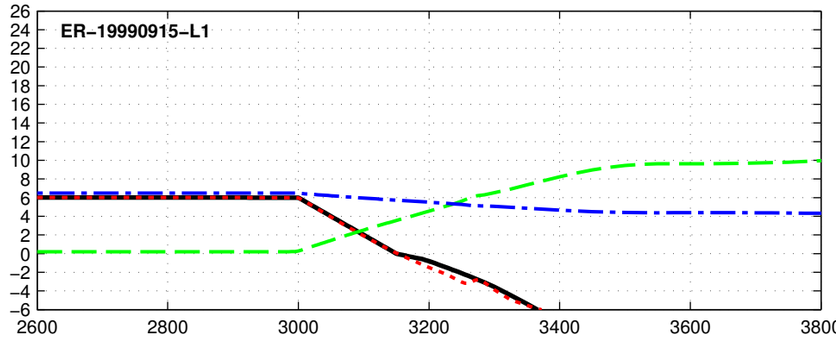
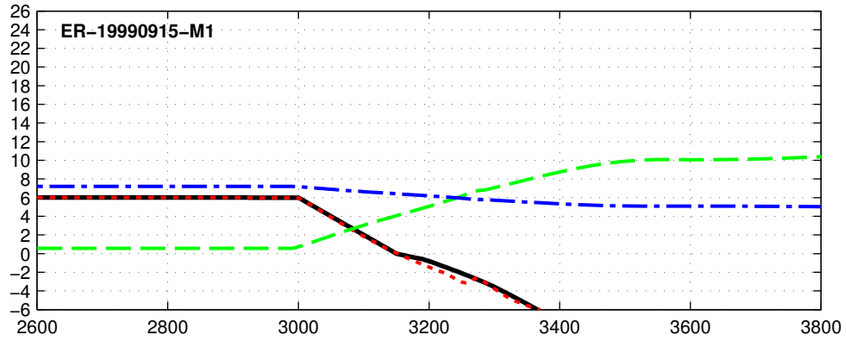
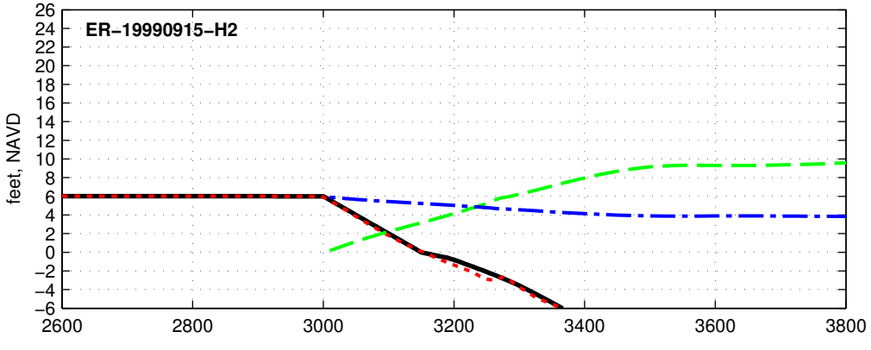
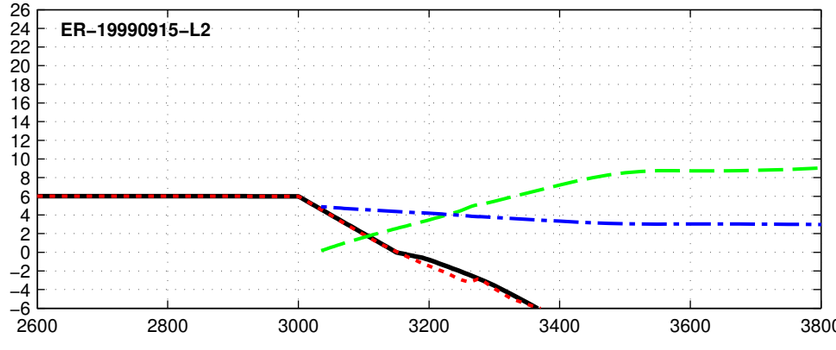
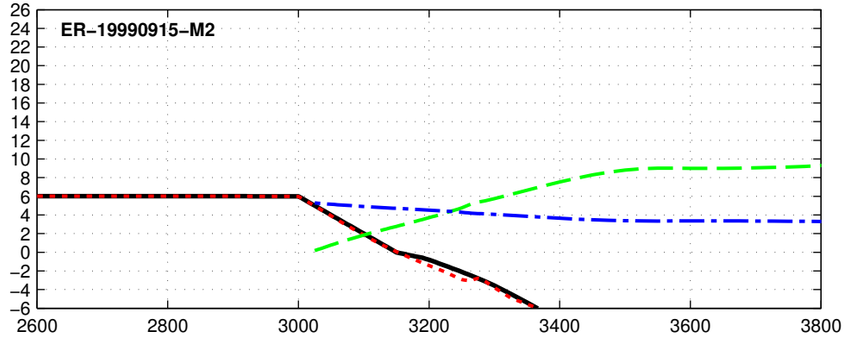


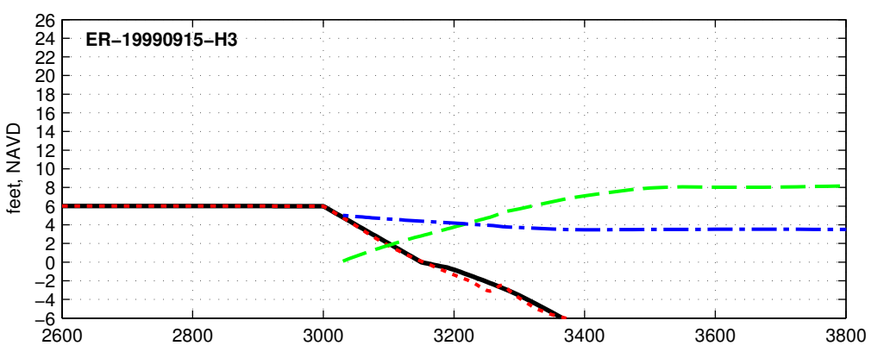


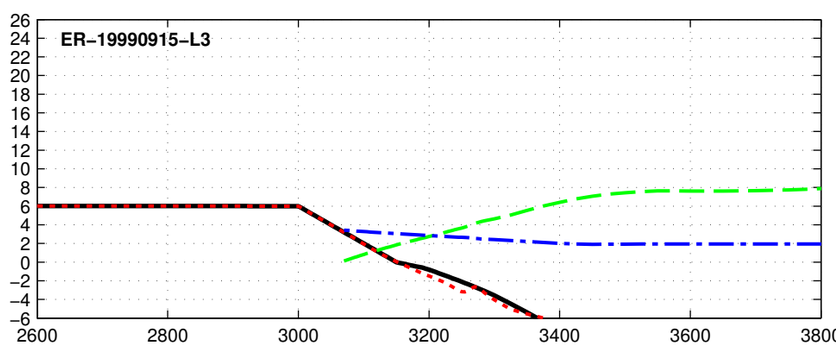


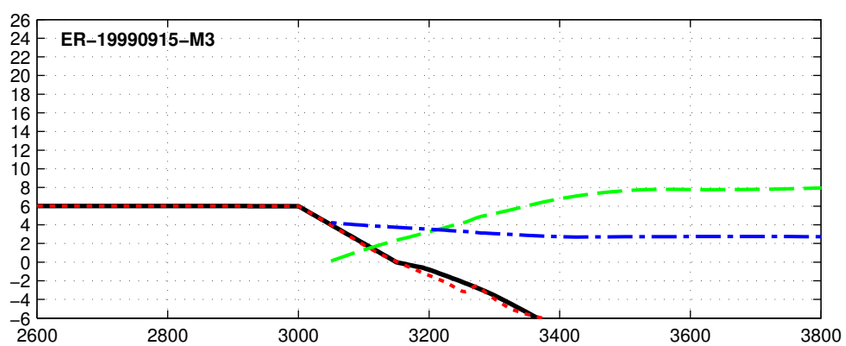


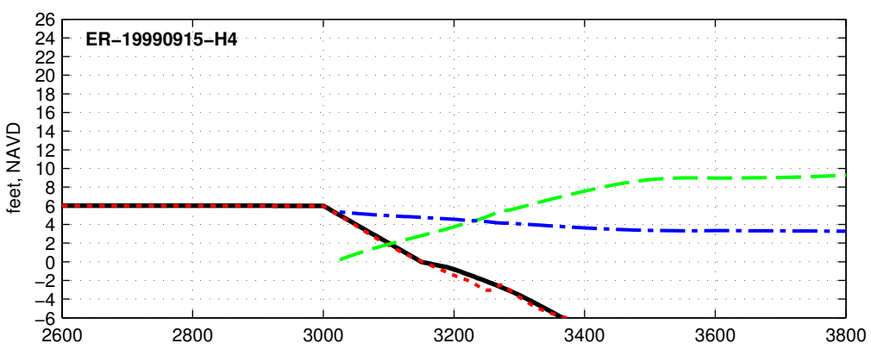


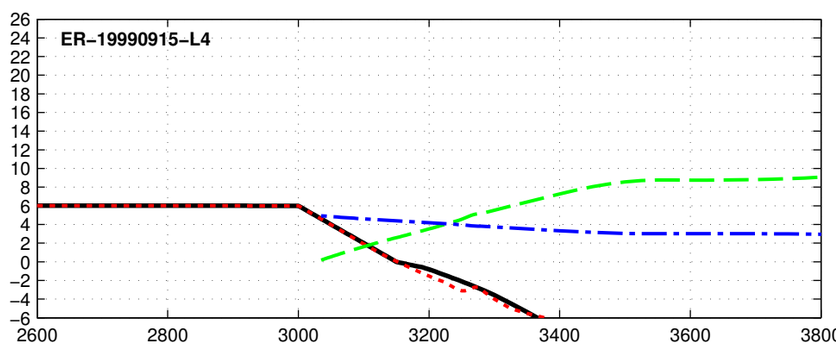


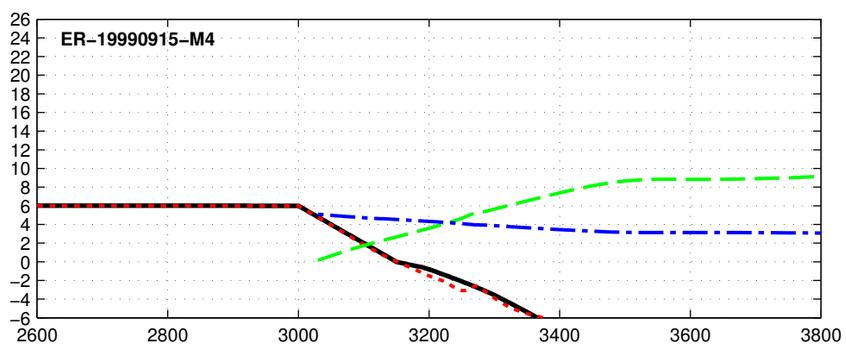






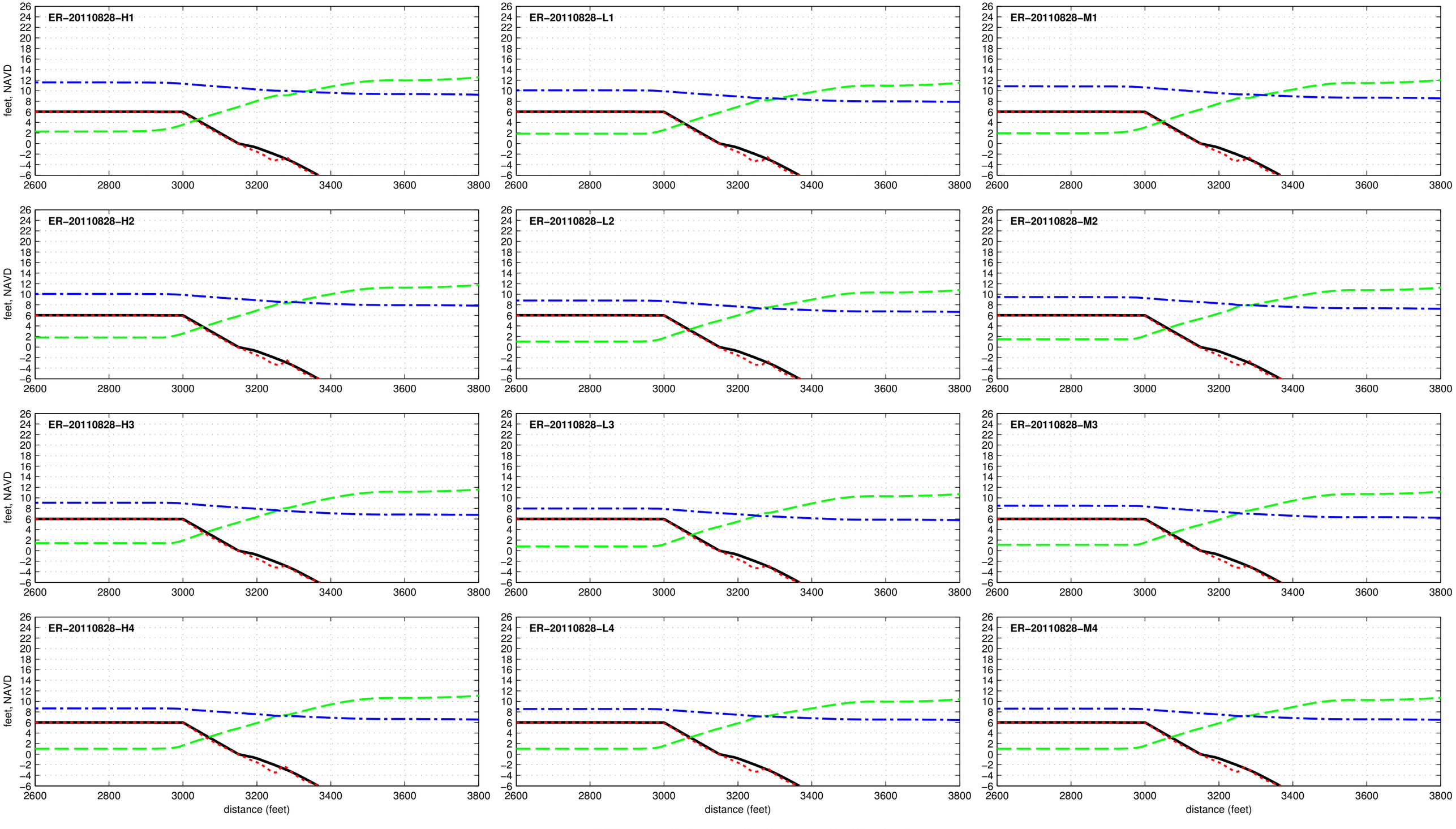






Initial Final Max Hs Max Water Level

Dune R1T1-6.0-000-000 - ER-20110828



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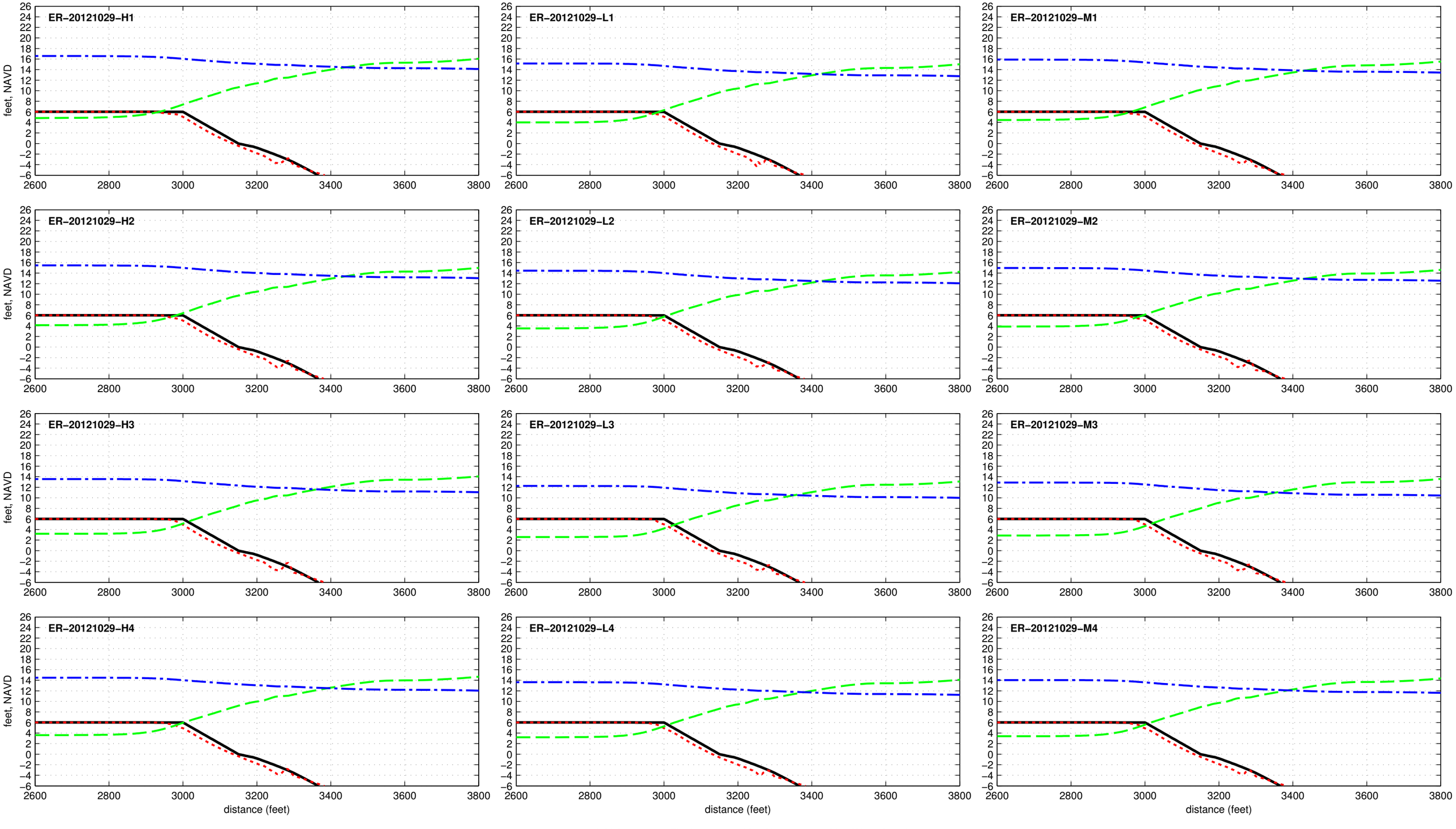
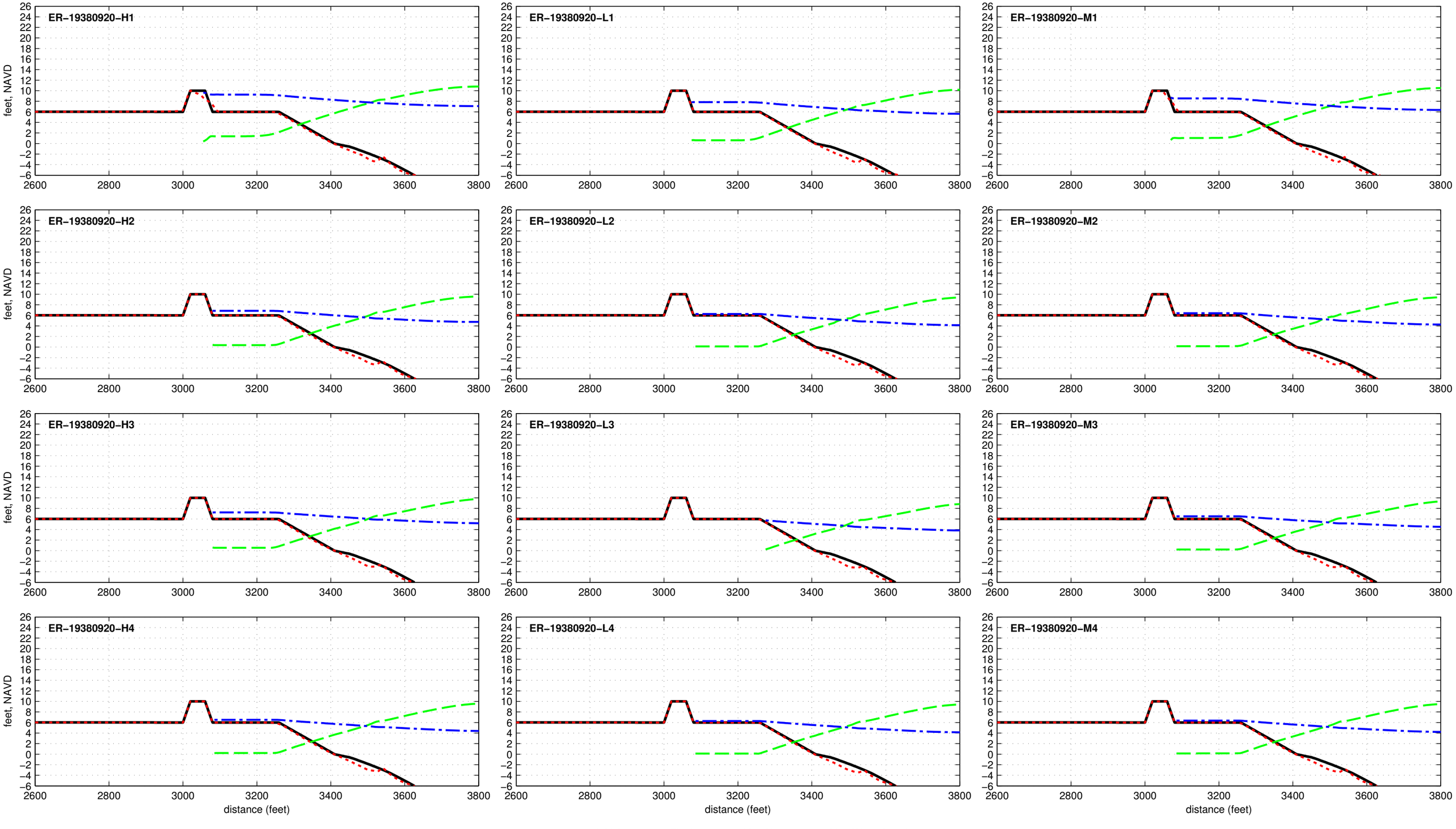


R1T1_10.0_040_180 - DUNE

Dune R1T1–10.0–040–180 – ER–19380920



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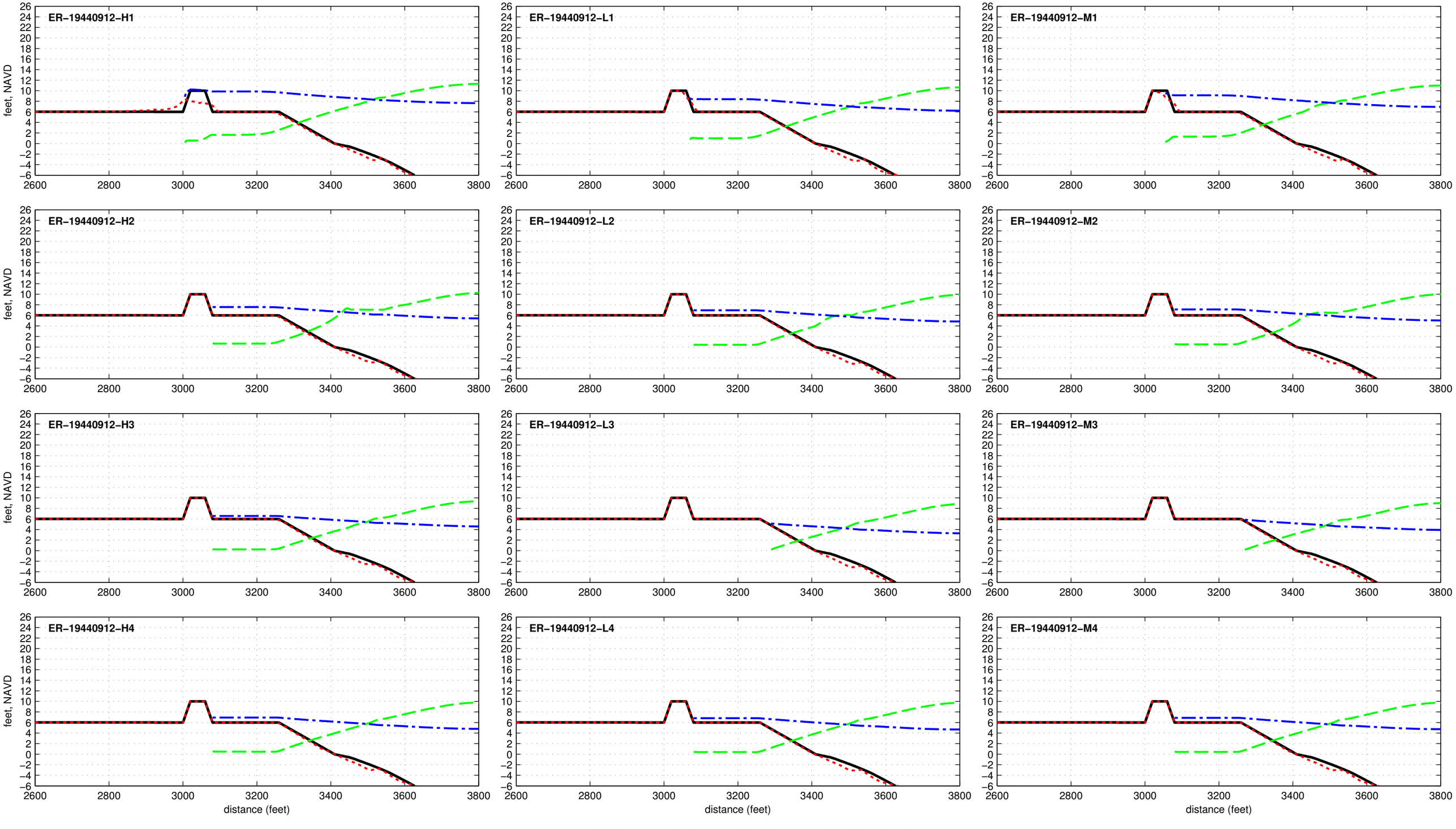

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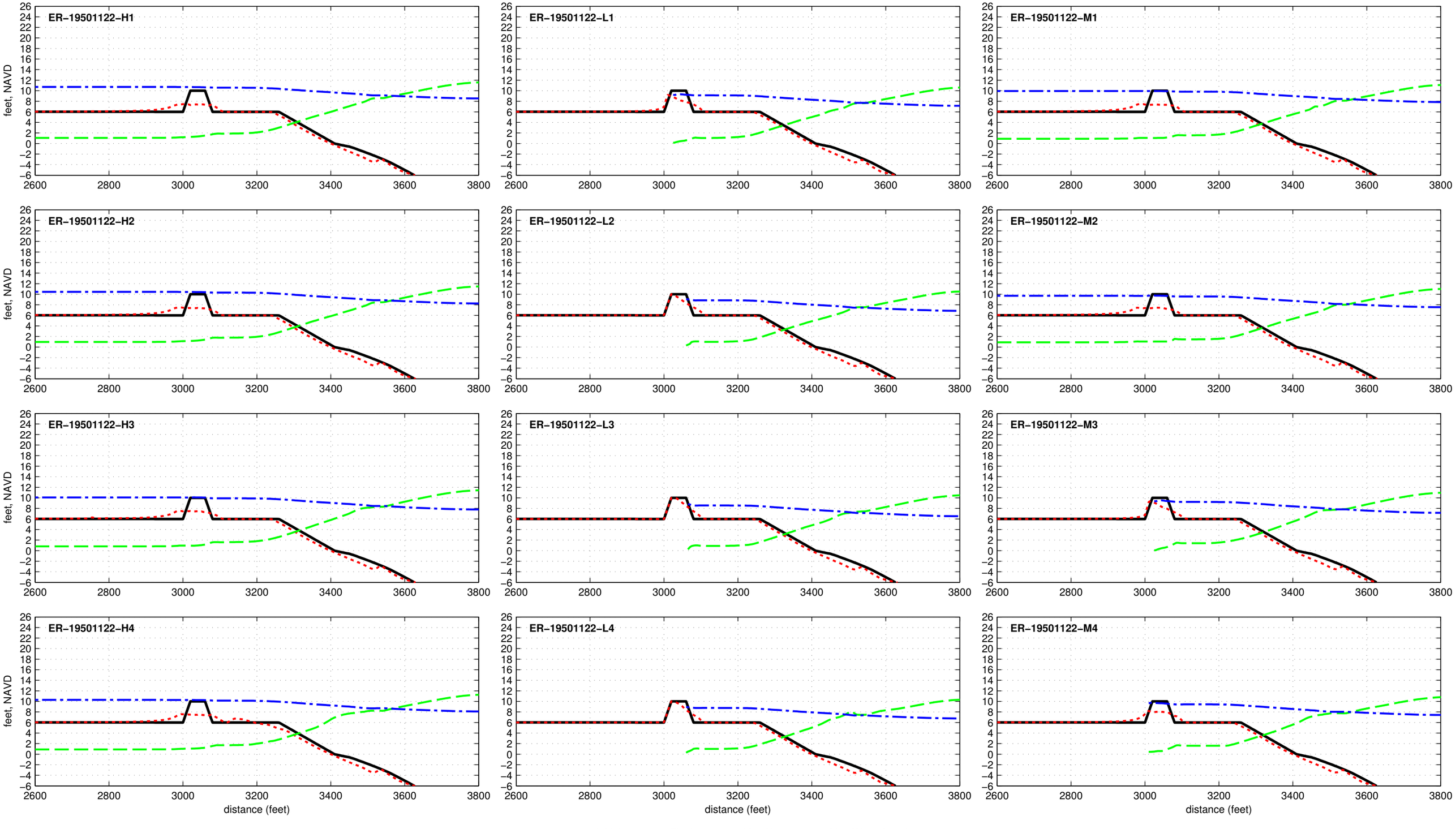

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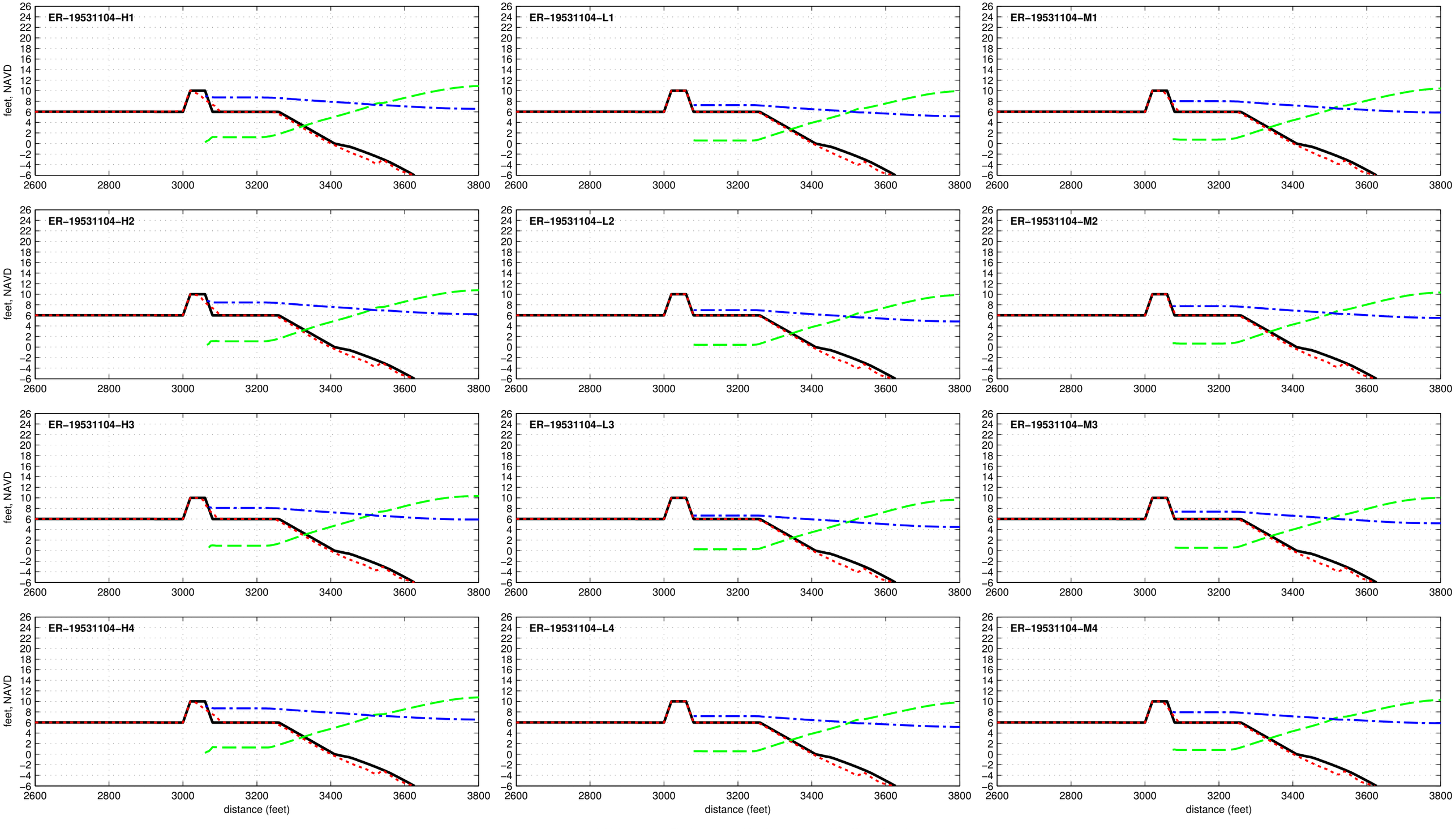

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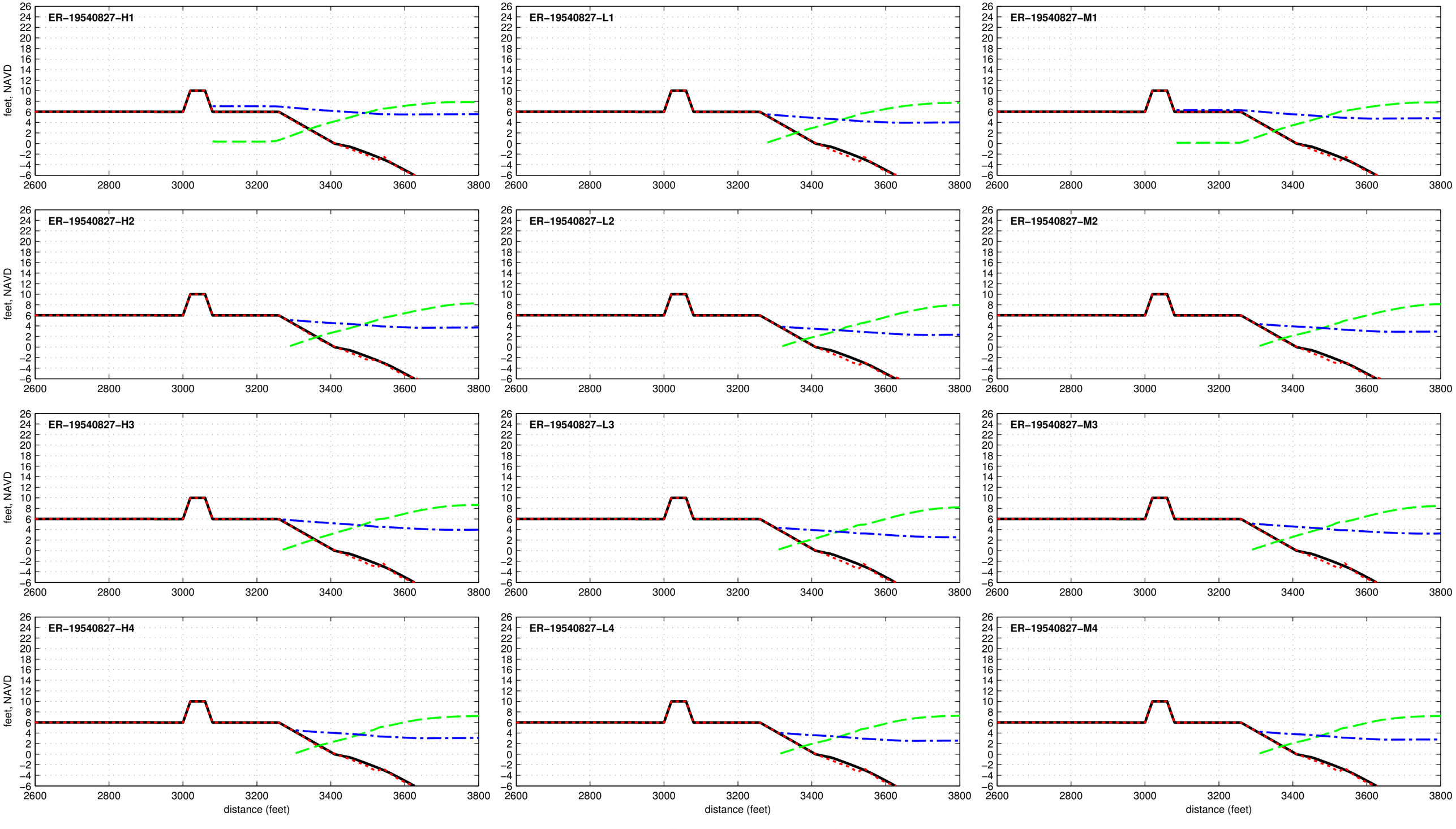

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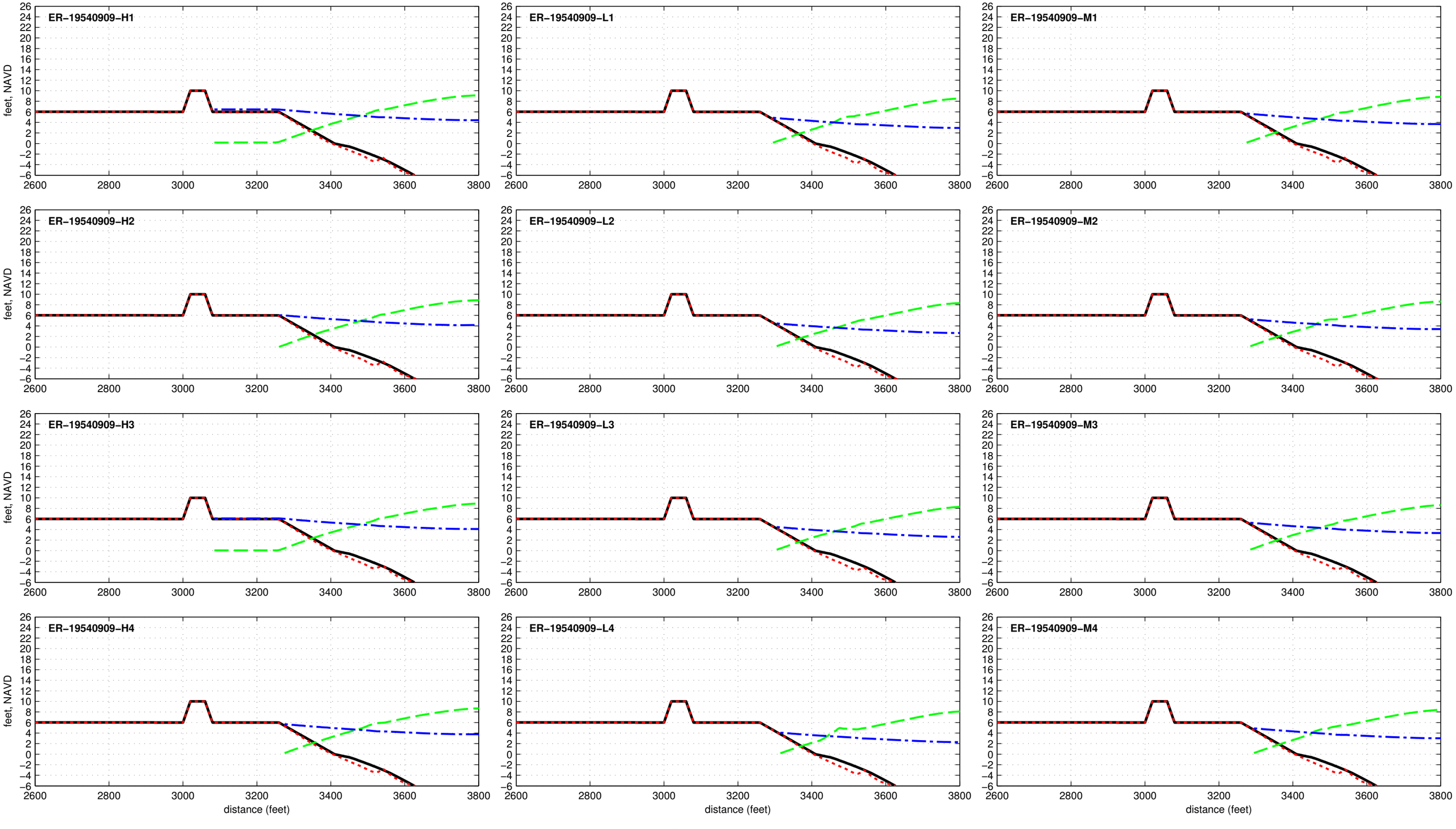

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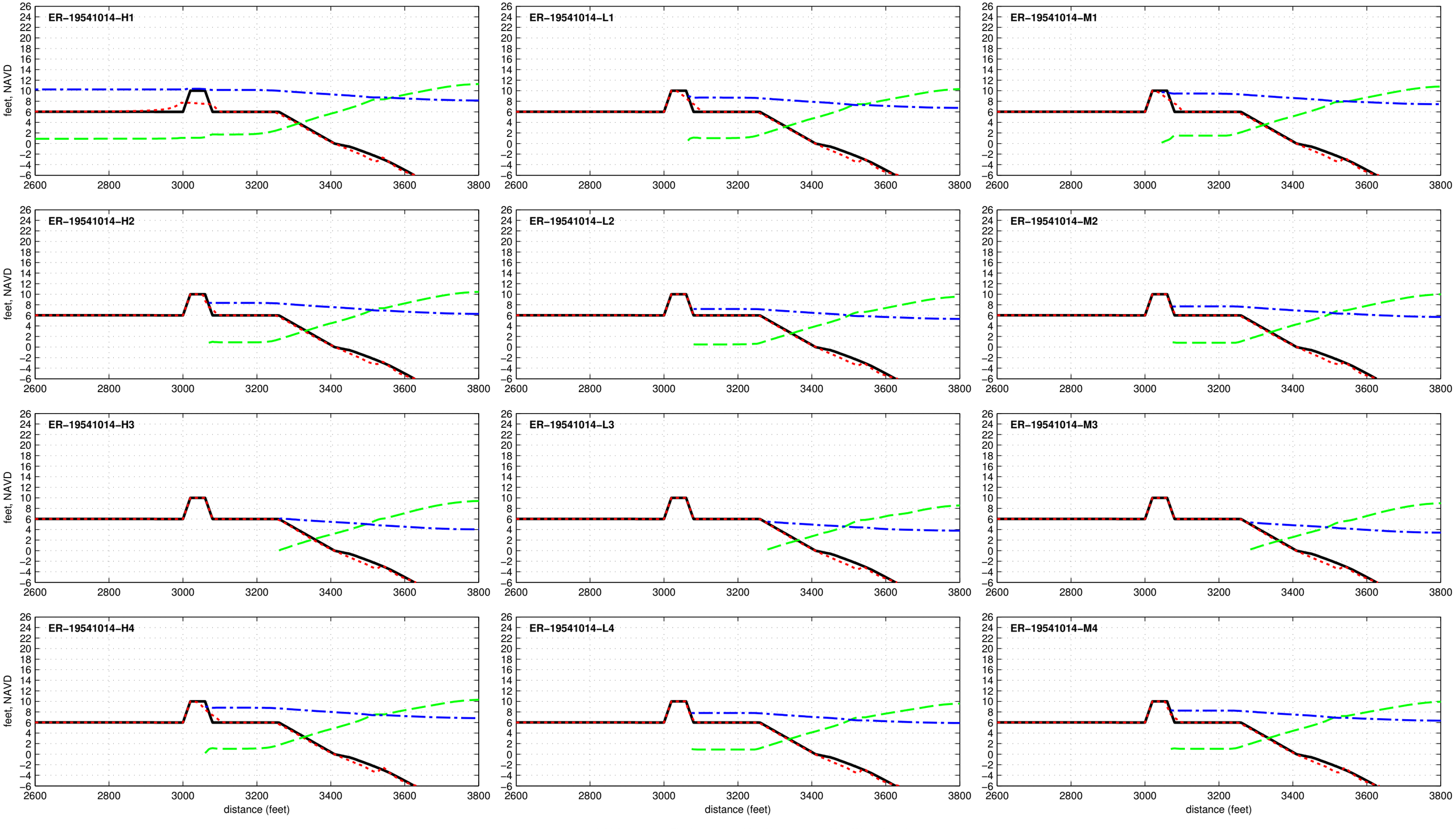

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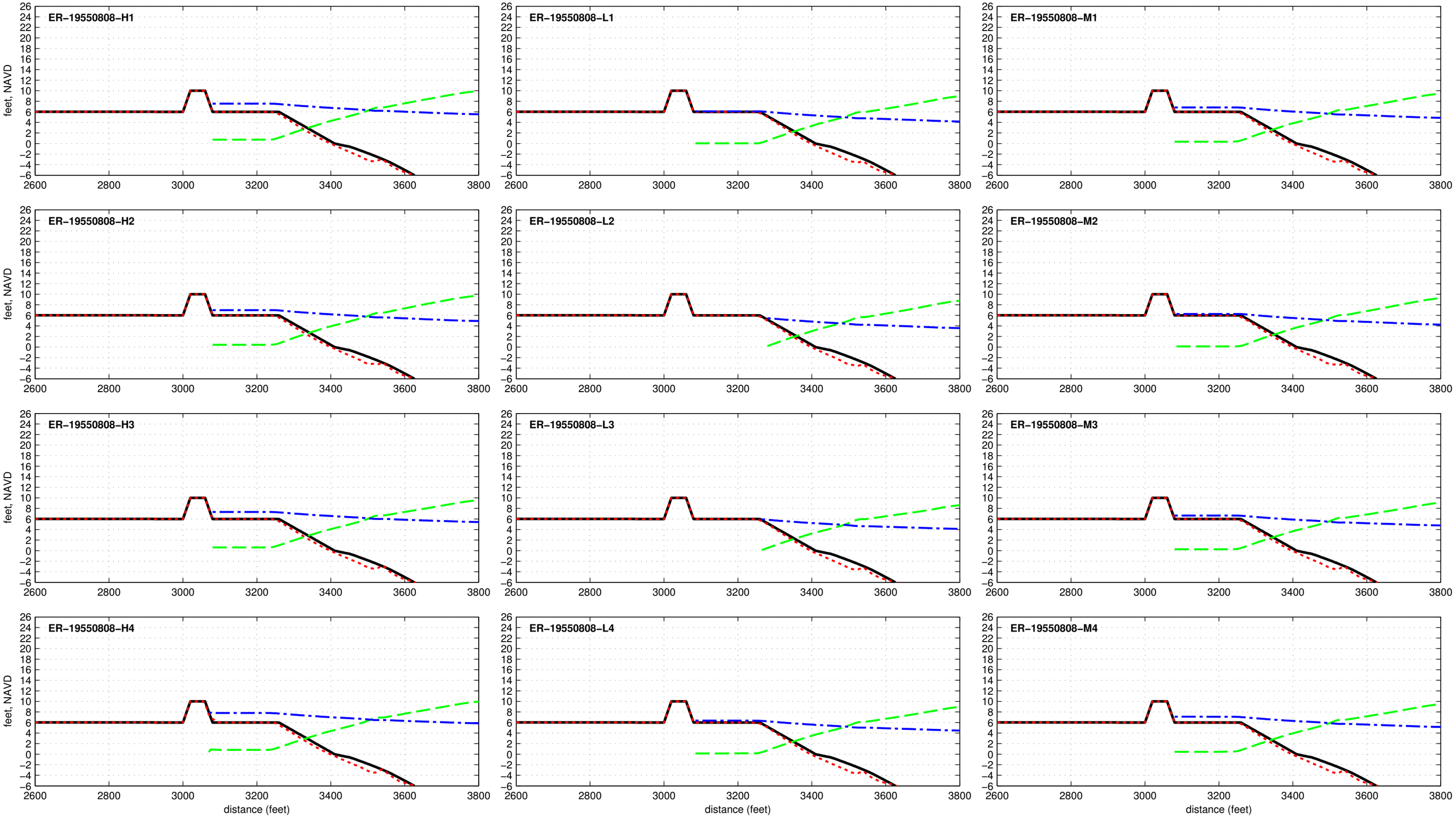

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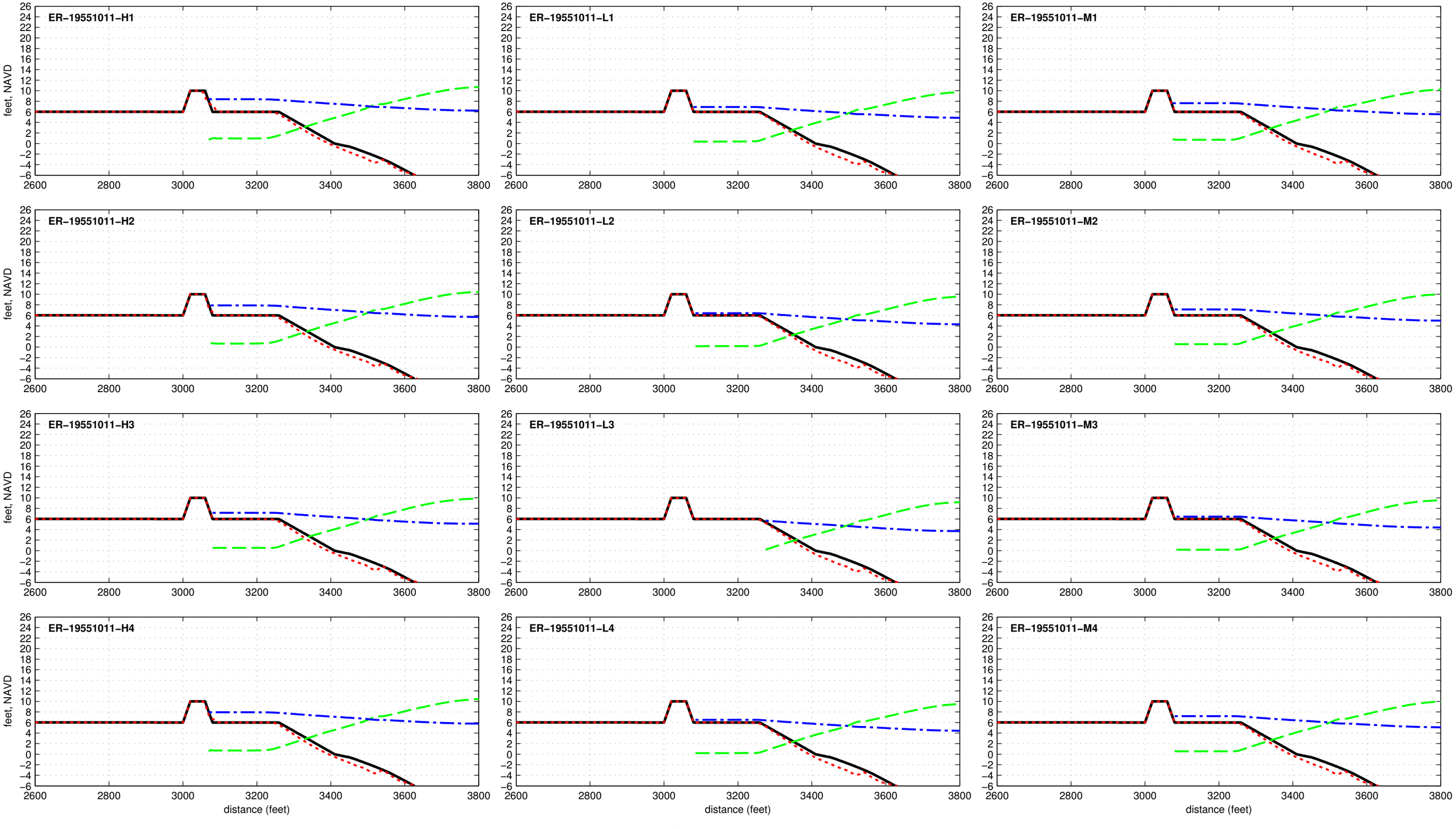

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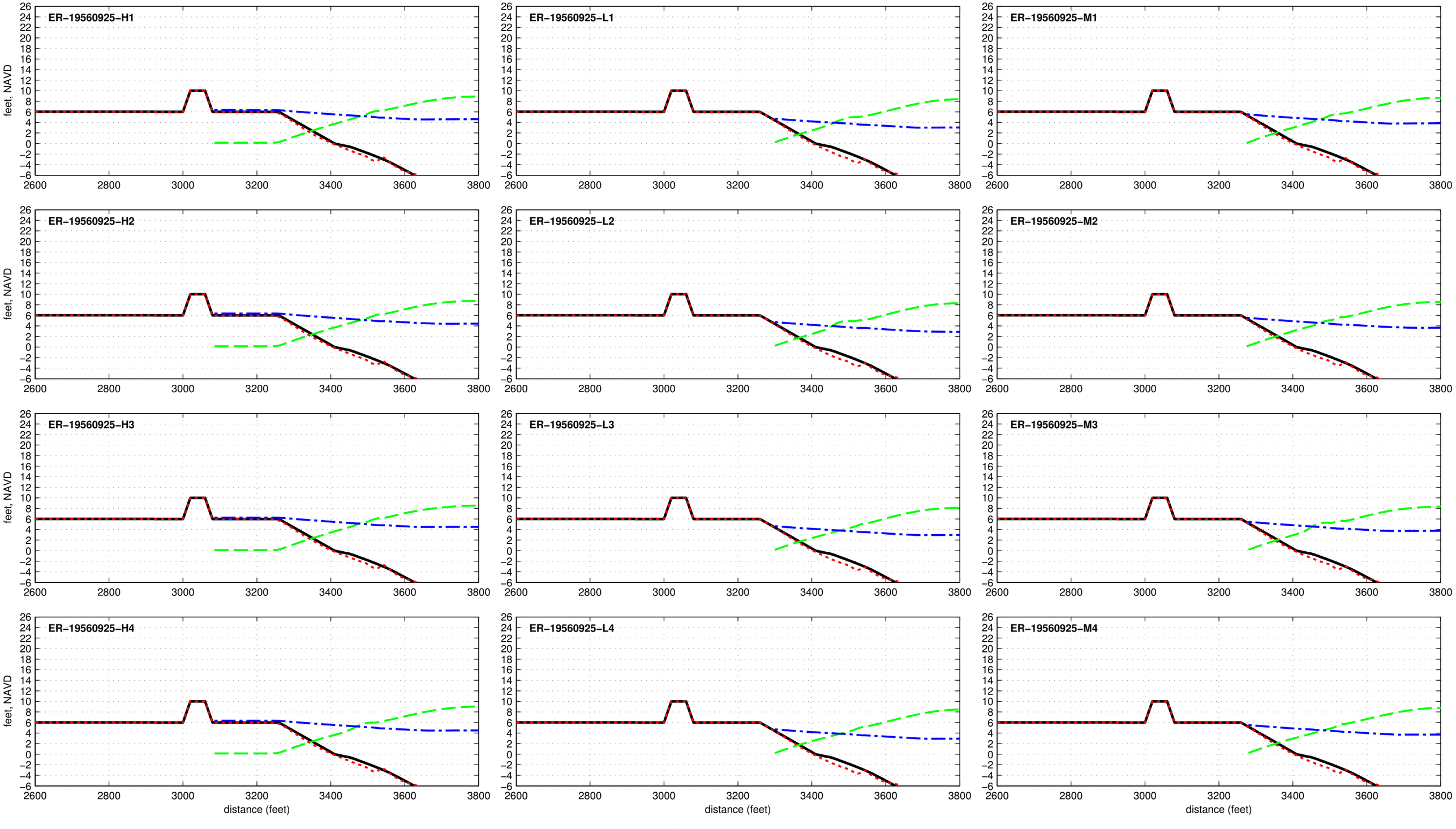

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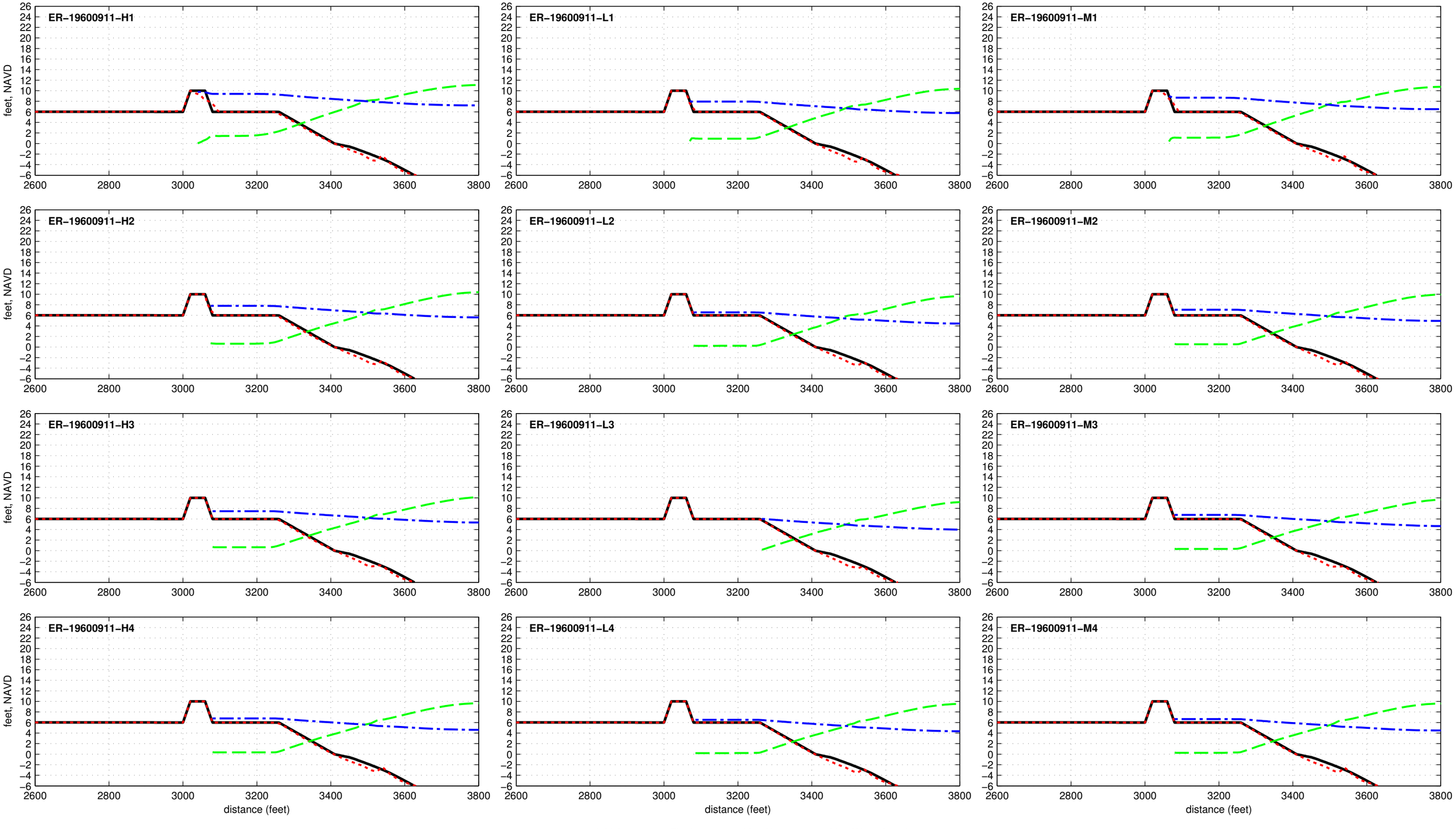

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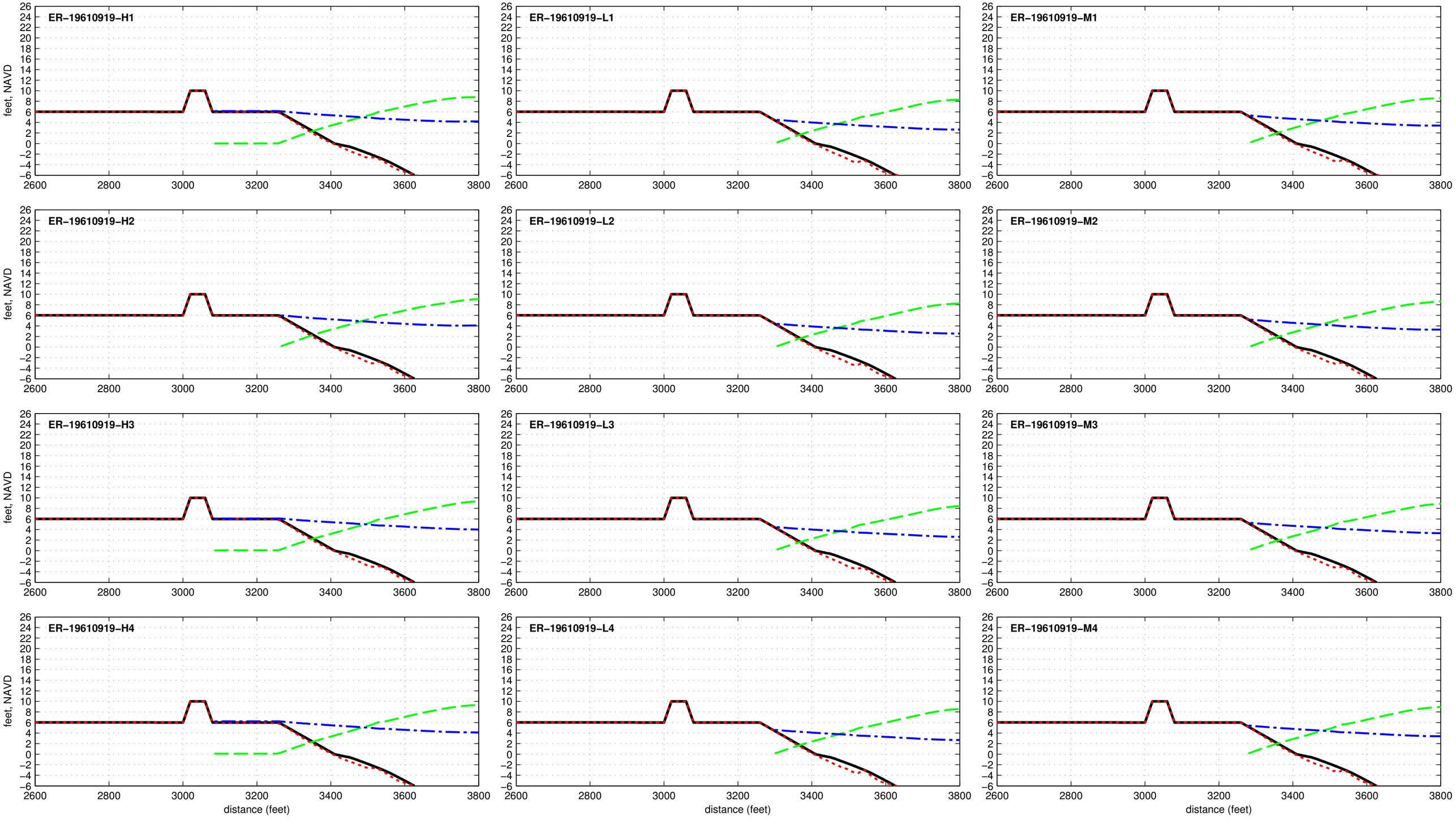

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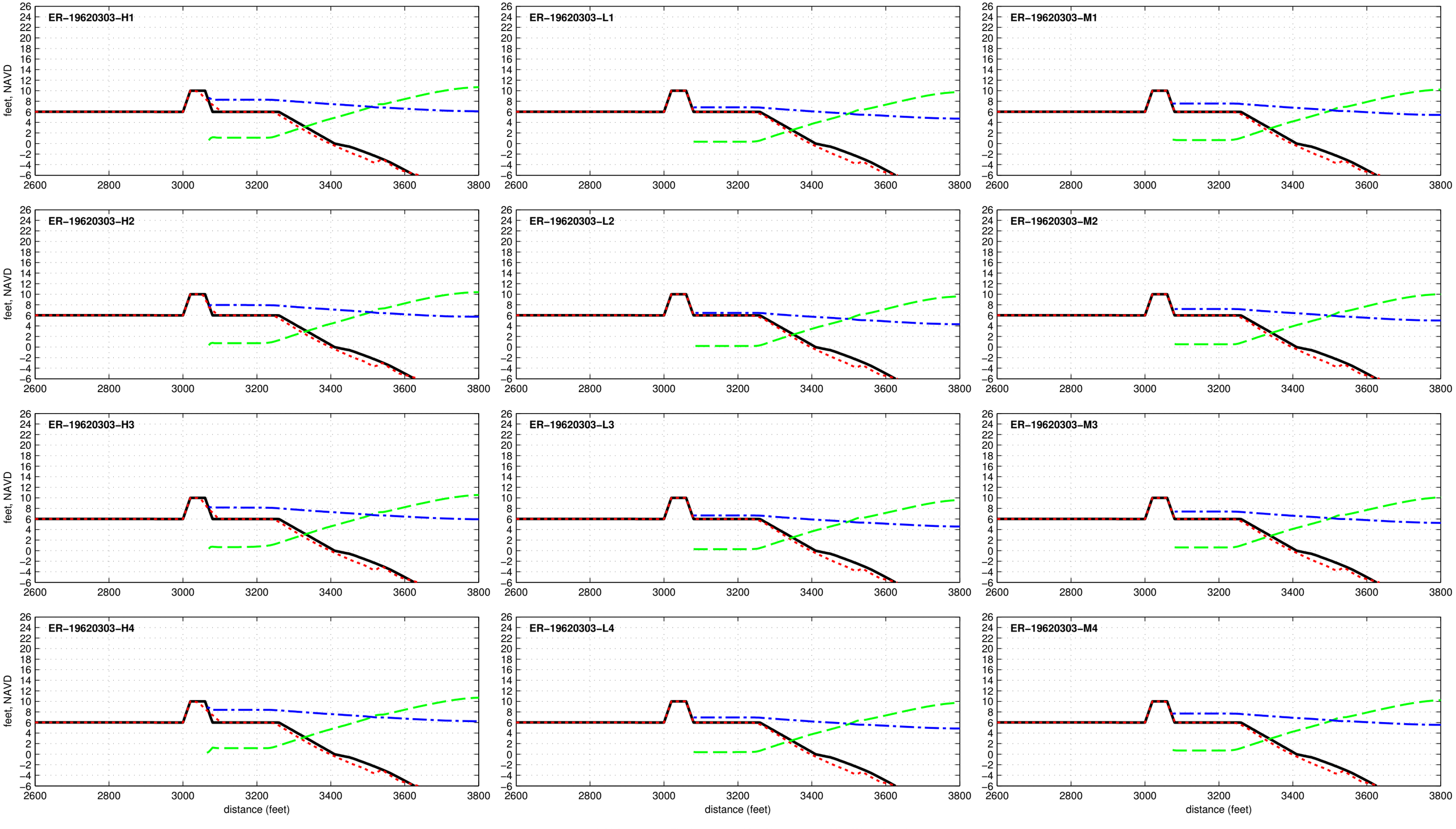

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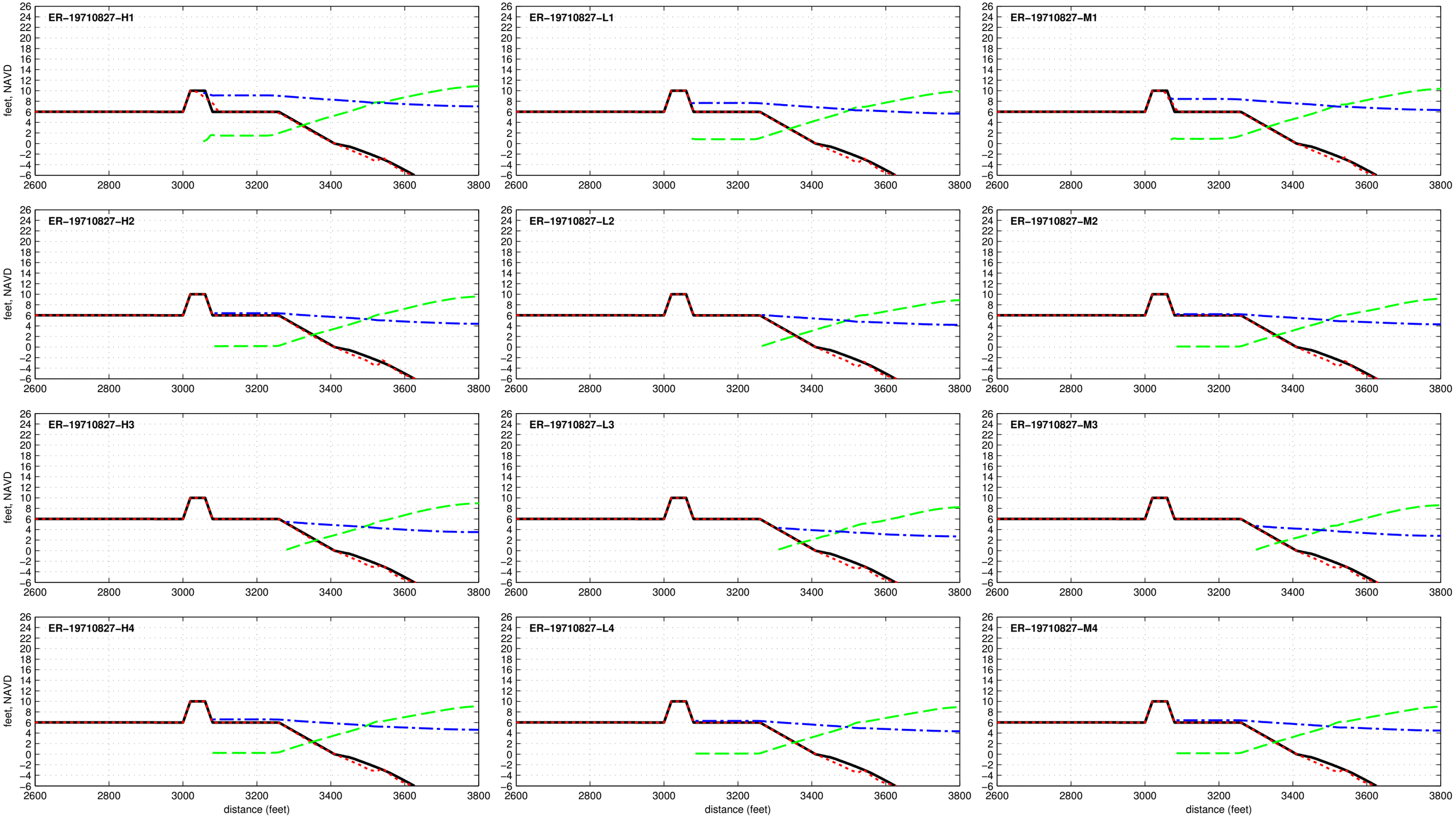

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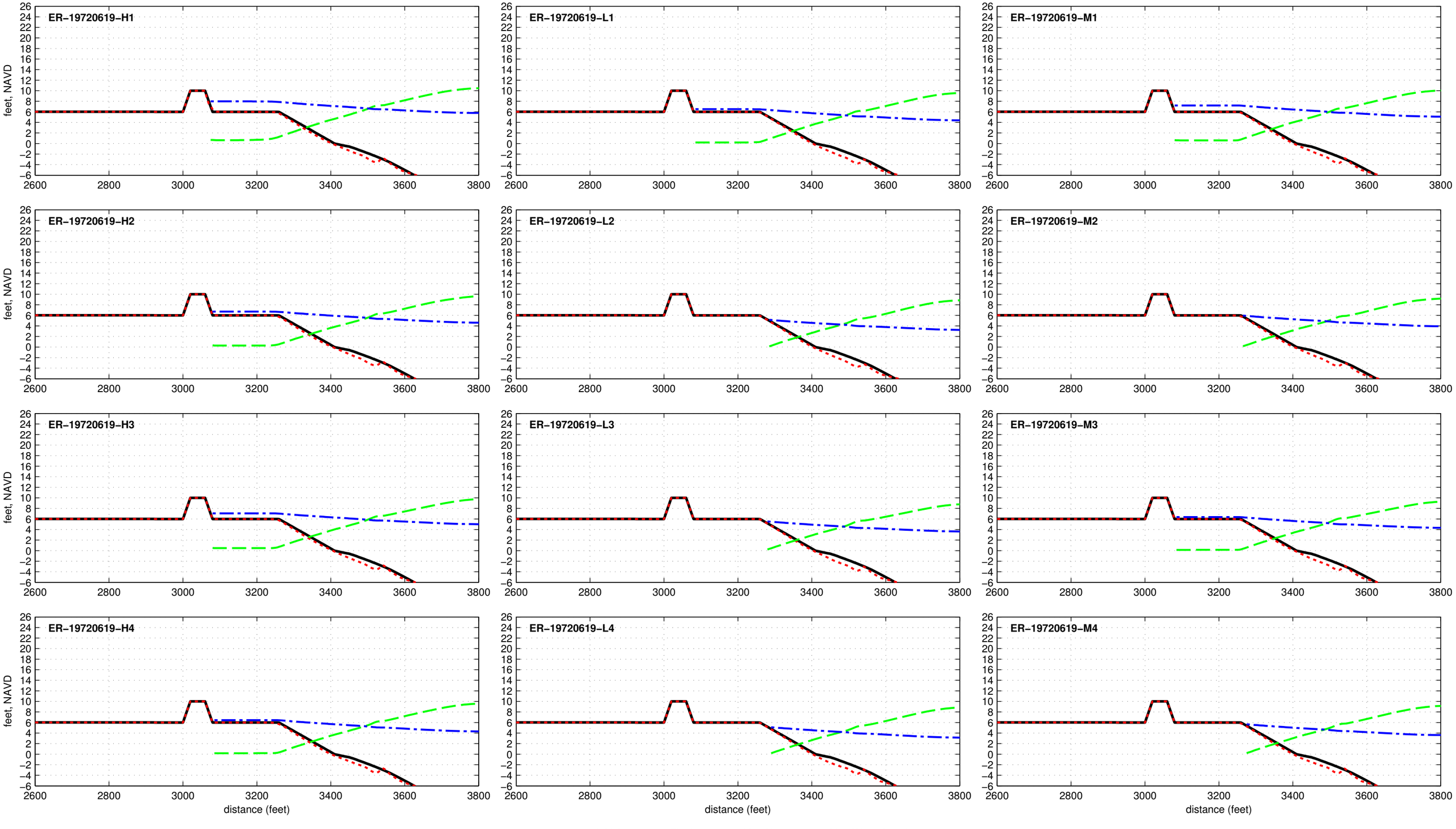

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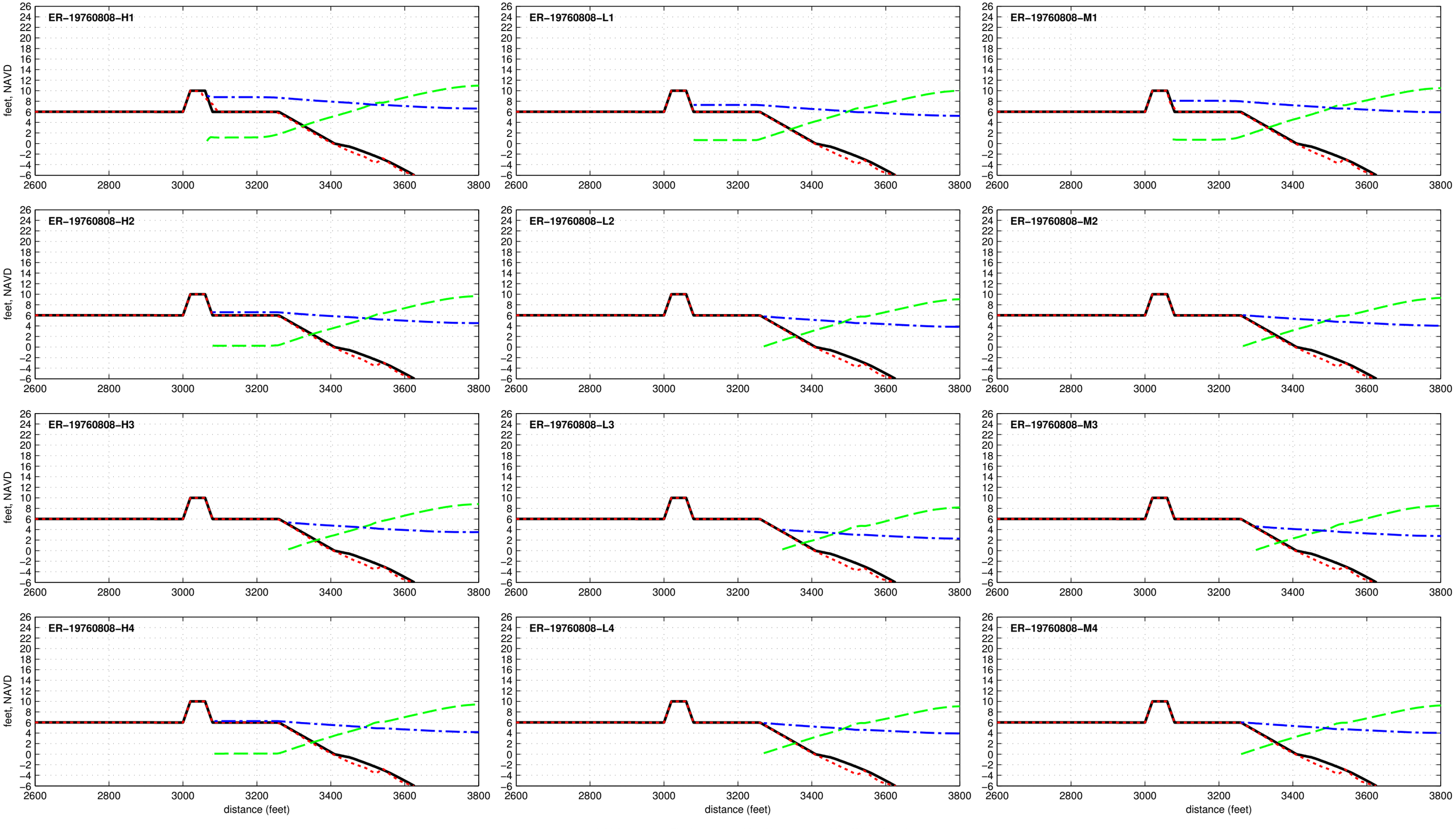

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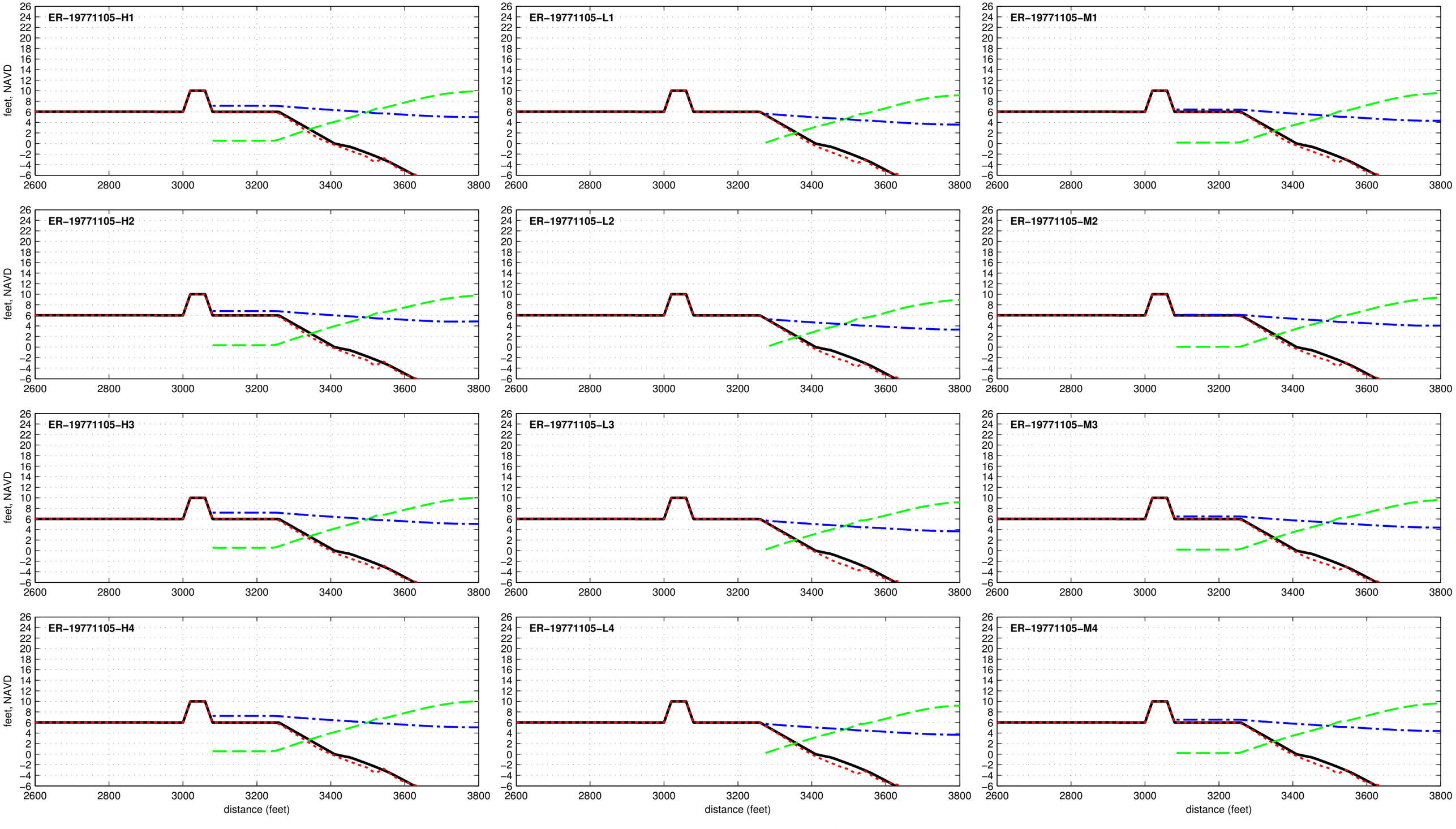
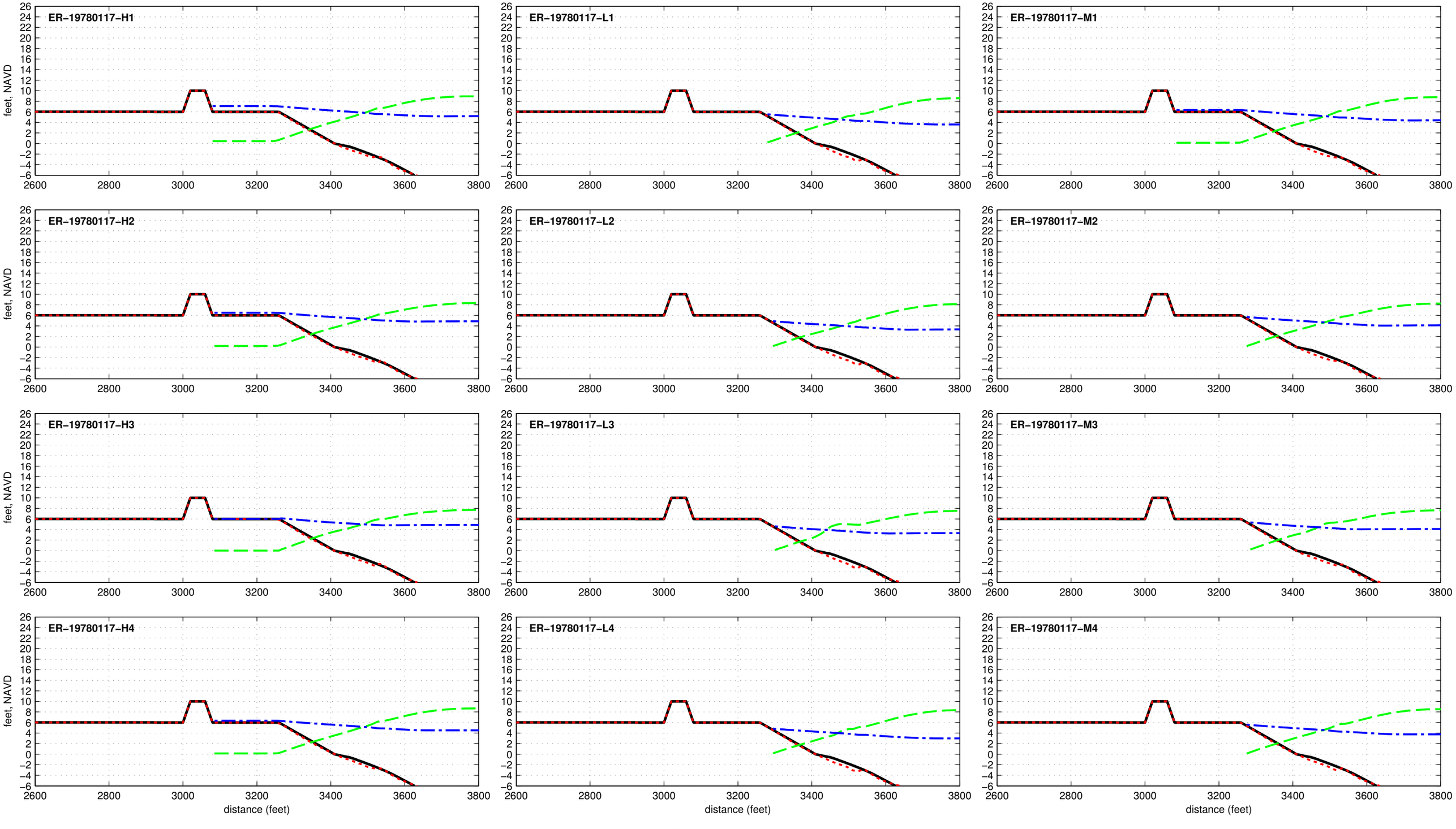


Dune R1T1–10.0–040–180 – ER–19780117



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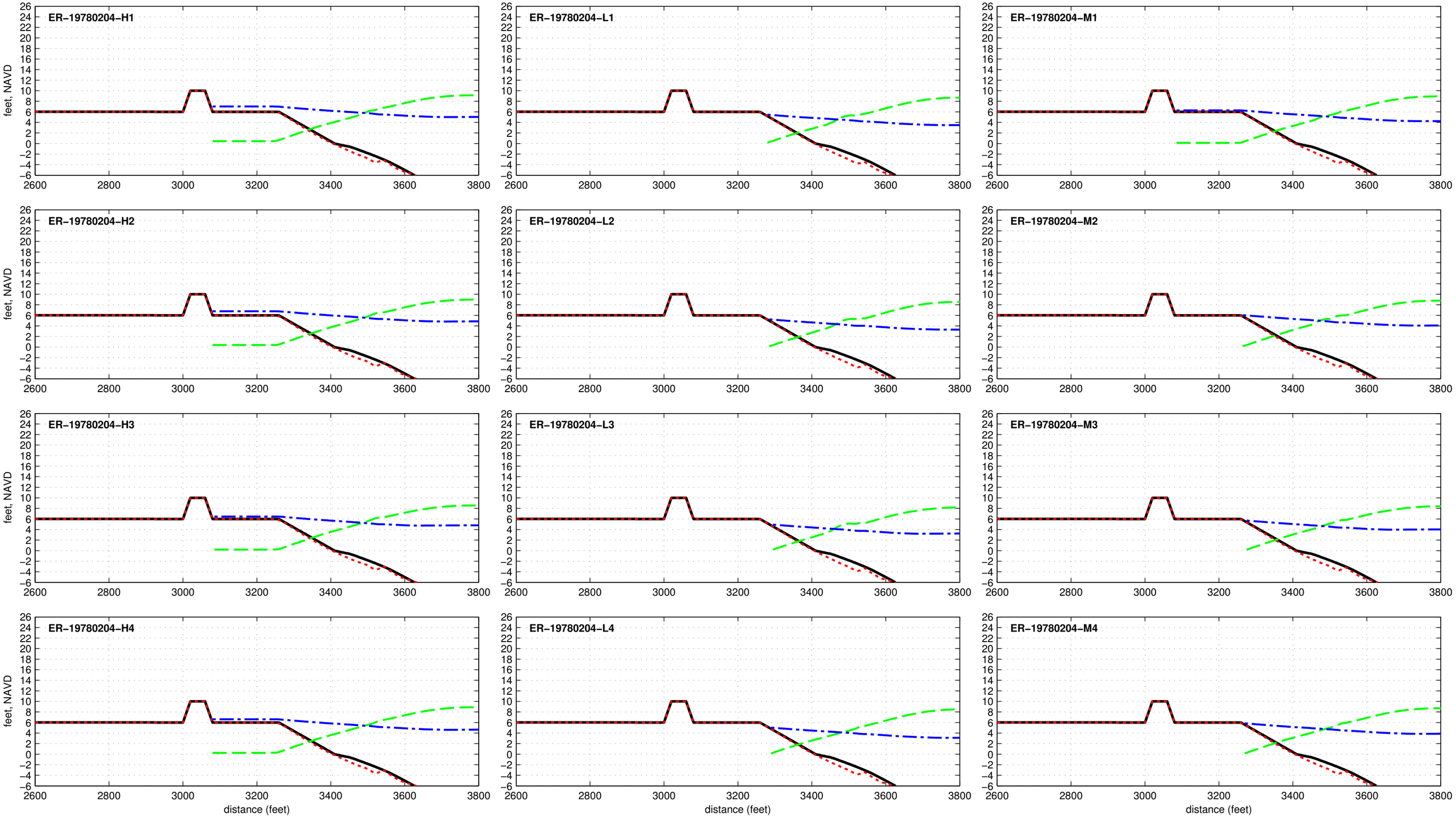

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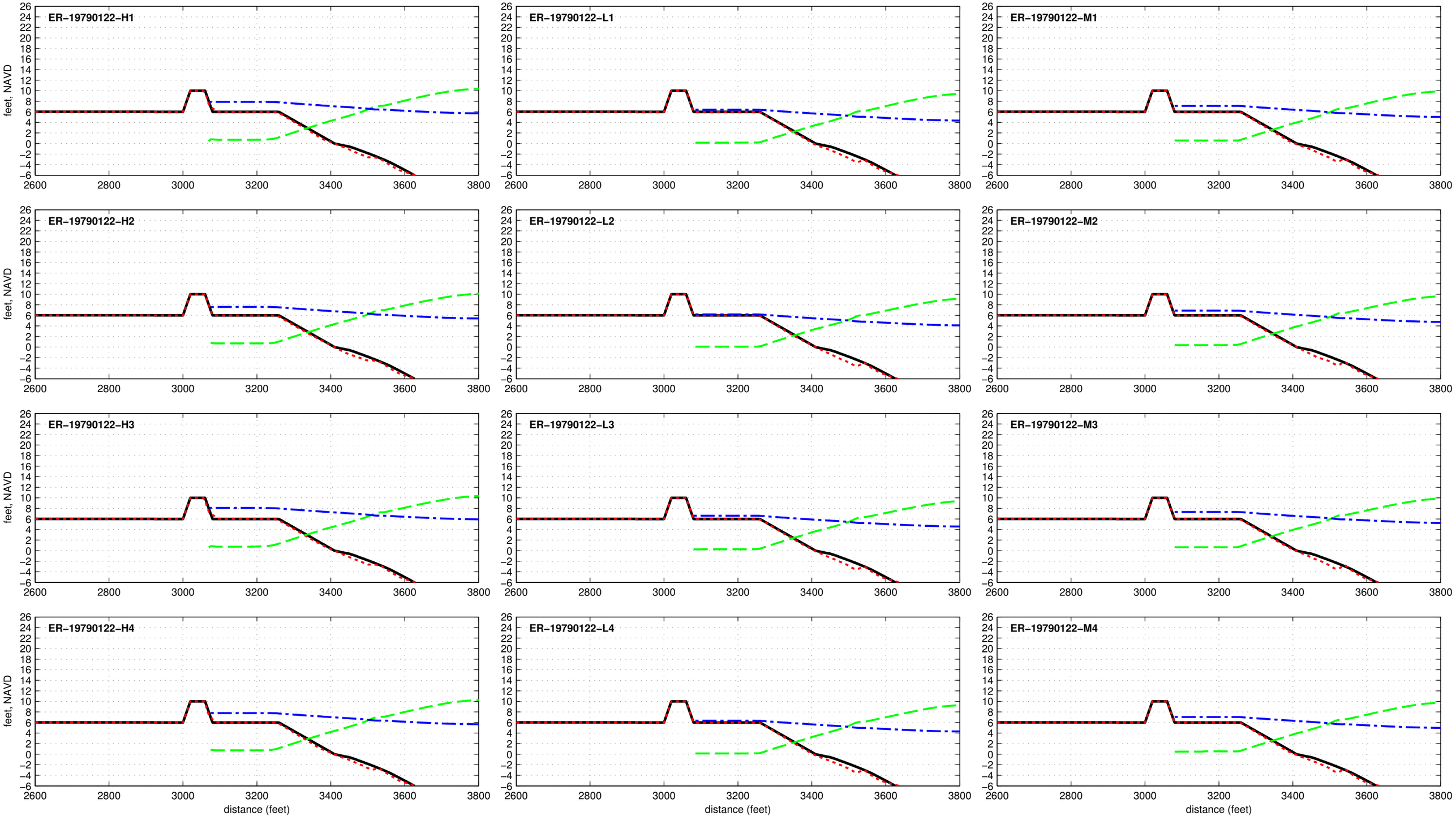
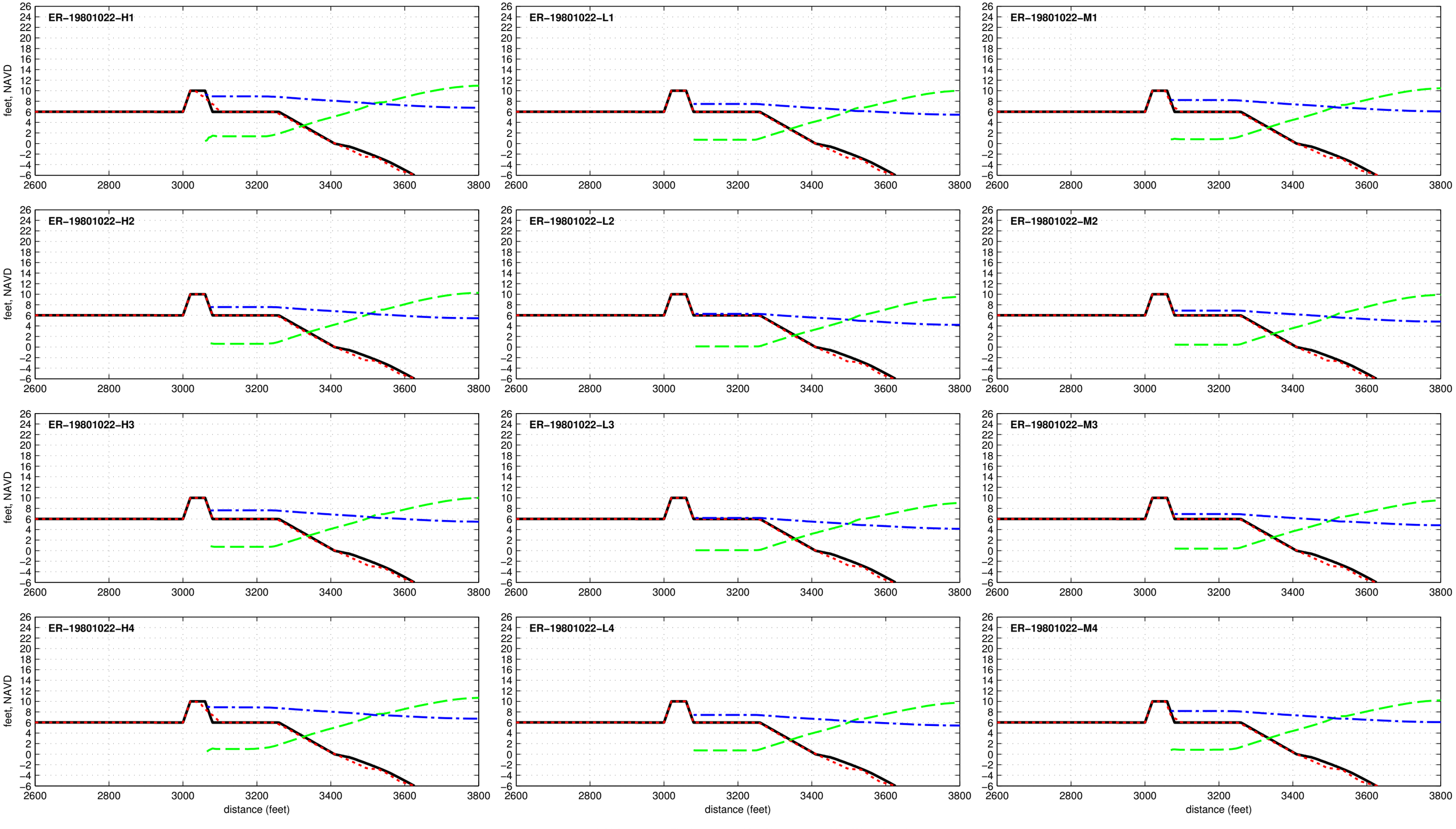
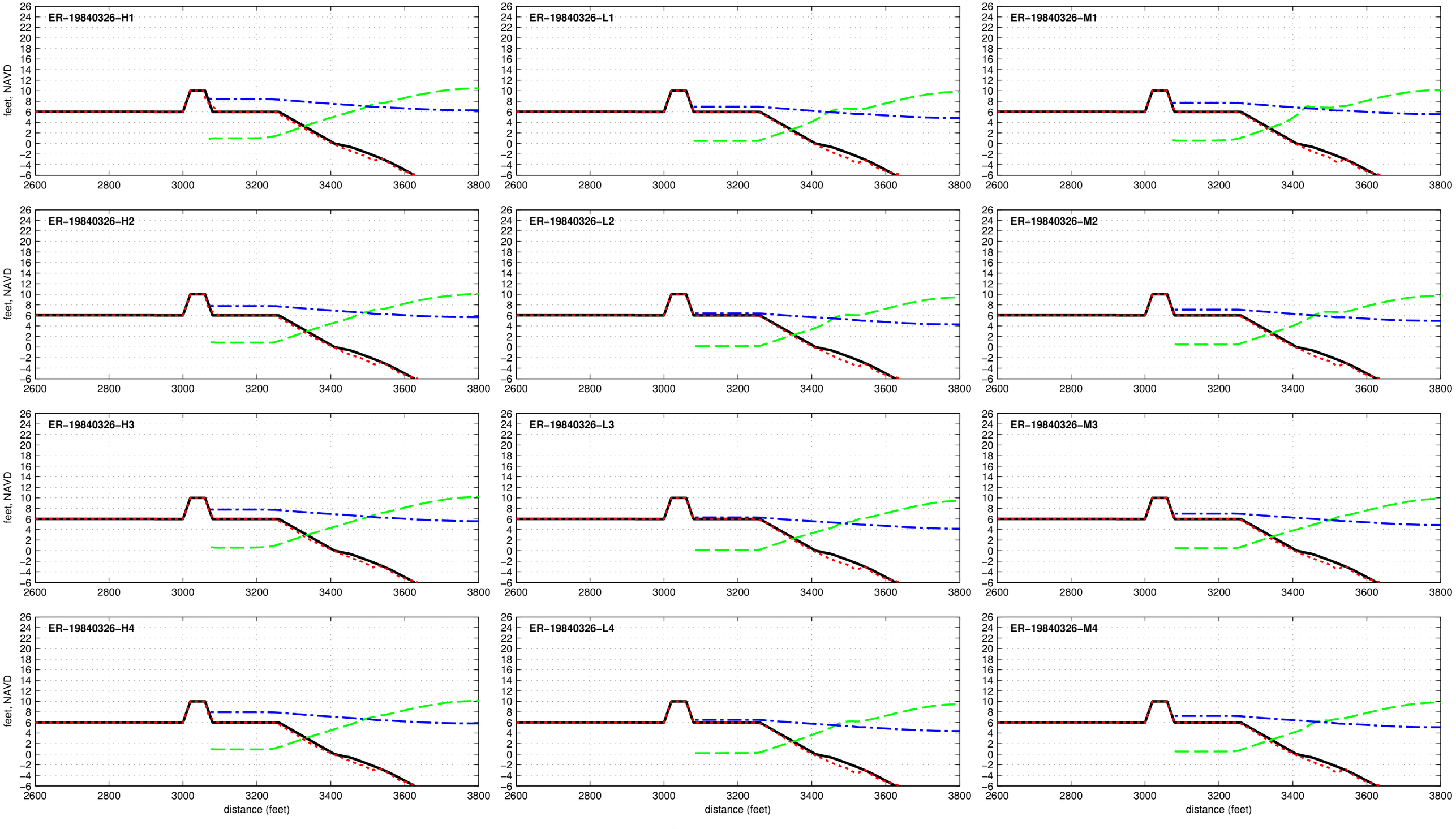


Dune R1T1–10.0–040–180 – ER–19801022



Dune R1T1–10.0–040–180 – ER–19840326



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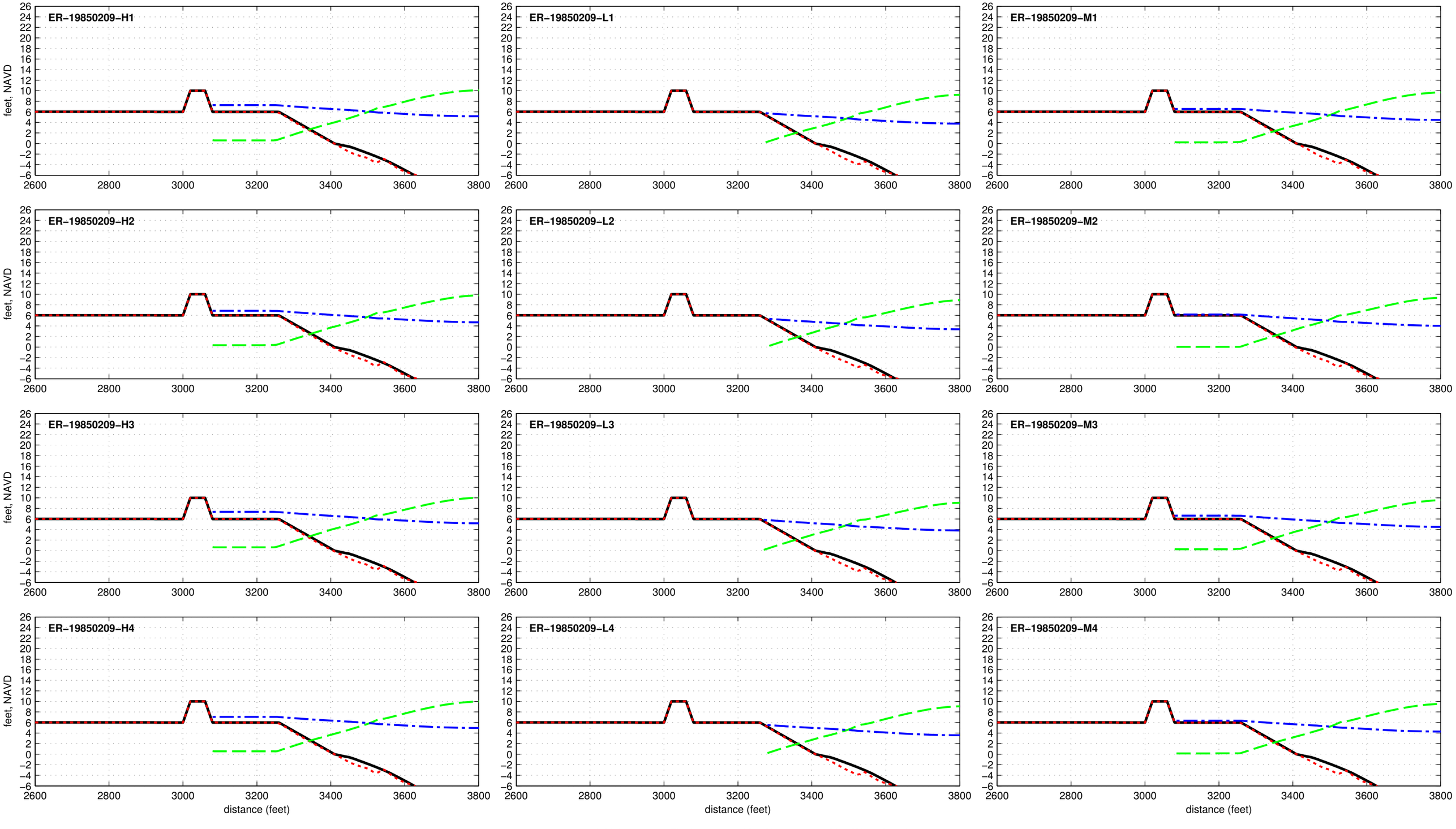

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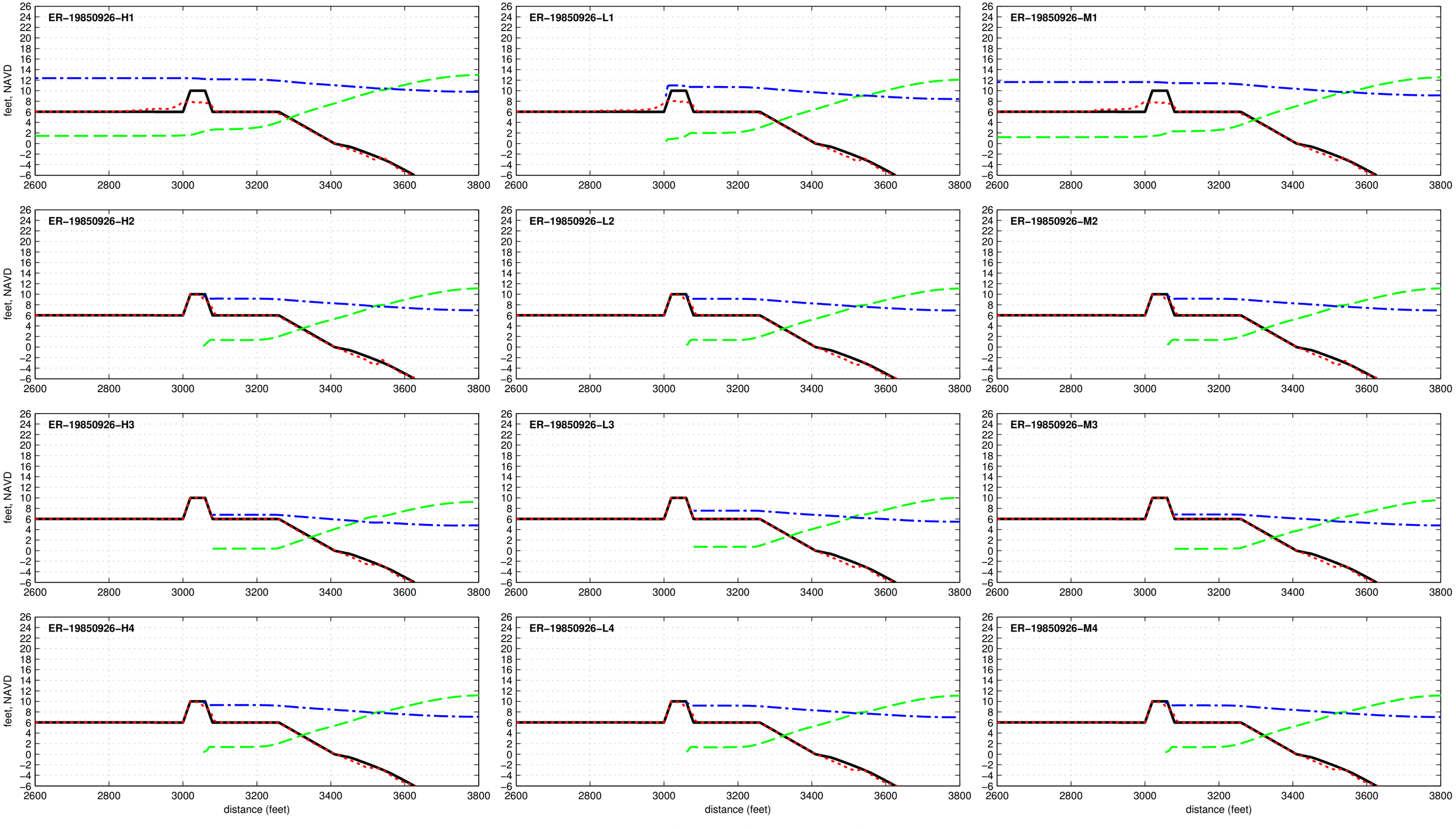

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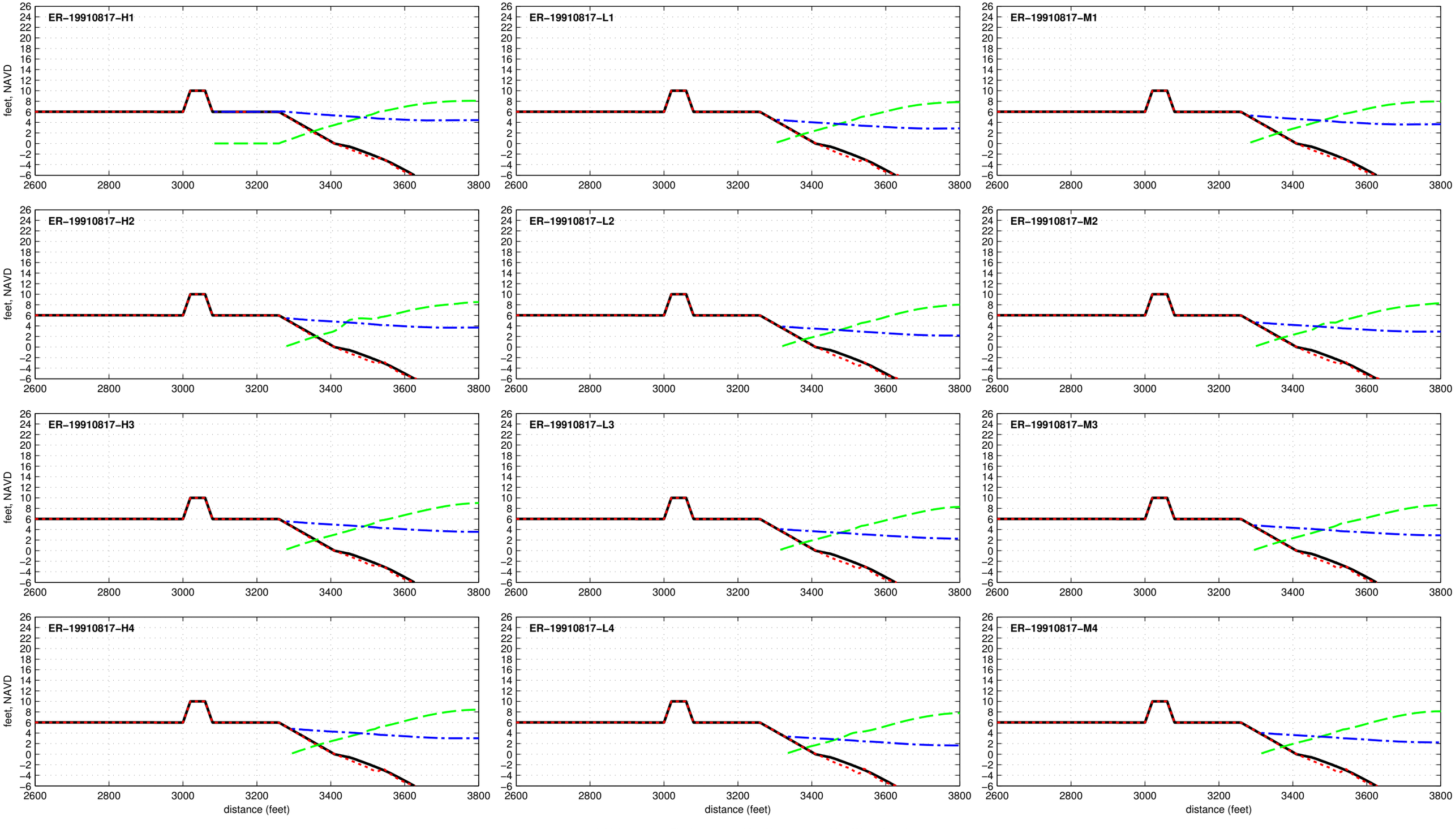

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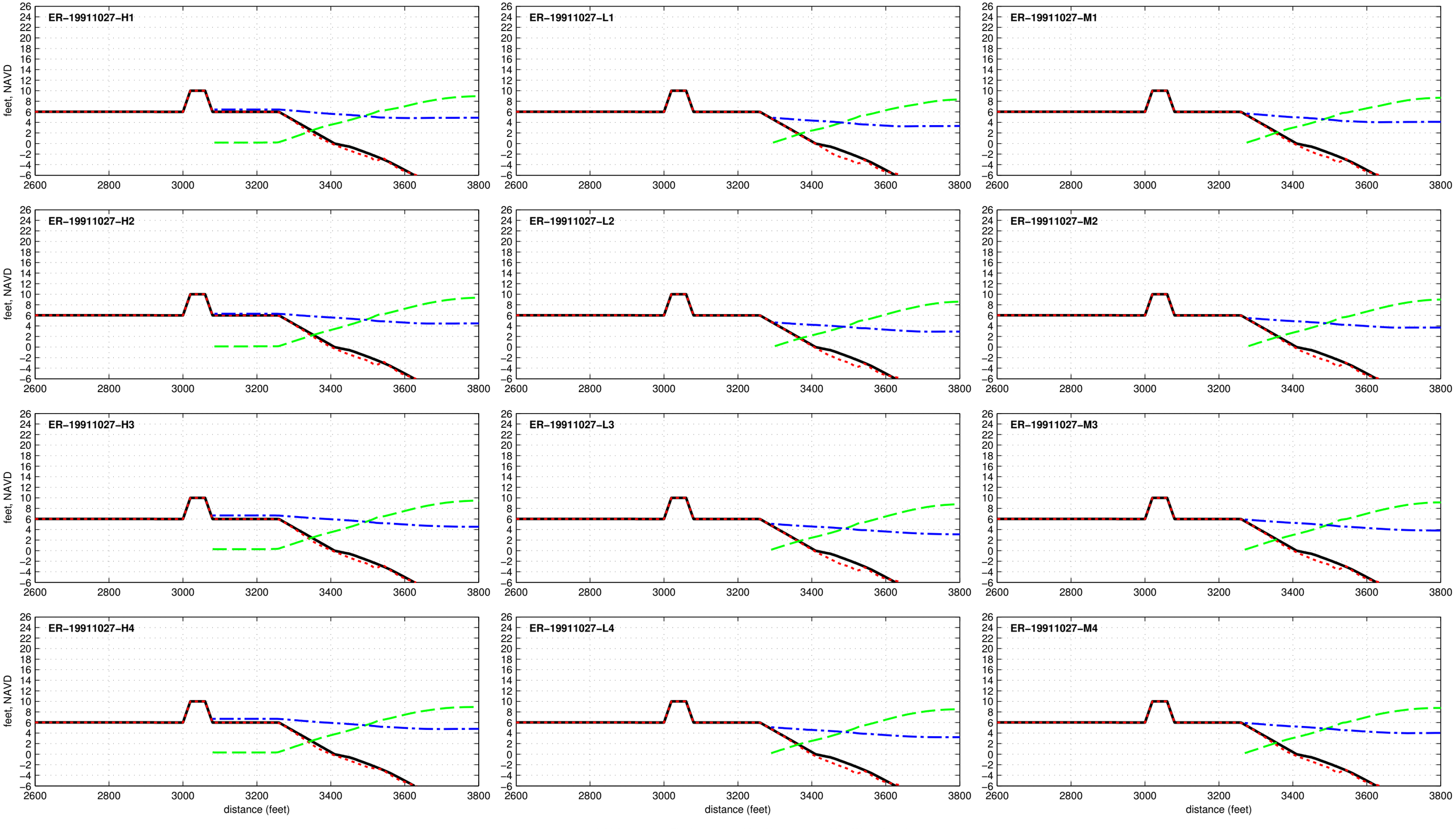

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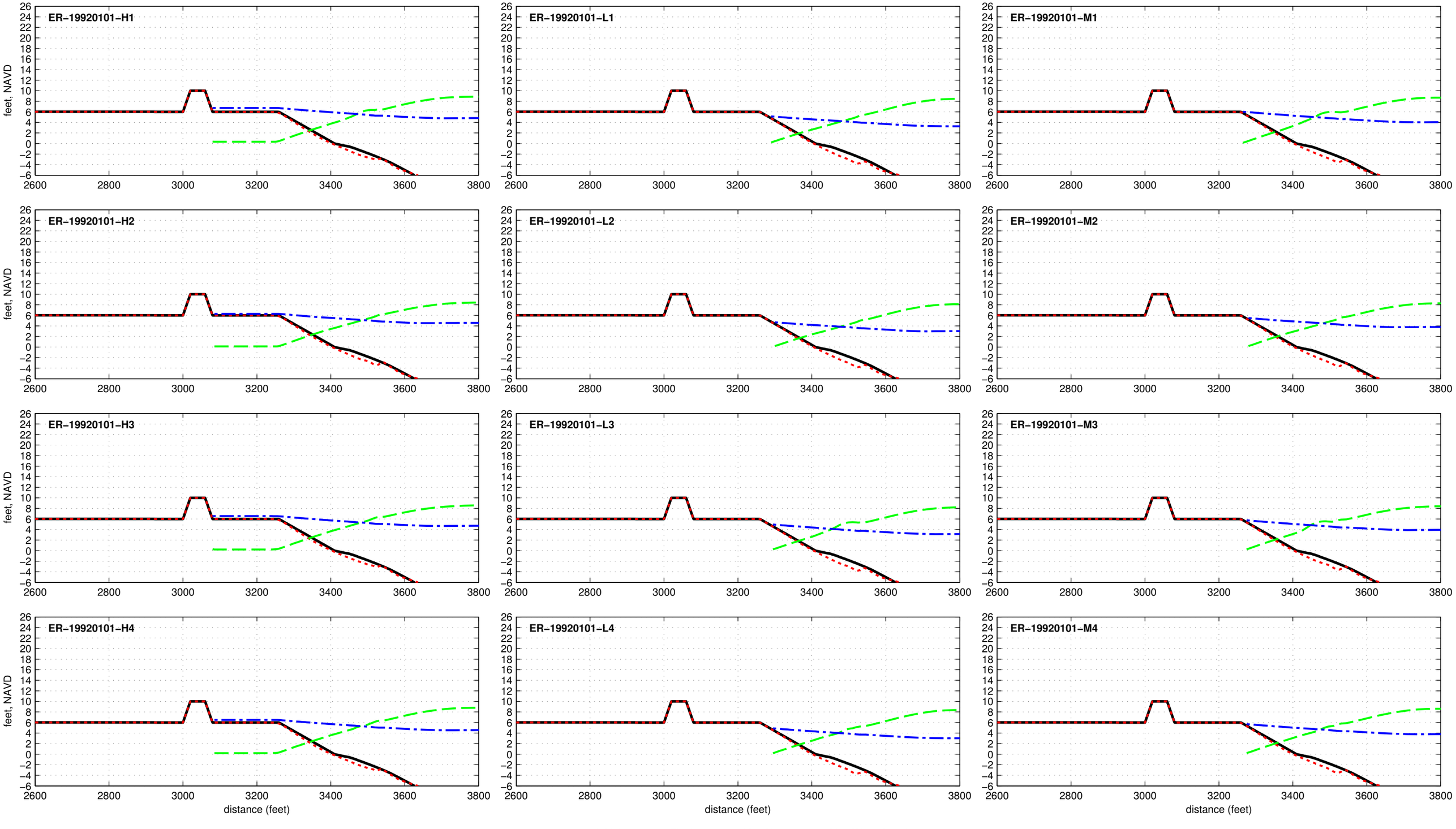

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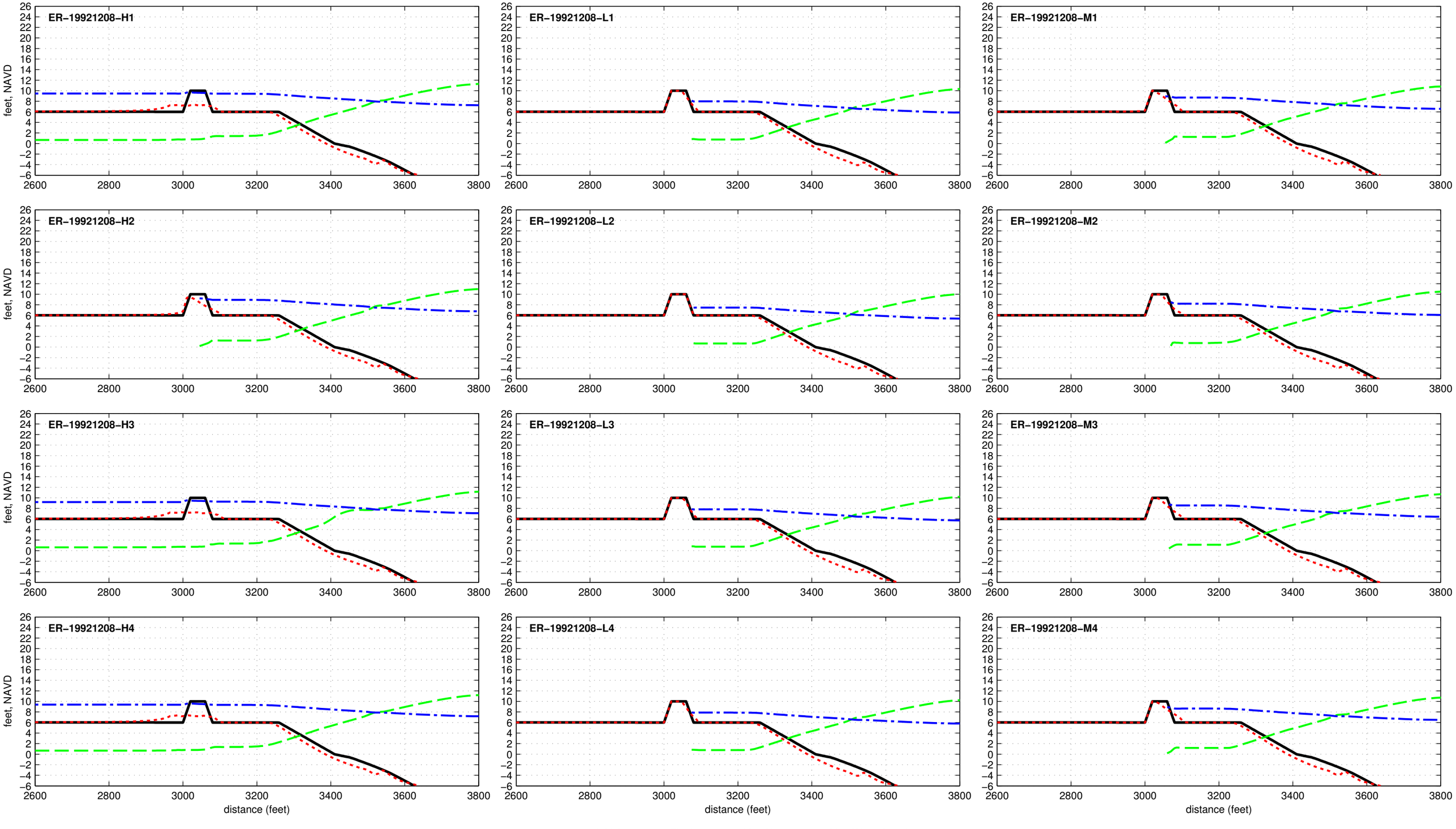

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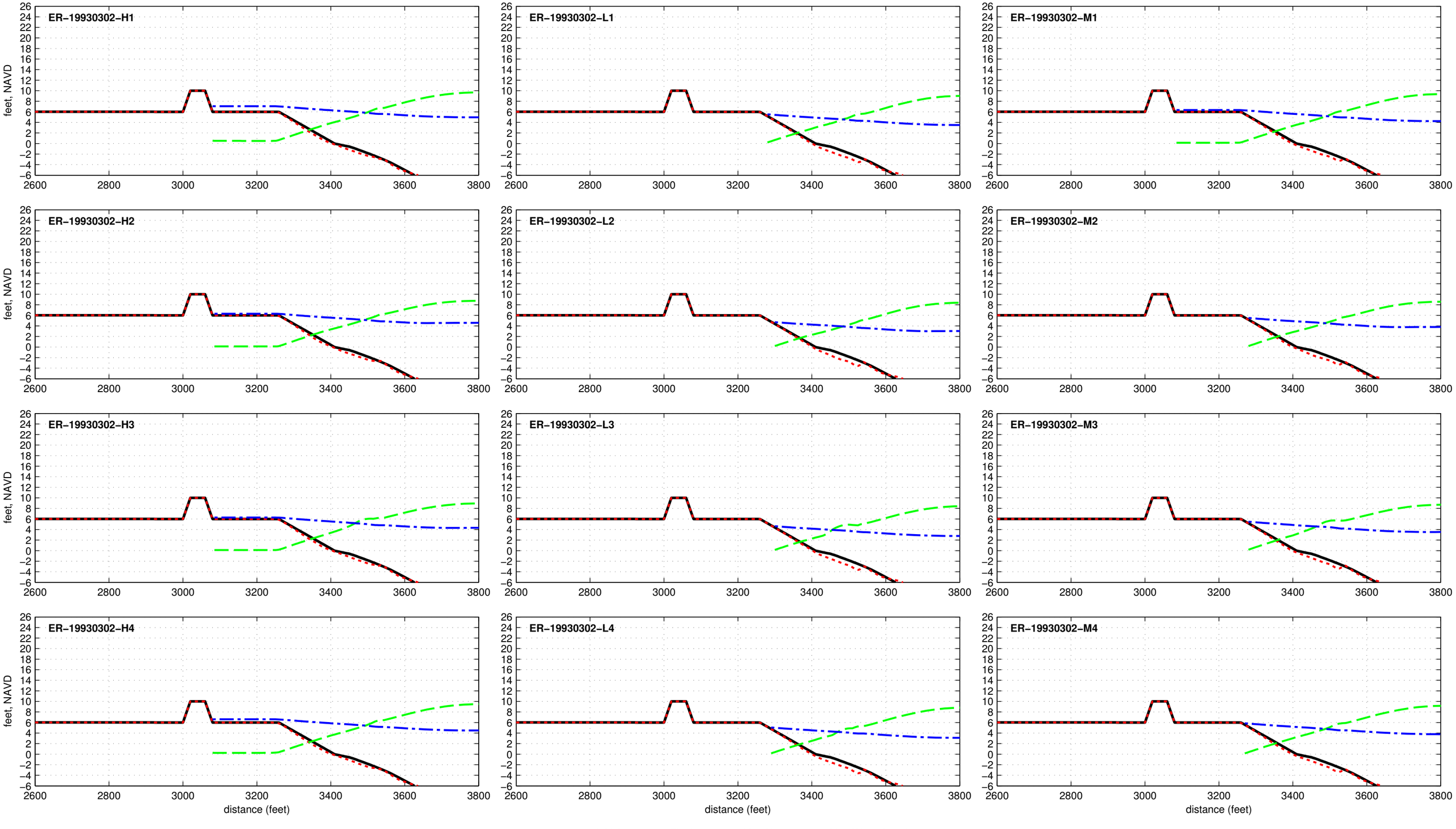

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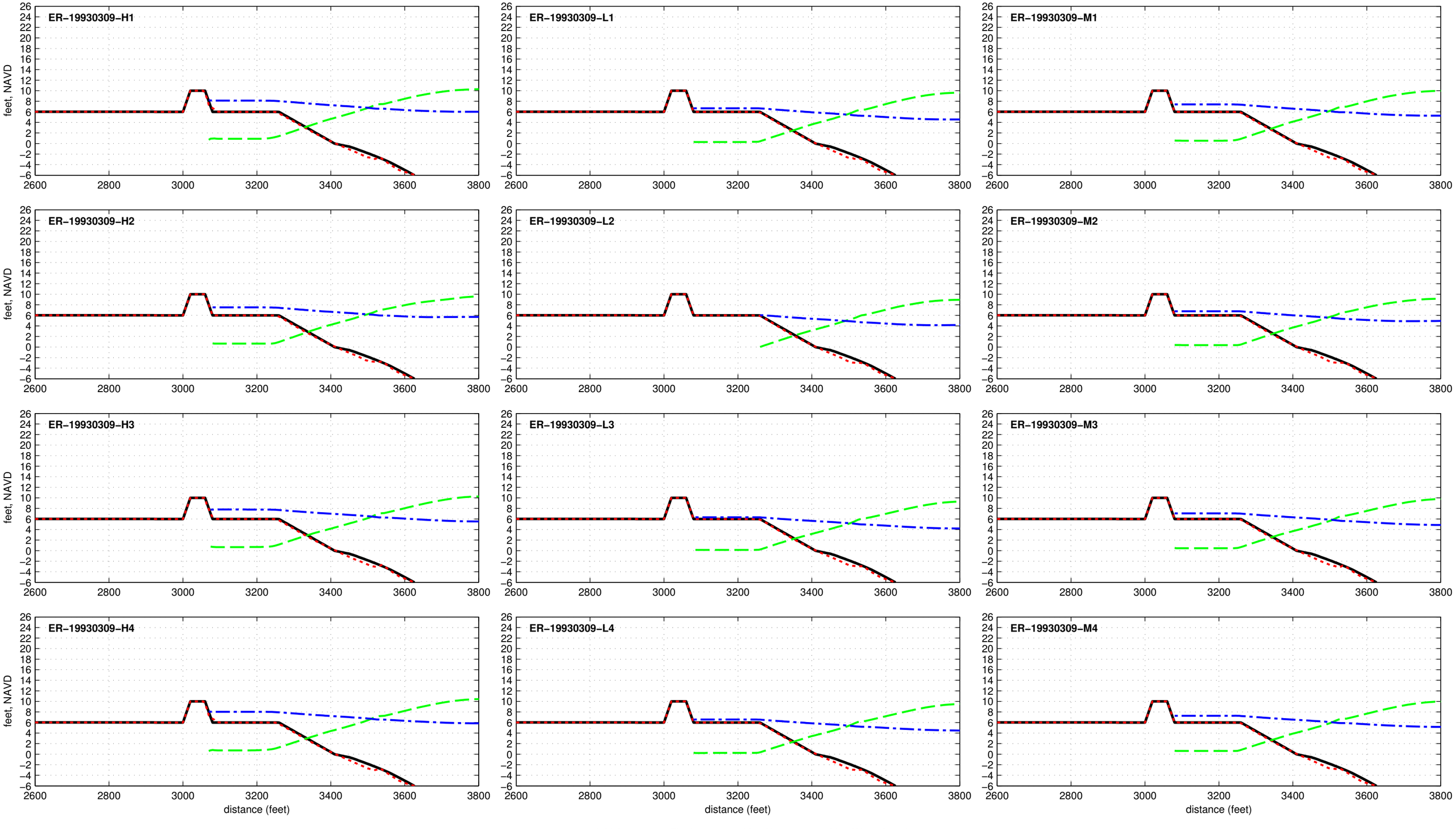

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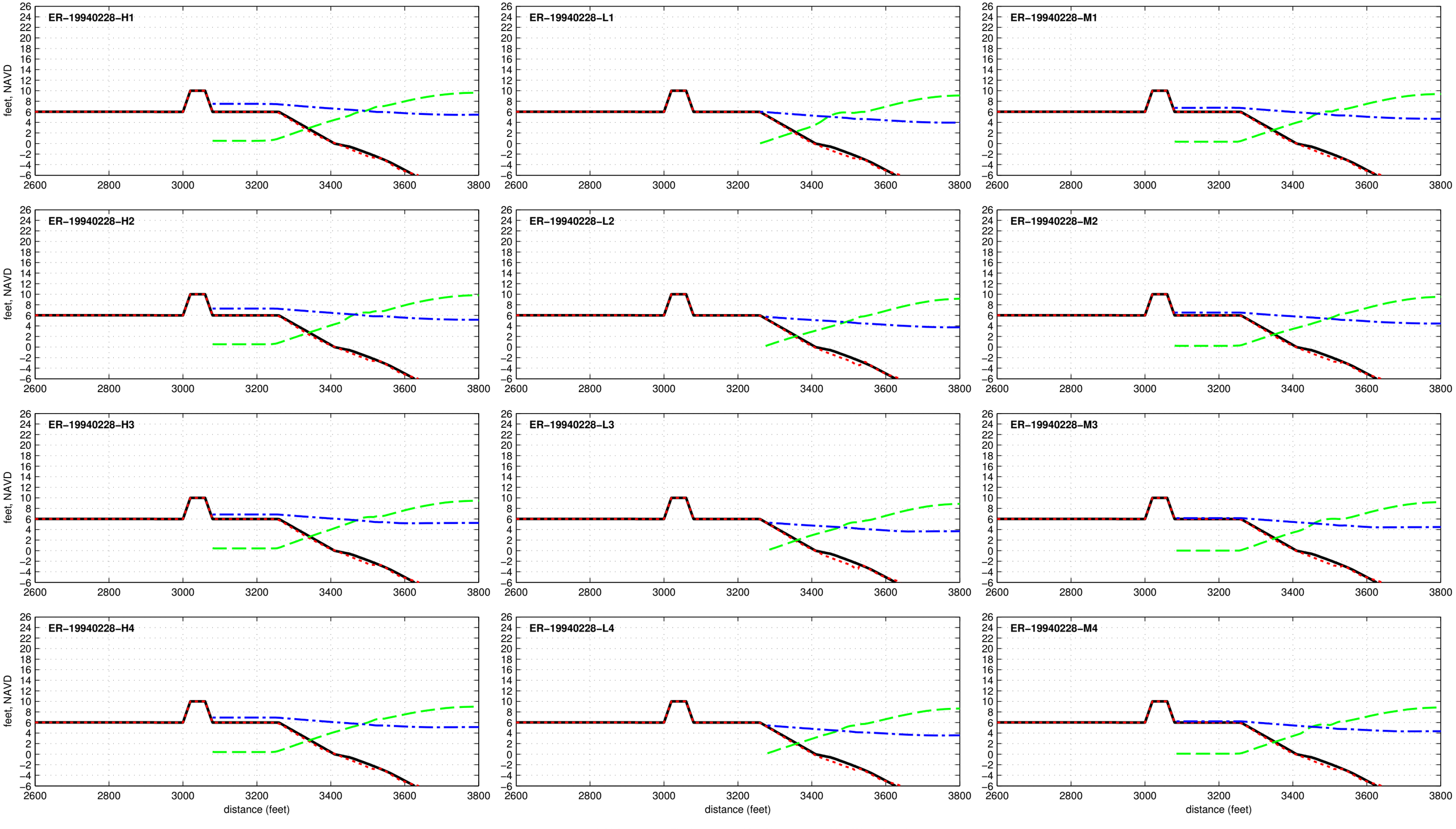

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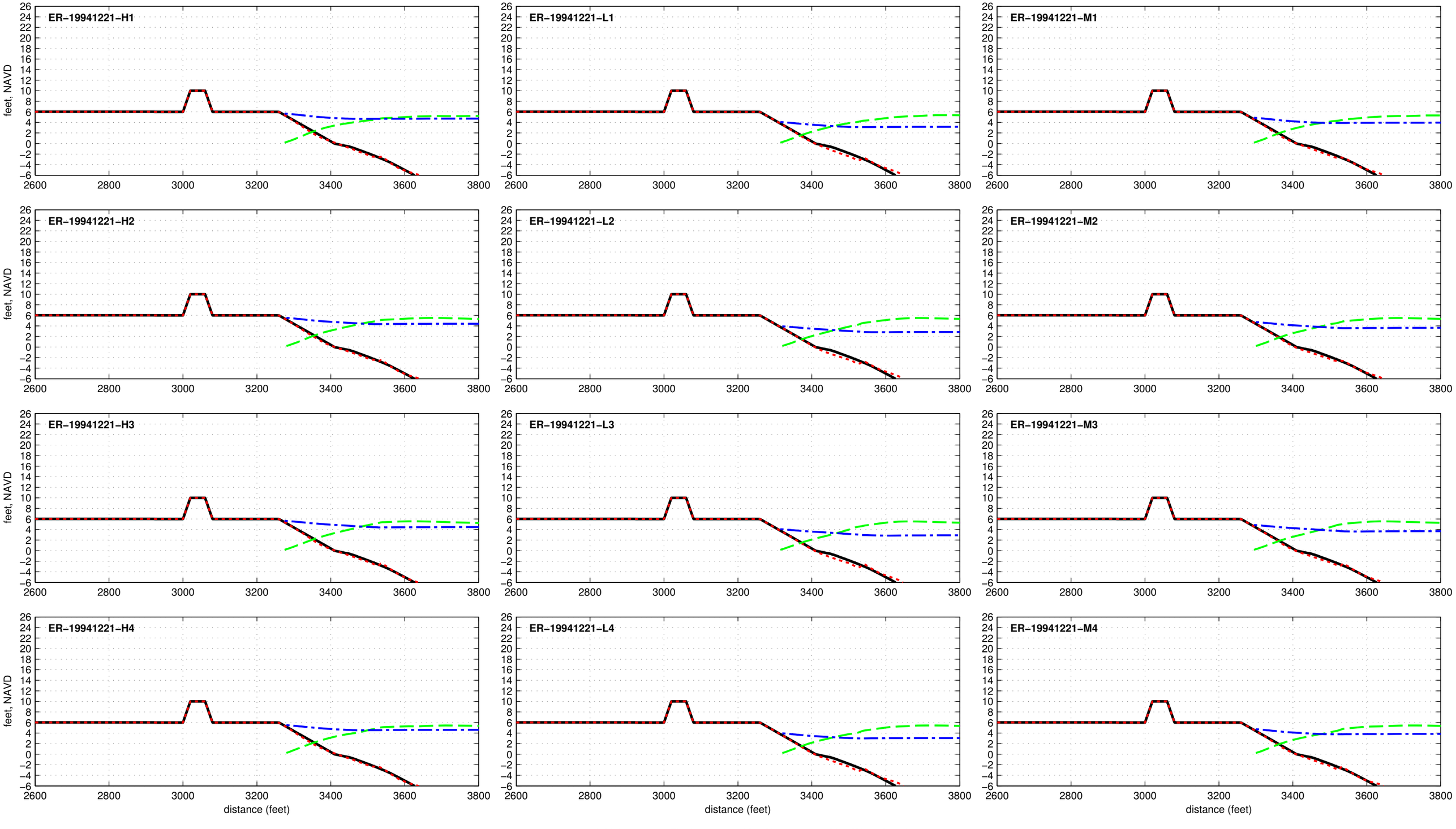

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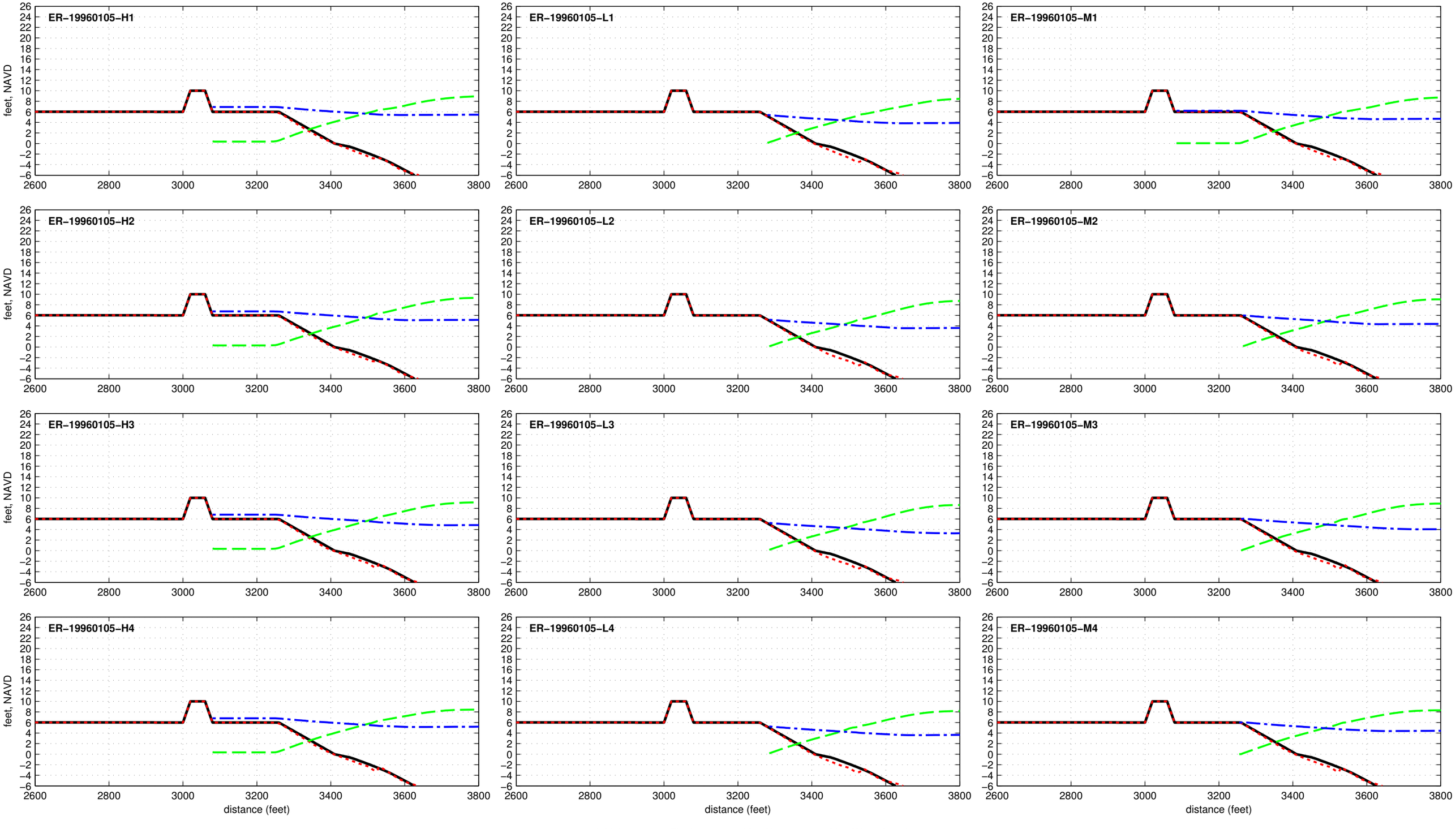

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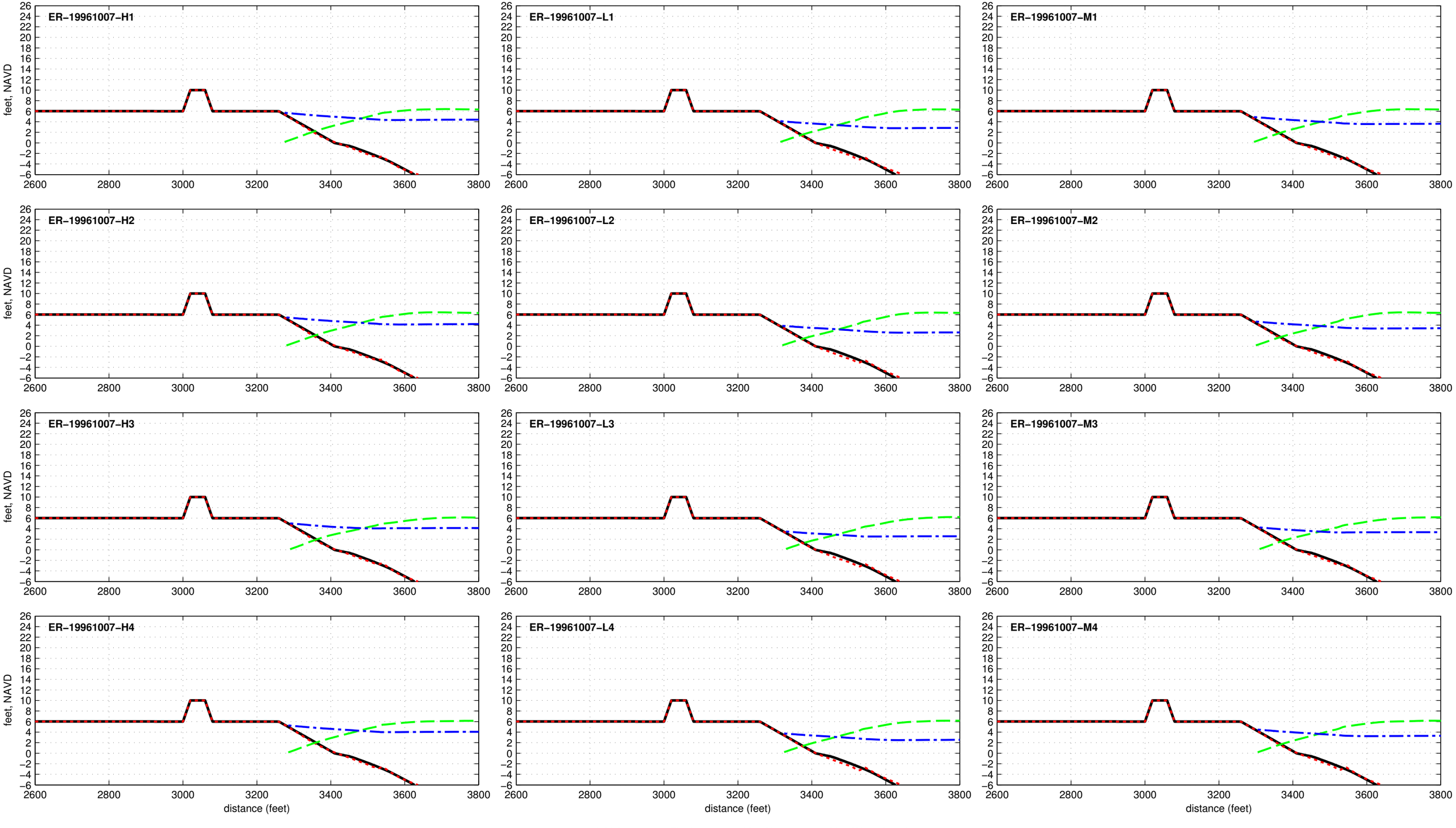

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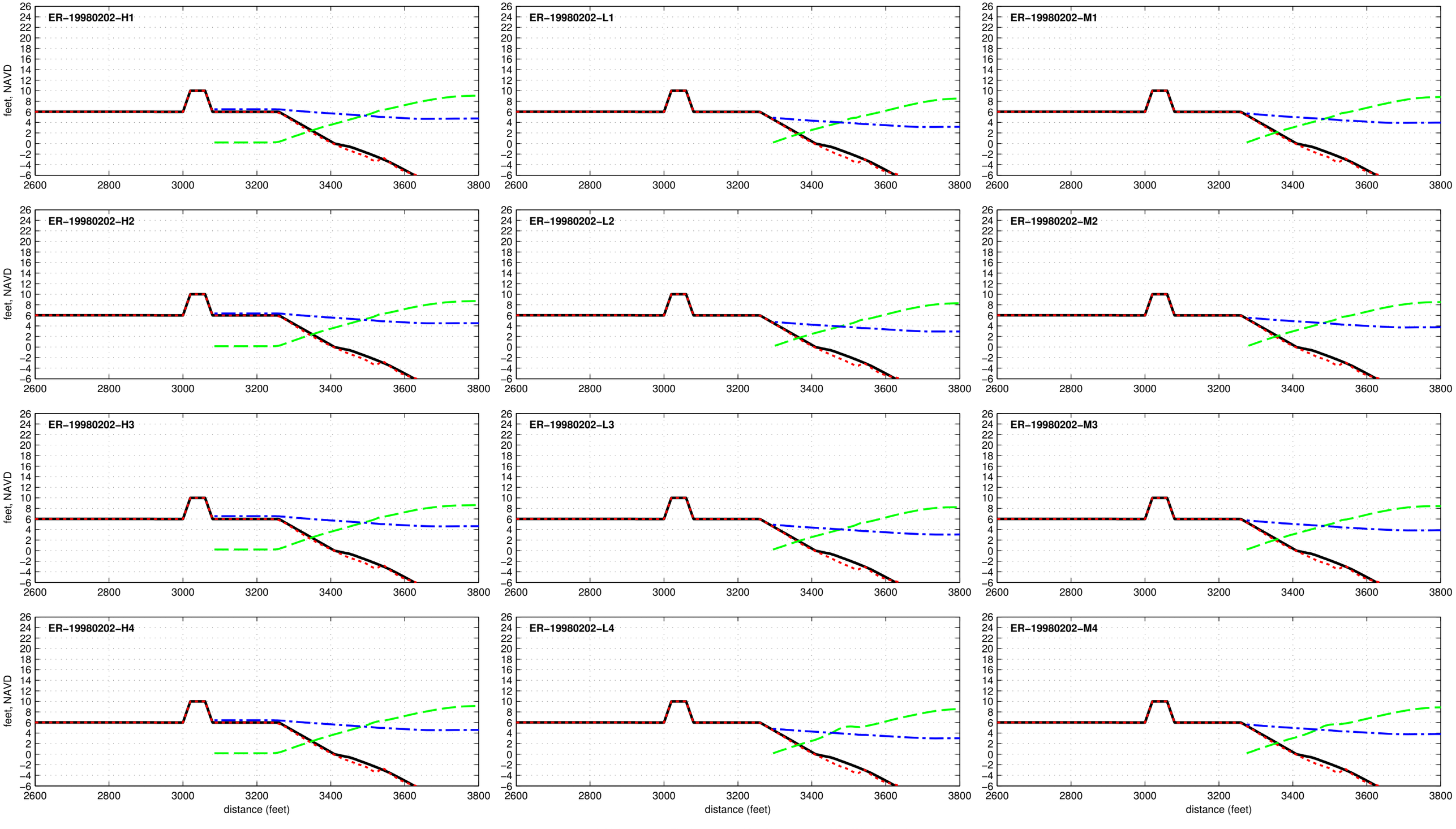

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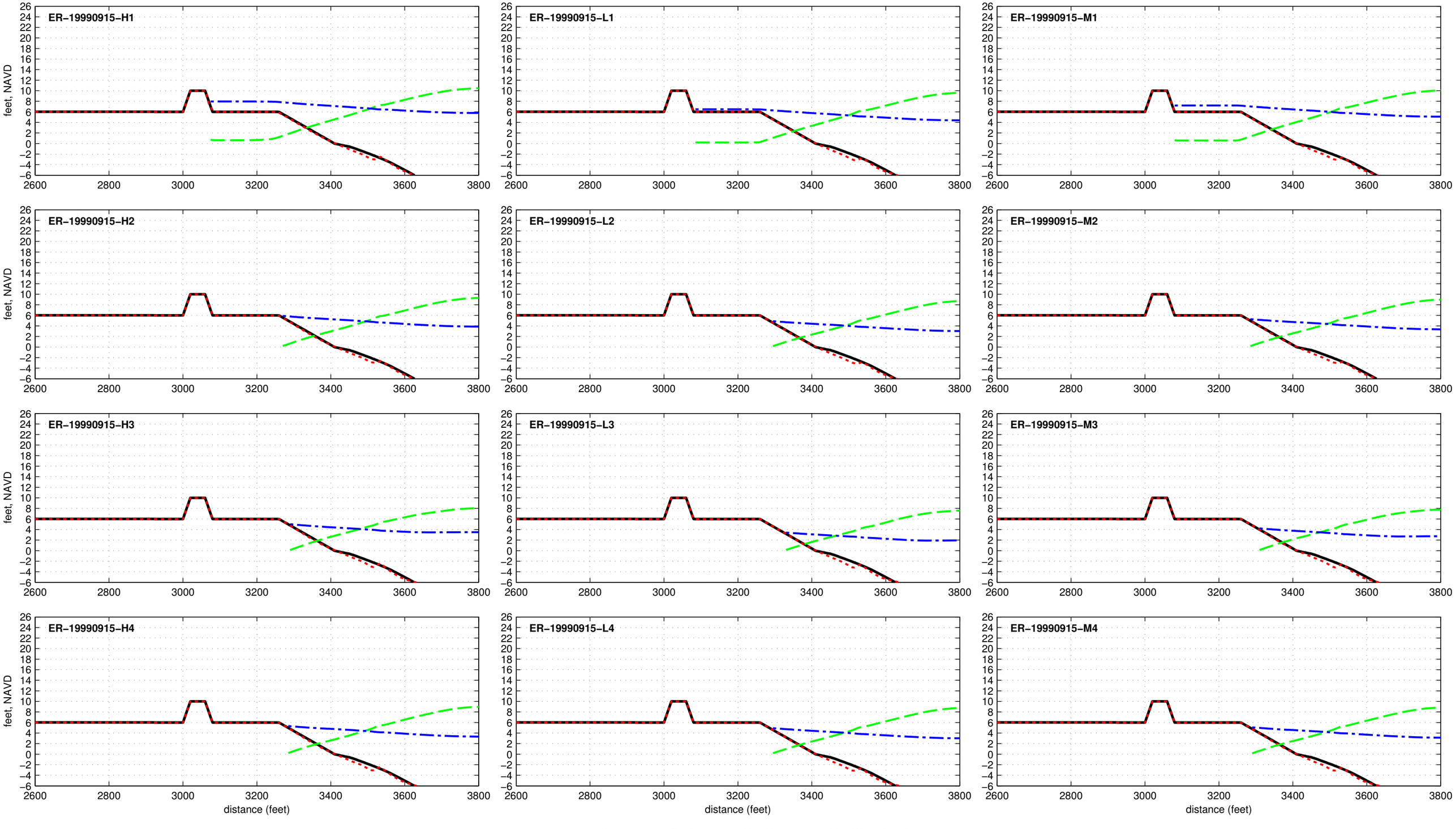

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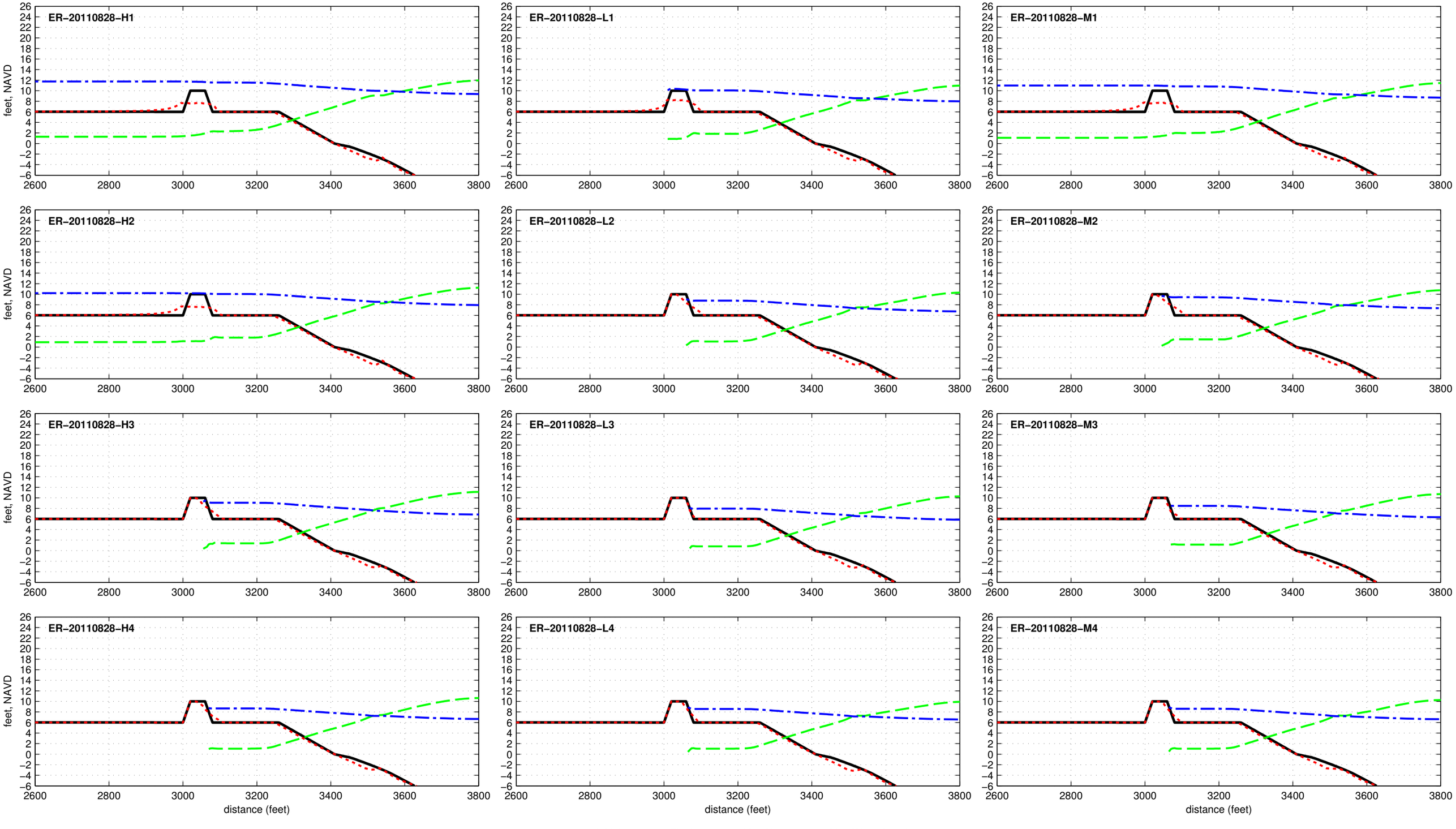

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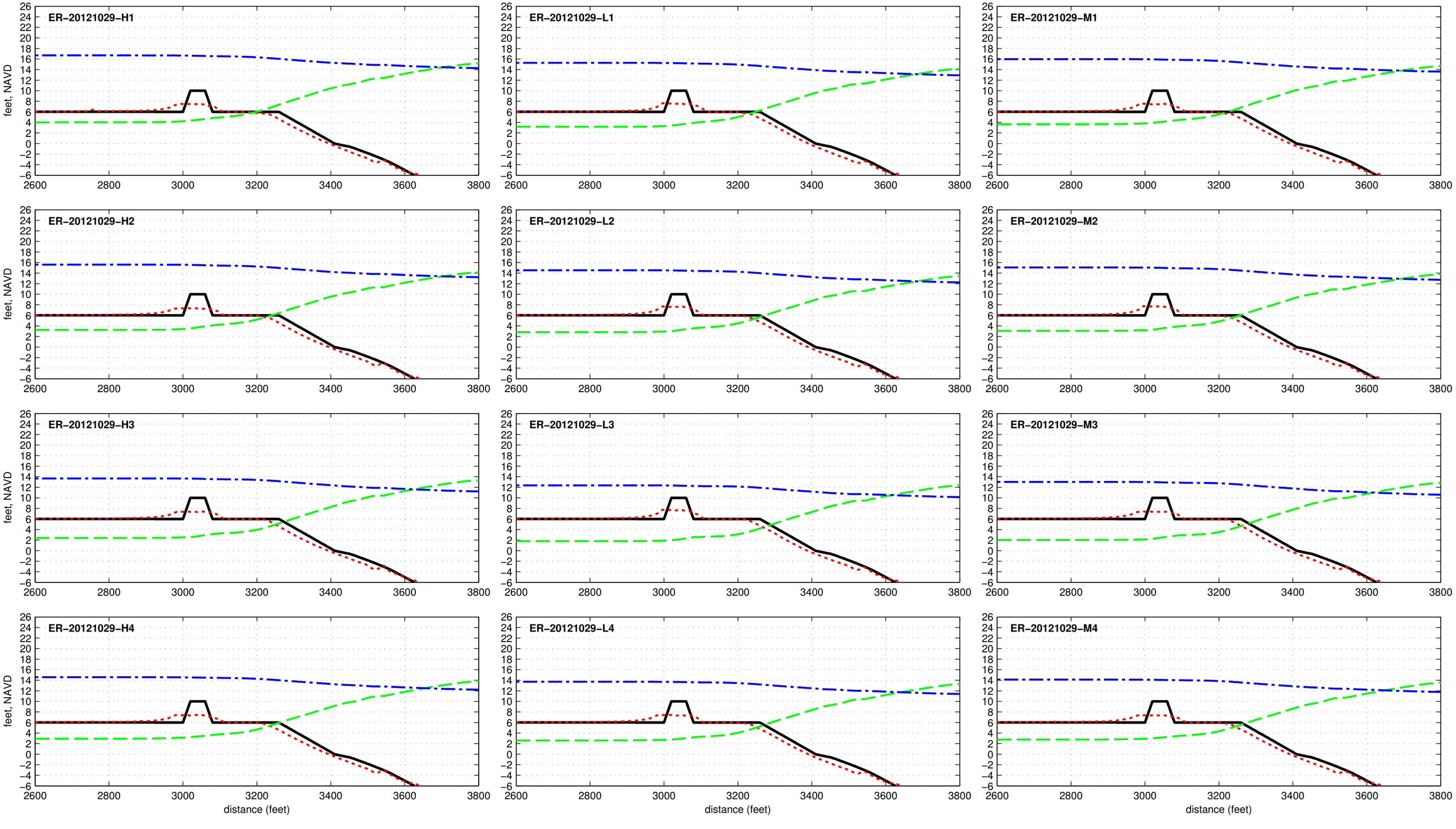
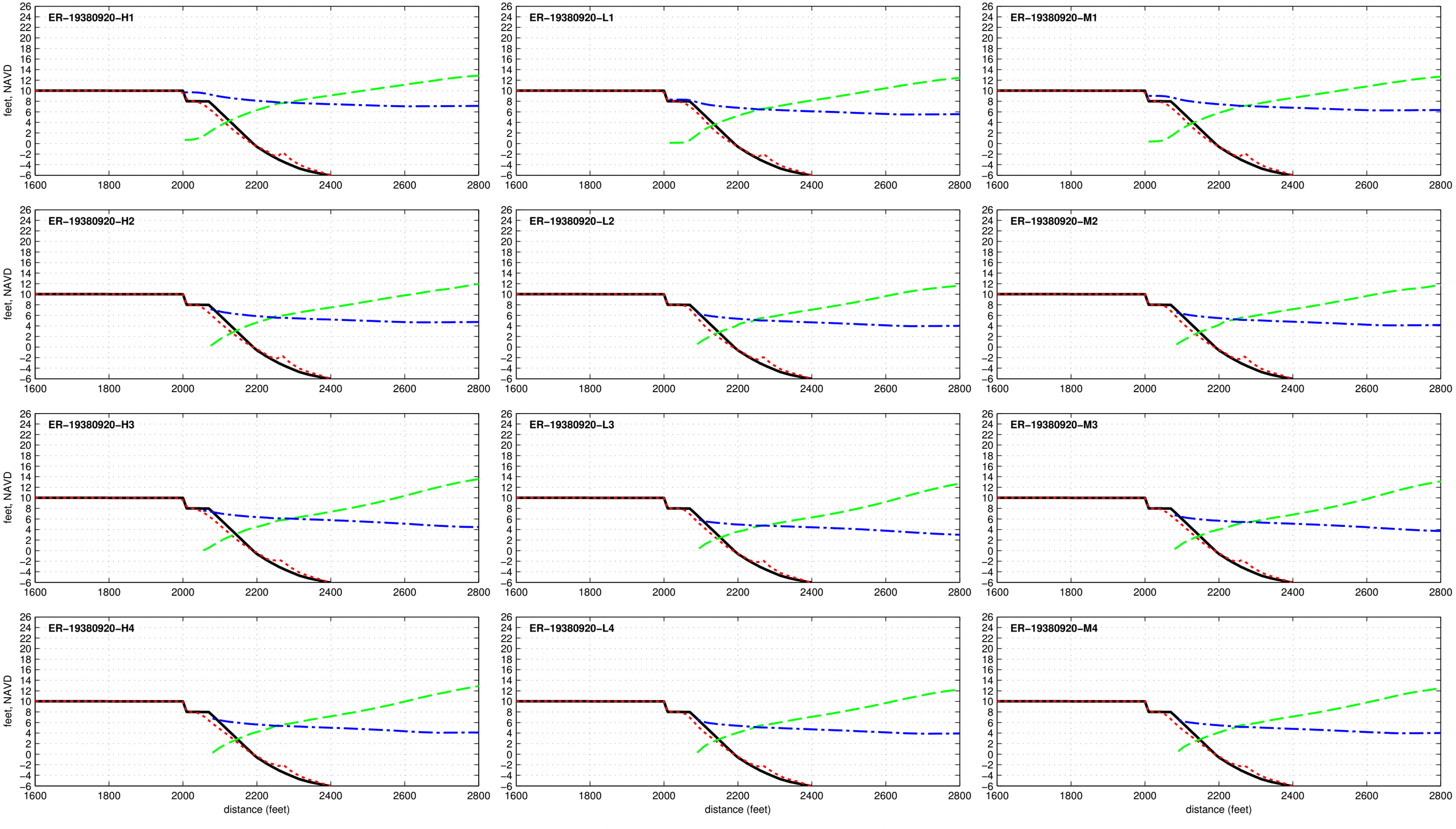


R2T1_10.0_000_060 - DUNE

Dune R2T1-10.0-000-060 - ER-19380920



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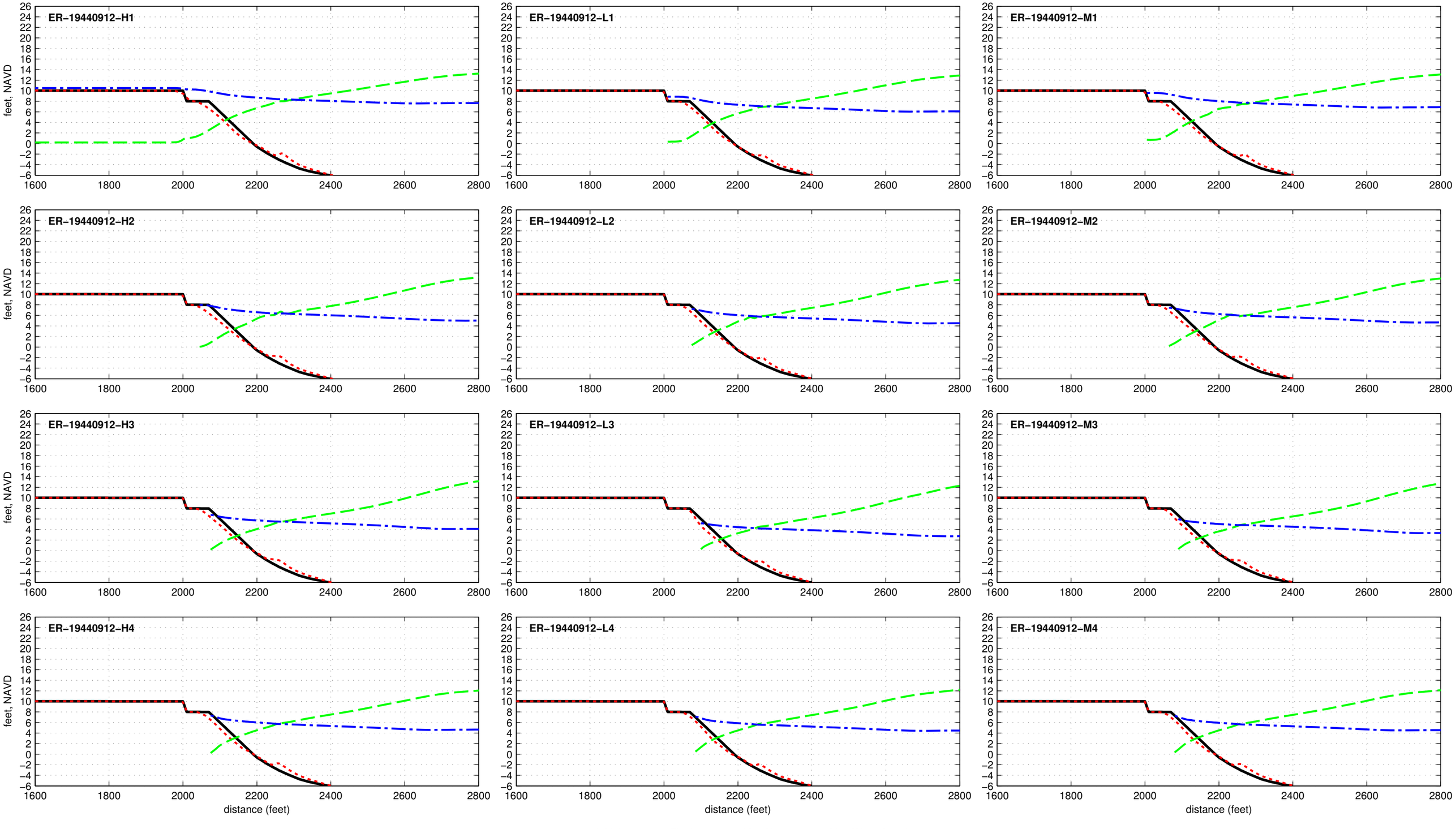

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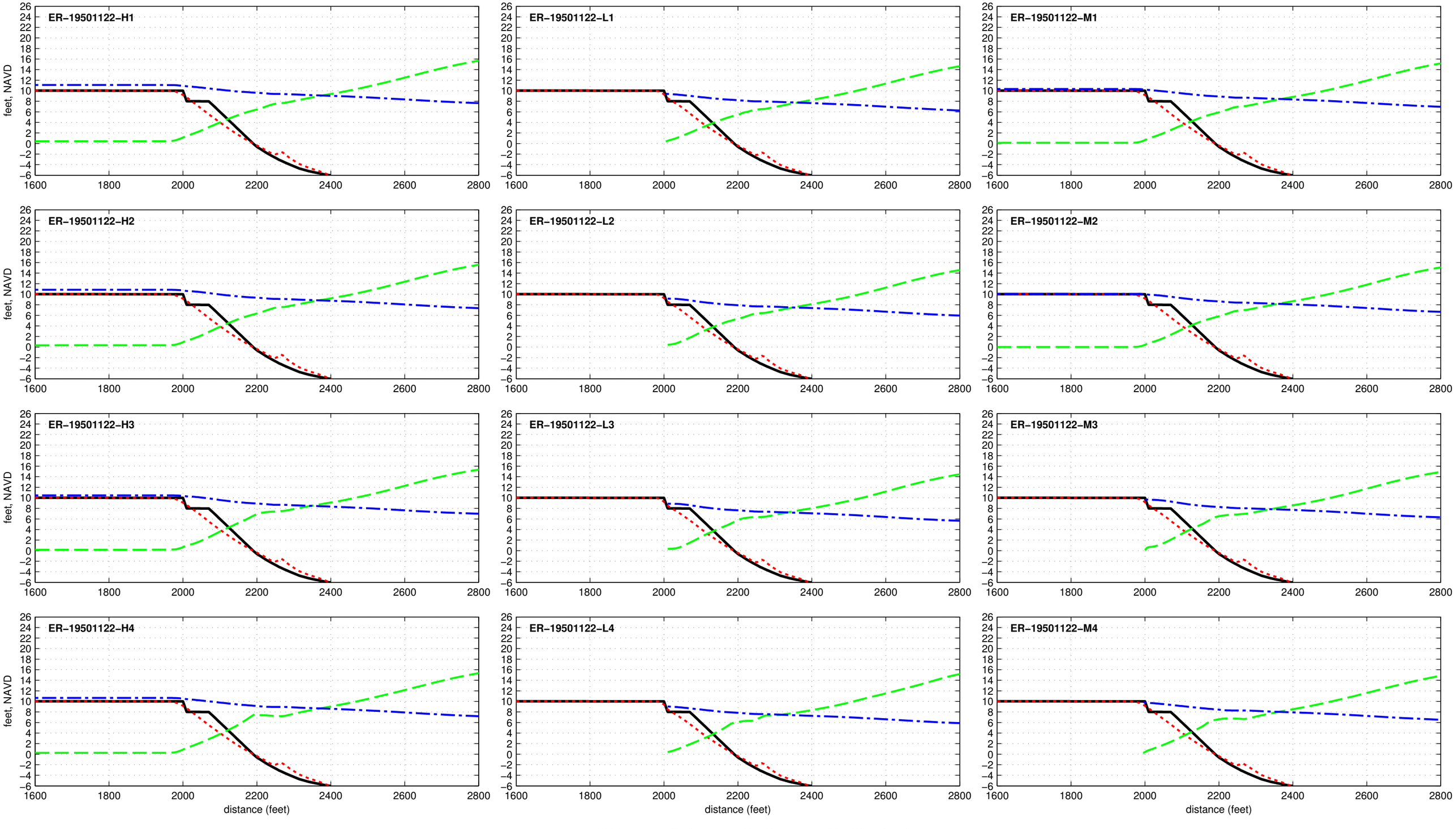

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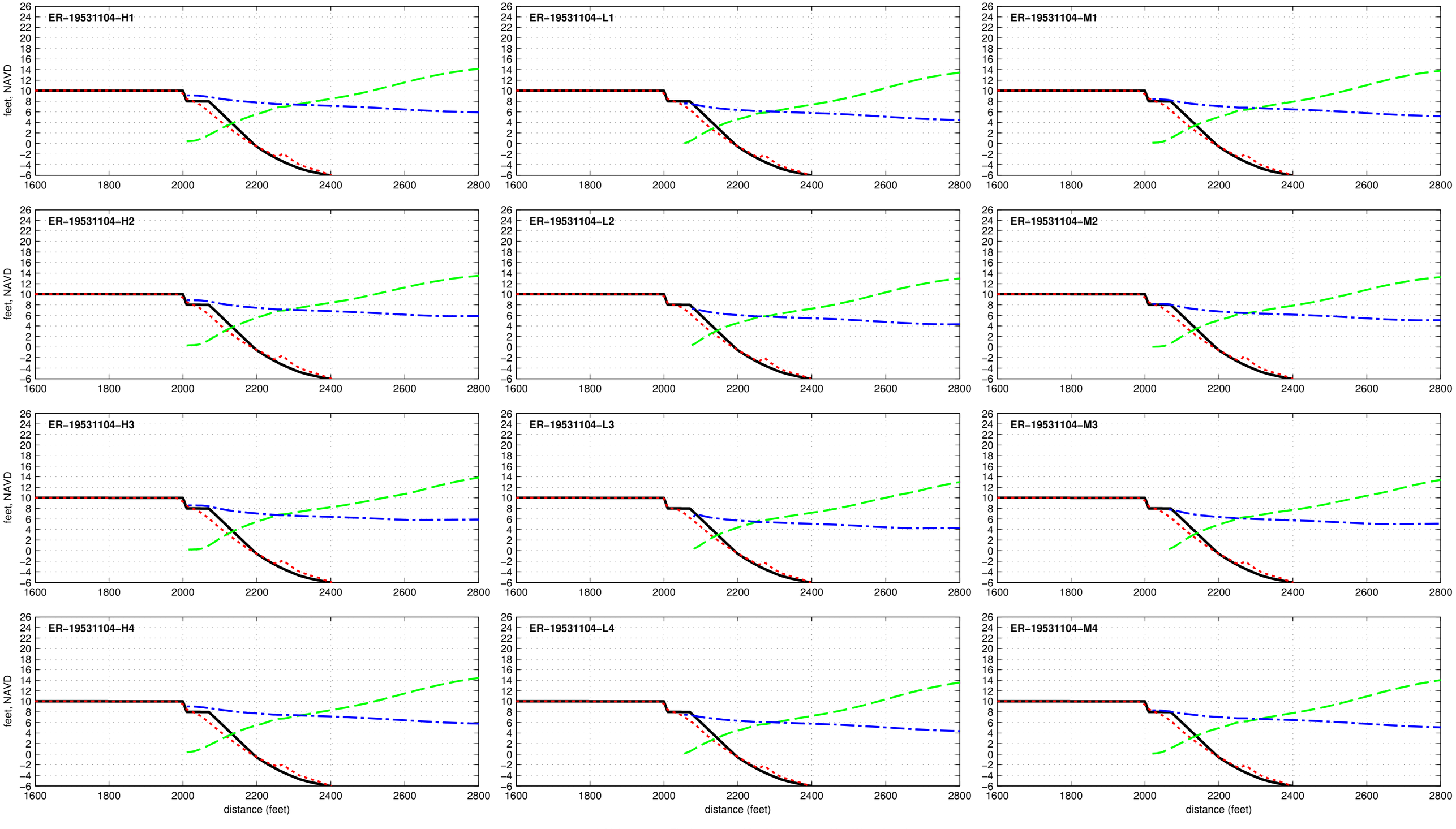

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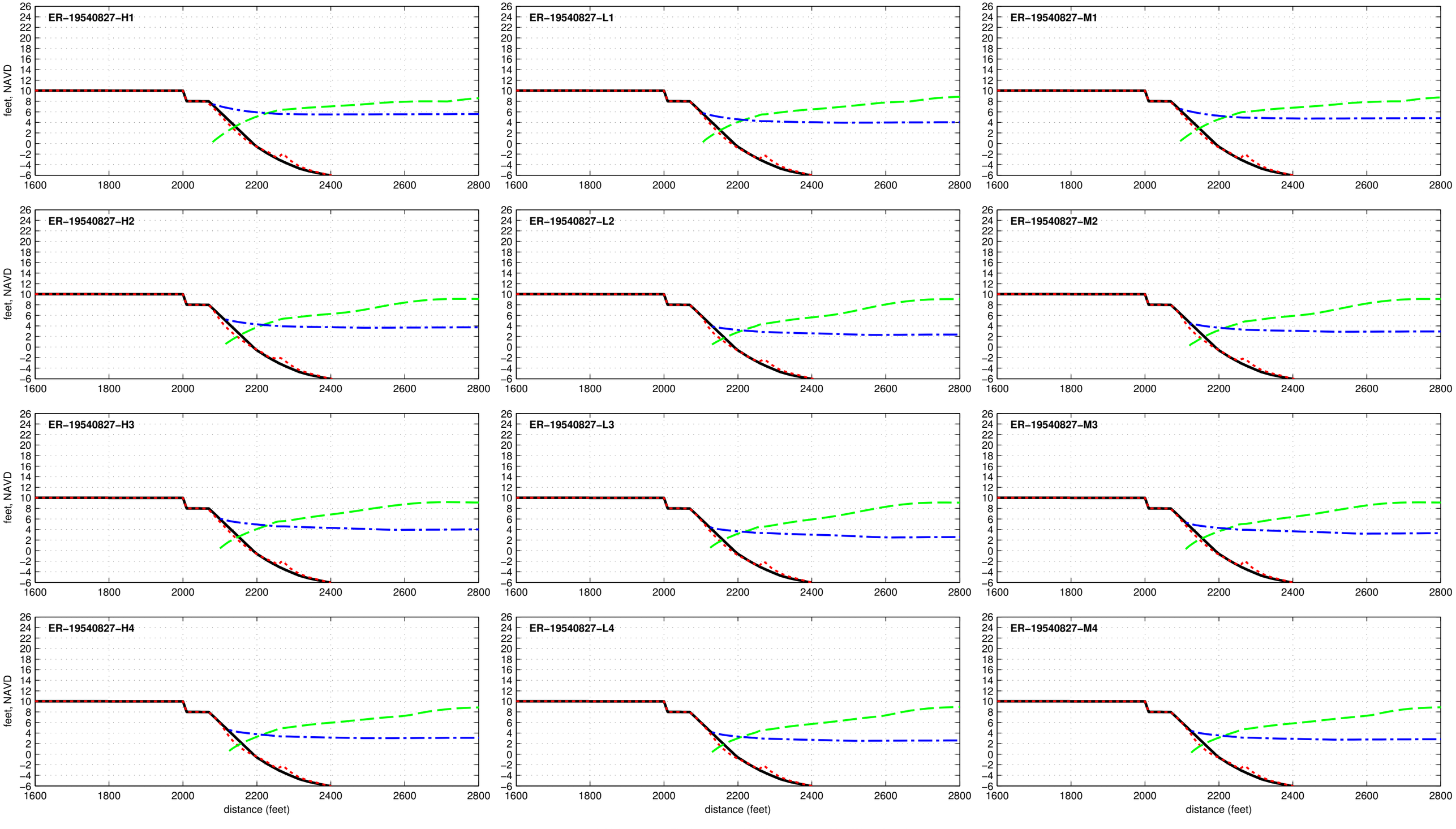

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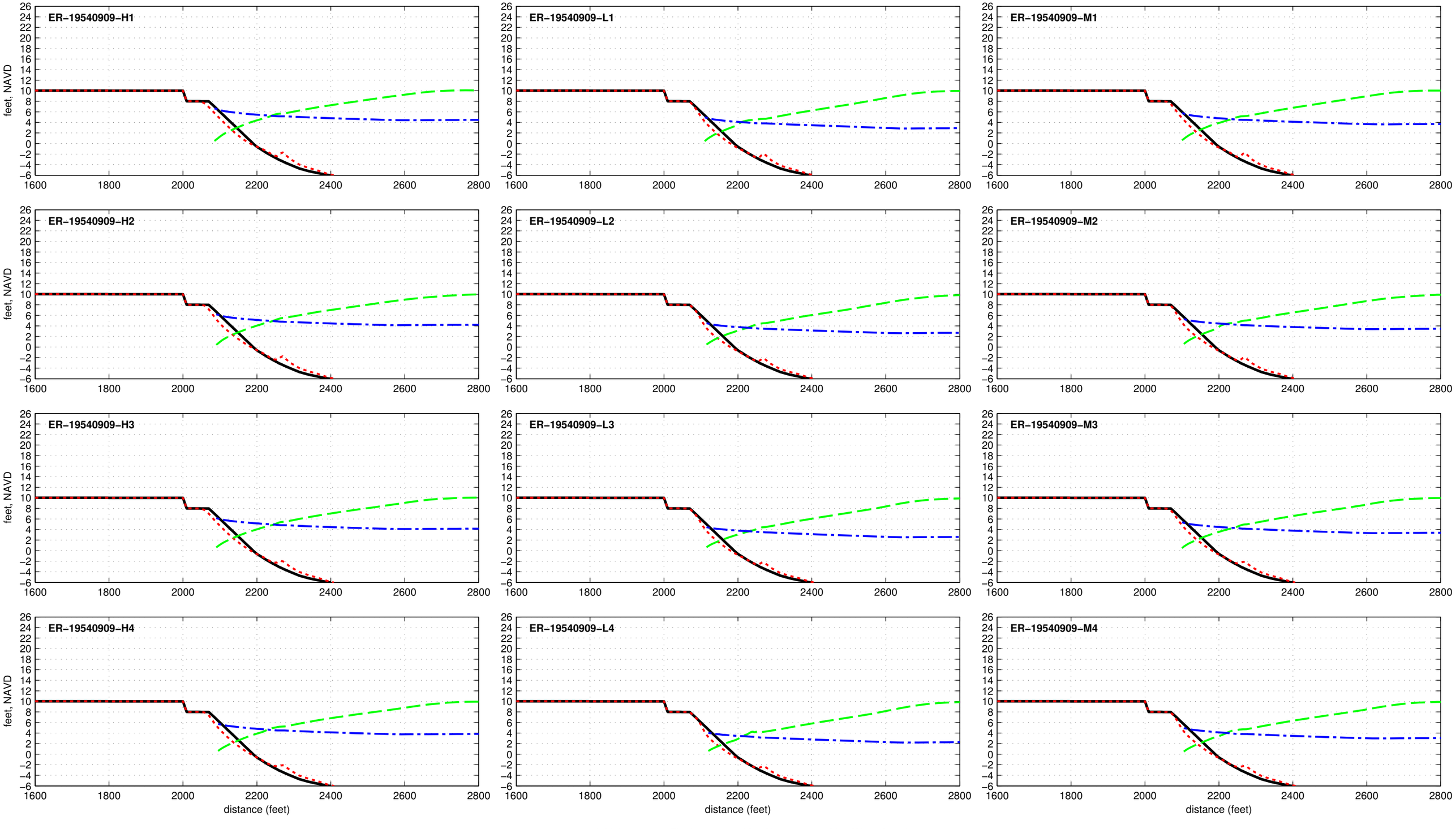

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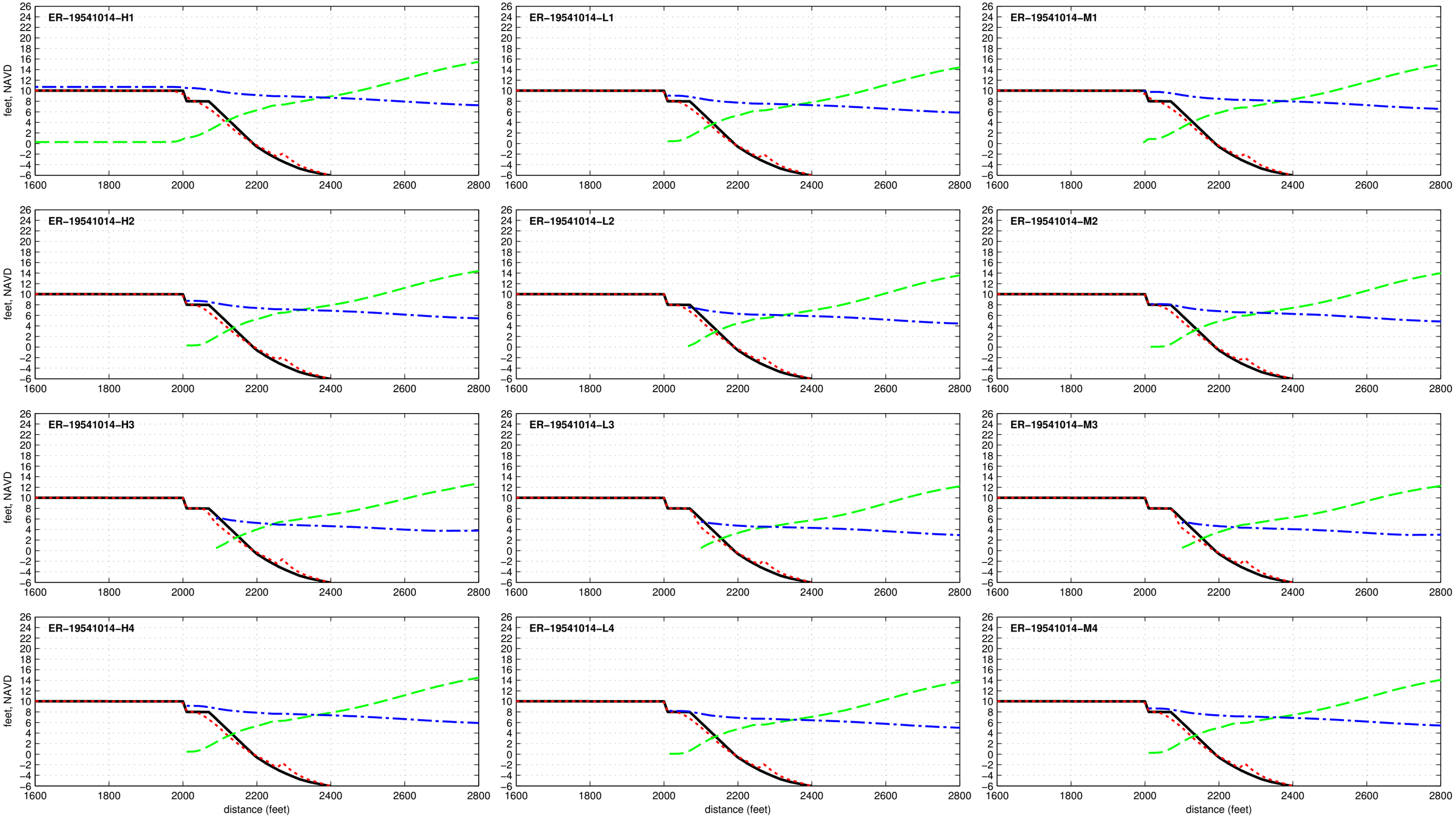

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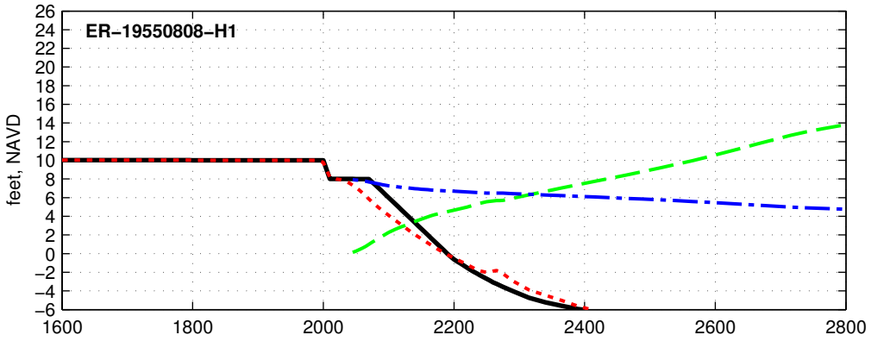
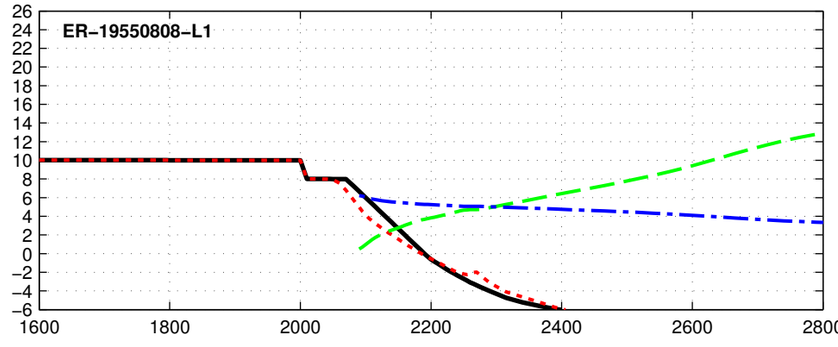
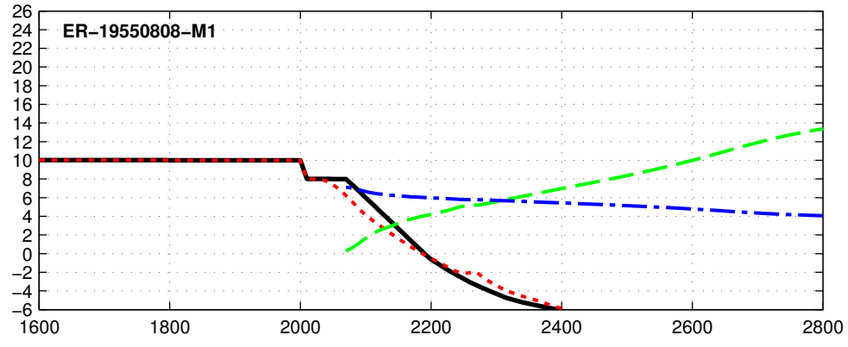
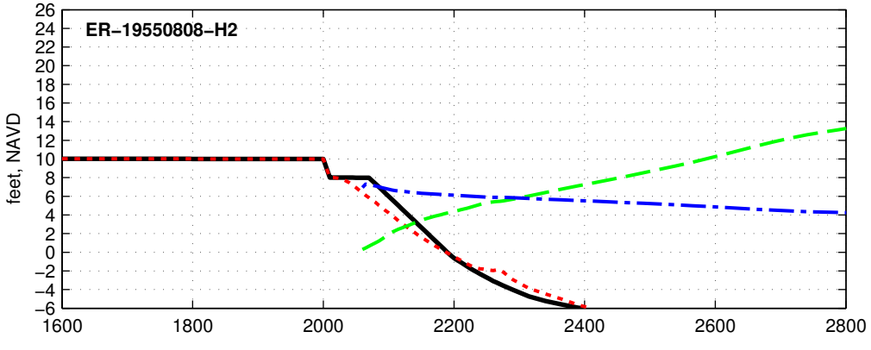
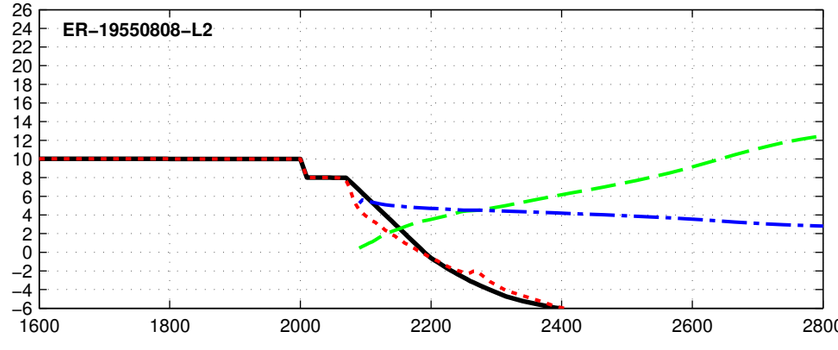
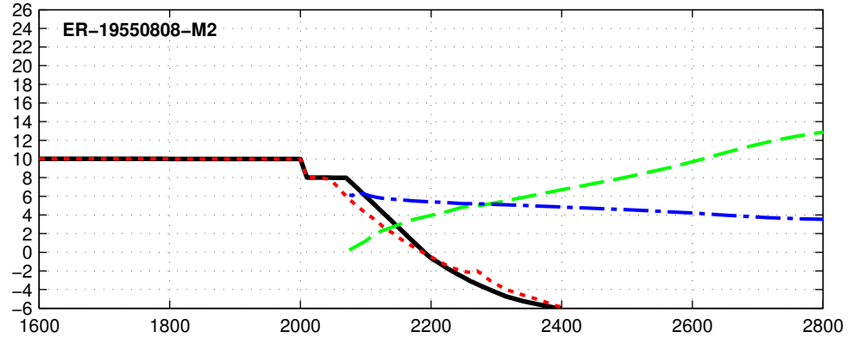


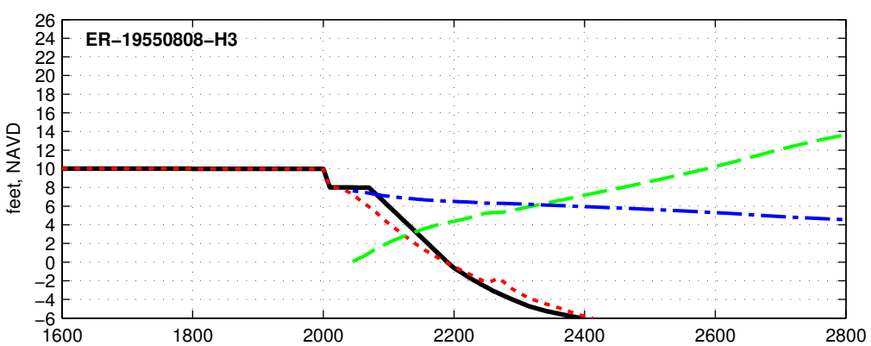


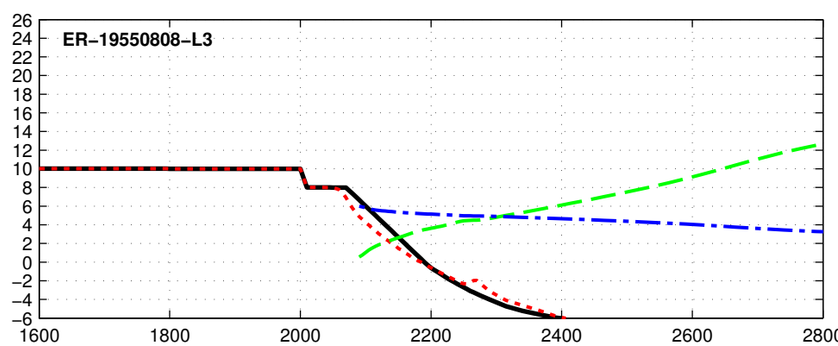


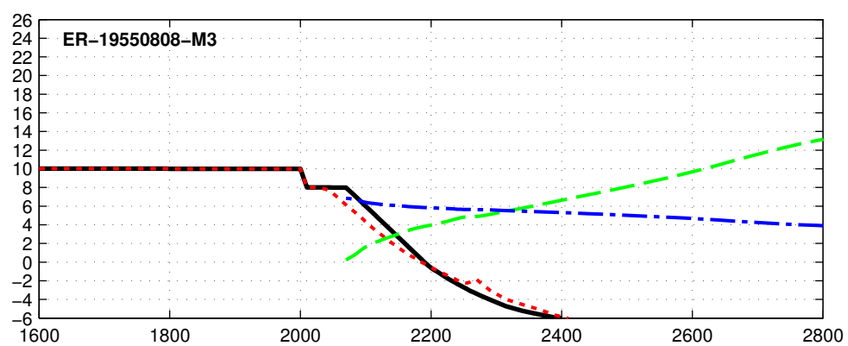


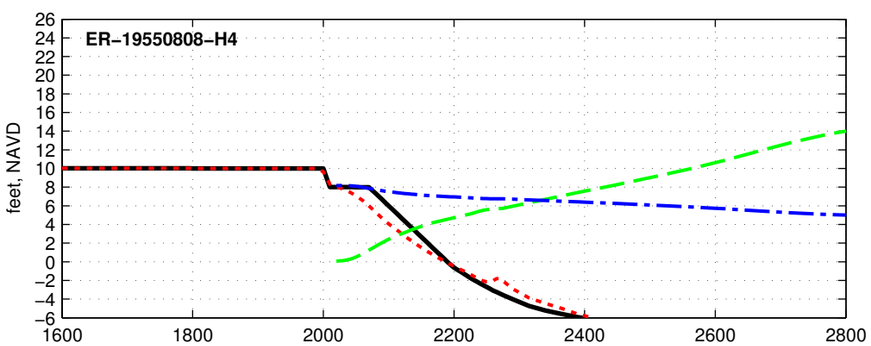


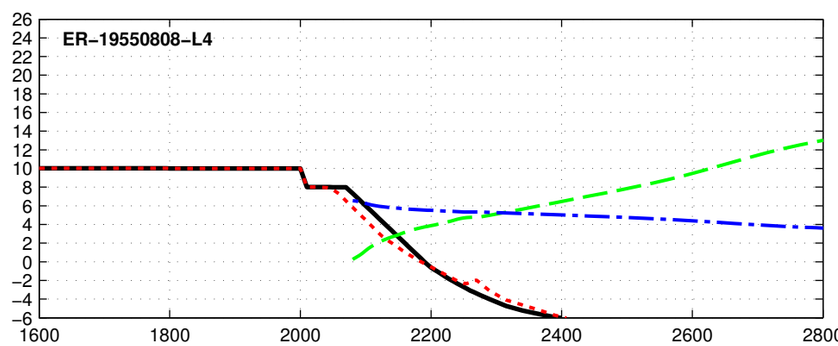


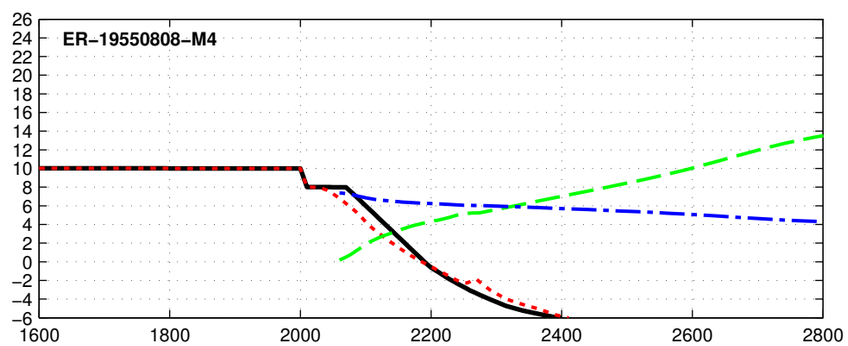






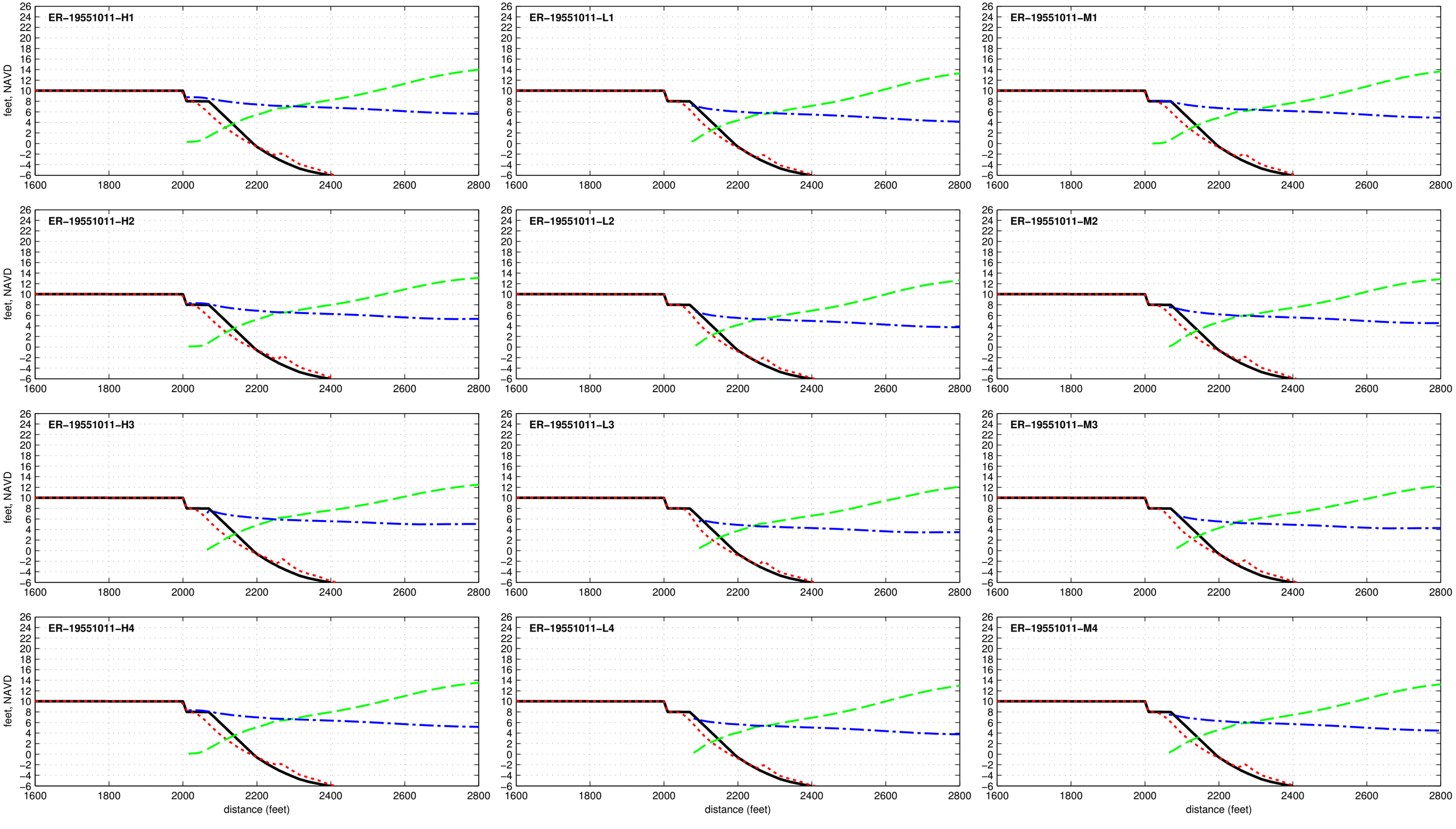






Initial Final Max Hs Max Water Level

Dune R2T1-10.0-000-060 - ER-19551011



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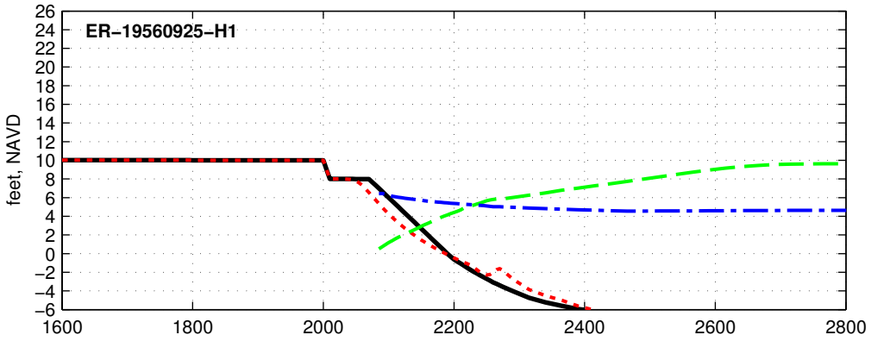
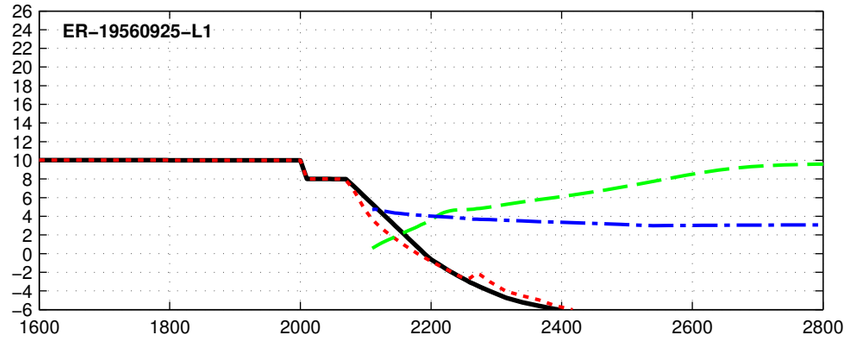
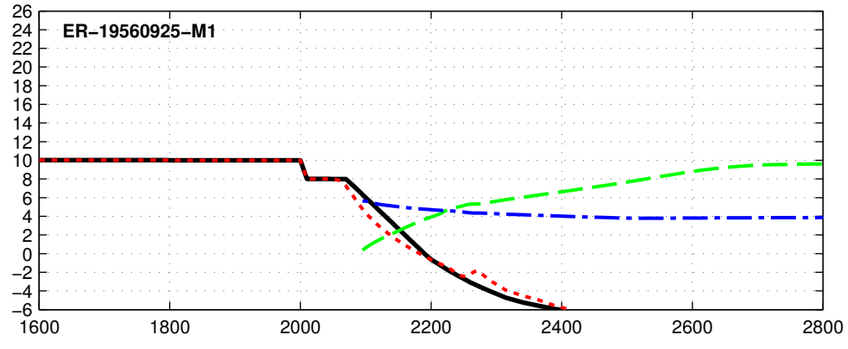
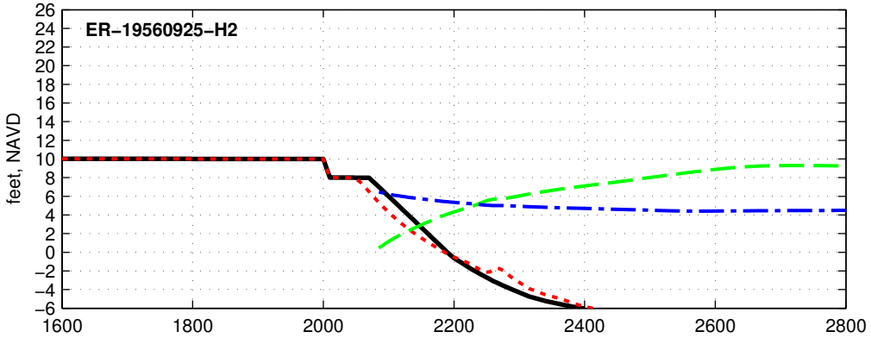
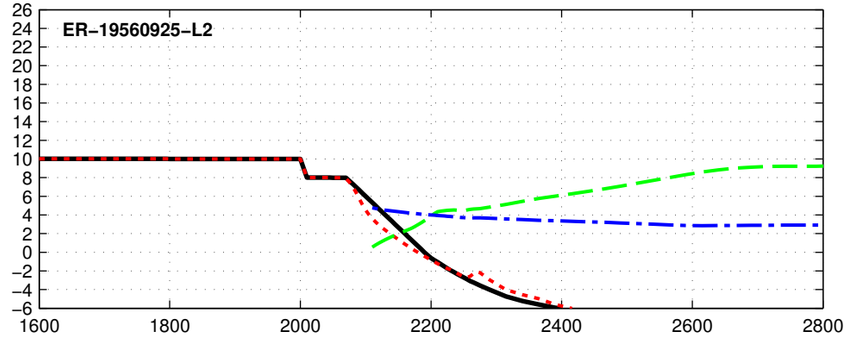
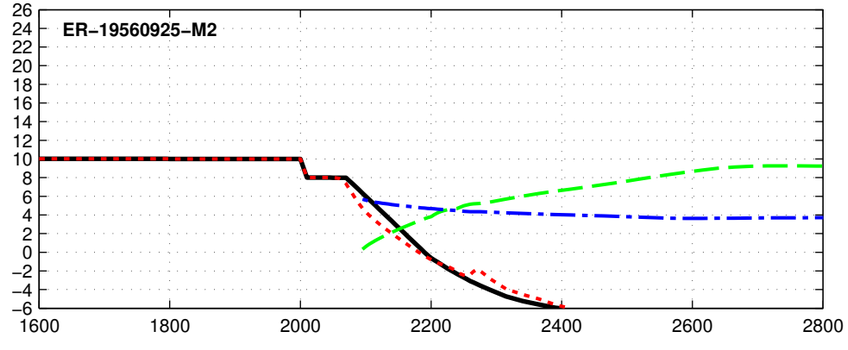


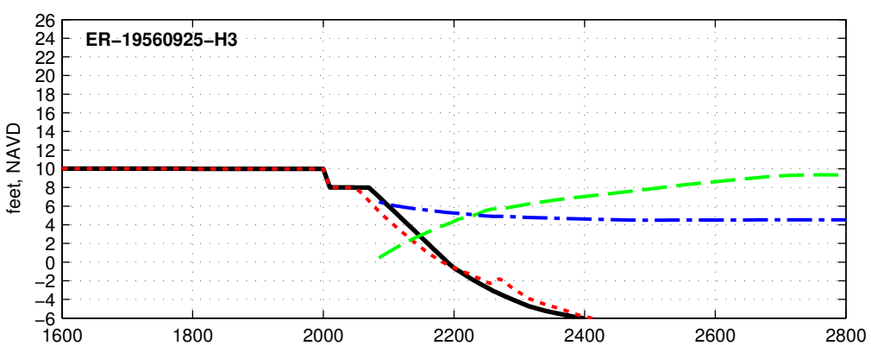


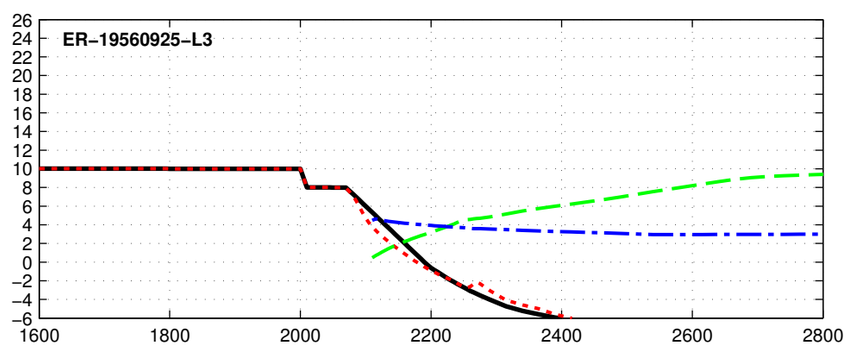


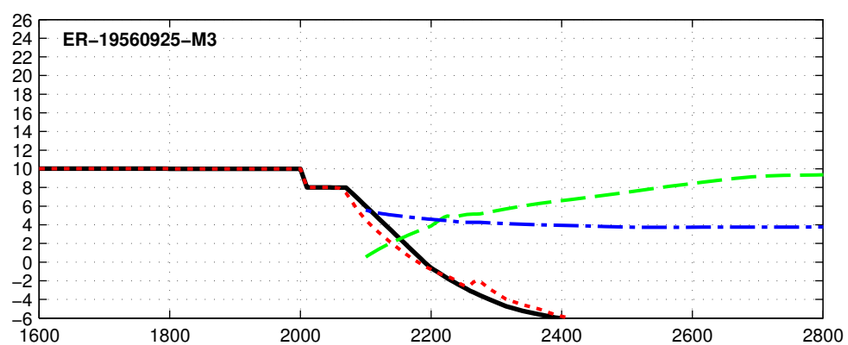


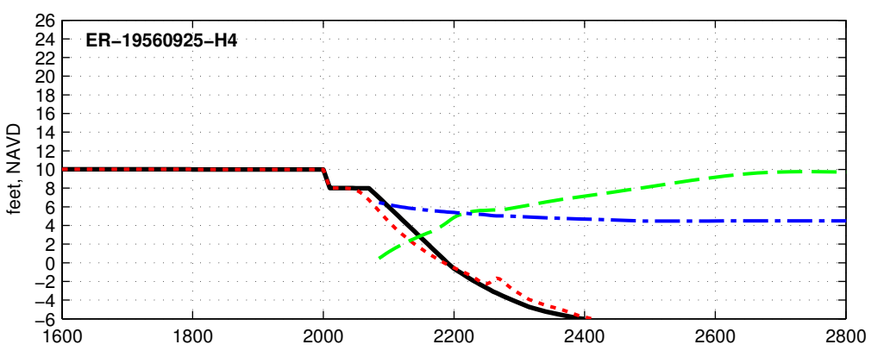


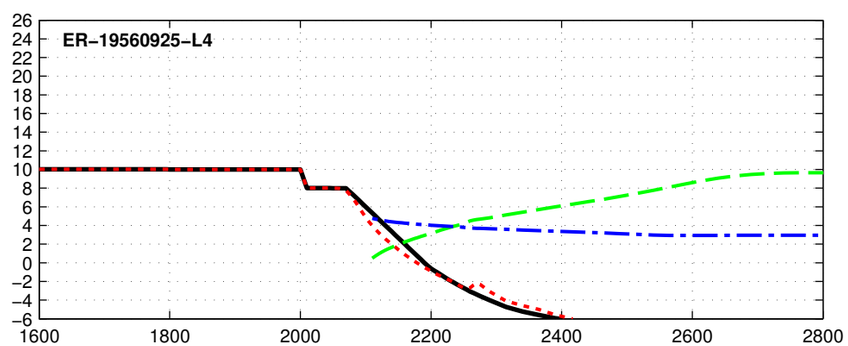


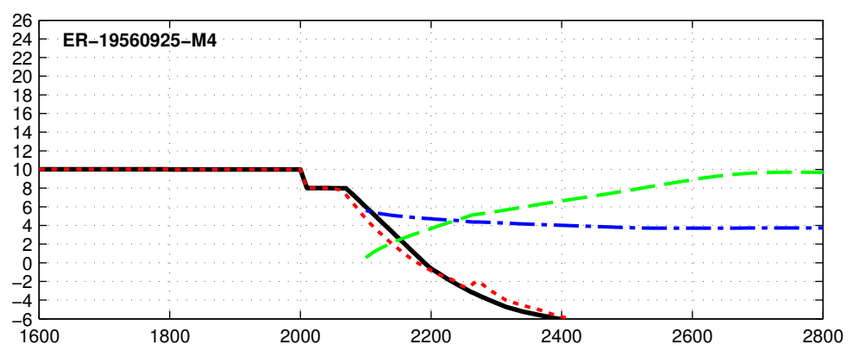






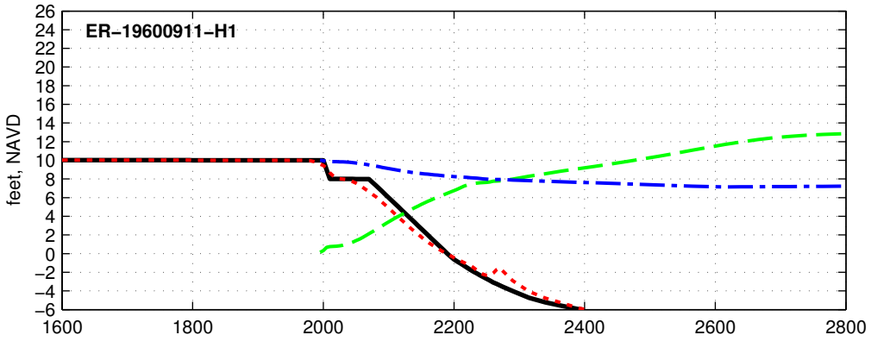


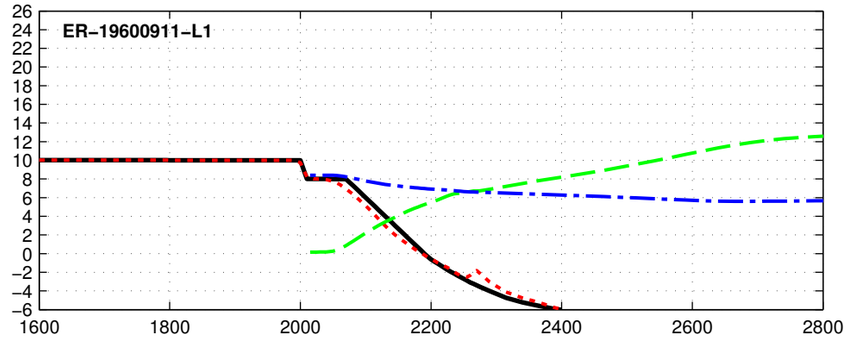


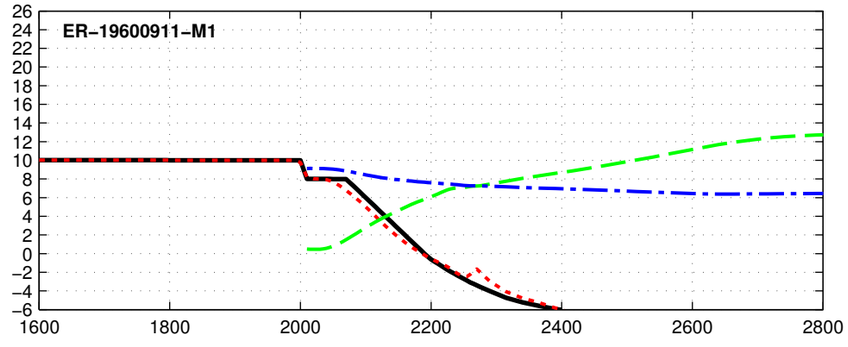


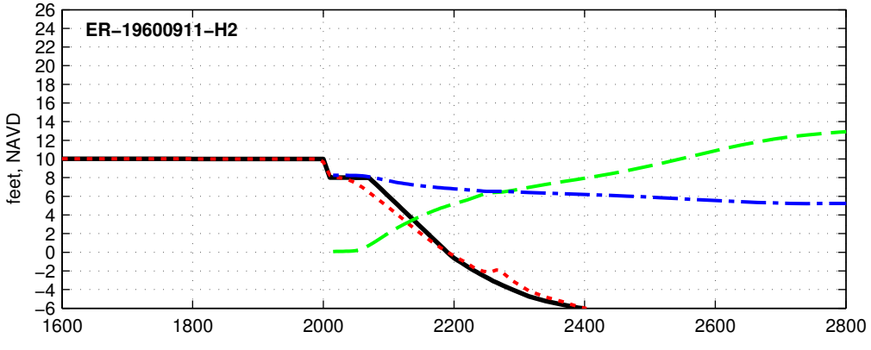
Initial Final Max Hs Max Water Level

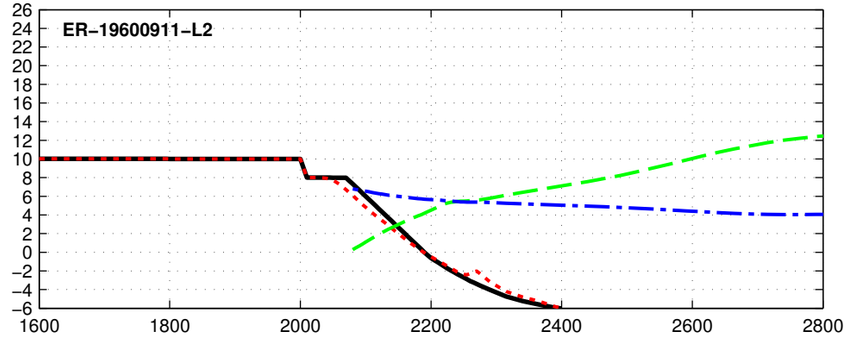
Dune R2T1-10.0-000-060 - ER-19600911

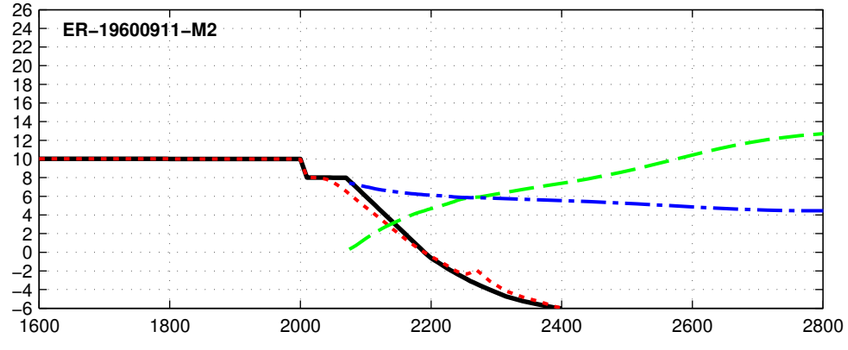


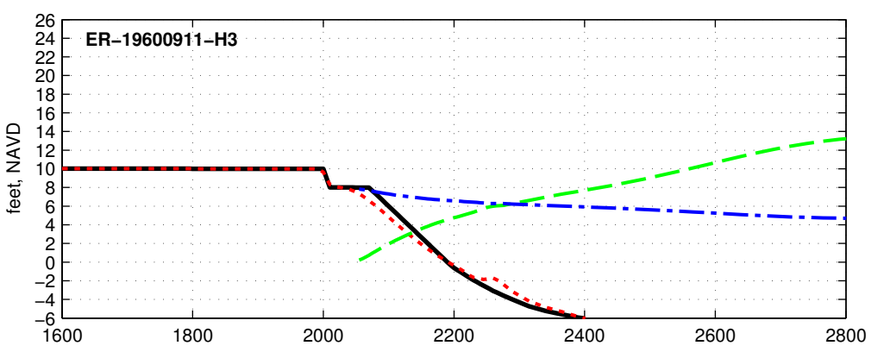


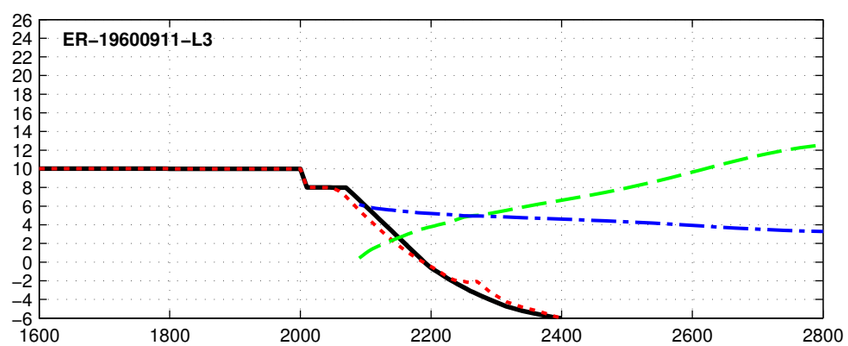


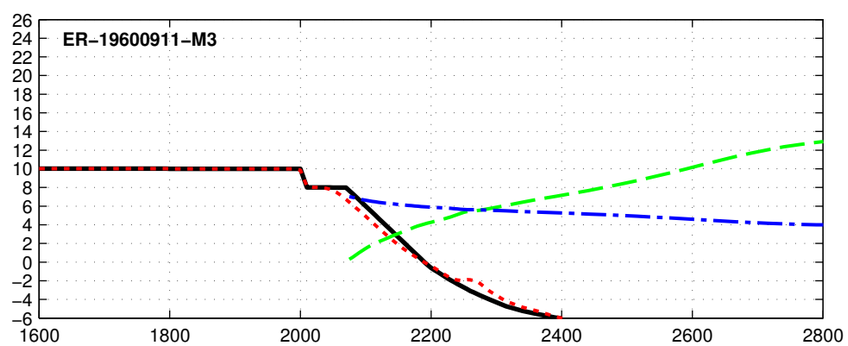


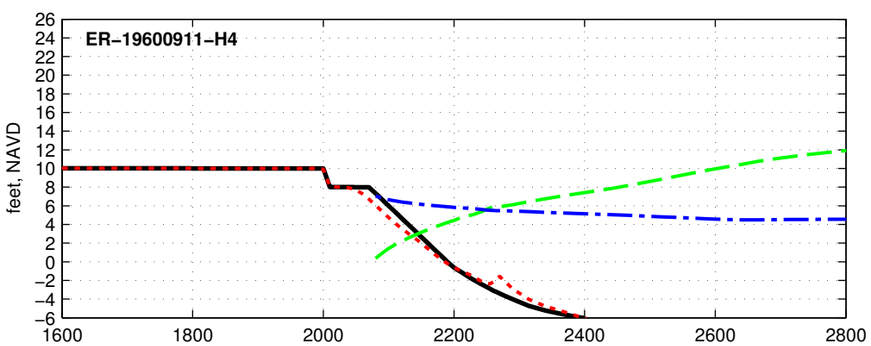


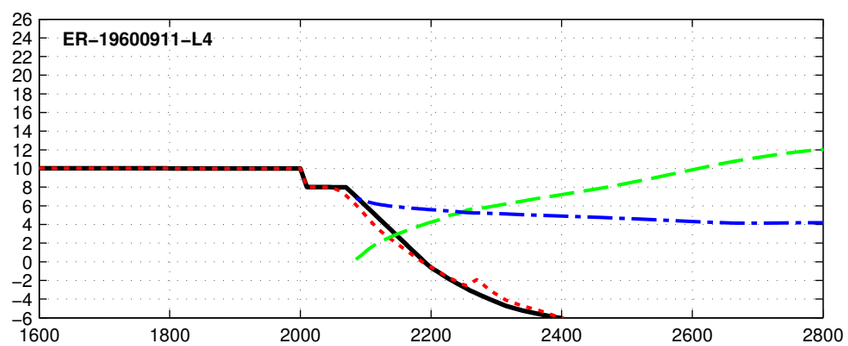


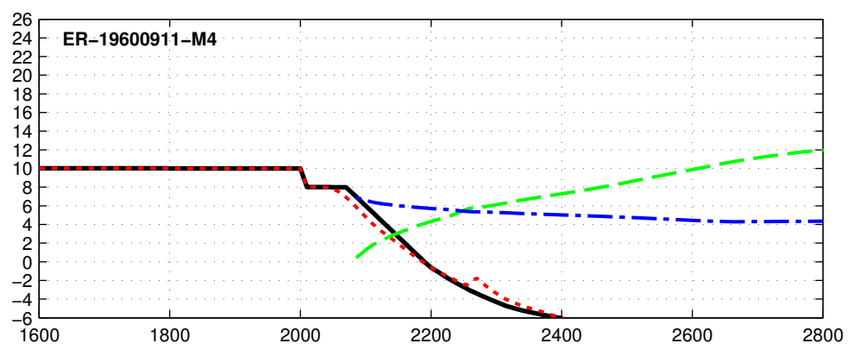






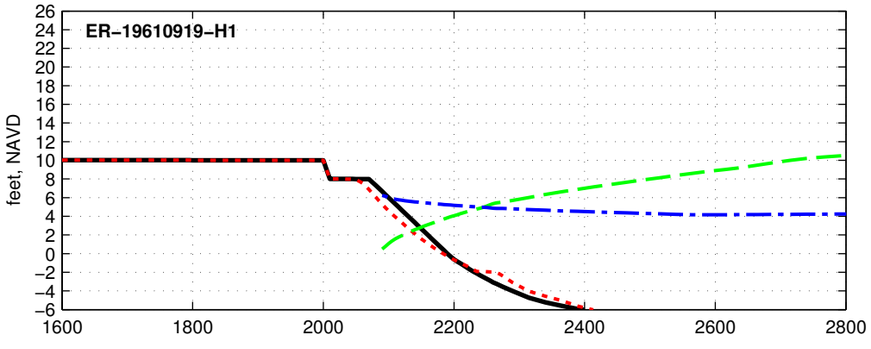


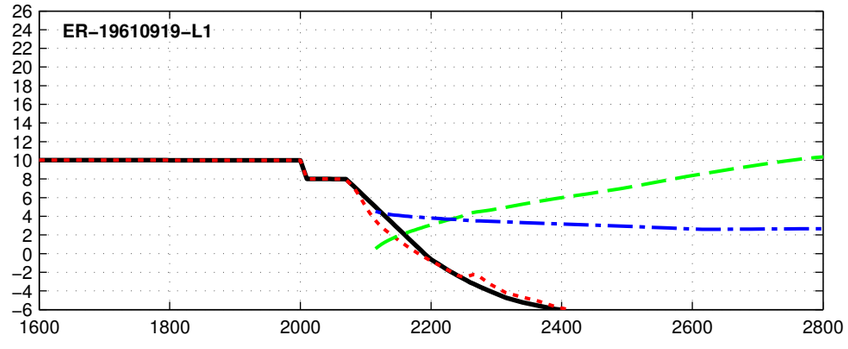


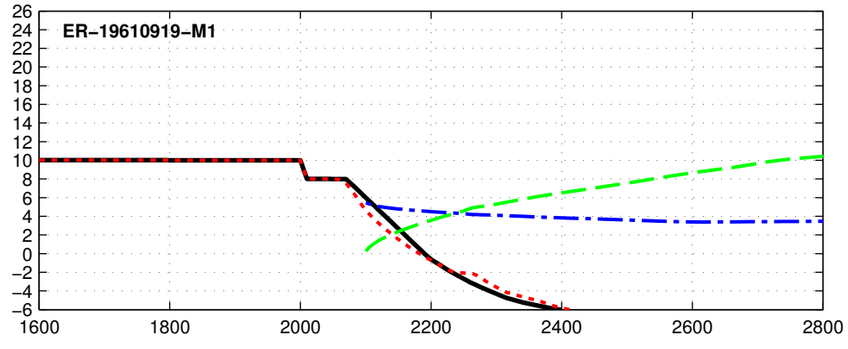


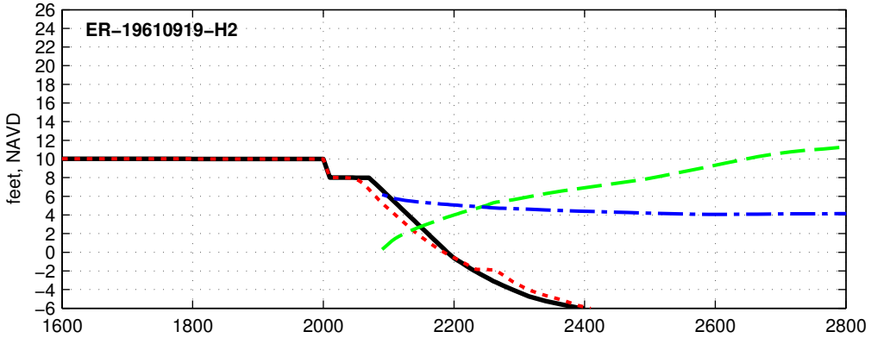
Initial Final Max Hs Max Water Level

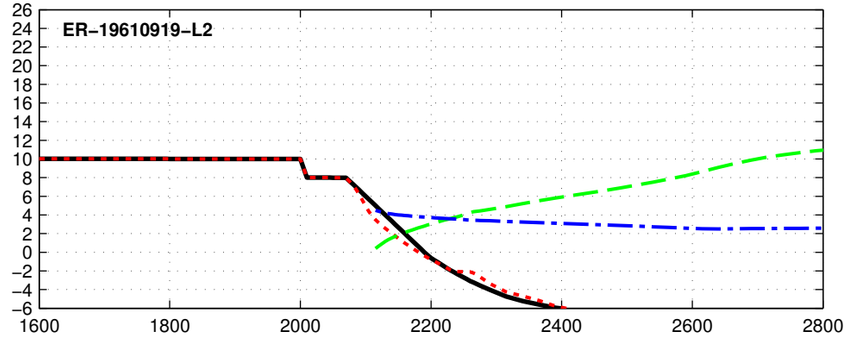
Dune R2T1-10.0-000-060 - ER-19610919

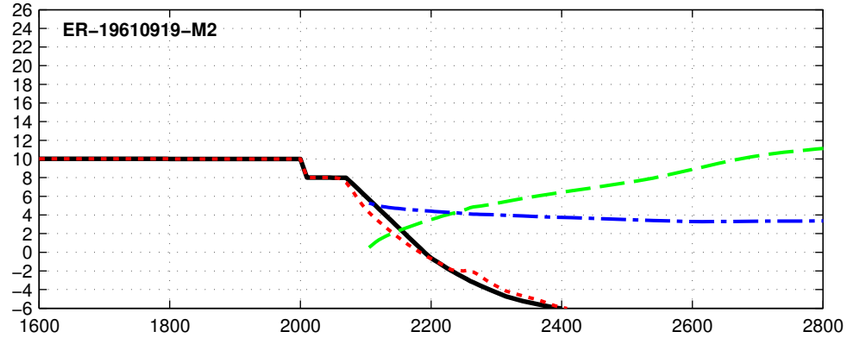


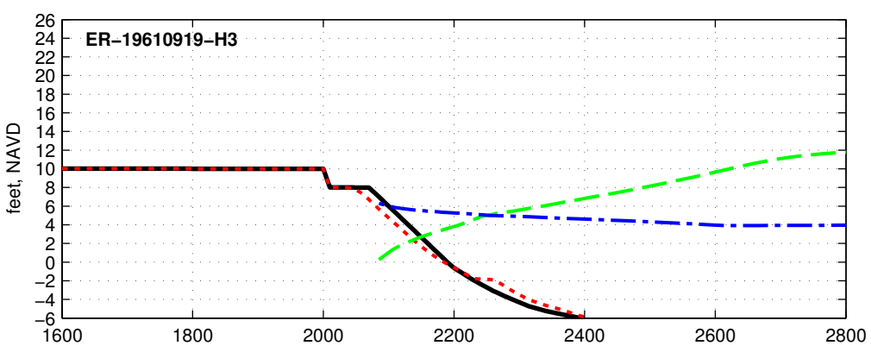


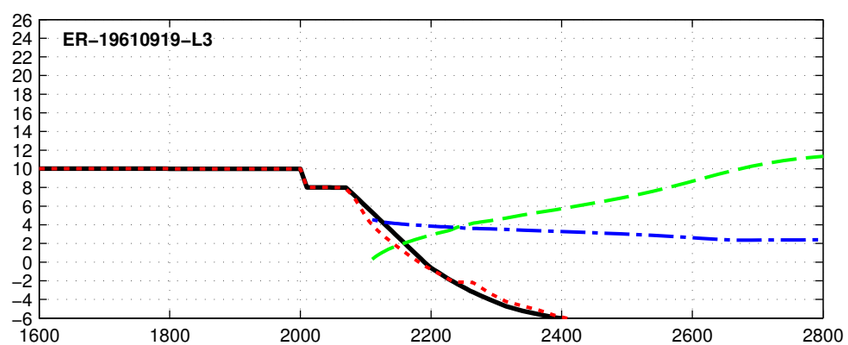


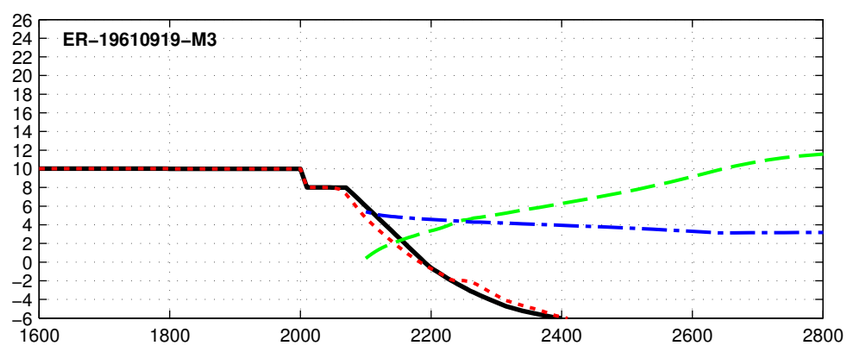


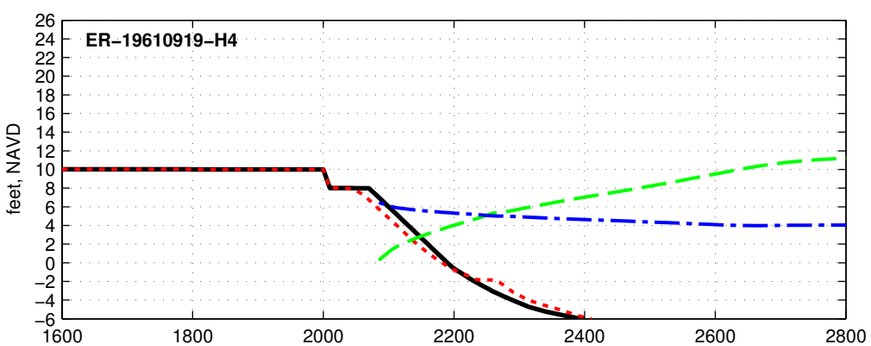


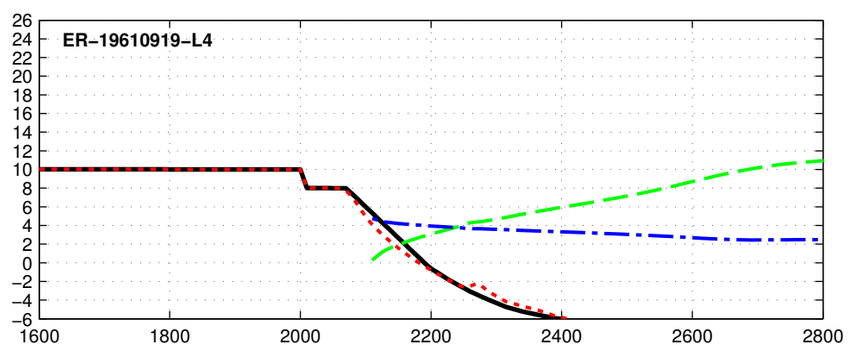


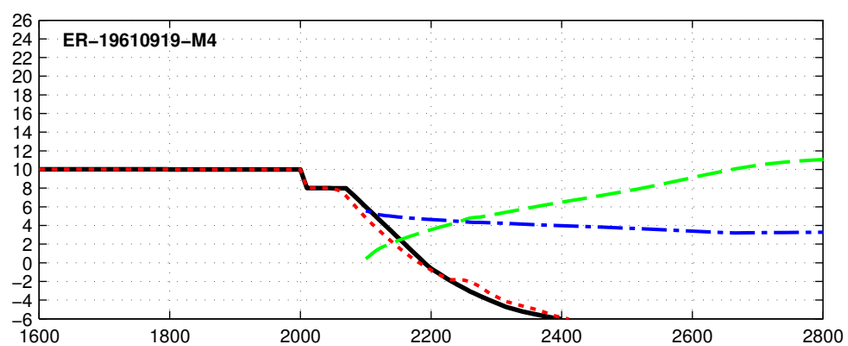






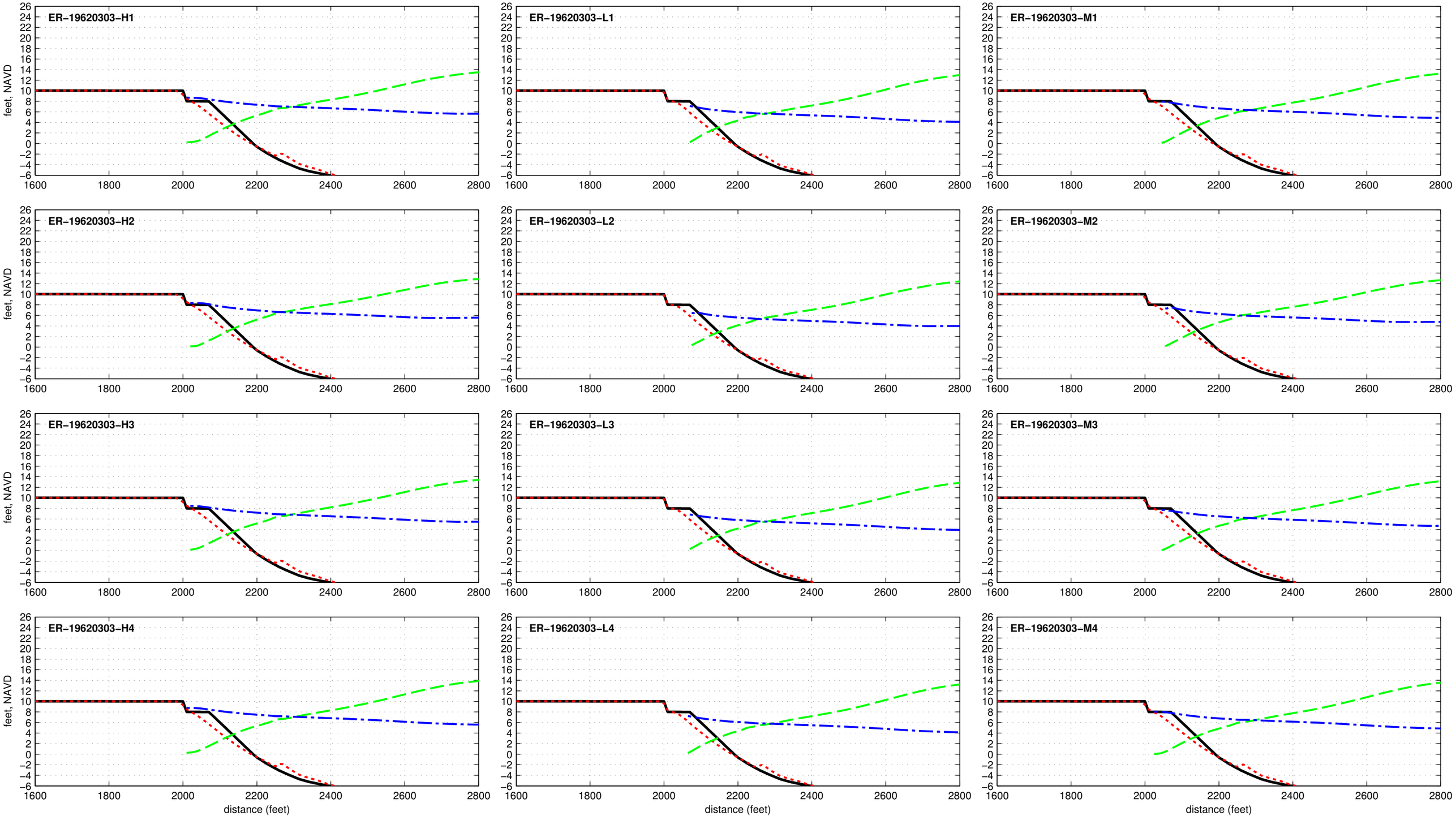






Initial Final Max Hs Max Water Level

Dune R2T1-10.0-000-060 - ER-19620303



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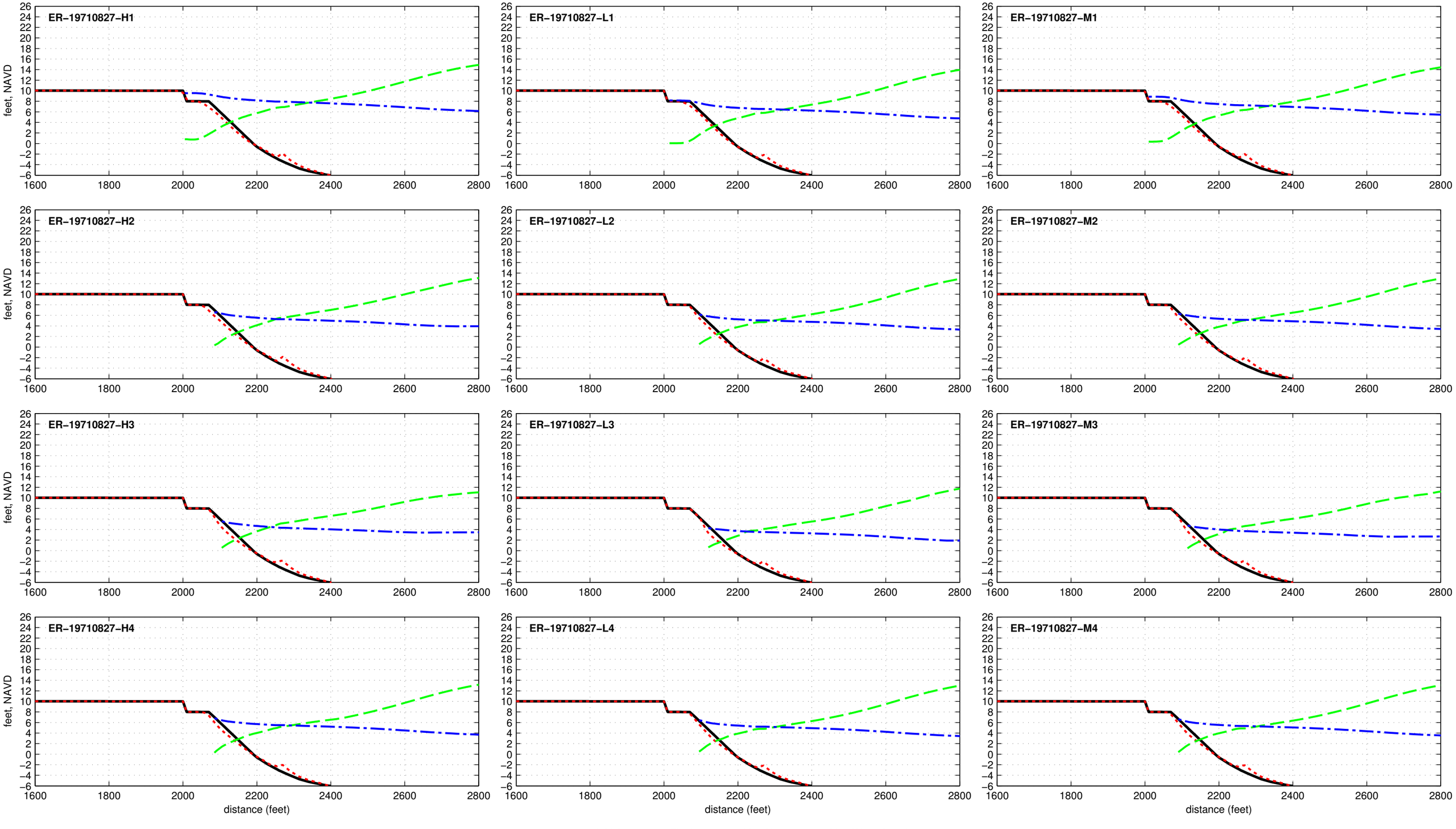

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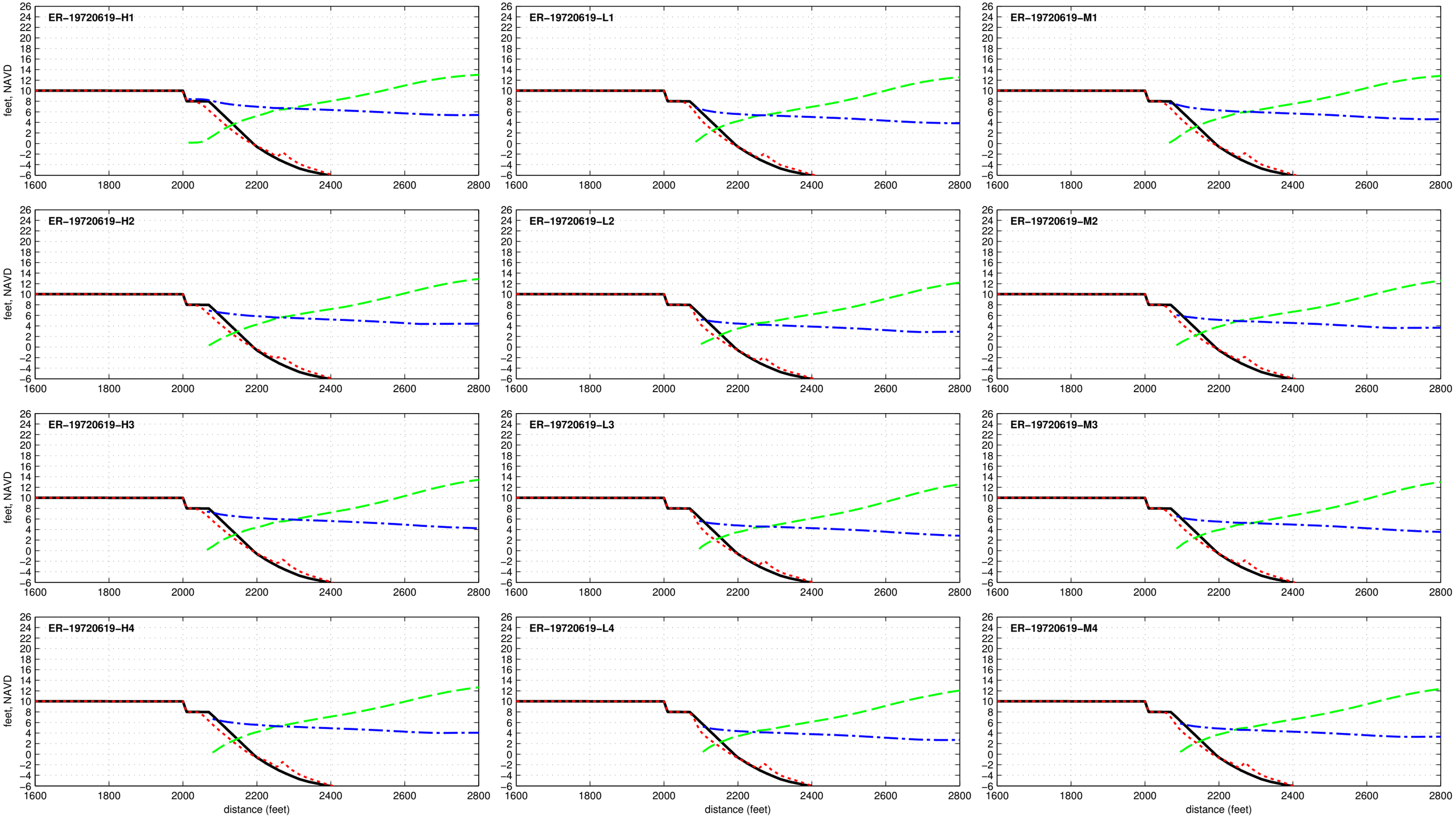

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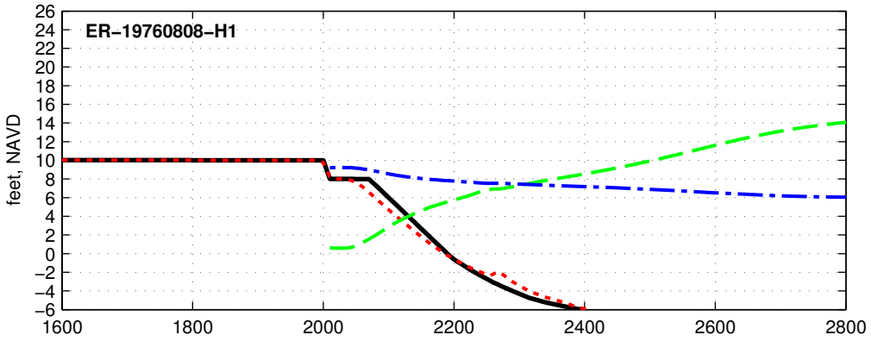
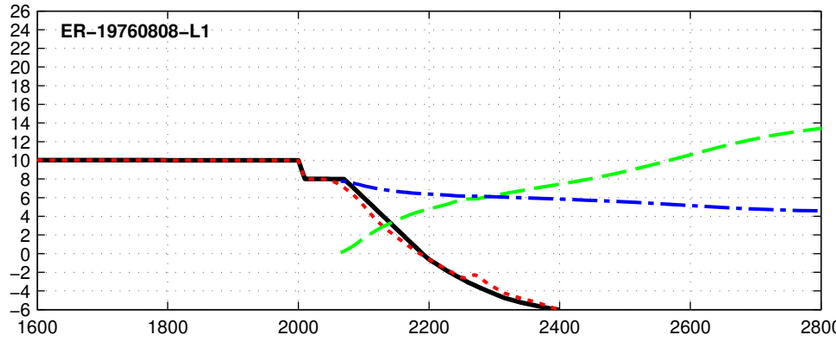
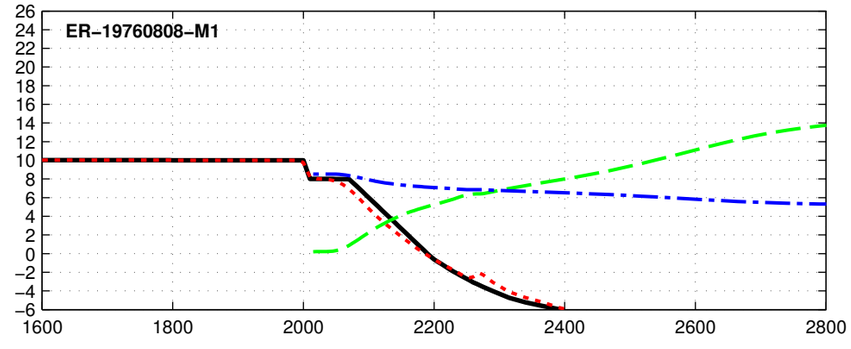
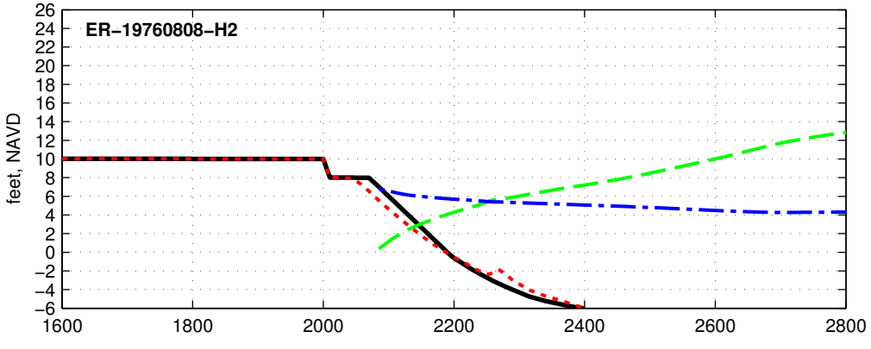
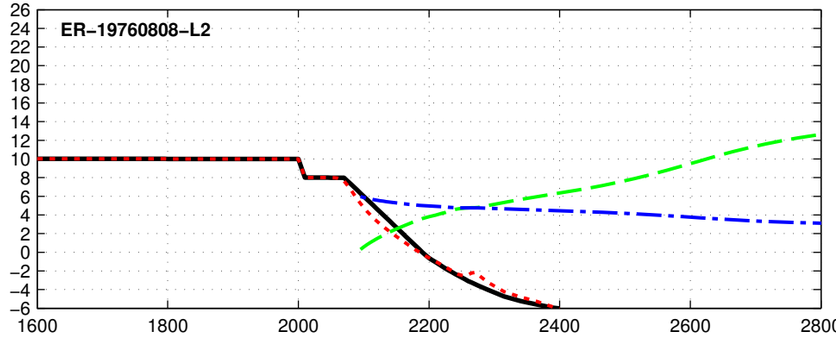
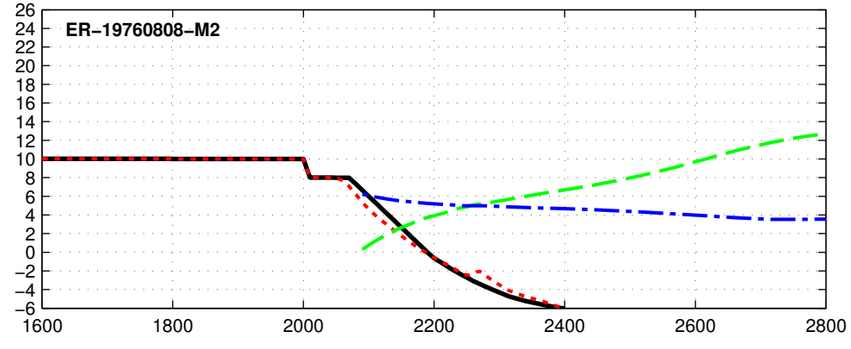


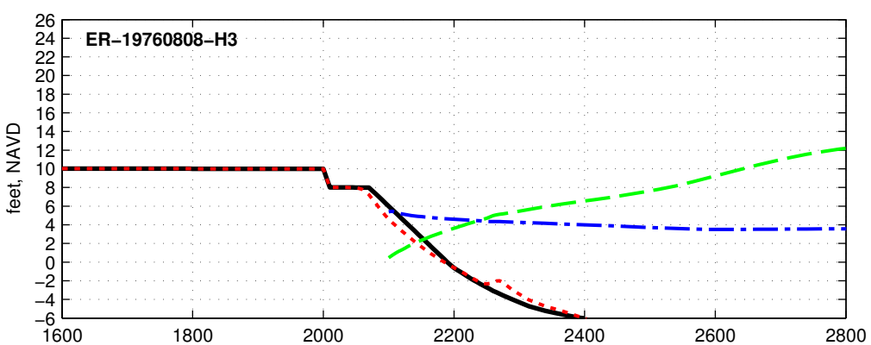


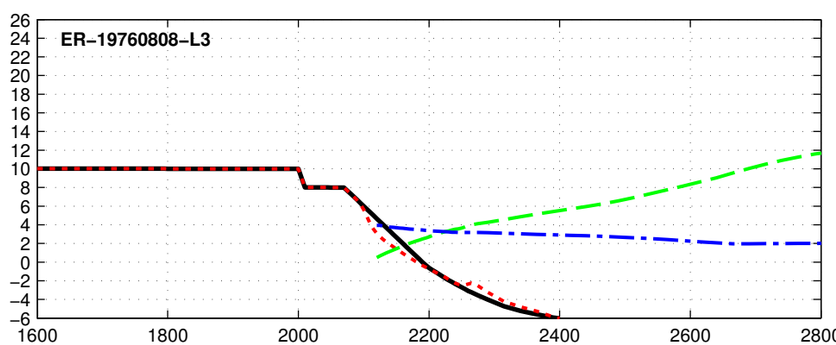


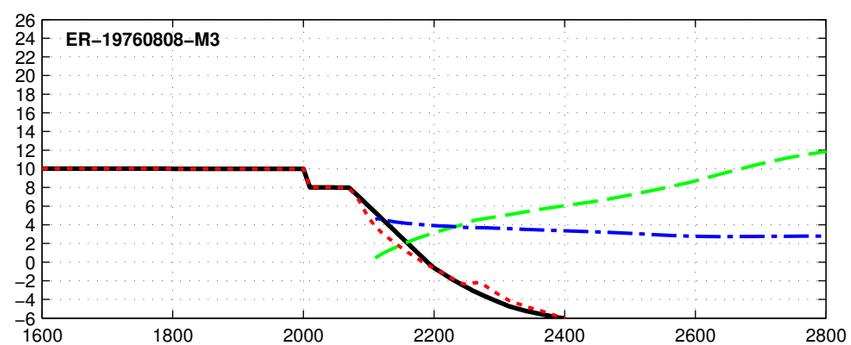


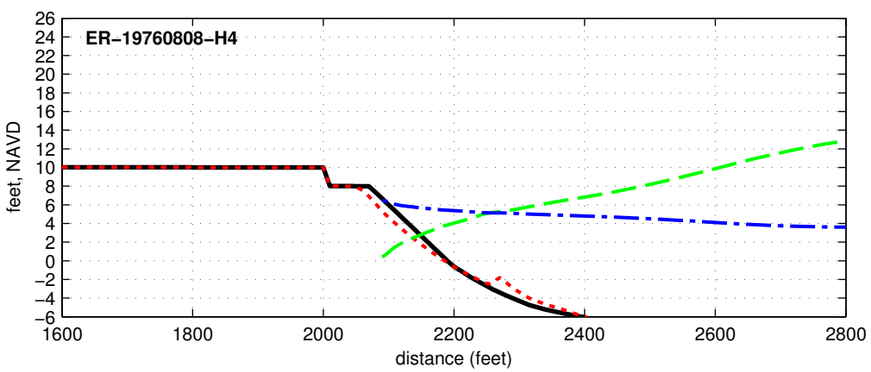


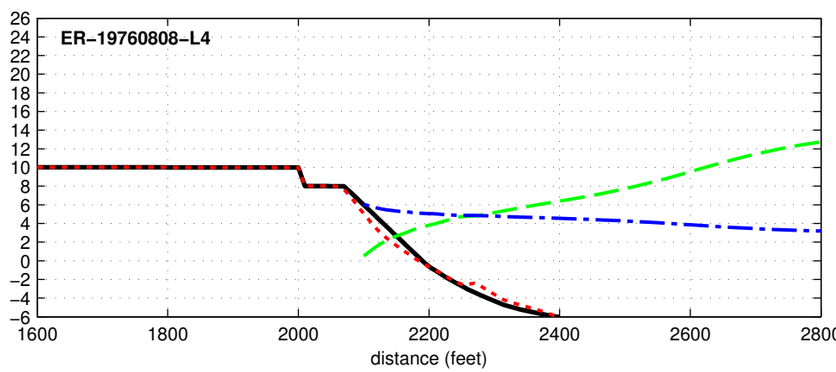


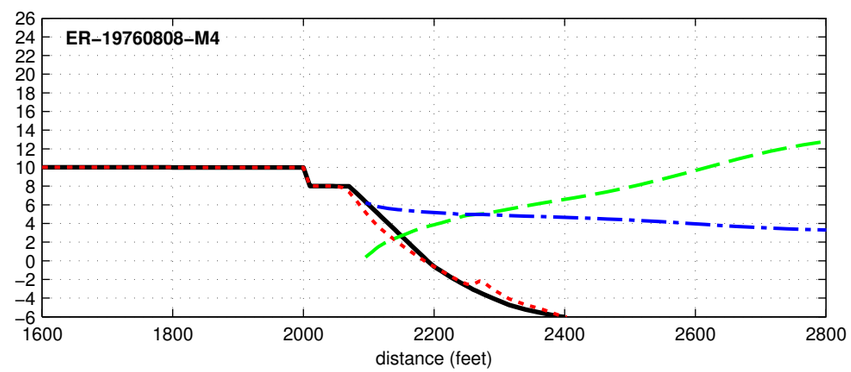






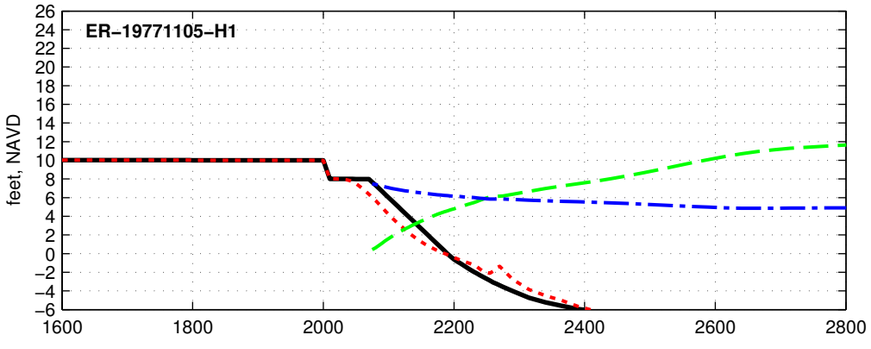


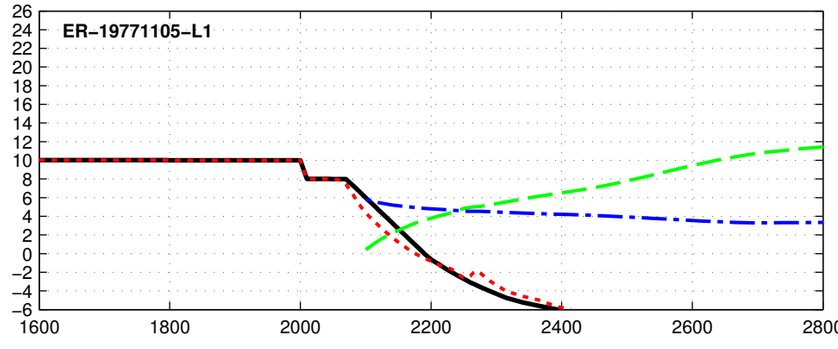


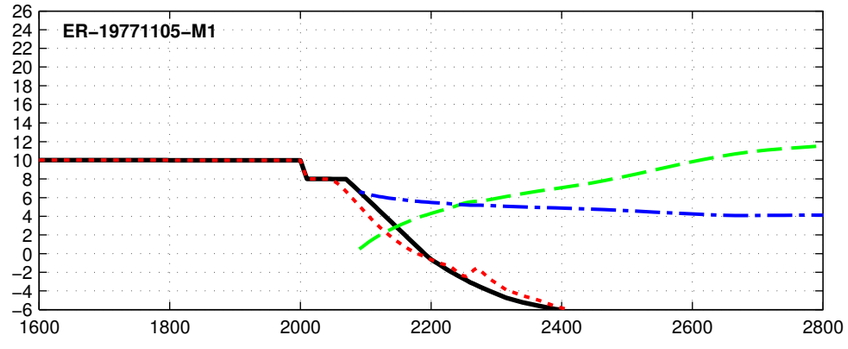


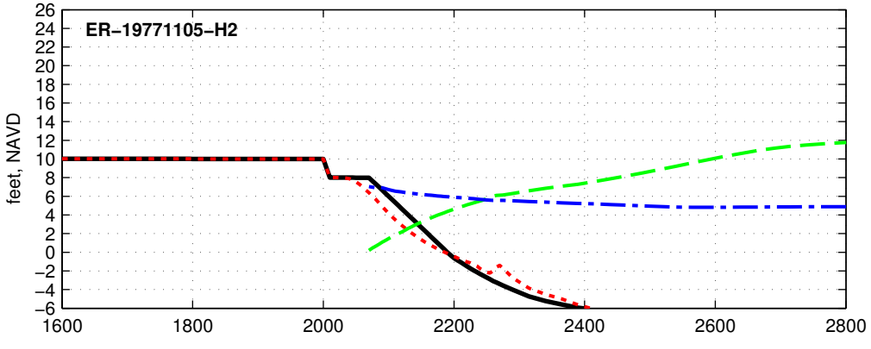
Initial Final Max Hs Max Water Level

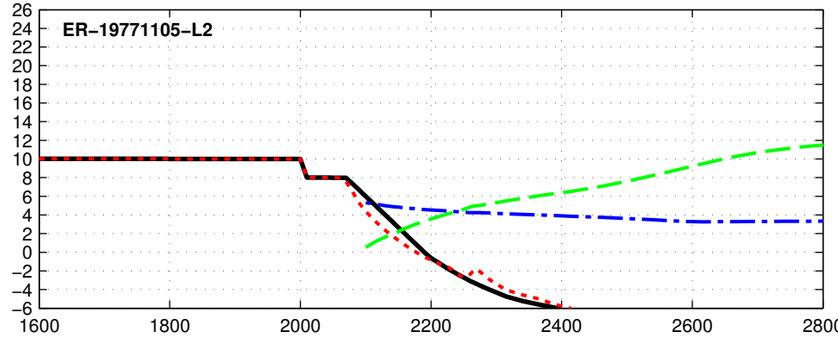
Dune R2T1-10.0-000-060 - ER-19771105

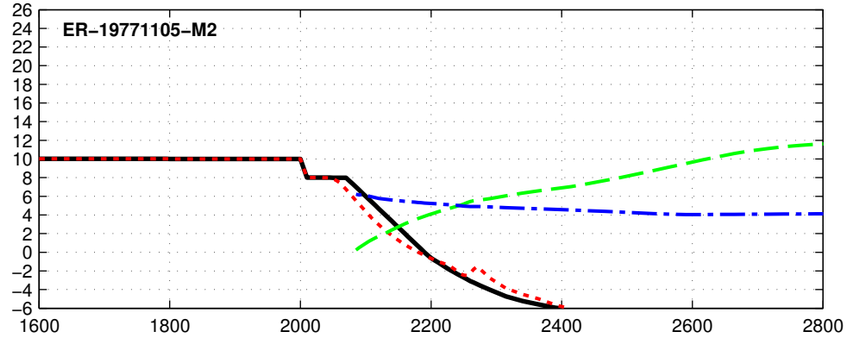


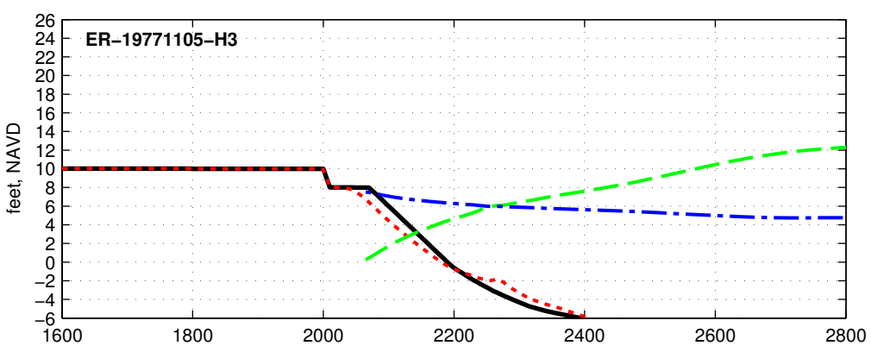


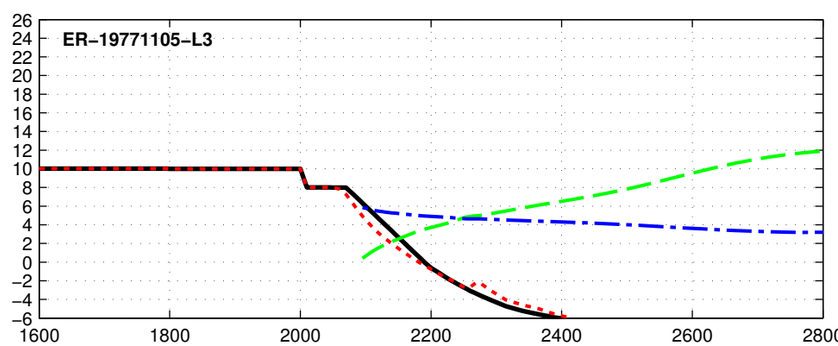


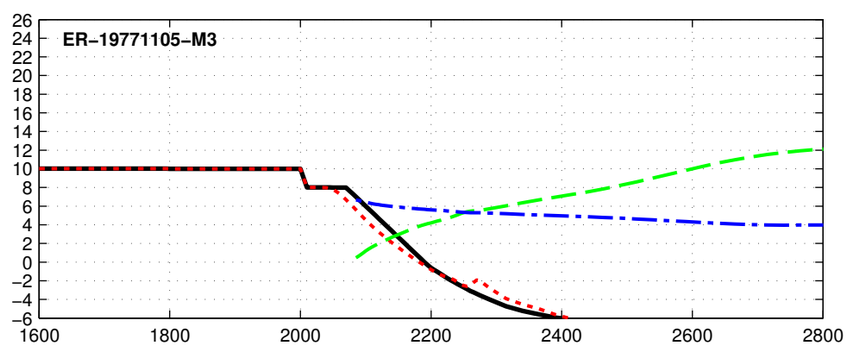


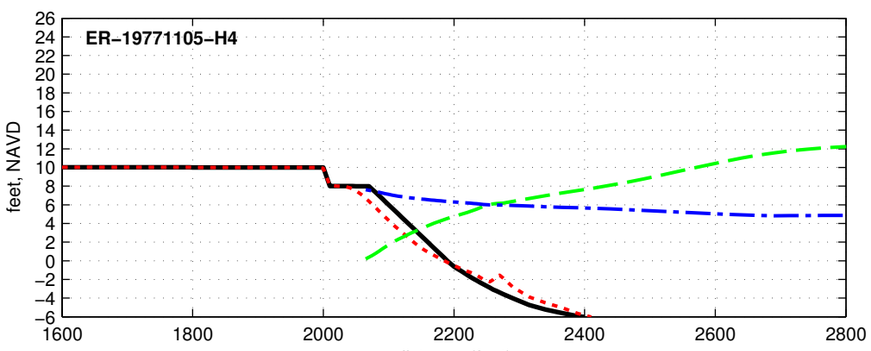


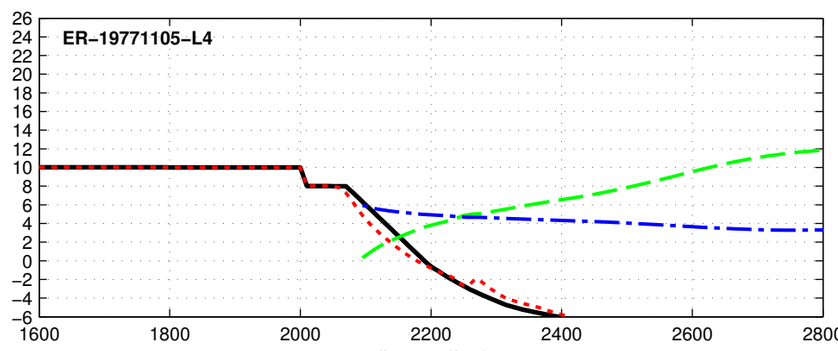


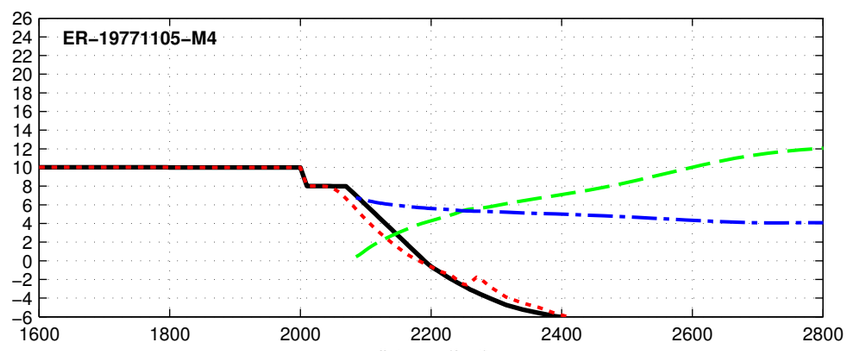






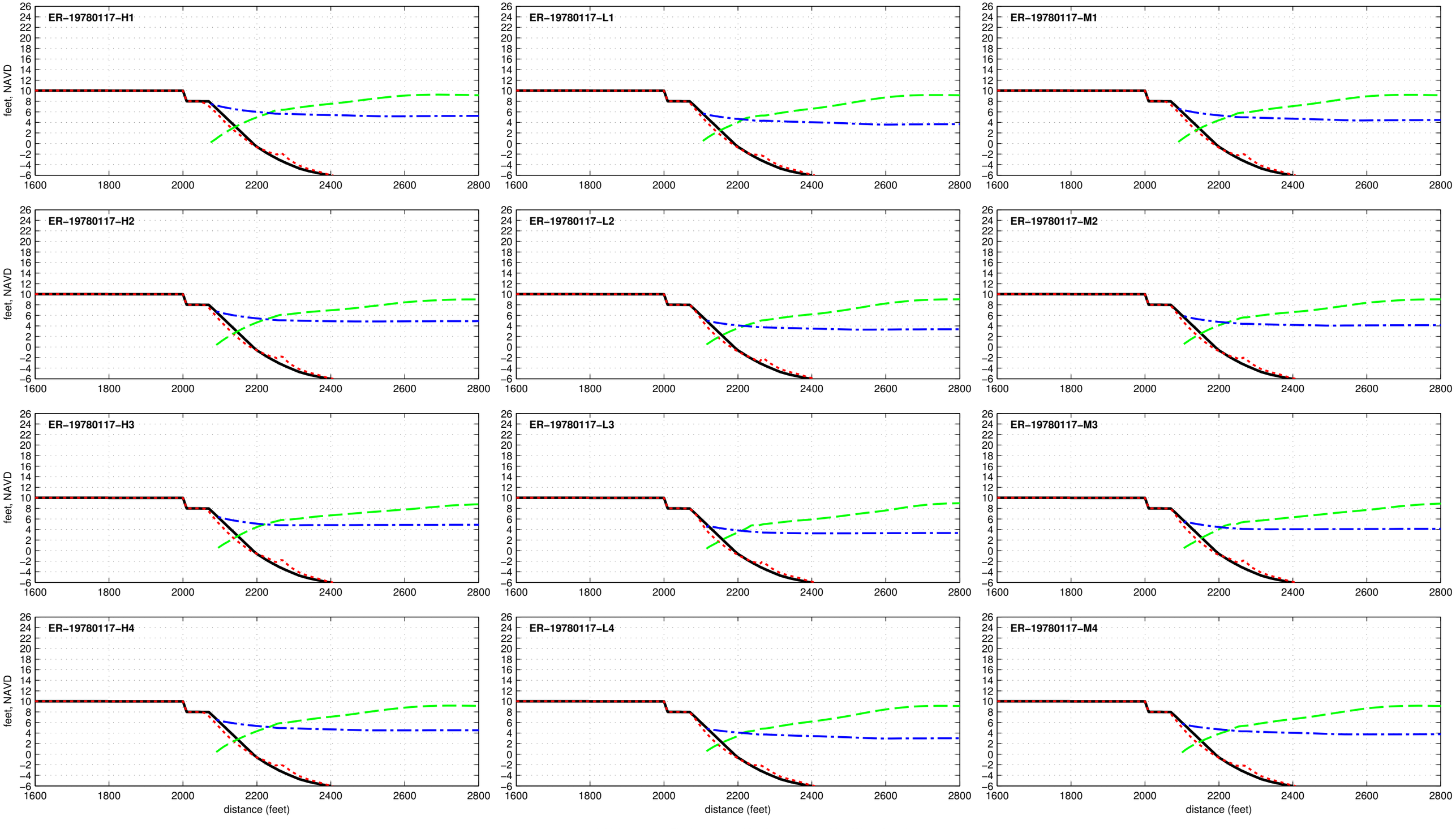




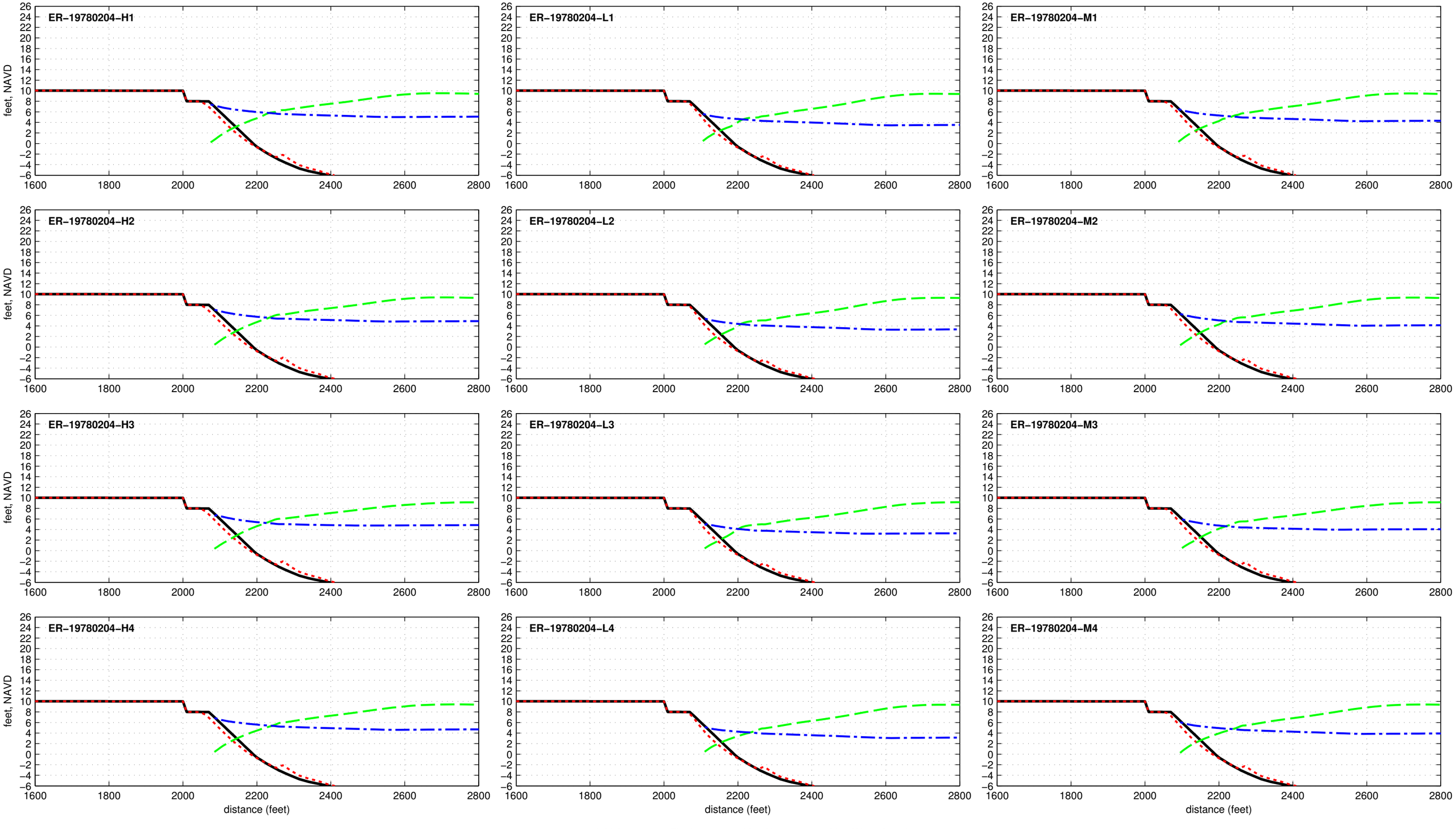


Initial Final Max Hs Max Water Level

Dune R2T1–10.0–000–060 – ER–19780117



Dune R2T1-10.0-000-060 - ER-19780204



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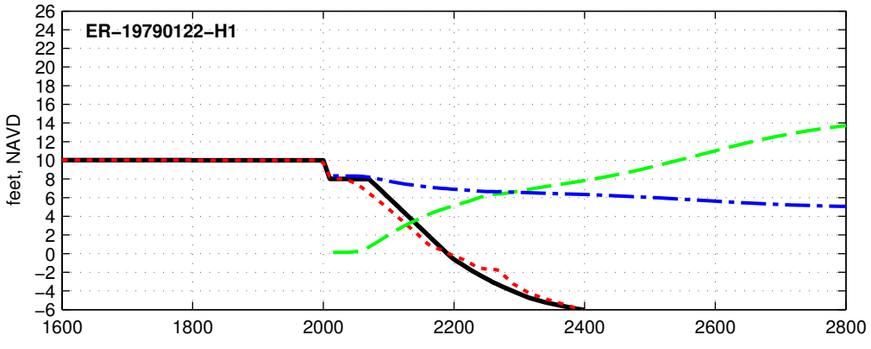
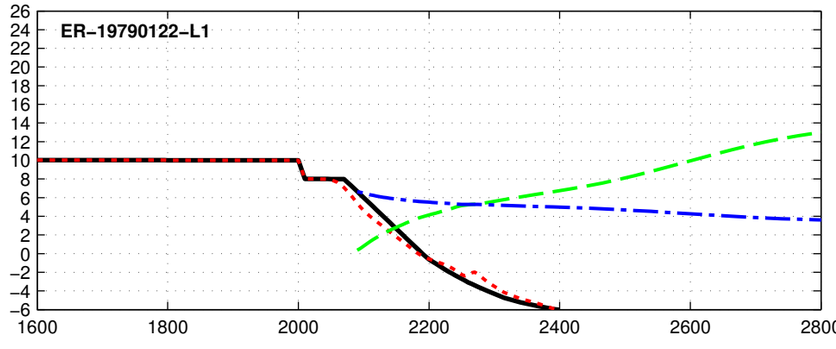
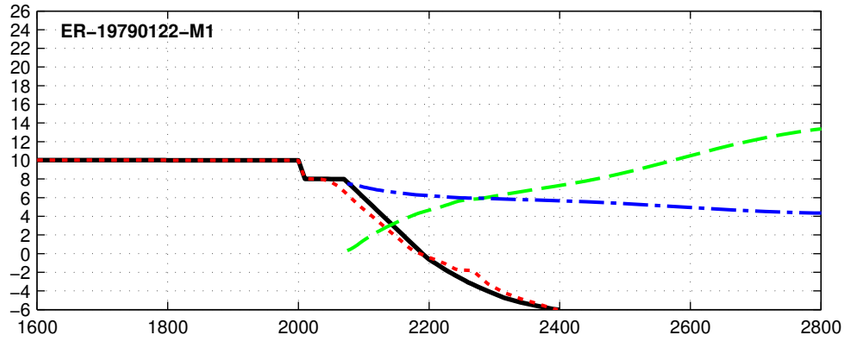
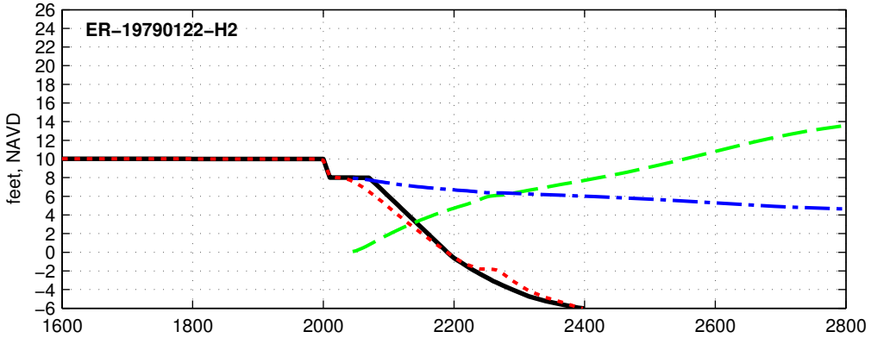
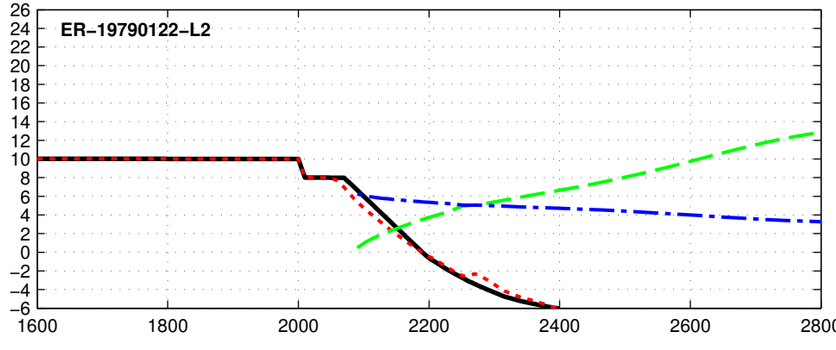
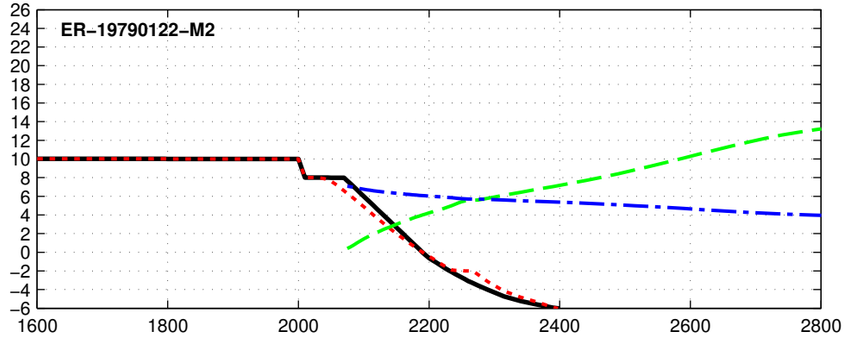


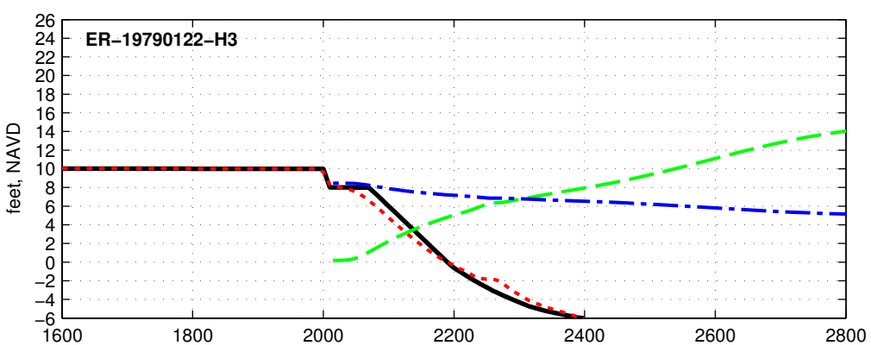


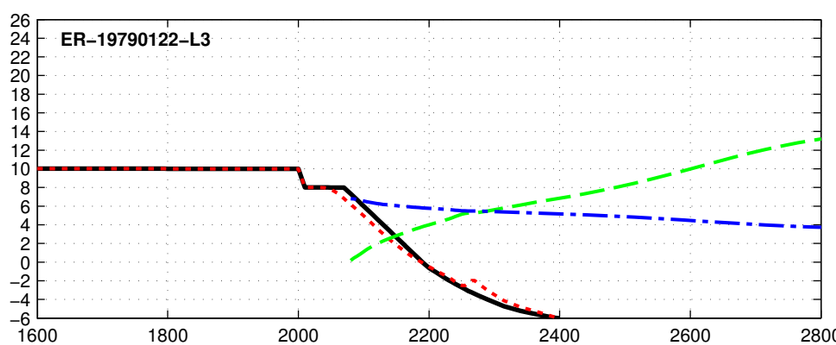


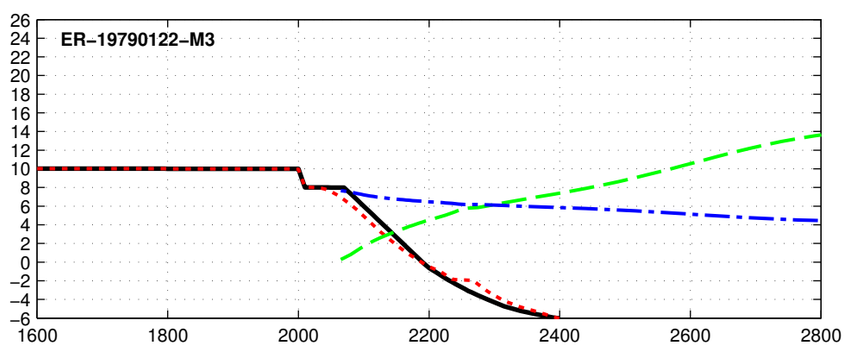


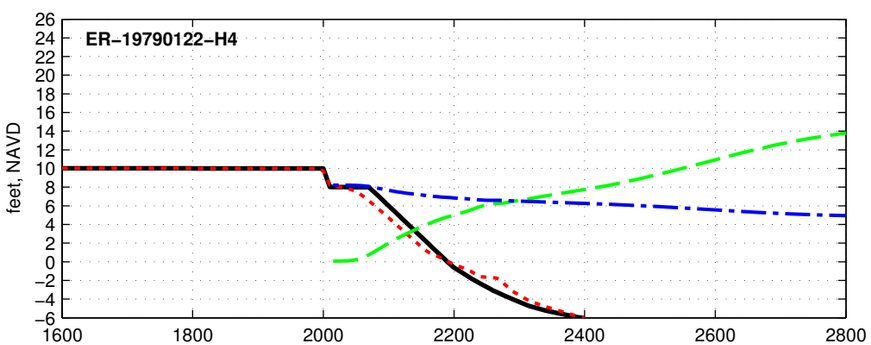


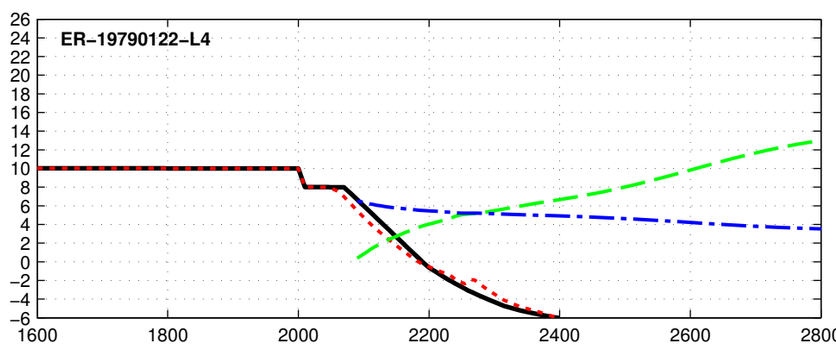


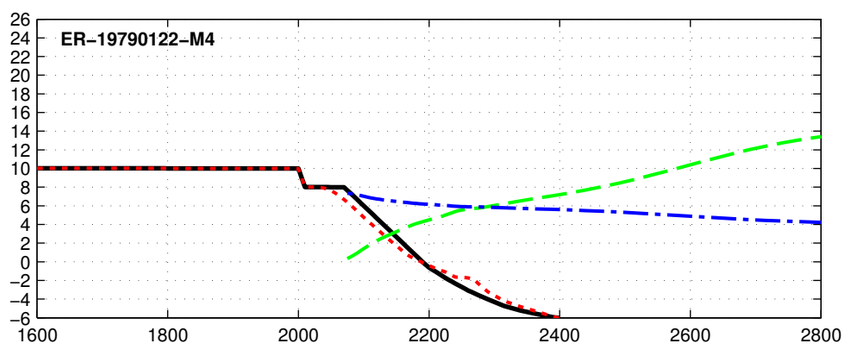










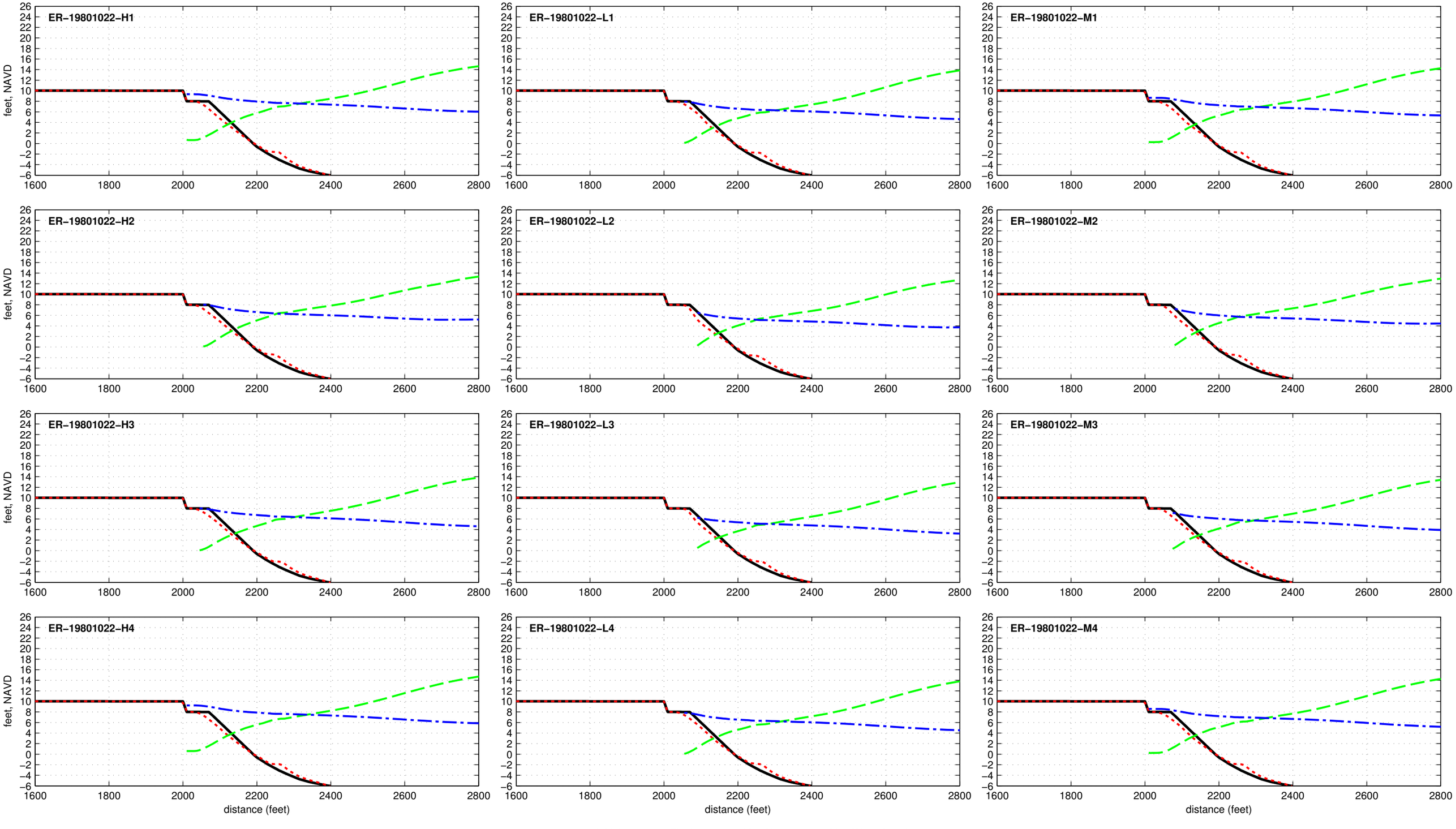


feet, NAVD

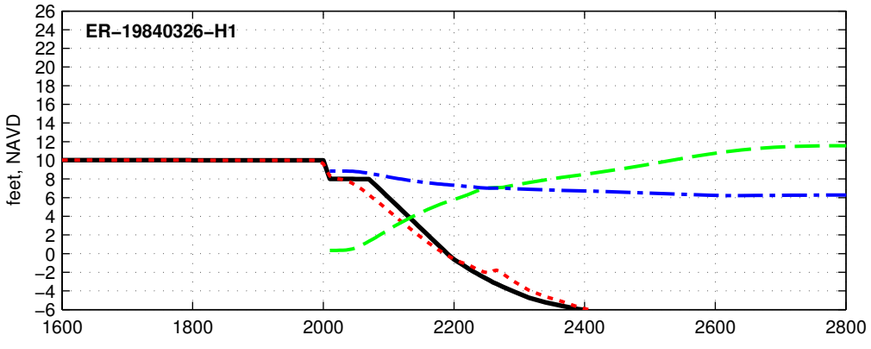
distance (feet)

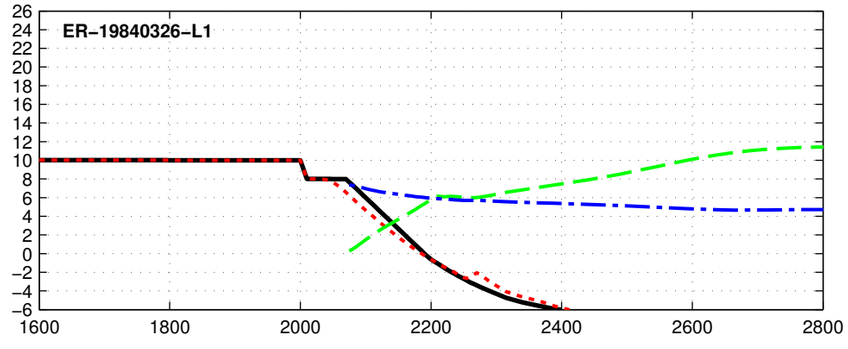
Initial Final Max Hs Max Water Level

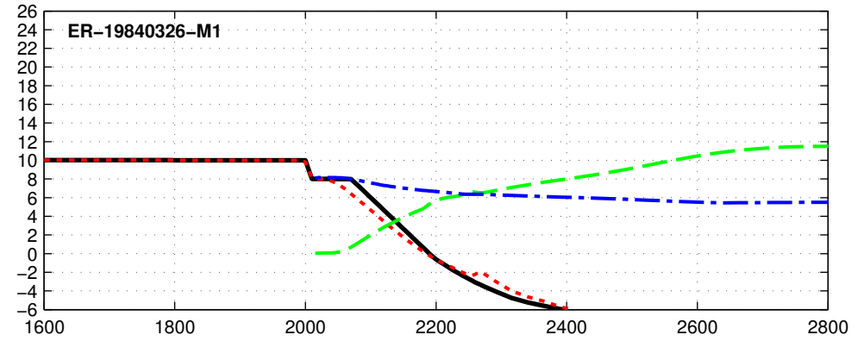
Dune R2T1-10.0-000-060 - ER-19801022

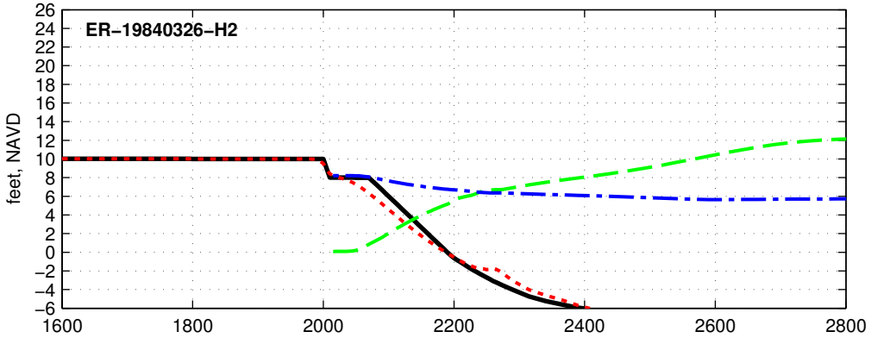


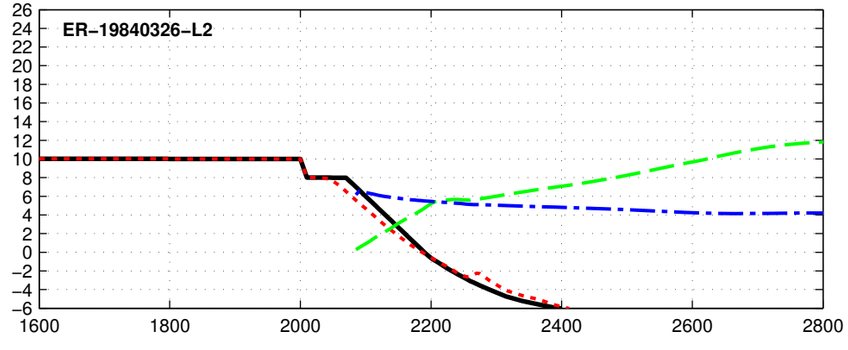
Dune R2T1-10.0-000-060 - ER-19840326

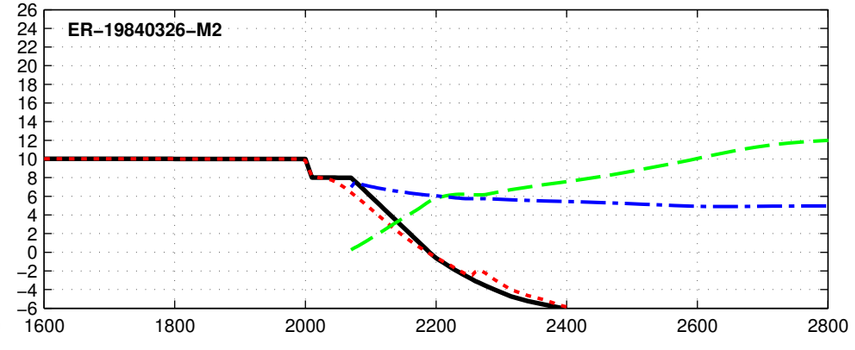


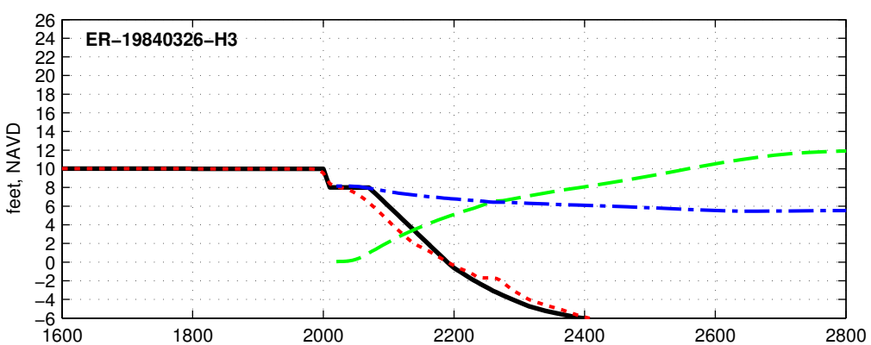


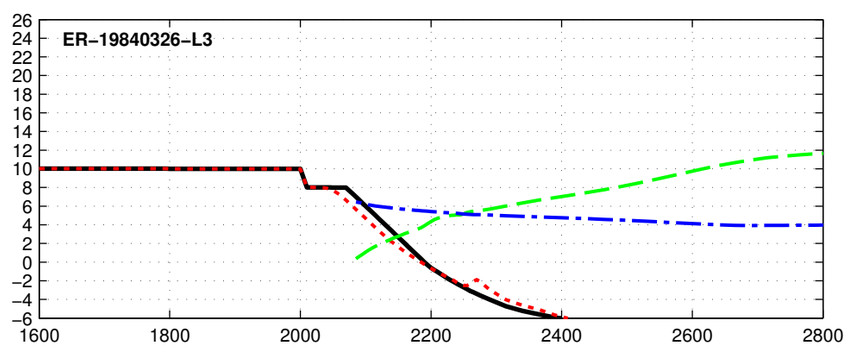


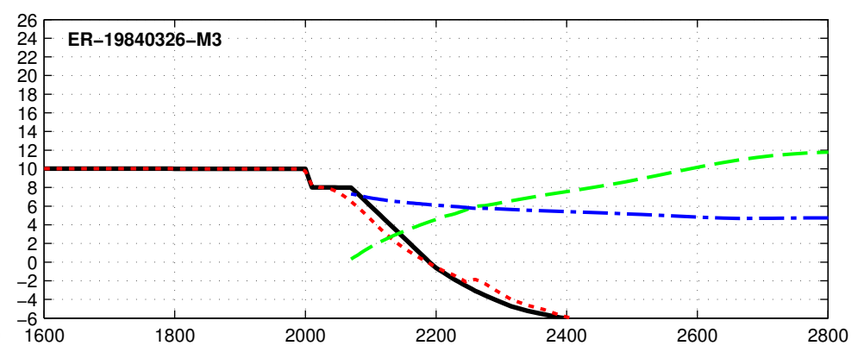


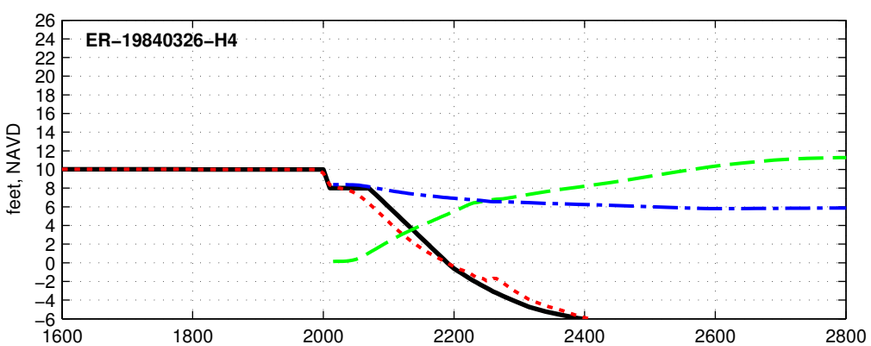


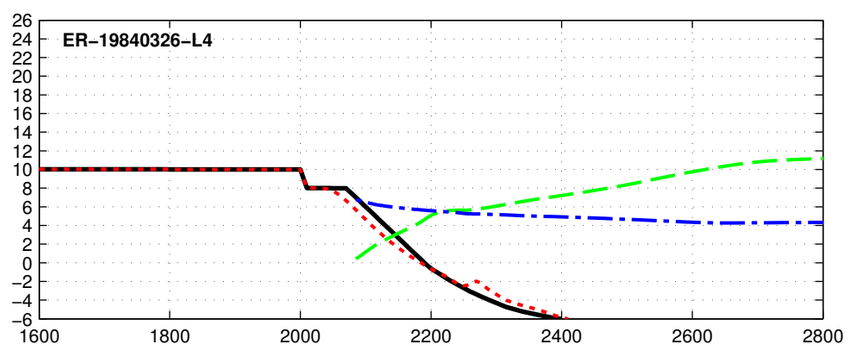


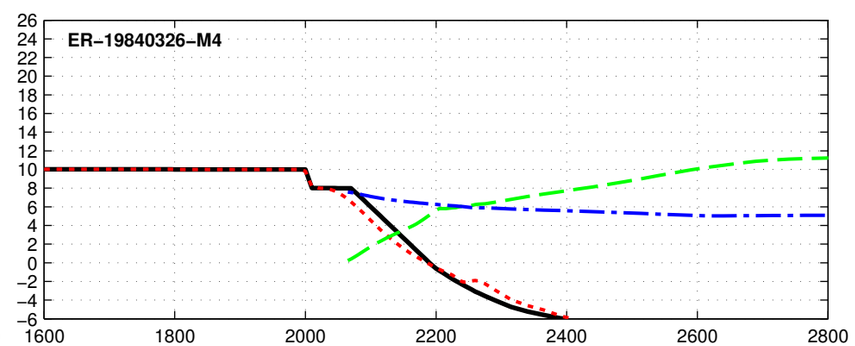






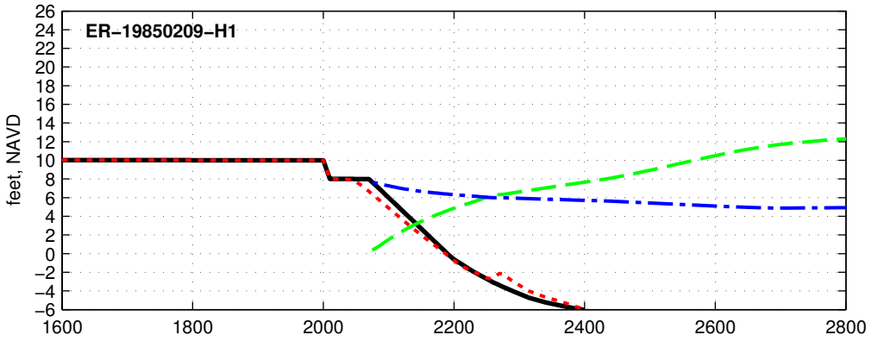


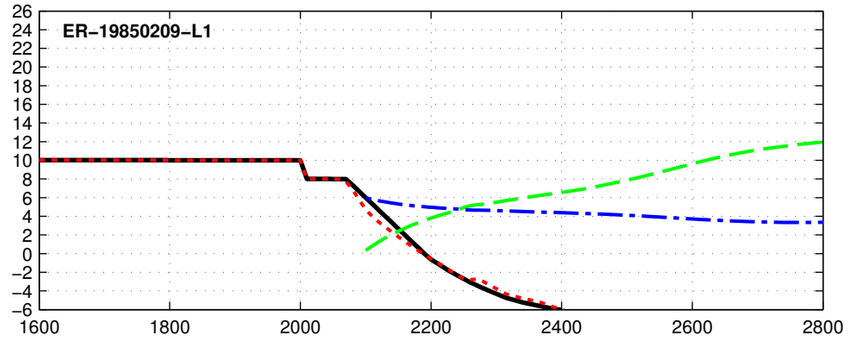


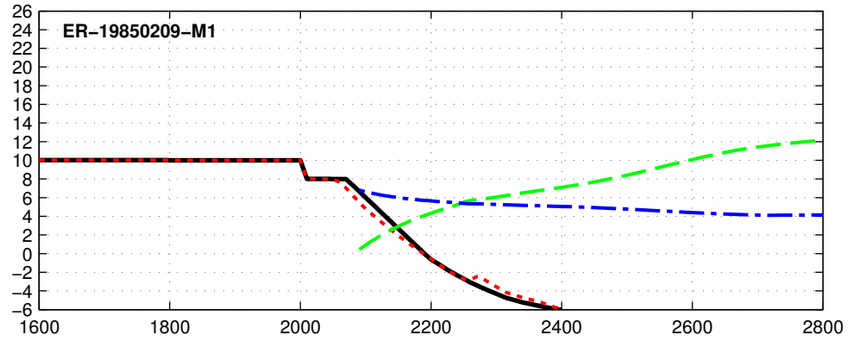


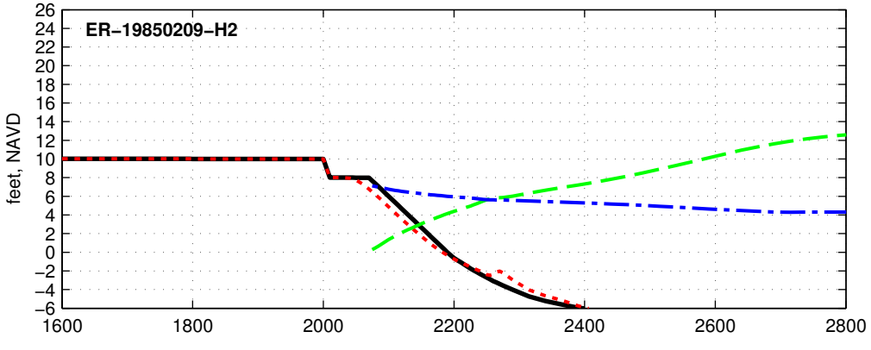
Initial Final Max Hs Max Water Level

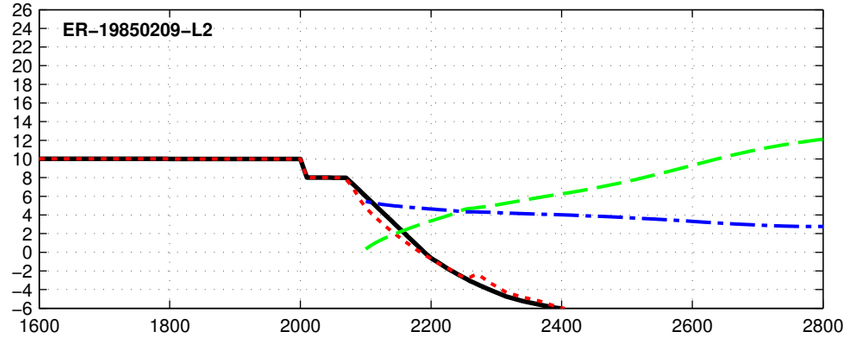
Dune R2T1-10.0-000-060 - ER-19850209

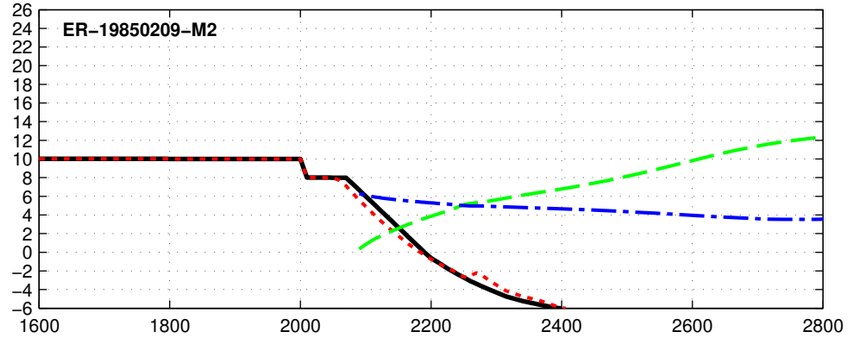


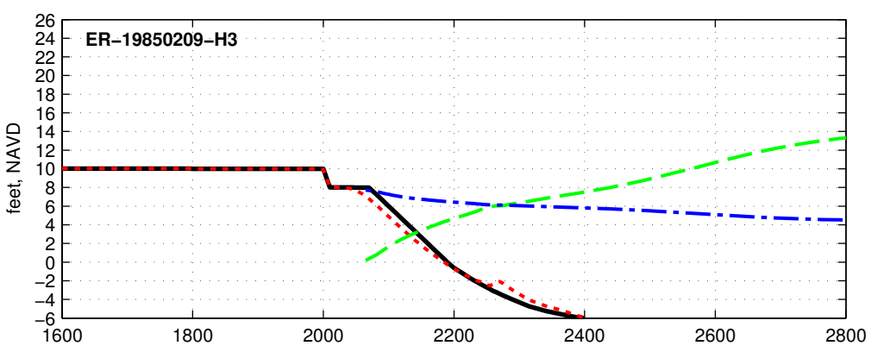


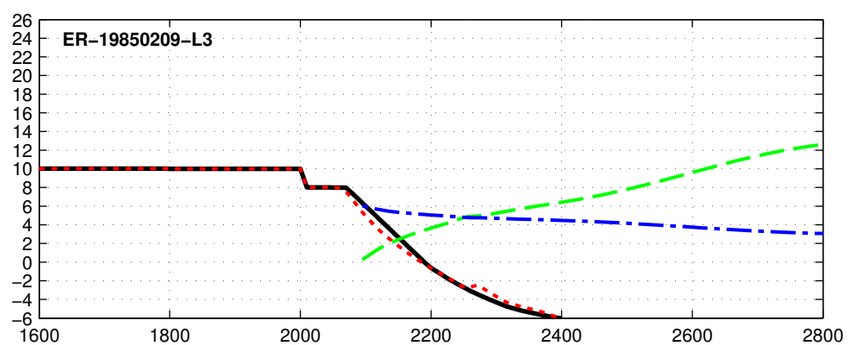


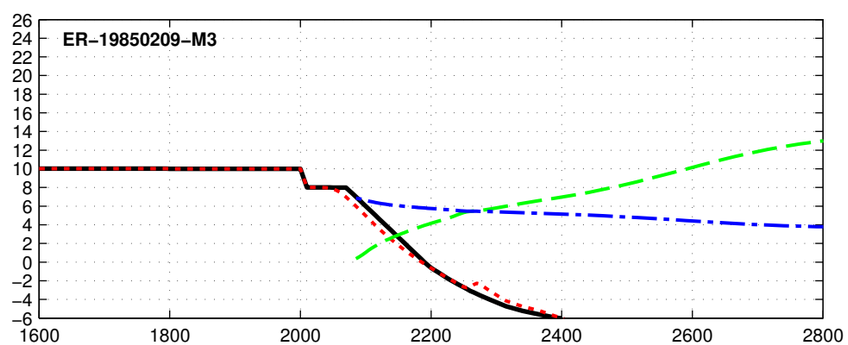


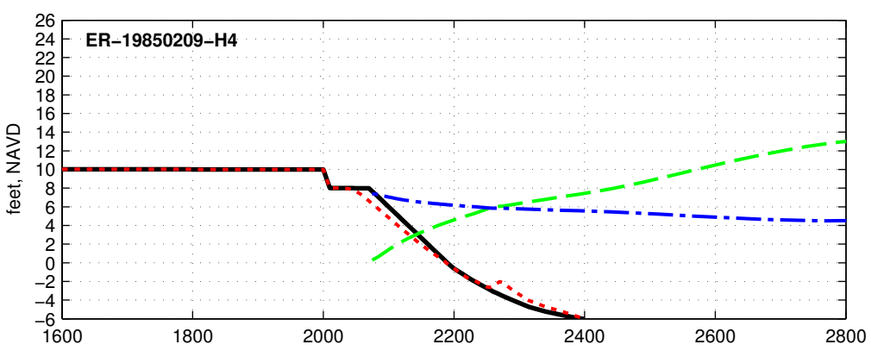


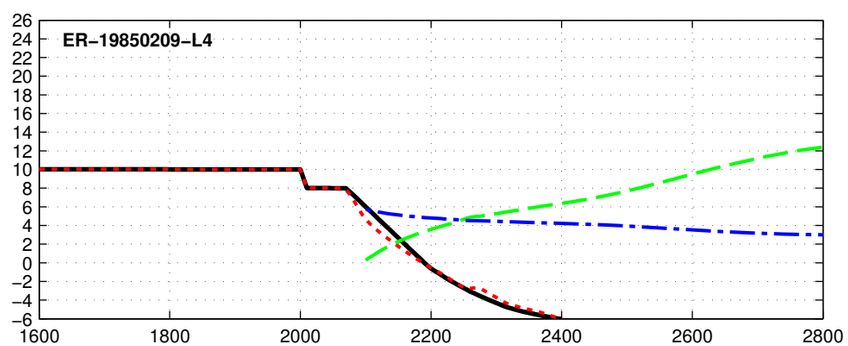


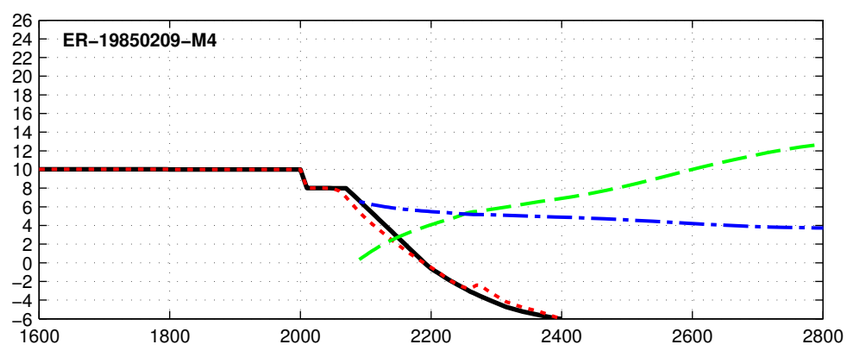






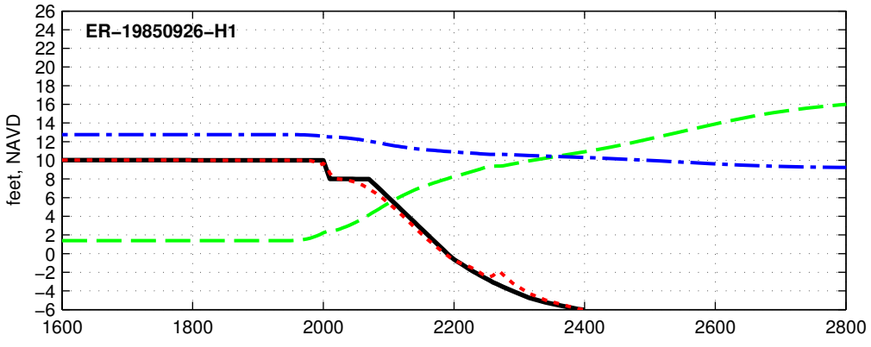


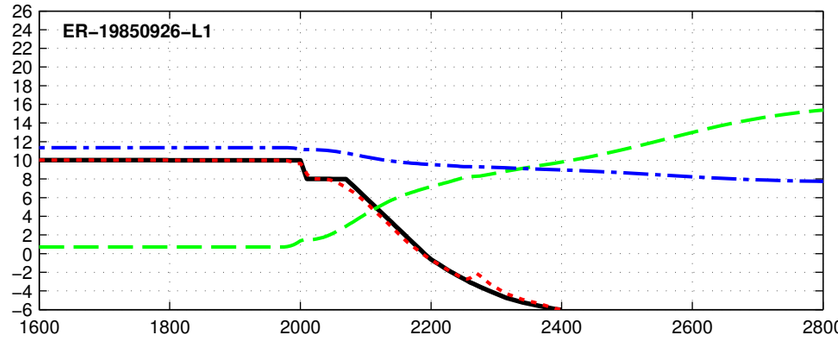


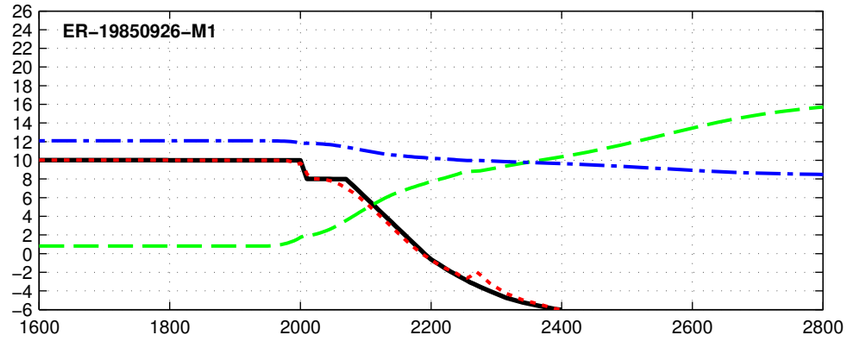


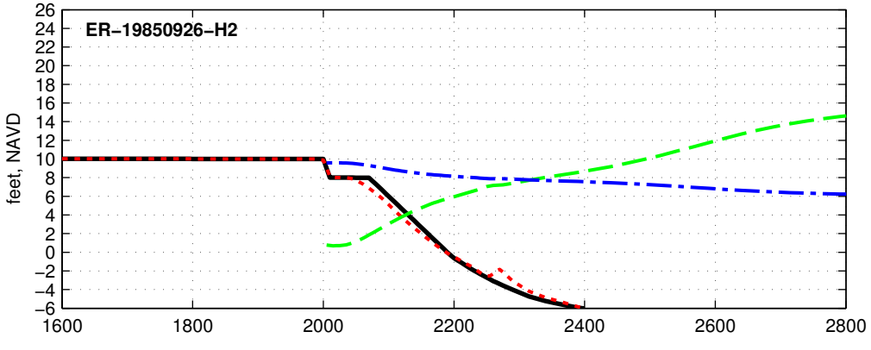
Initial Final Max Hs Max Water Level

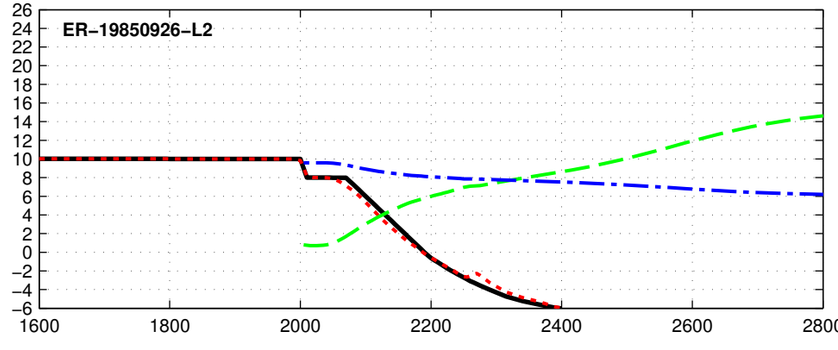
Dune R2T1-10.0-000-060 - ER-19850926

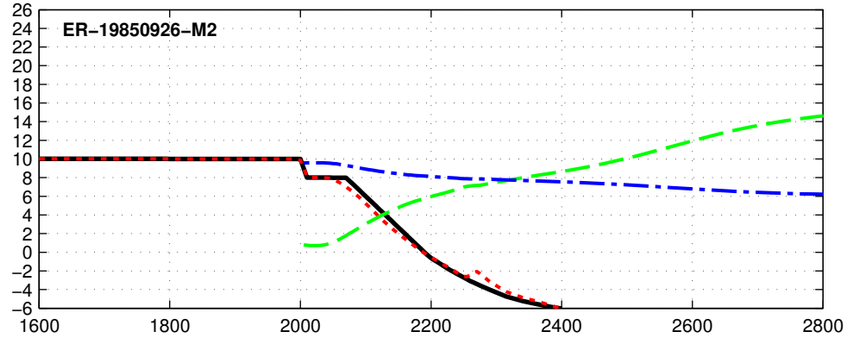


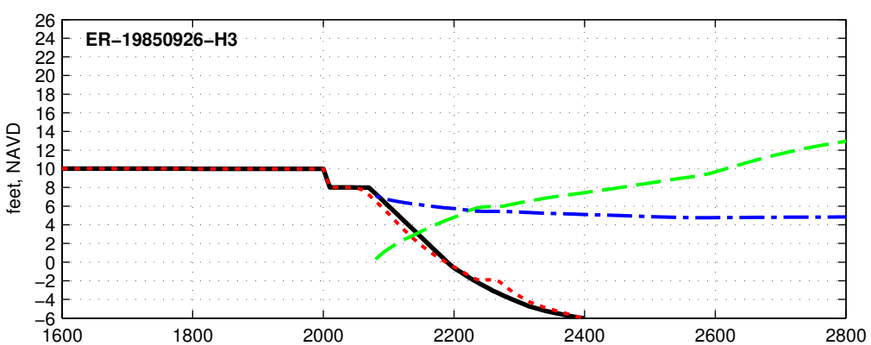


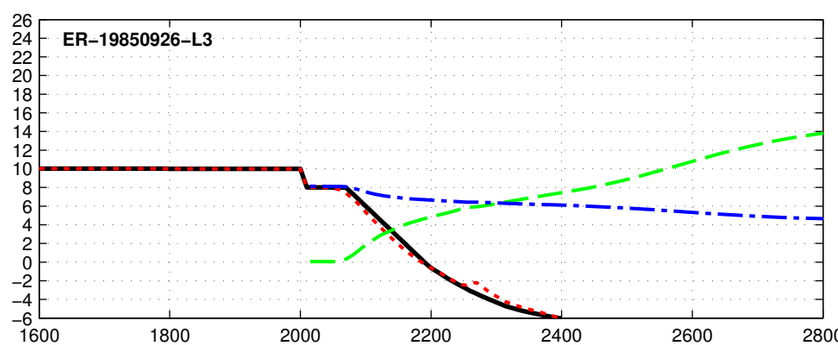


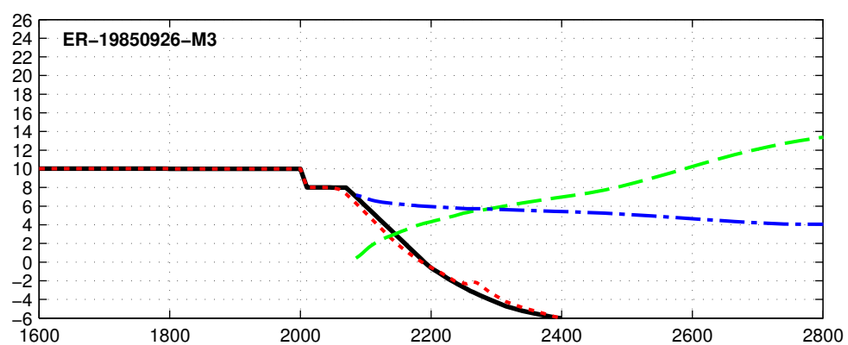


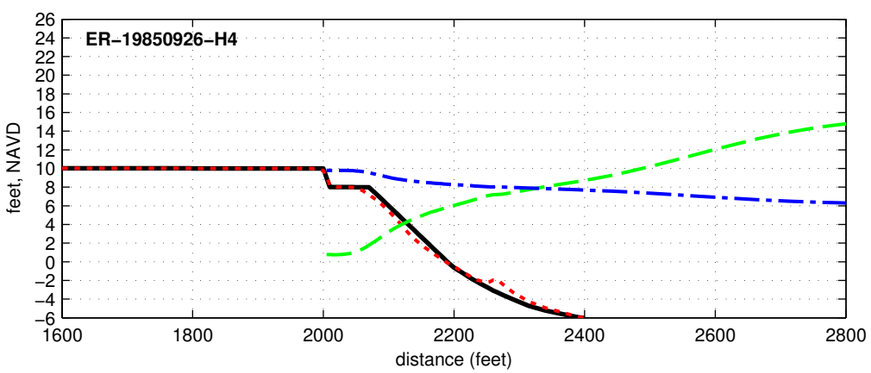


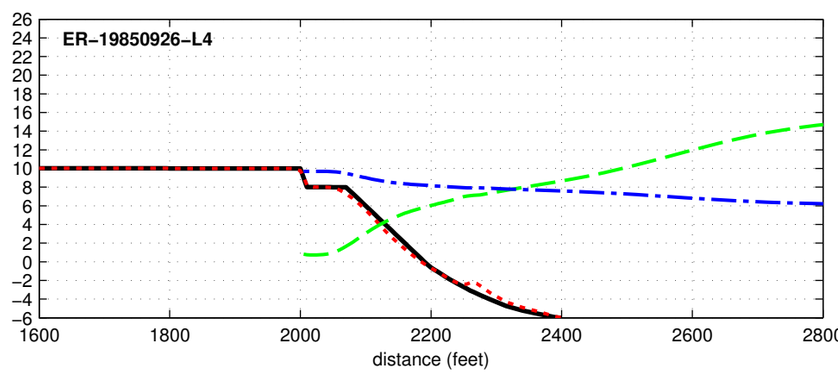


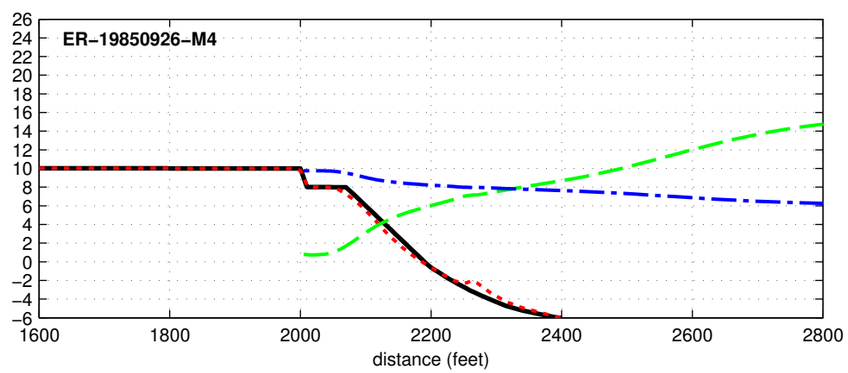






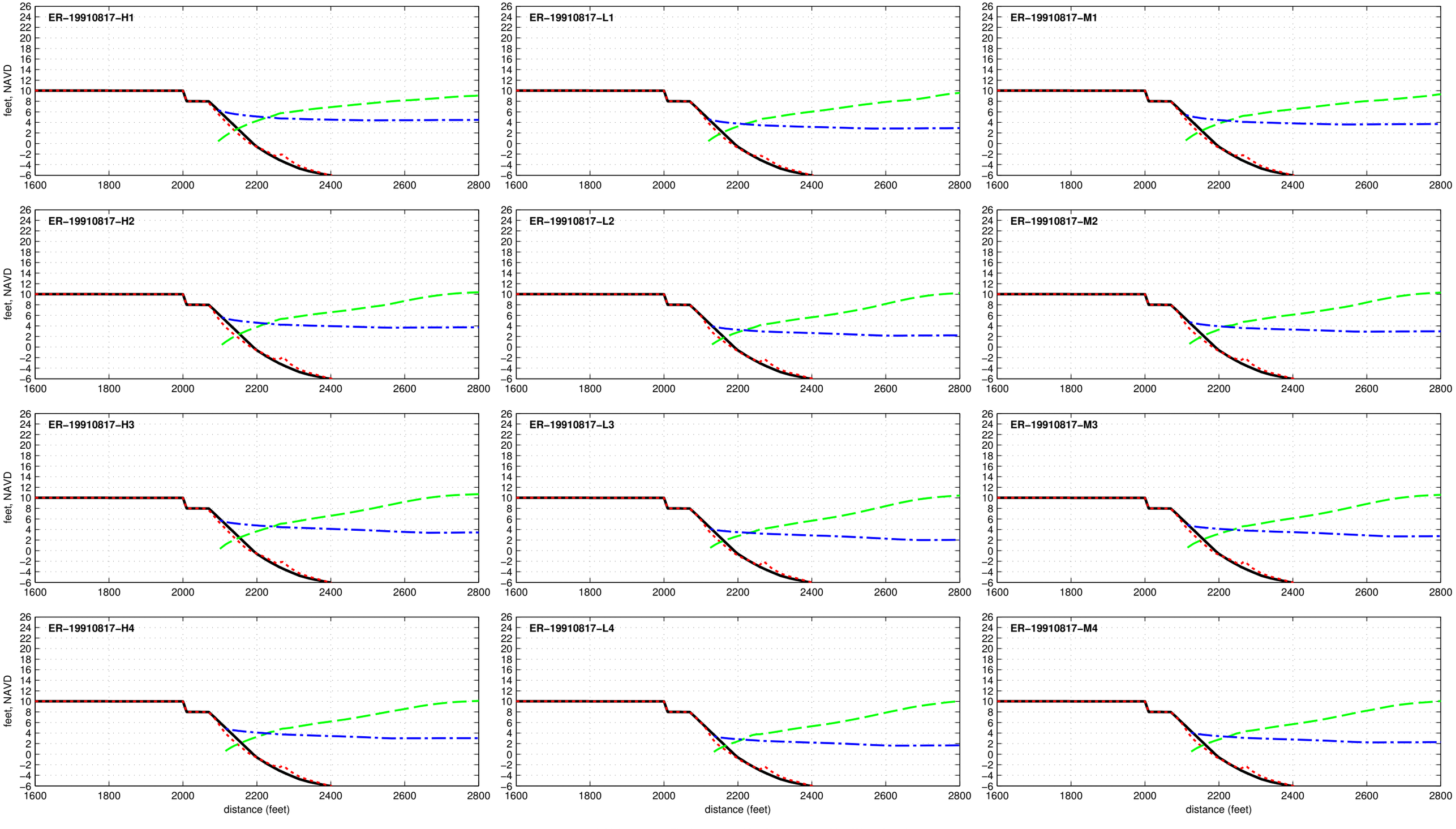




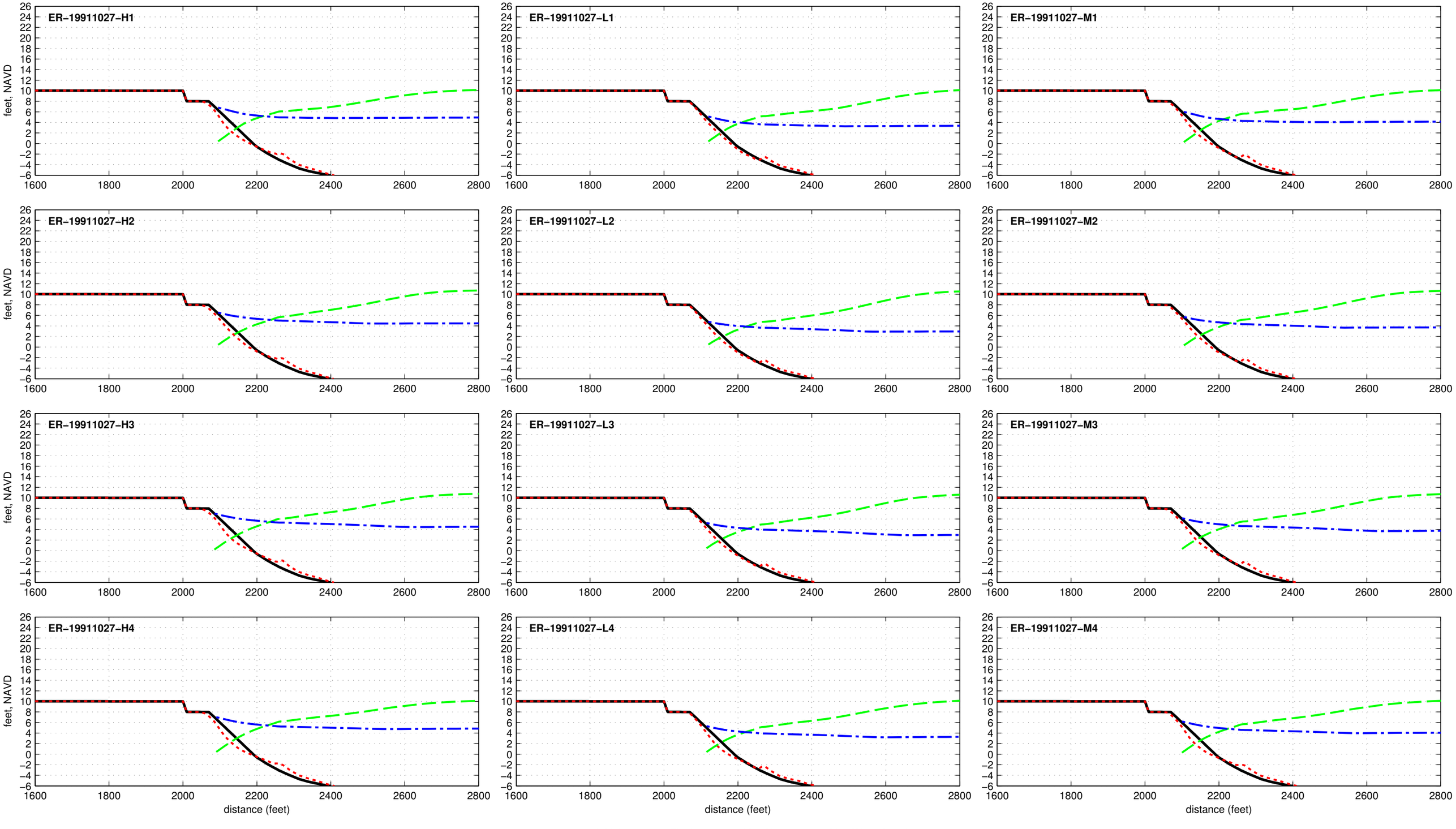


Initial Final Max Hs Max Water Level

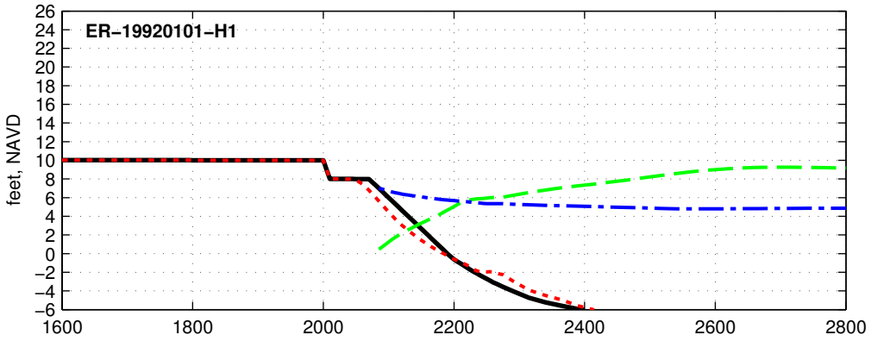
Dune R2T1-10.0-000-060 - ER-19910817

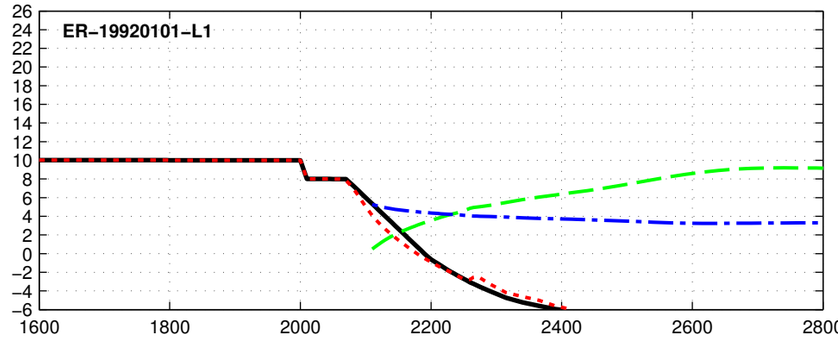


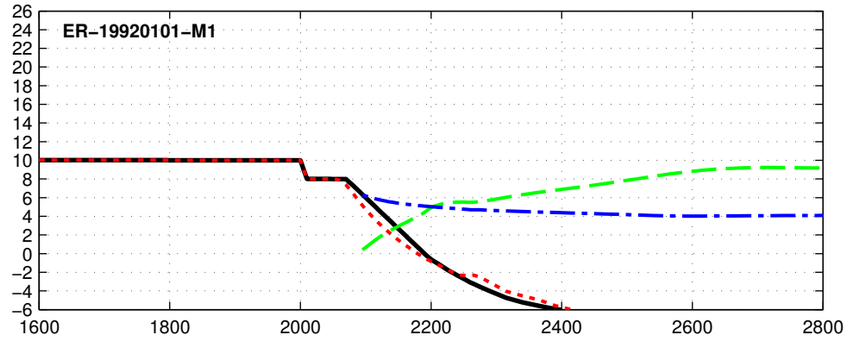
Dune R2T1-10.0-000-060 - ER-19911027

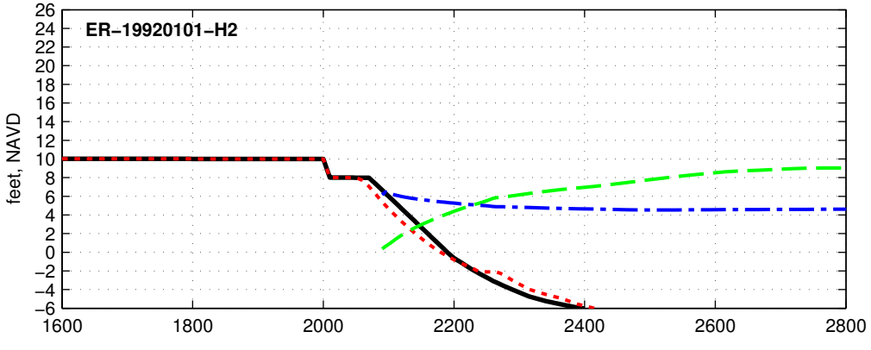


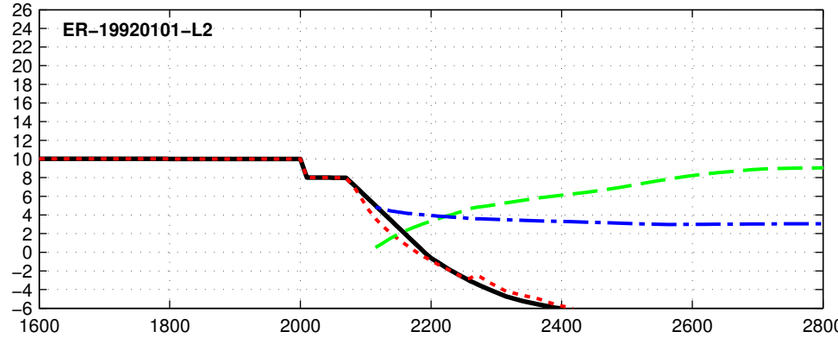
Dune R2T1-10.0-000-060 - ER-19920101

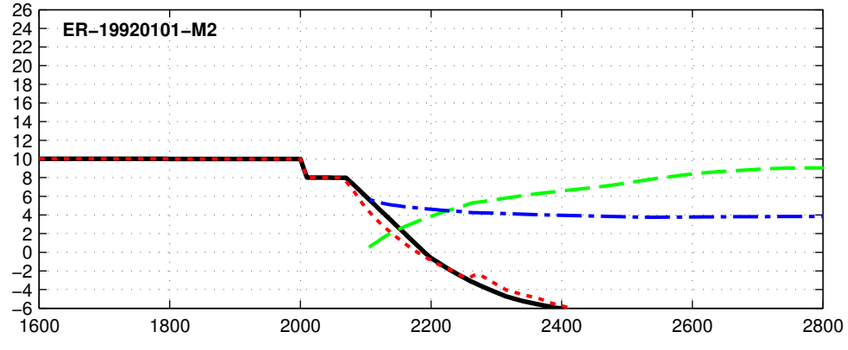


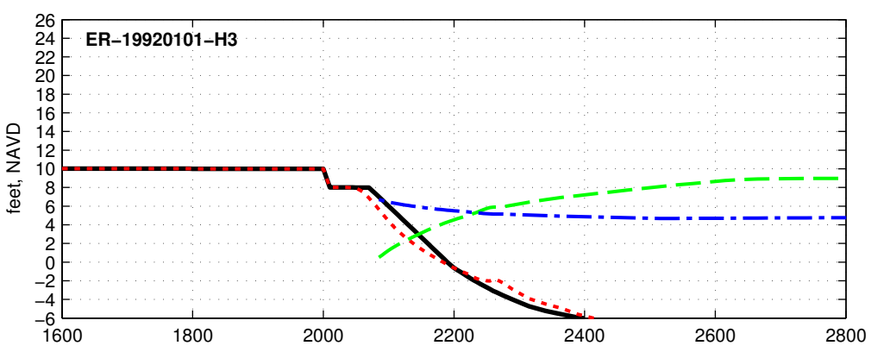


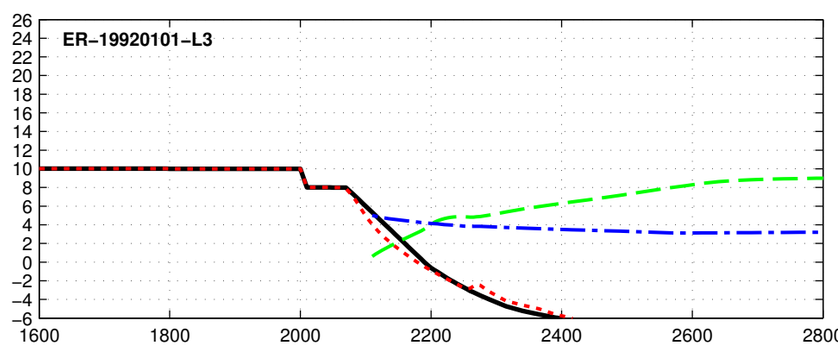


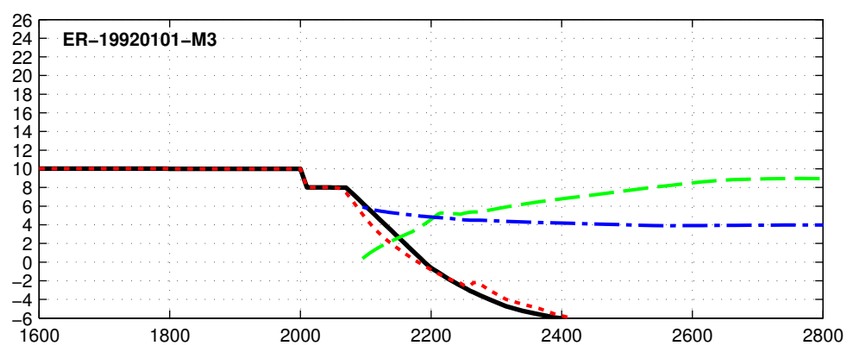


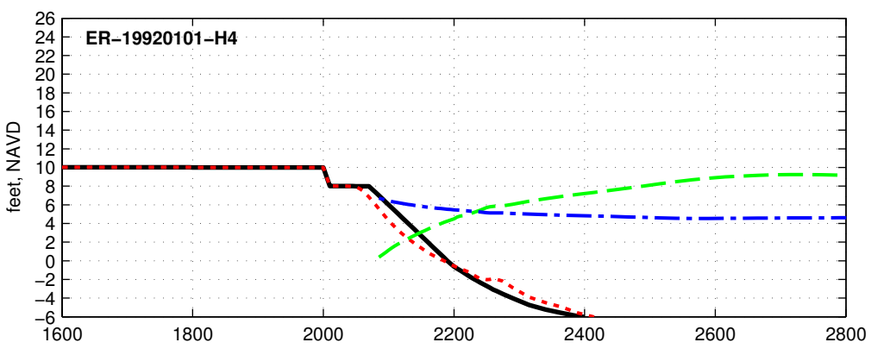


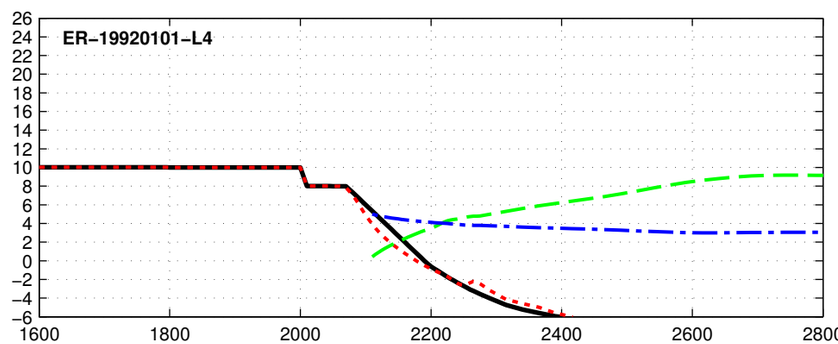


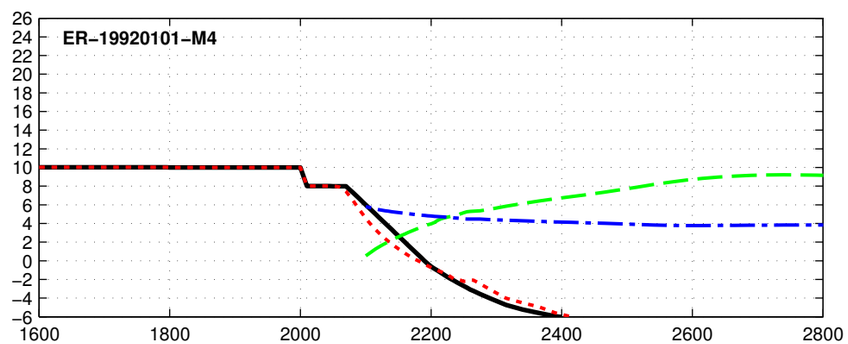






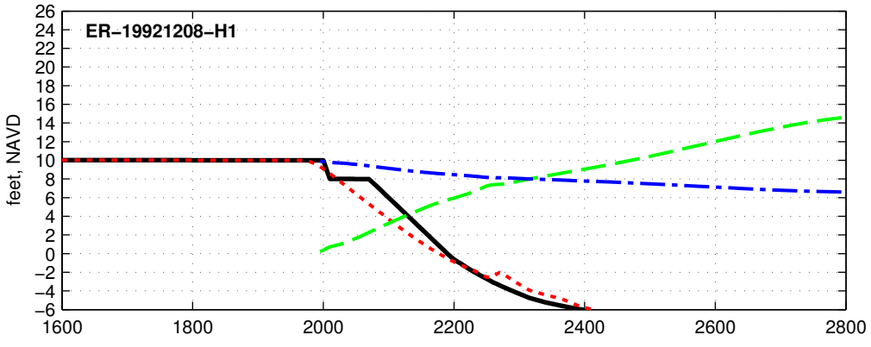


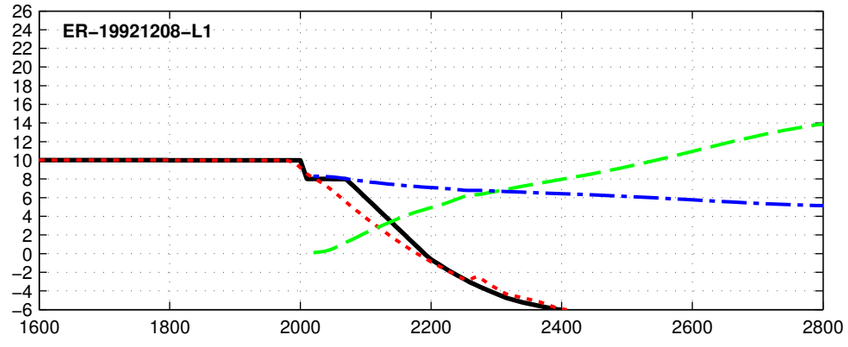


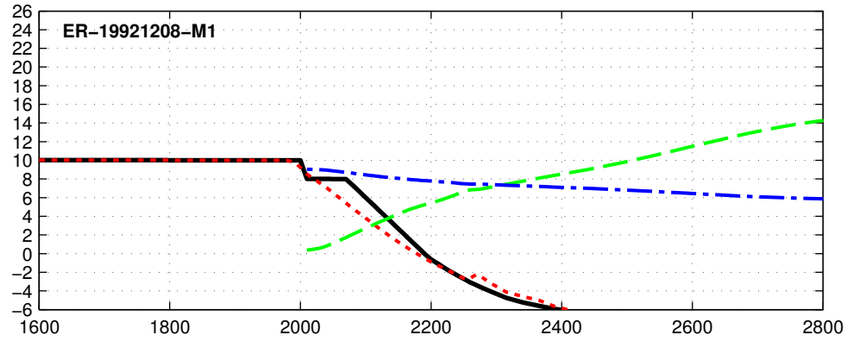


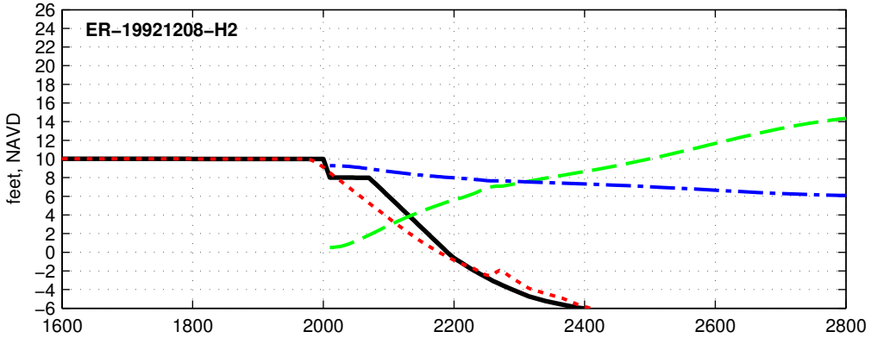
Initial Final Max Hs Max Water Level

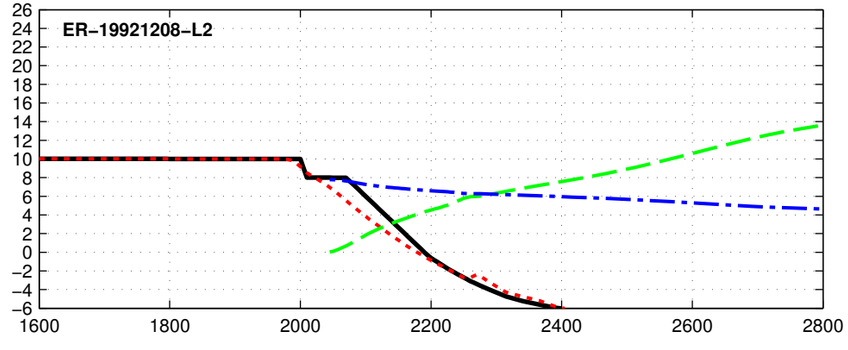
Dune R2T1-10.0-000-060 - ER-19921208

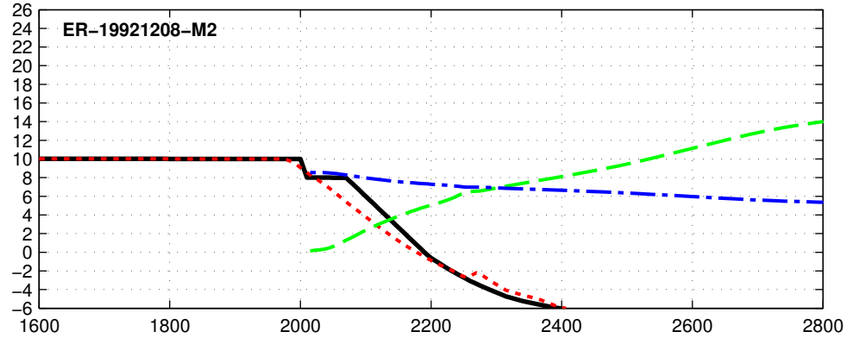


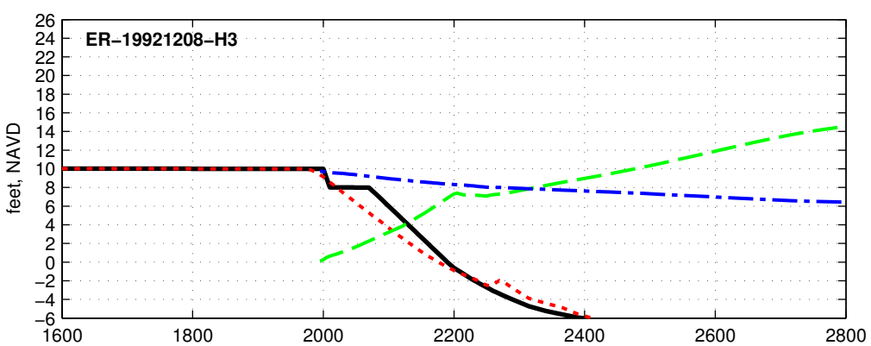


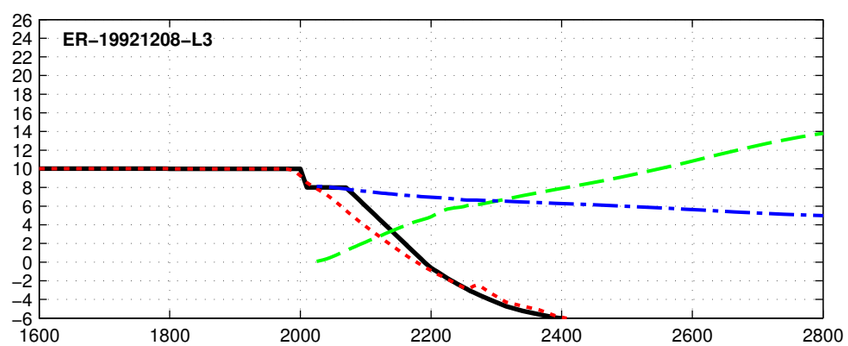


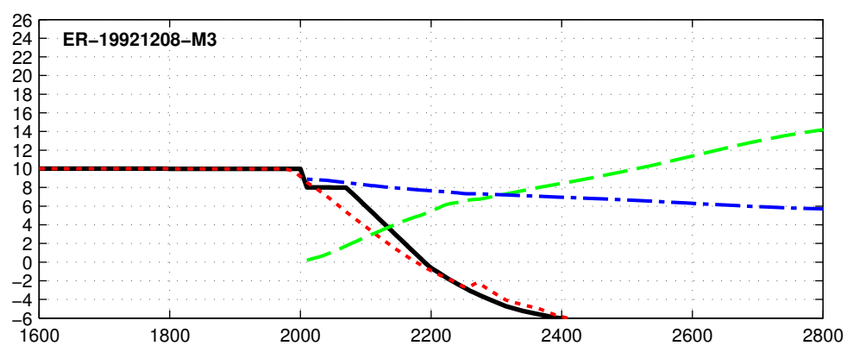


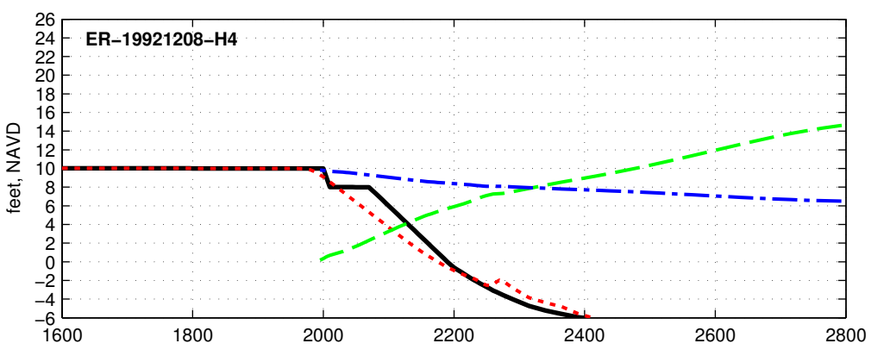


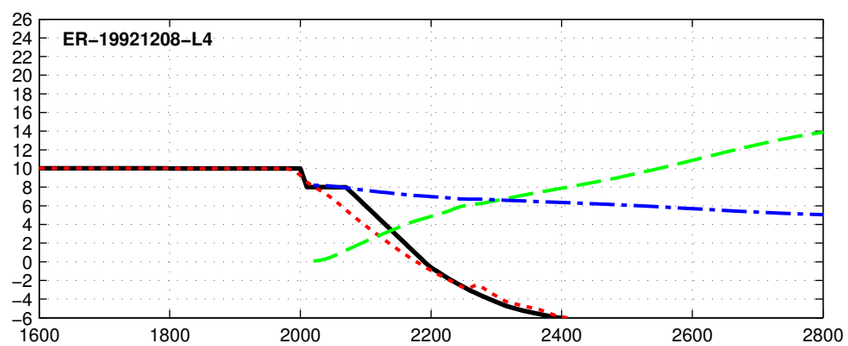


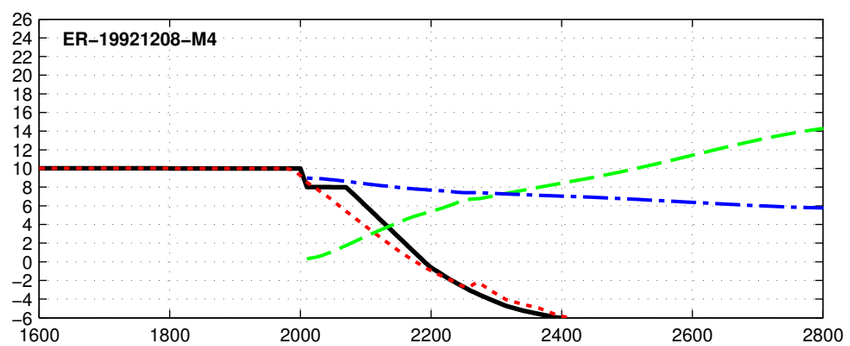






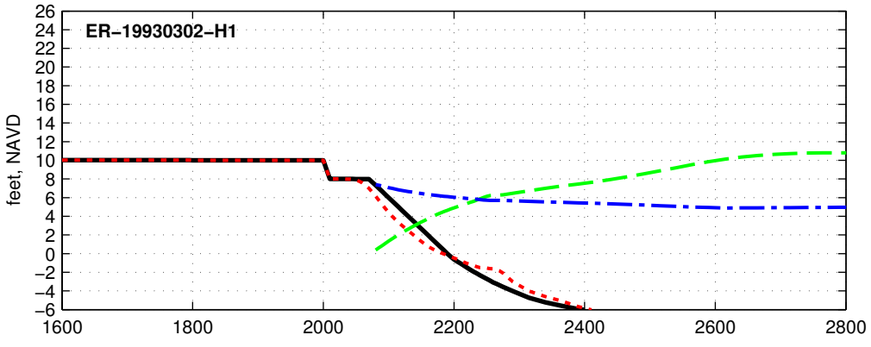


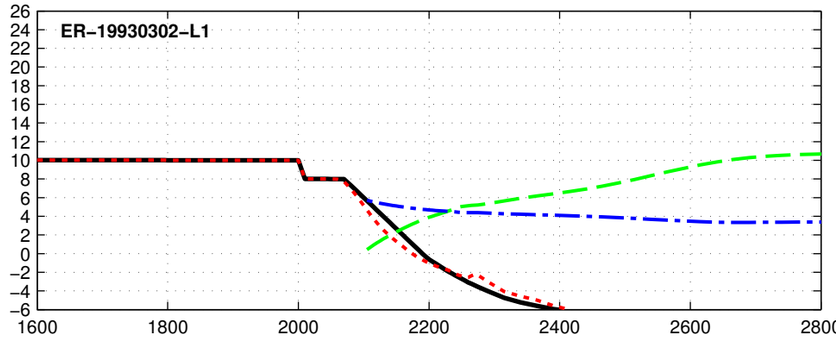


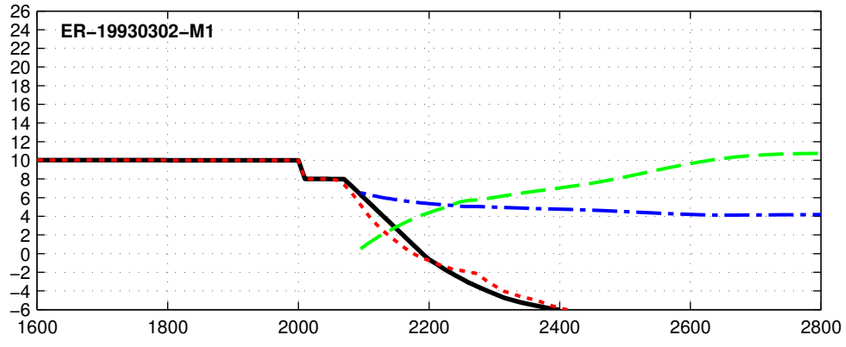


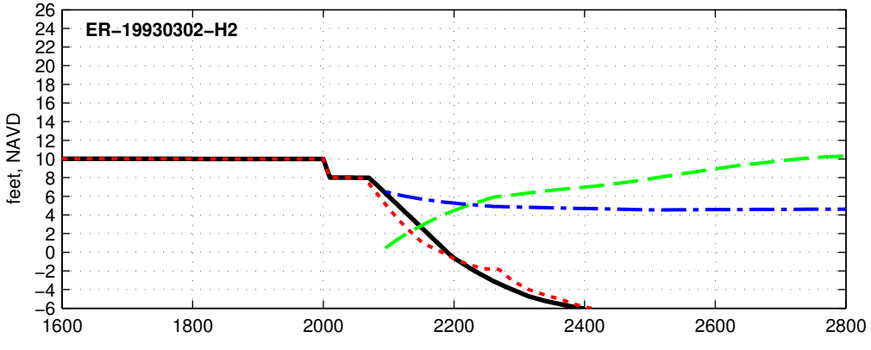
Initial Final Max Hs Max Water Level

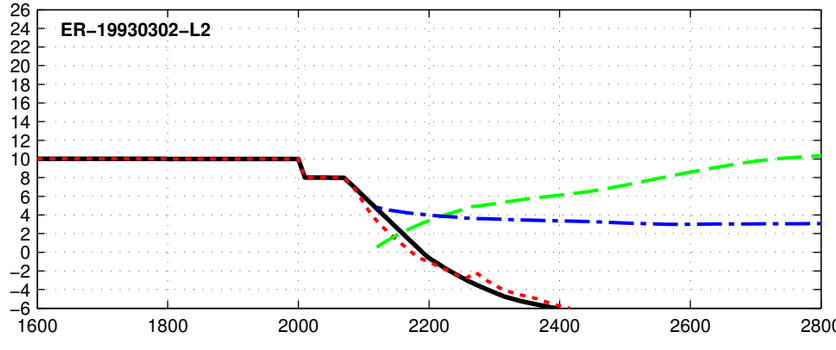
Dune R2T1-10.0-000-060 - ER-19930302

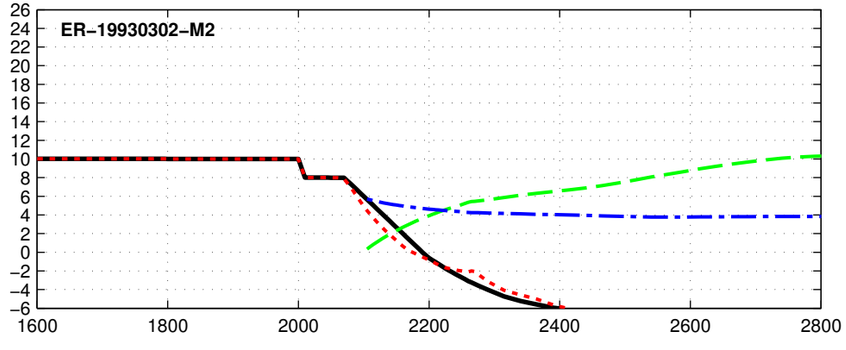


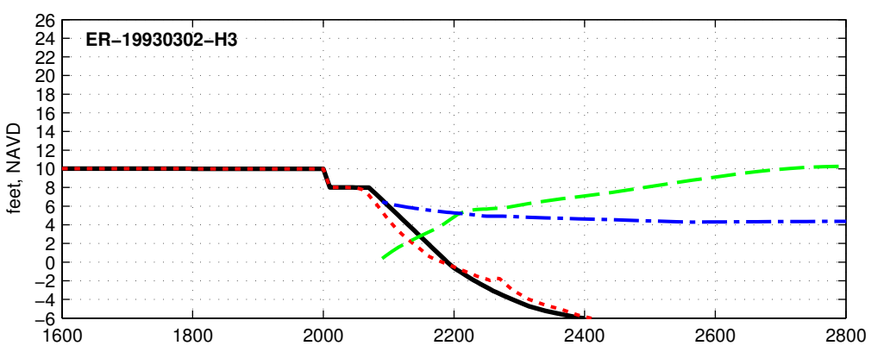


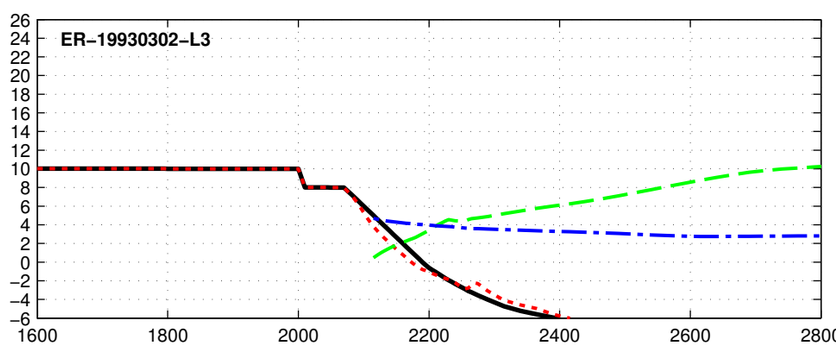


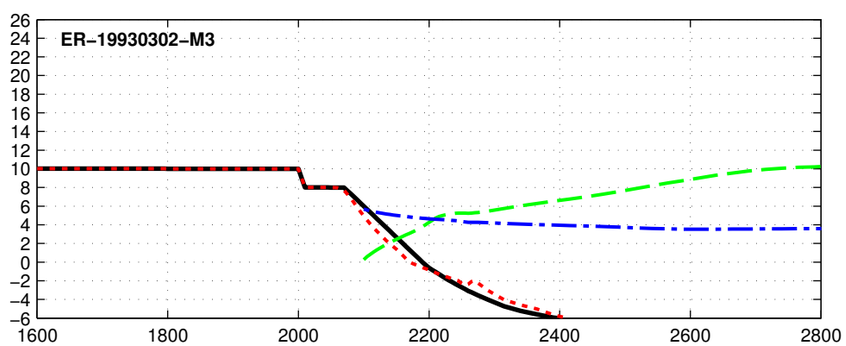


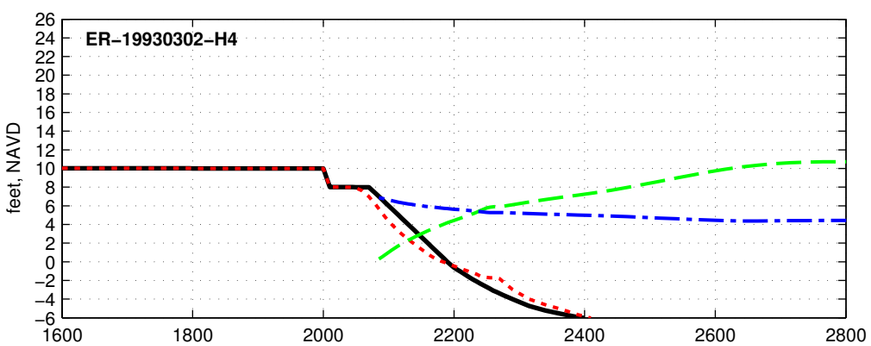


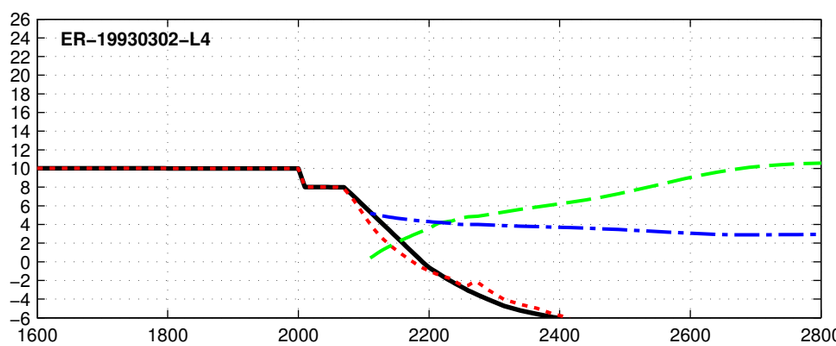


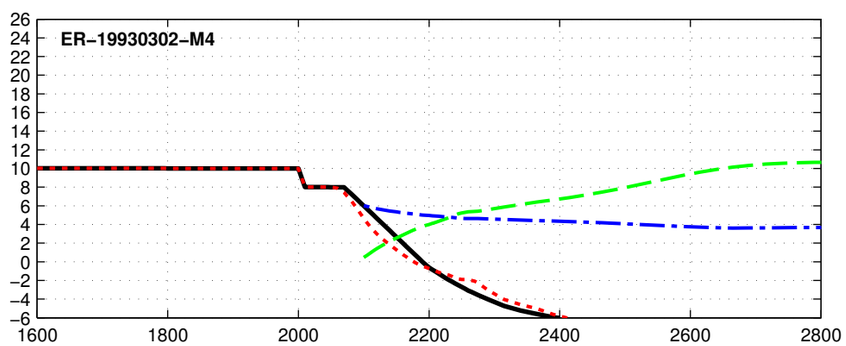






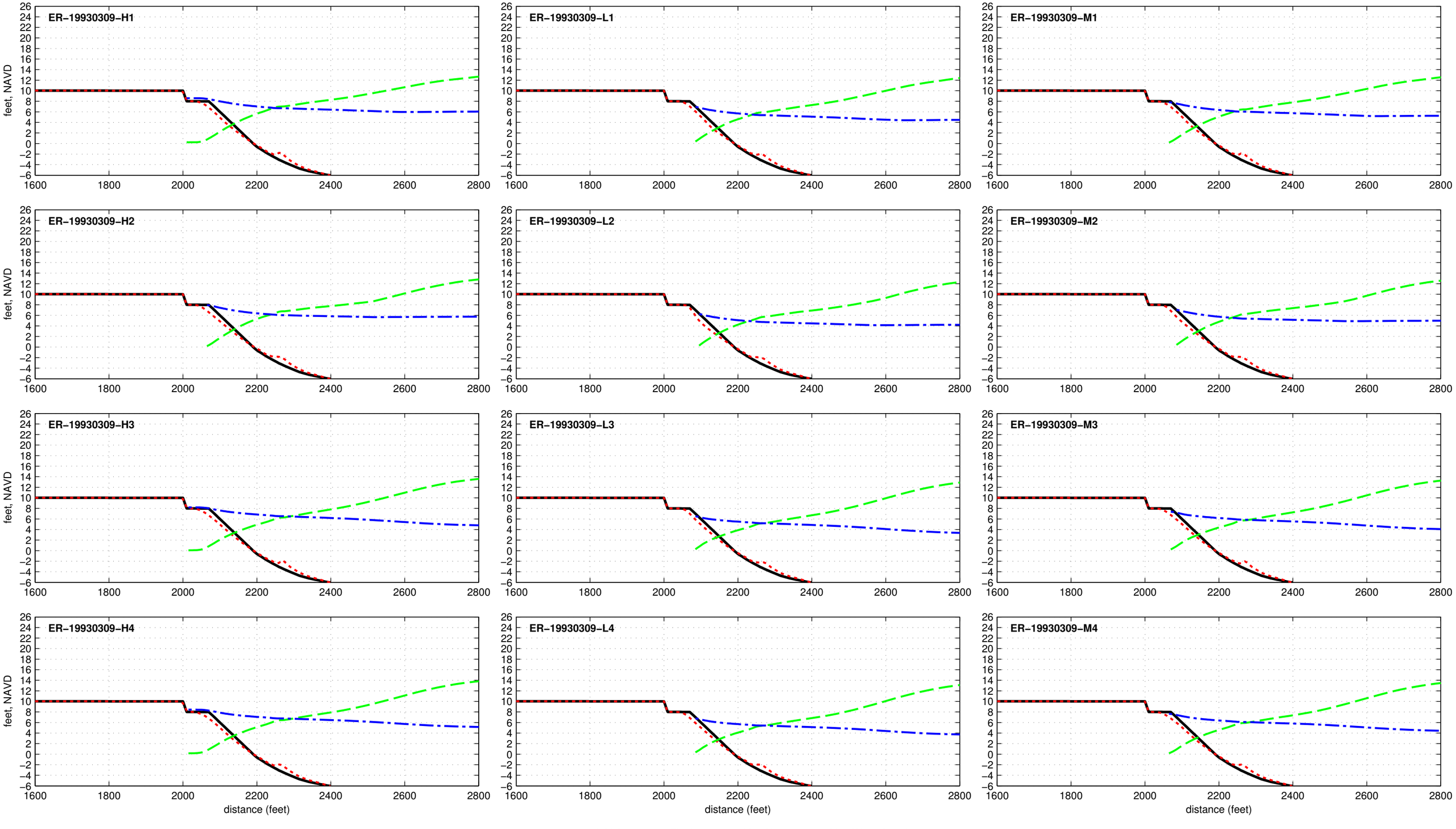




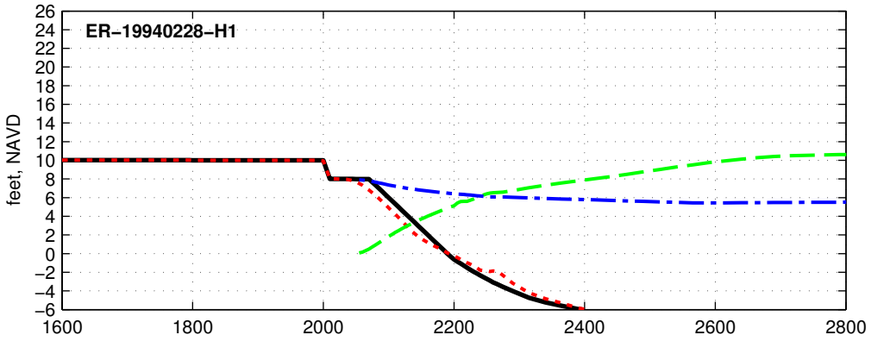


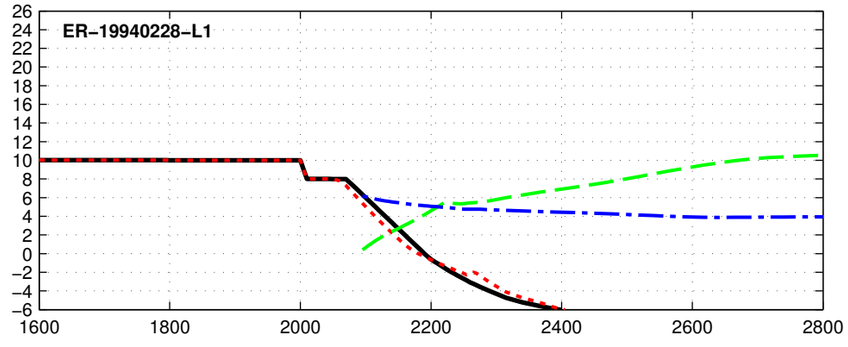
Initial Final Max Hs Max Water Level

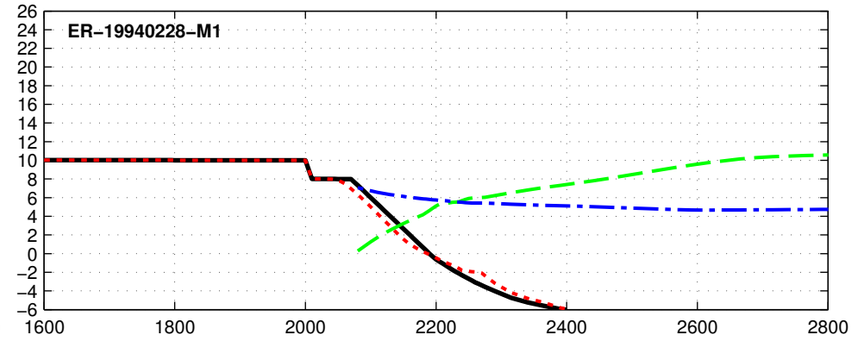
Dune R2T1-10.0-000-060 - ER-19930309

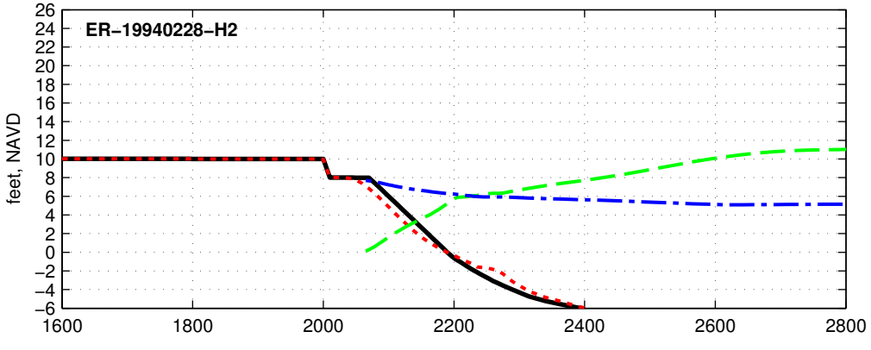


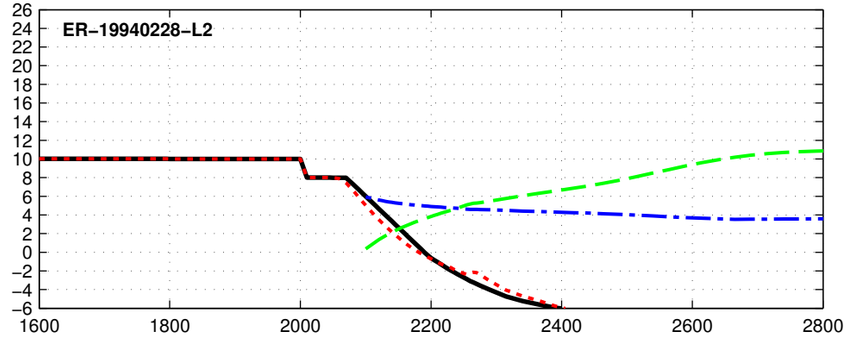
Dune R2T1-10.0-000-060 - ER-19940228

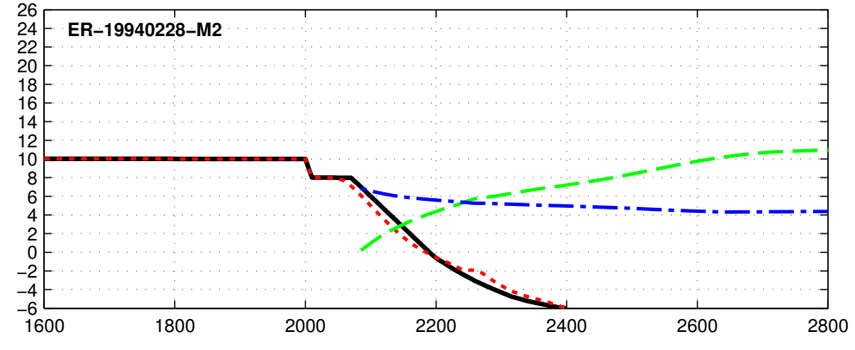


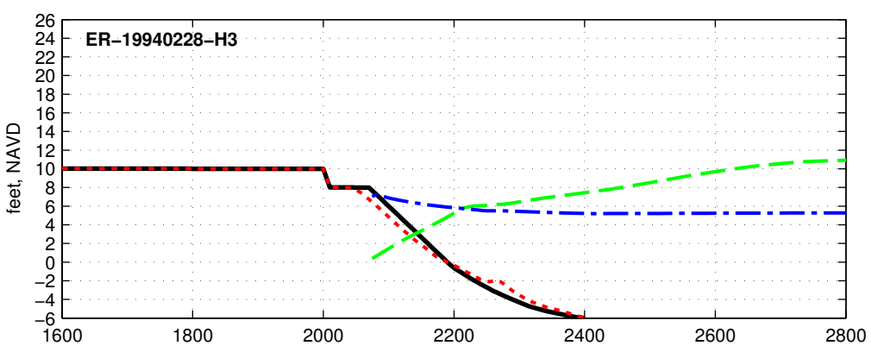


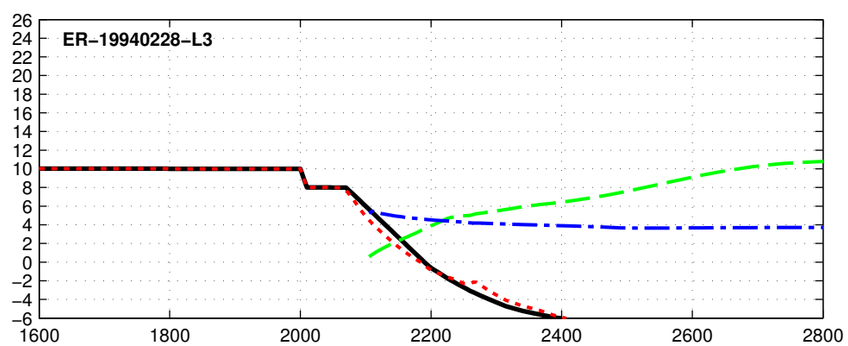


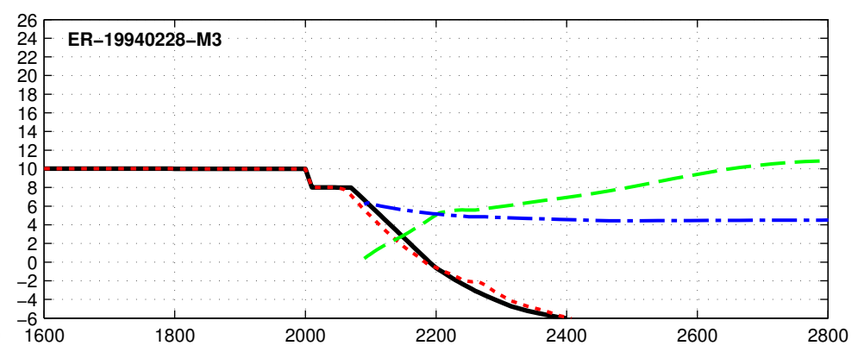


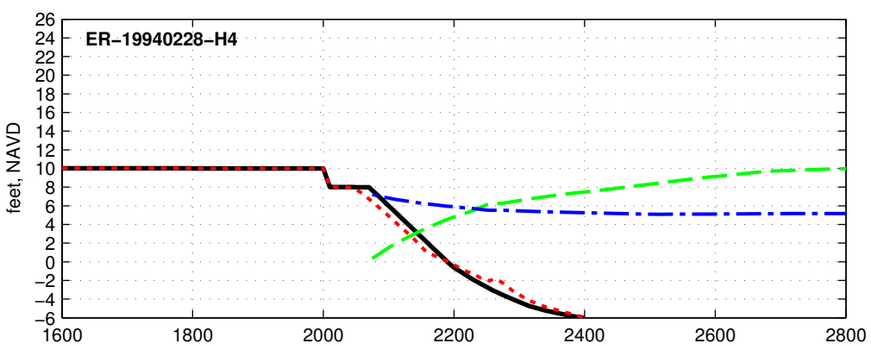


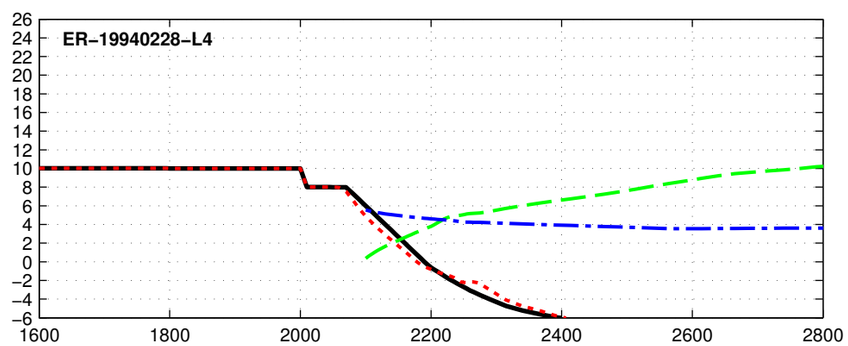


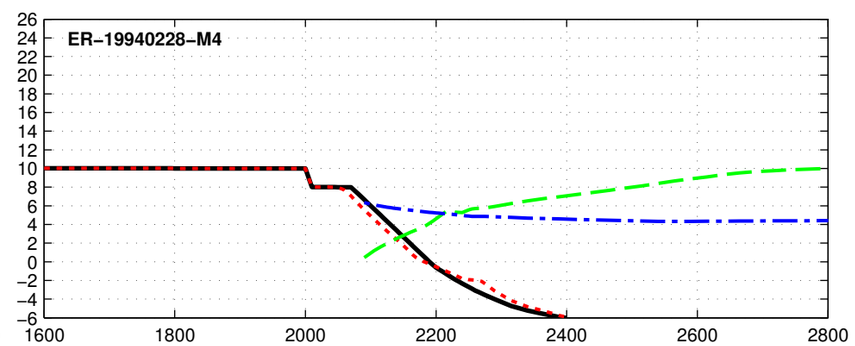






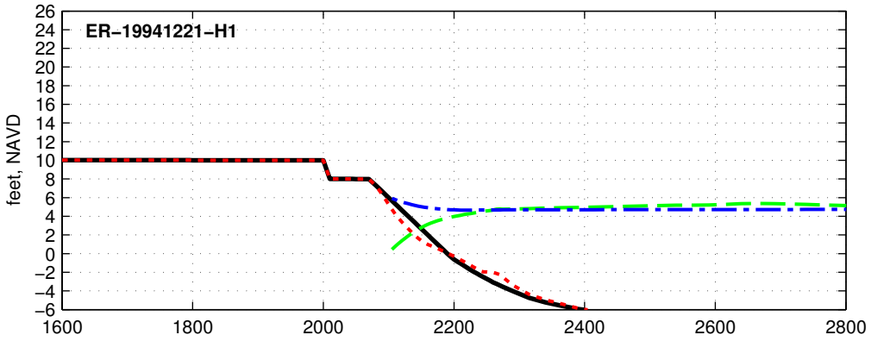


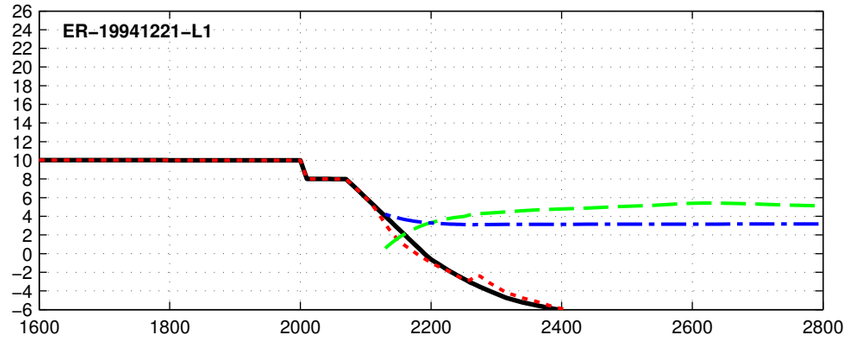


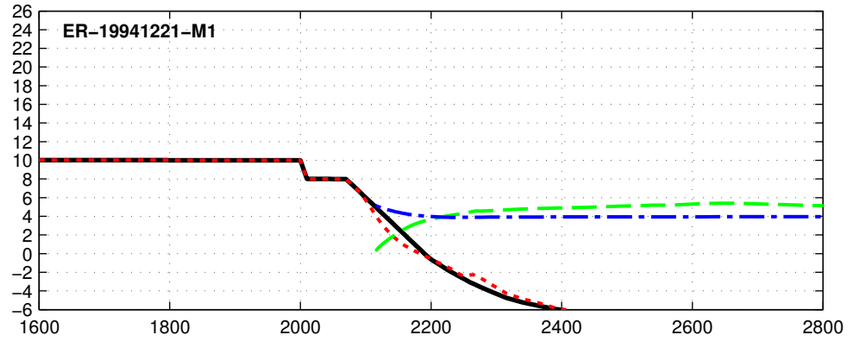


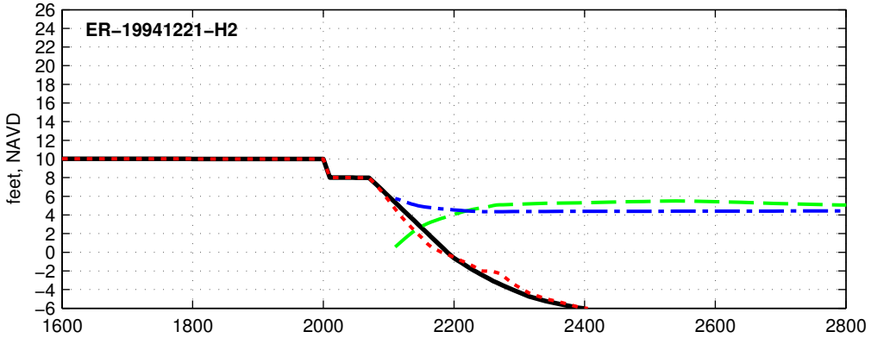
Initial Final Max Hs Max Water Level

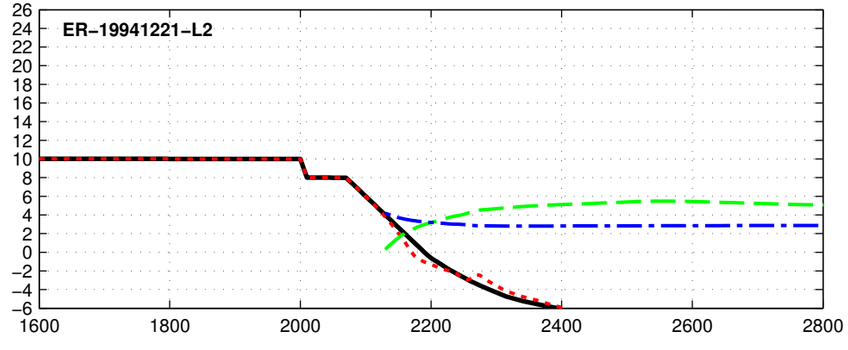
Dune R2T1-10.0-000-060 - ER-19941221

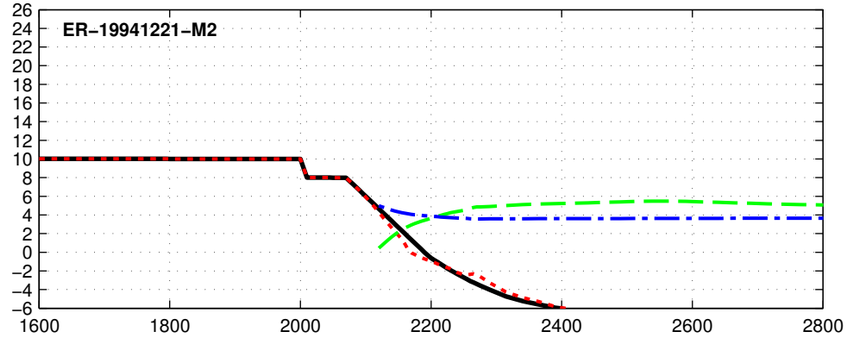


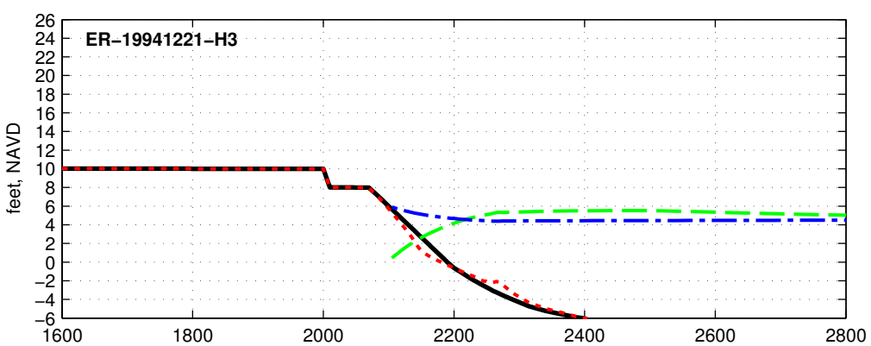


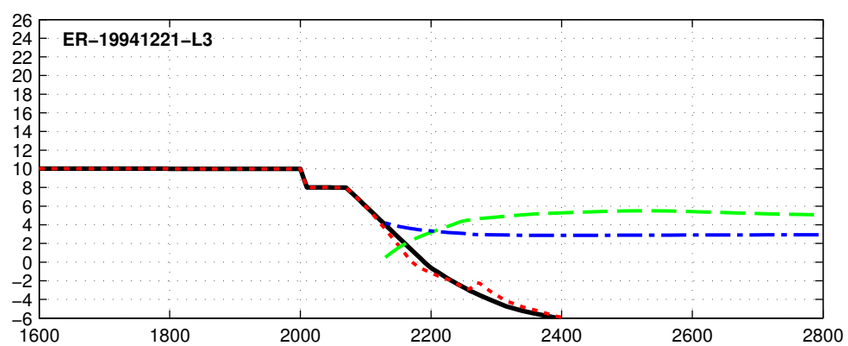


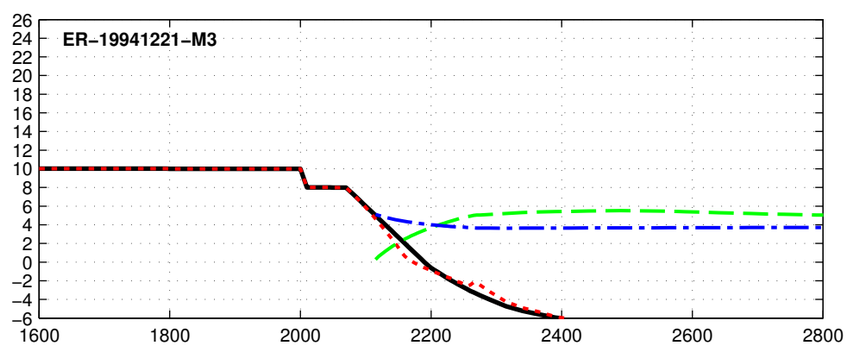


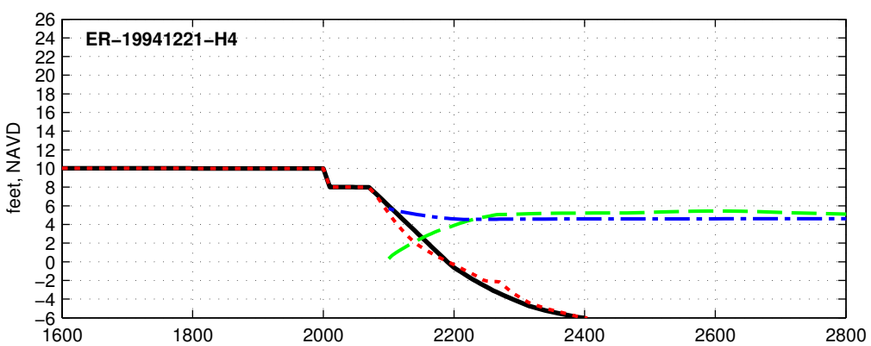


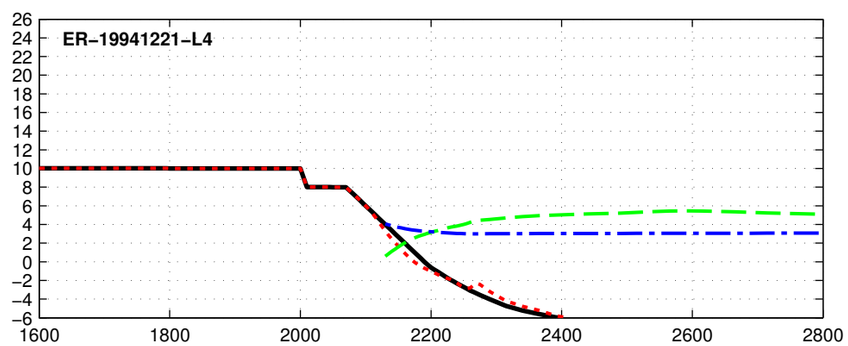


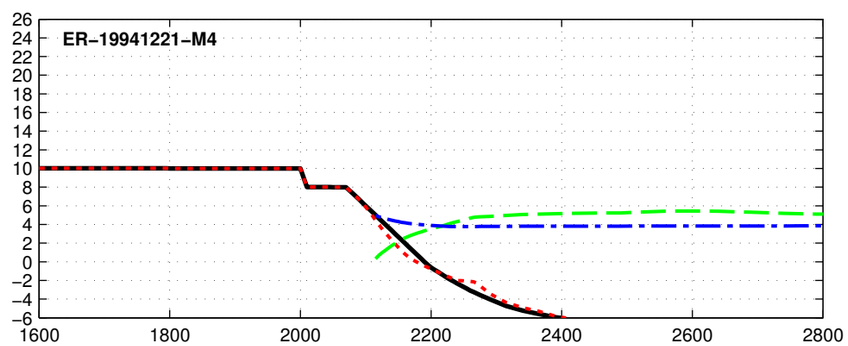






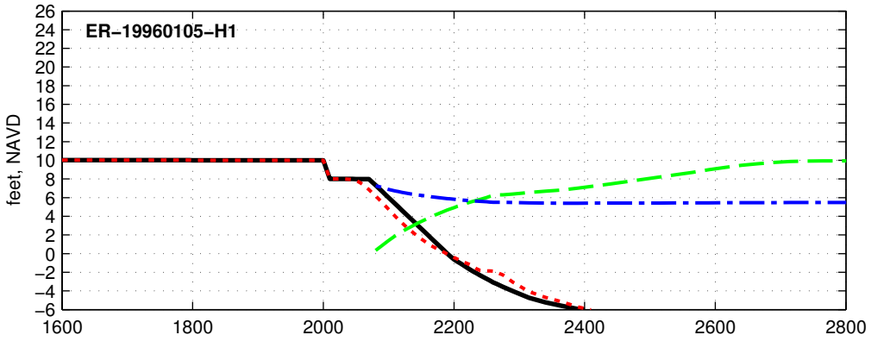


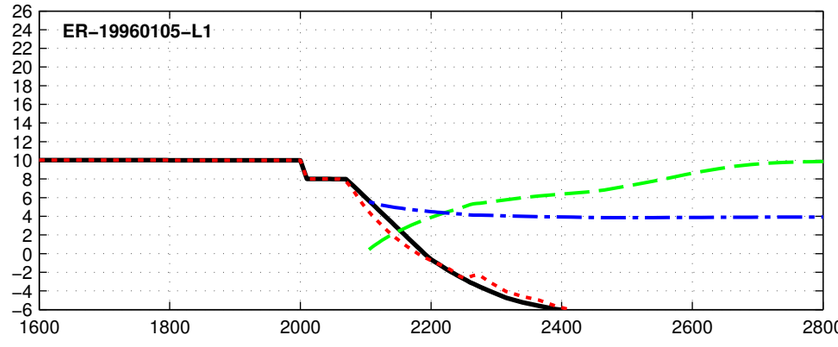


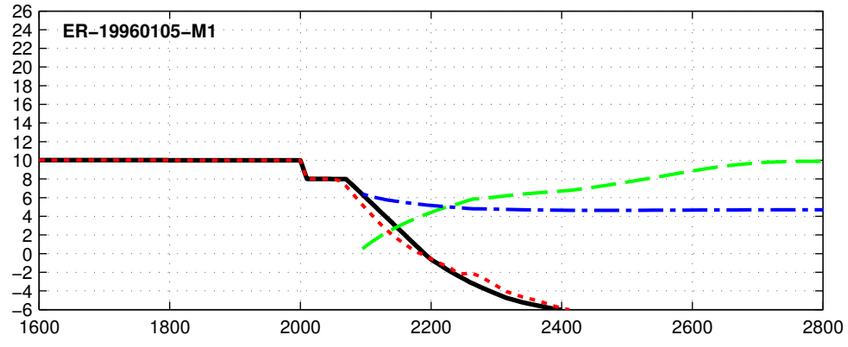


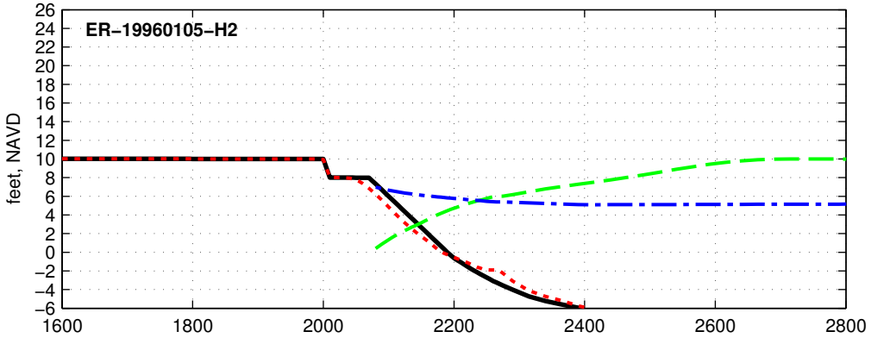
Initial Final Max Hs Max Water Level

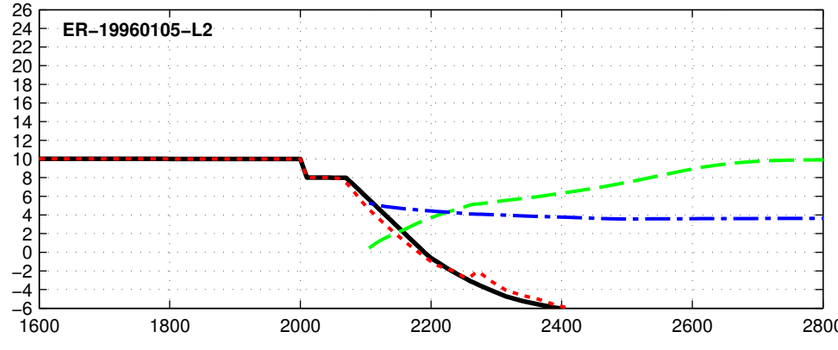
Dune R2T1-10.0-000-060 - ER-19960105

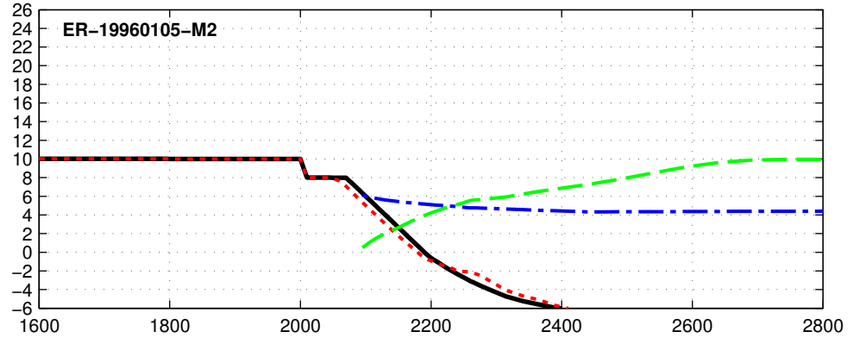


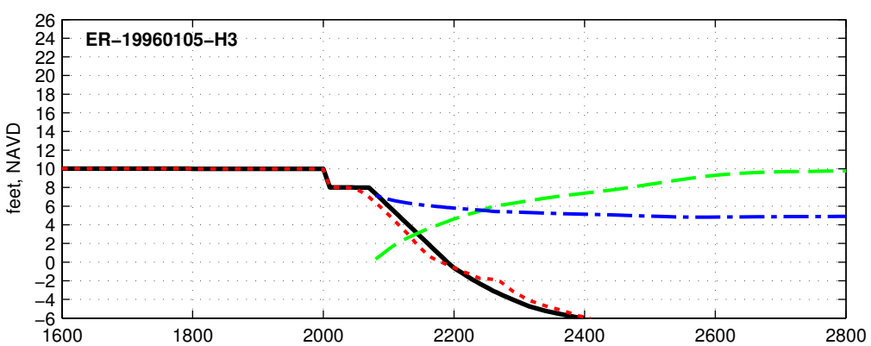


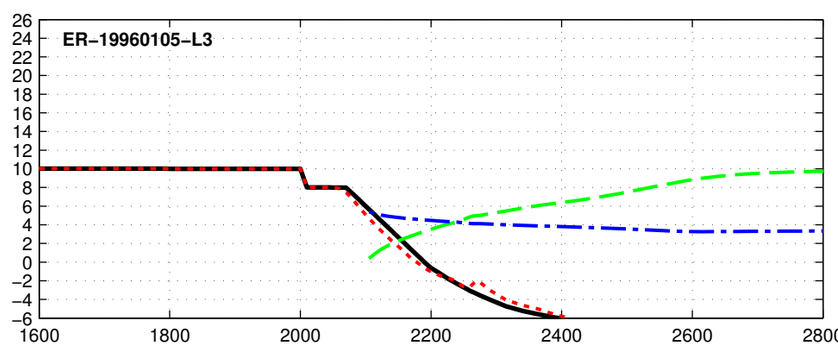


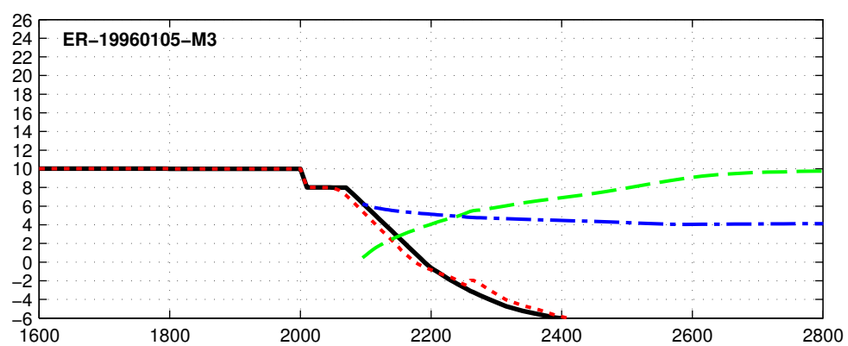


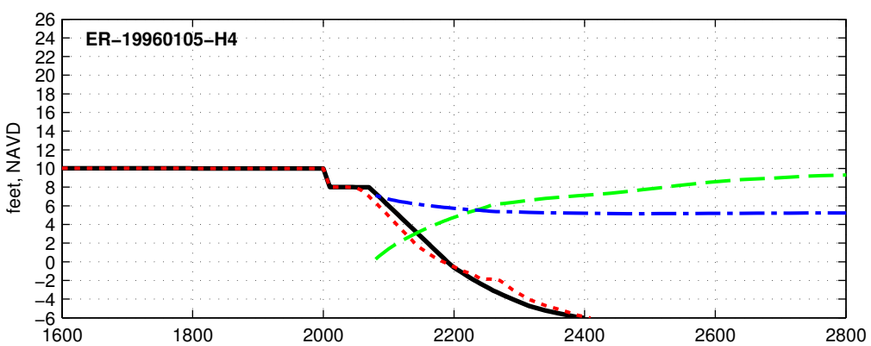


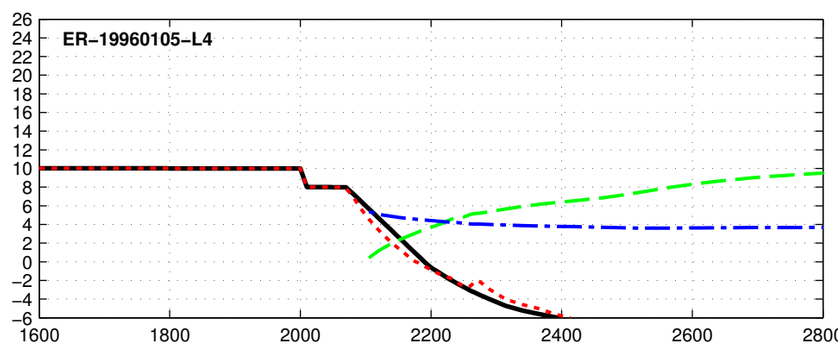


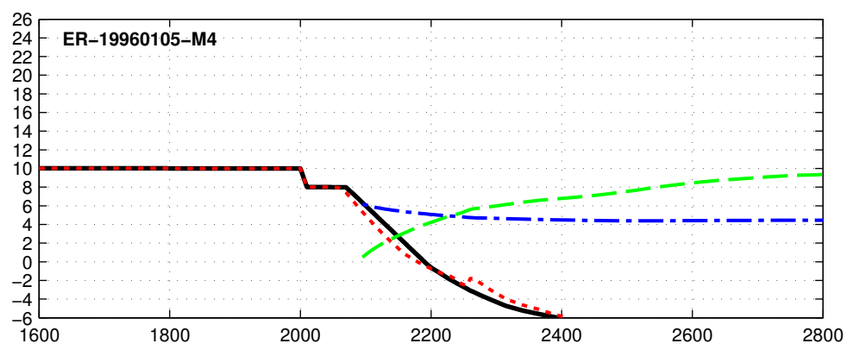






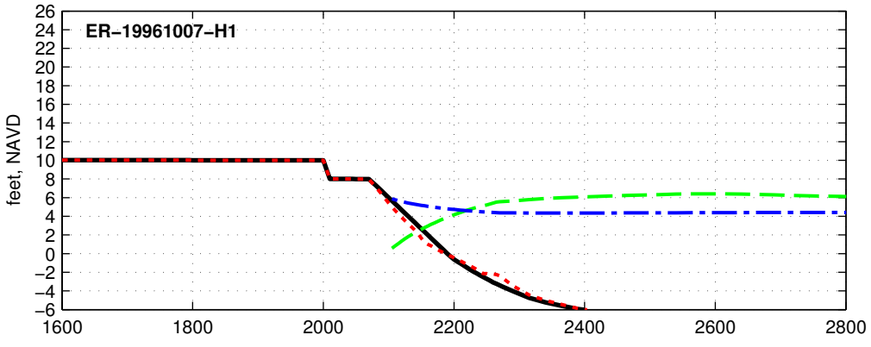


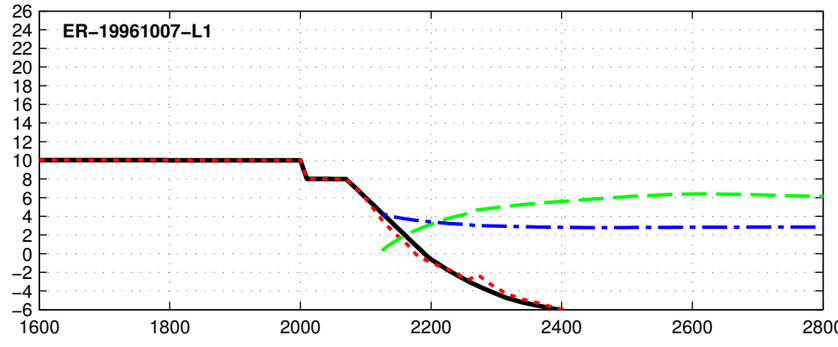


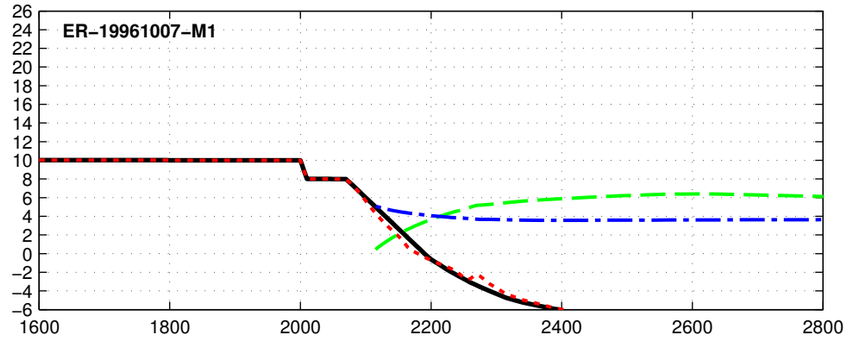


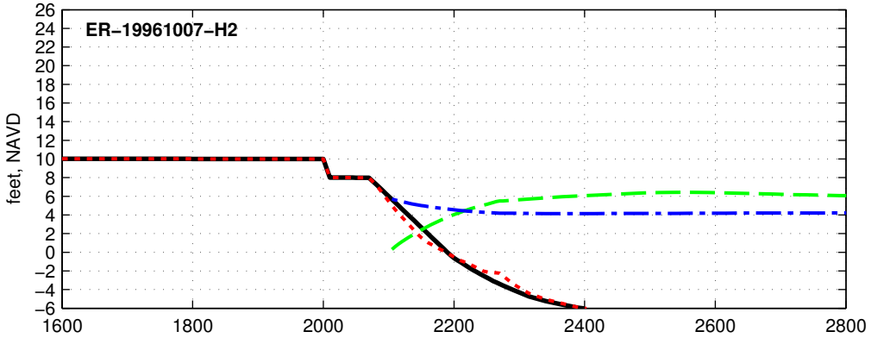
Initial Final Max Hs Max Water Level

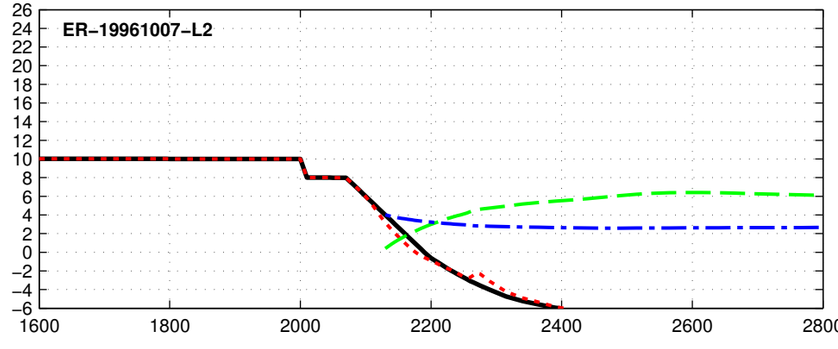
Dune R2T1-10.0-000-060 - ER-19961007

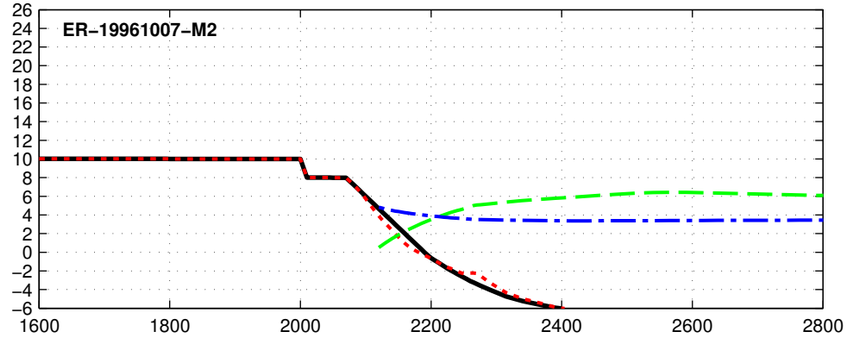


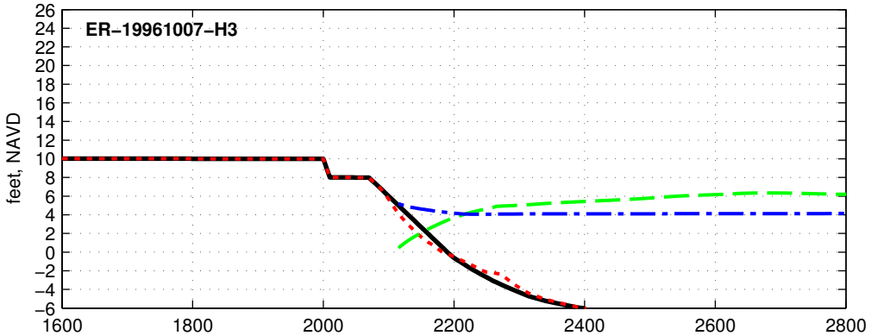


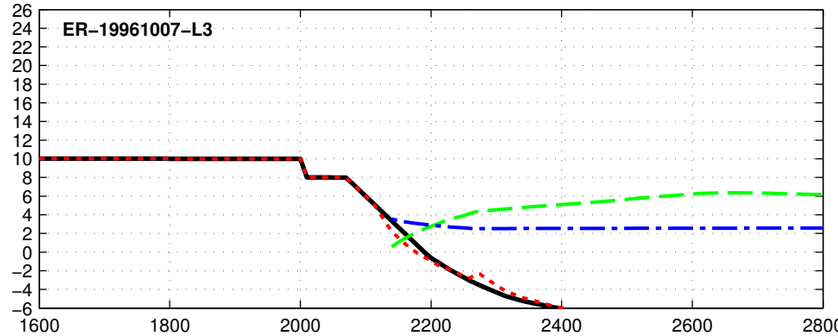


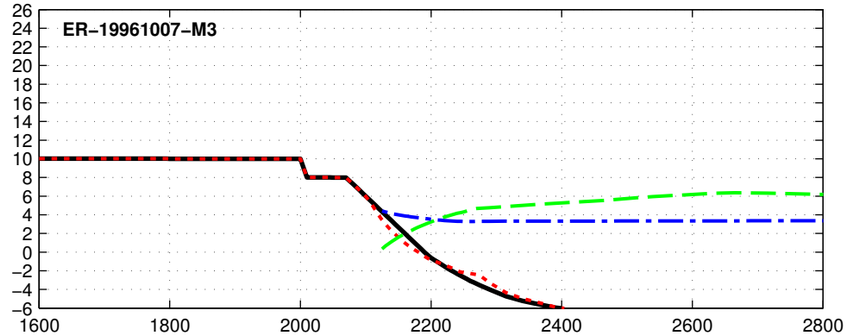


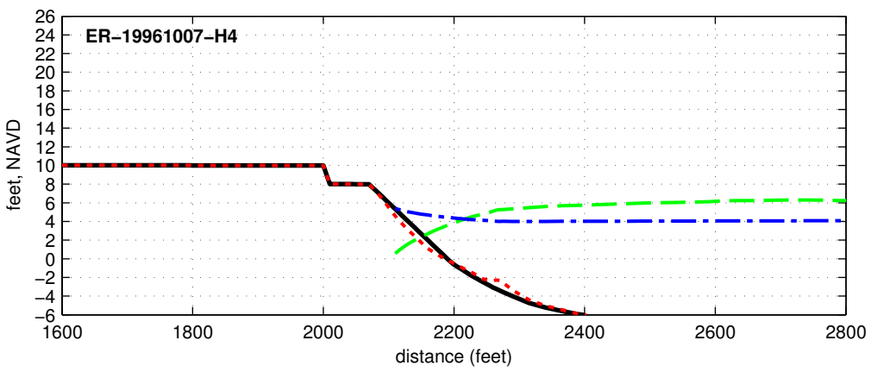


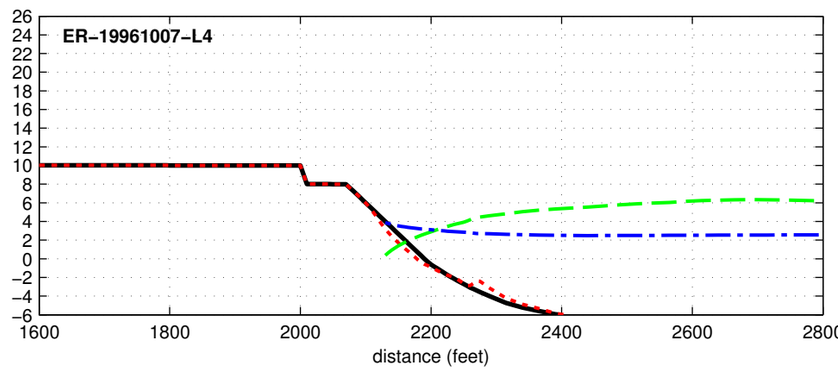


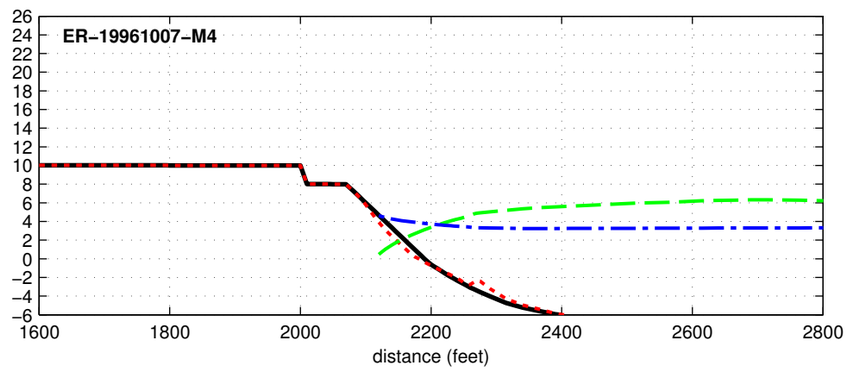






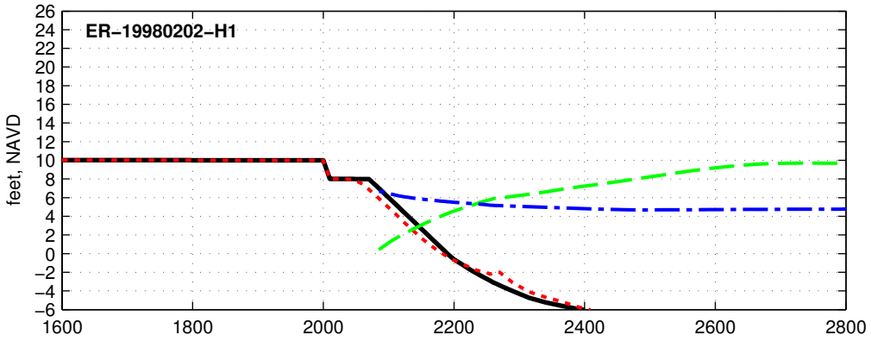


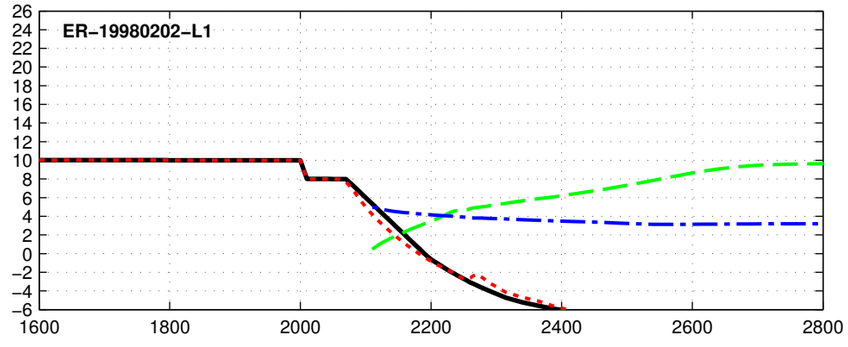


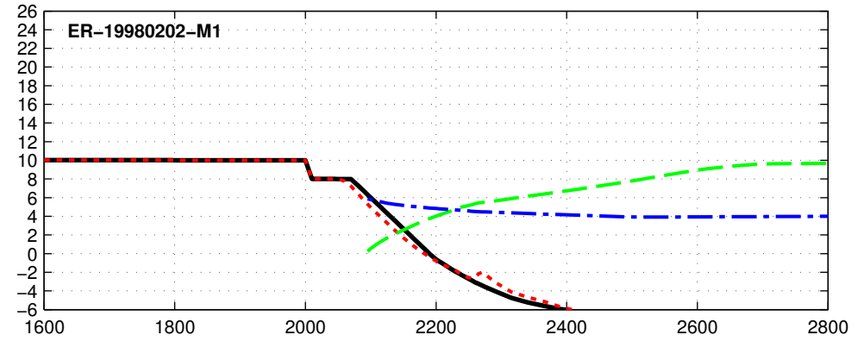


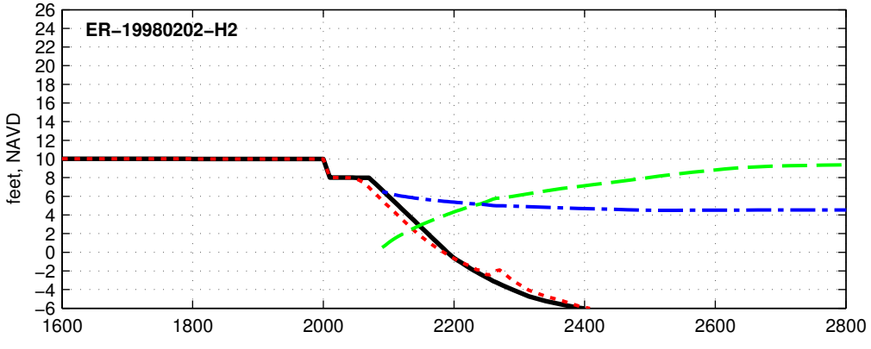
Initial Final Max Hs Max Water Level

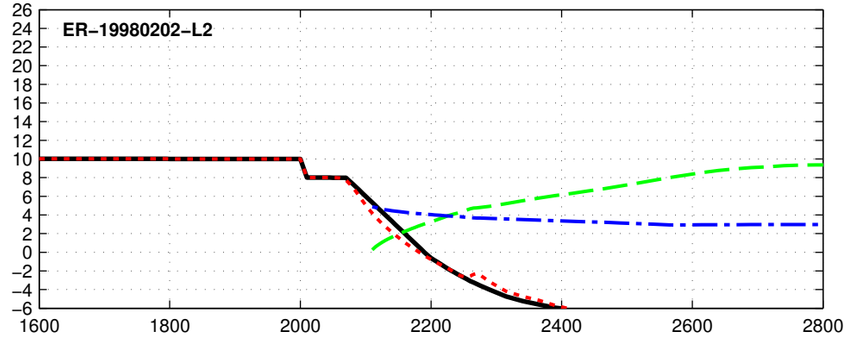
Dune R2T1-10.0-000-060 - ER-19980202

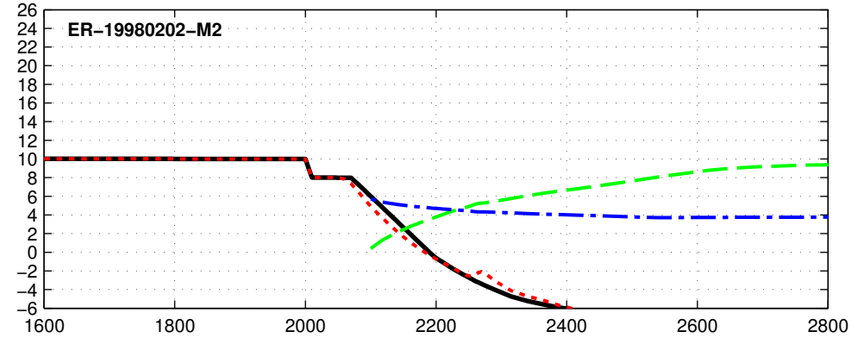


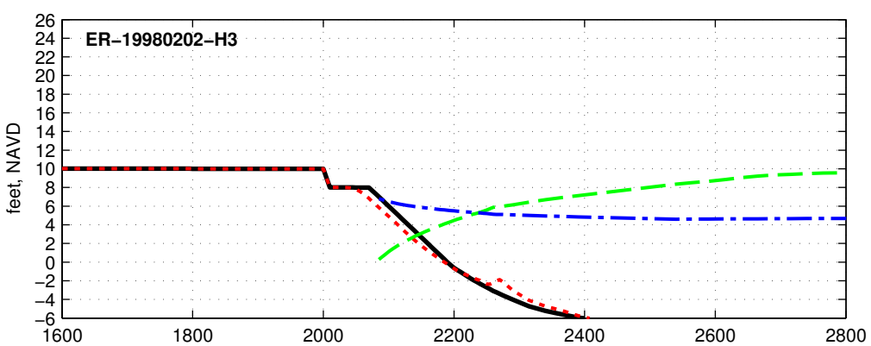


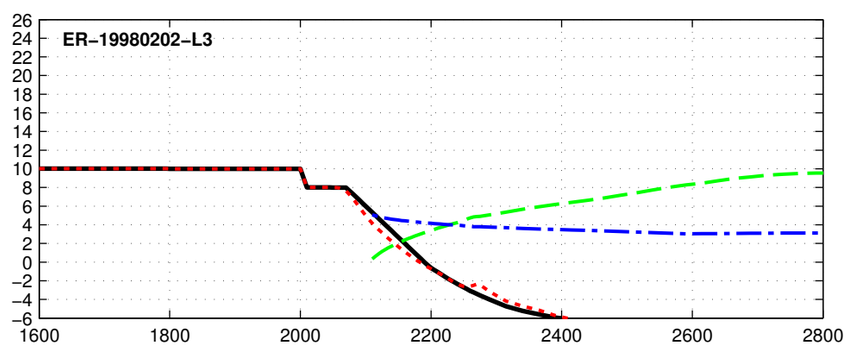


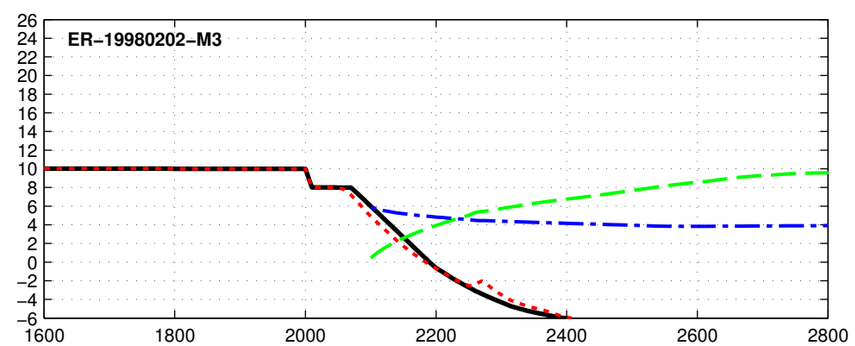


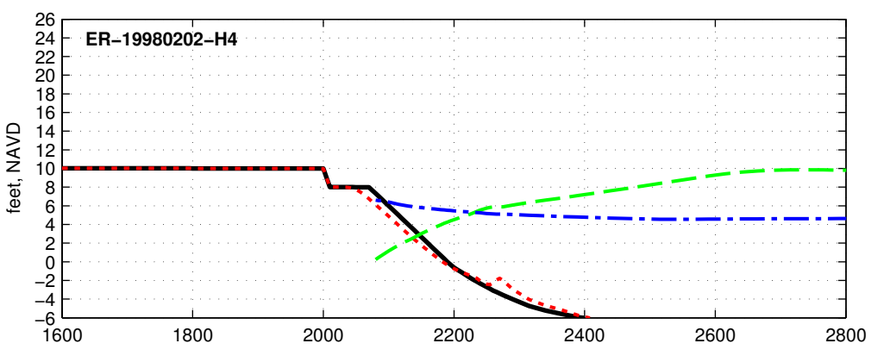


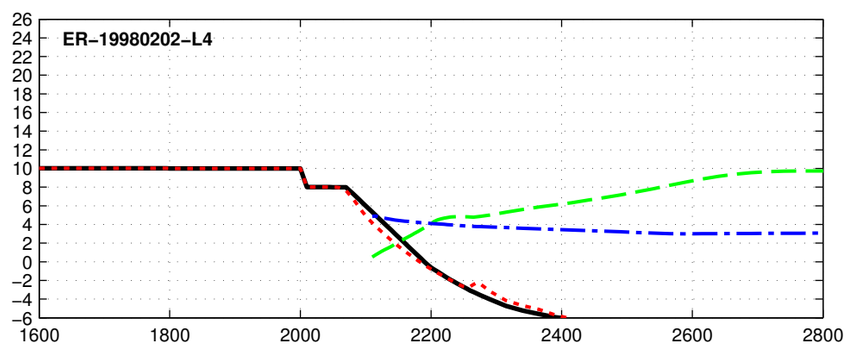


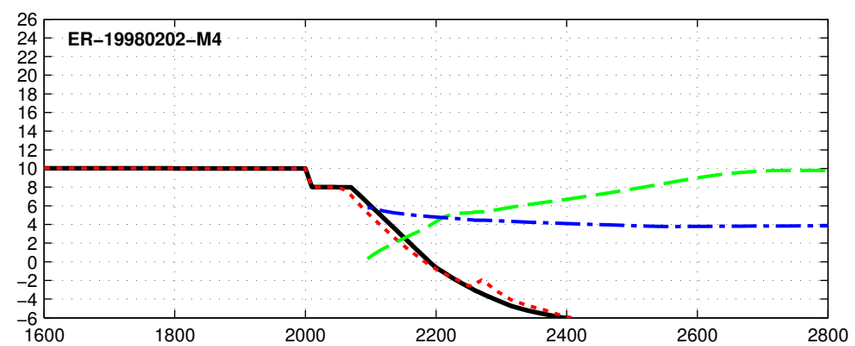






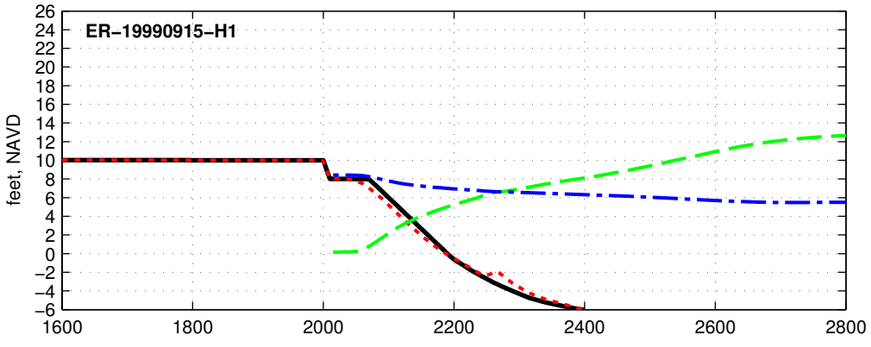


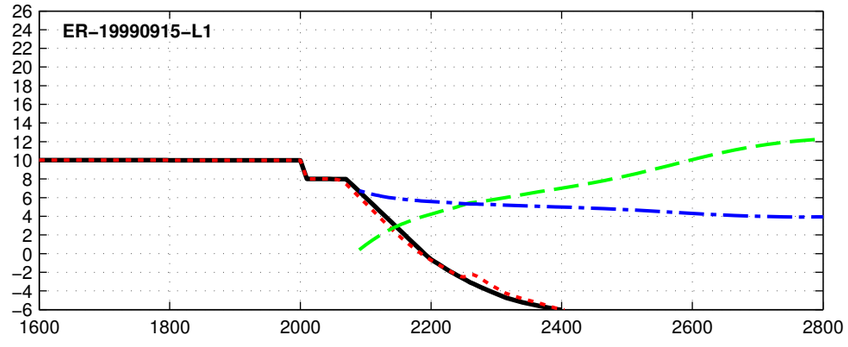


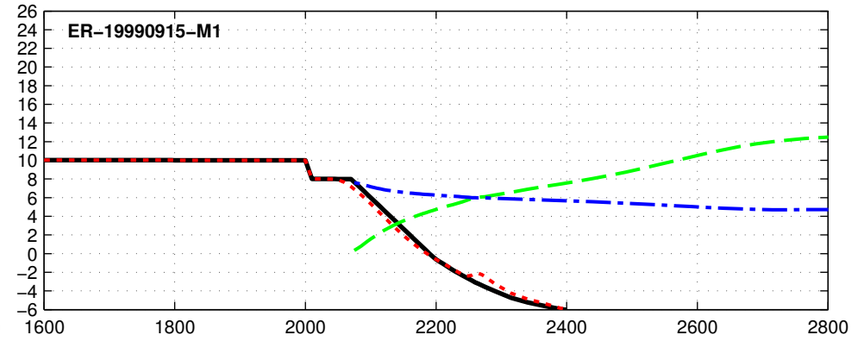


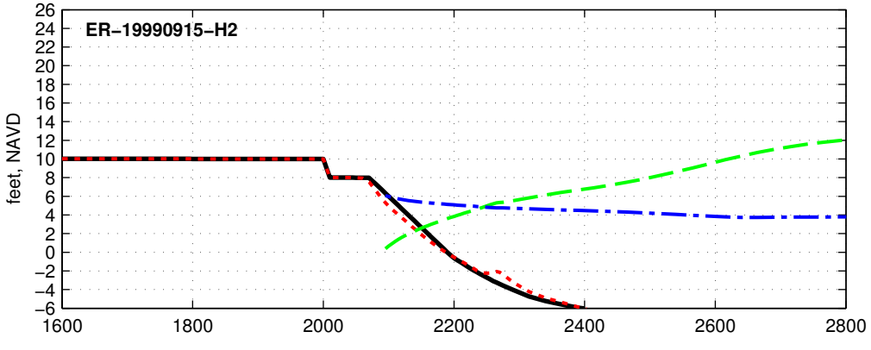
Initial Final Max Hs Max Water Level

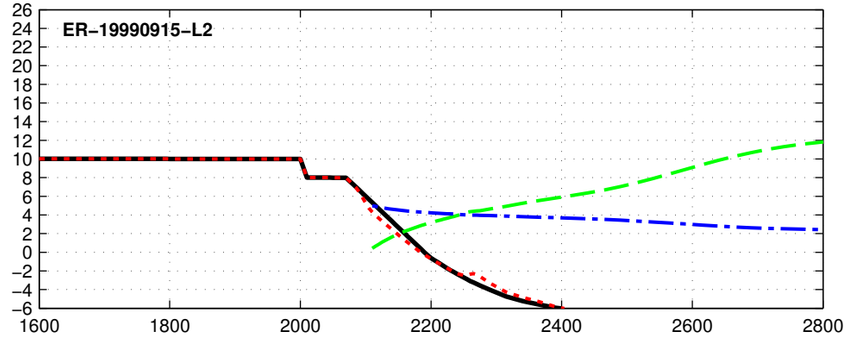
Dune R2T1-10.0-000-060 - ER-19990915

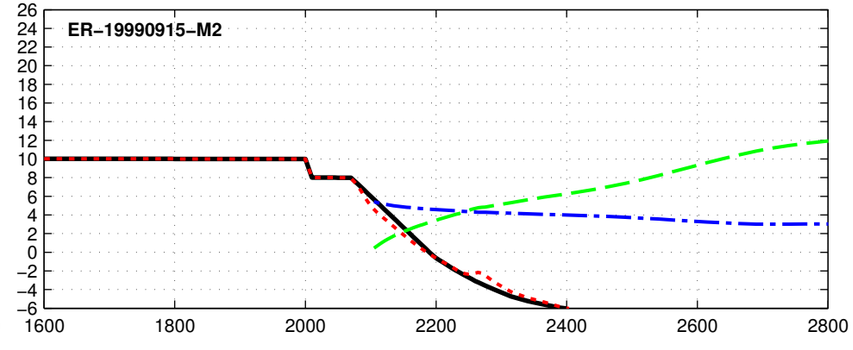


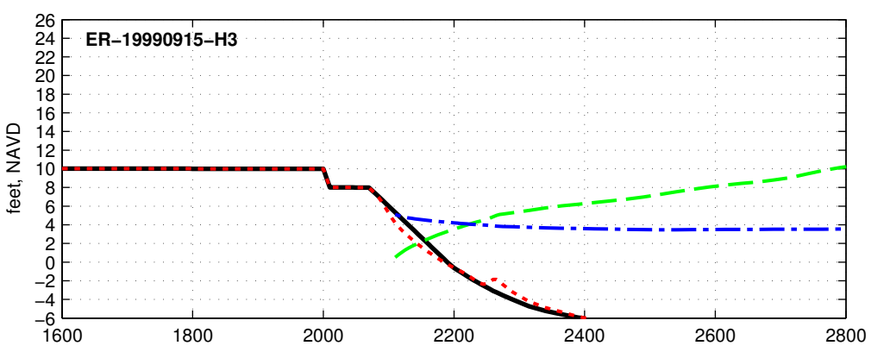


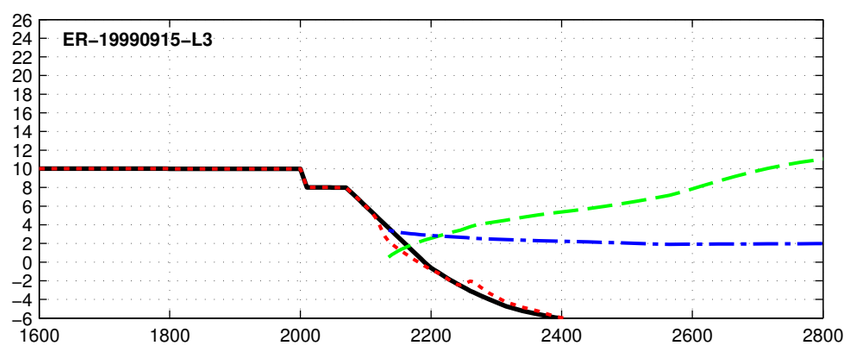


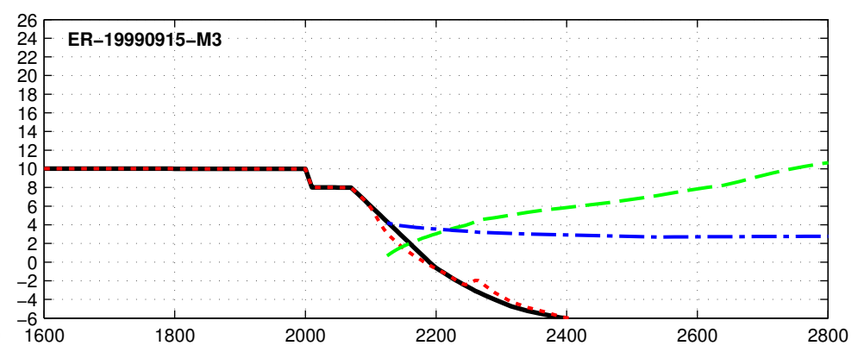


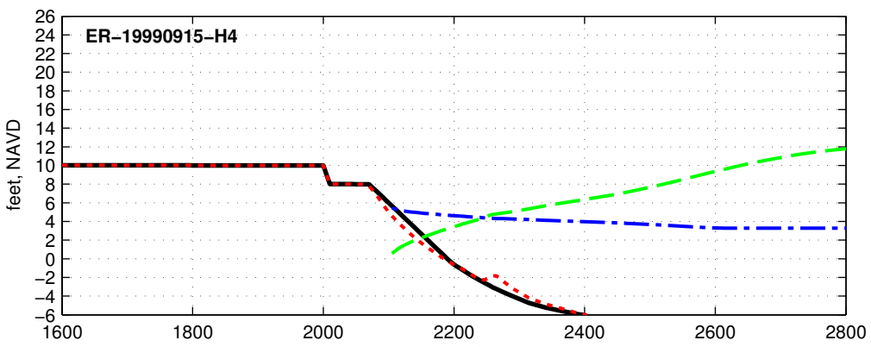


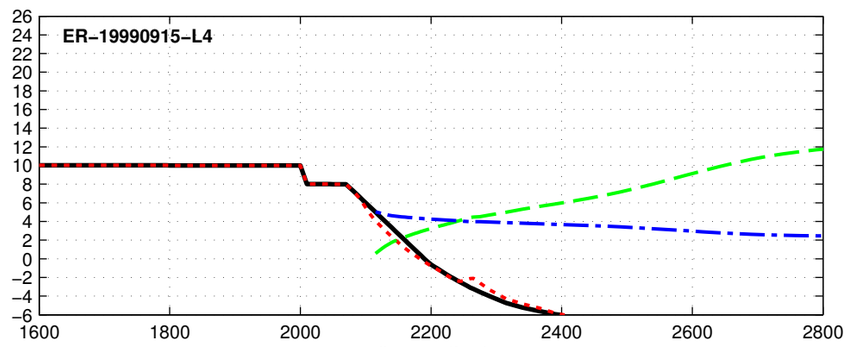


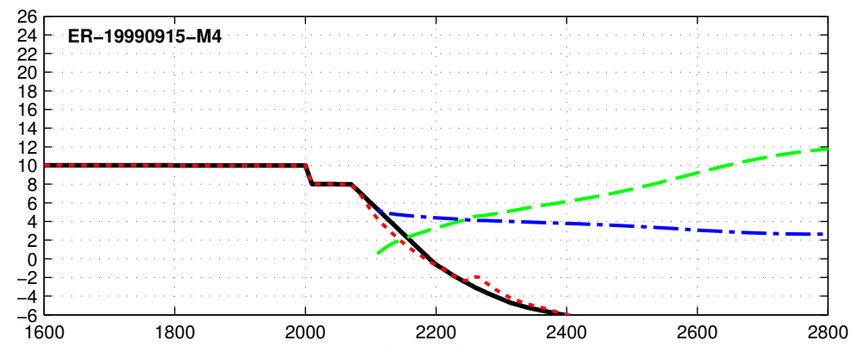






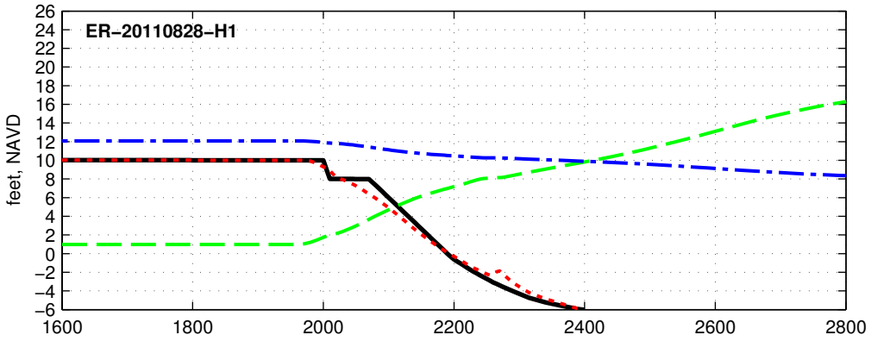


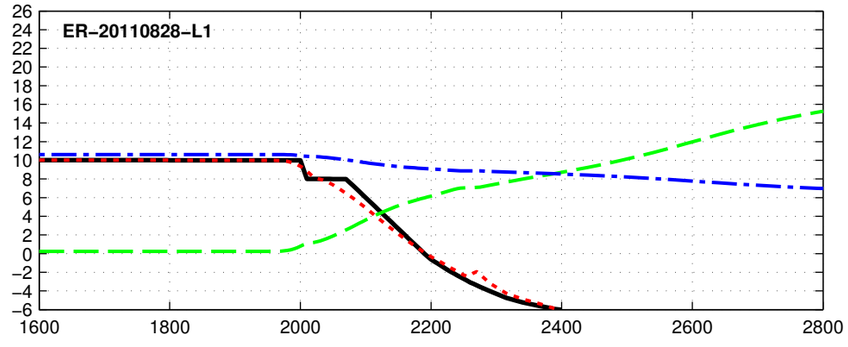


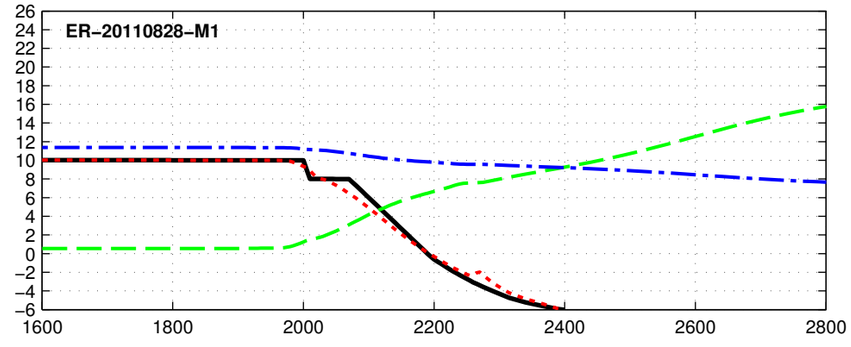


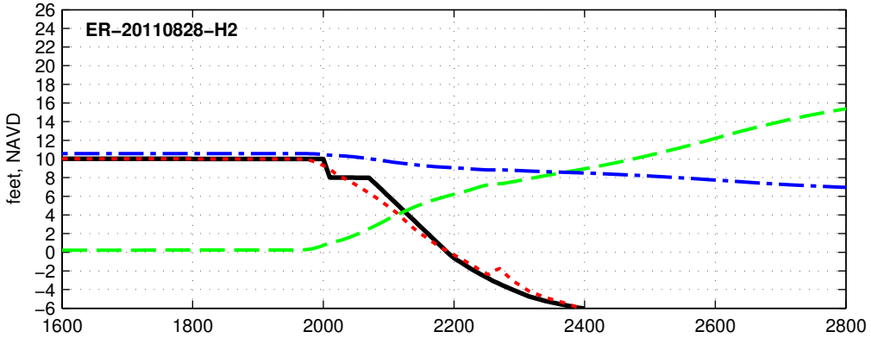
Initial Final Max Hs Max Water Level

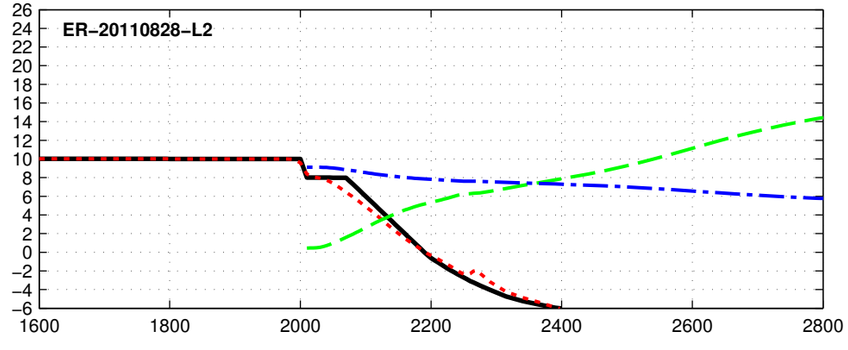
Dune R2T1-10.0-000-060 - ER-20110828

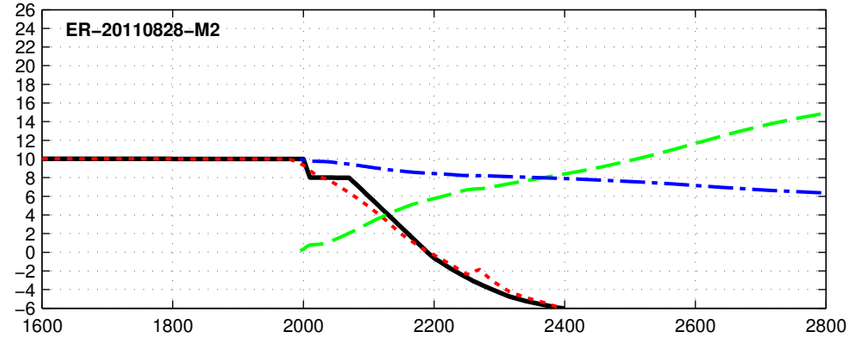


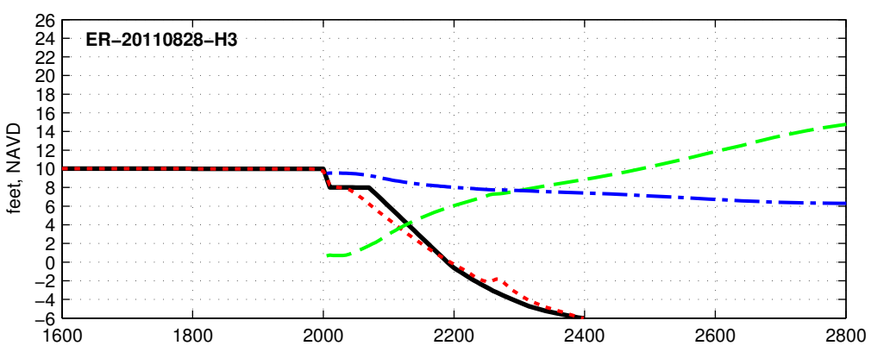


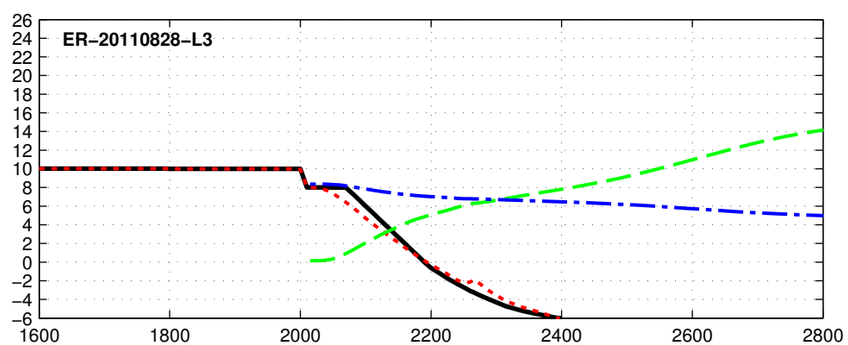


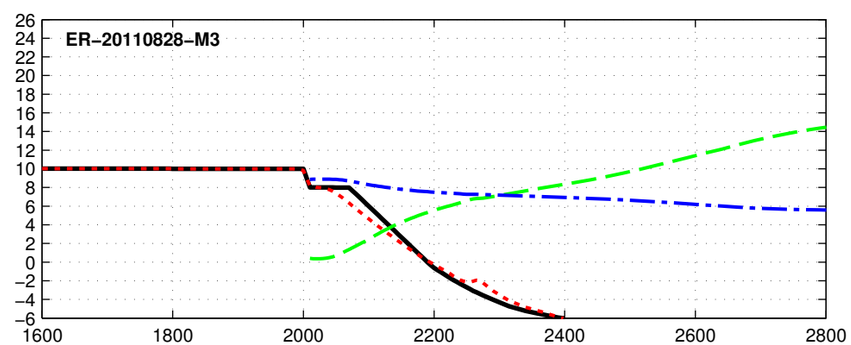


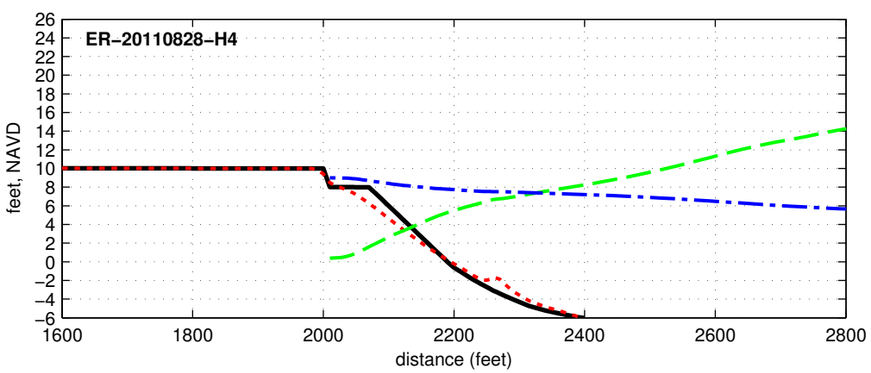


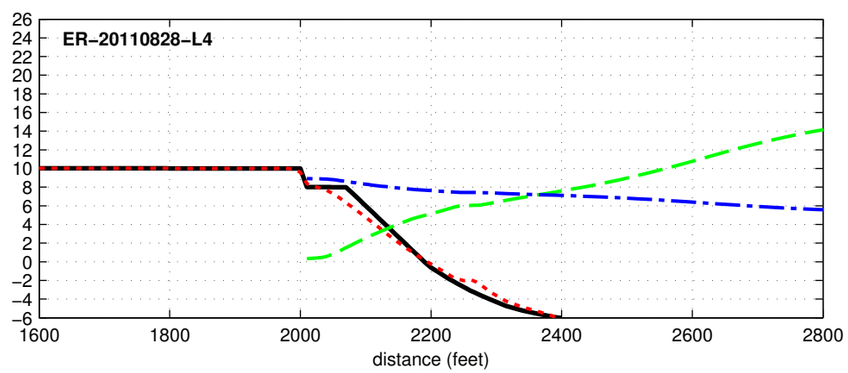


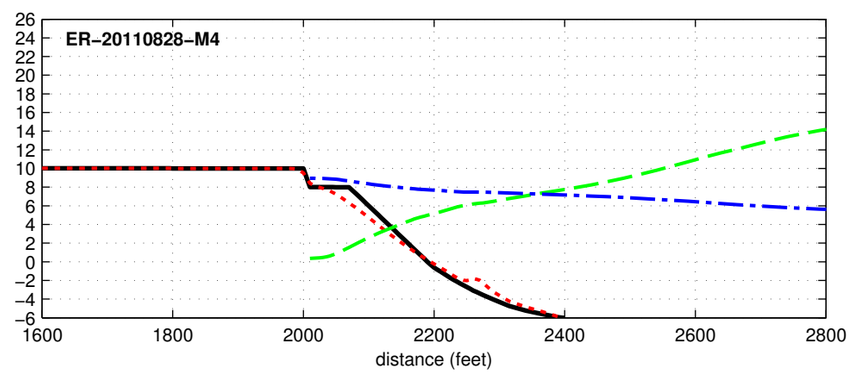






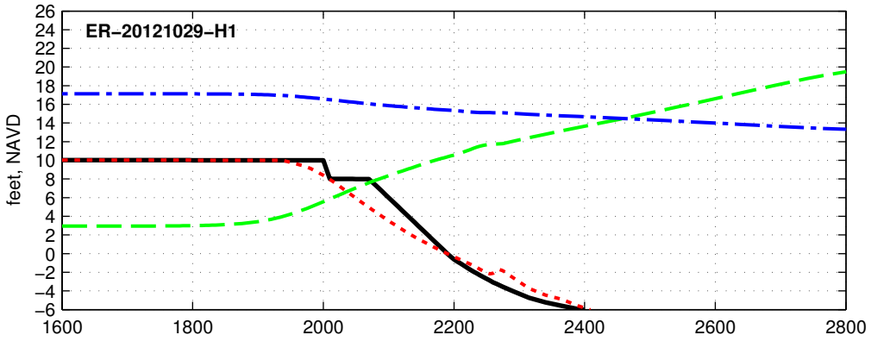


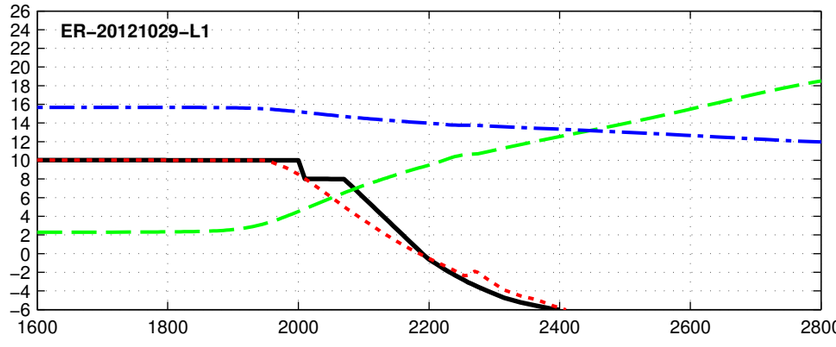


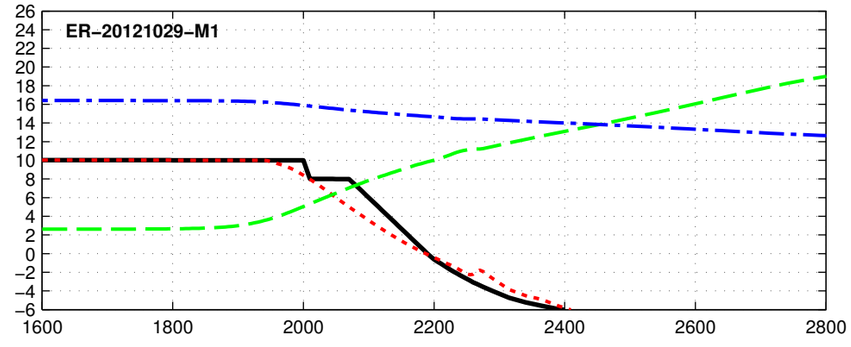


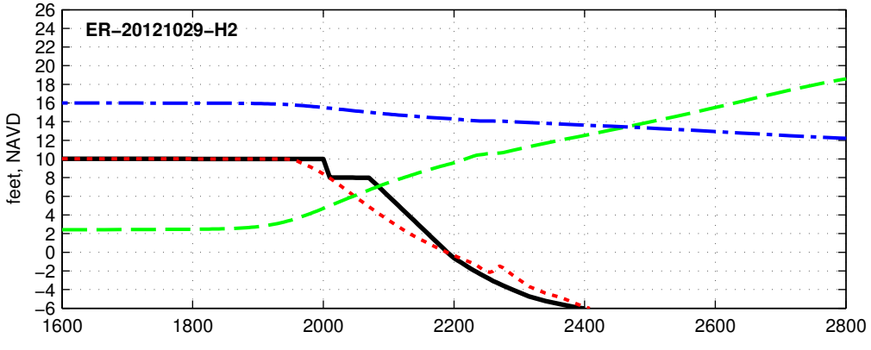
Initial Final Max Hs Max Water Level

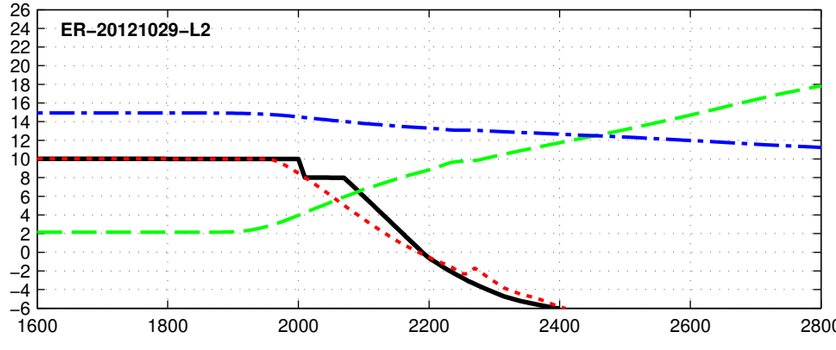
Dune R2T1-10.0-000-060 - ER-20121029

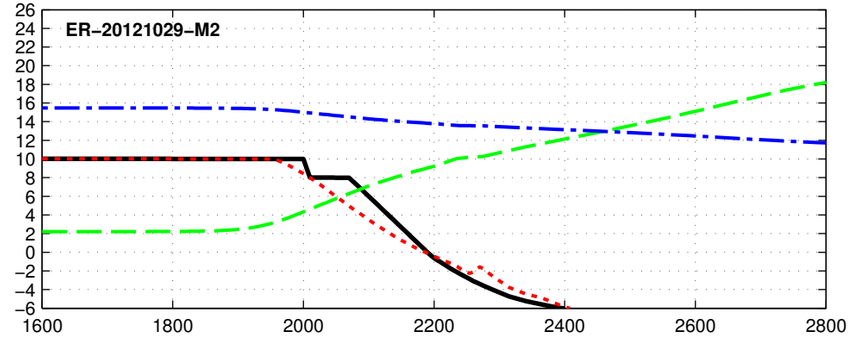


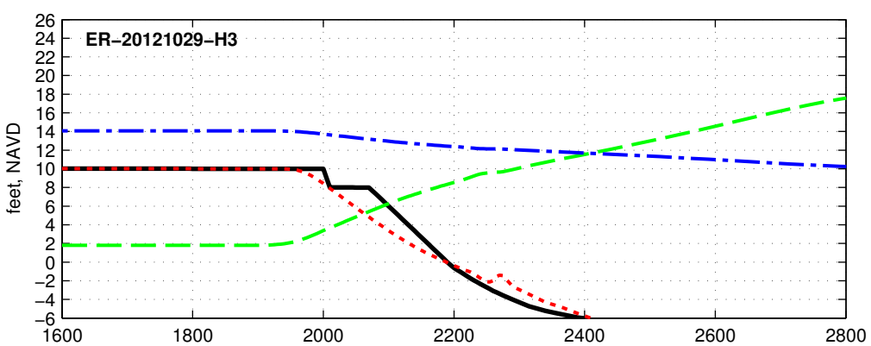


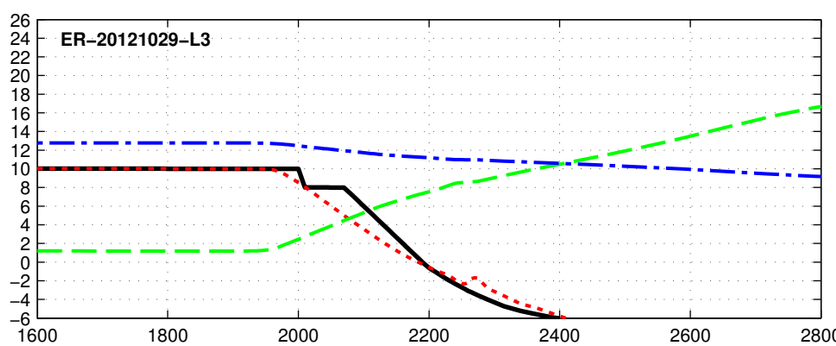


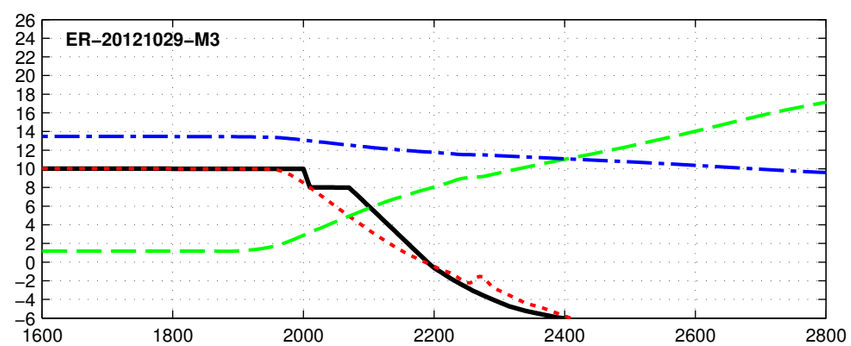


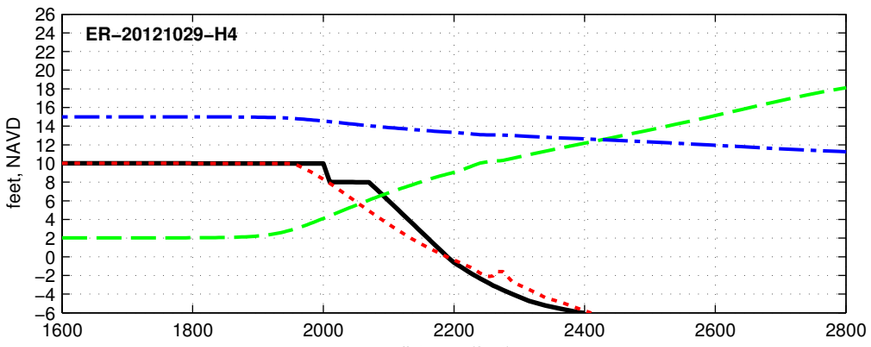


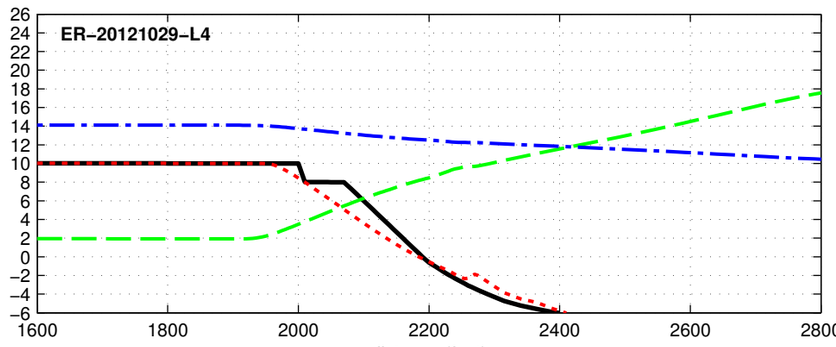


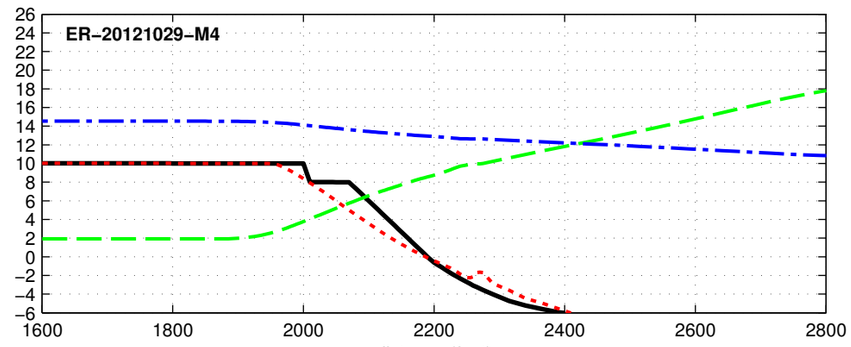








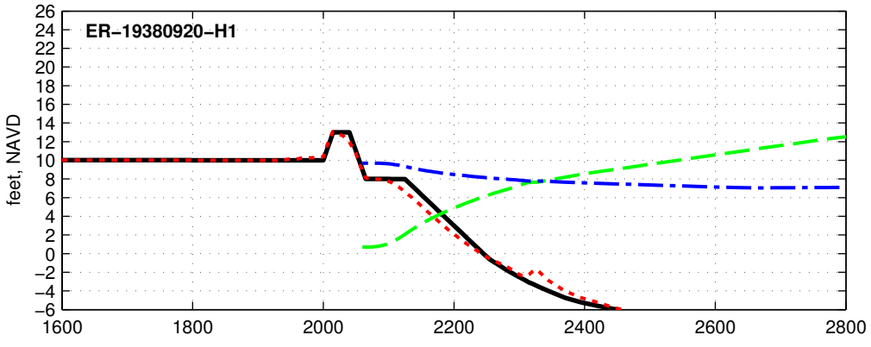


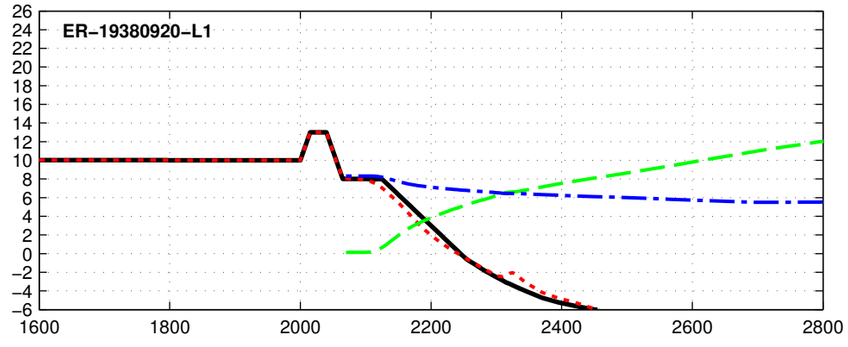


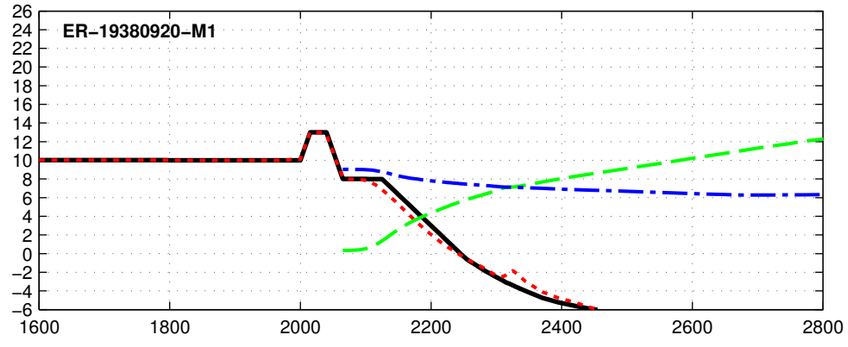
Initial Final Max Hs Max Water Level

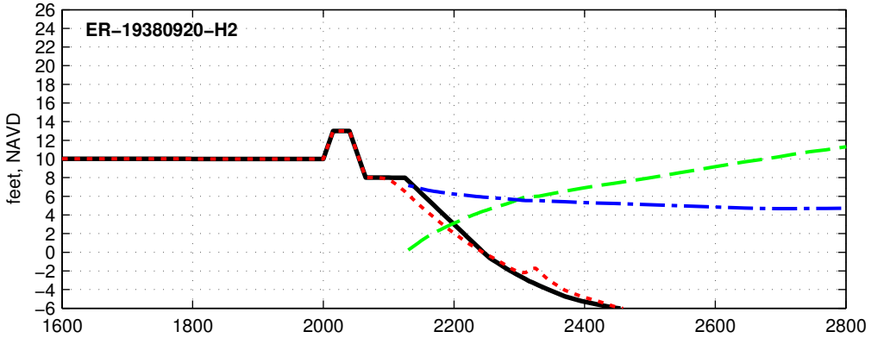
R2T1_13.0_025_060 - DUNE

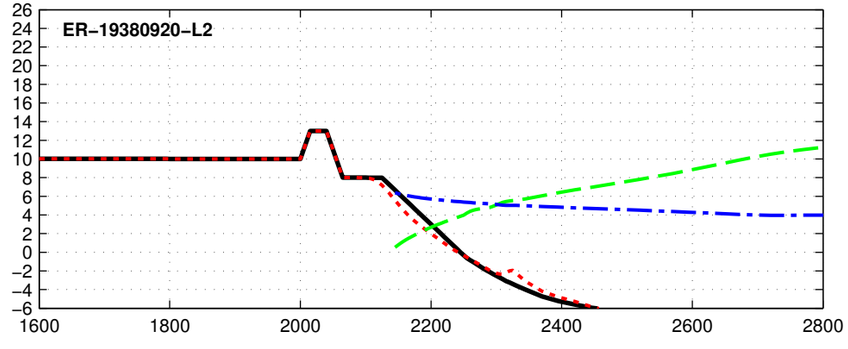
Dune R2T1-13.0-025-060 - ER-19380920

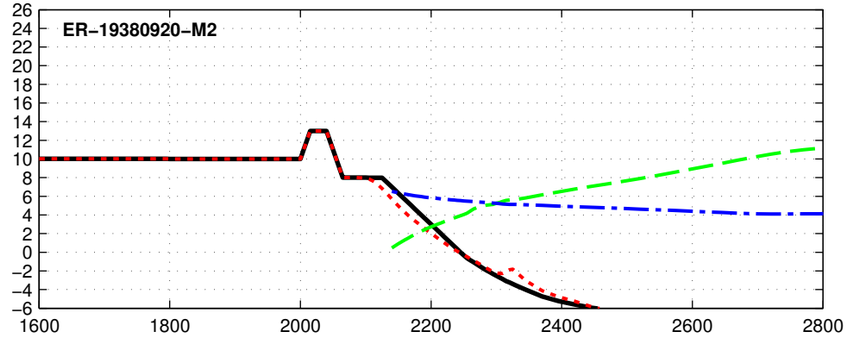


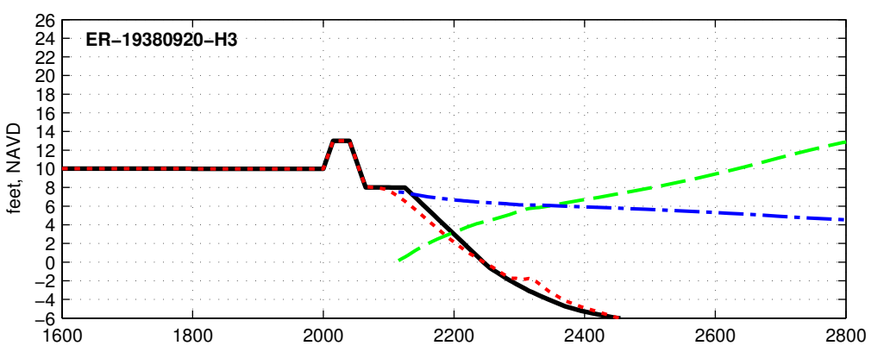


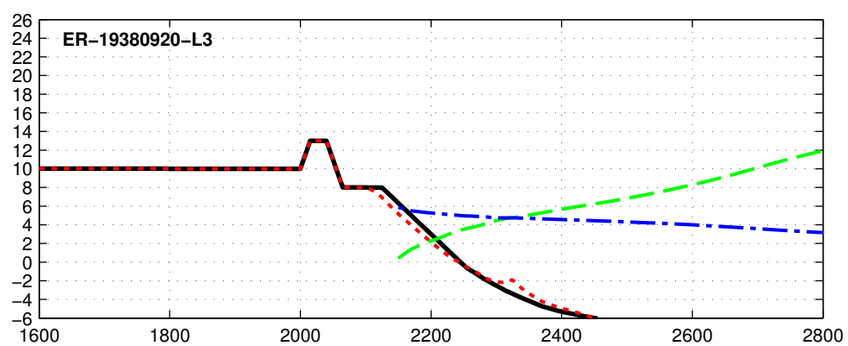


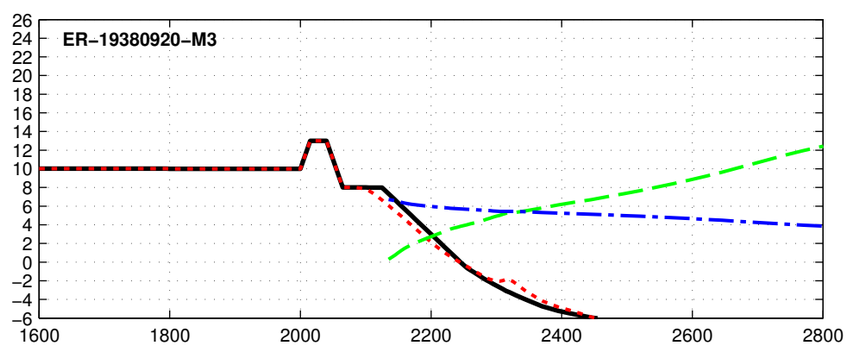


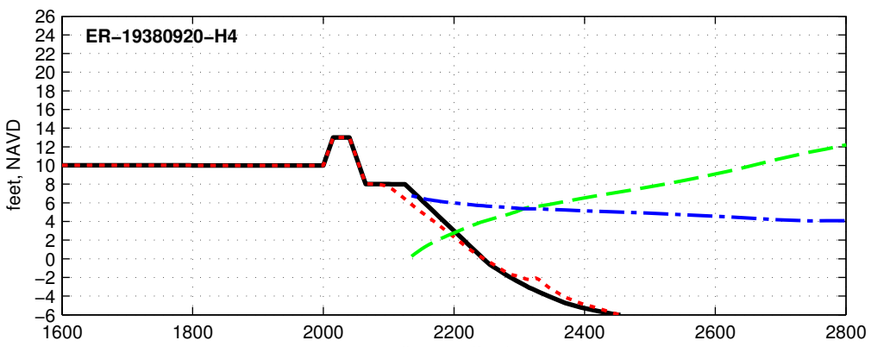


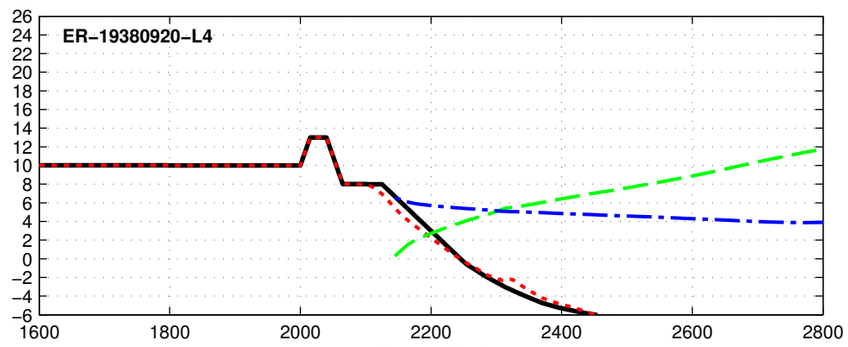


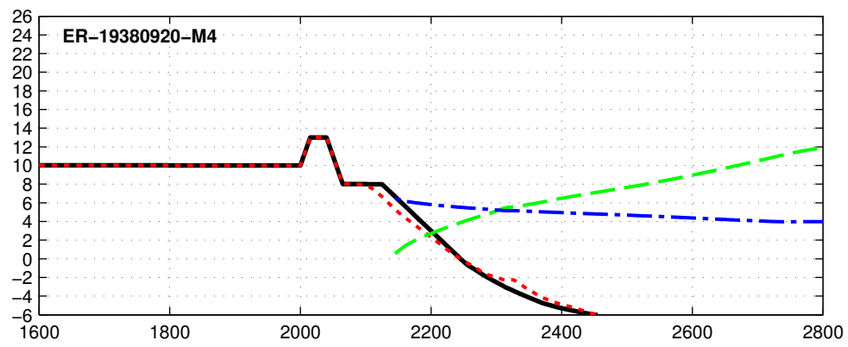






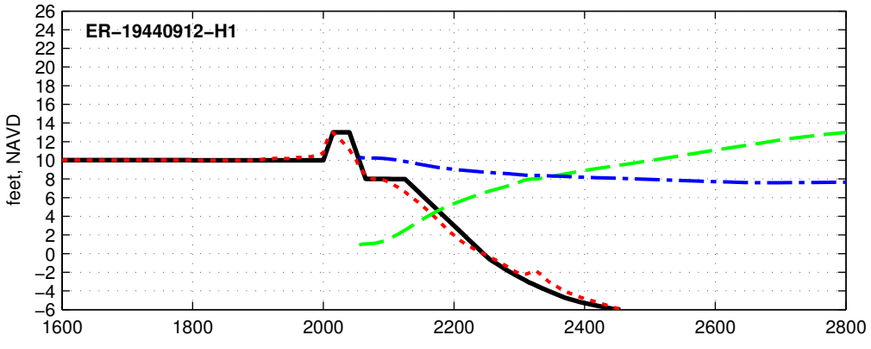


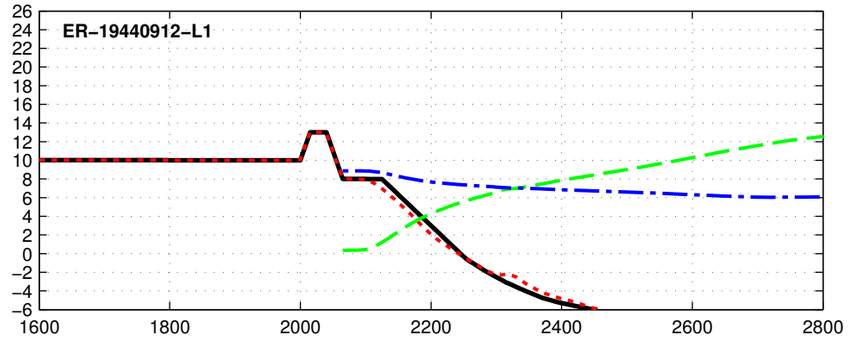


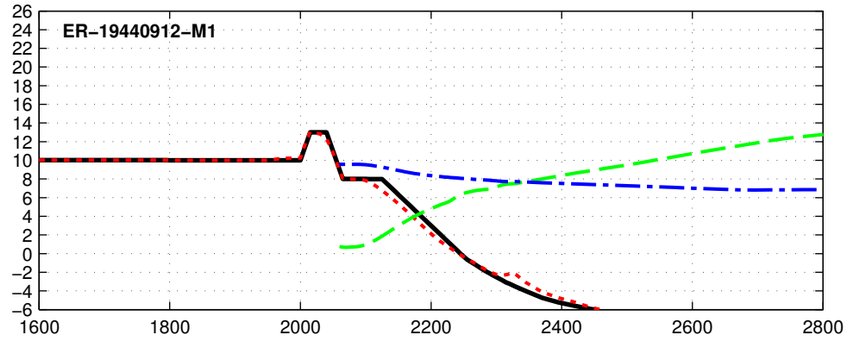


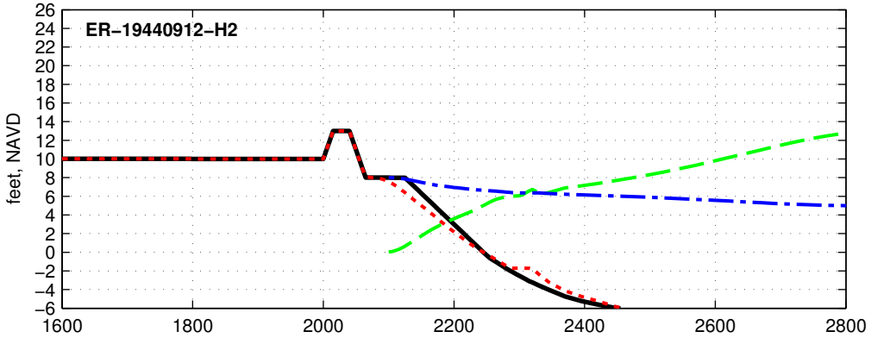
Initial Final Max Hs Max Water Level

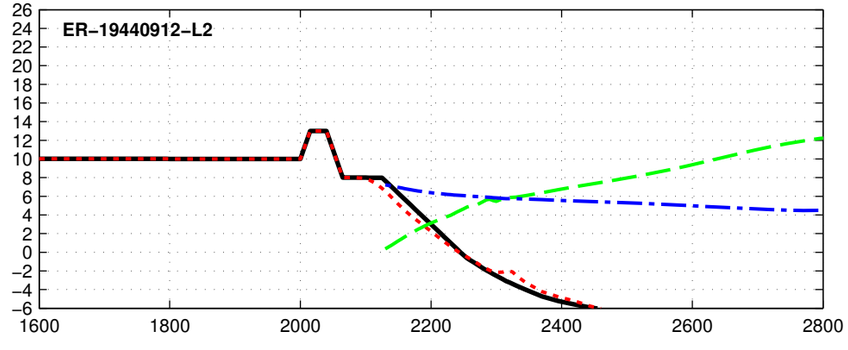
Dune R2T1–13.0–025–060 – ER–19440912

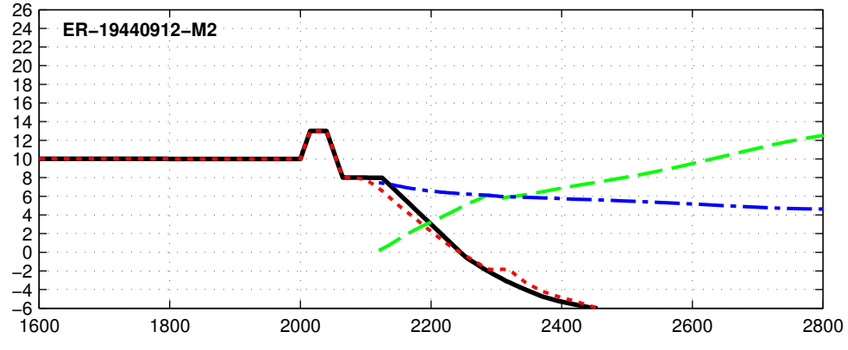


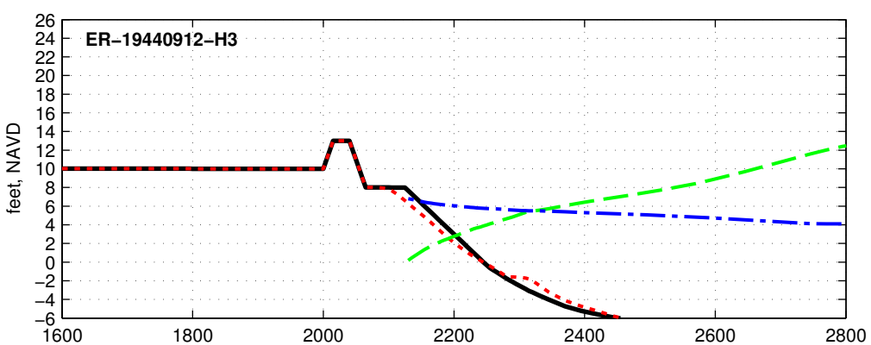


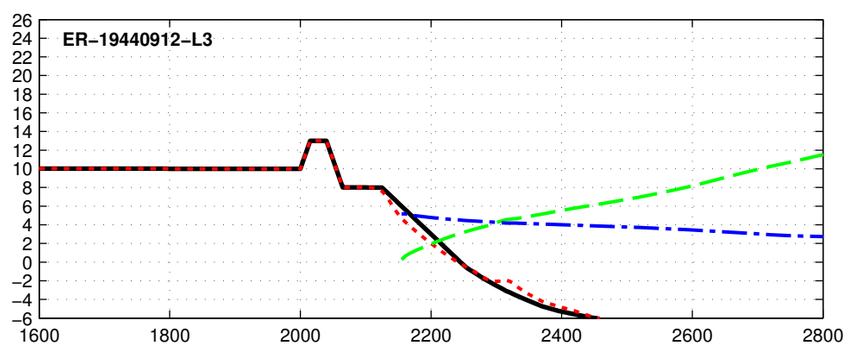


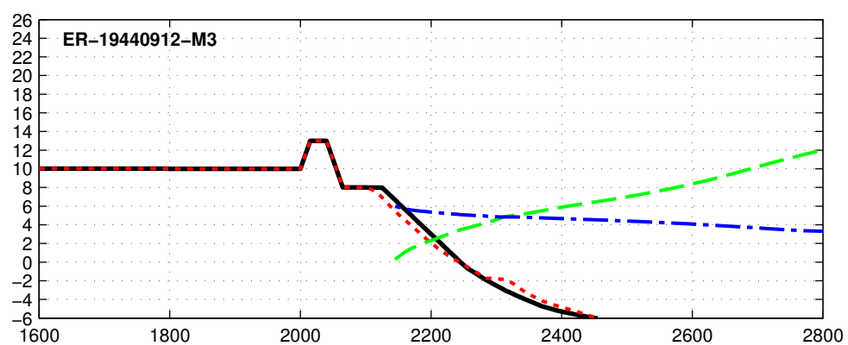


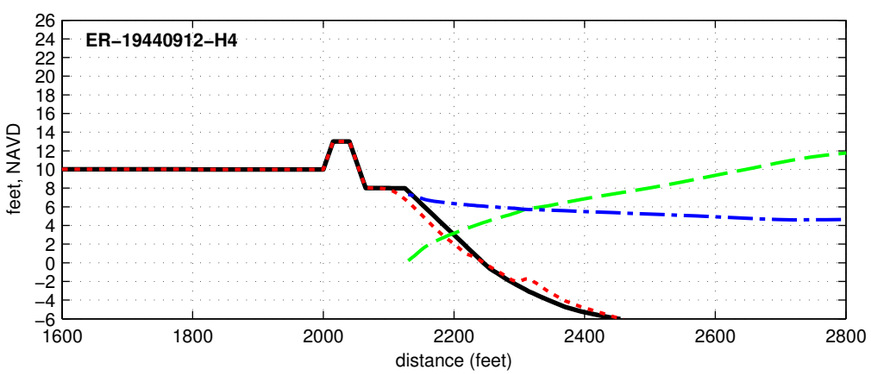


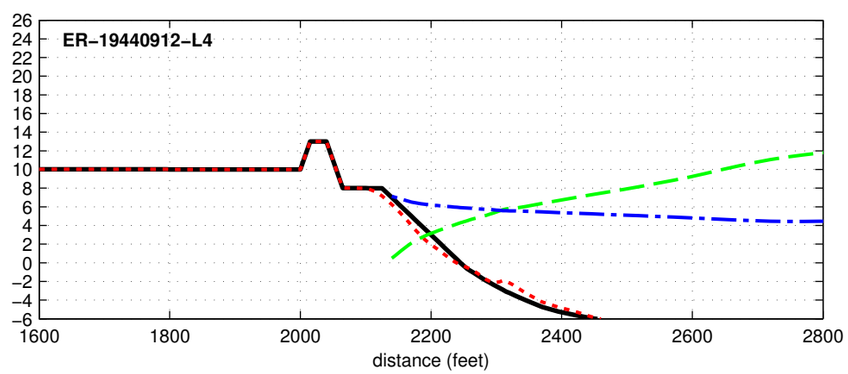


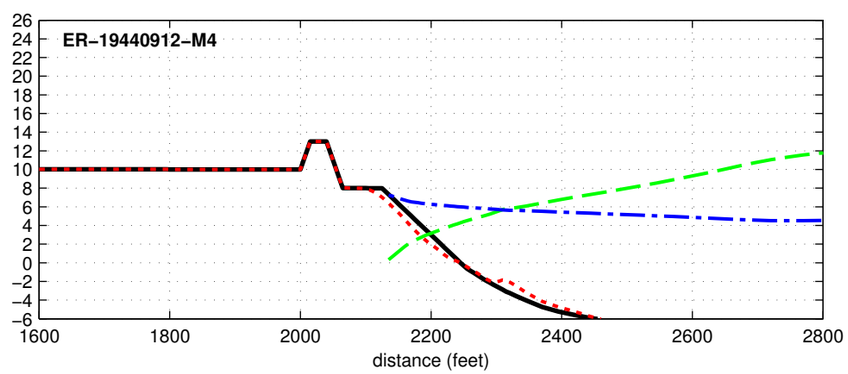






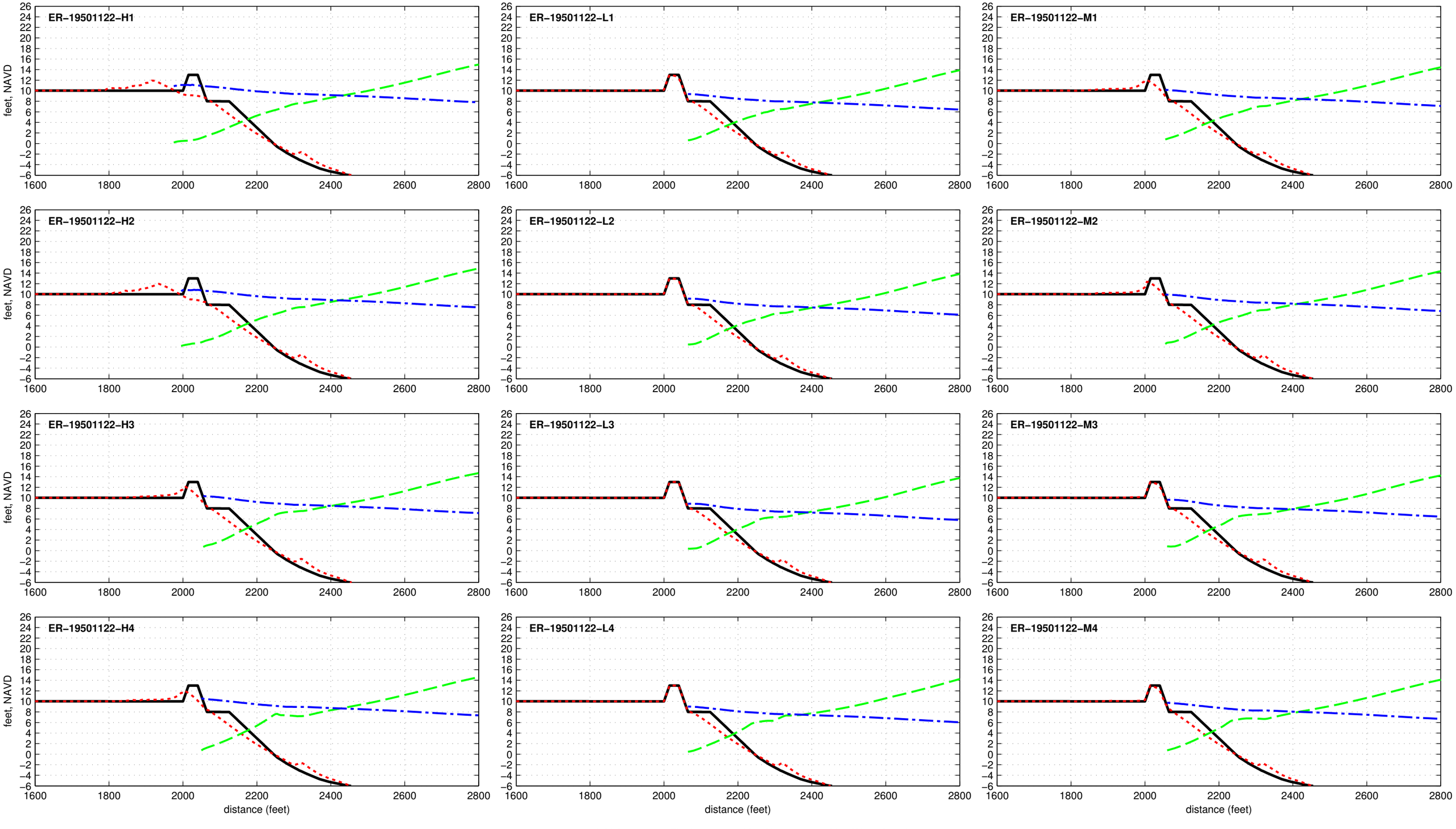






Initial Final Max Hs Max Water Level

Dune R2T1-13.0-025-060 - ER-19501122



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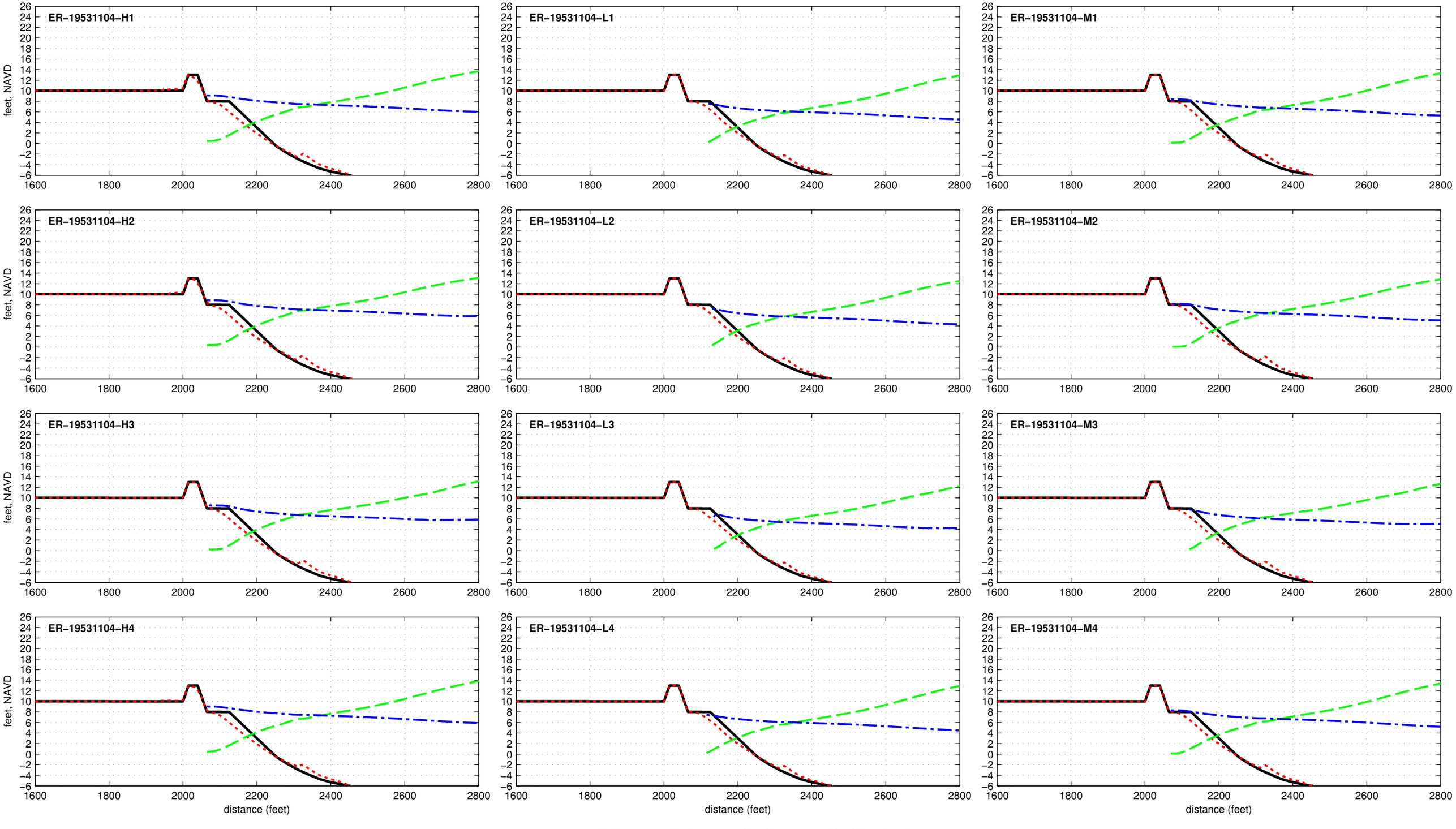

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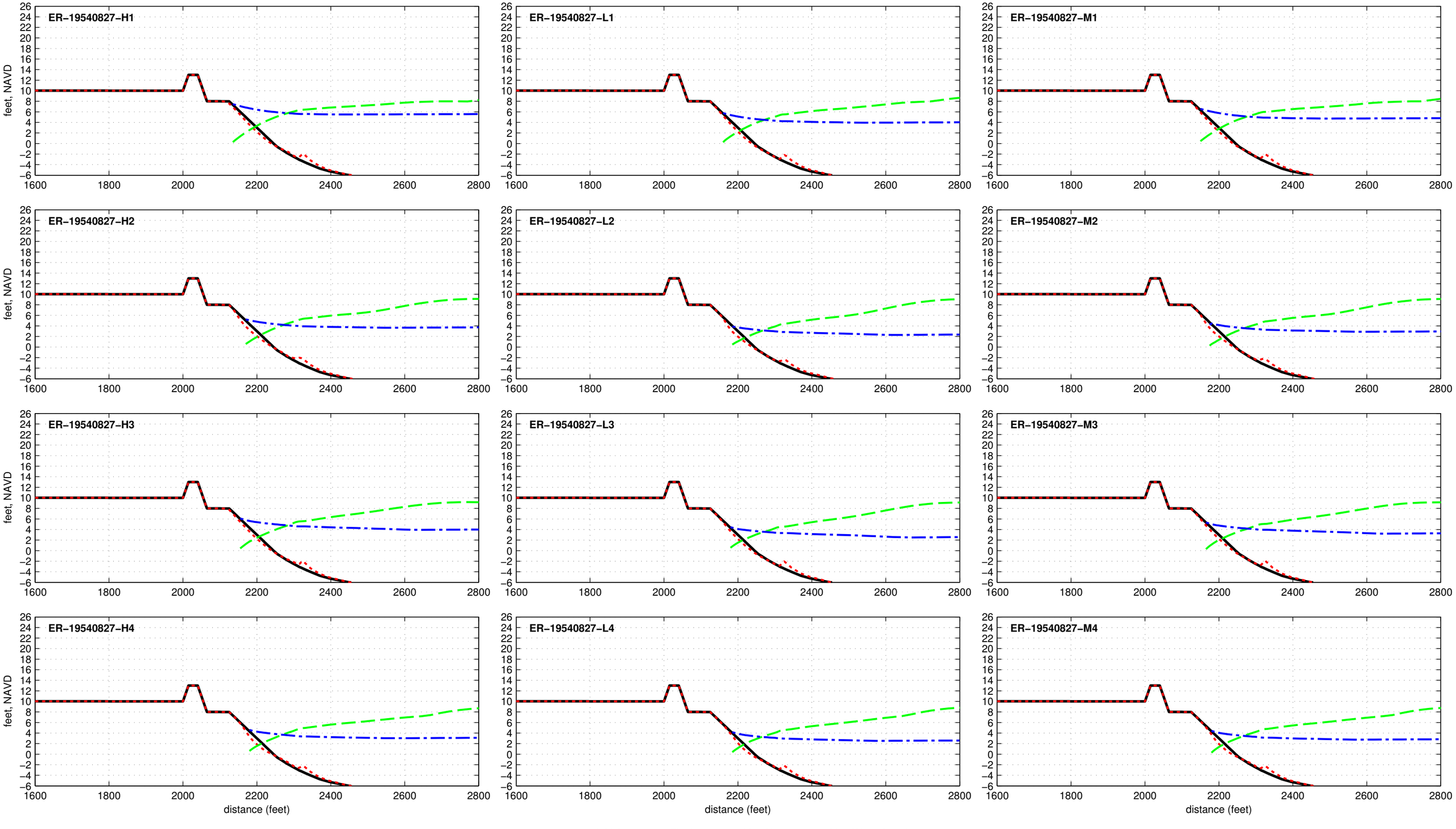
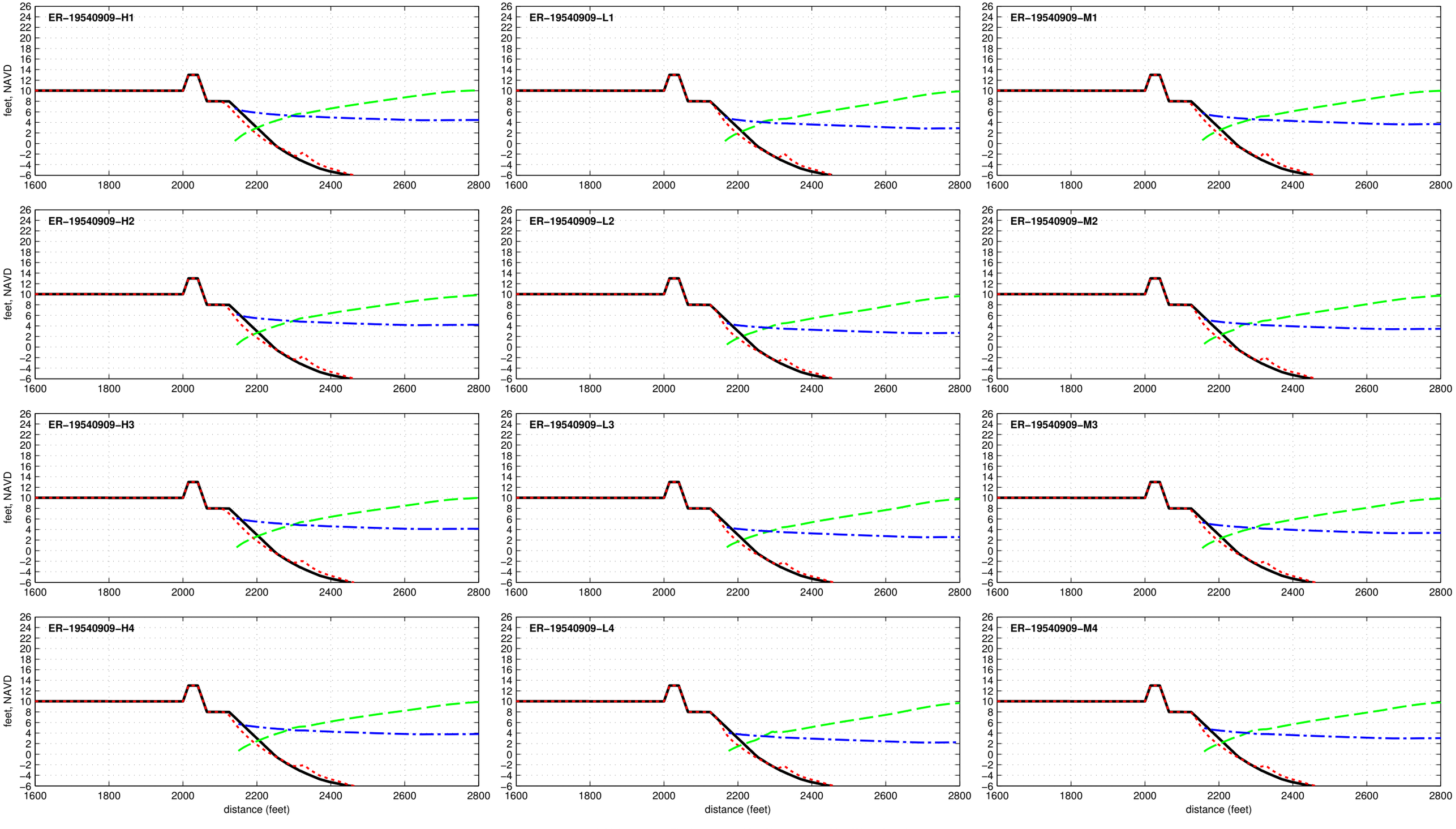
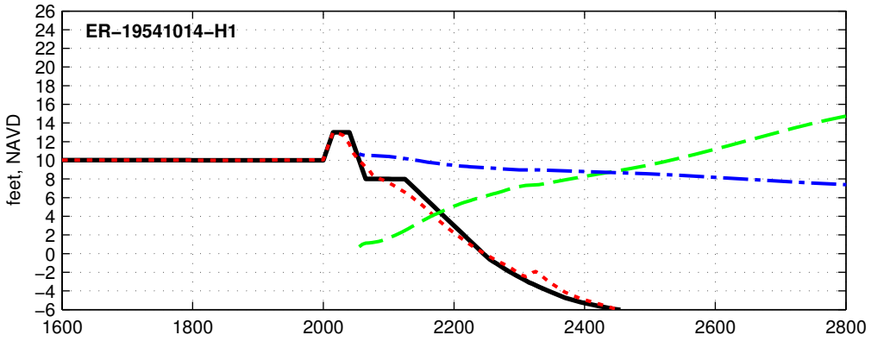
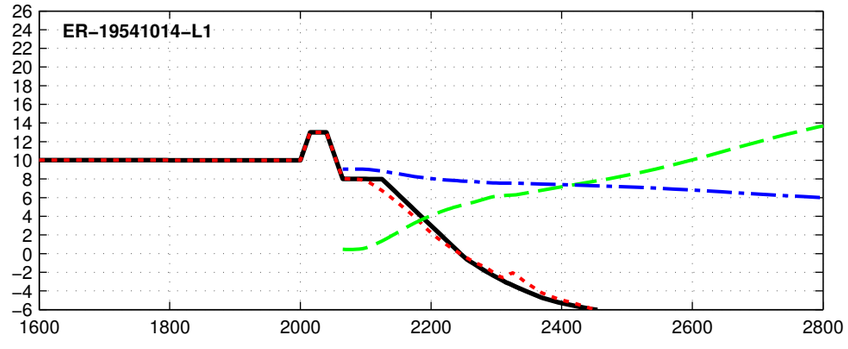


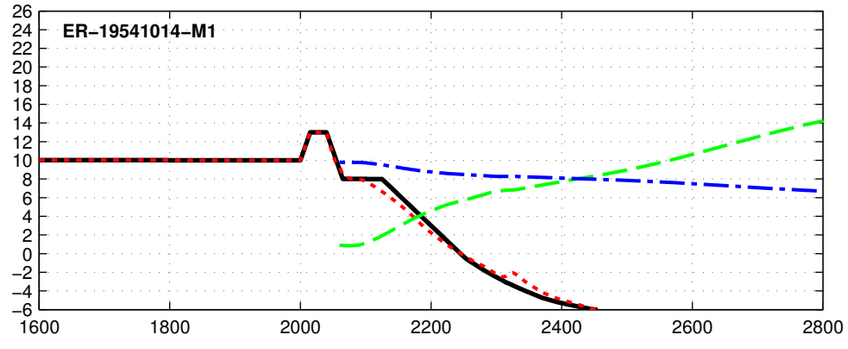
Dune R2T1−13.0−025−060 − ER−19540909

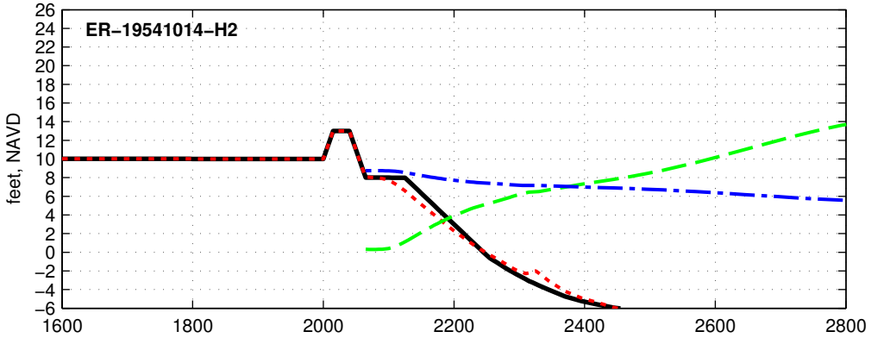


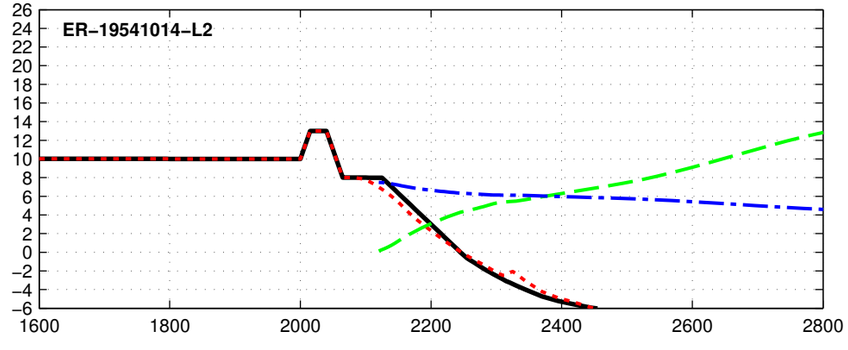
Dune R2T1–13.0–025–060 – ER–19541014

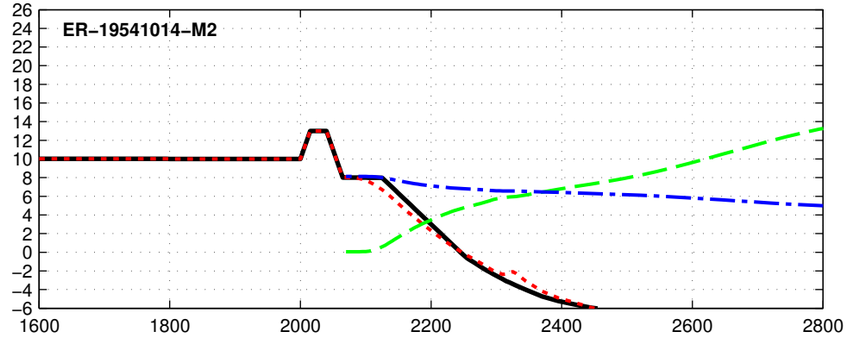


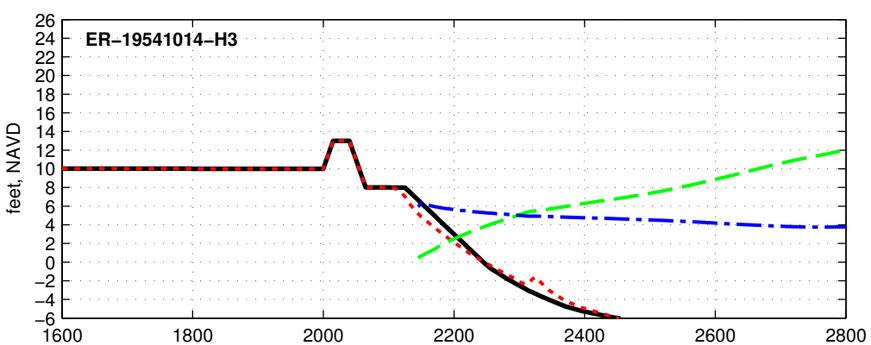


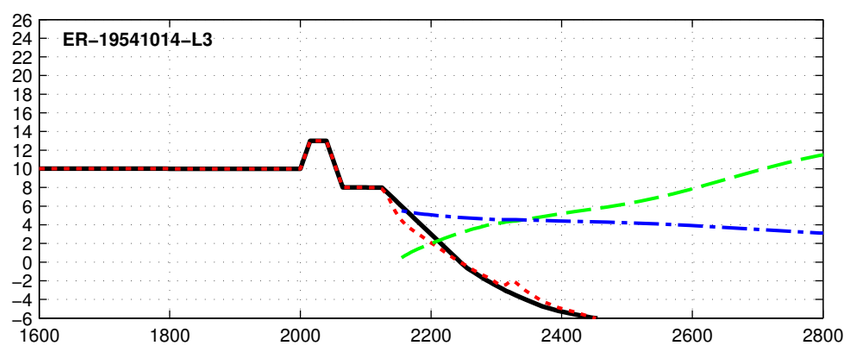


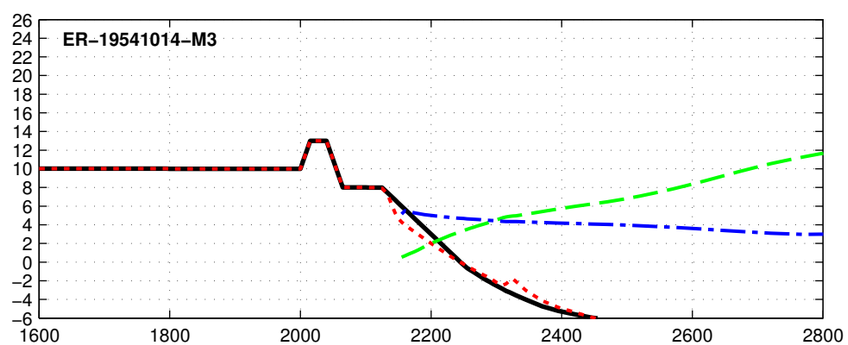


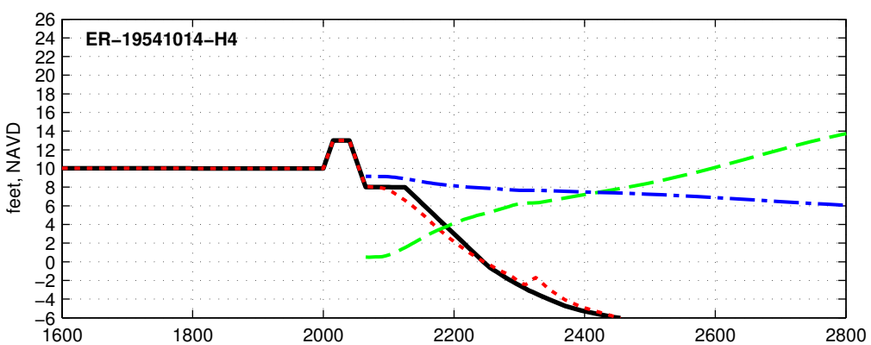


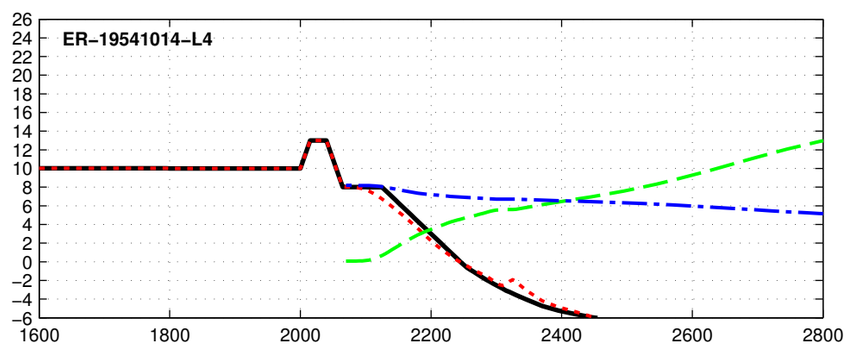


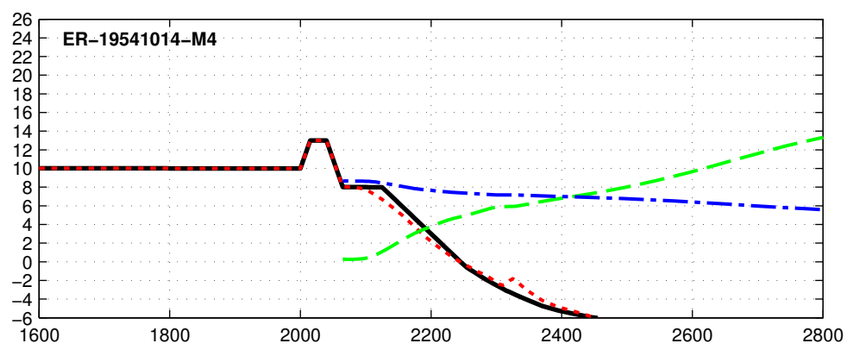






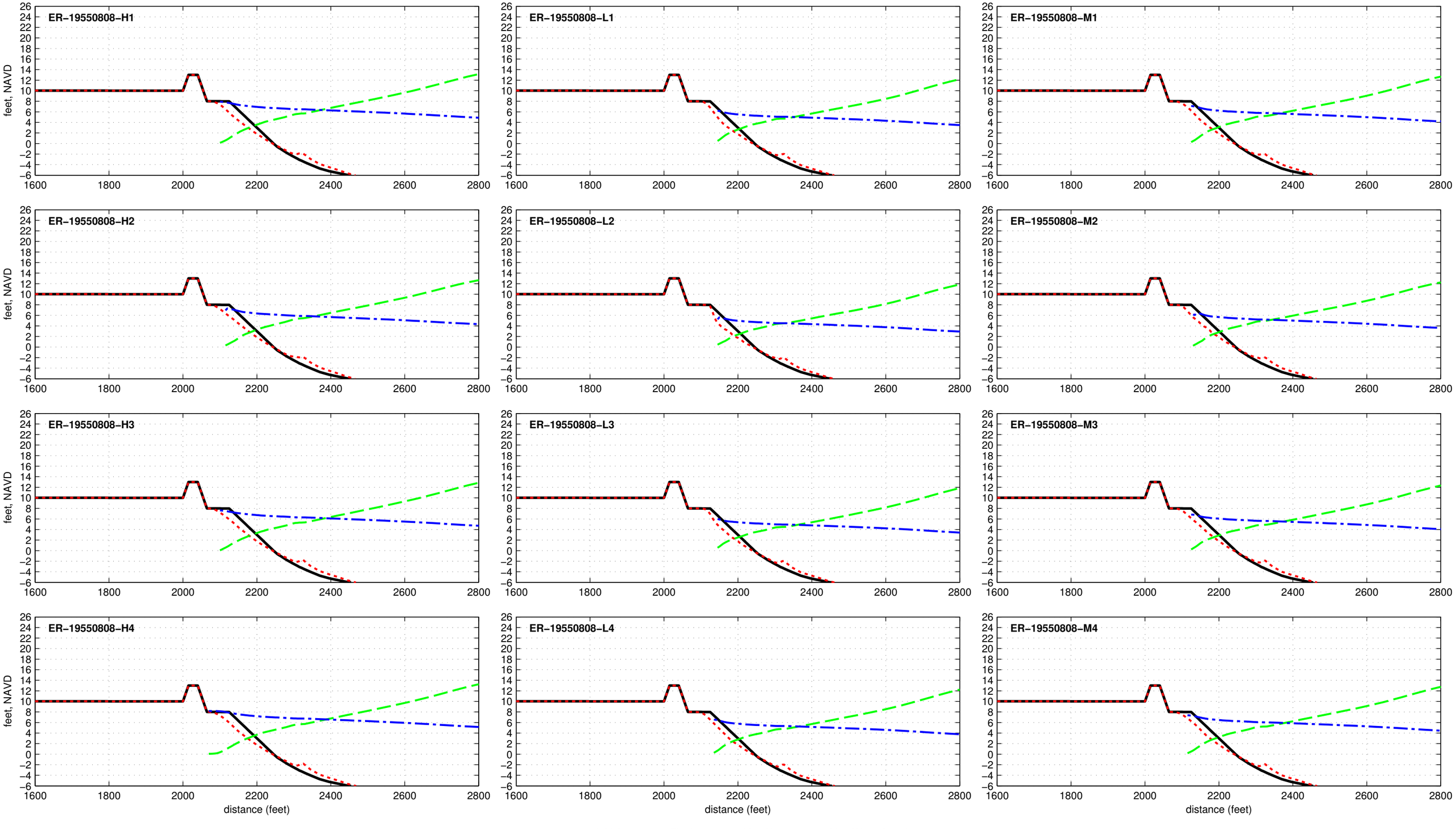




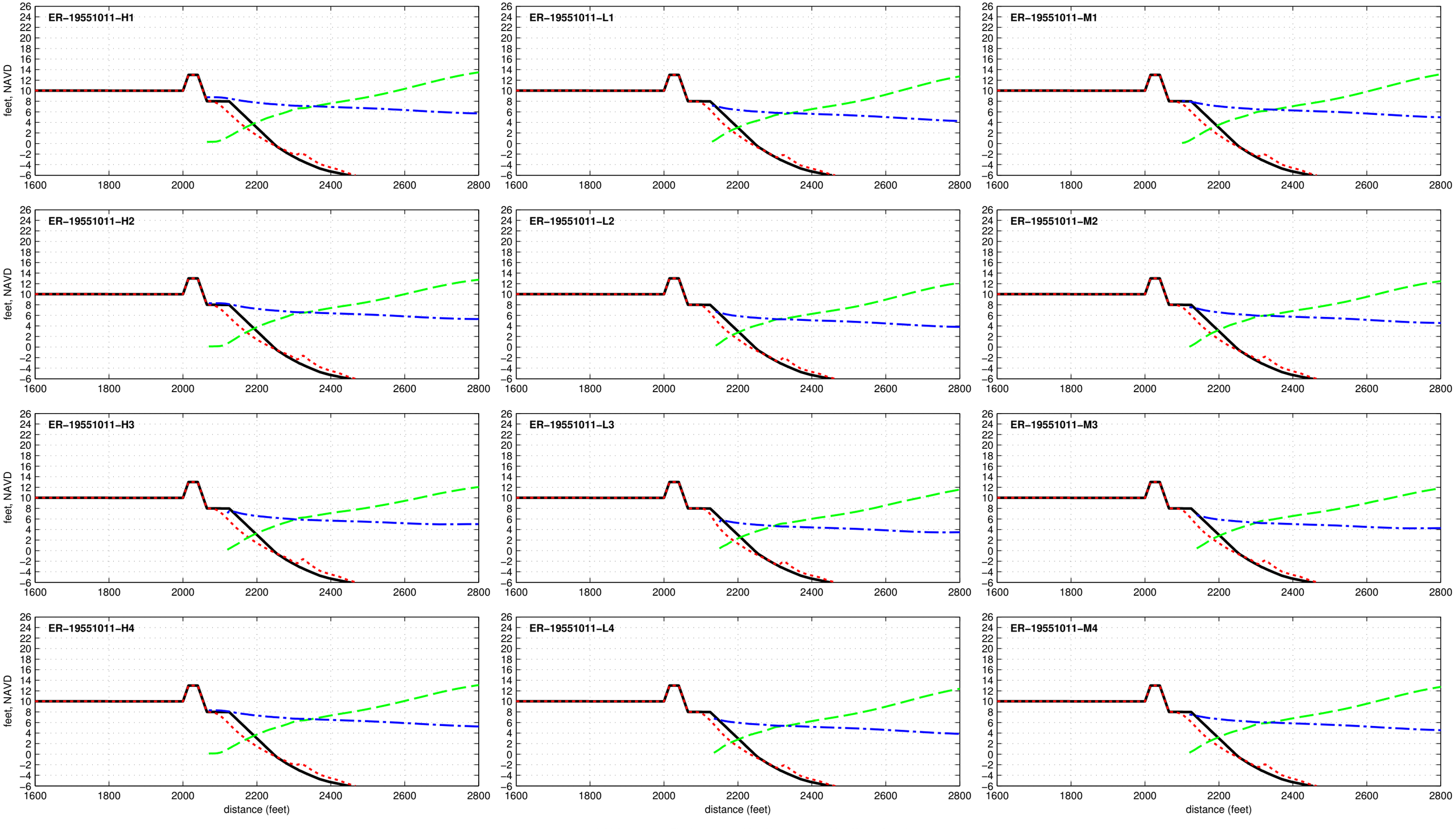


Initial Final Max Hs Max Water Level

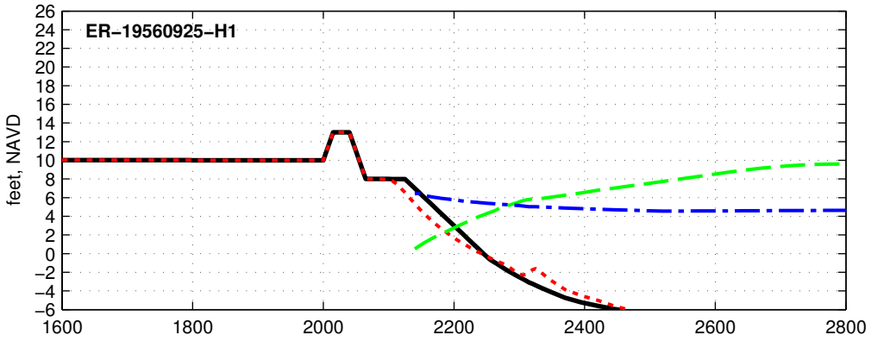
Dune R2T1−13.0−025−060 − ER−19550808

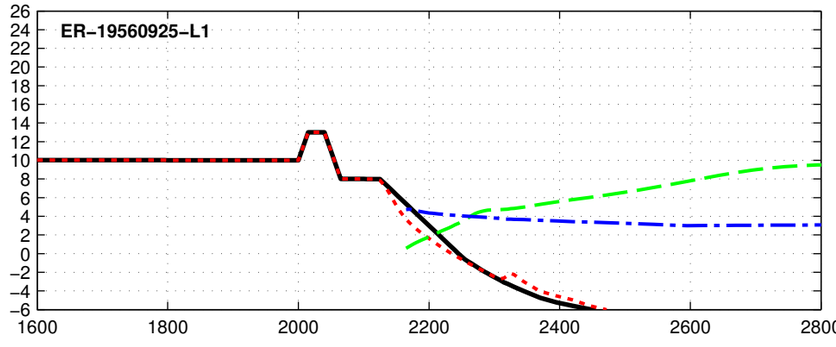


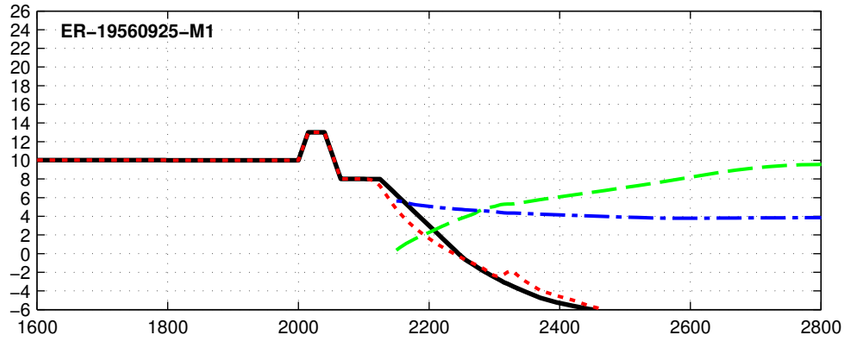
Dune R2T1-13.0-025-060 - ER-19551011

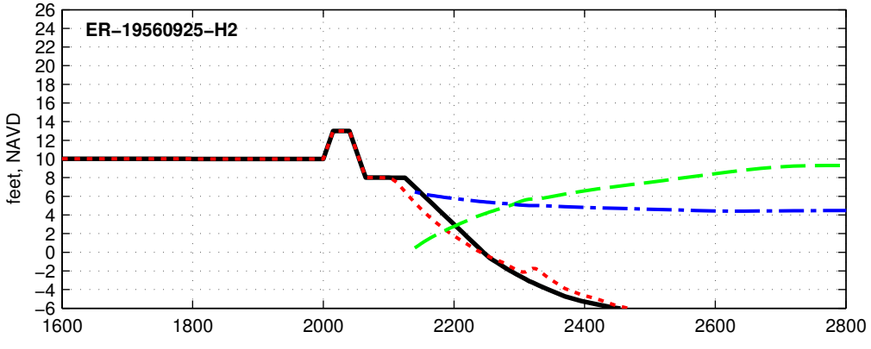


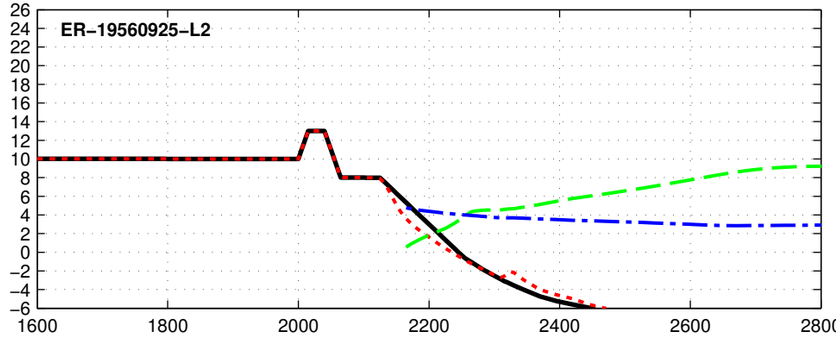
Dune R2T1-13.0-025-060 - ER-19560925

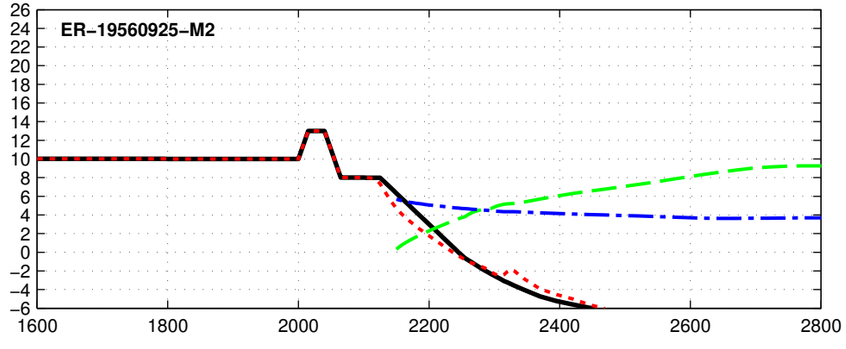


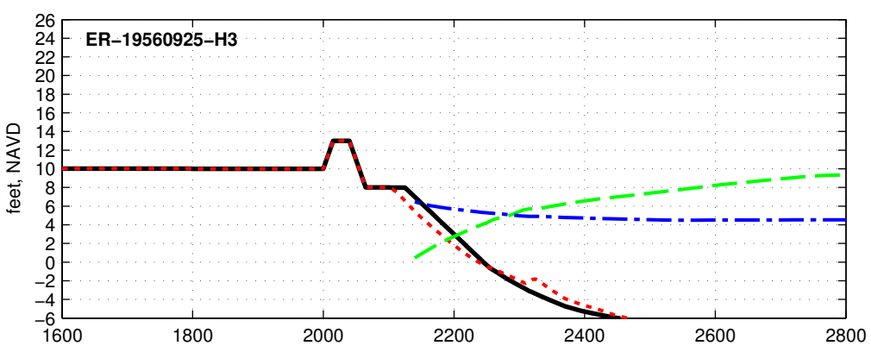


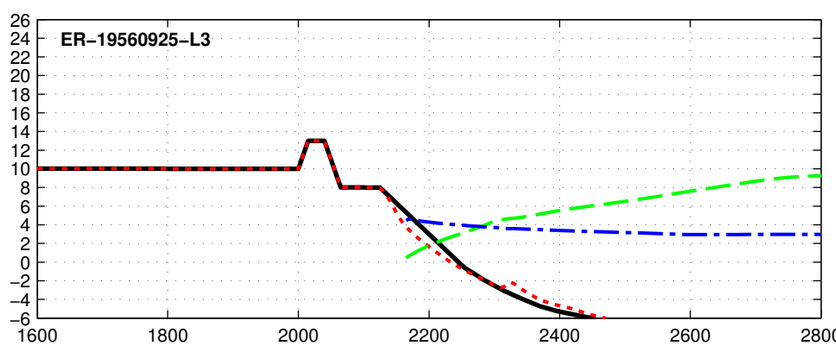


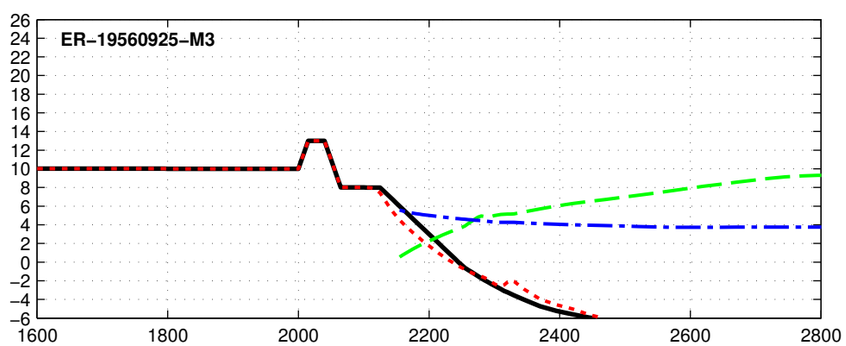


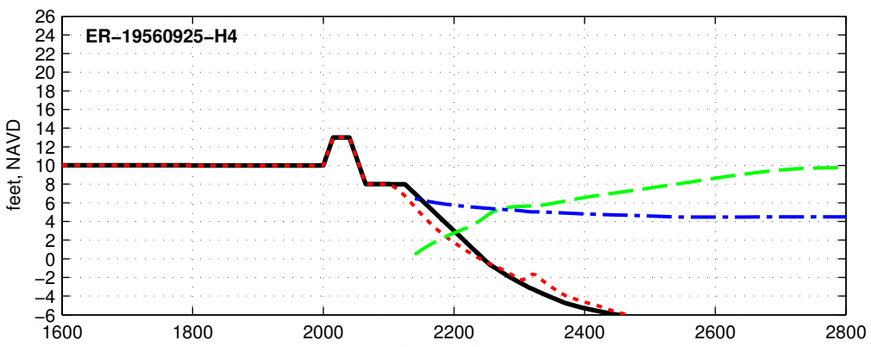


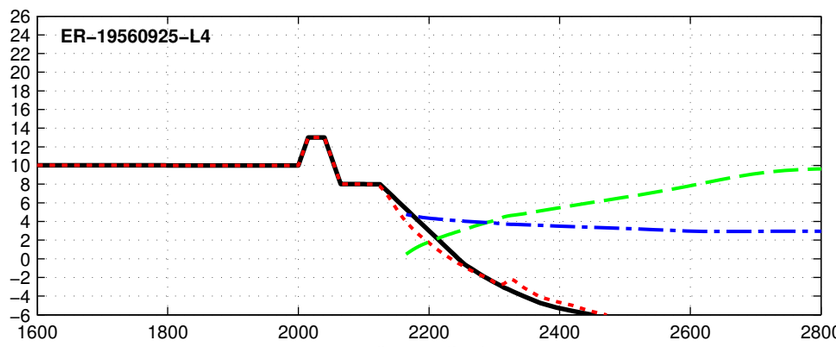


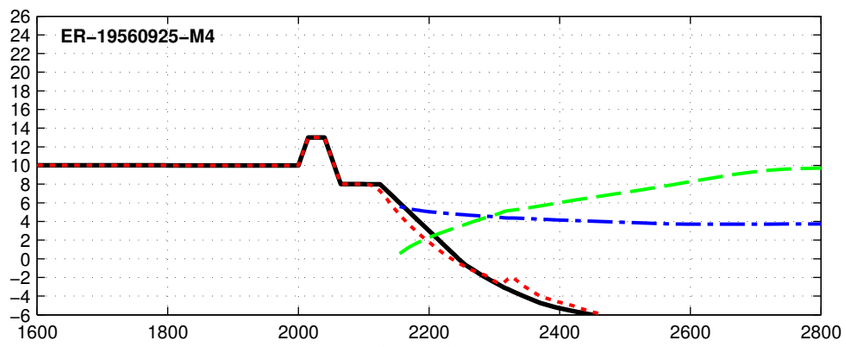






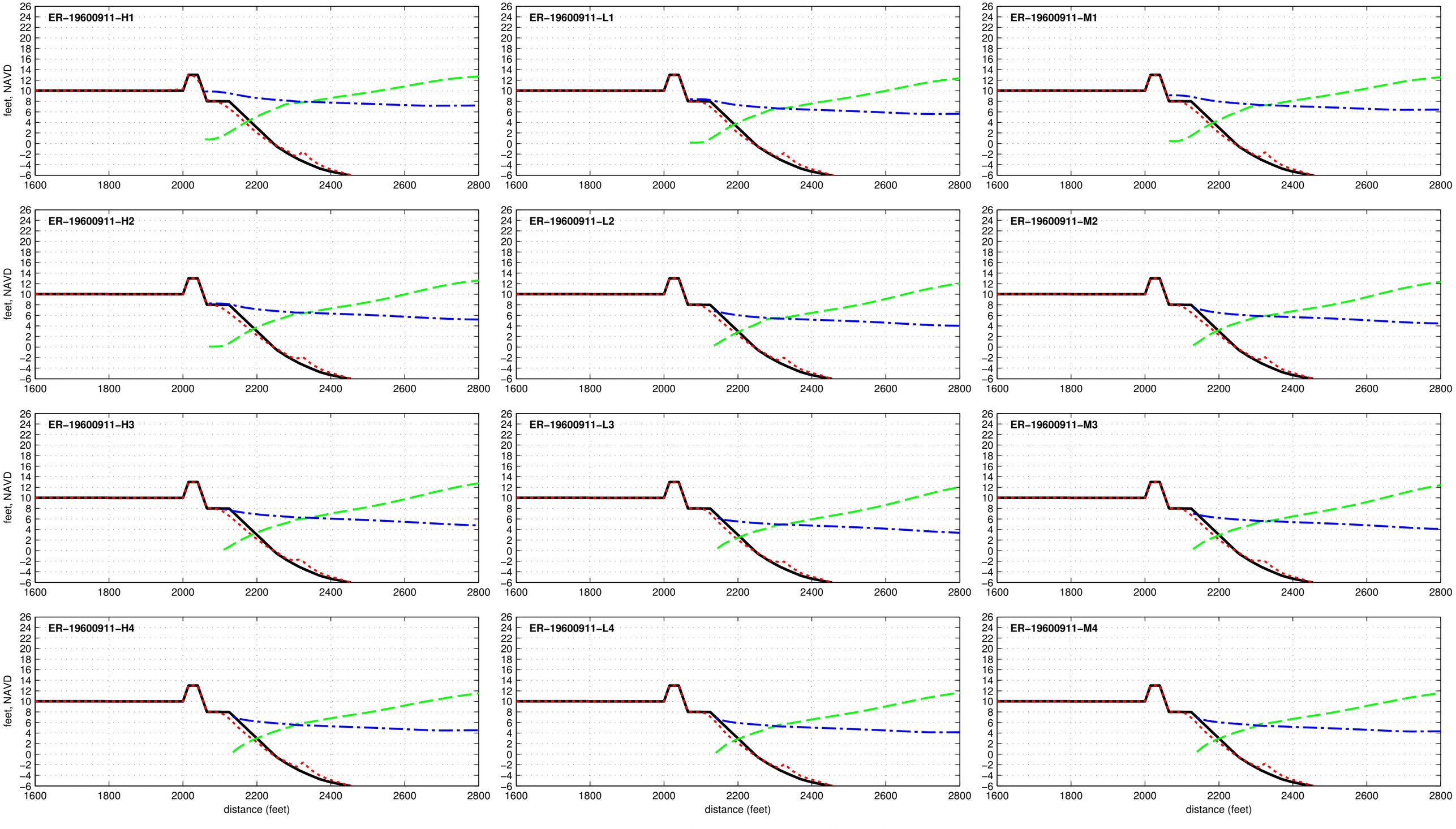




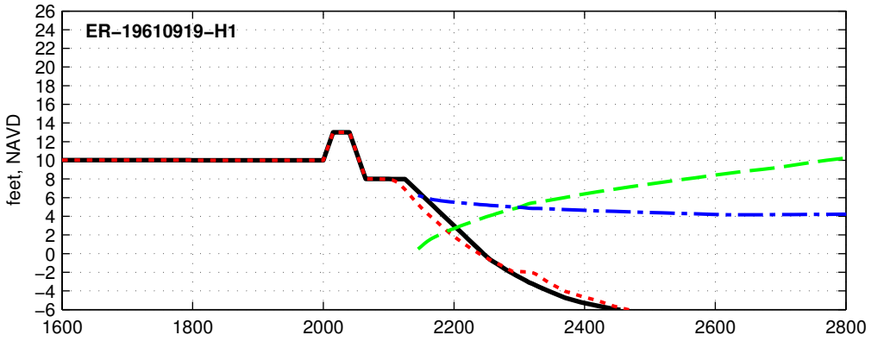


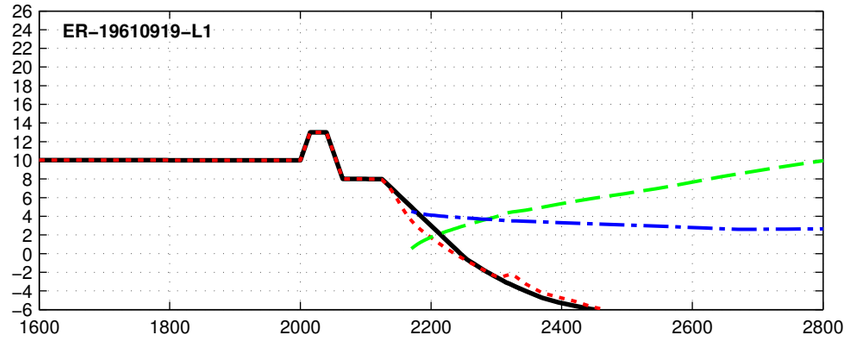
Initial Final Max Hs Max Water Level

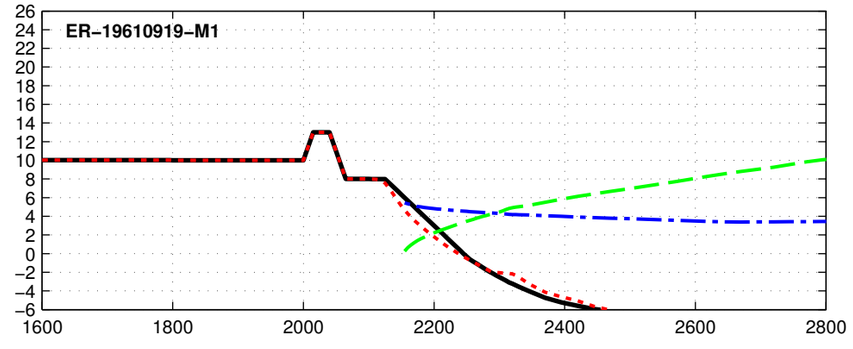
Dune R2T1-13.0-025-060 - ER-19600911

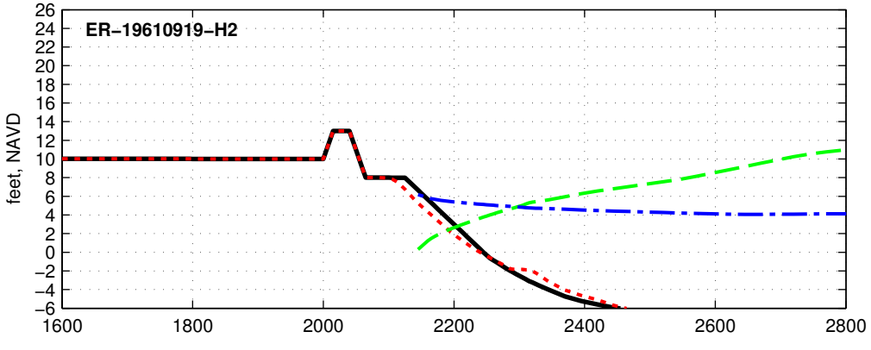


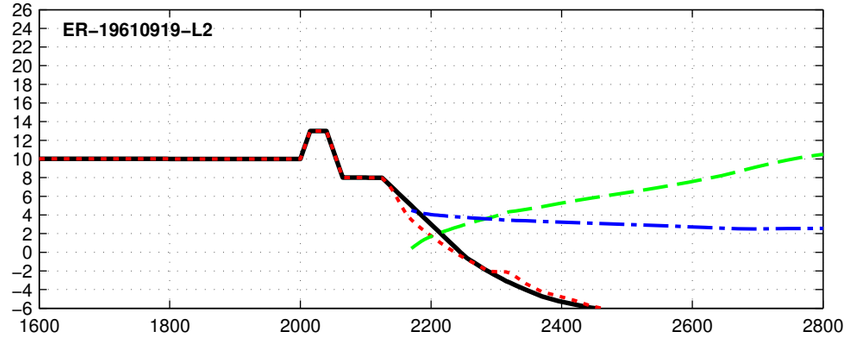
Dune R2T1-13.0-025-060 - ER-19610919

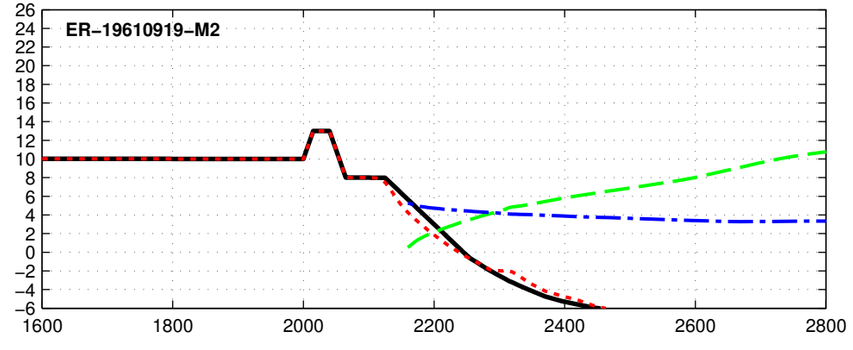


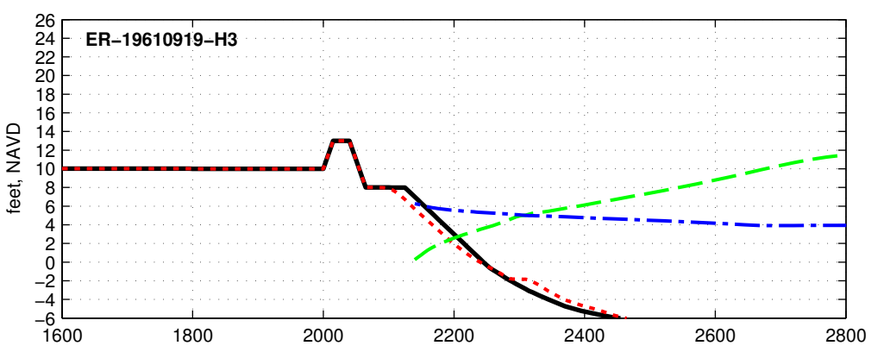


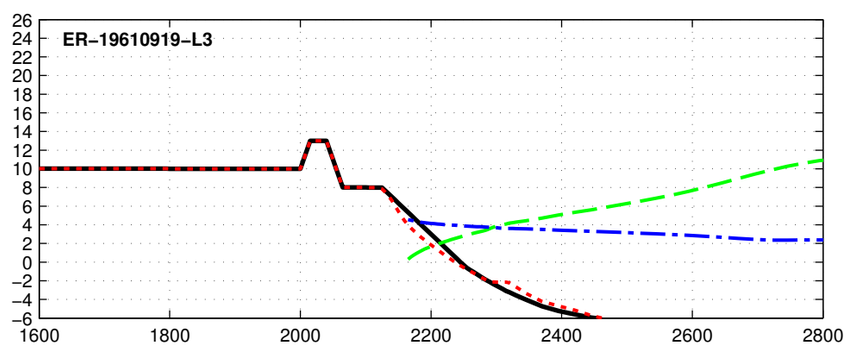


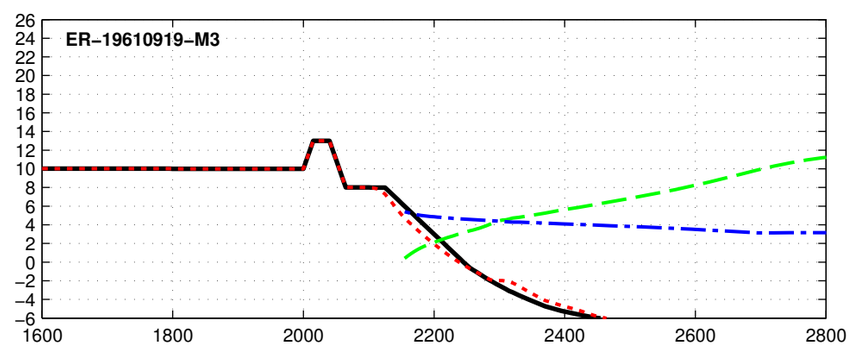


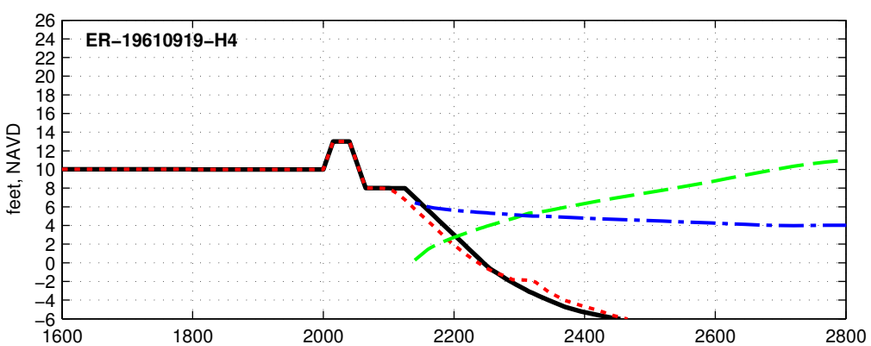


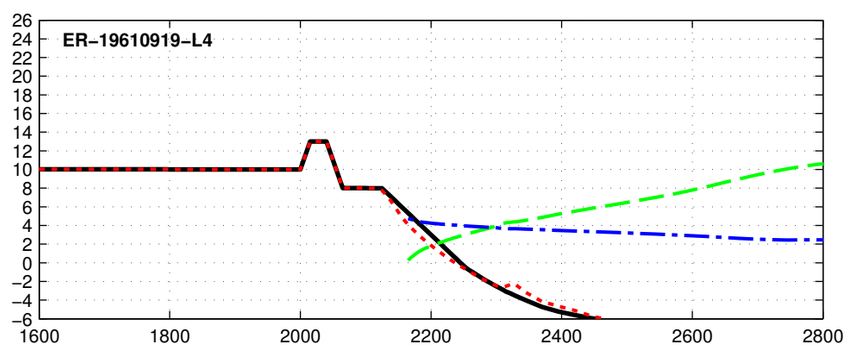


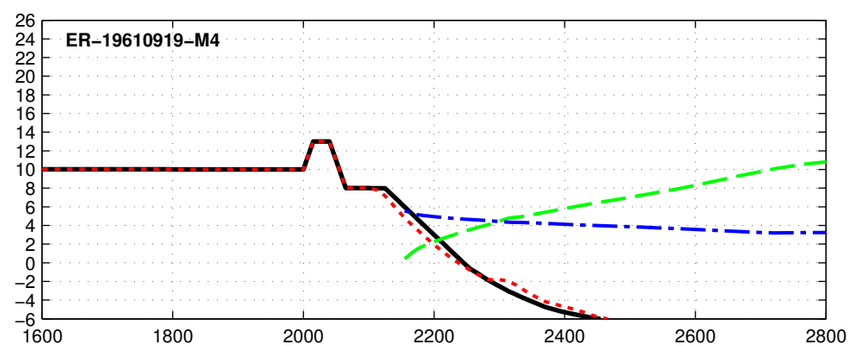






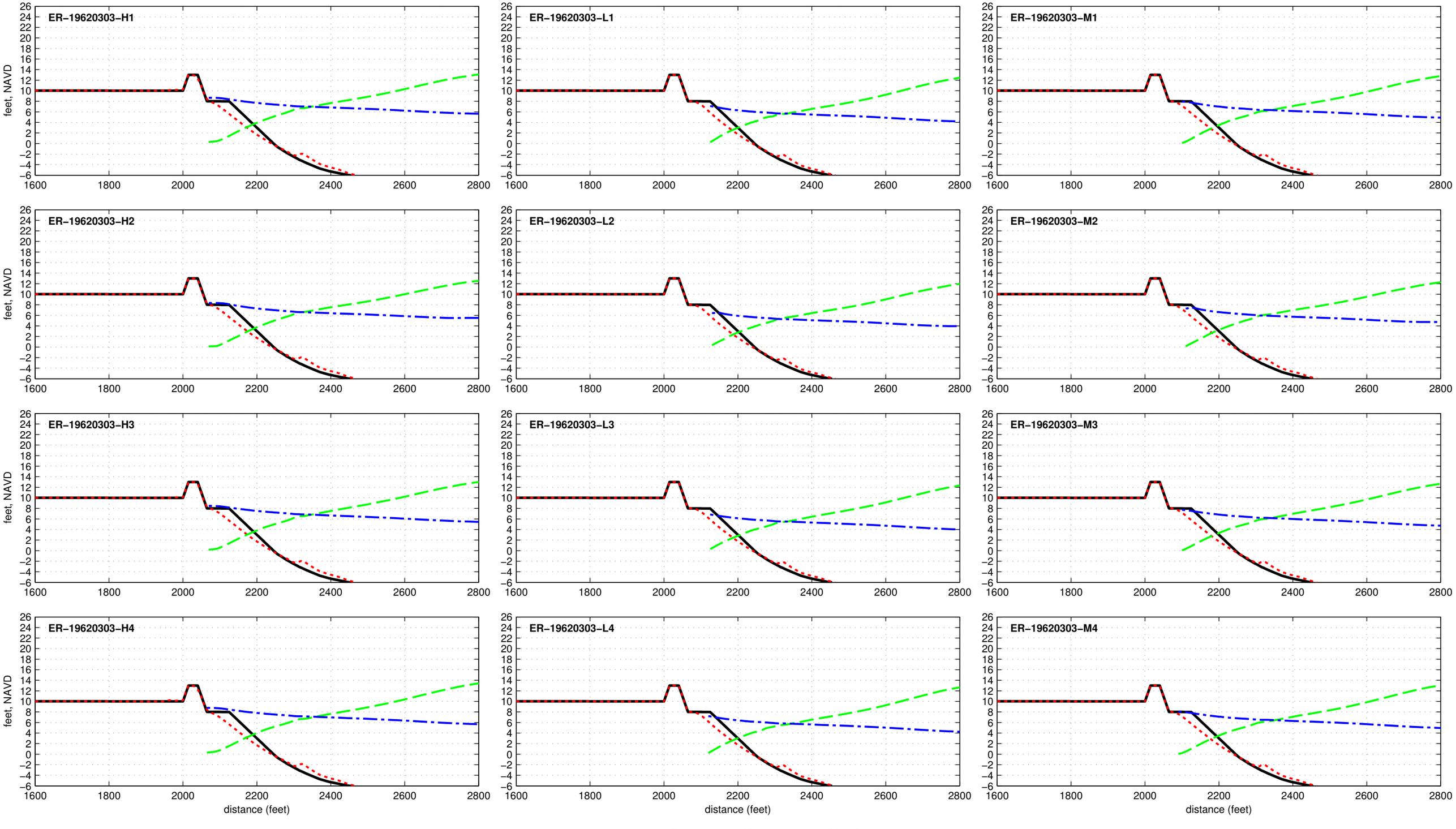




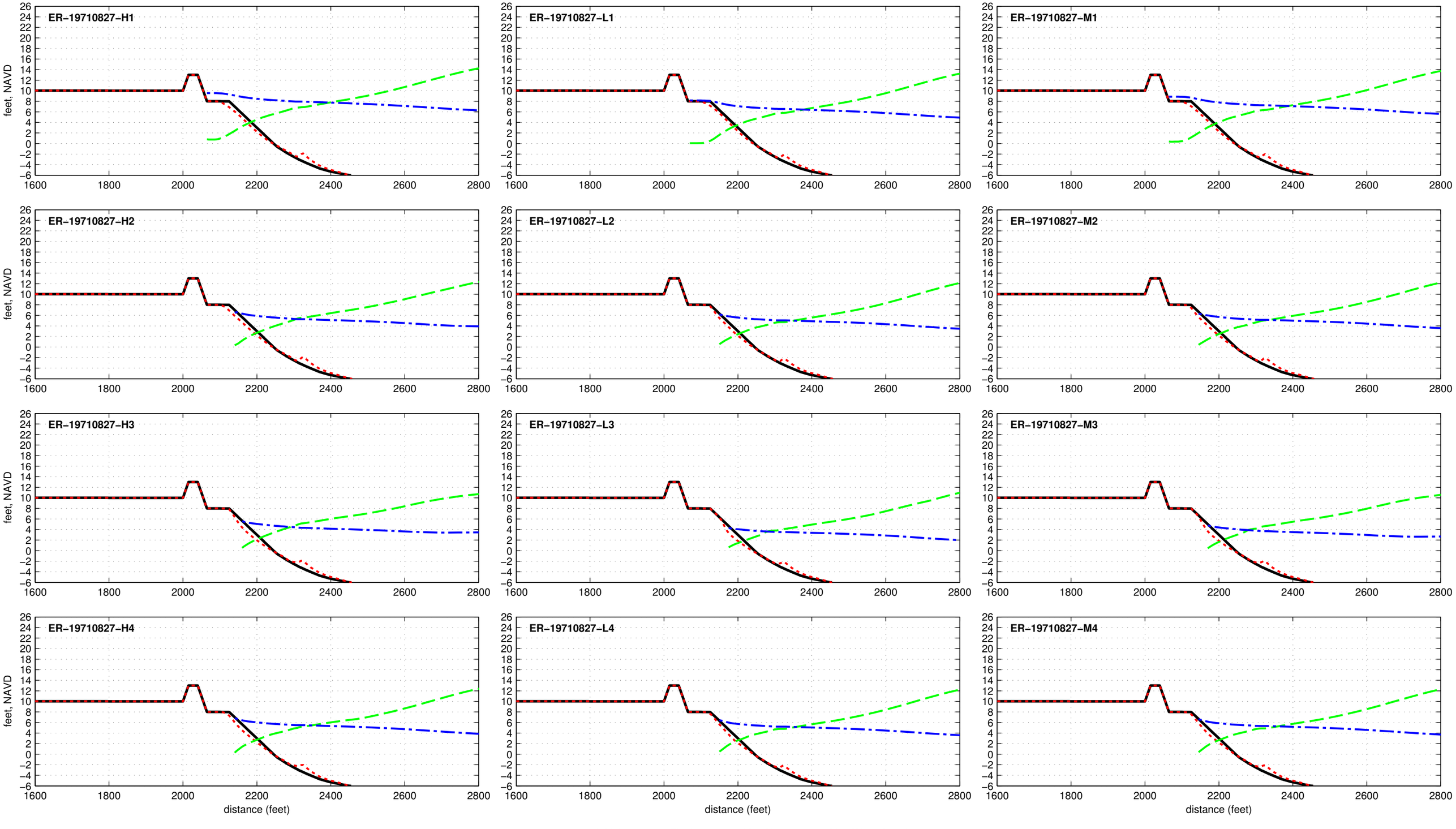


Initial Final Max Hs Max Water Level

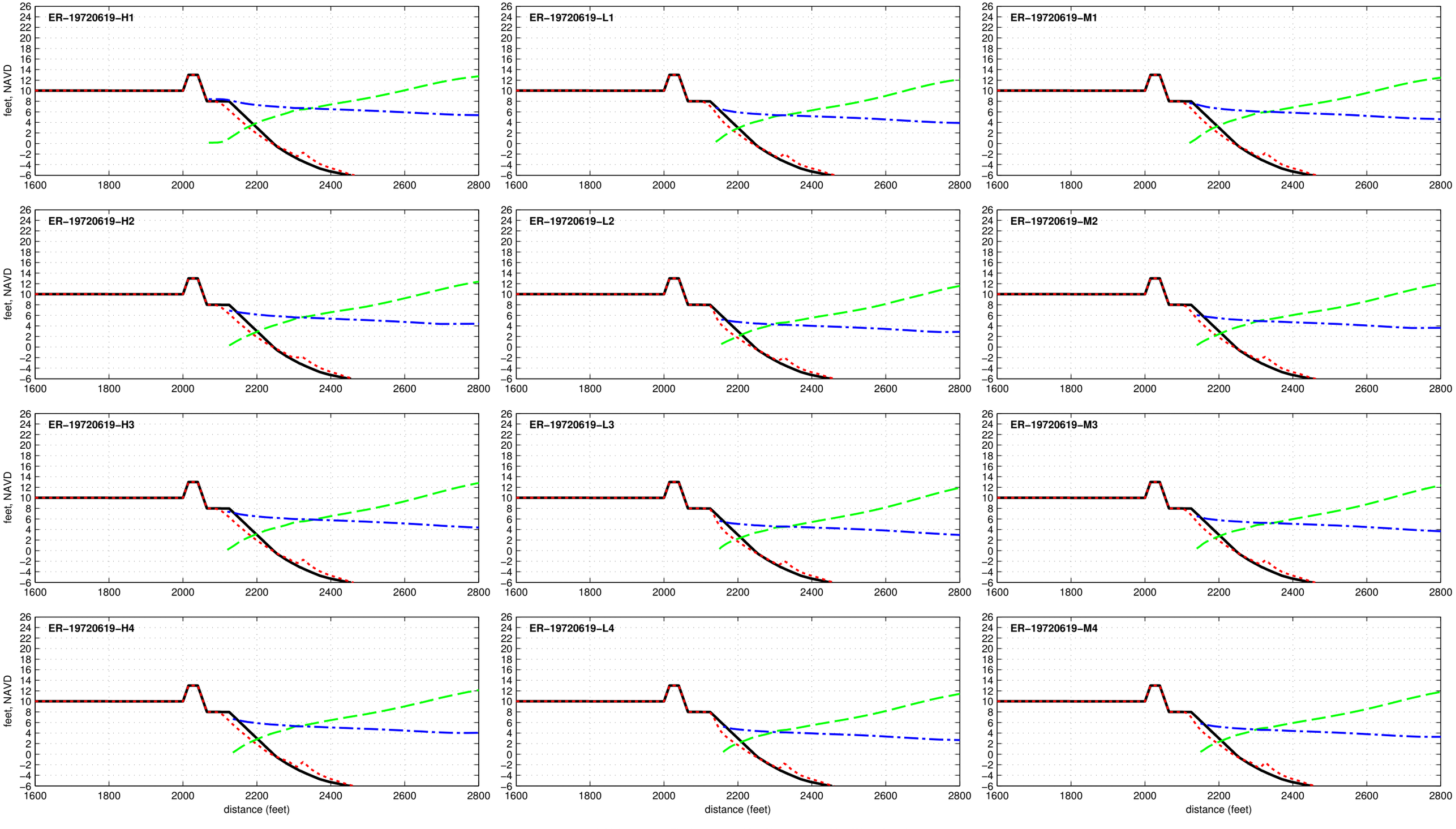
Dune R2T1–13.0–025–060 – ER–19620303



Dune R2T1–13.0–025–060 – ER–19710827



Dune R2T1-13.0-025-060 - ER-19720619



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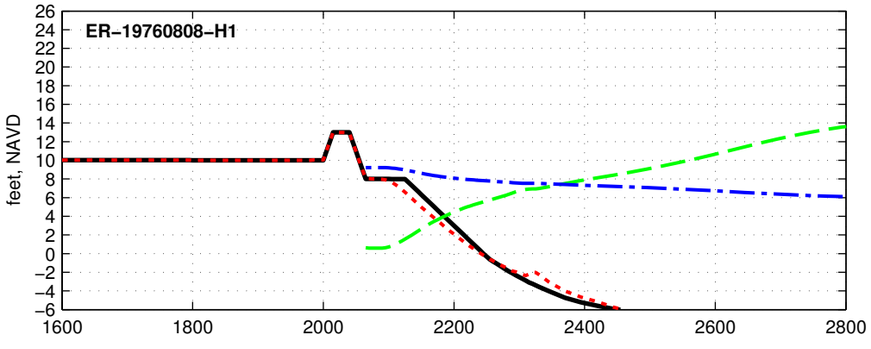
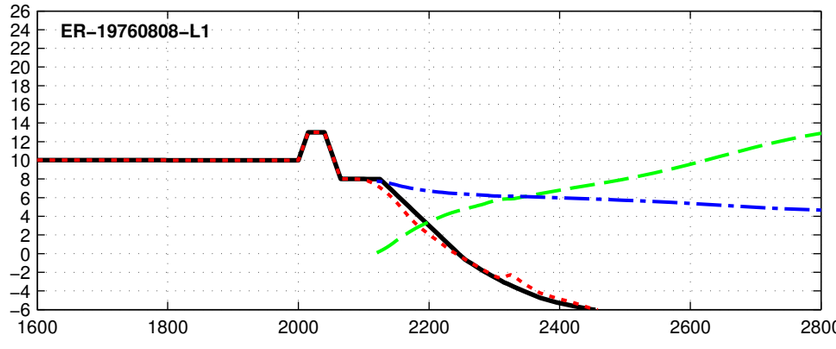
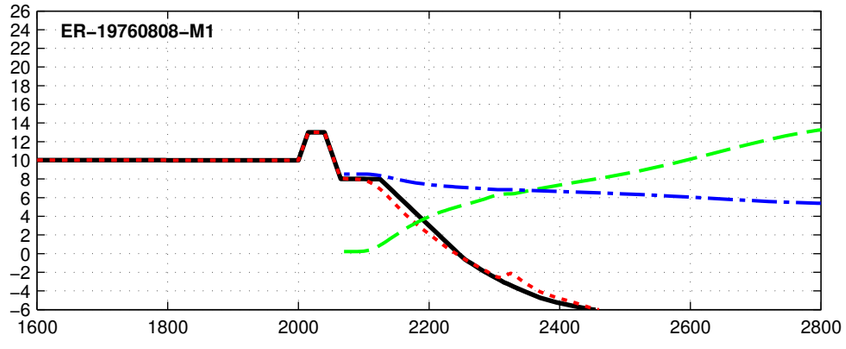
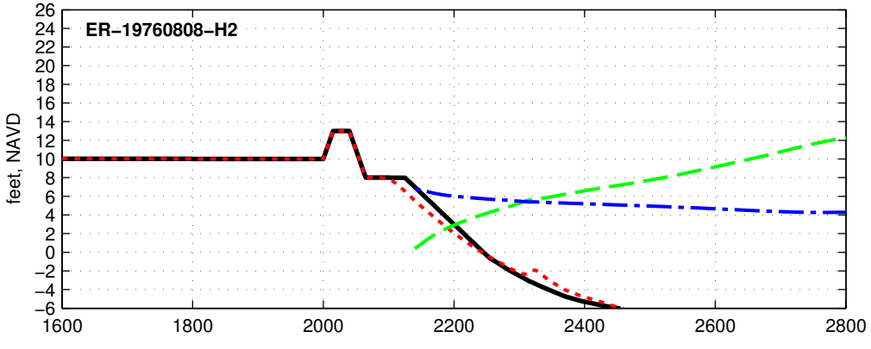
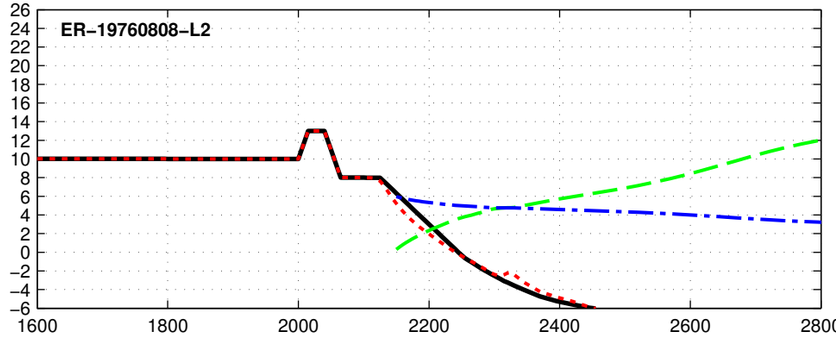
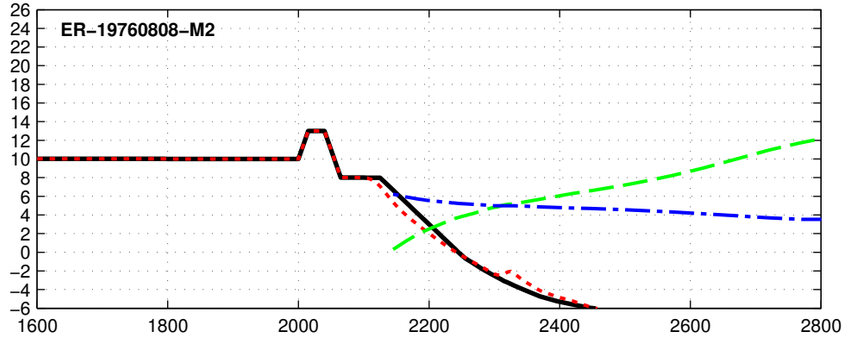


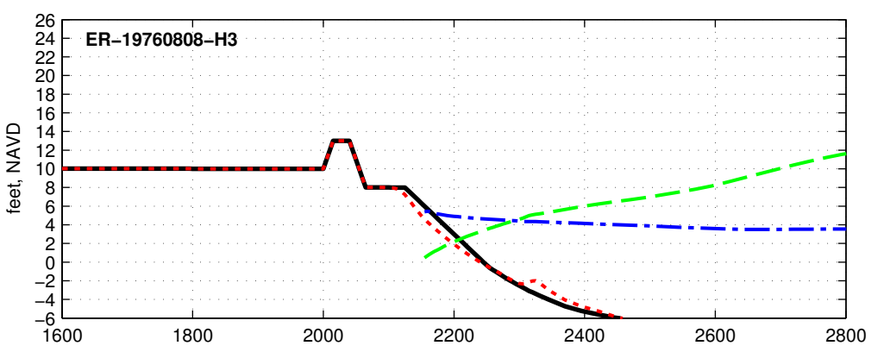


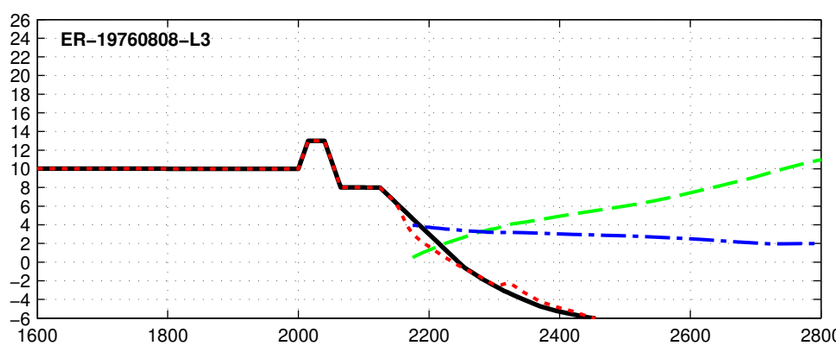


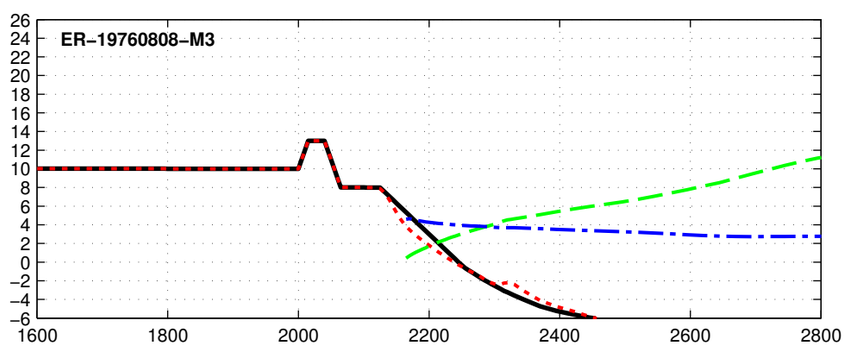


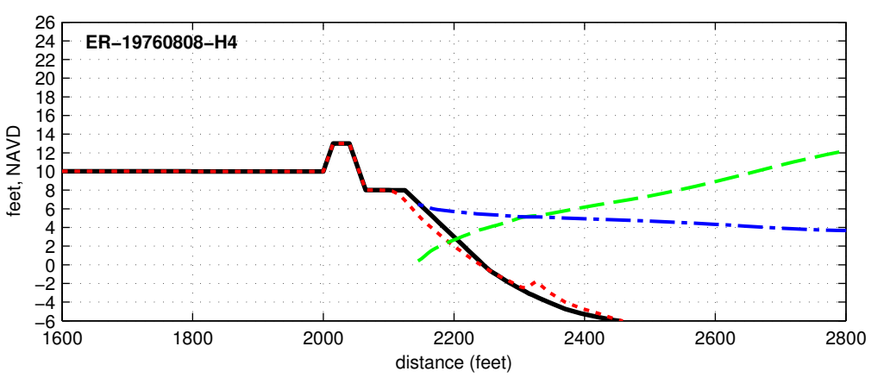


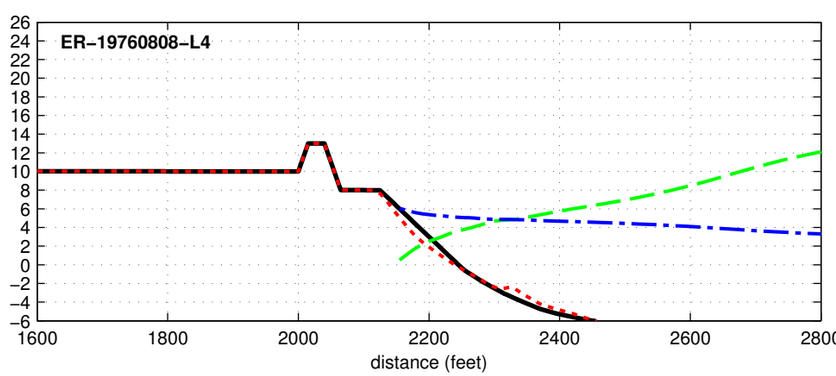


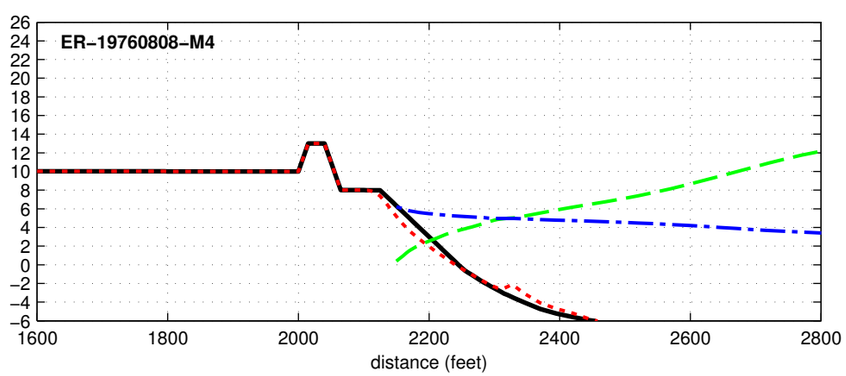






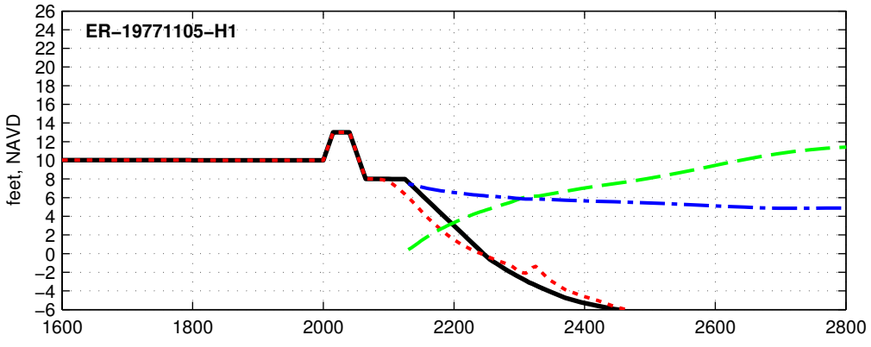


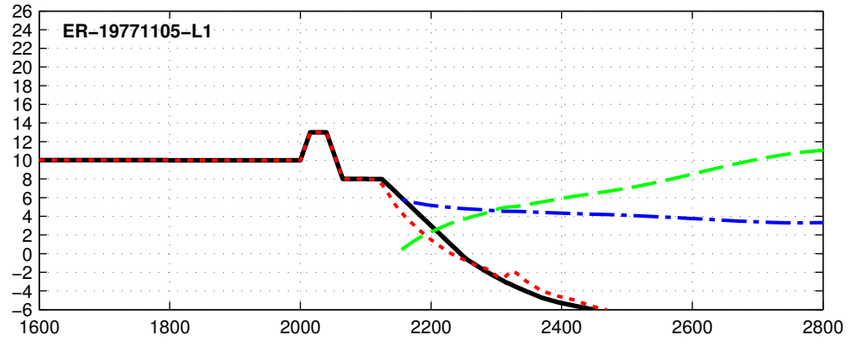


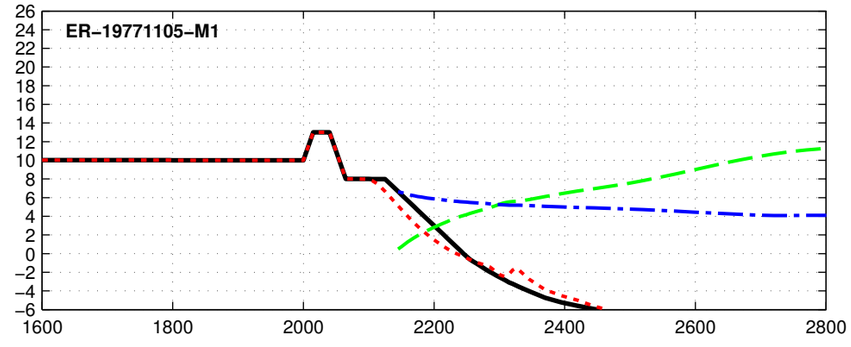


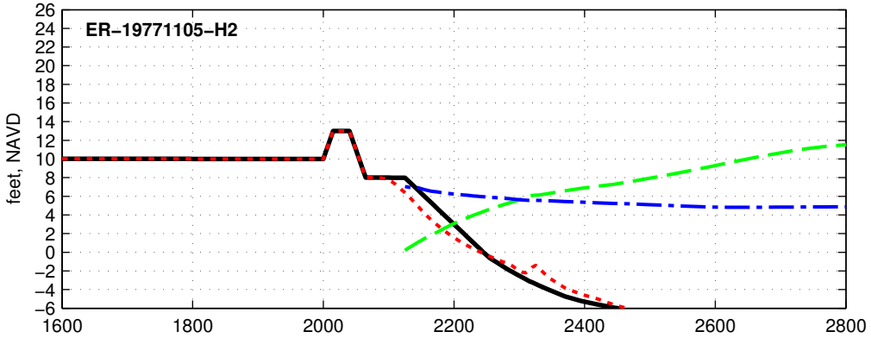
Initial Final Max Hs Max Water Level

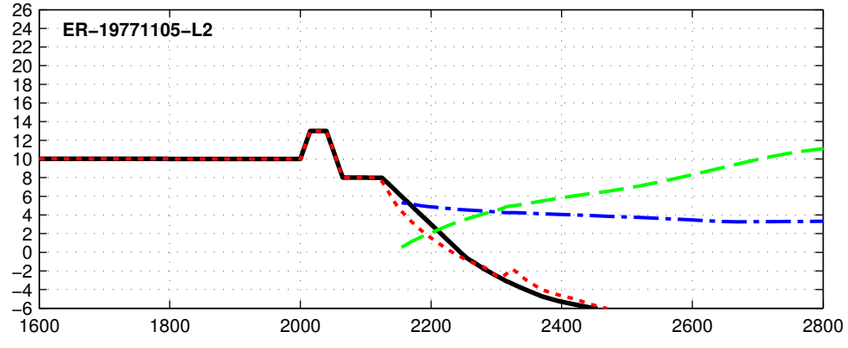
Dune R2T1-13.0-025-060 - ER-19771105

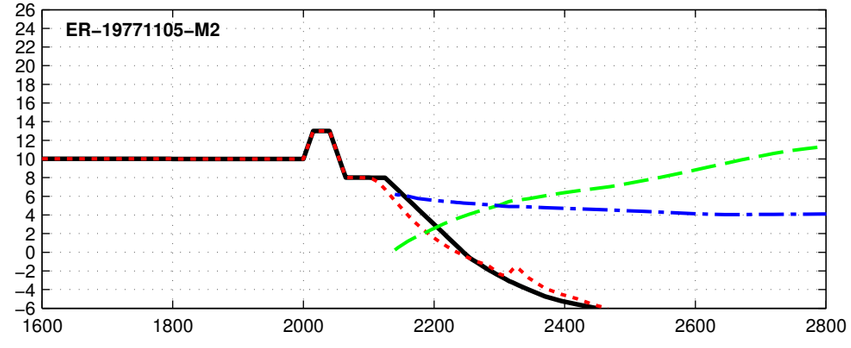


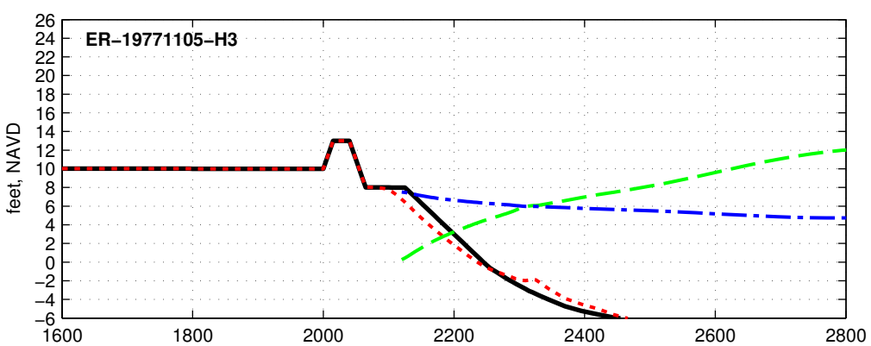


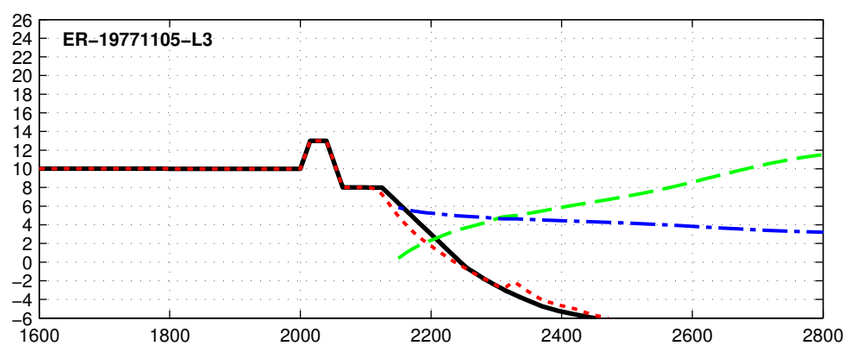


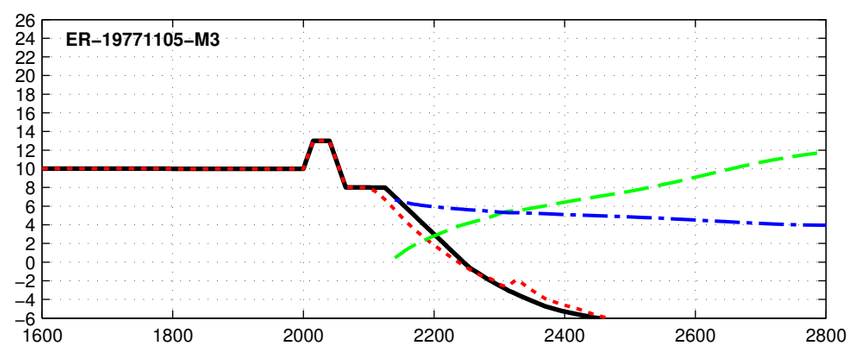


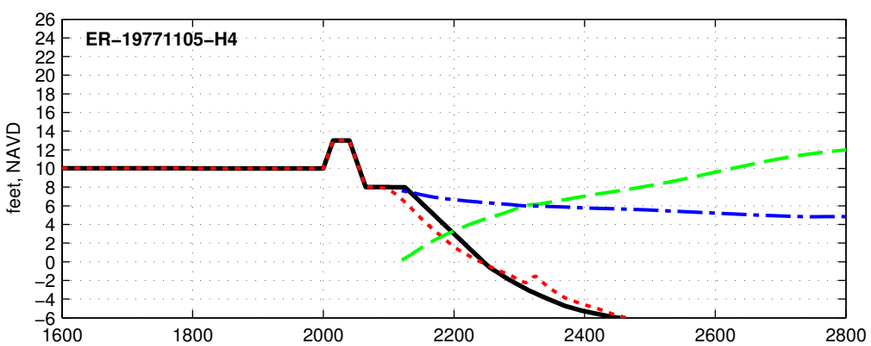


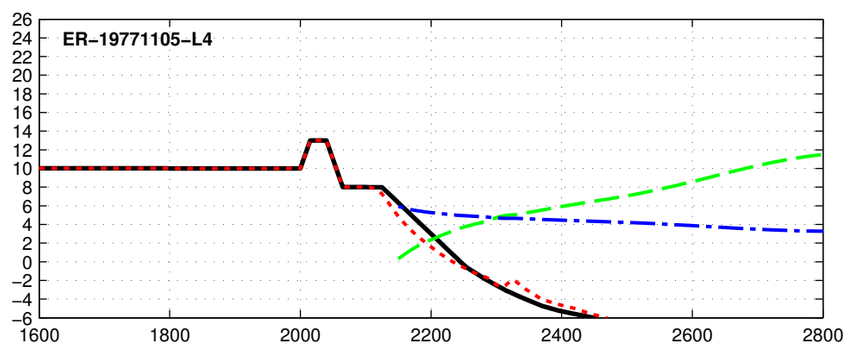


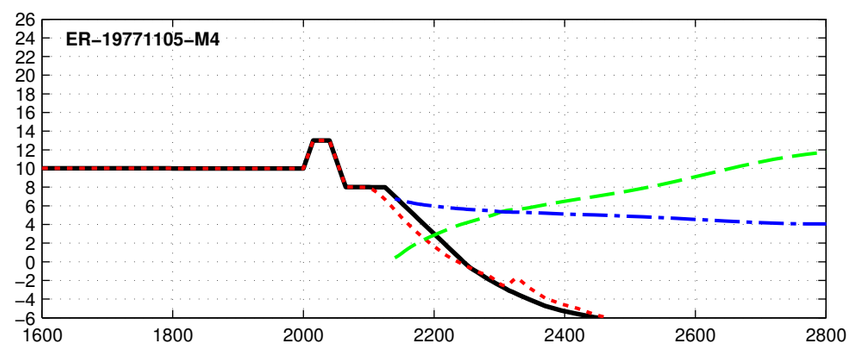






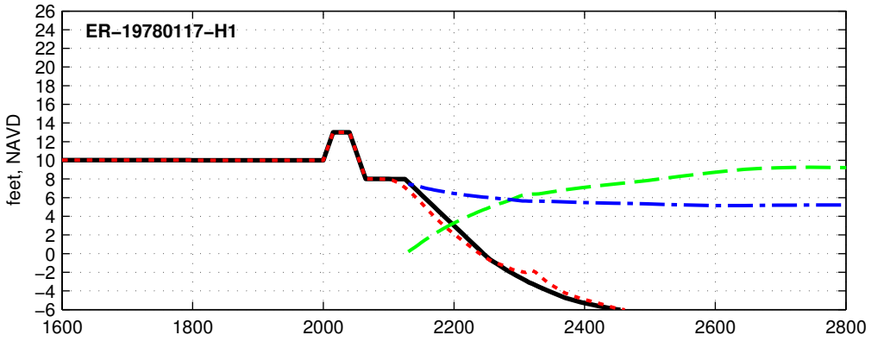


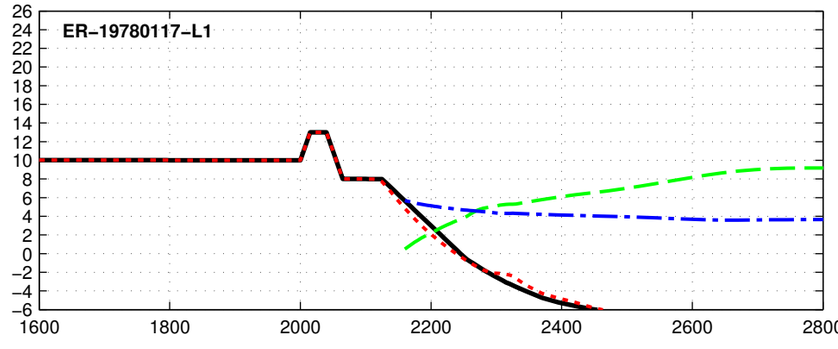


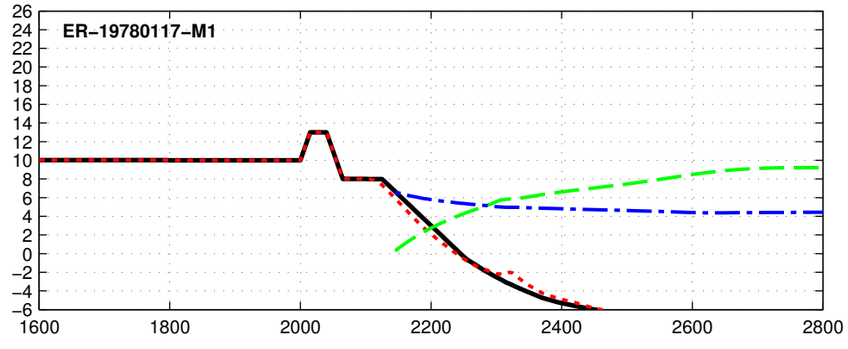


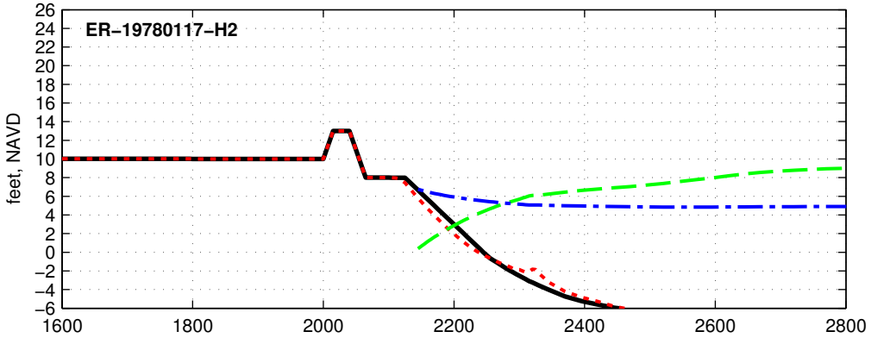
Initial Final Max Hs Max Water Level

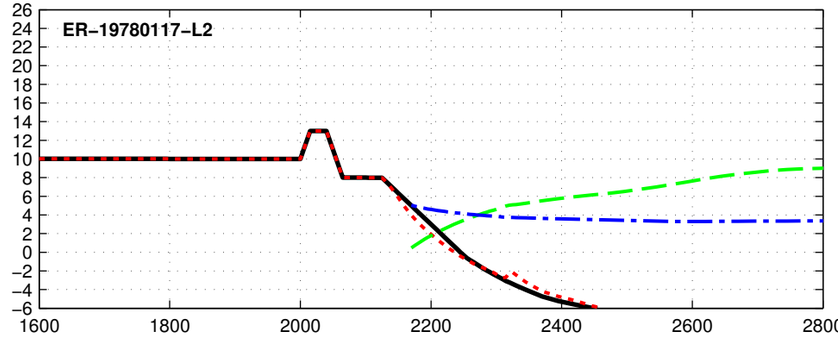
Dune R2T1–13.0–025–060 – ER–19780117

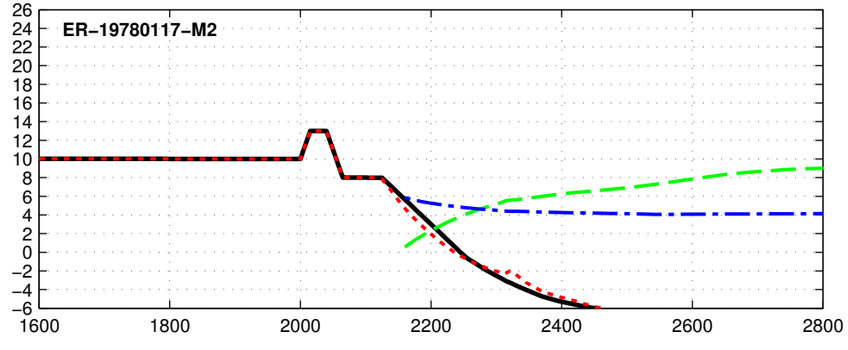


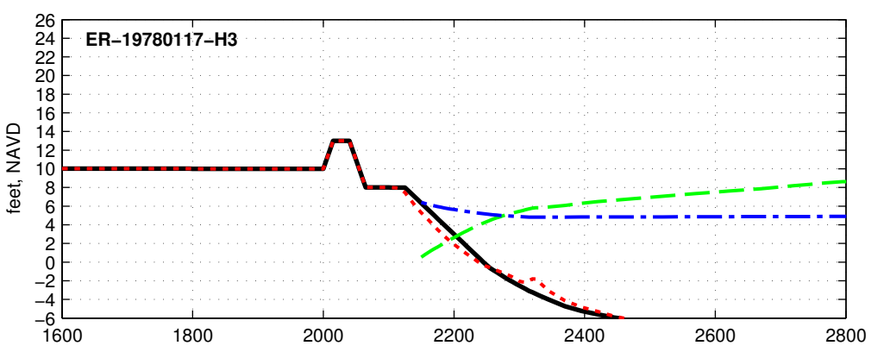


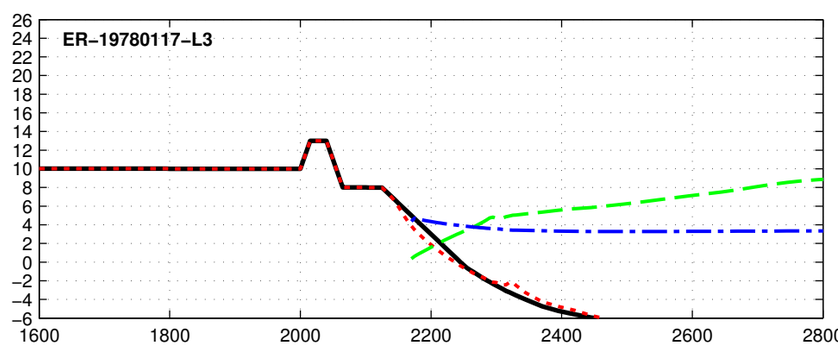


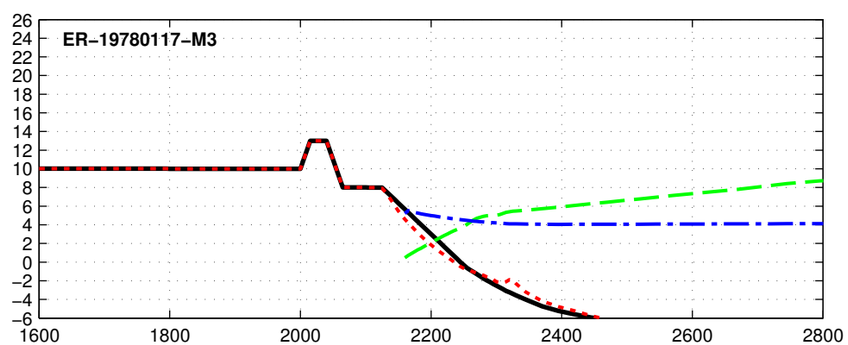


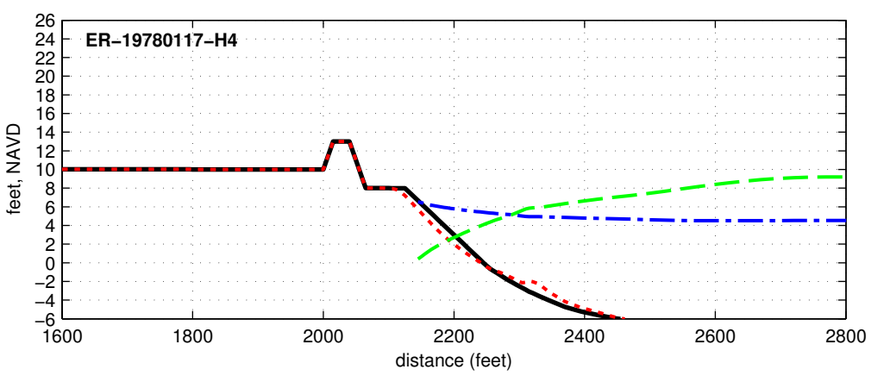


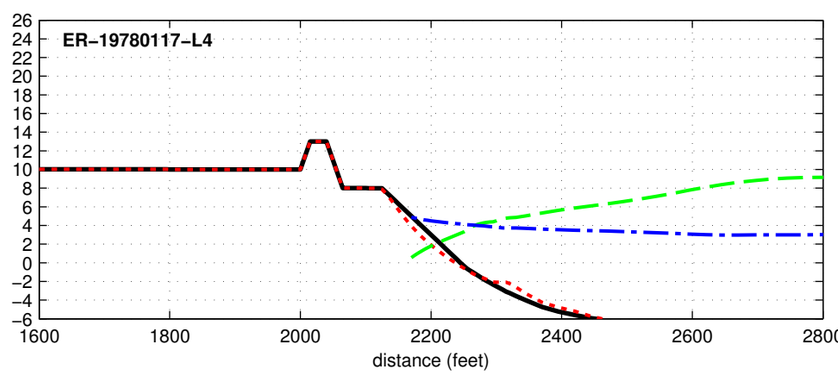


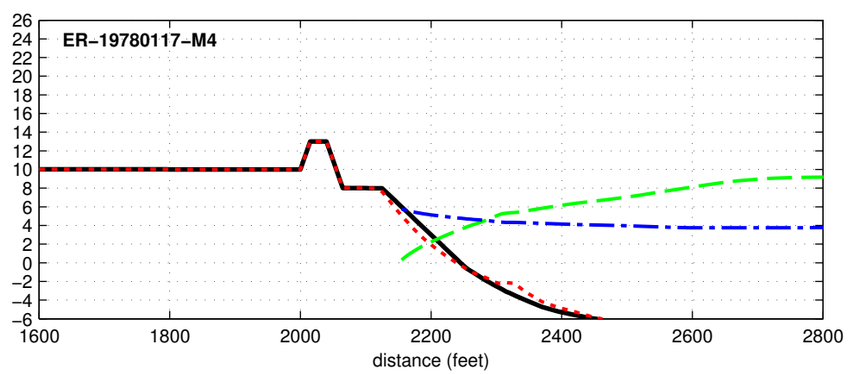






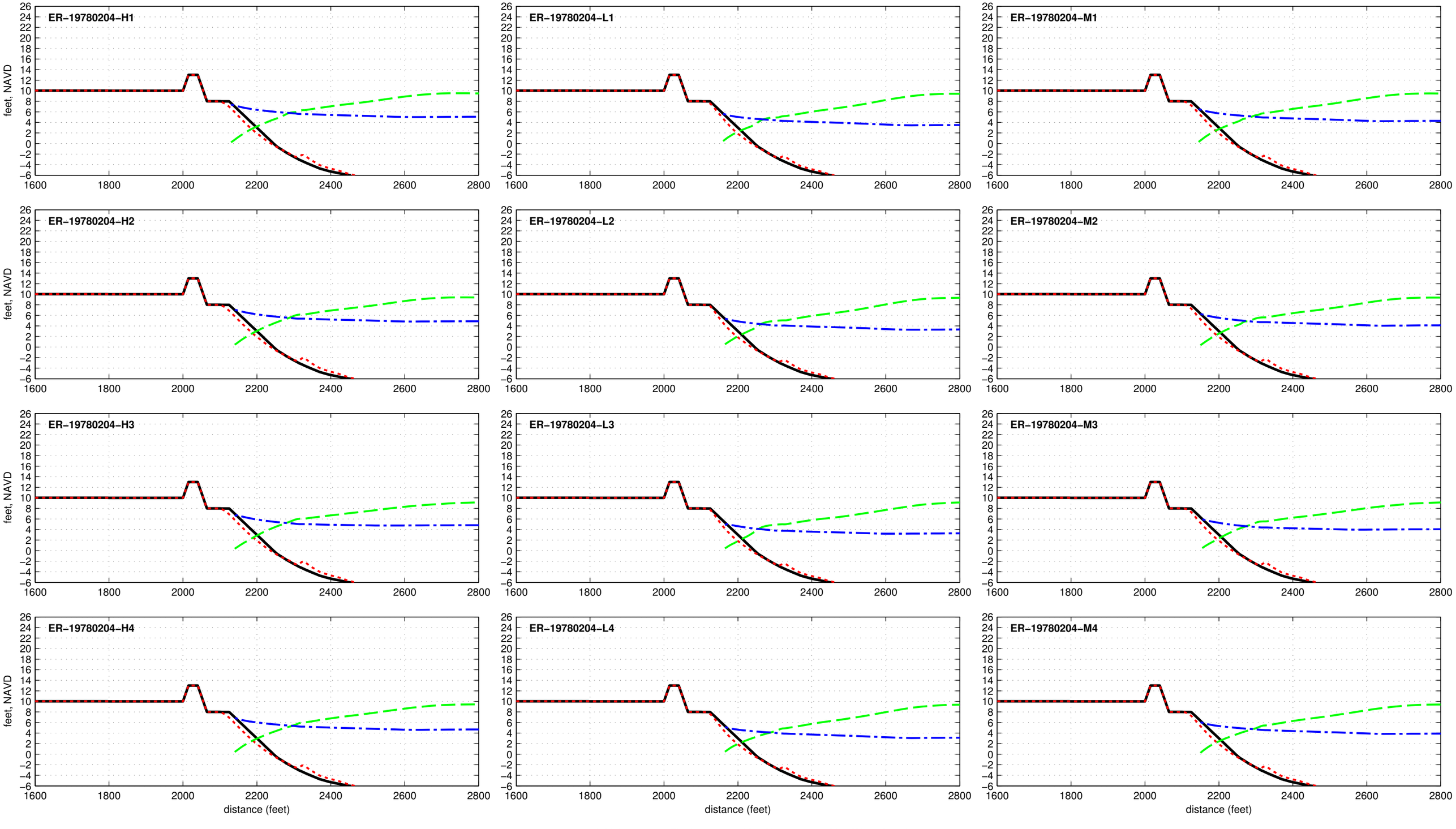




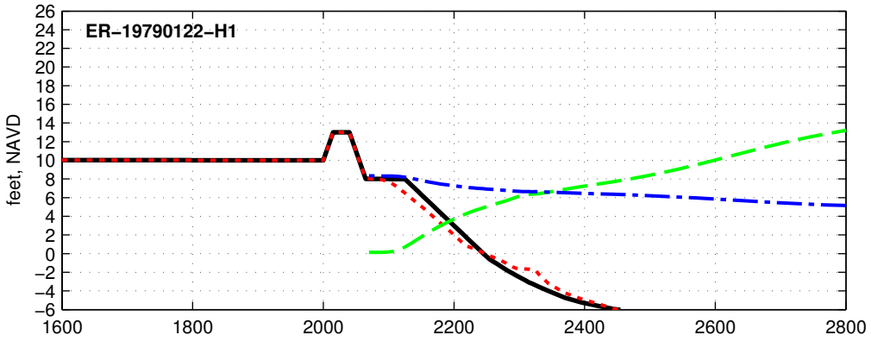


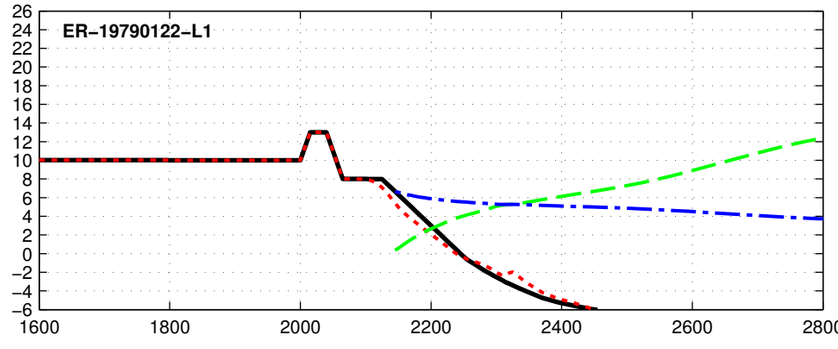
Initial Final Max Hs Max Water Level

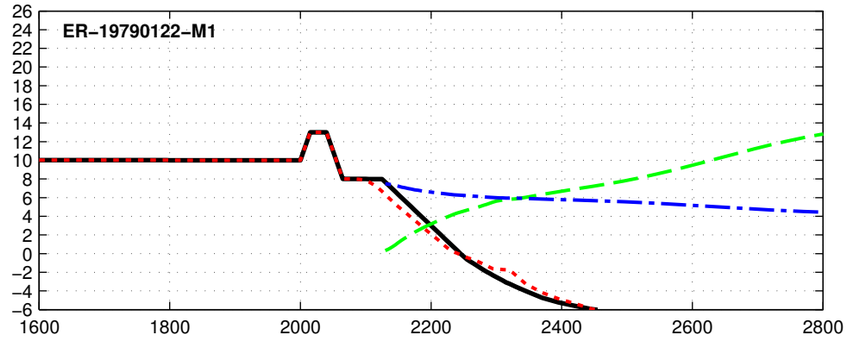
Dune R2T1-13.0-025-060 - ER-19780204

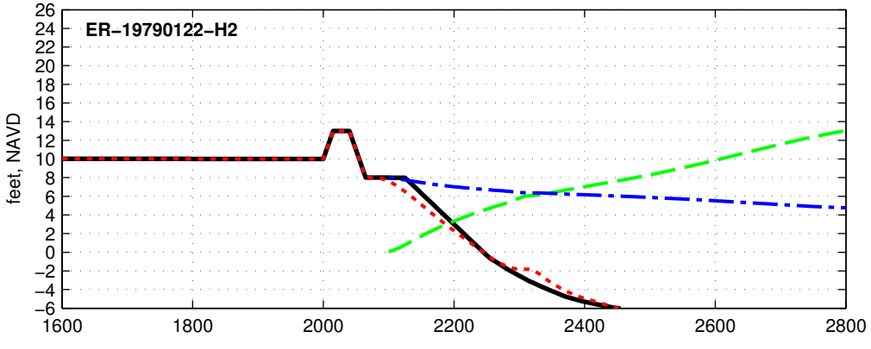


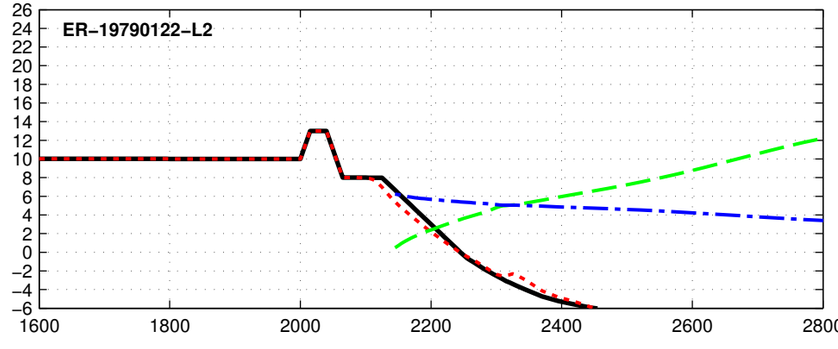
Dune R2T1-13.0-025-060 - ER-19790122

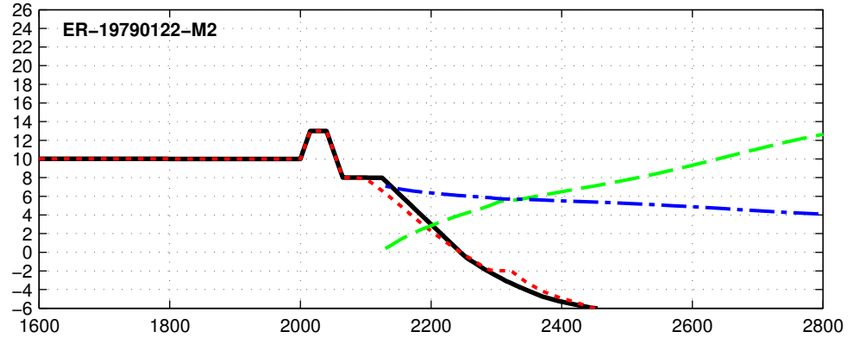


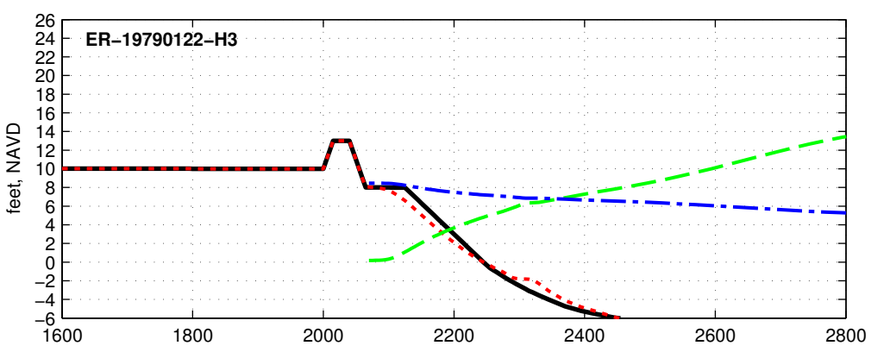


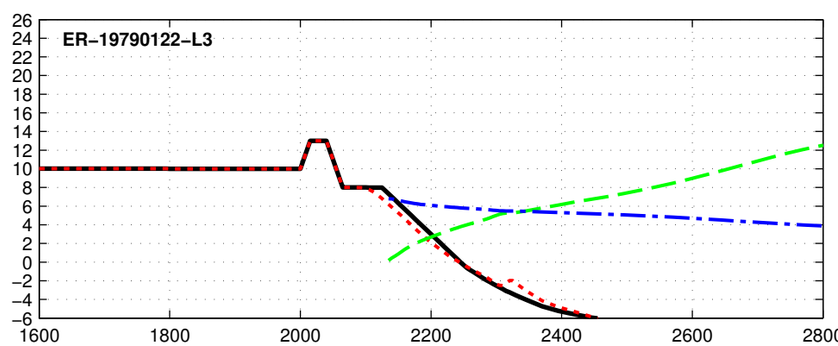


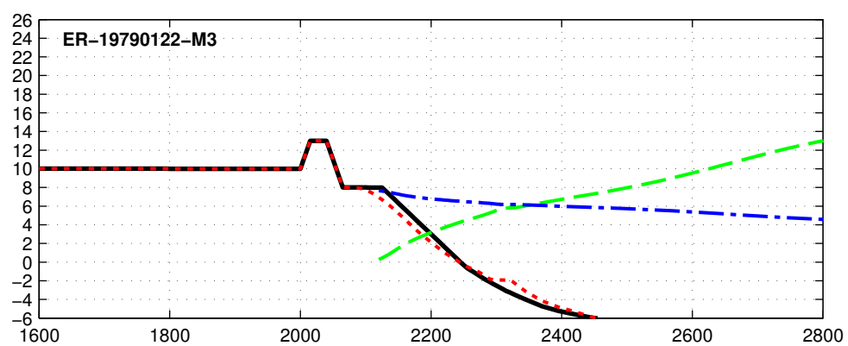


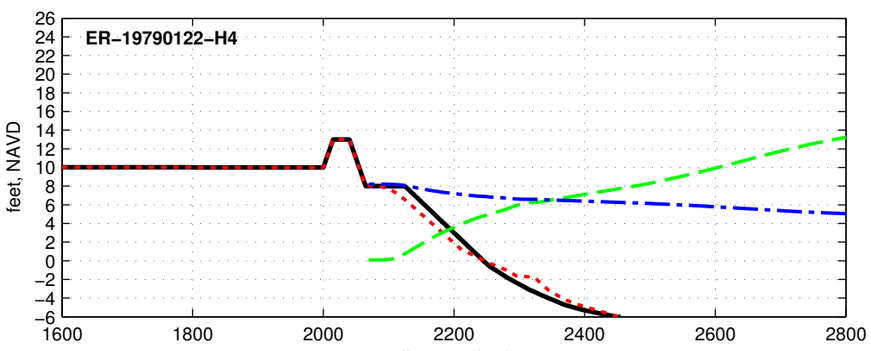


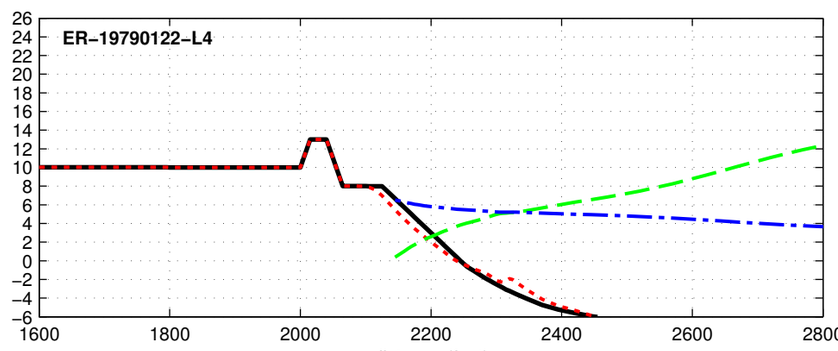


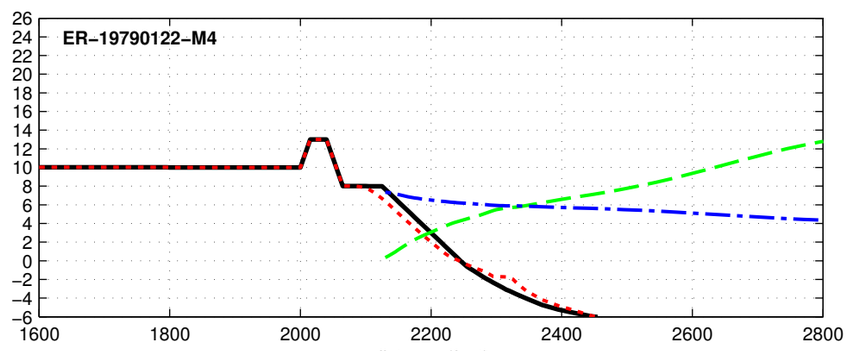






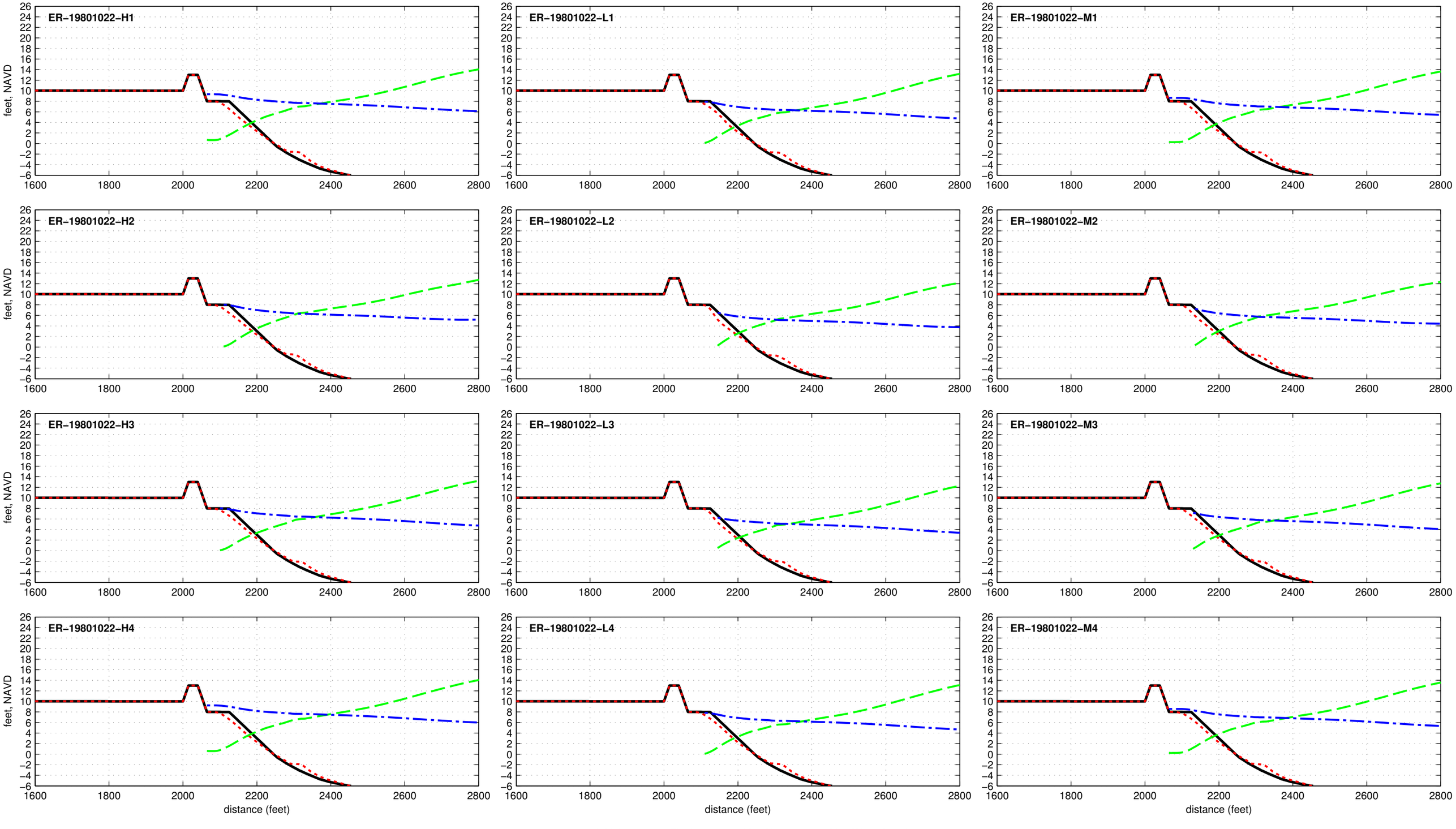




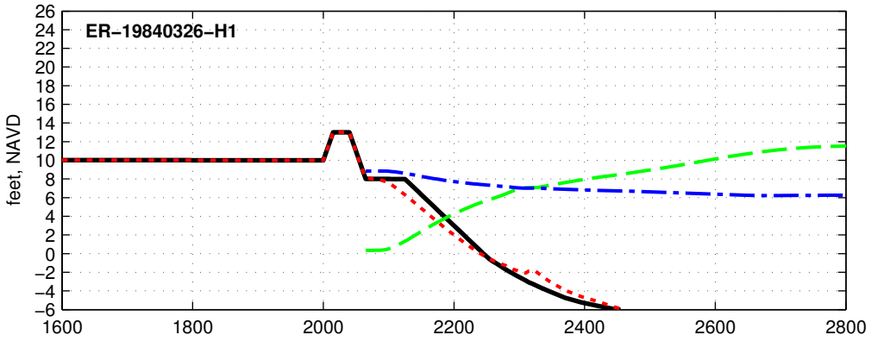


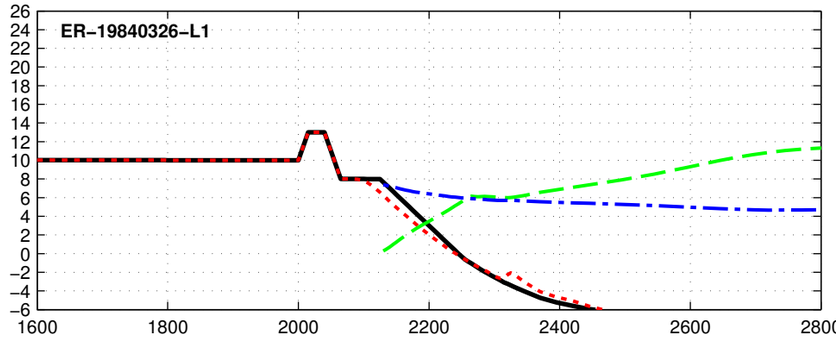
Initial Final Max Hs Max Water Level

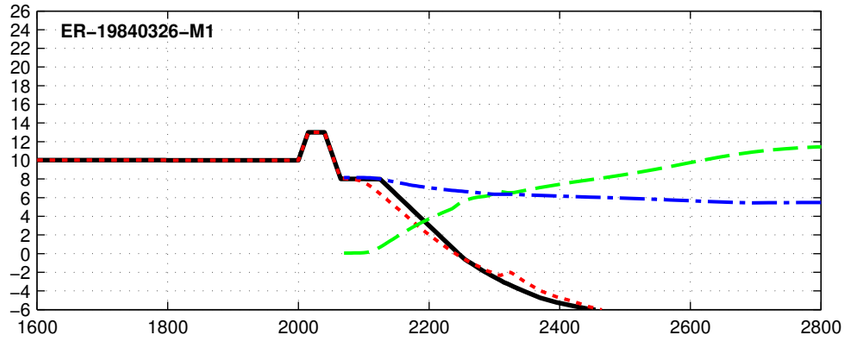
Dune R2T1-13.0-025-060 - ER-19801022

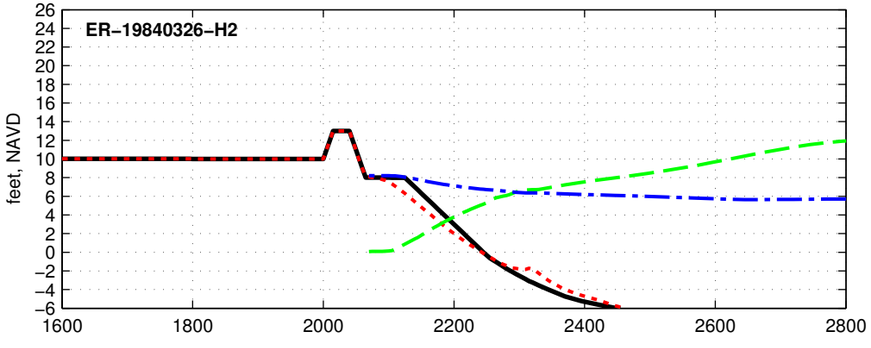


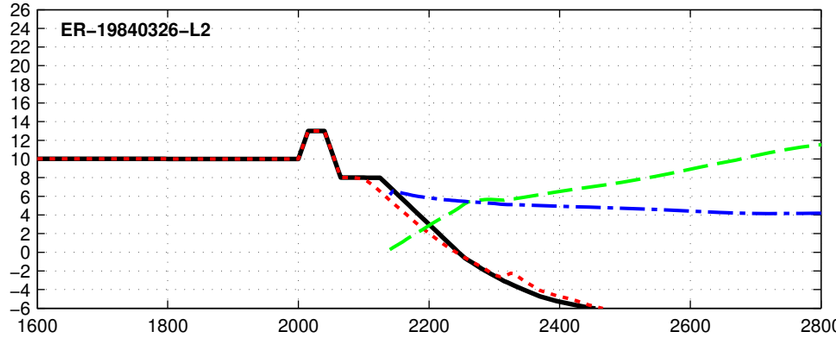
Dune R2T1-13.0-025-060 - ER-19840326

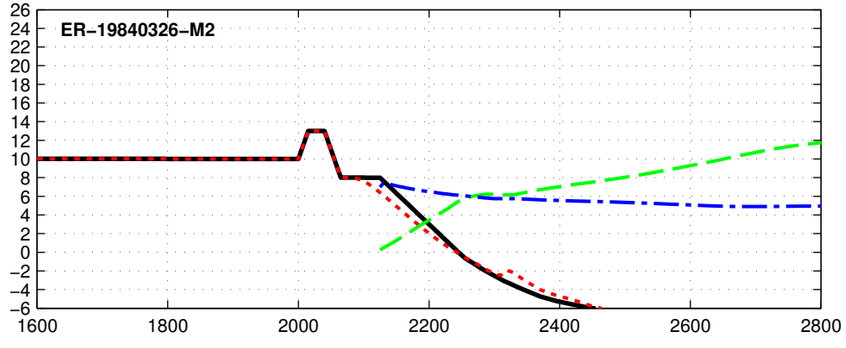


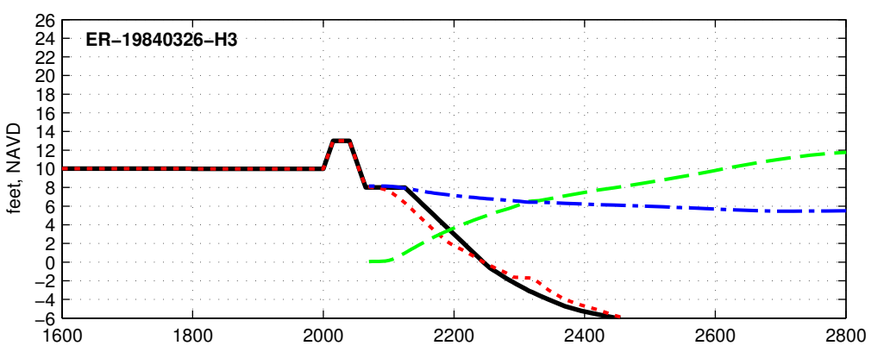


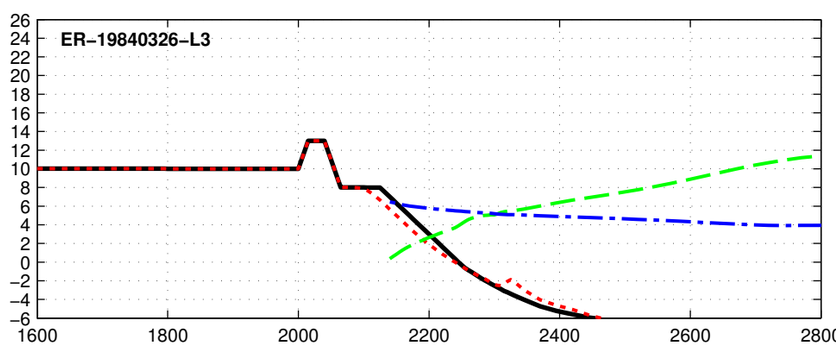


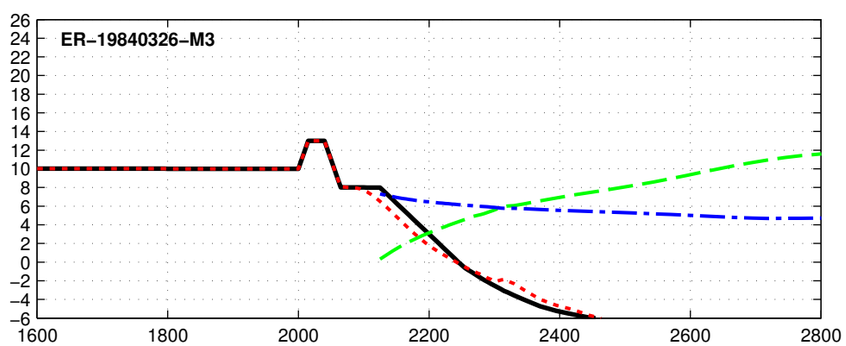


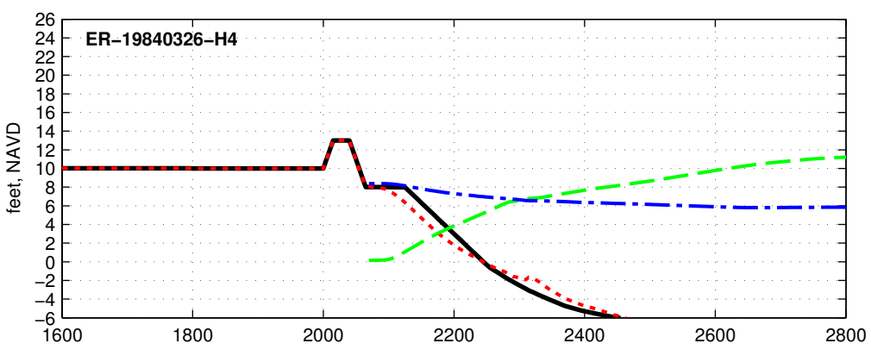


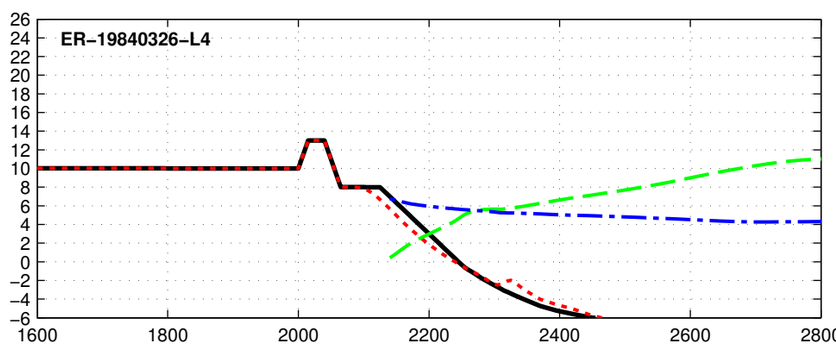


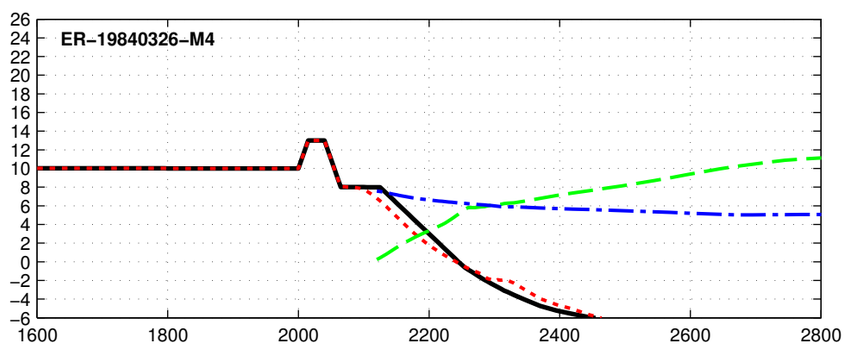










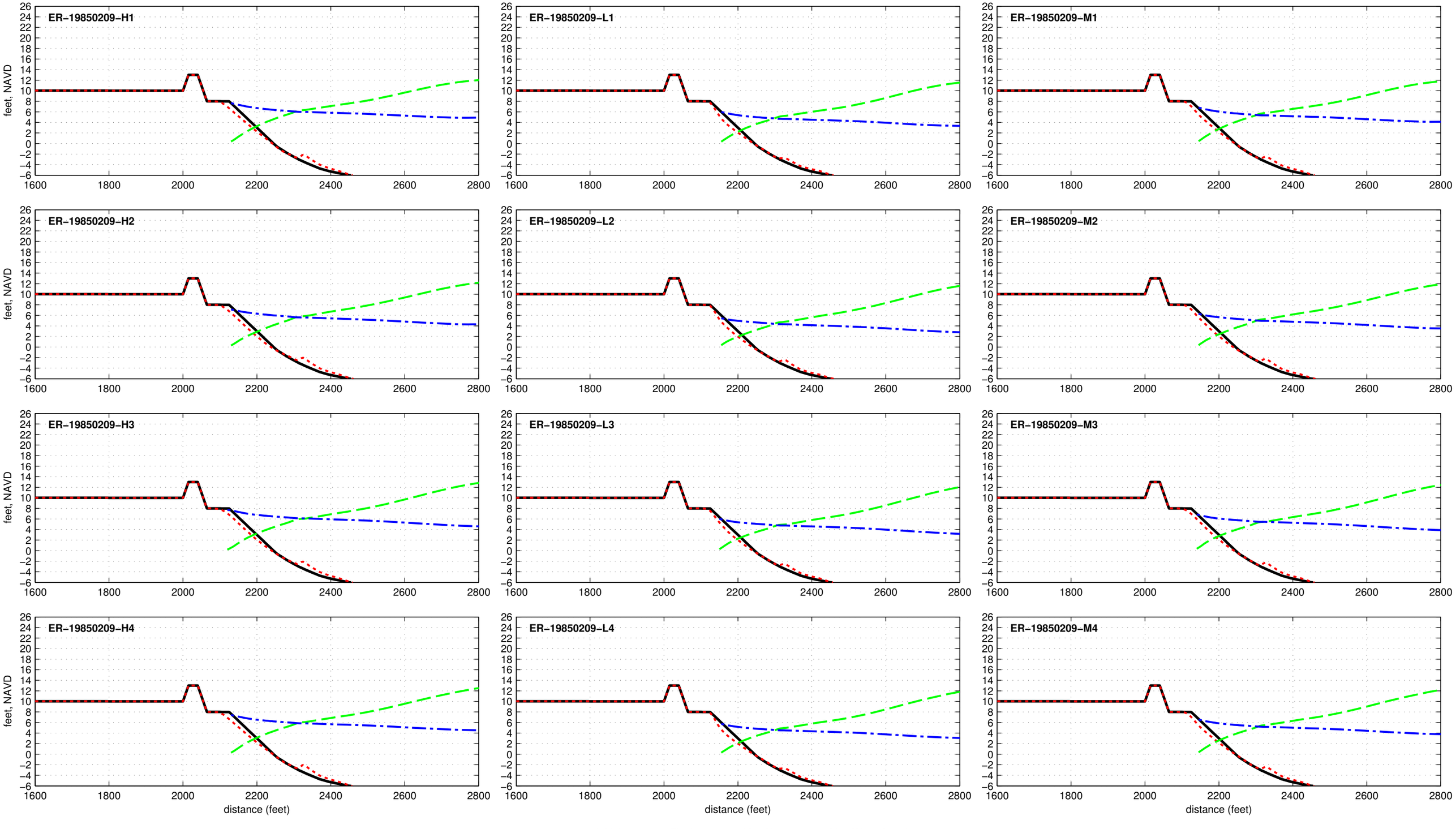


feet, NAVD

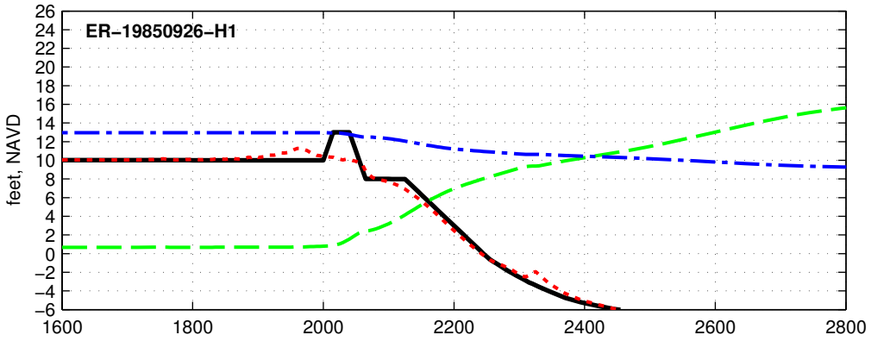
distance (feet)

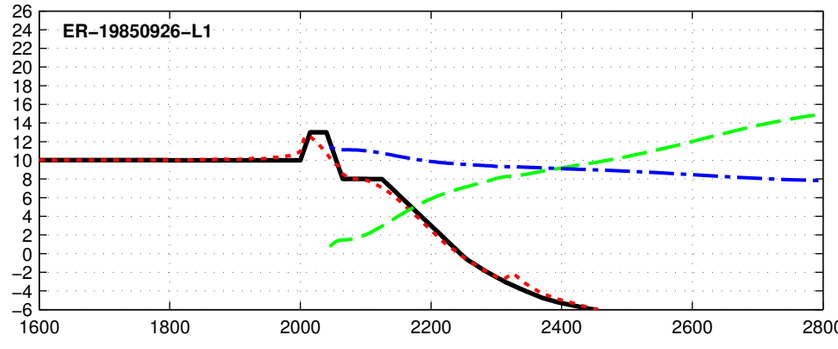
Initial Final Max Hs Max Water Level

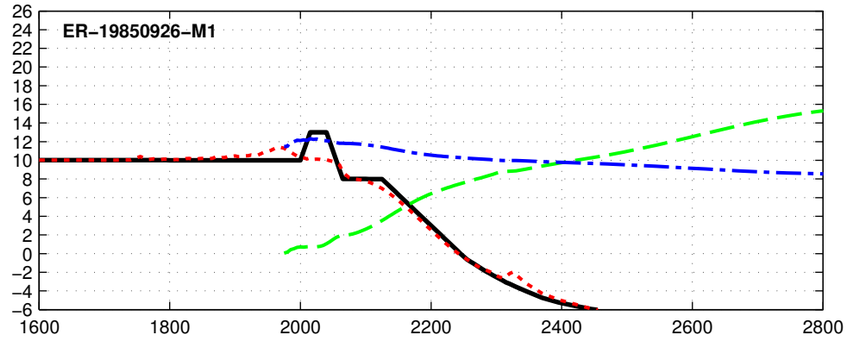
Dune R2T1-13.0-025-060 - ER-19850209

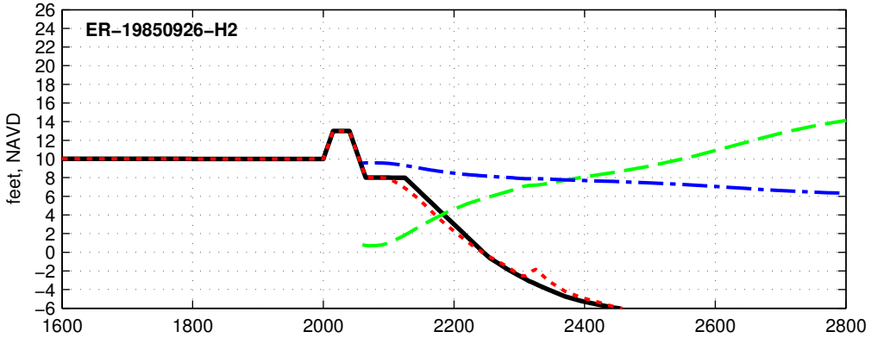


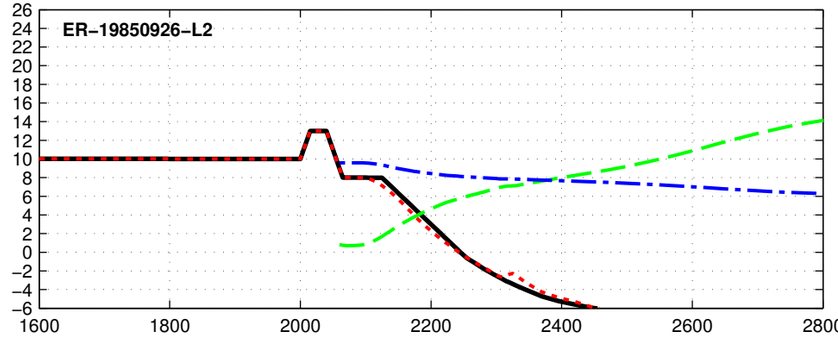
Dune R2T1-13.0-025-060 - ER-19850926

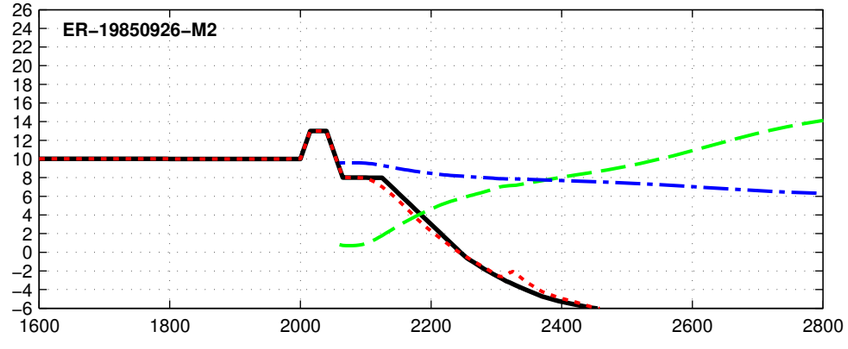


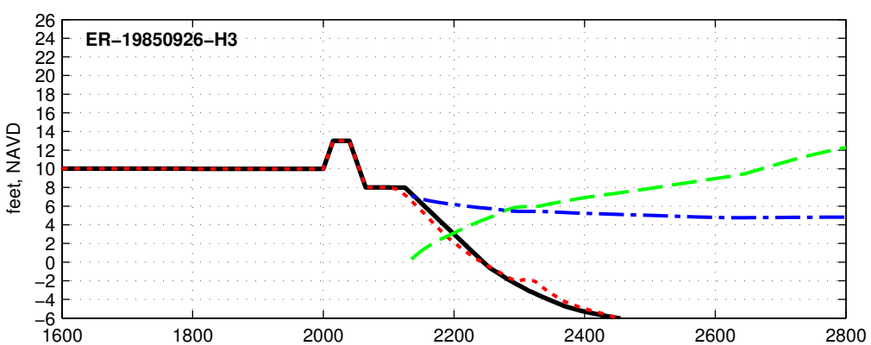


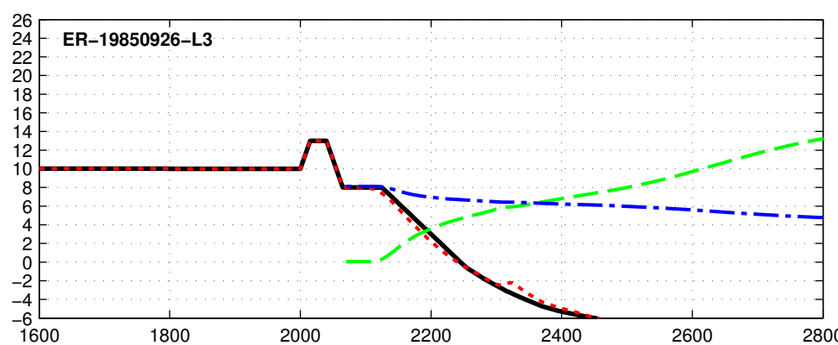


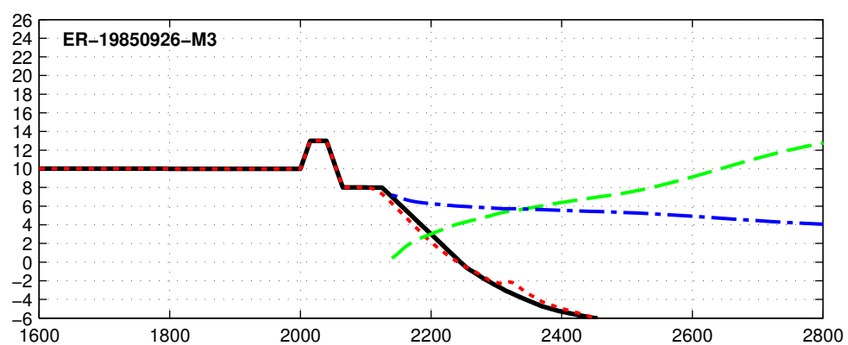


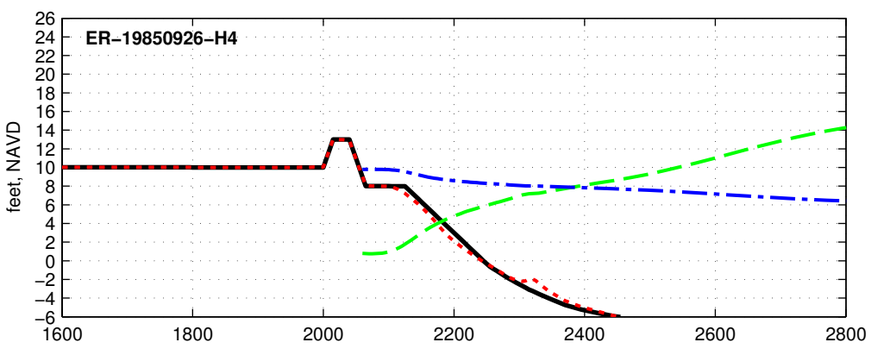


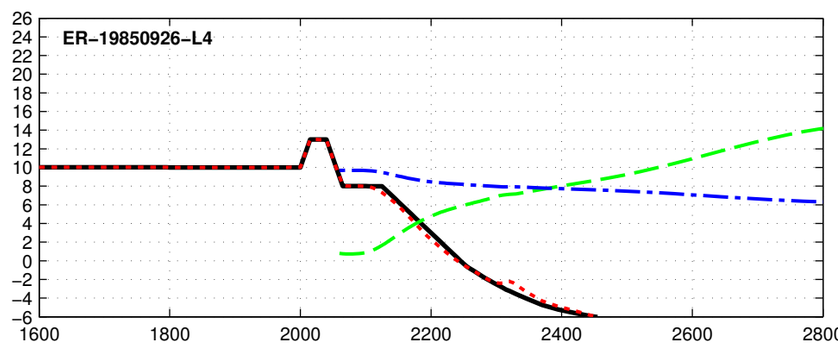


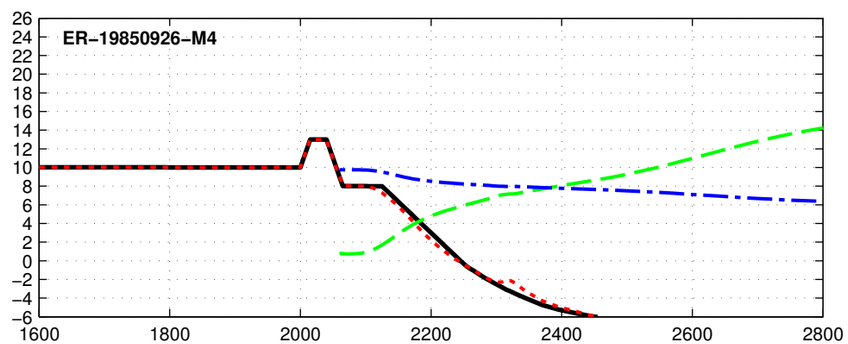






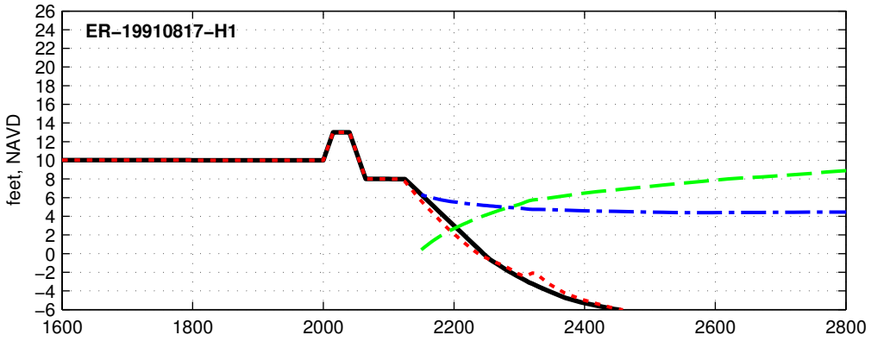


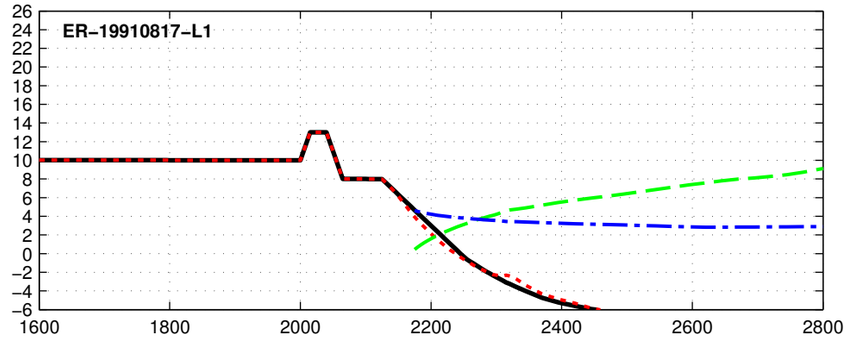


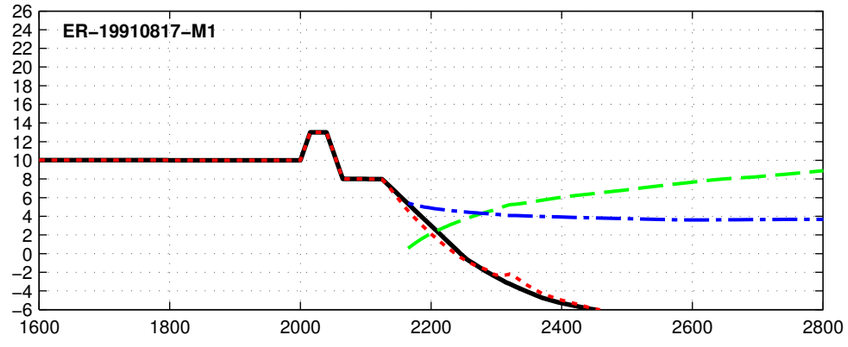


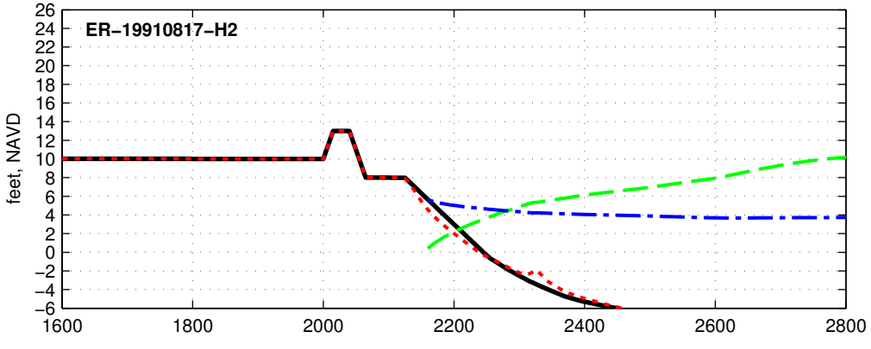
Initial Final Max Hs Max Water Level

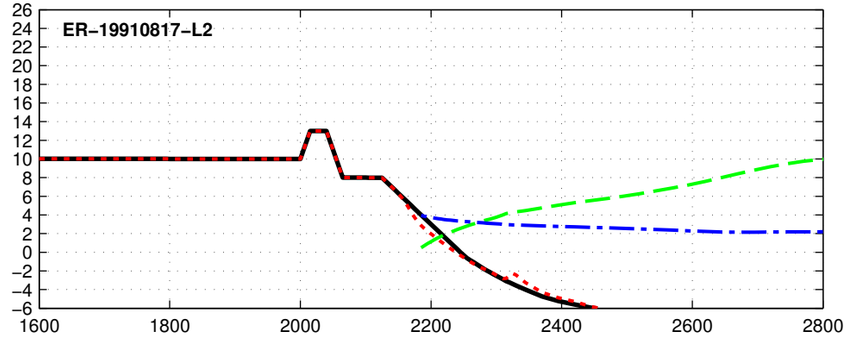
Dune R2T1–13.0–025–060 – ER–19910817

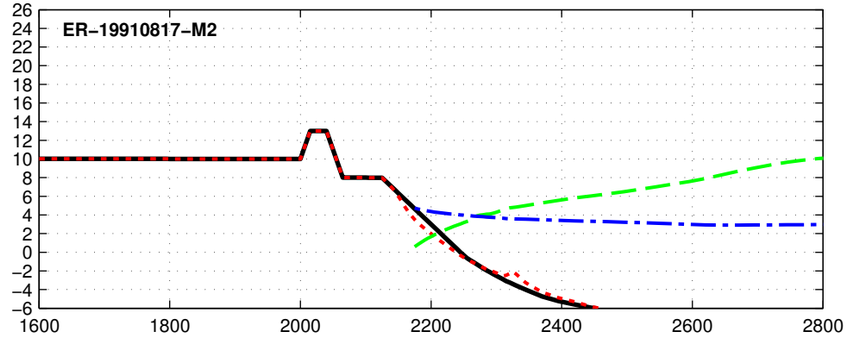


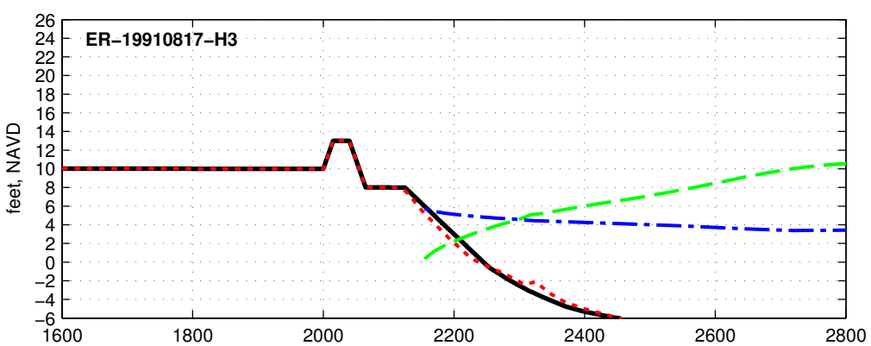


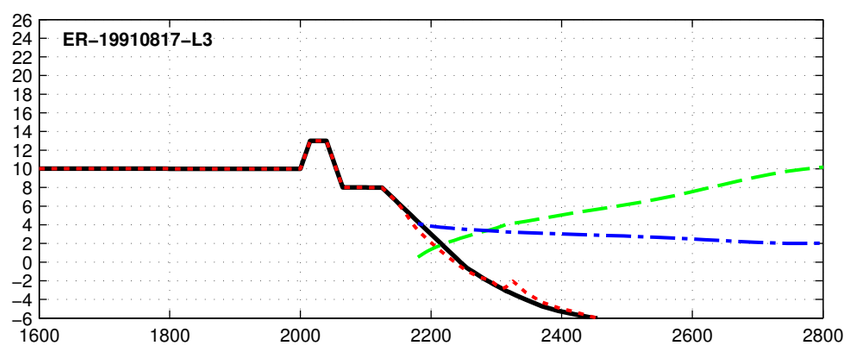


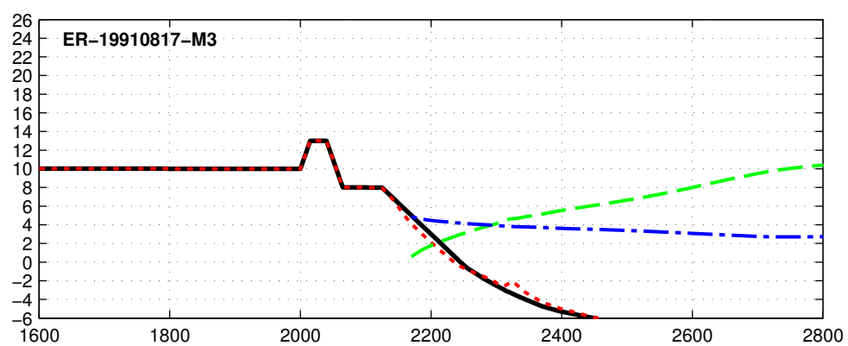


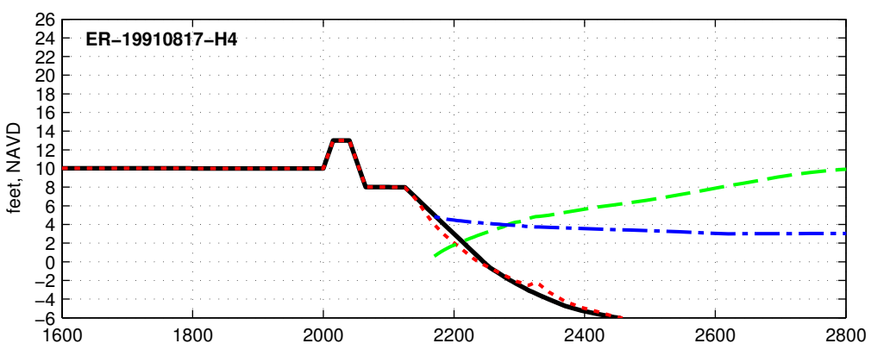


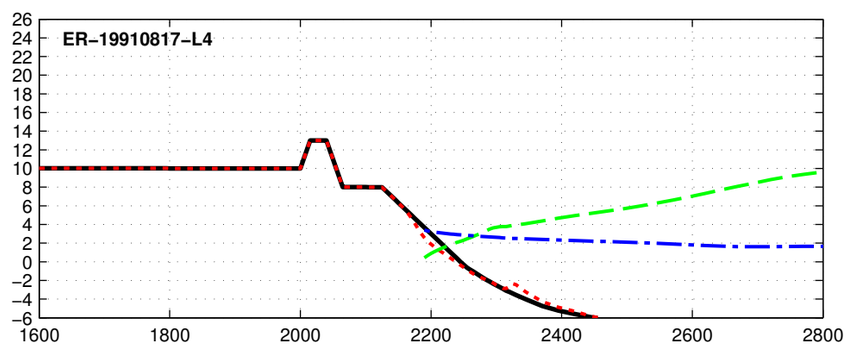


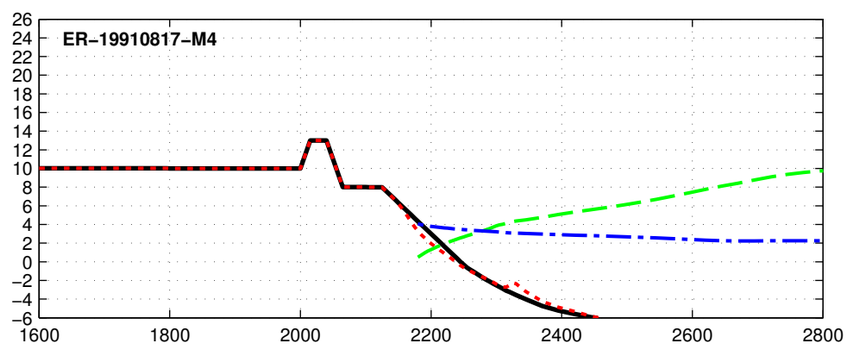






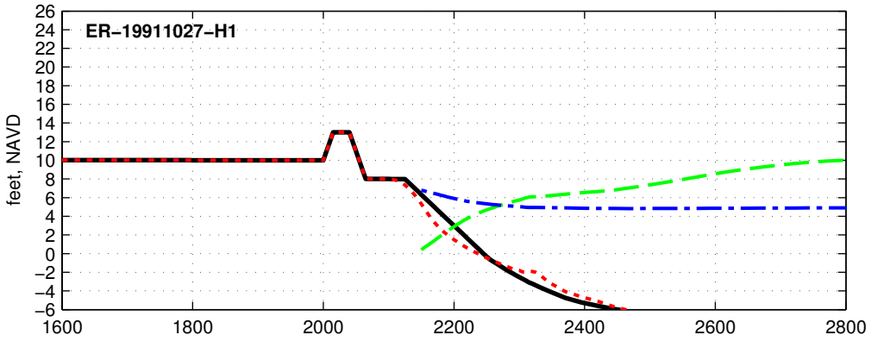


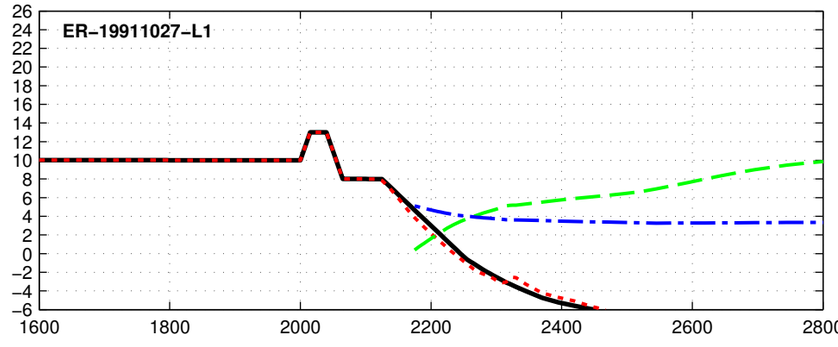


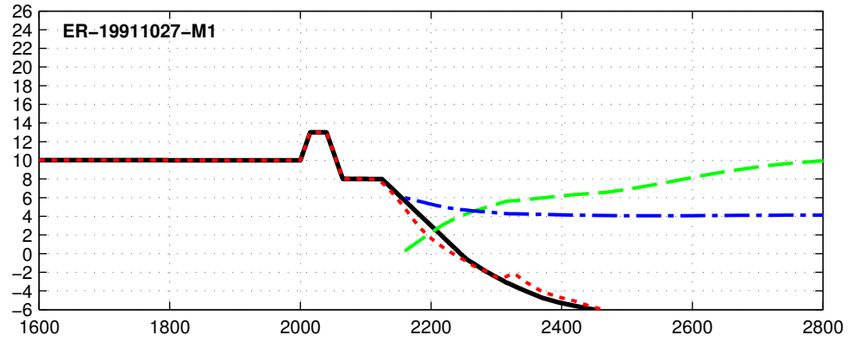


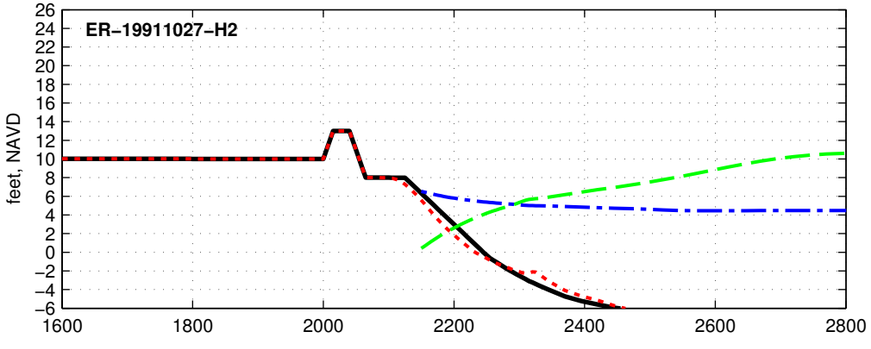
Initial Final Max Hs Max Water Level

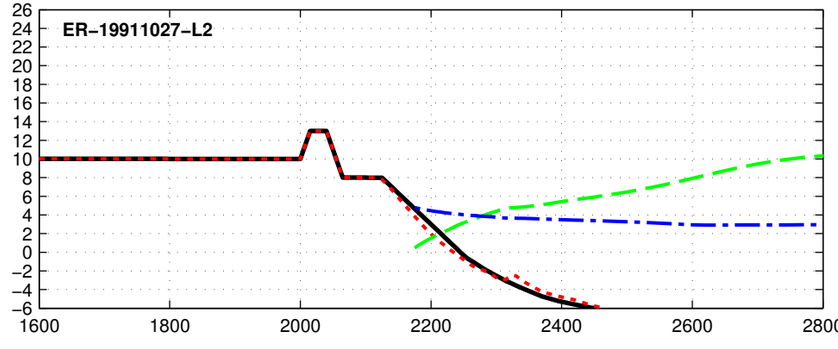
Dune R2T1–13.0–025–060 – ER–19911027

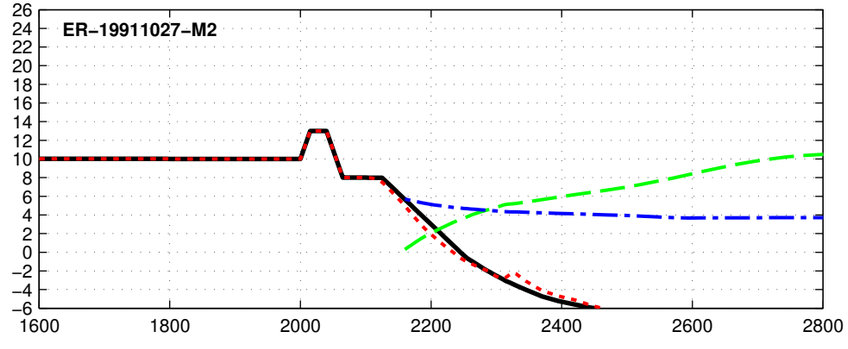


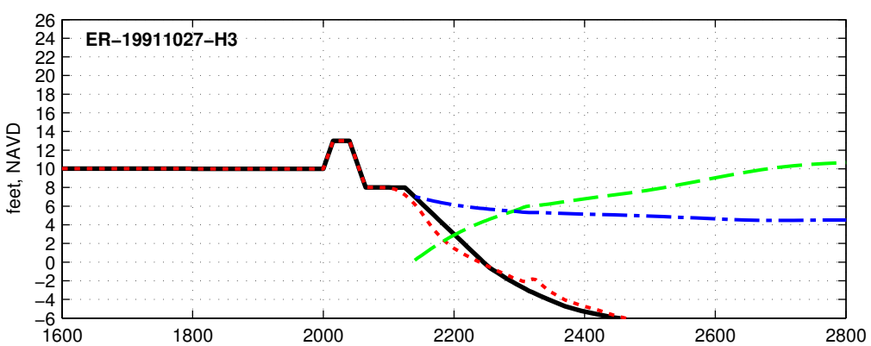


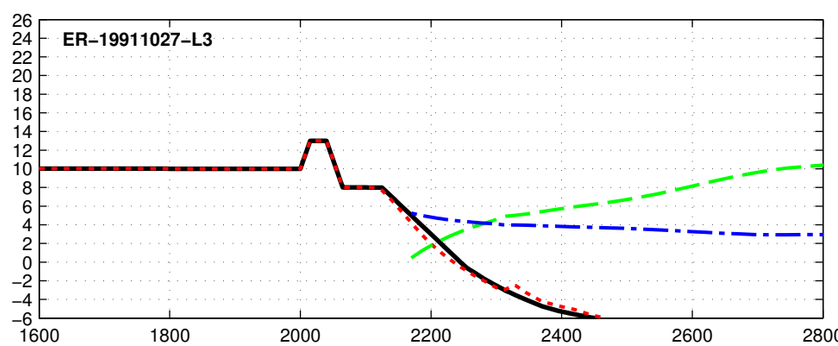


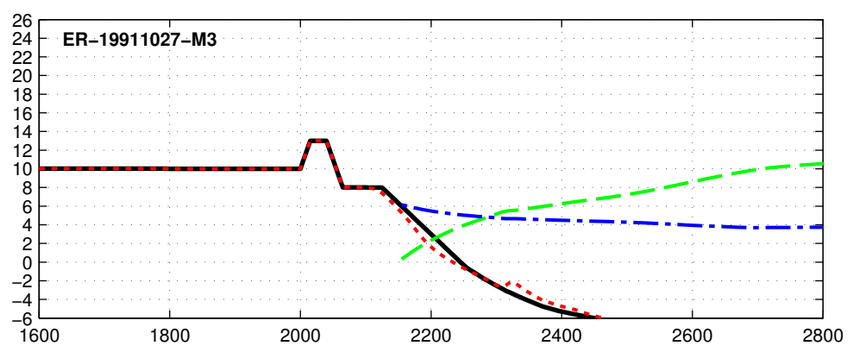


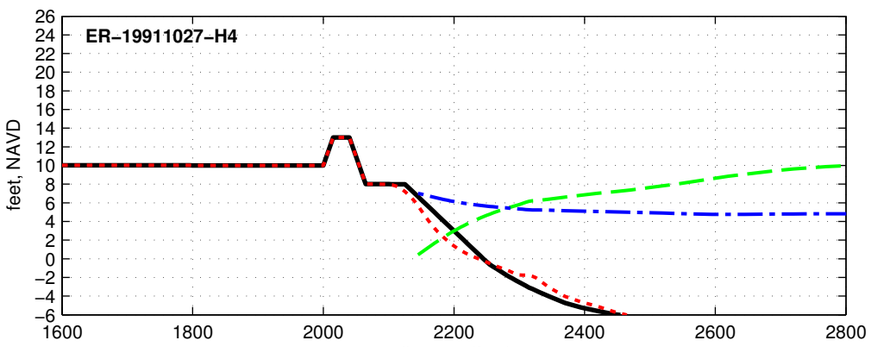


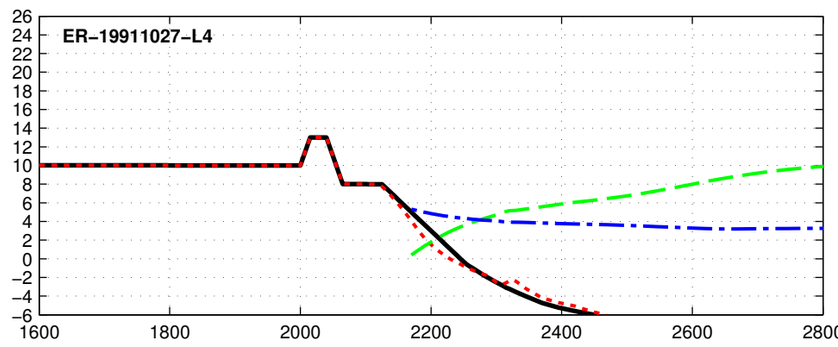


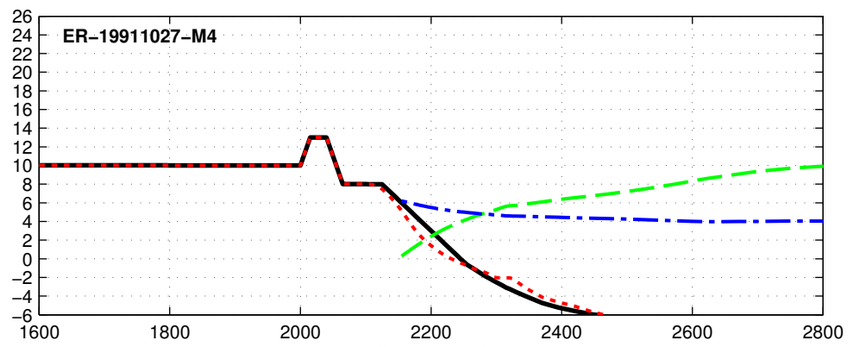






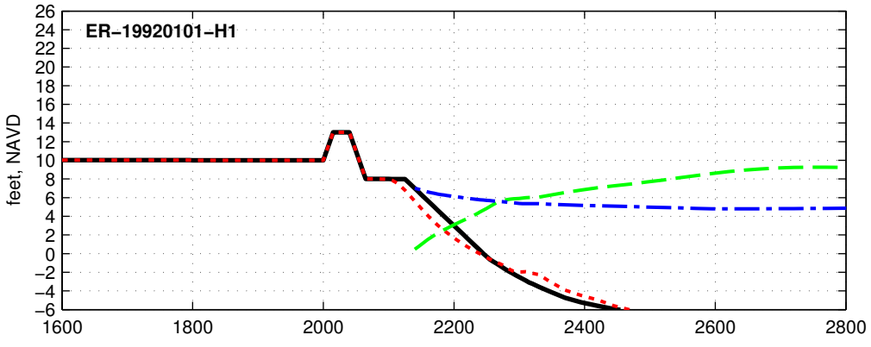


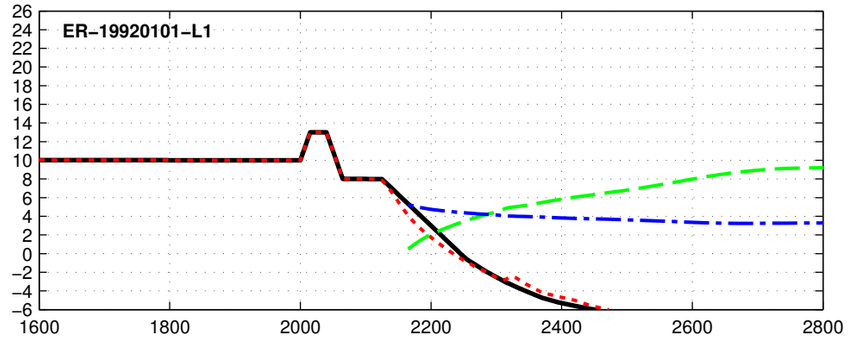


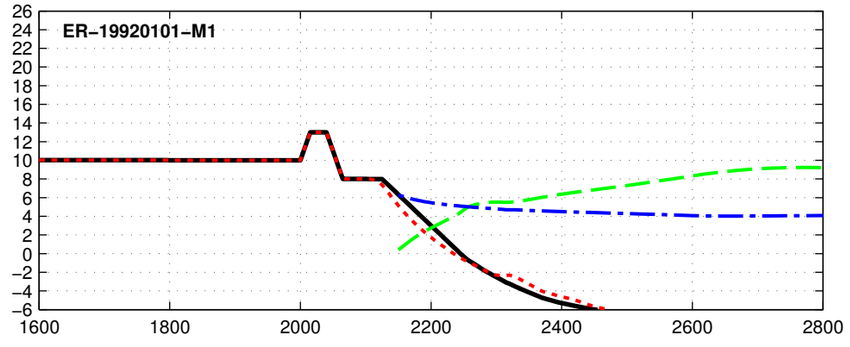


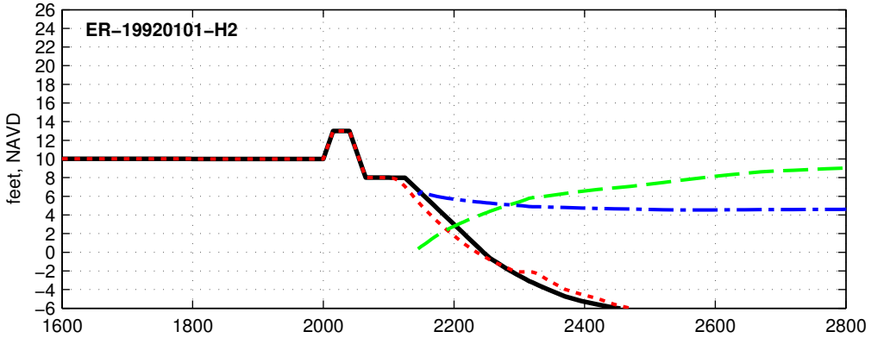
Initial Final Max Hs Max Water Level

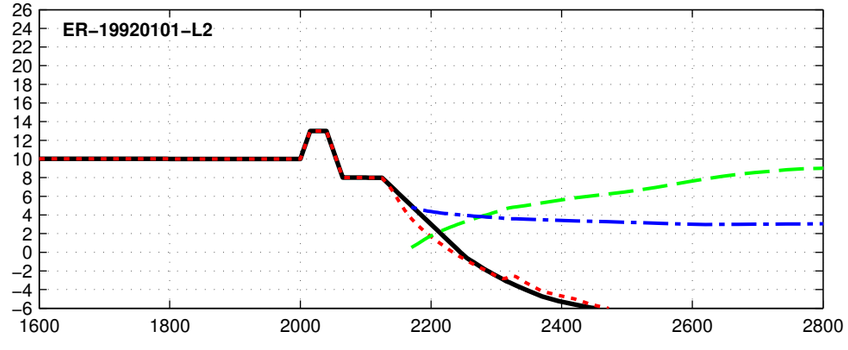
Dune R2T1-13.0-025-060 - ER-19920101

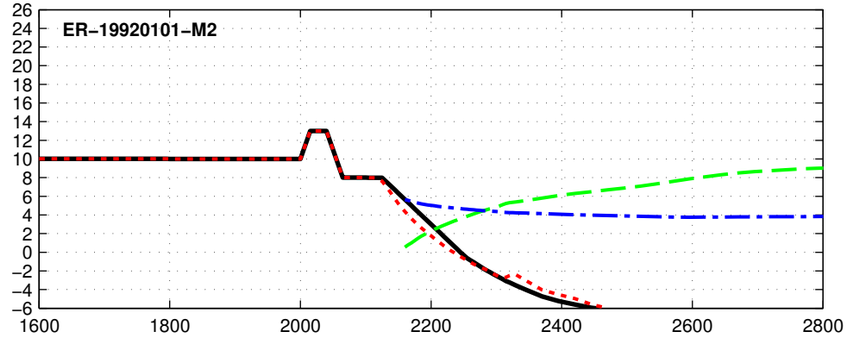


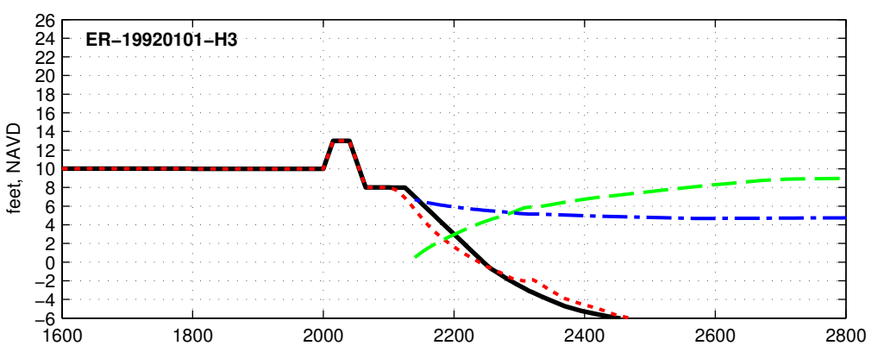


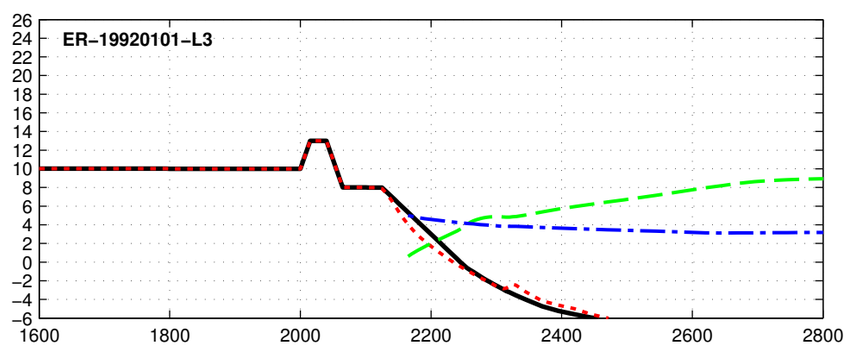


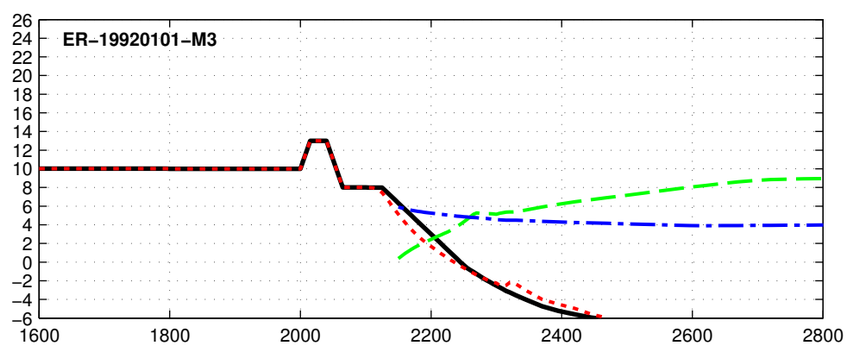


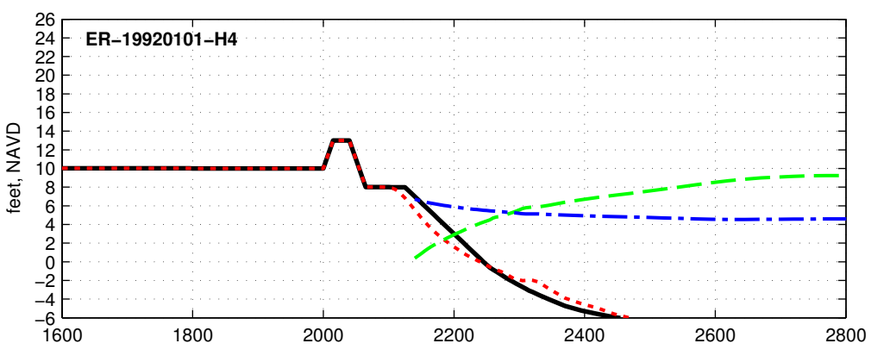


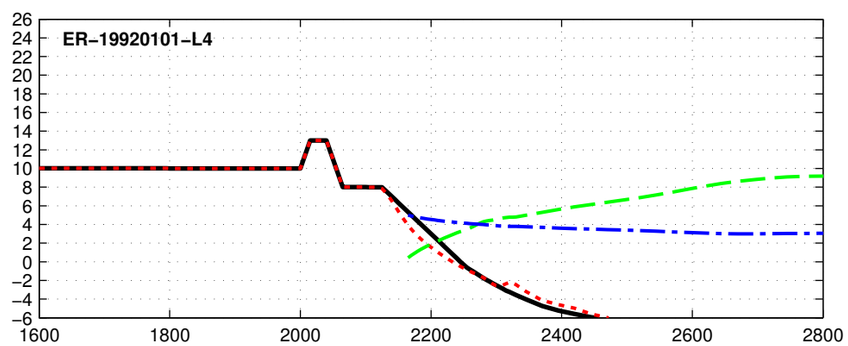


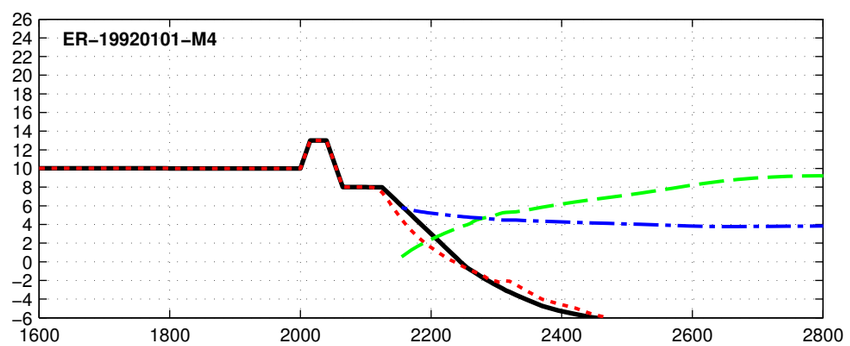






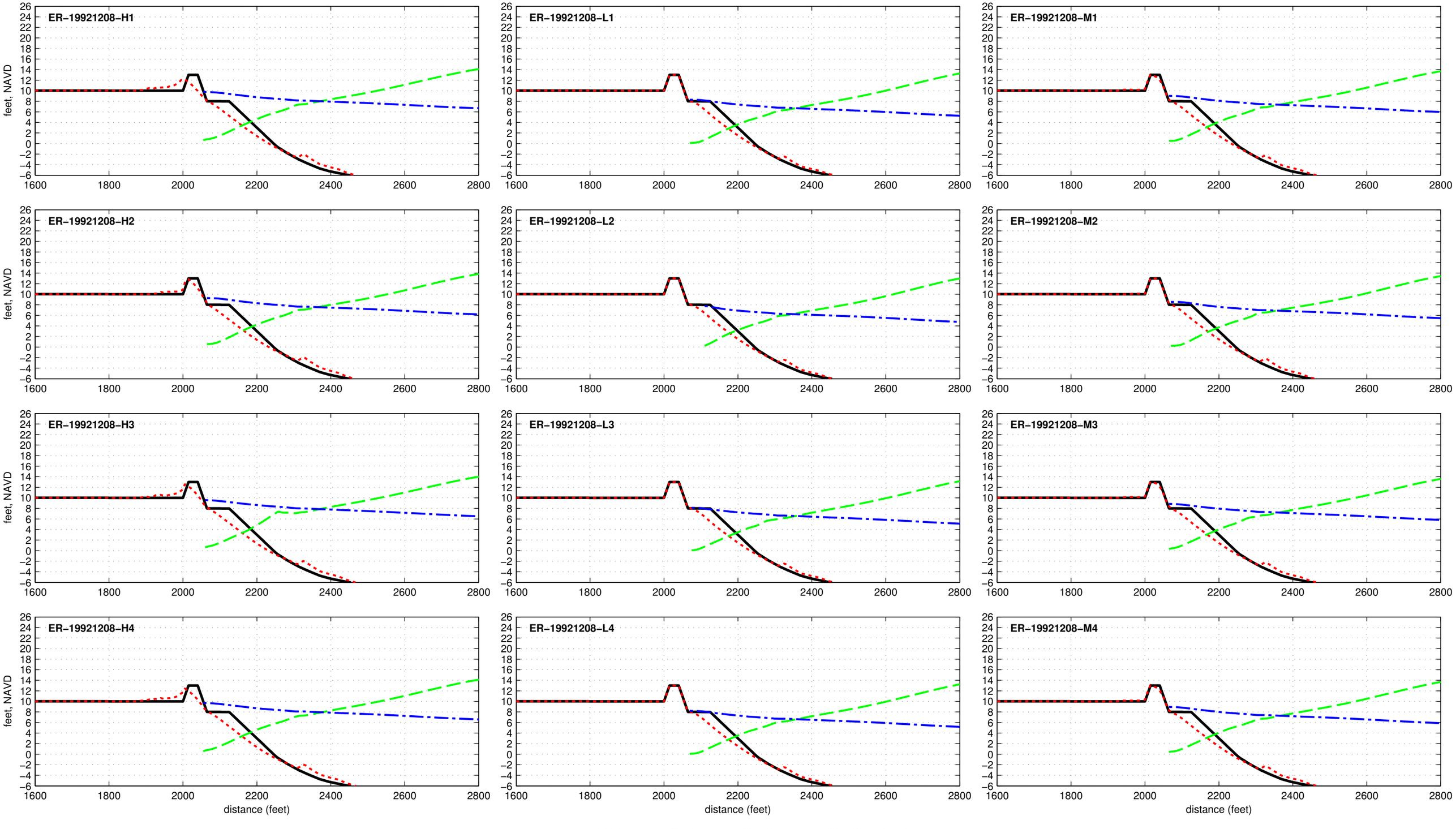




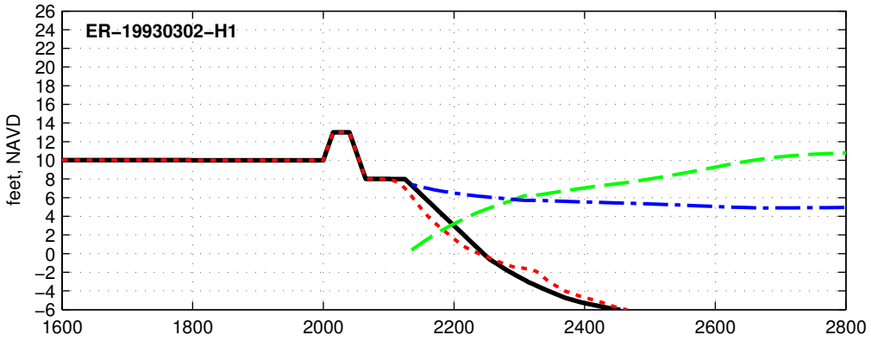


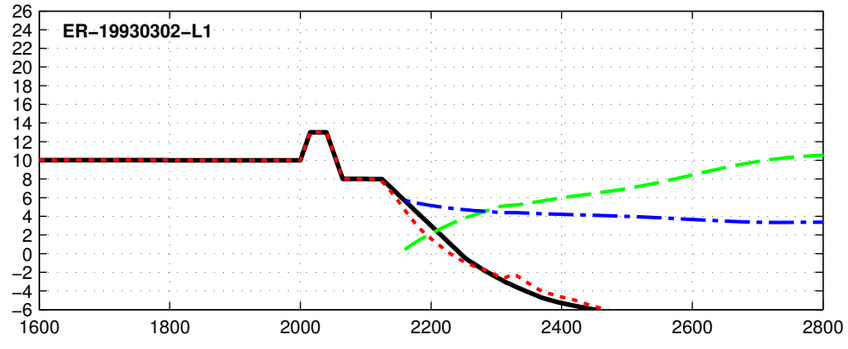
Initial Final Max Hs Max Water Level

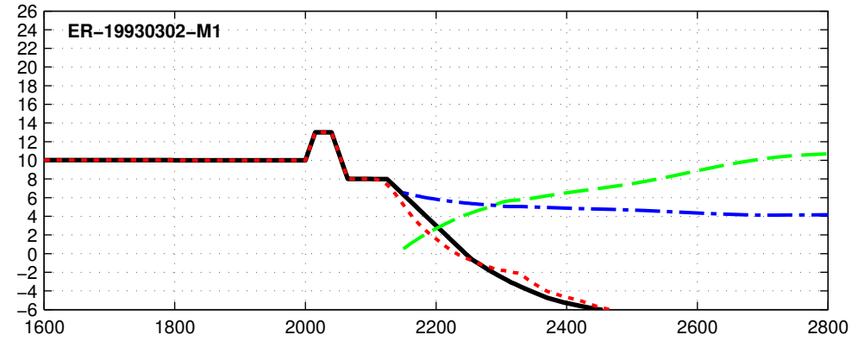
Dune R2T1-13.0-025-060 - ER-19921208

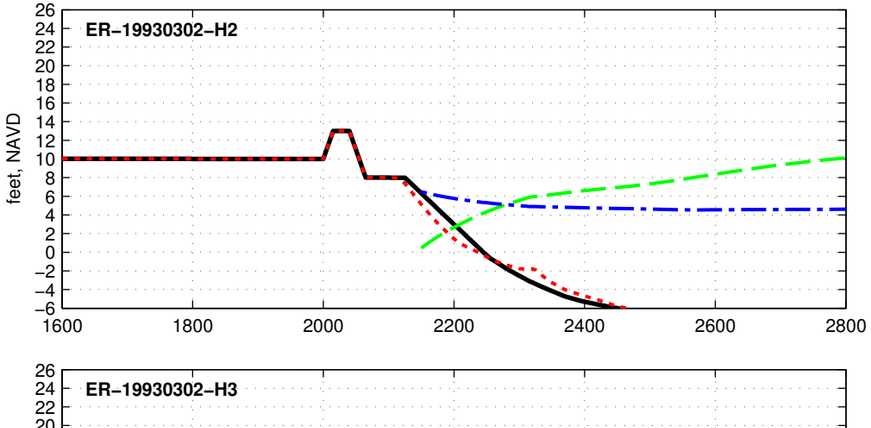


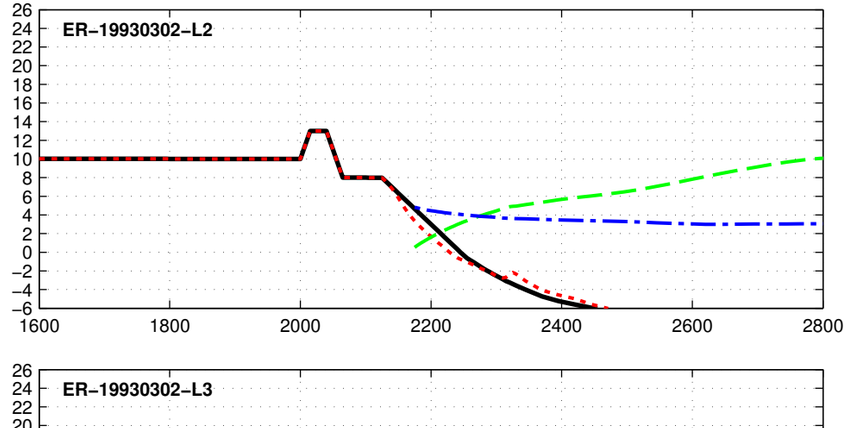
Dune R2T1-13.0-025-060 - ER-19930302

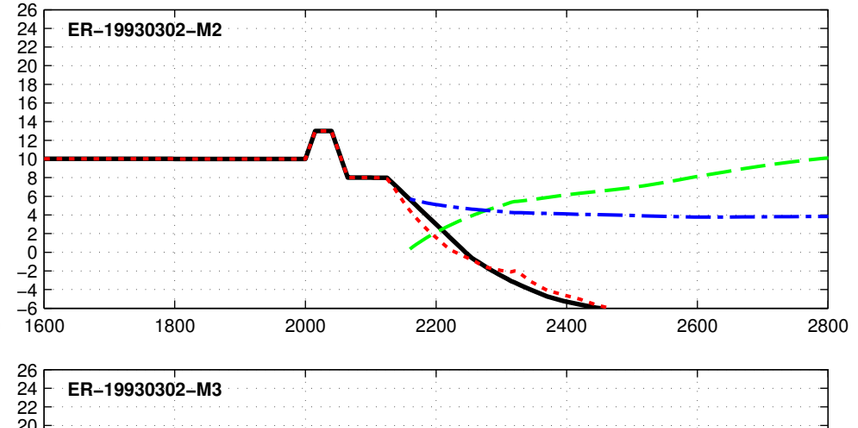


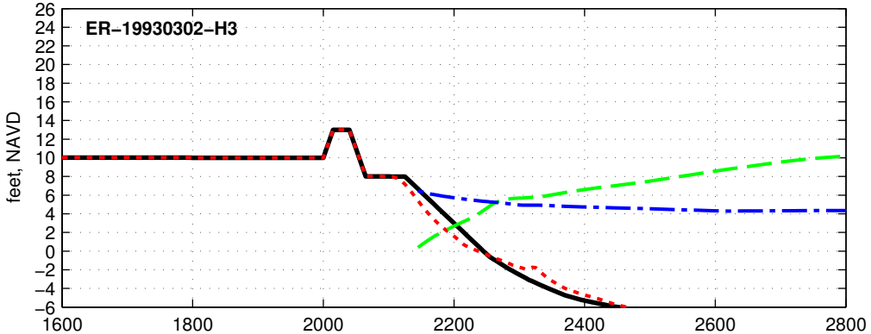


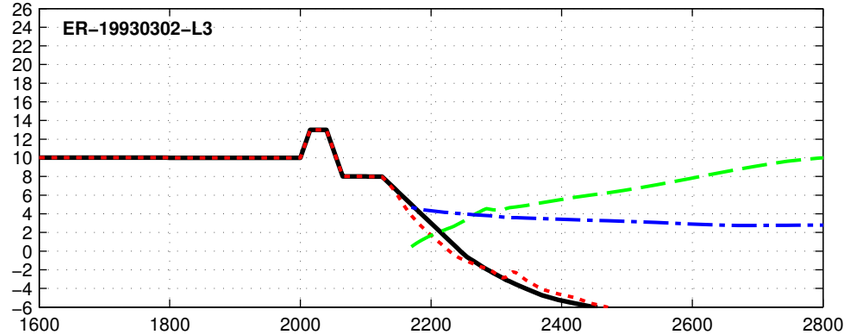


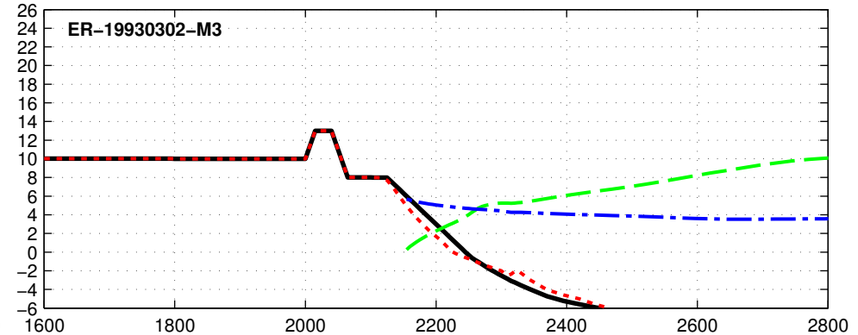


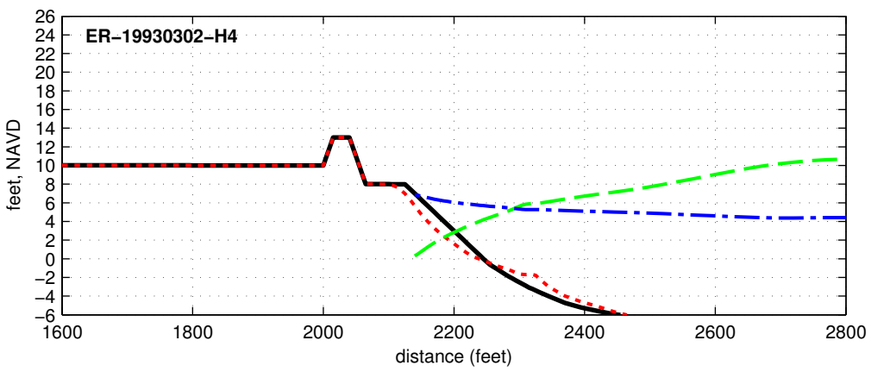


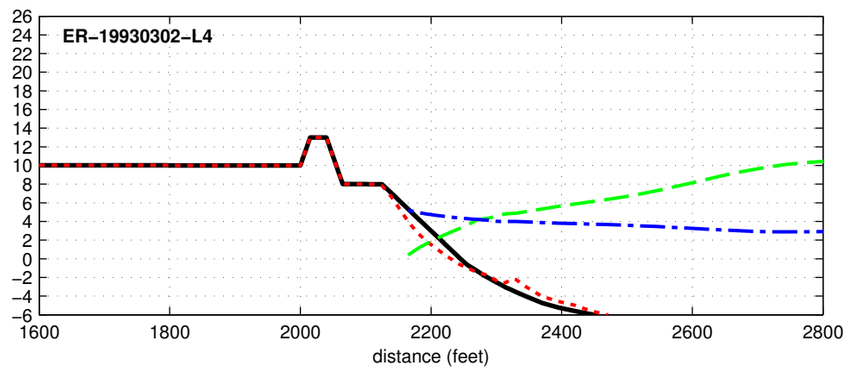


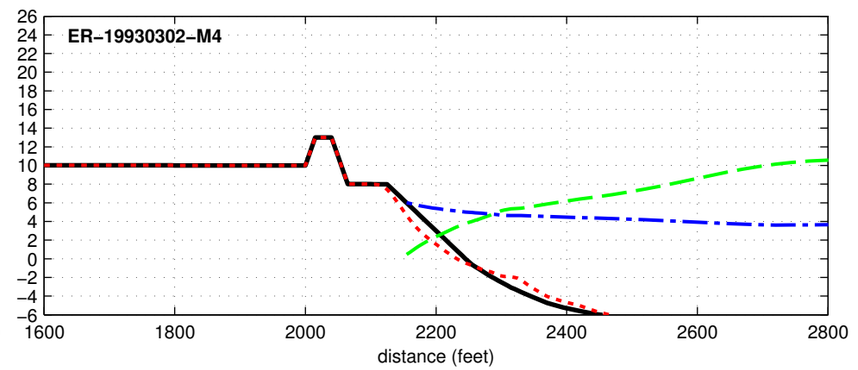






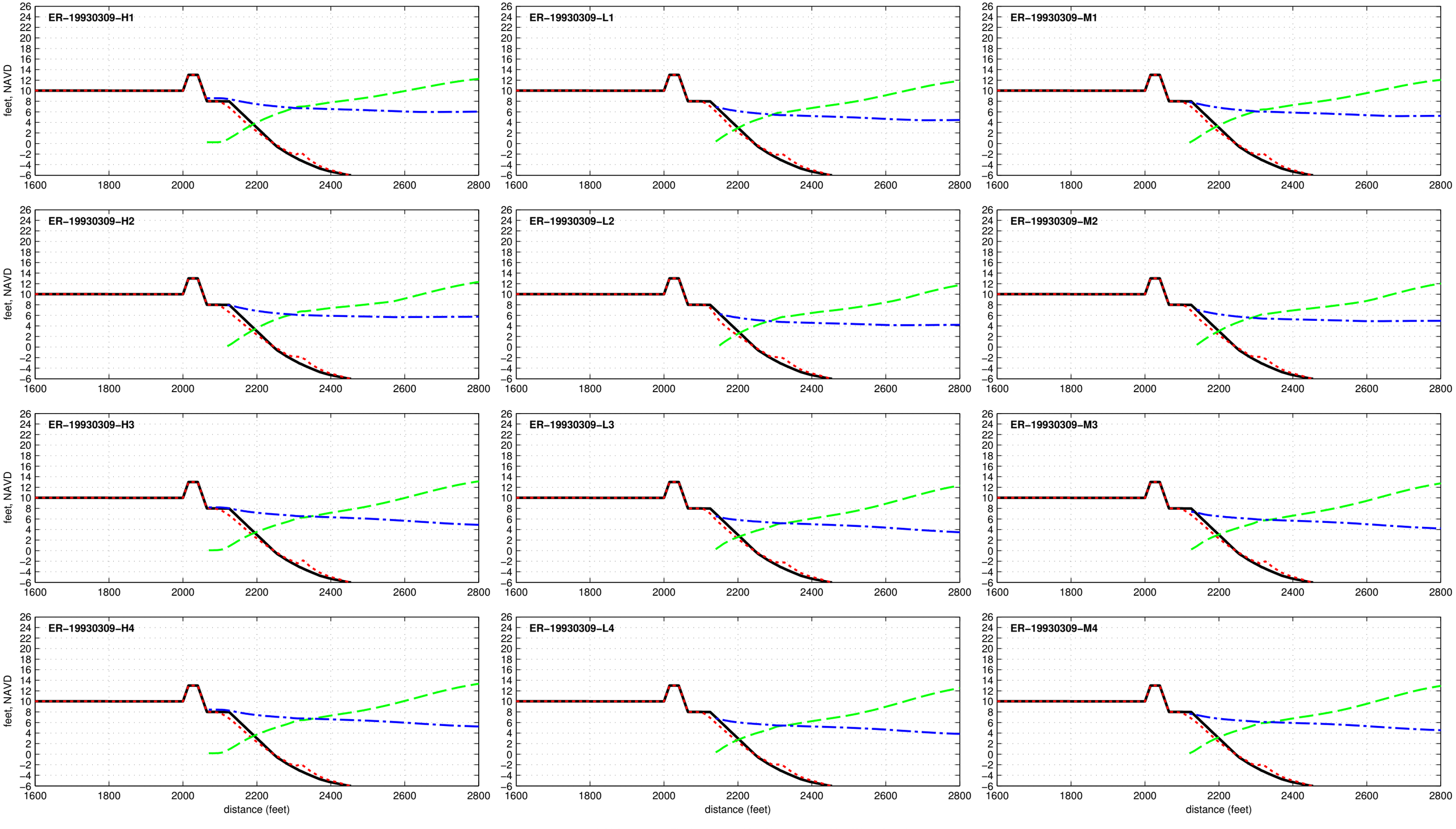






Initial Final Max Hs Max Water Level

Dune R2T1-13.0-025-060 - ER-19930309



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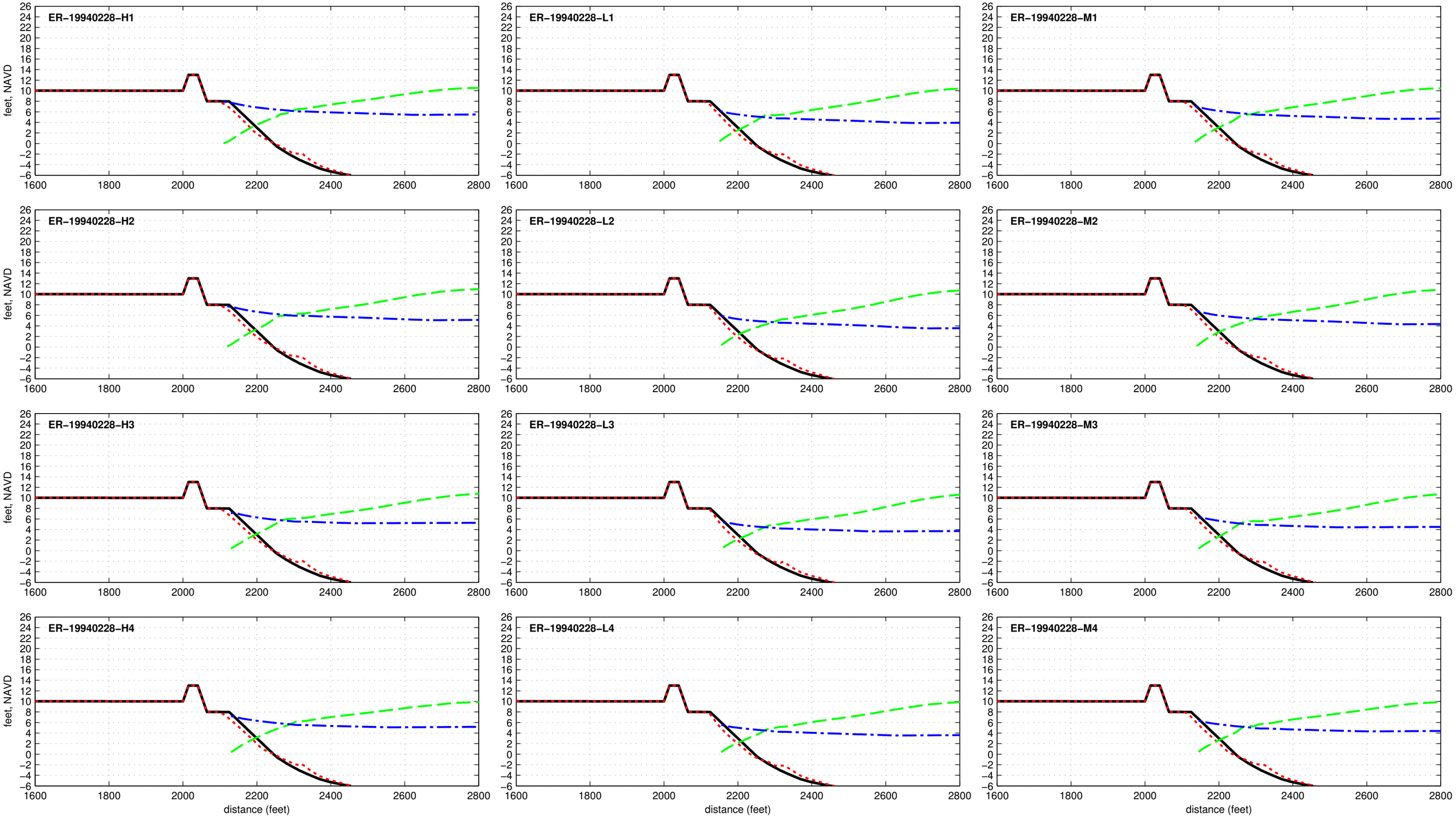

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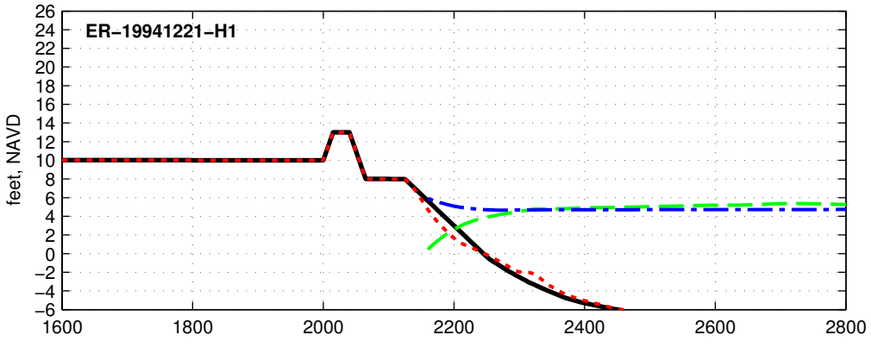
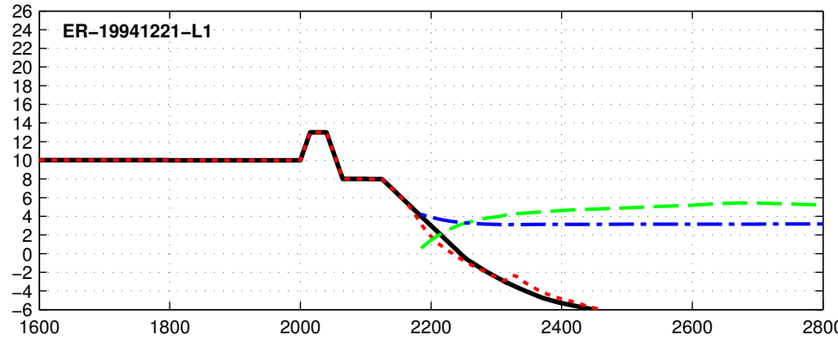
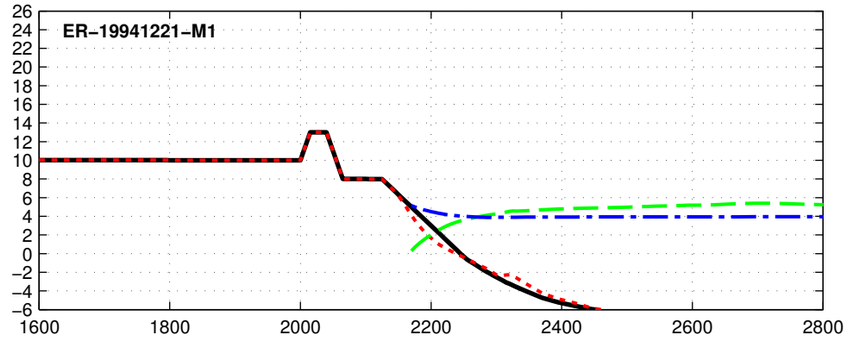
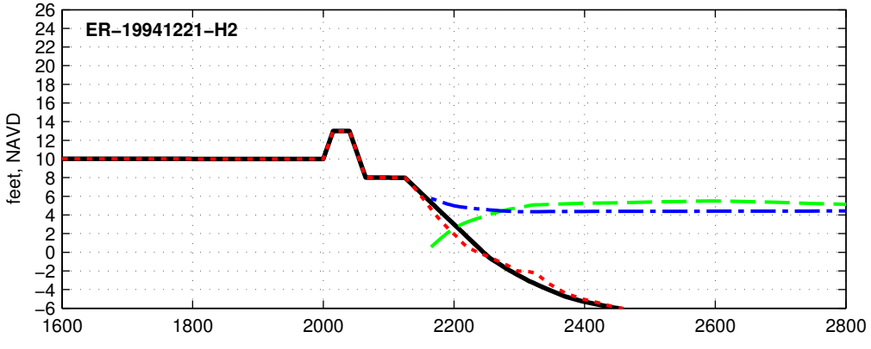
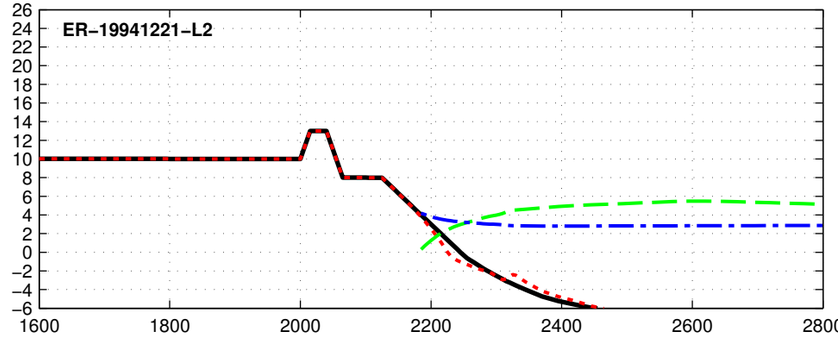
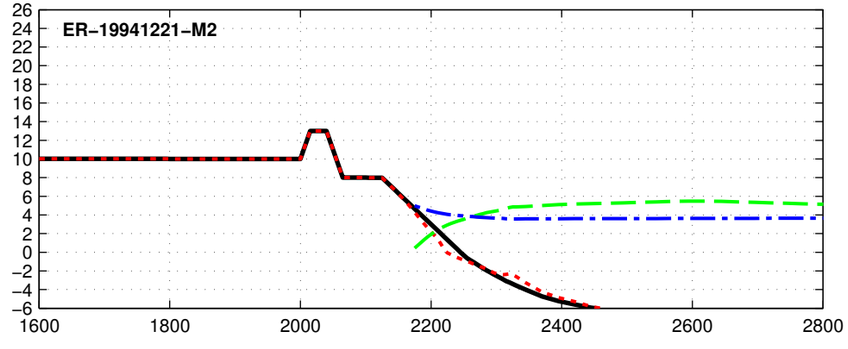


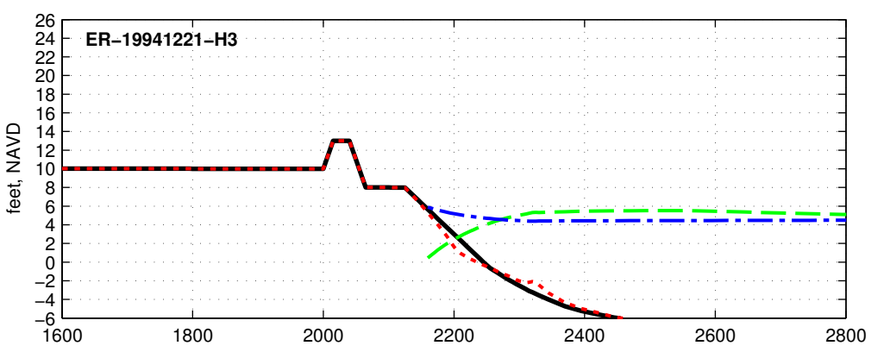


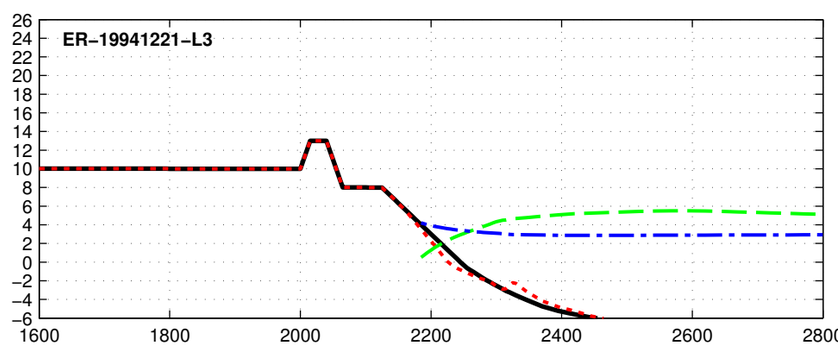


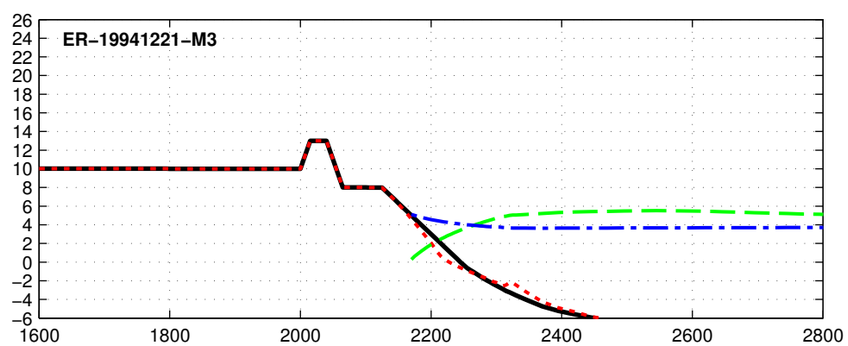


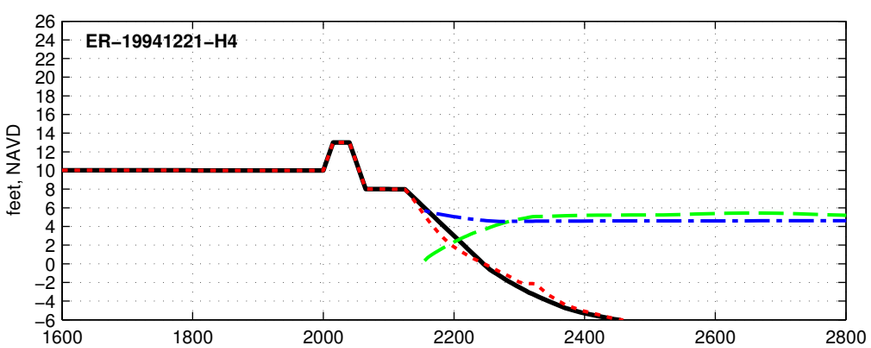


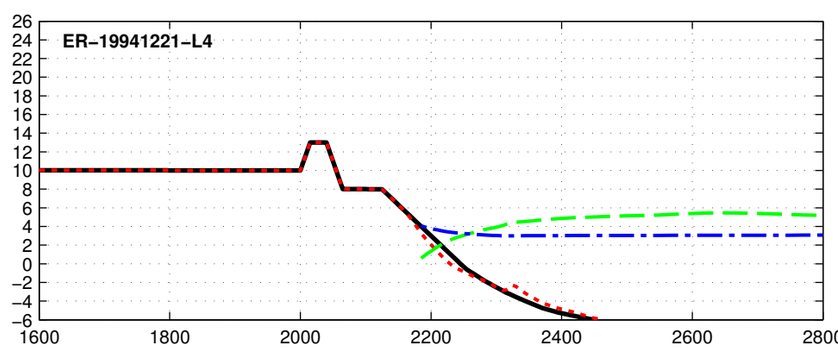


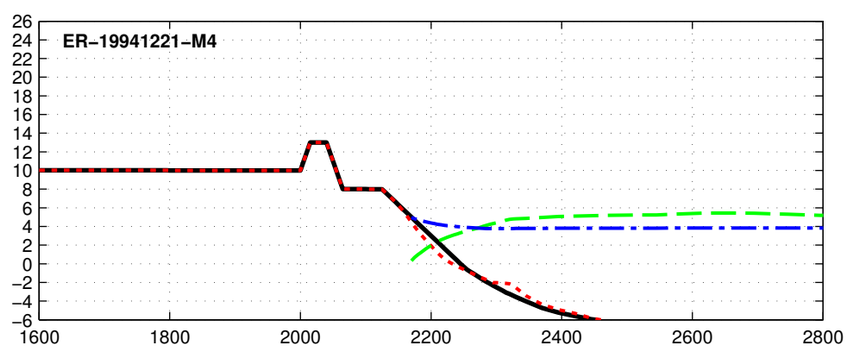






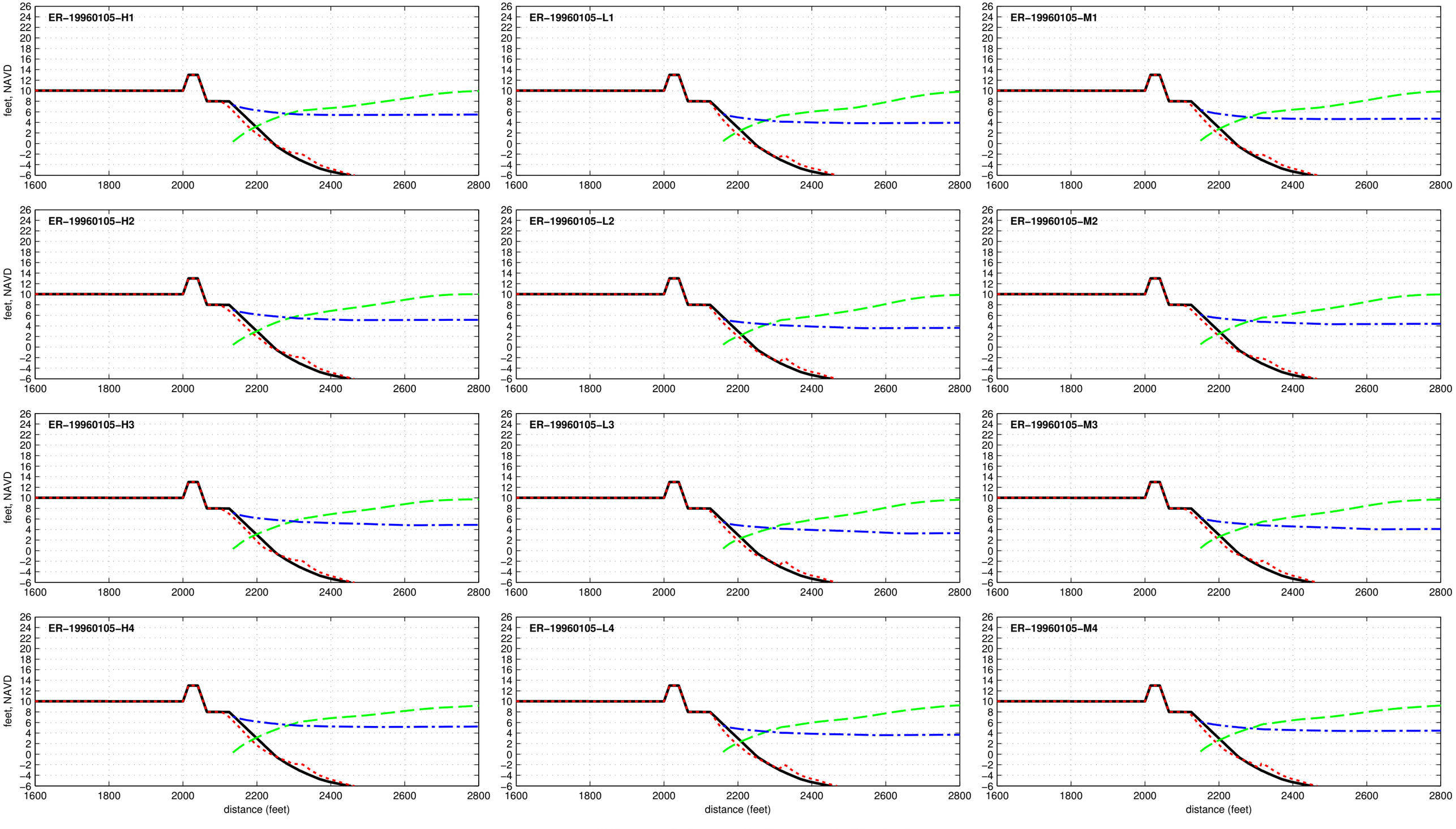




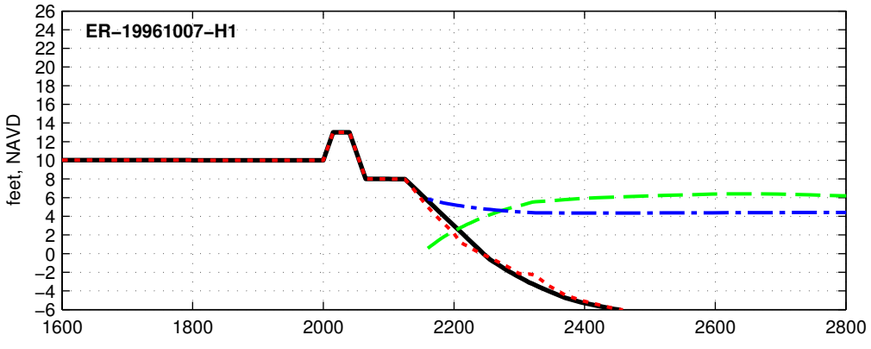


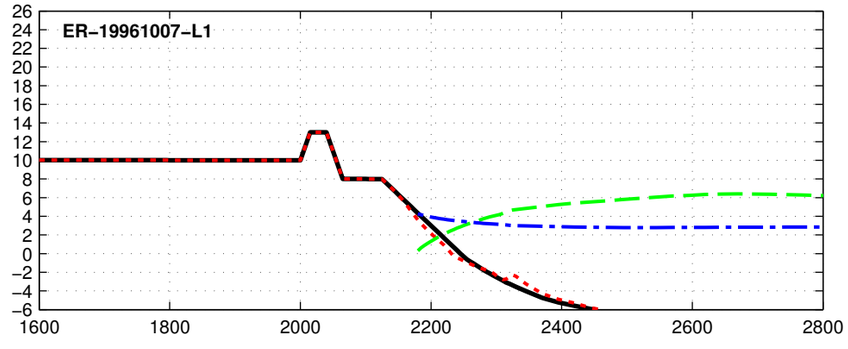
Initial Final Max Hs Max Water Level

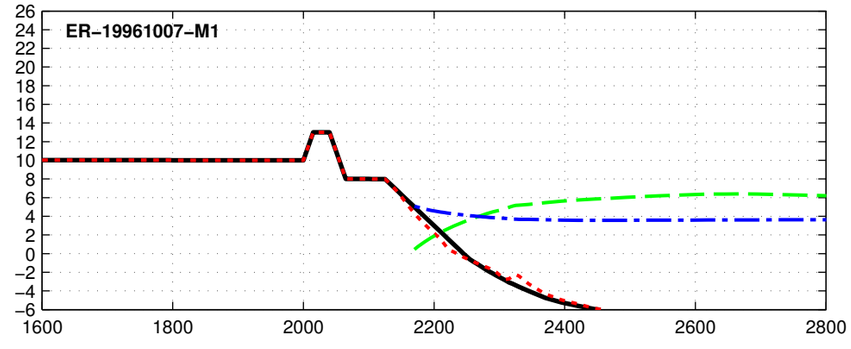
Dune R2T1-13.0-025-060 - ER-19960105

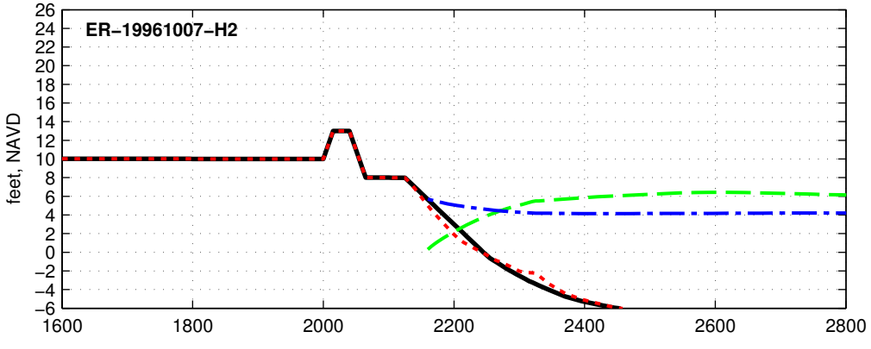


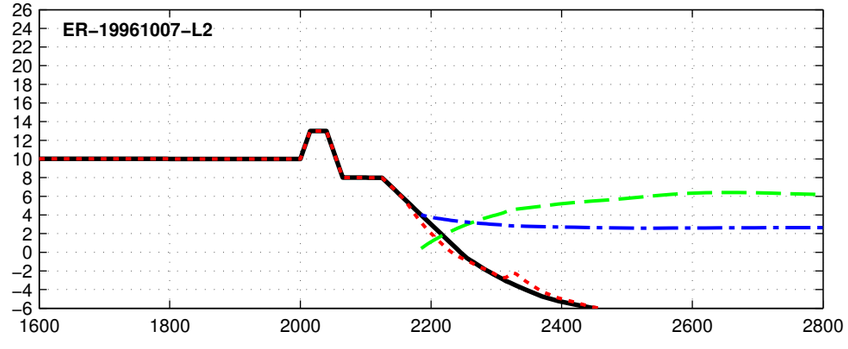
Dune R2T1–13.0–025–060 – ER–19961007

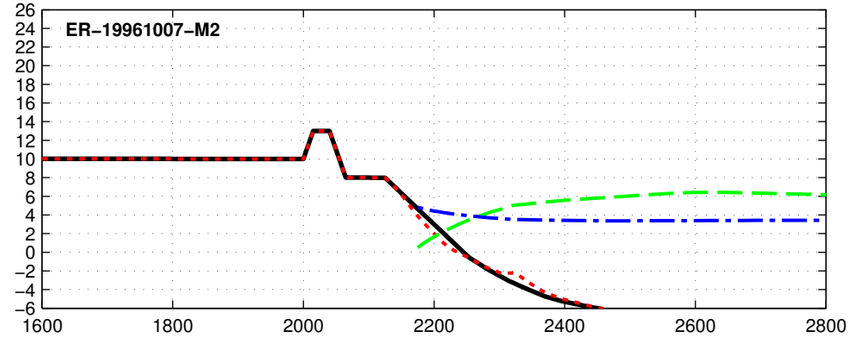


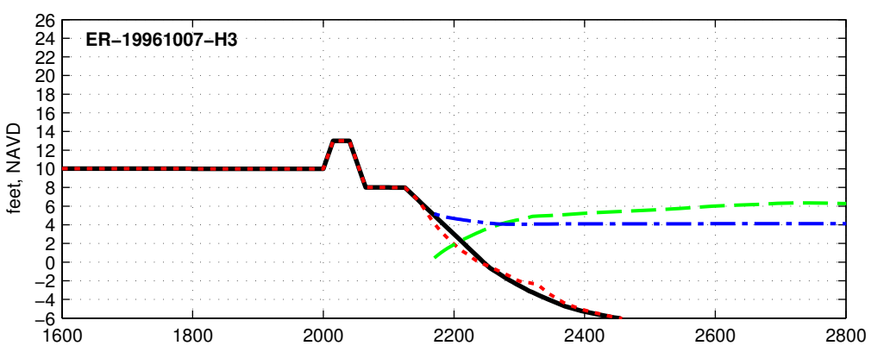


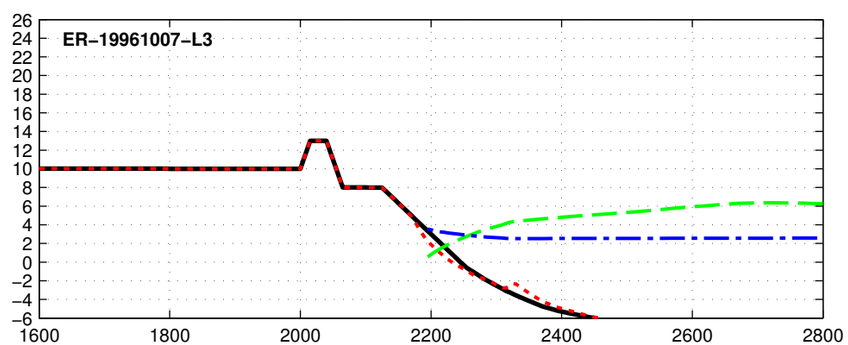


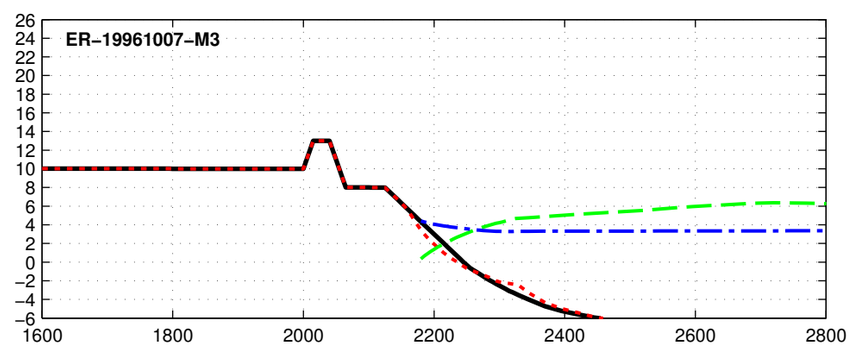


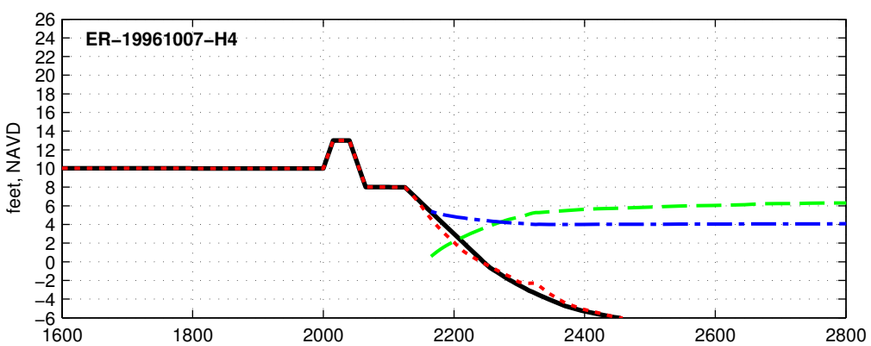


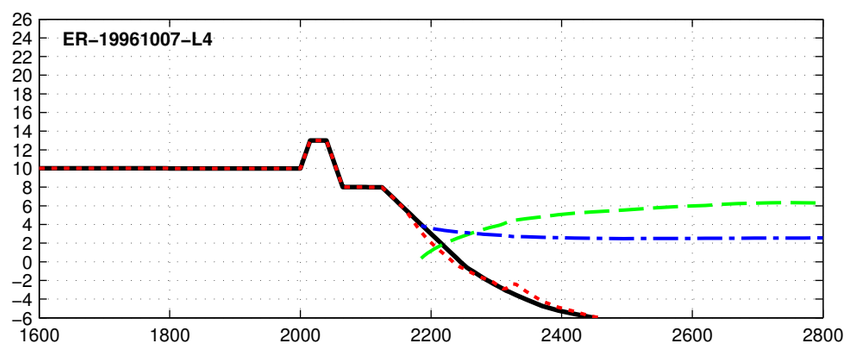


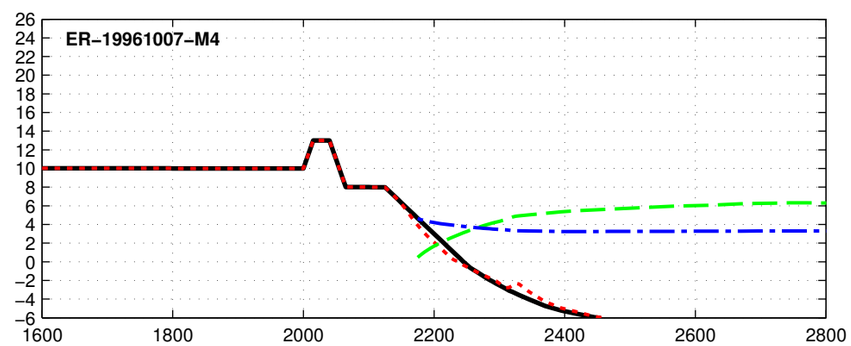






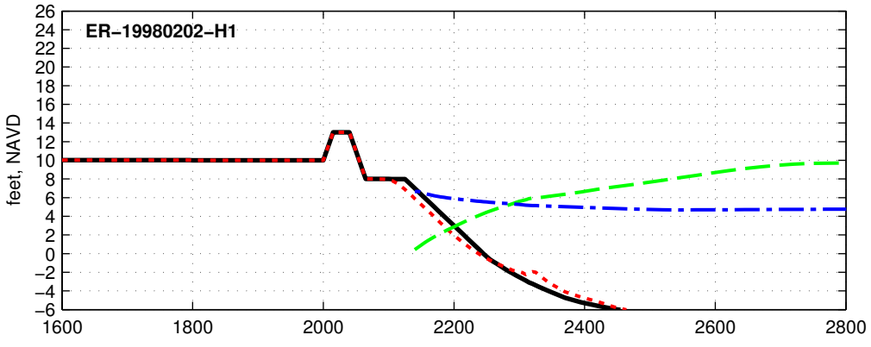


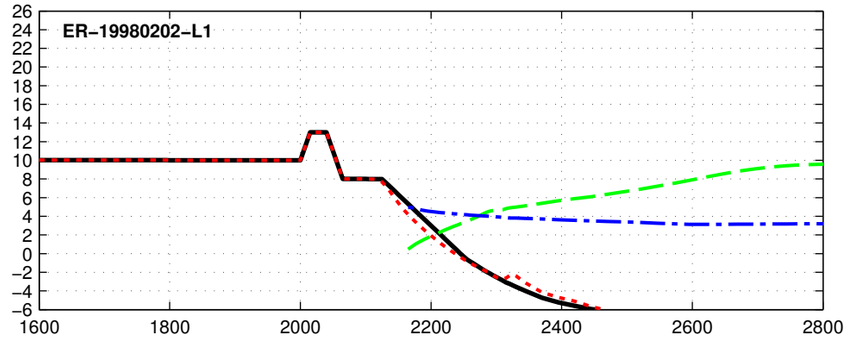


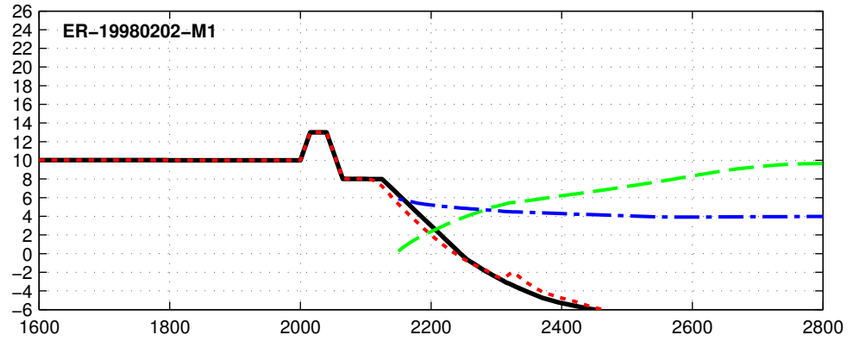


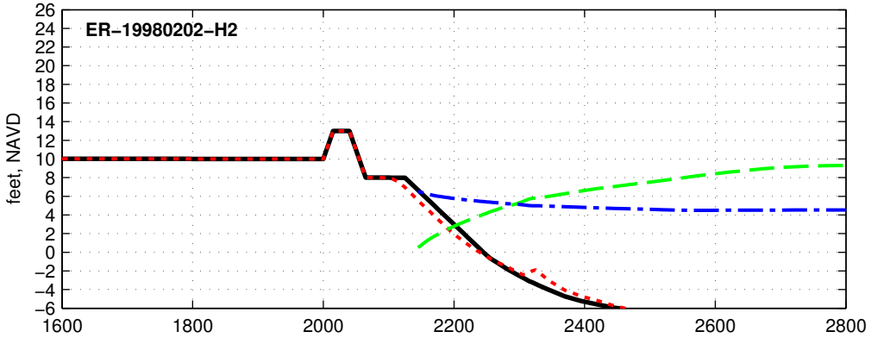
Initial Final Max Hs Max Water Level

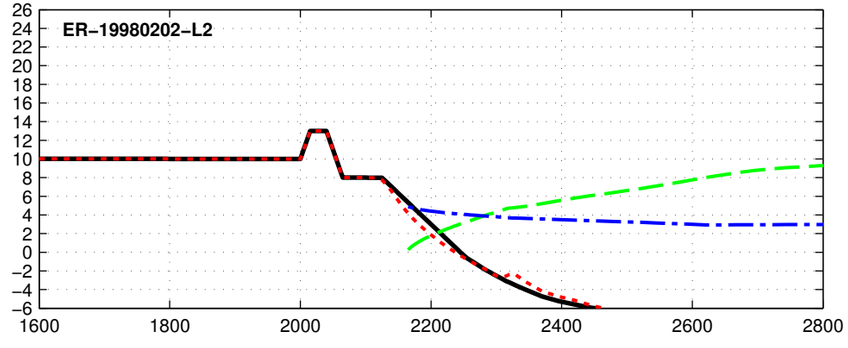
Dune R2T1–13.0–025–060 – ER–19980202

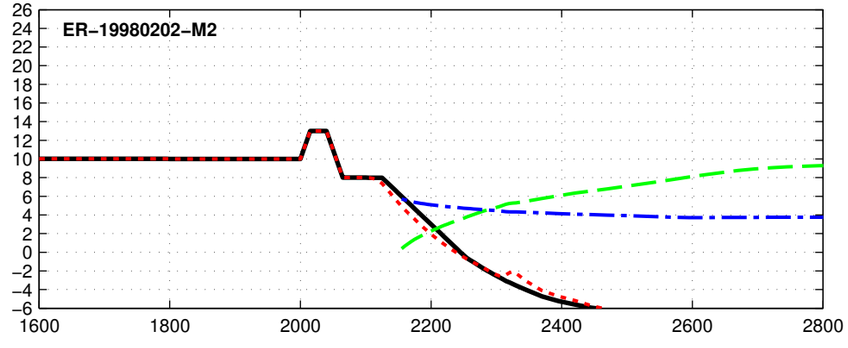


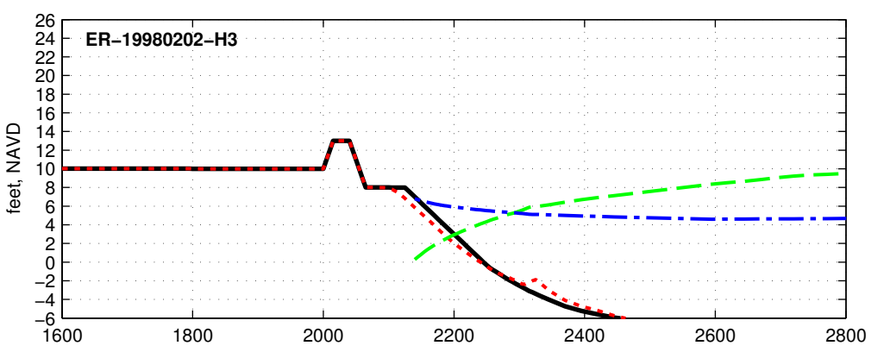


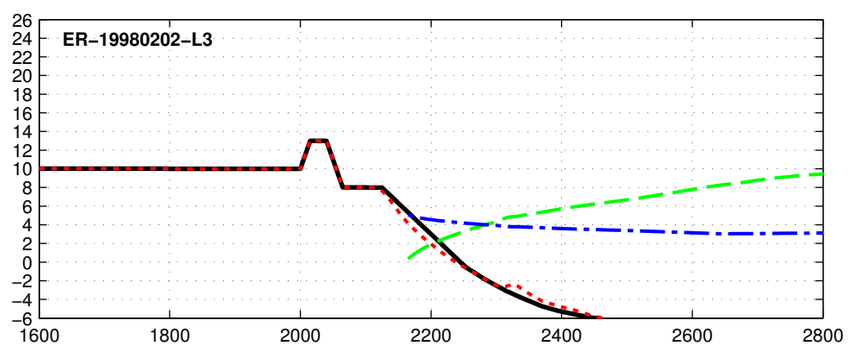


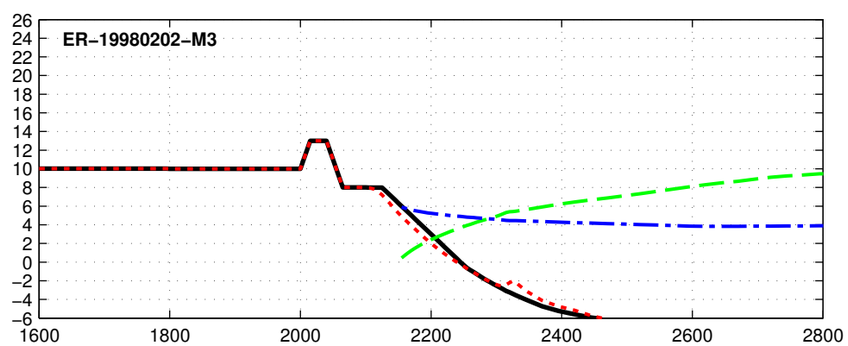


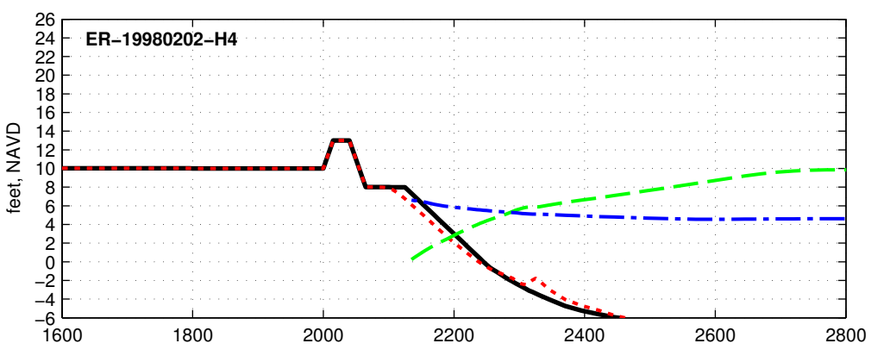


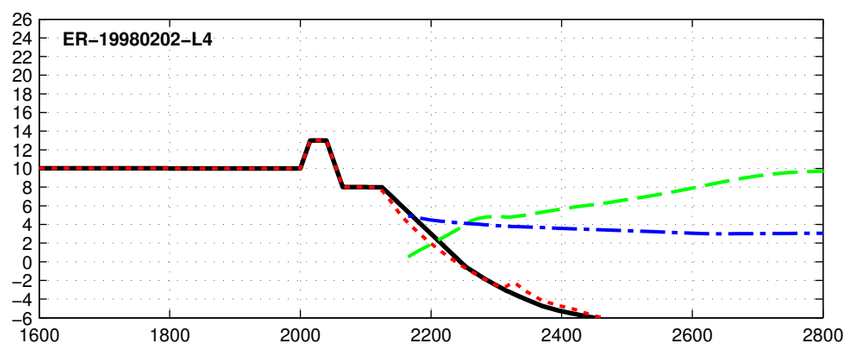


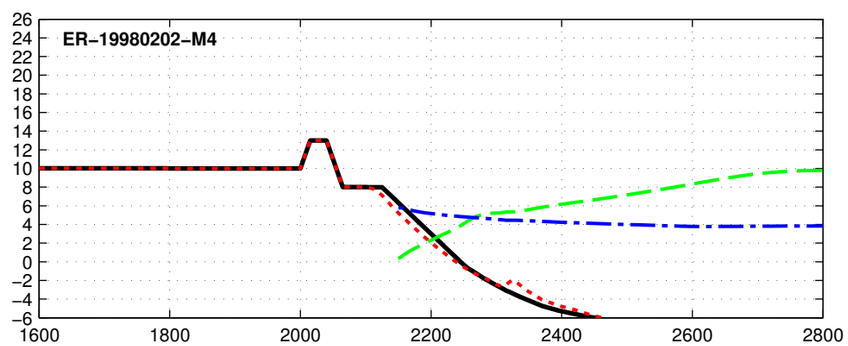






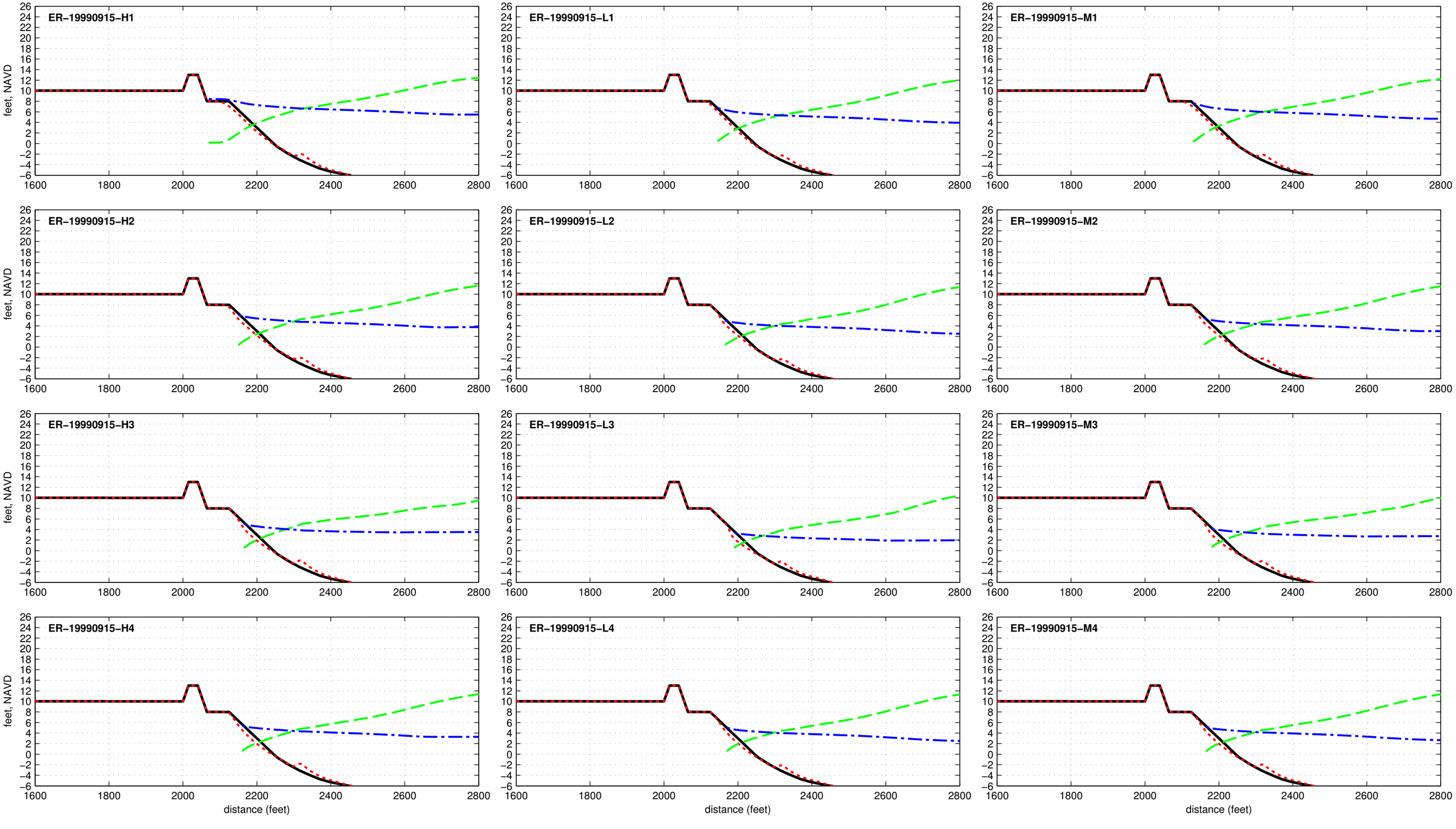




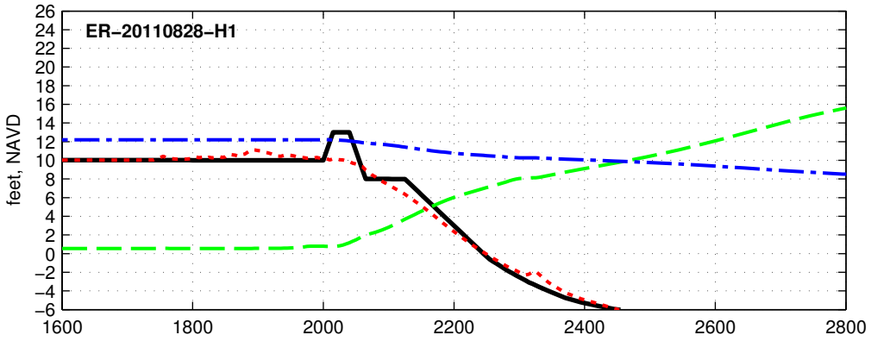


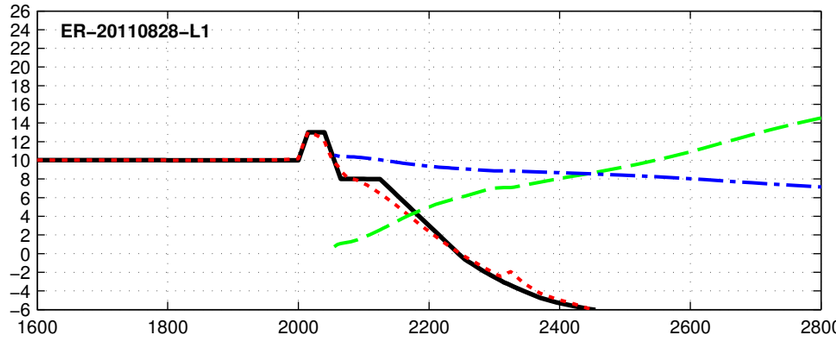
Initial Final Max Hs Max Water Level

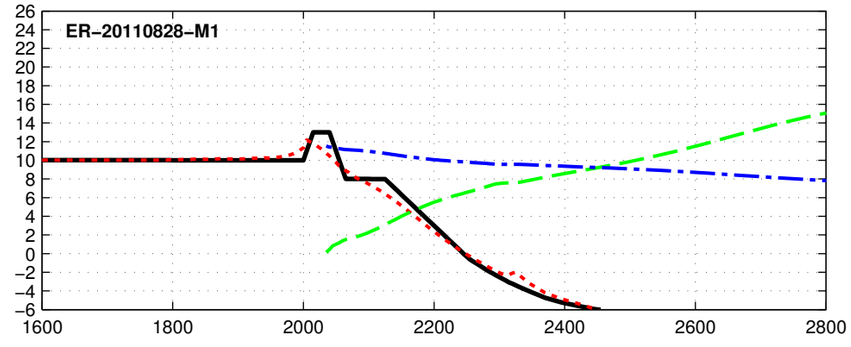
Dune R2T1-13.0-025-060 - ER-19990915

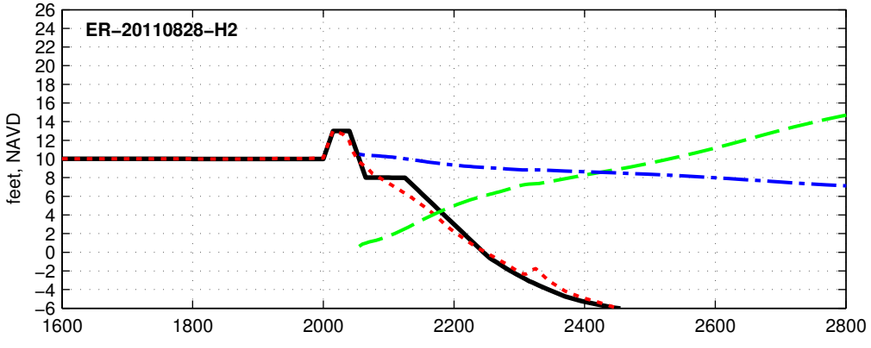


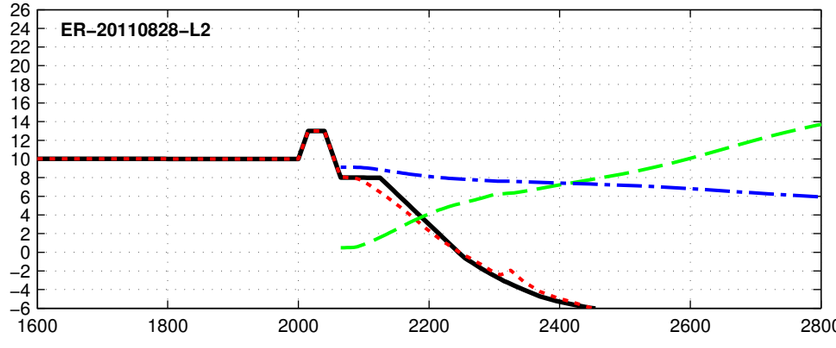
Dune R2T1−13.0−025−060 − ER−20110828

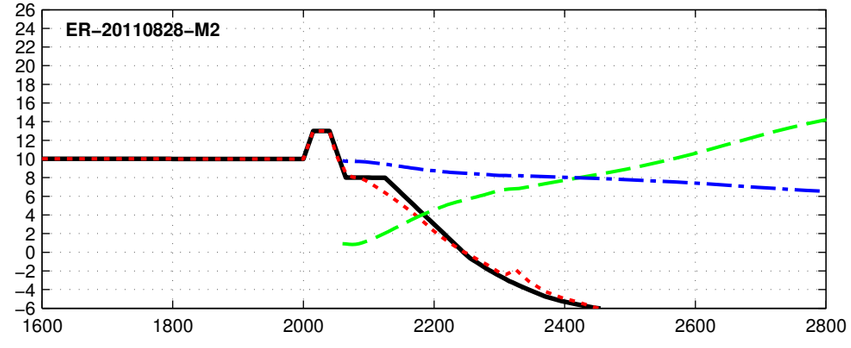


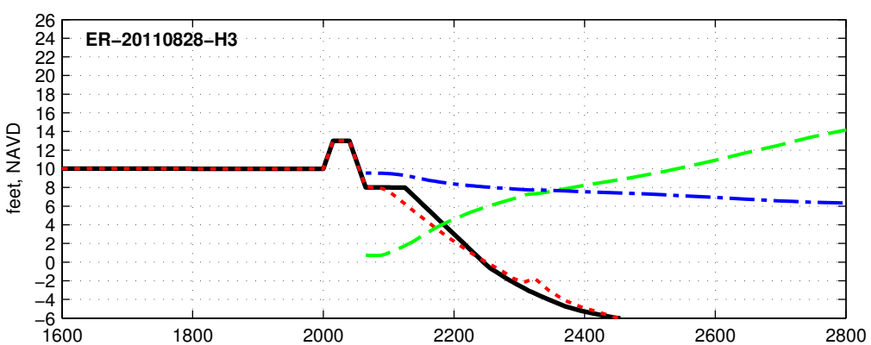


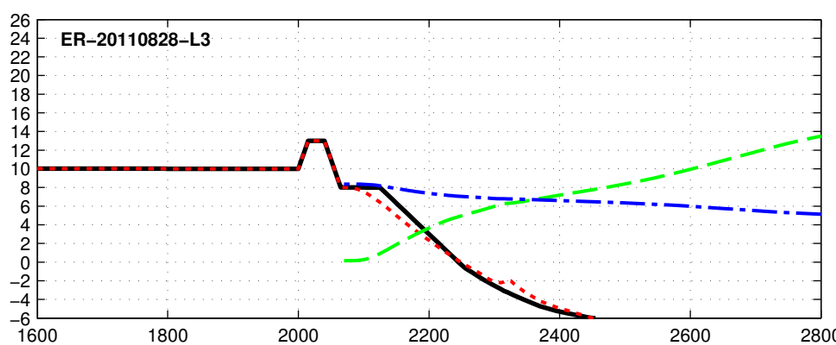


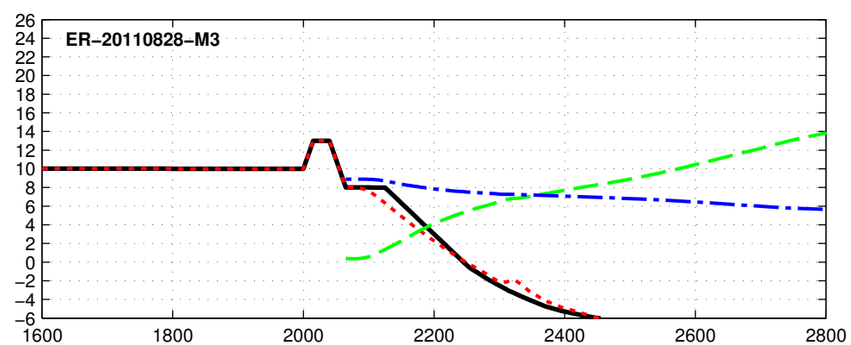


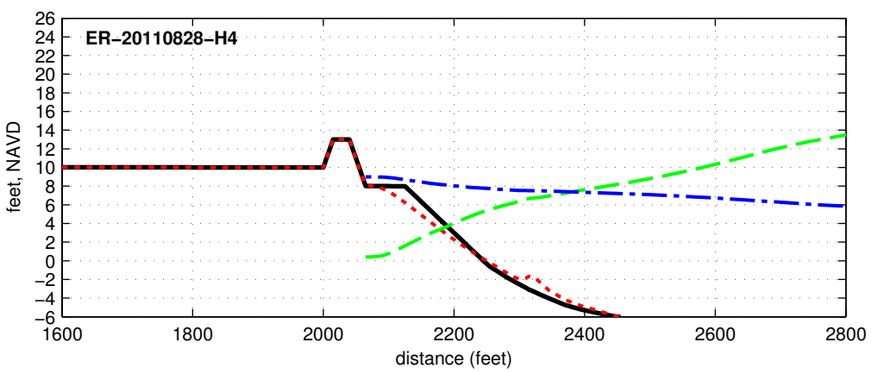


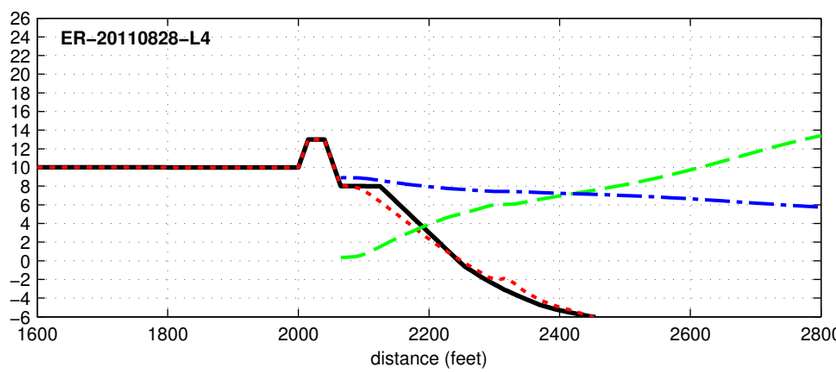


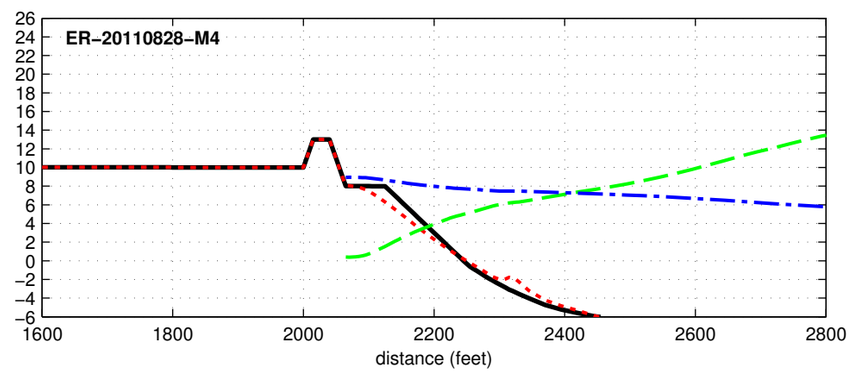






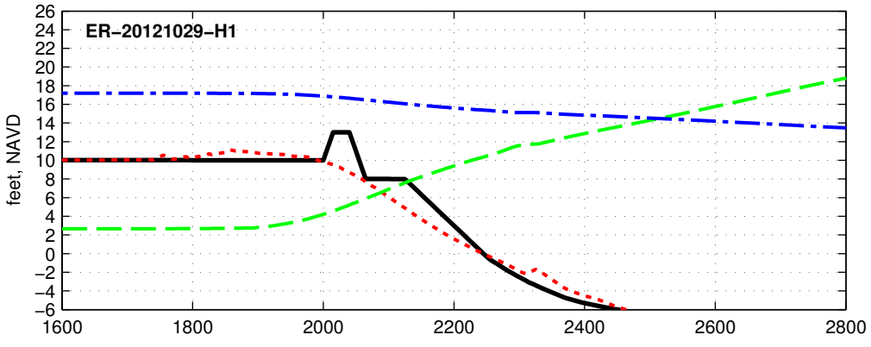


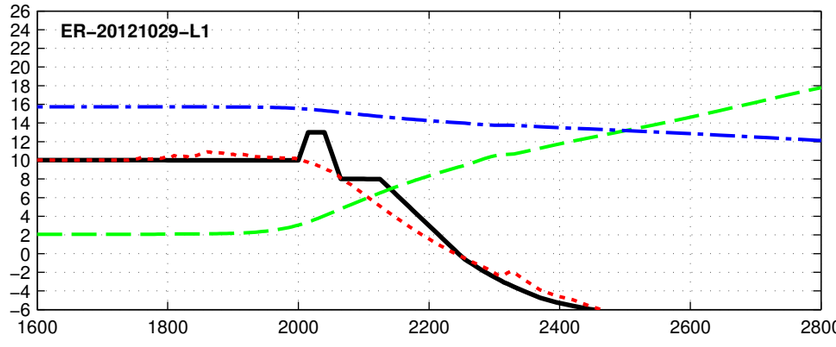


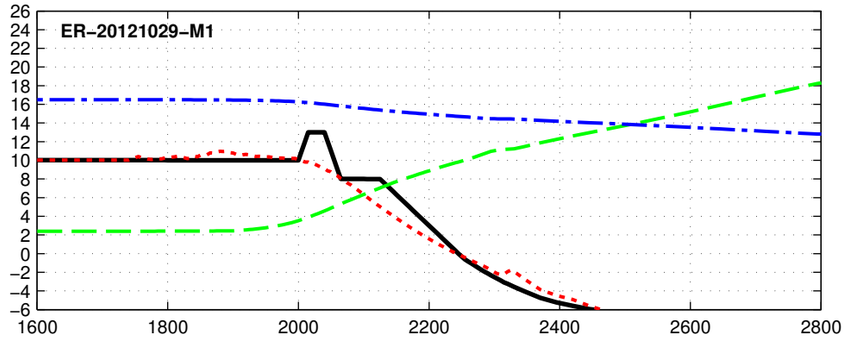


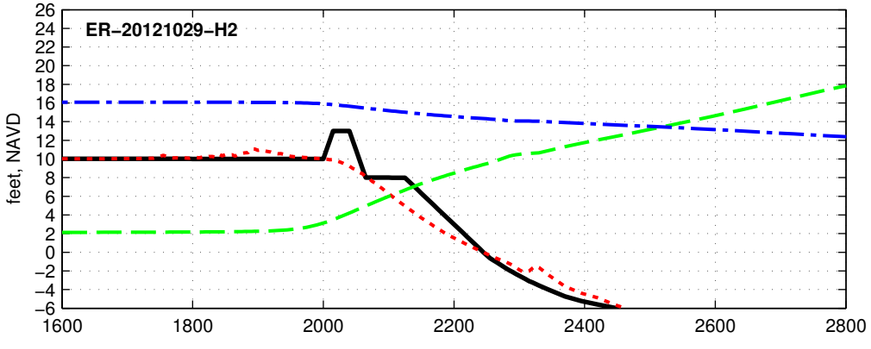
Initial Final Max Hs Max Water Level

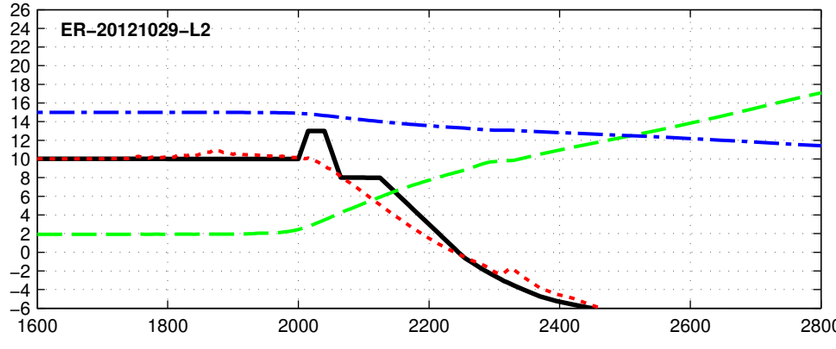
Dune R2T1–13.0–025–060 – ER–20121029

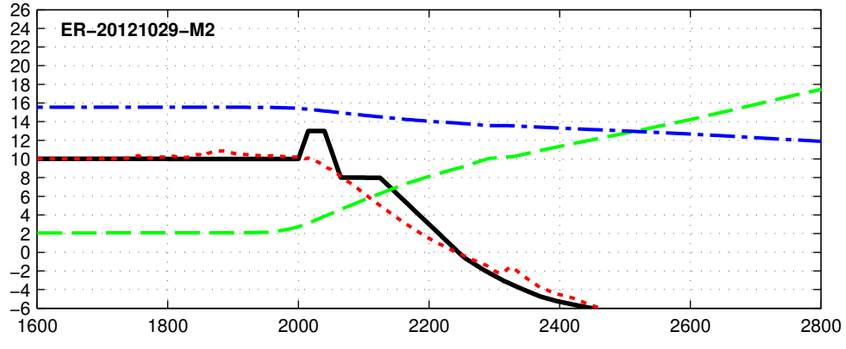


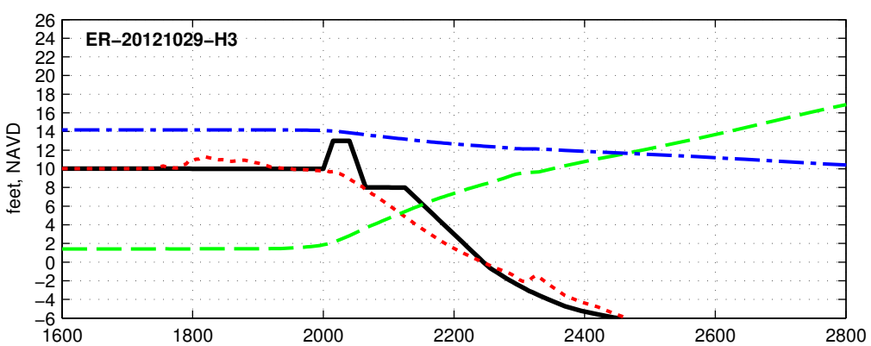


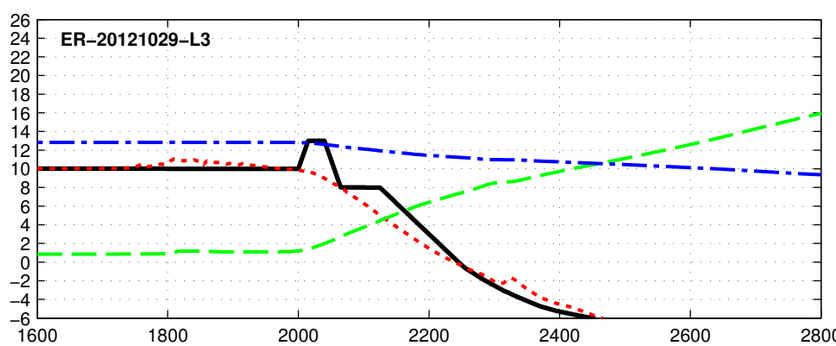


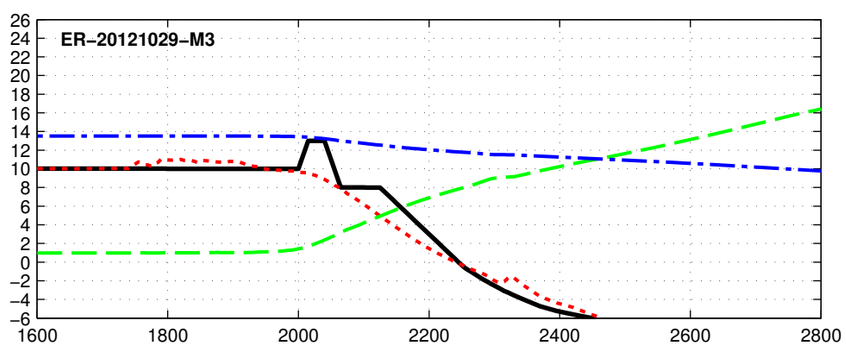


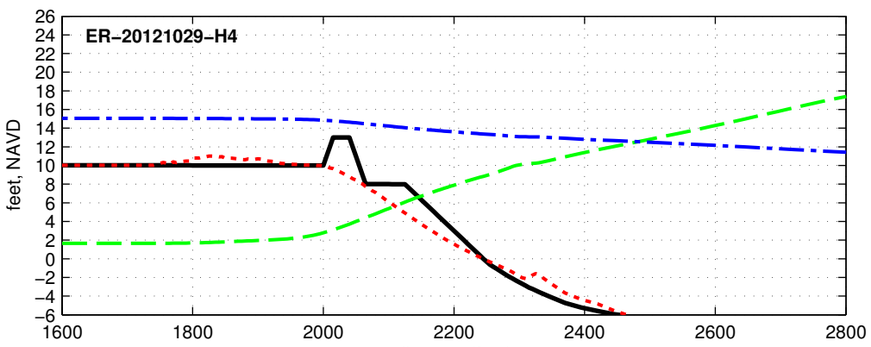


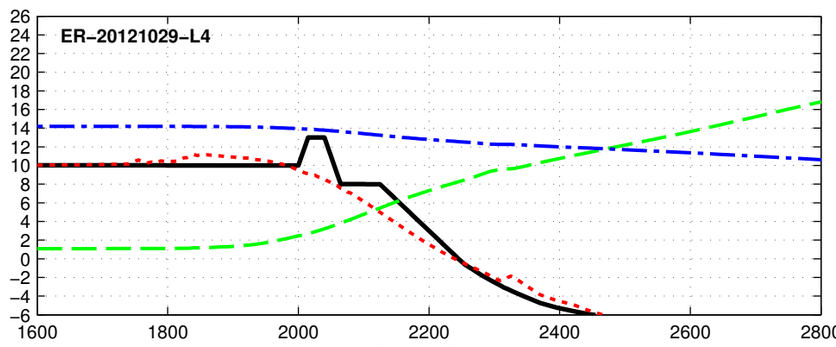


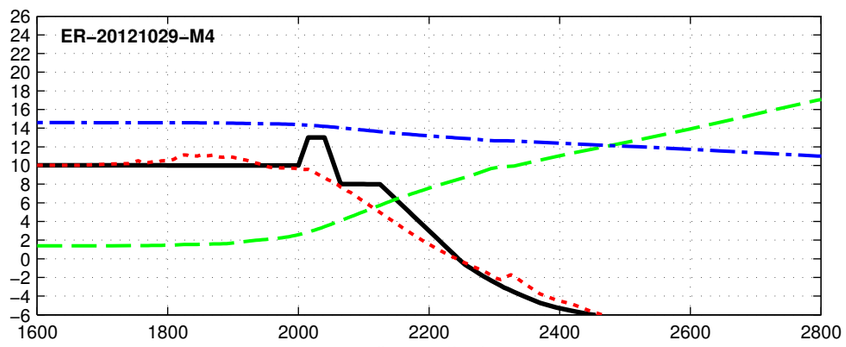








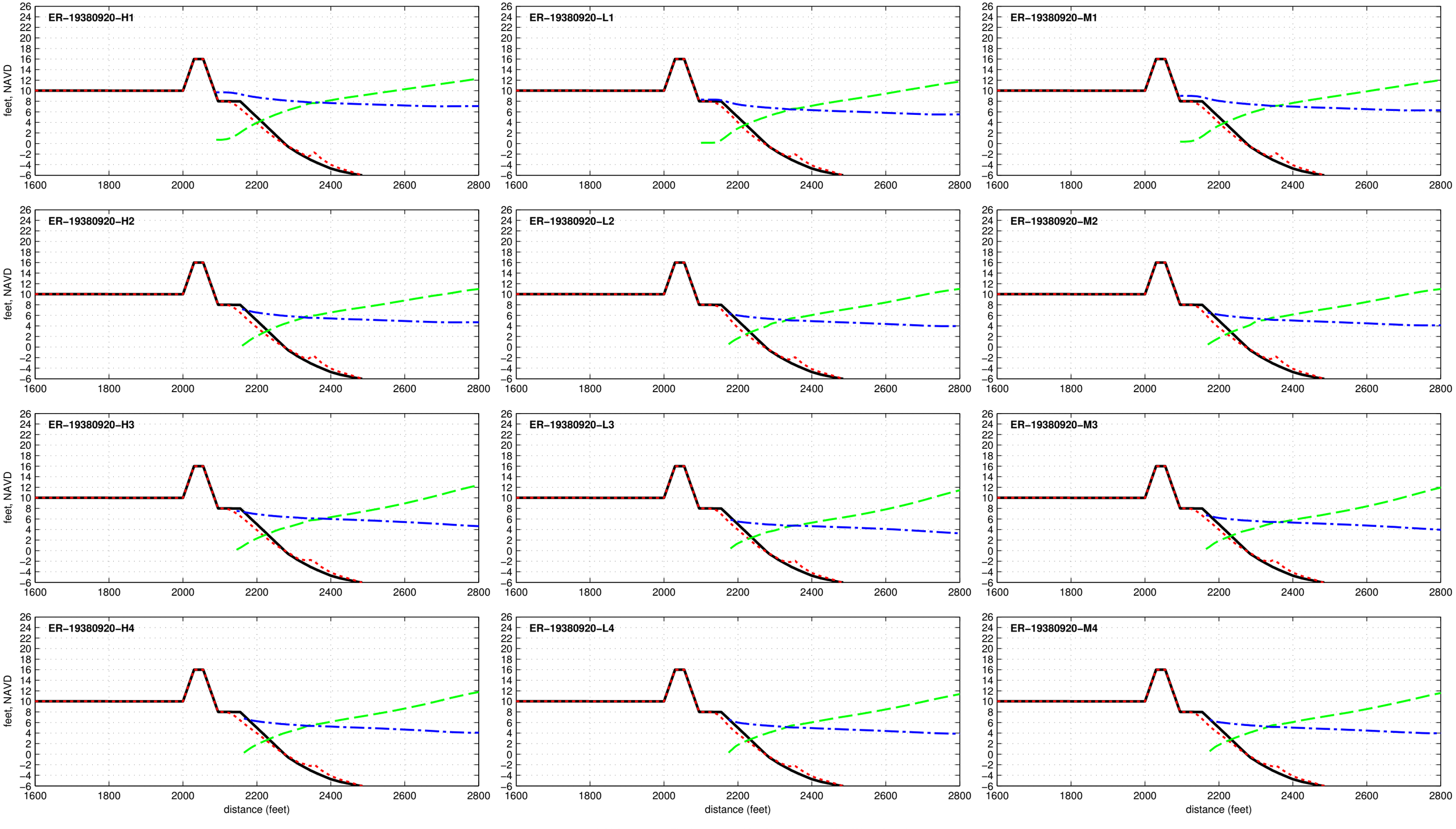




Initial Final Max Hs Max Water Level

R2T1_16.0_025_060 – DUNE

Dune R2T1-16.0-025-060 - ER-19380920



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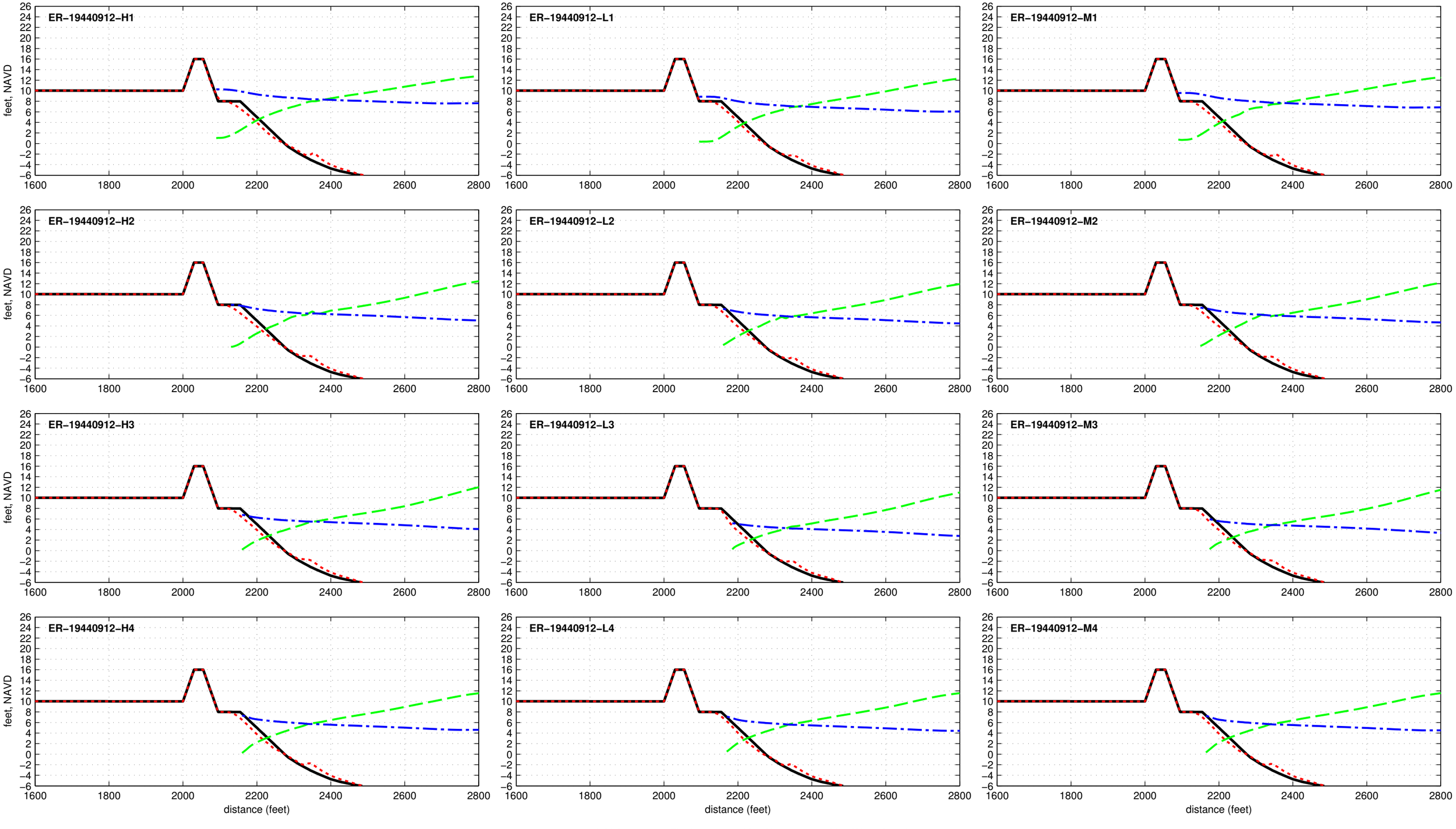

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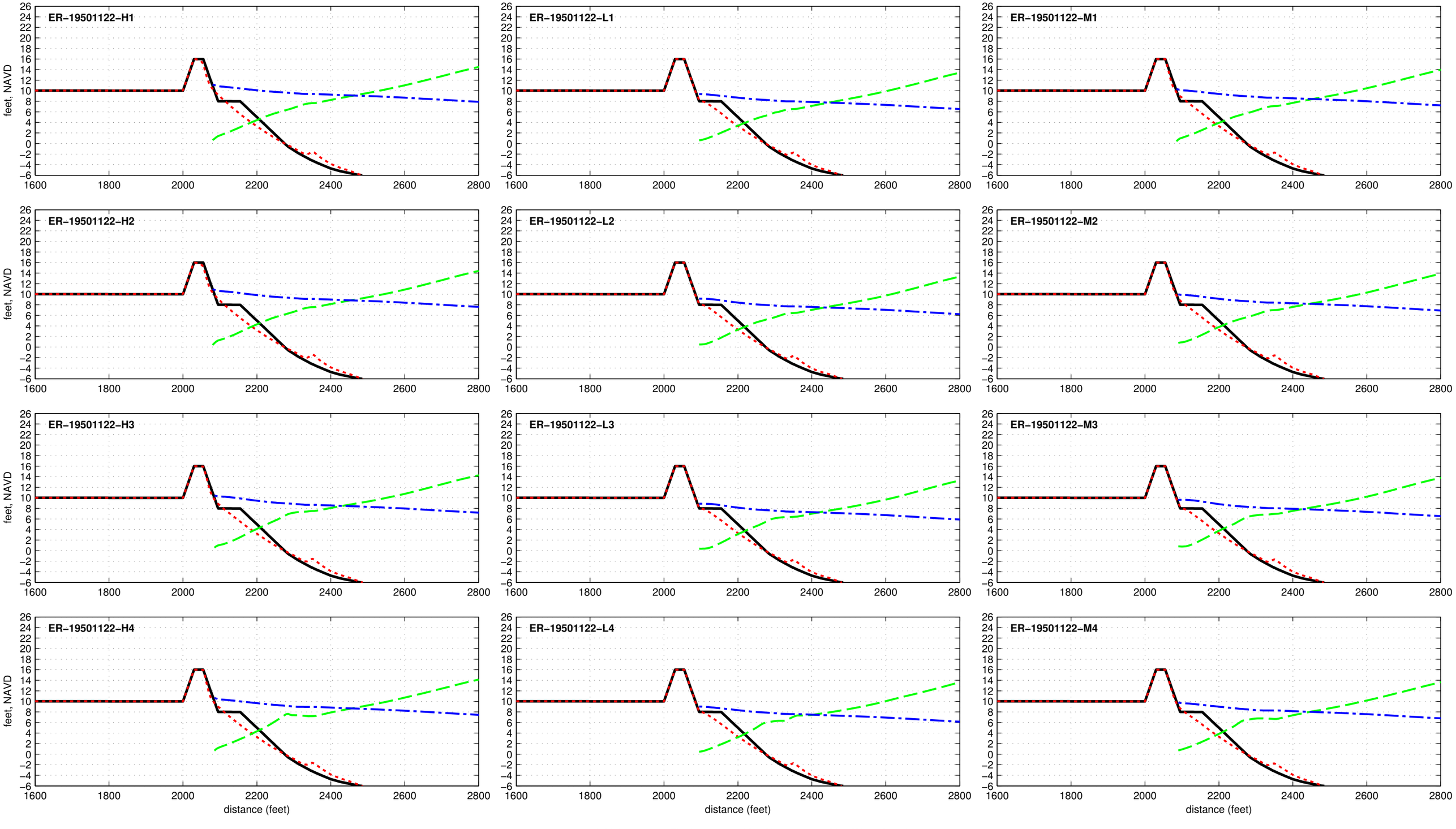

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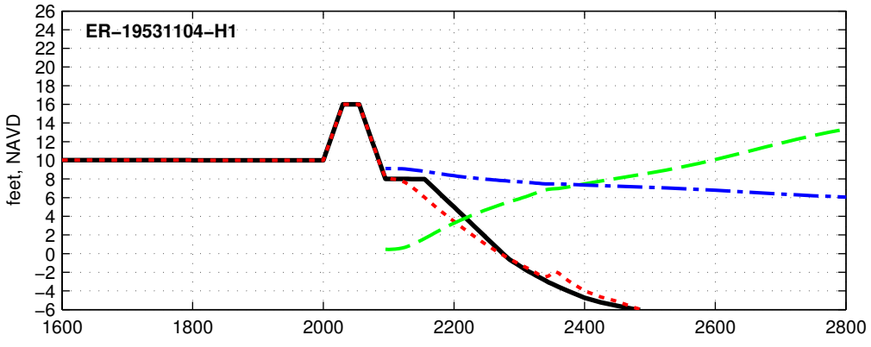
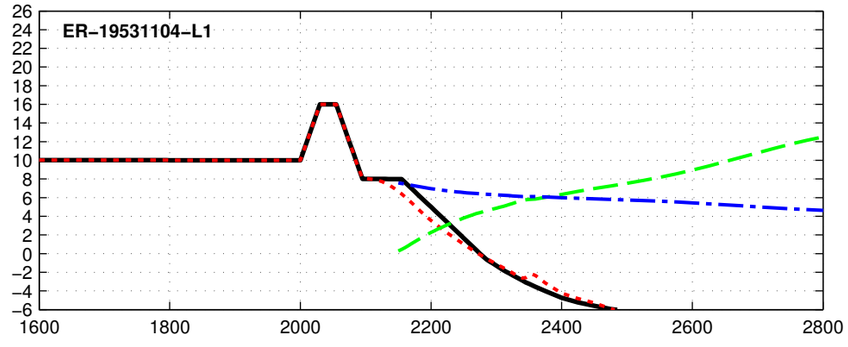
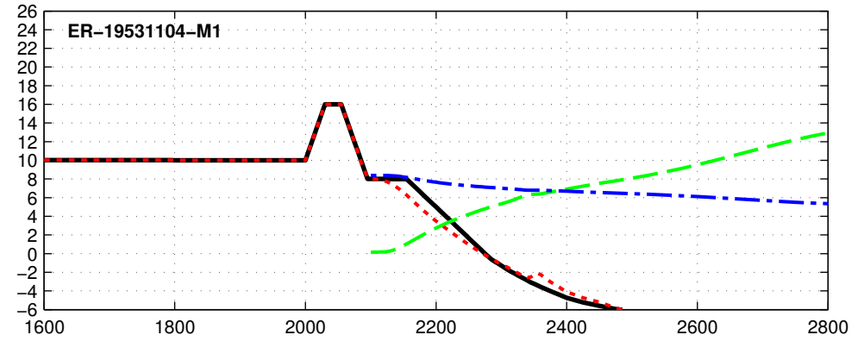
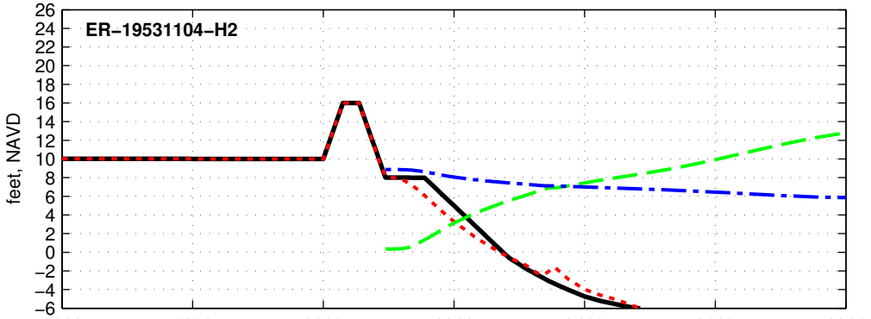
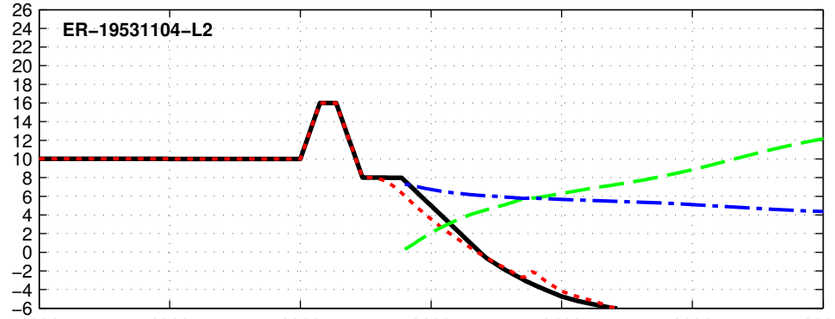
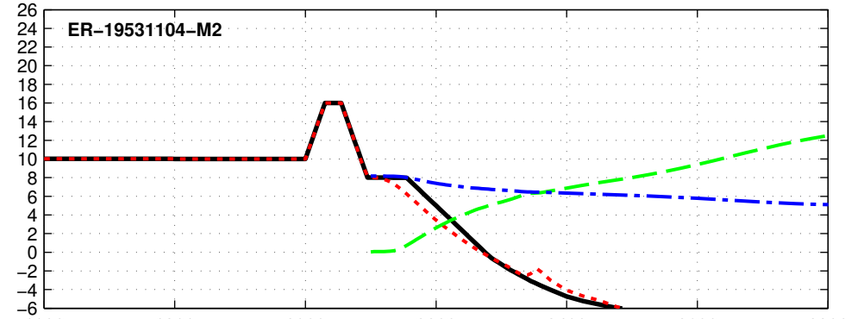


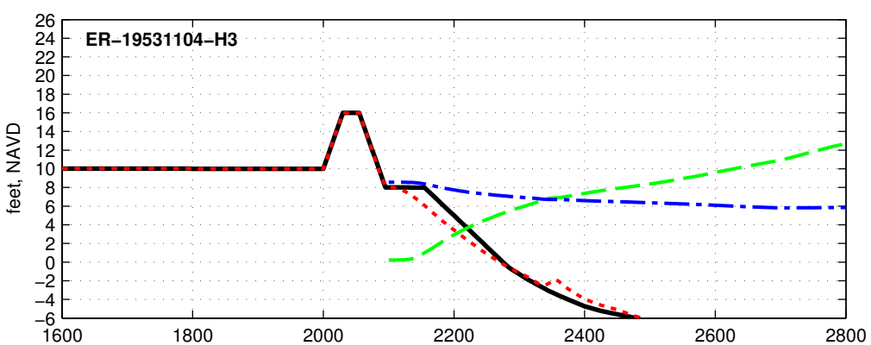


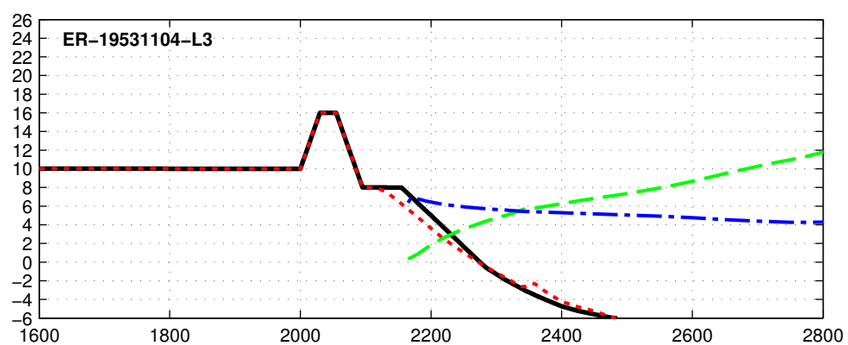


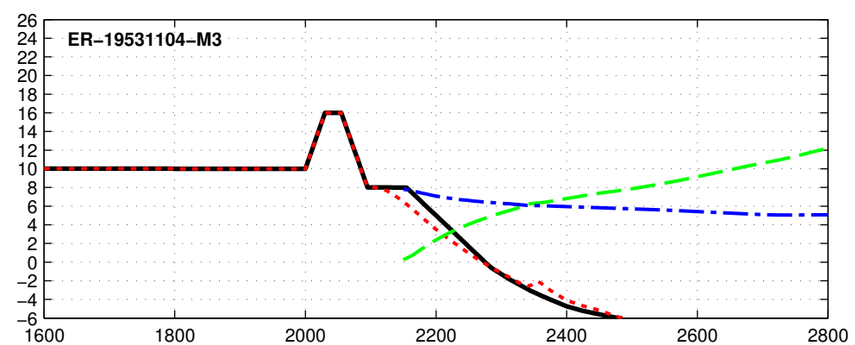


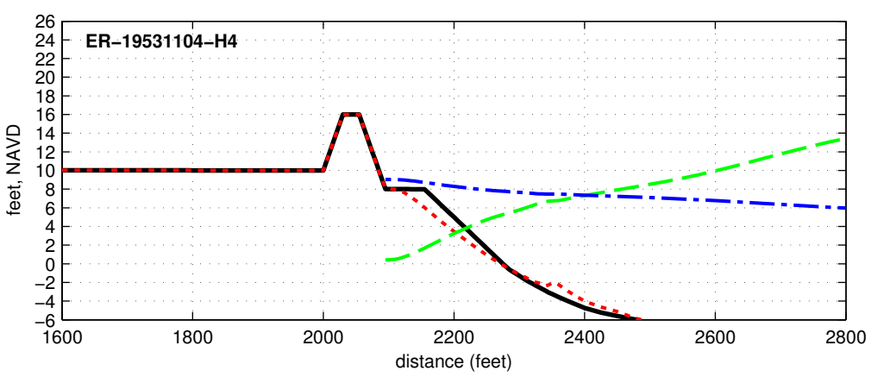


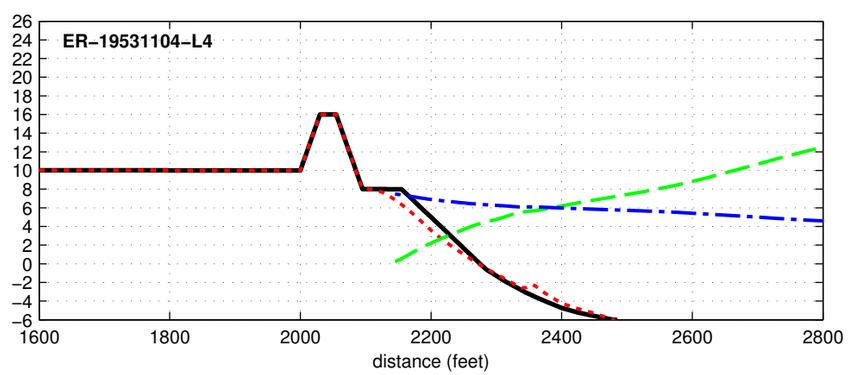


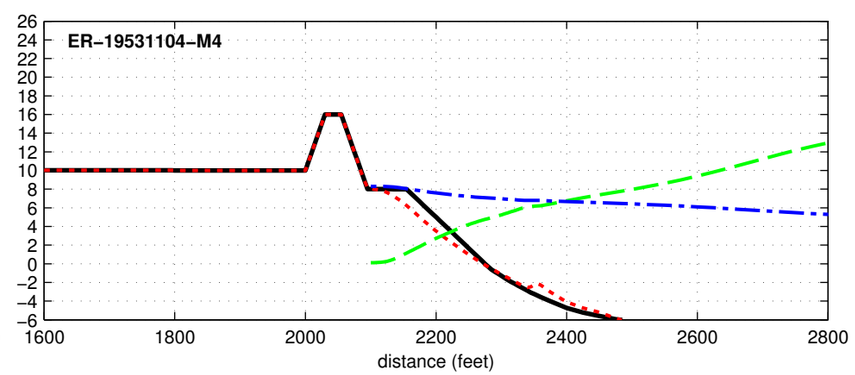






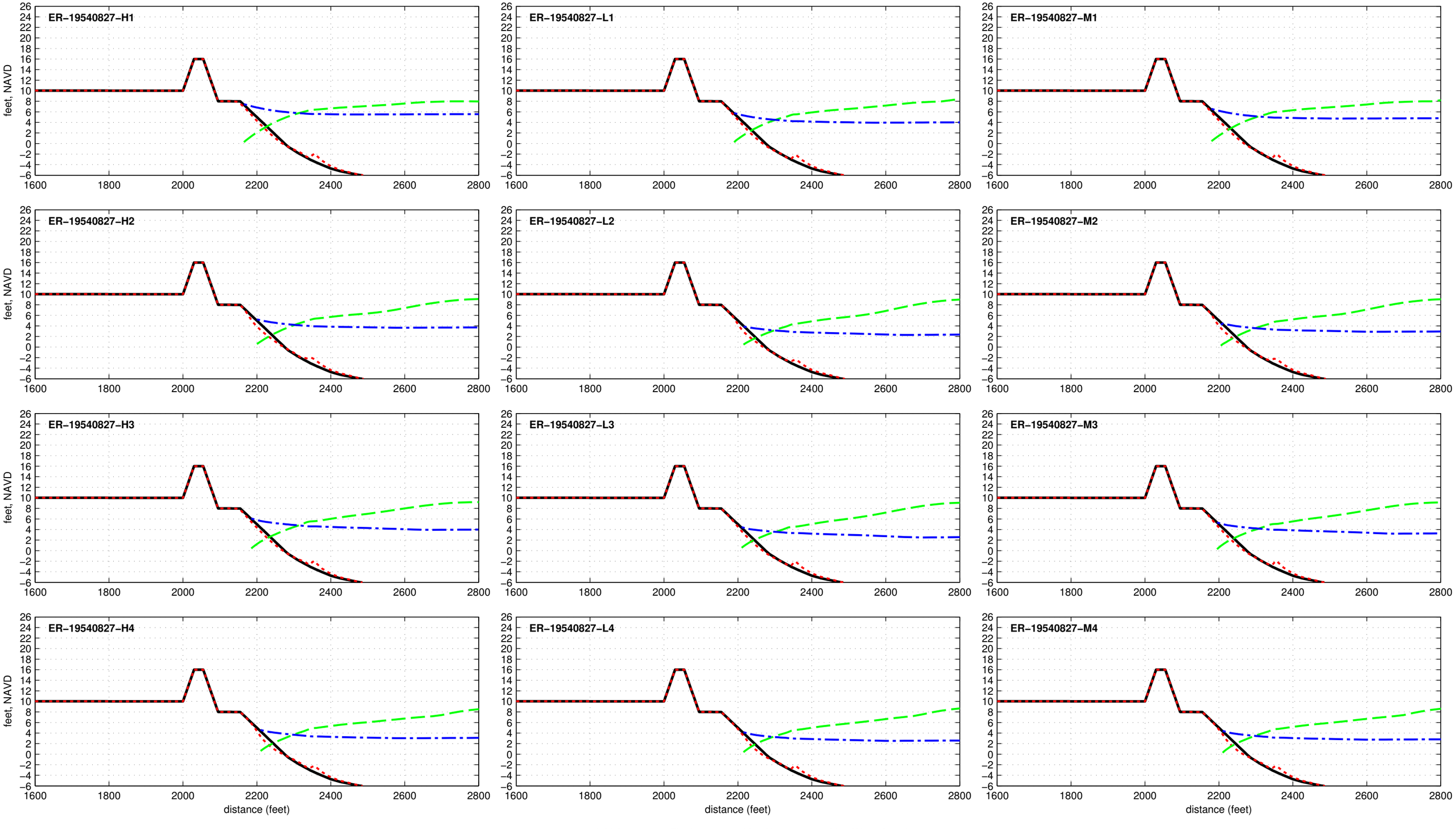






Initial Final Max Hs Max Water Level

Dune R2T1-16.0-025-060 - ER-19540827



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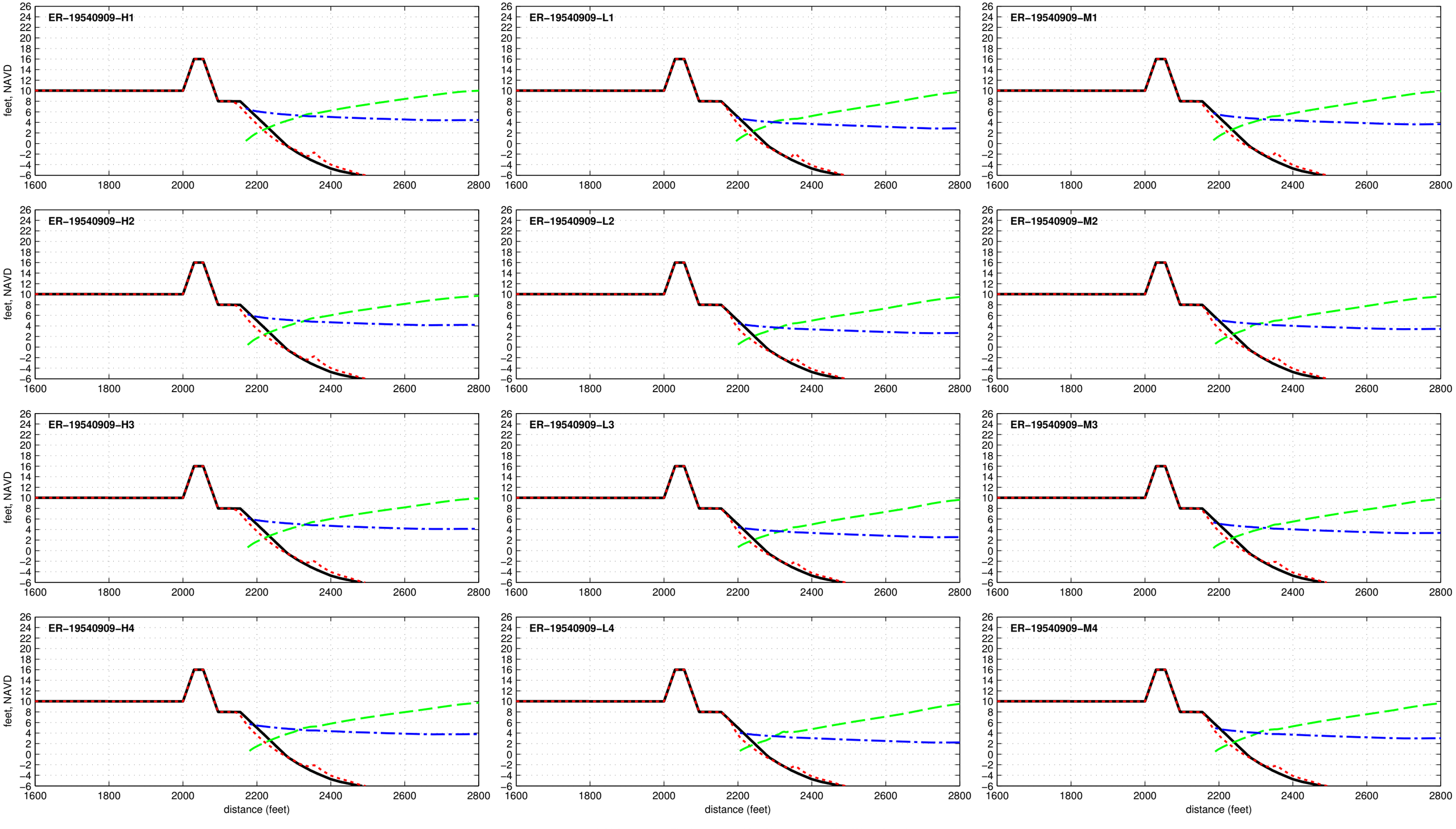

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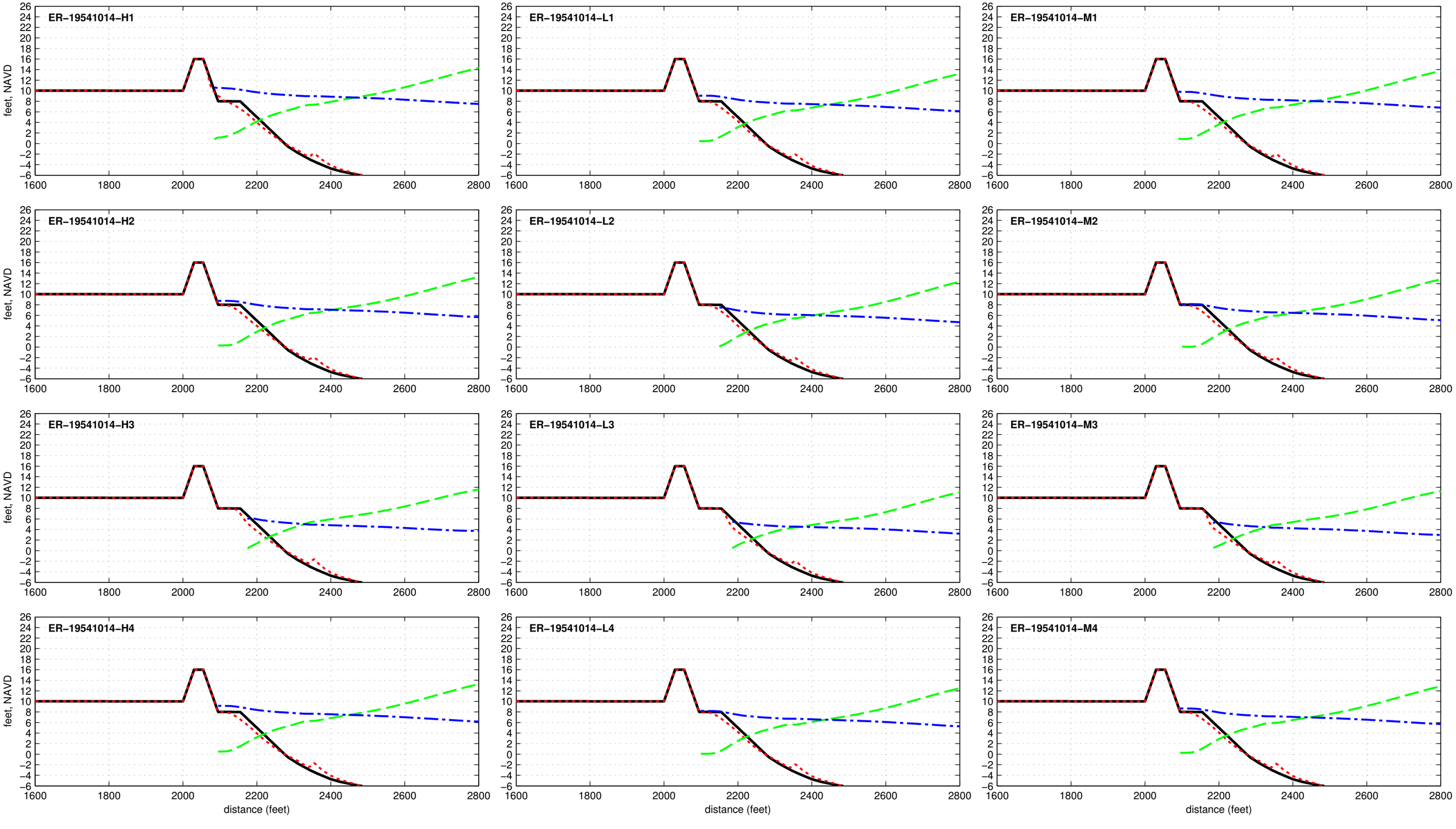

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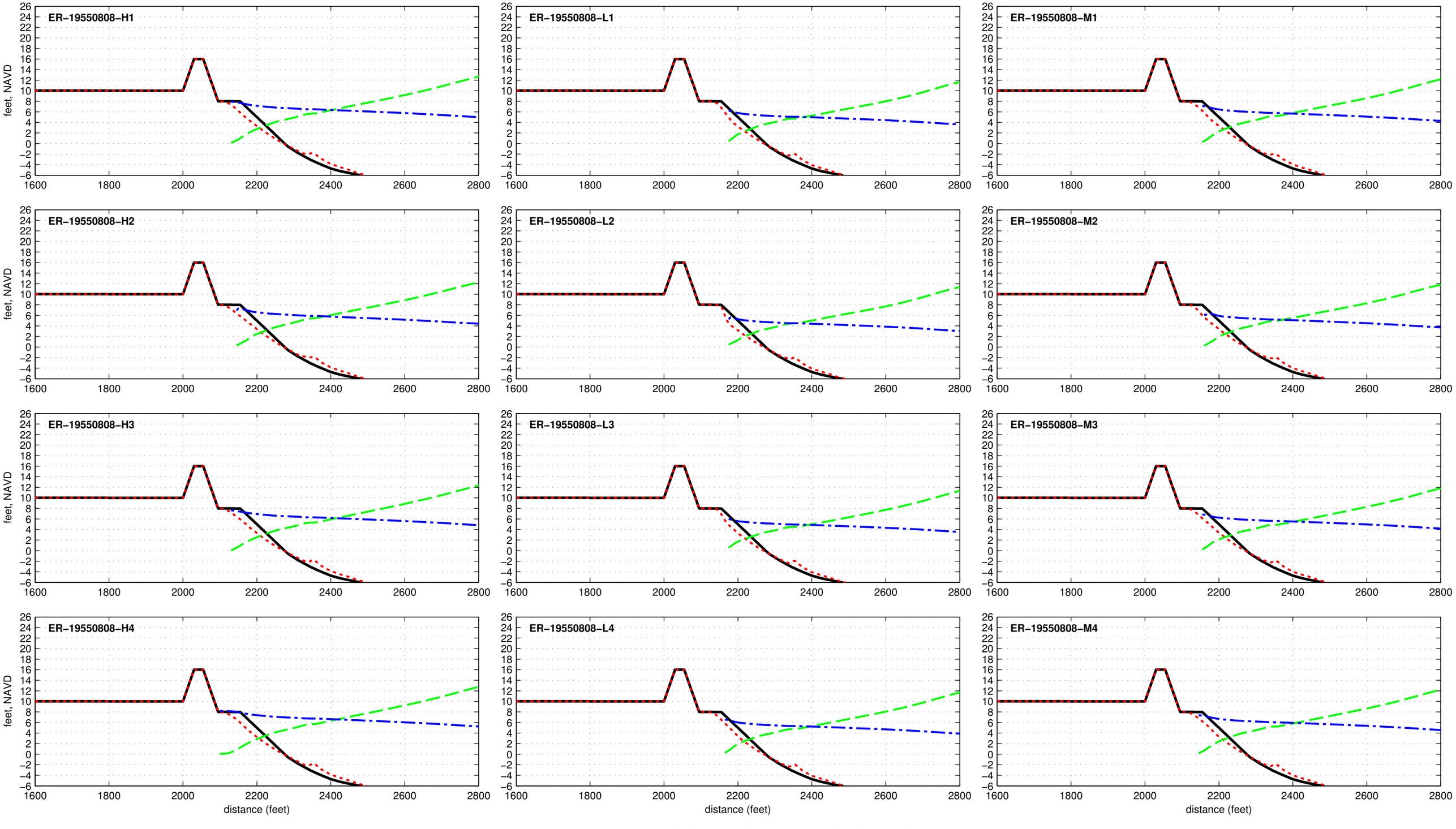

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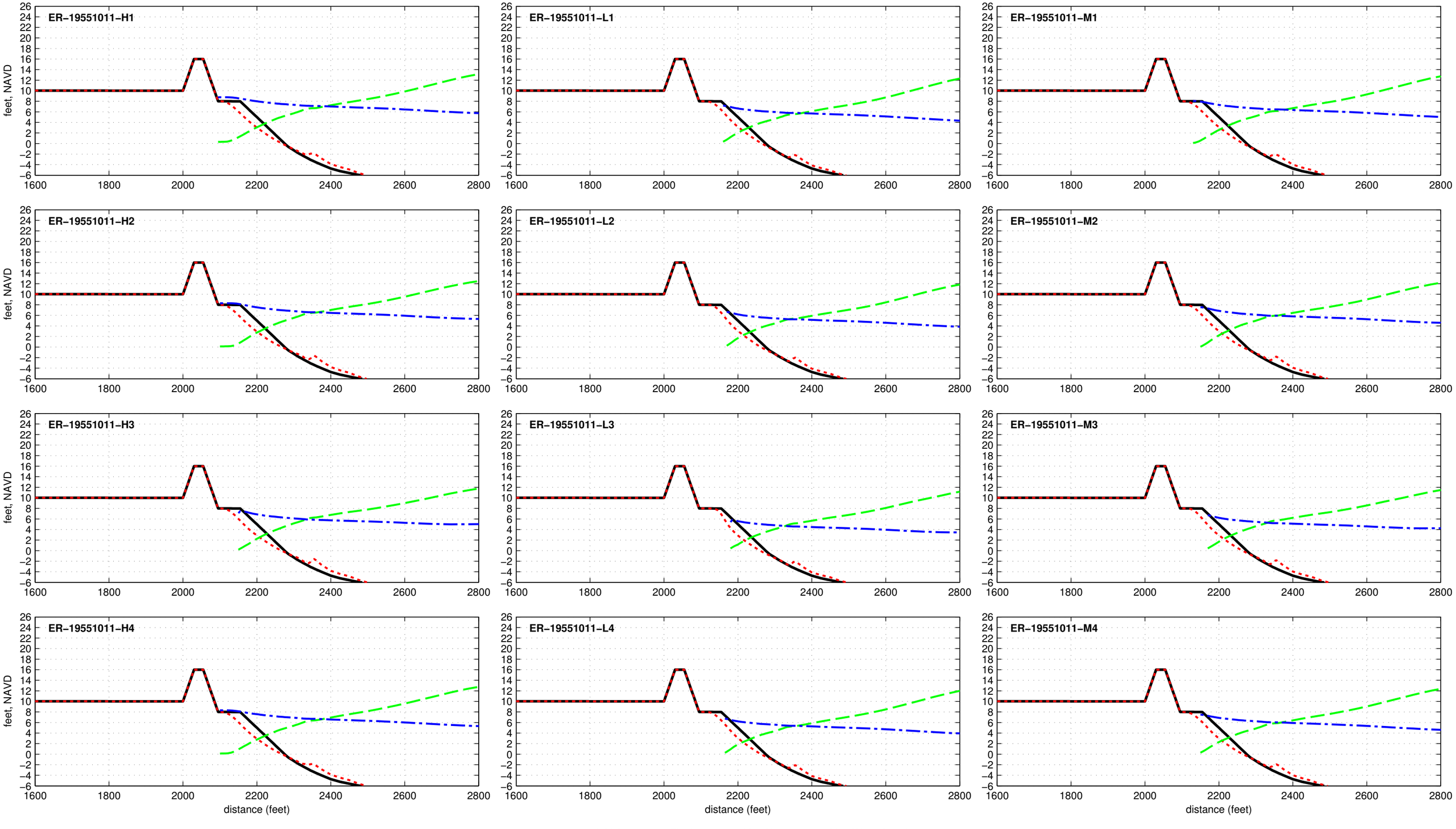

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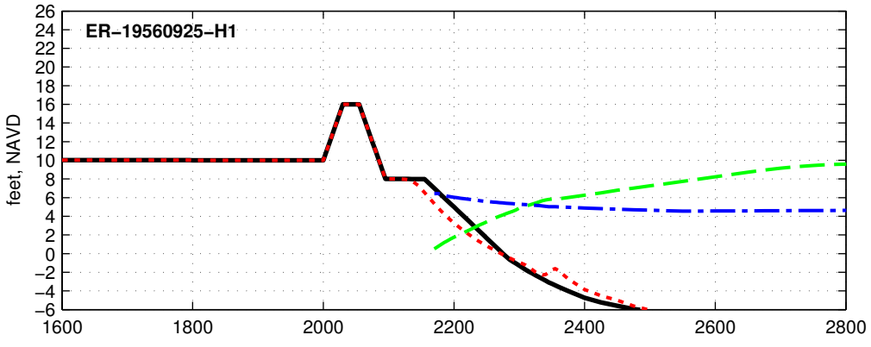
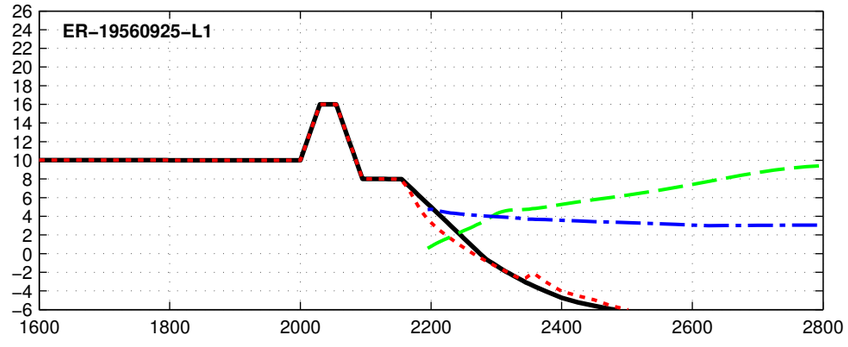
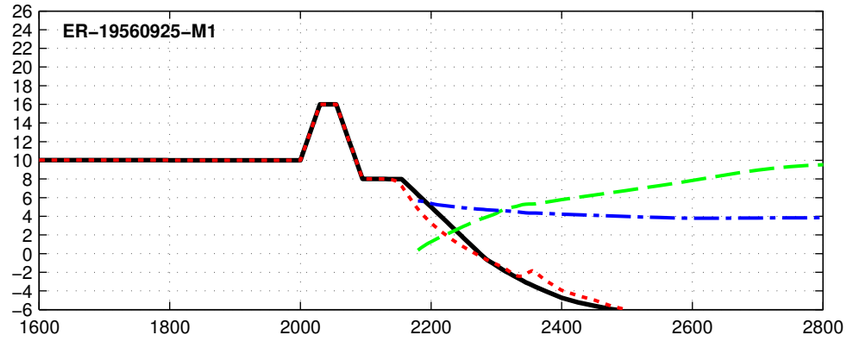
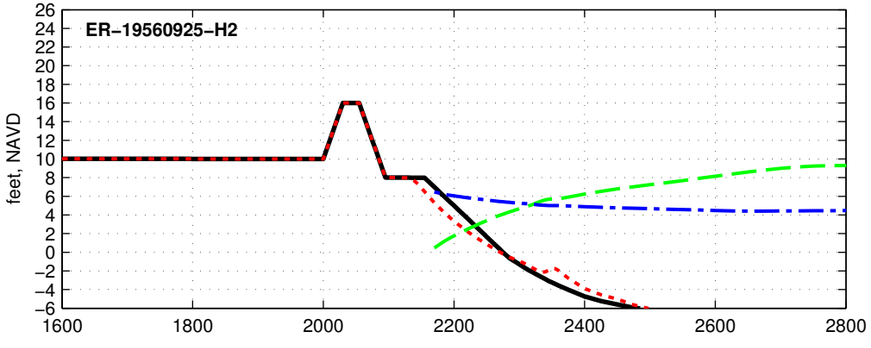
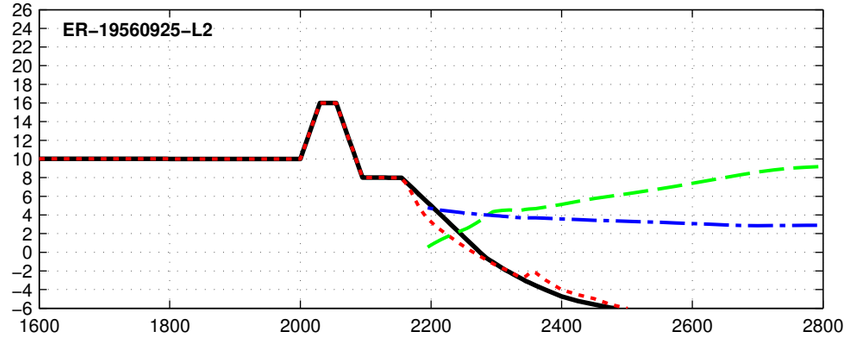
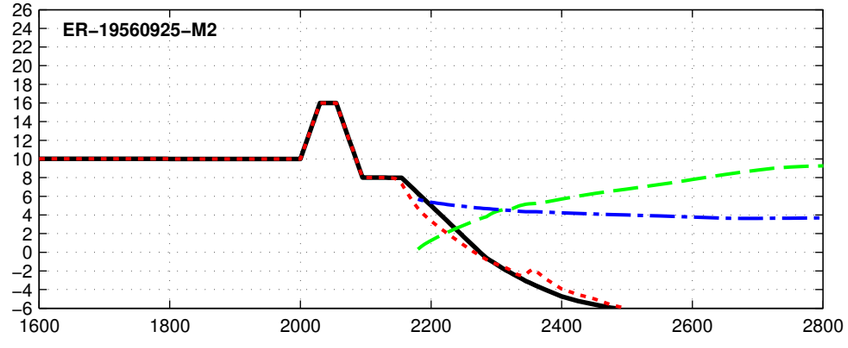


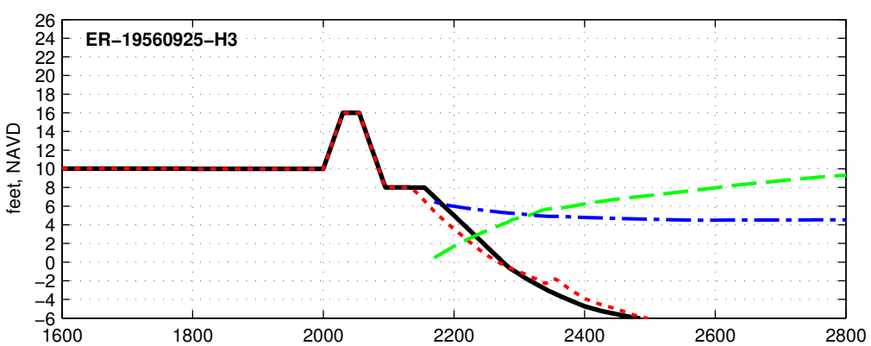


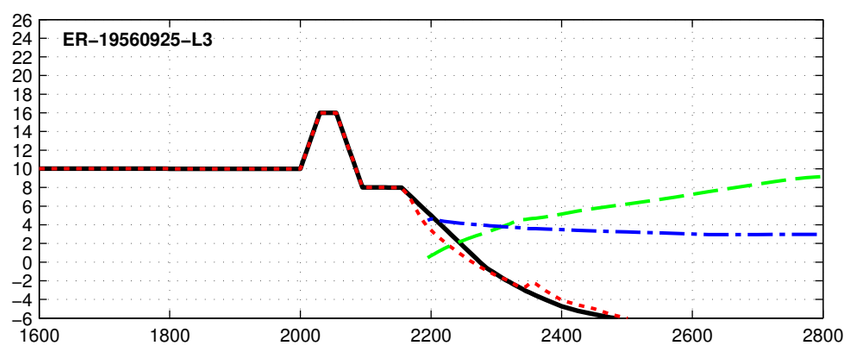


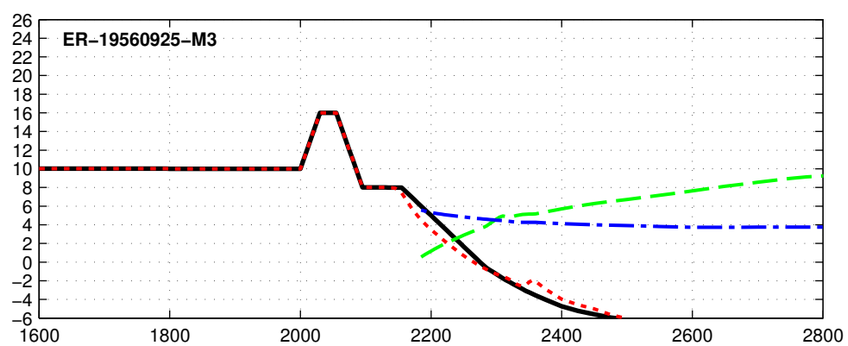


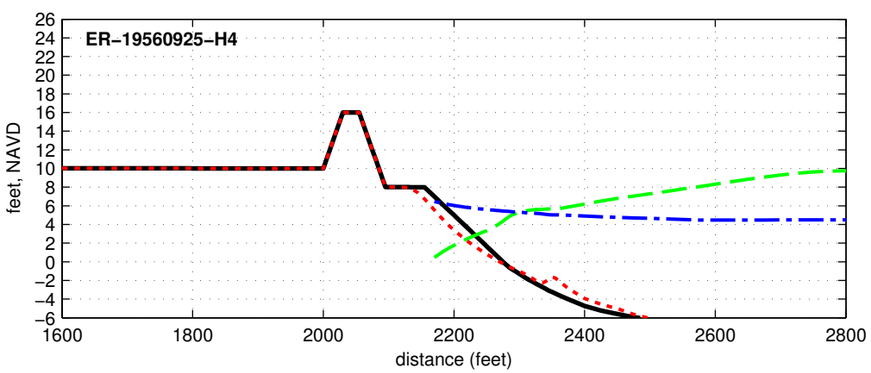


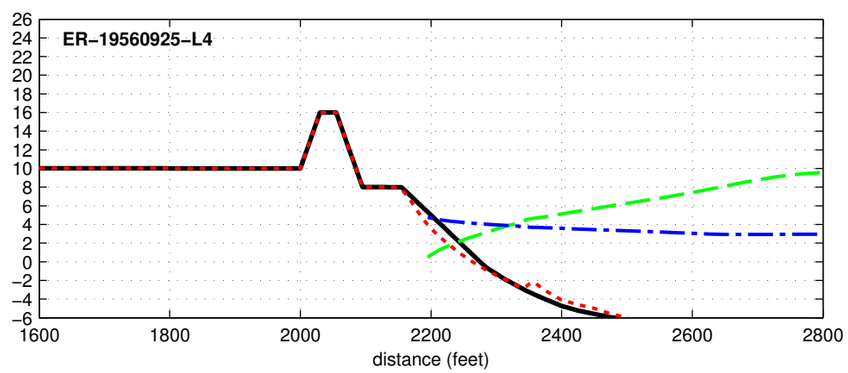


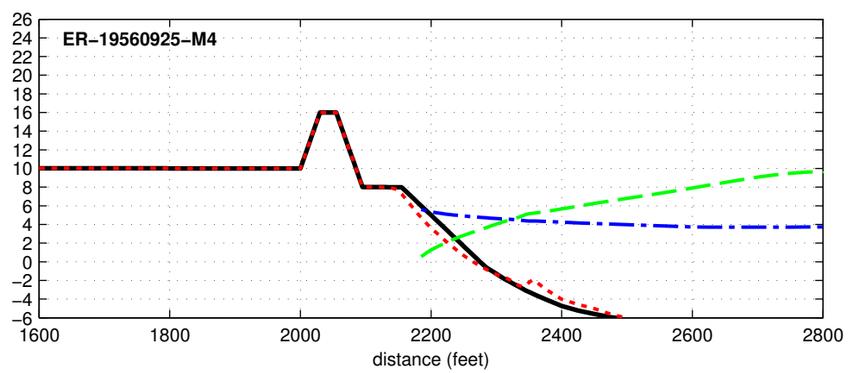






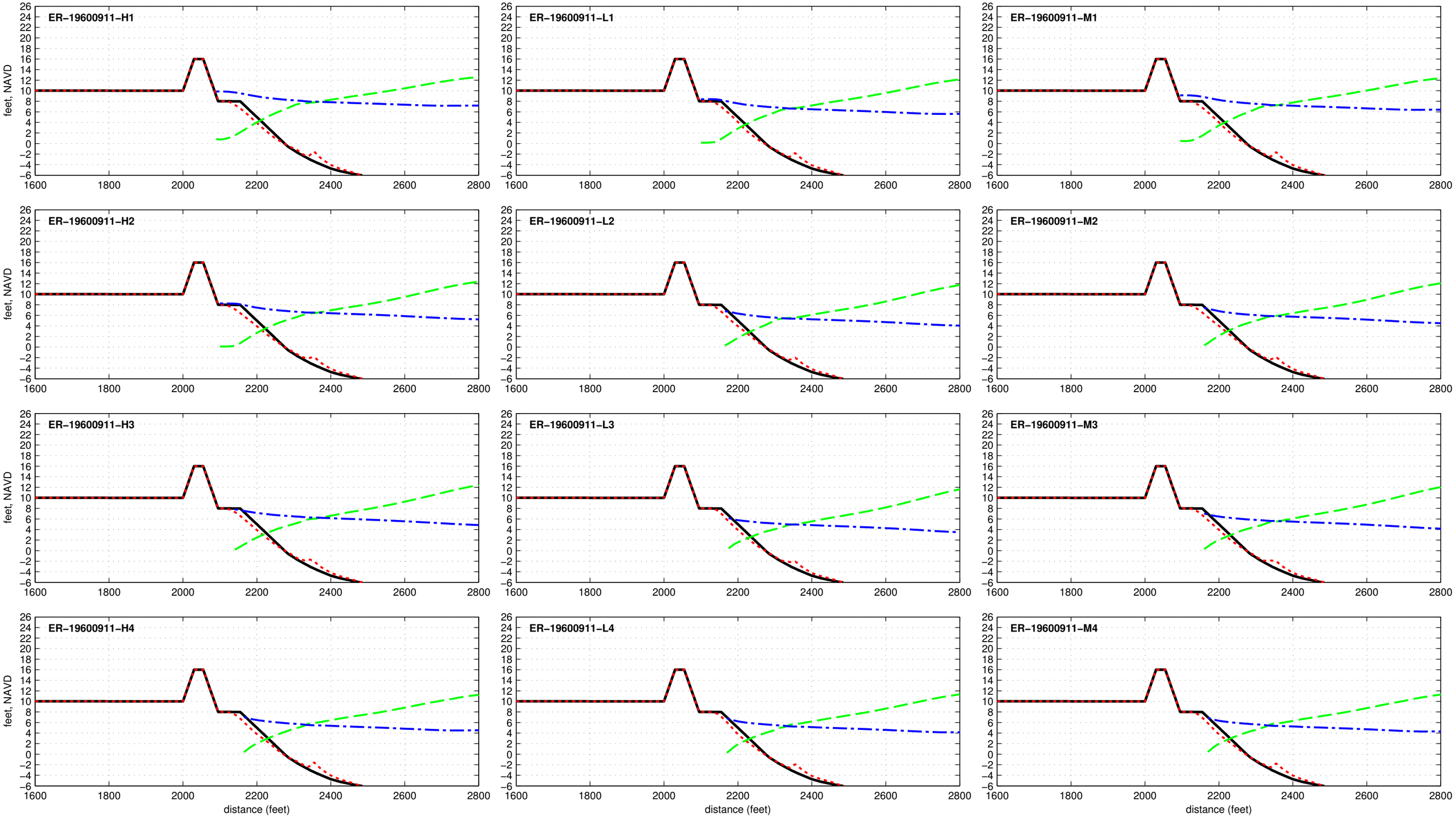




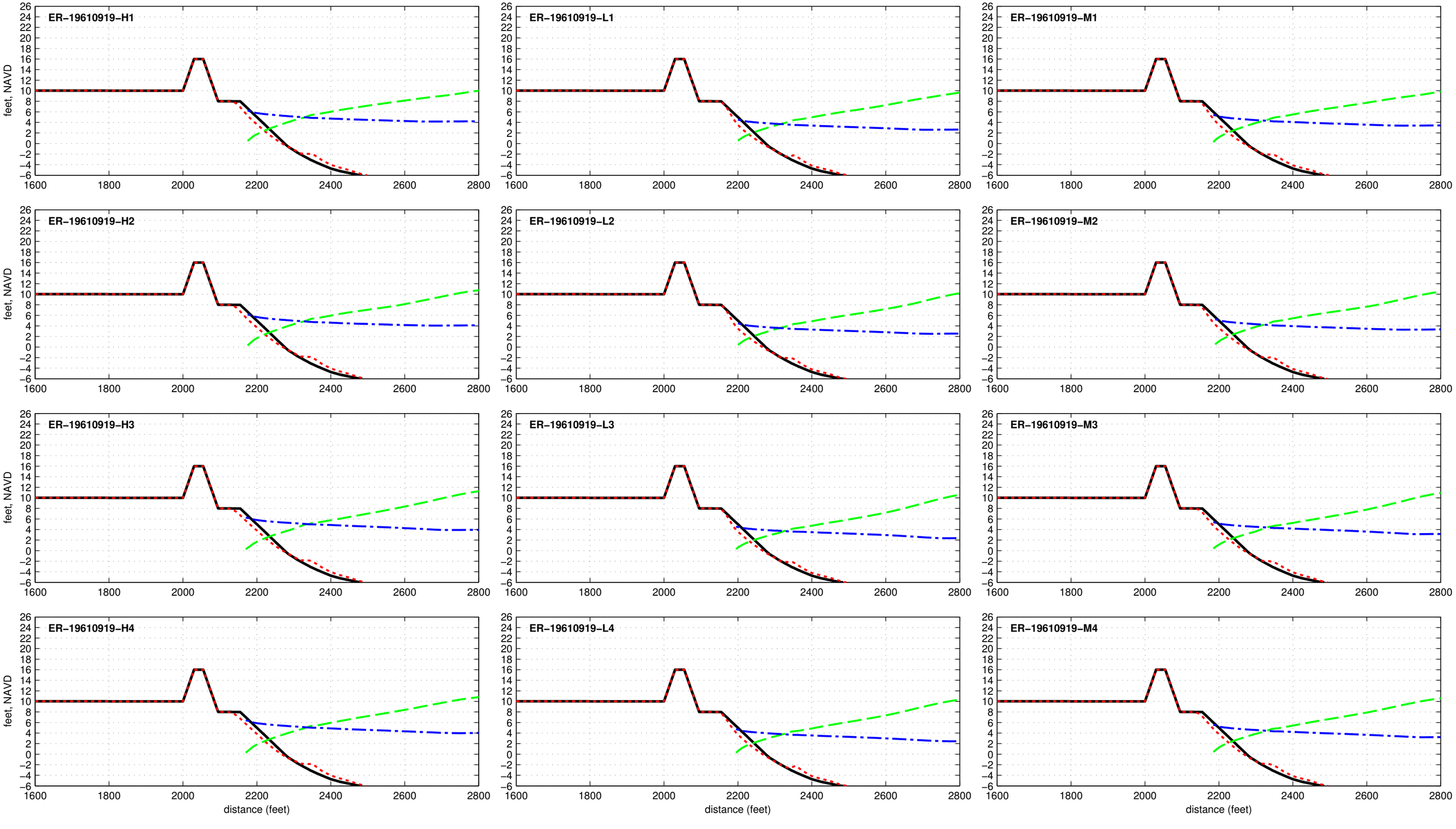


Initial Final Max Hs Max Water Level

Dune R2T1-16.0-025-060 - ER-19600911



Dune R2T1-16.0-025-060 - ER-19610919



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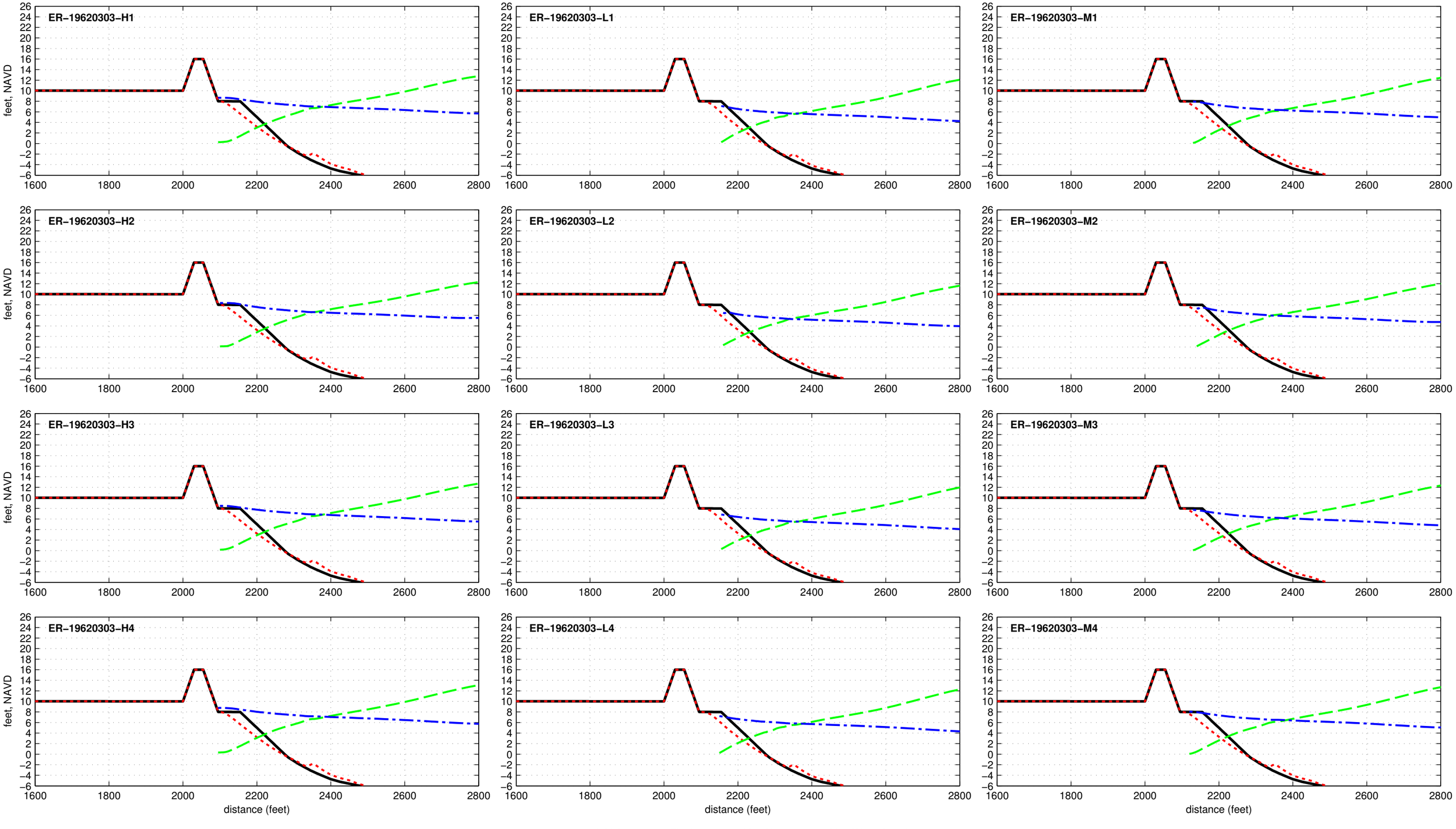
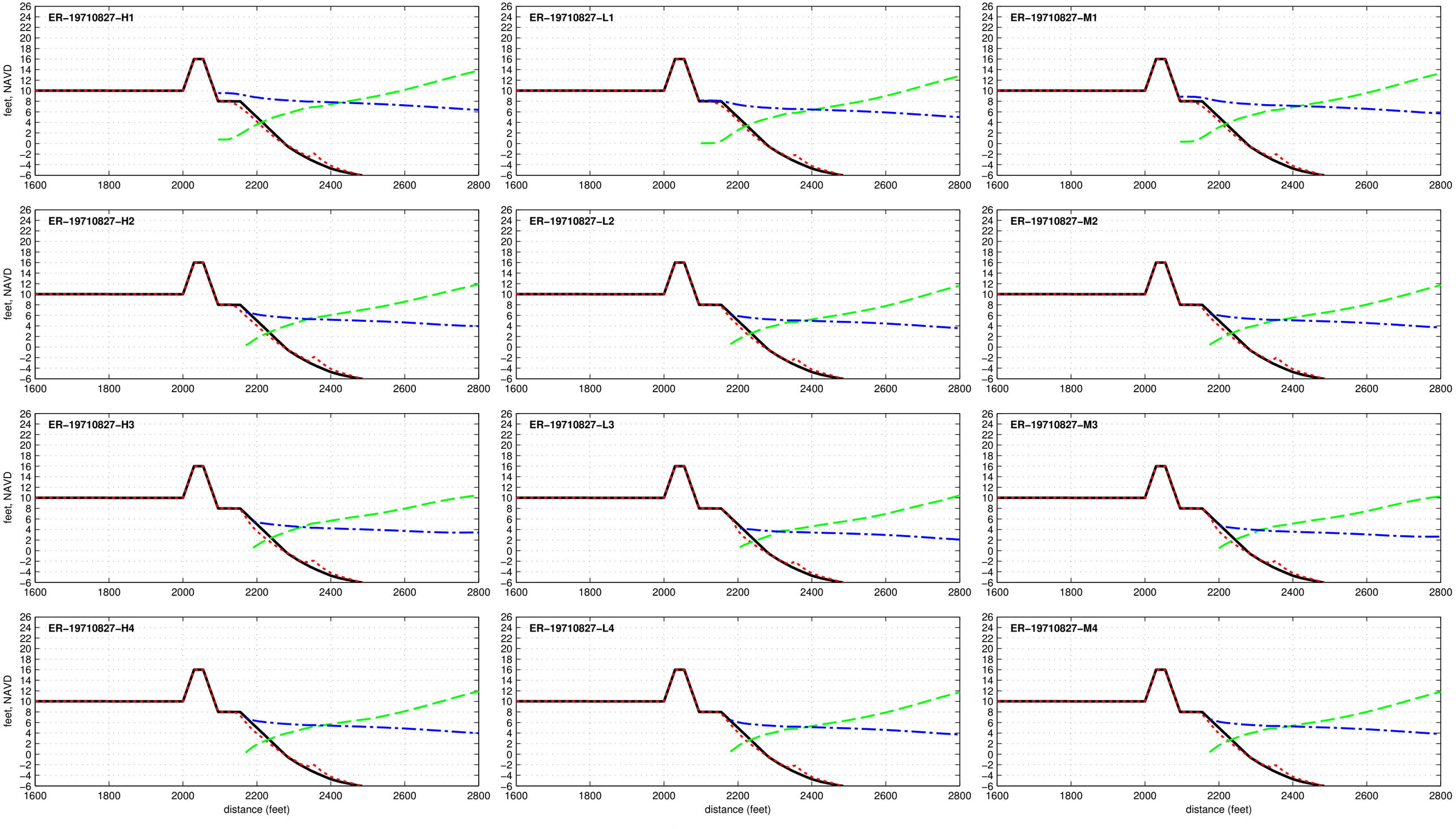


Dune R2T1-16.0-025-060 - ER-19710827



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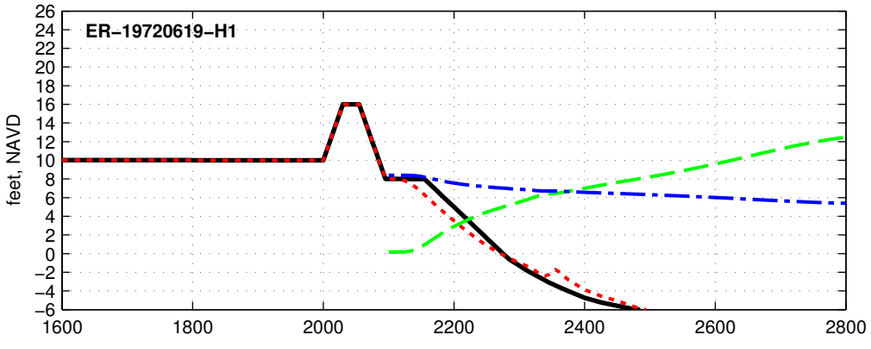
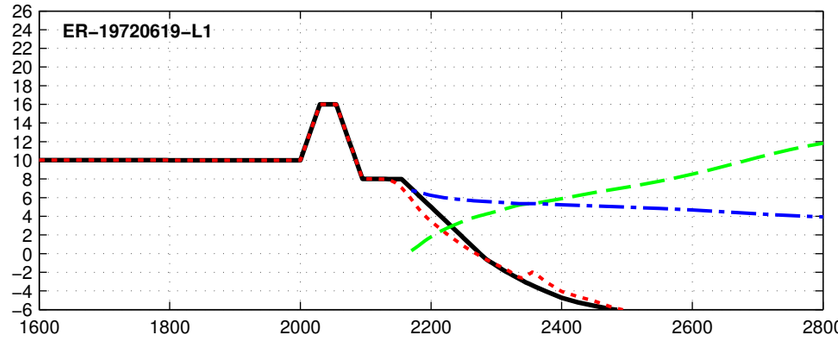
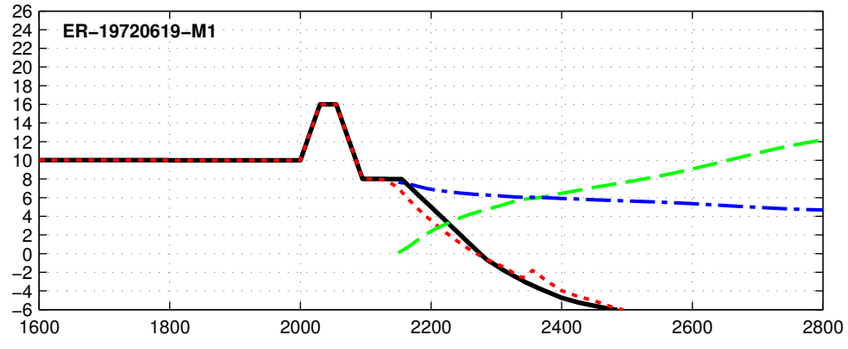
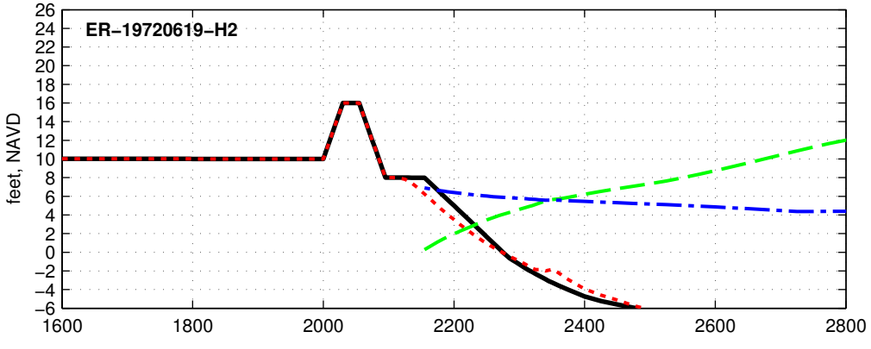
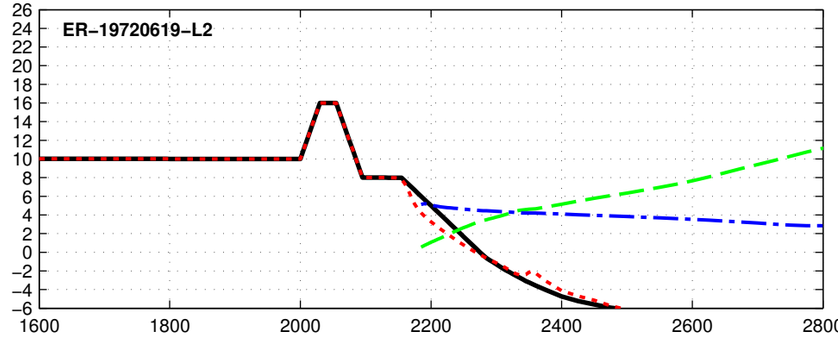
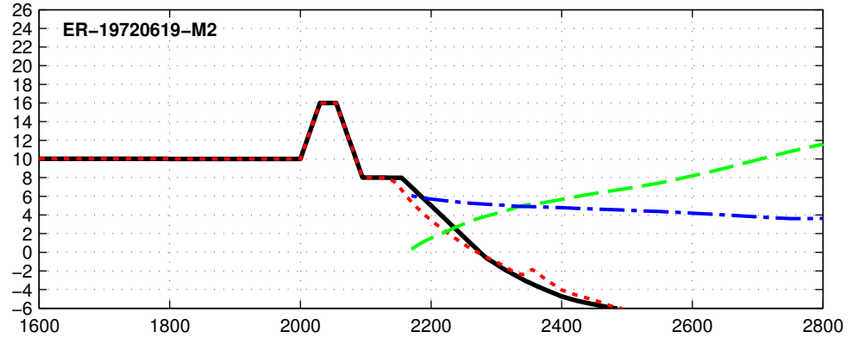


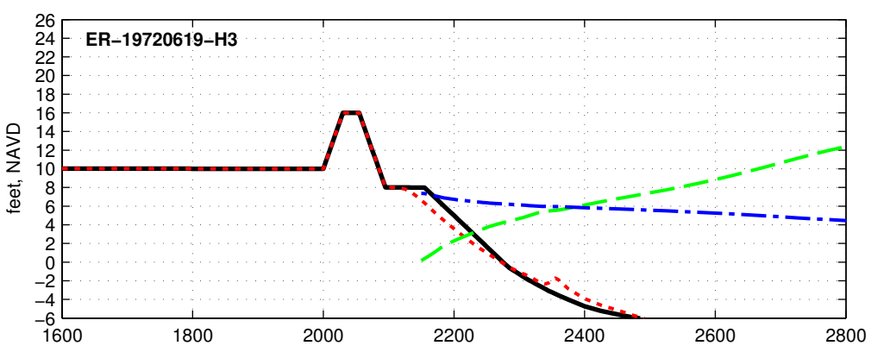


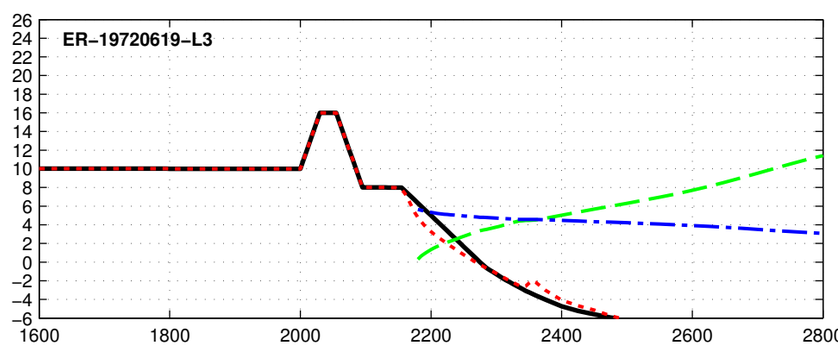


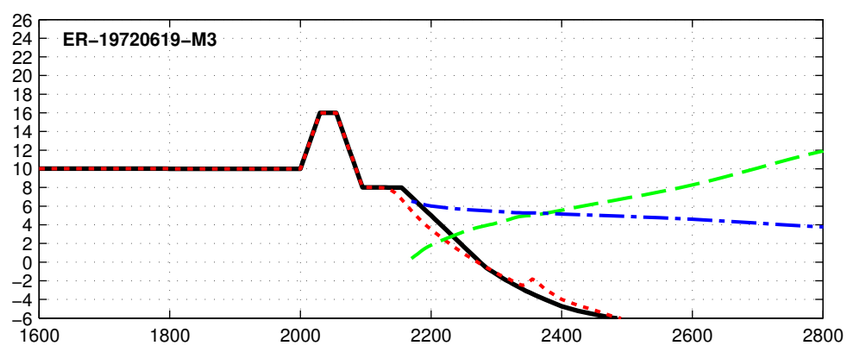


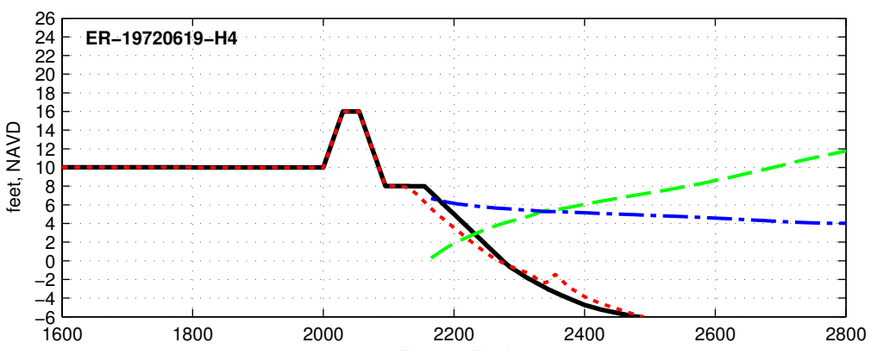


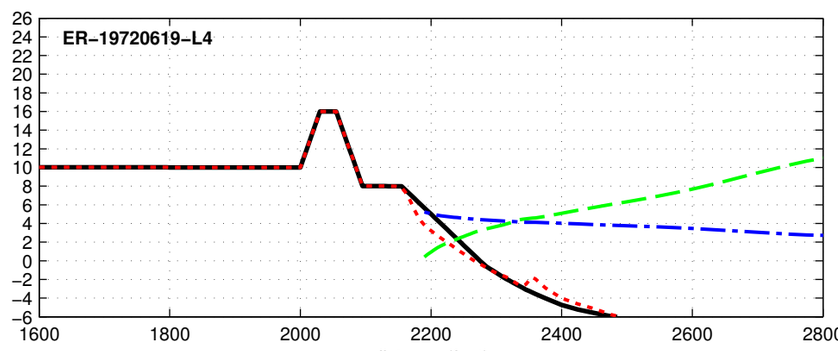


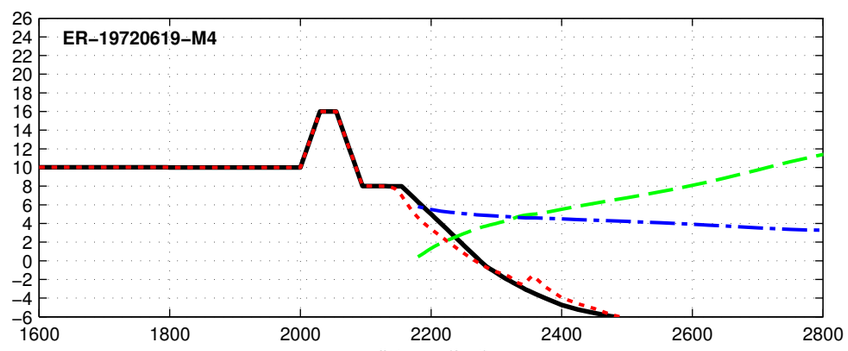






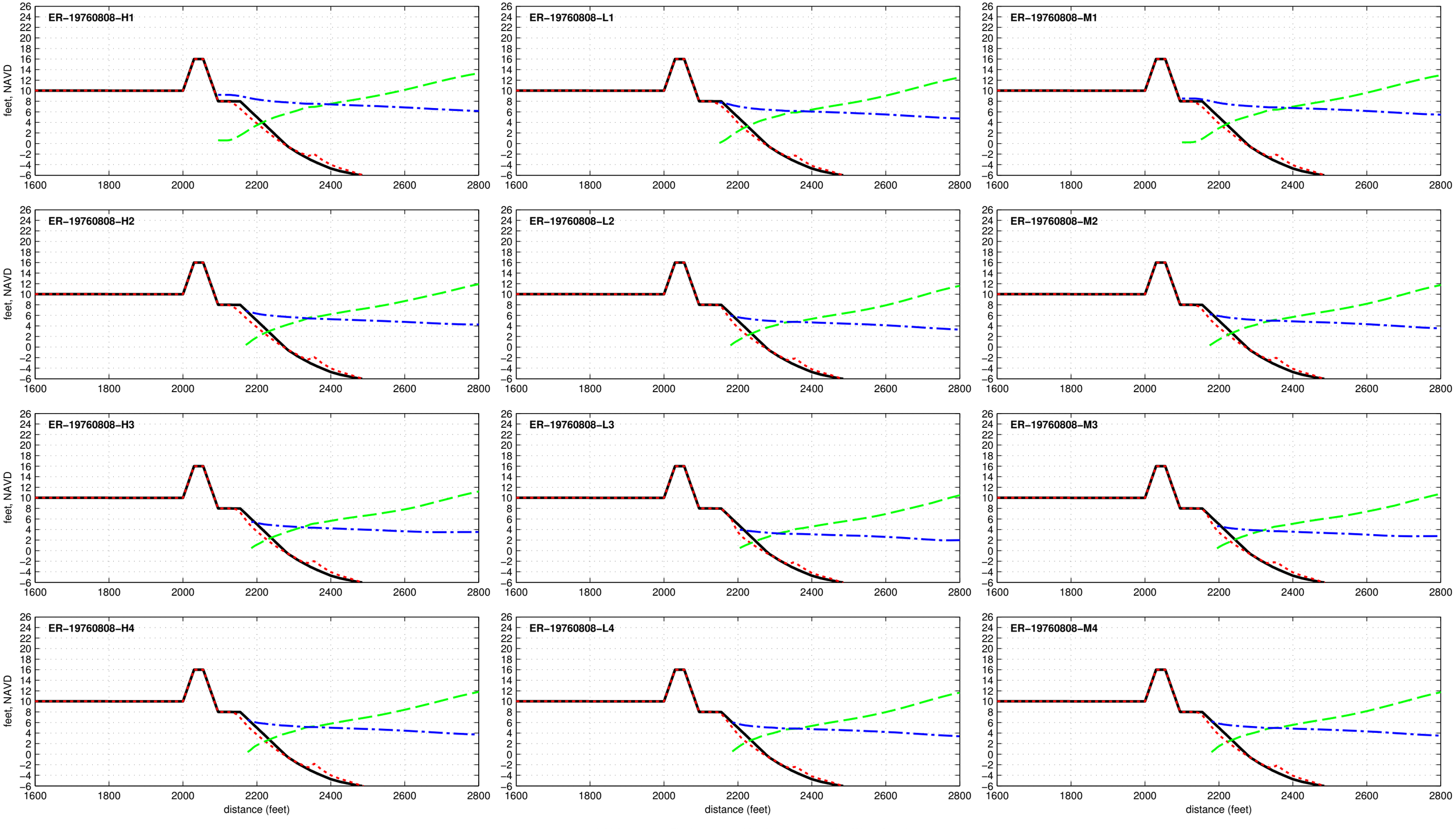




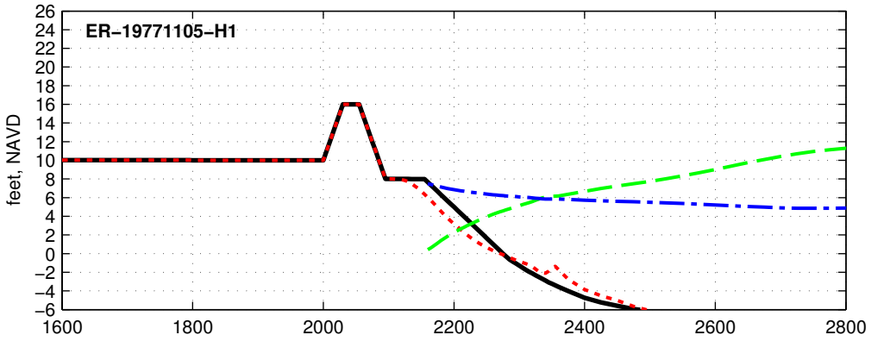


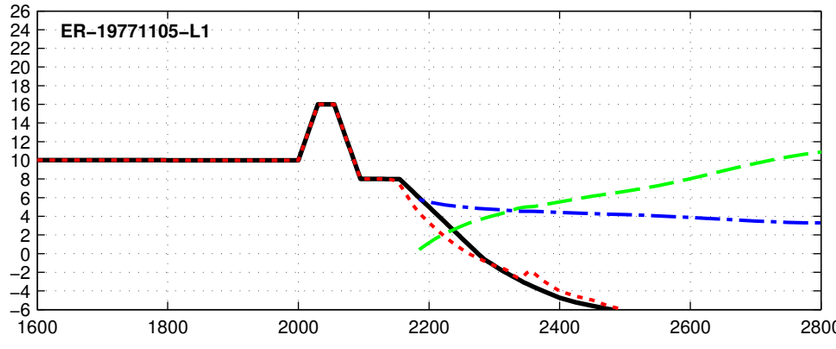
Initial Final Max Hs Max Water Level

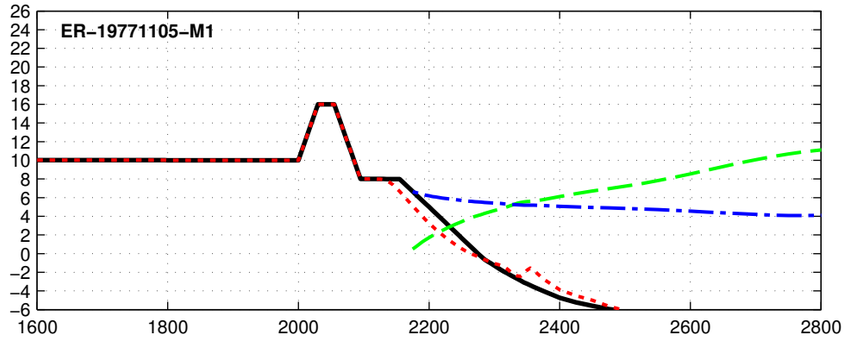
Dune R2T1-16.0-025-060 - ER-19760808

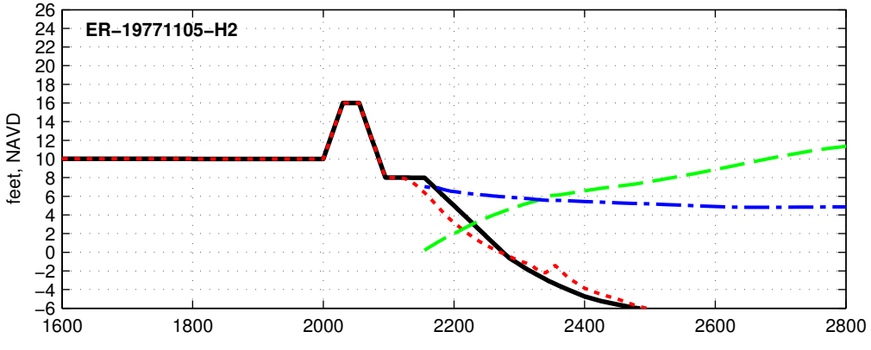


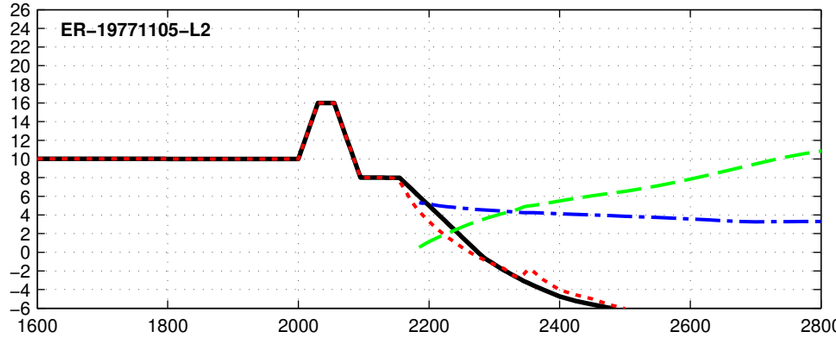
Dune R2T1-16.0-025-060 - ER-19771105

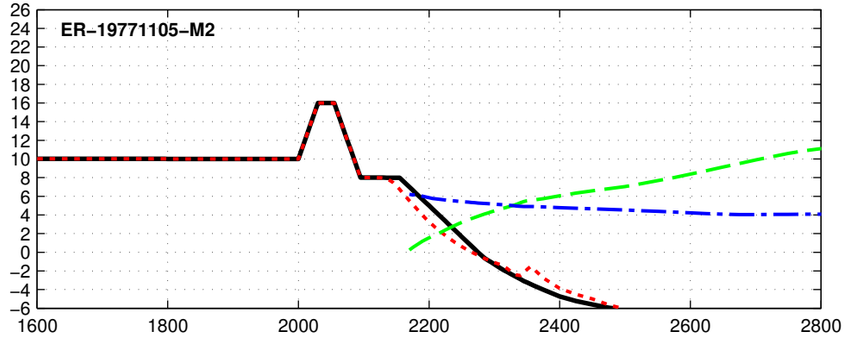


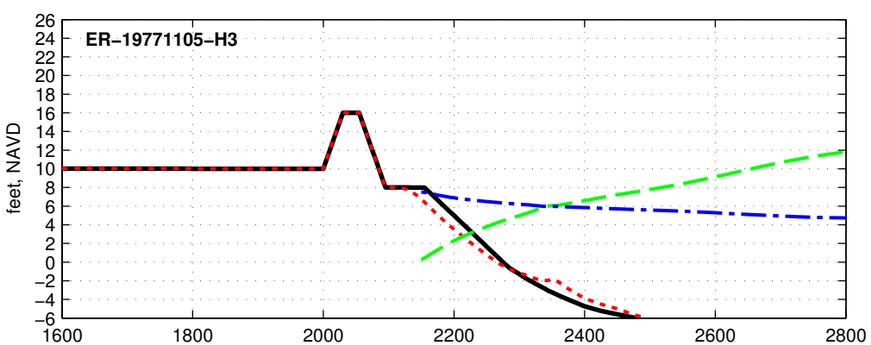


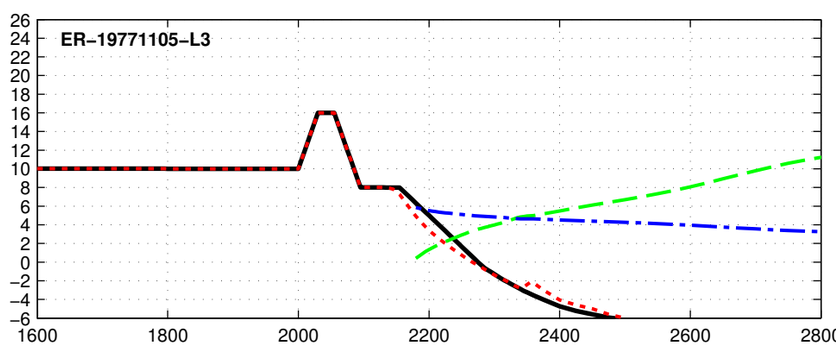


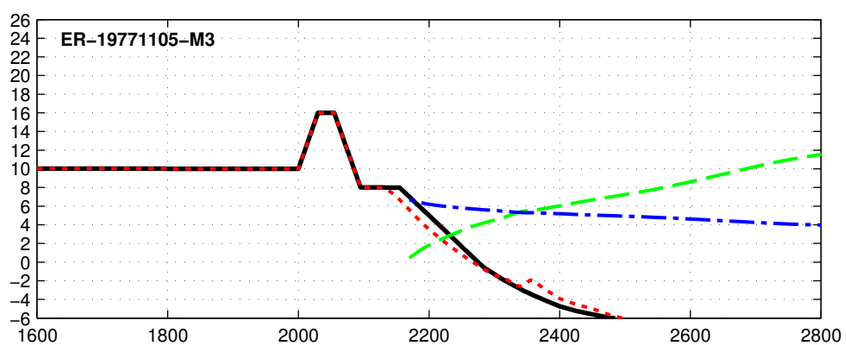


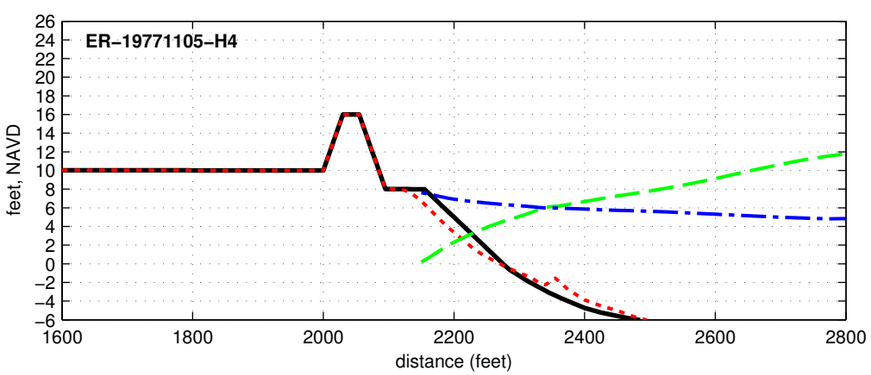


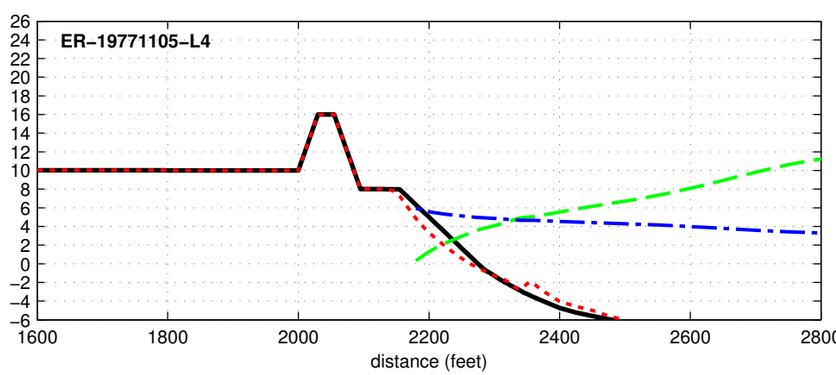


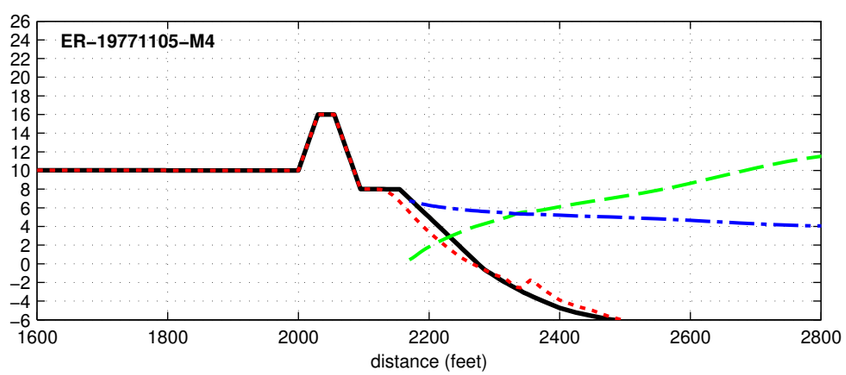






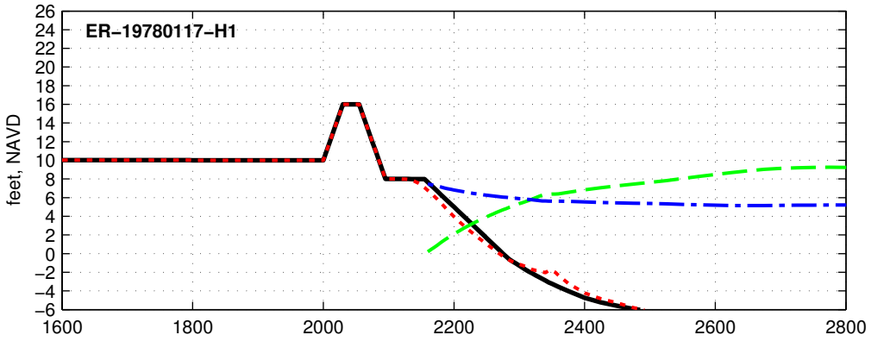


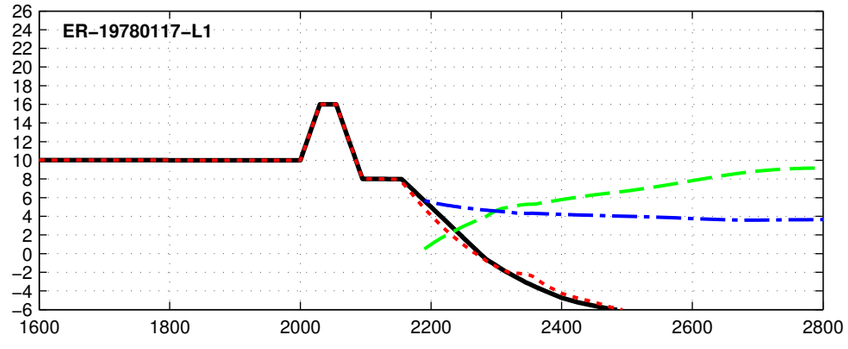


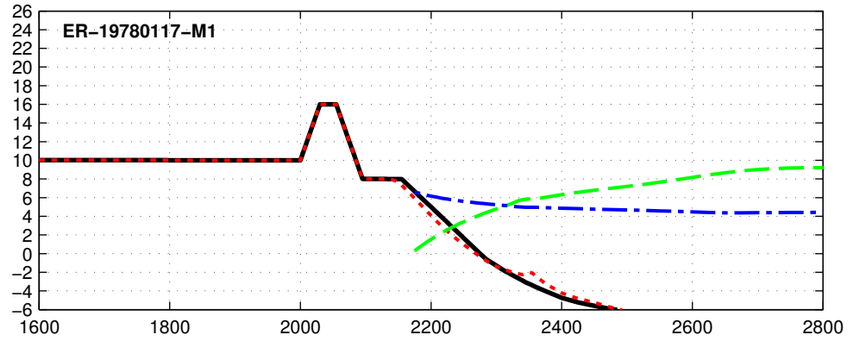


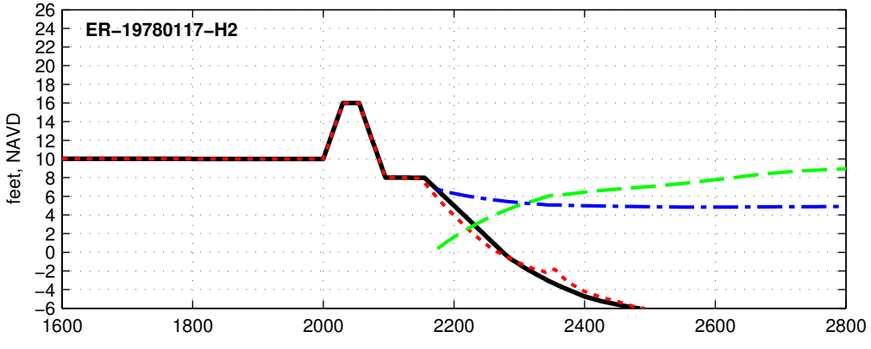
Initial Final Max Hs Max Water Level

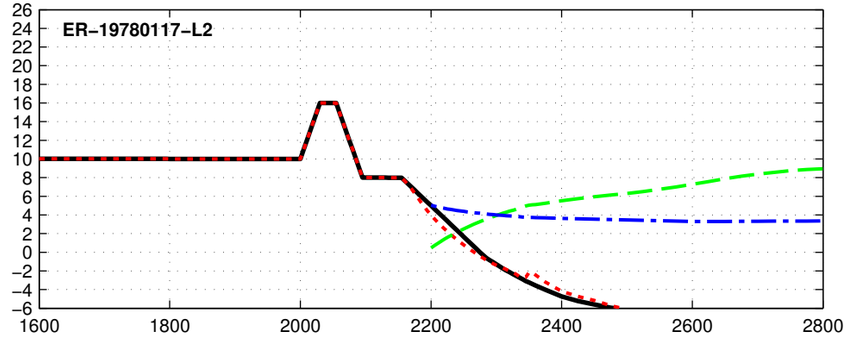
Dune R2T1-16.0-025-060 - ER-19780117

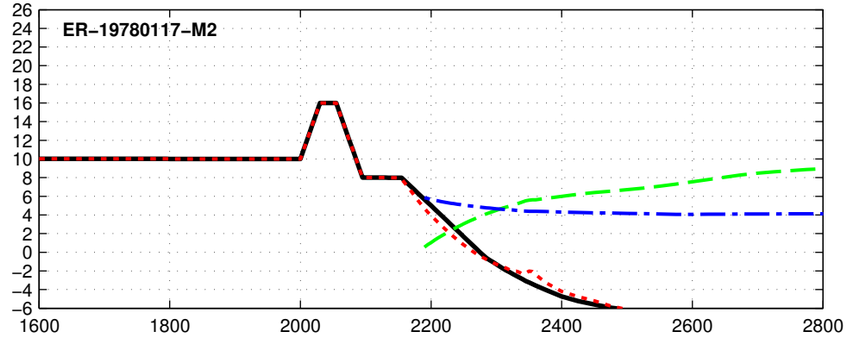


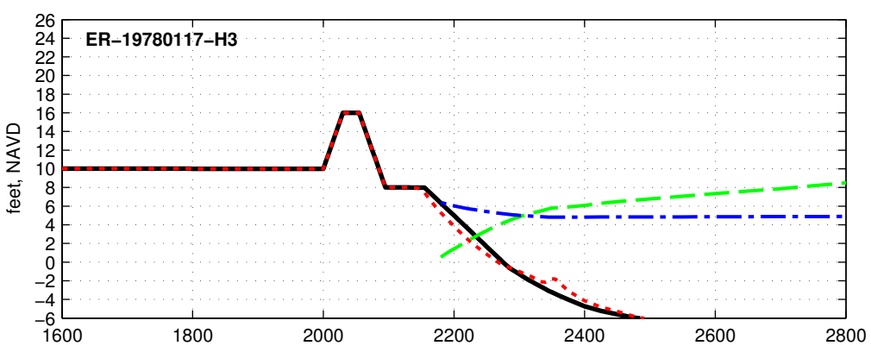


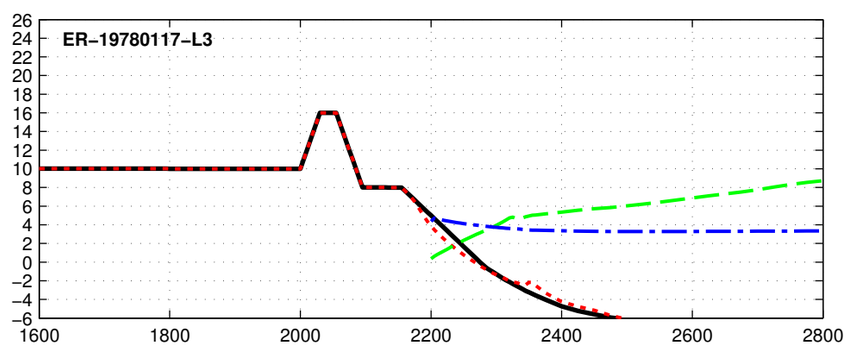


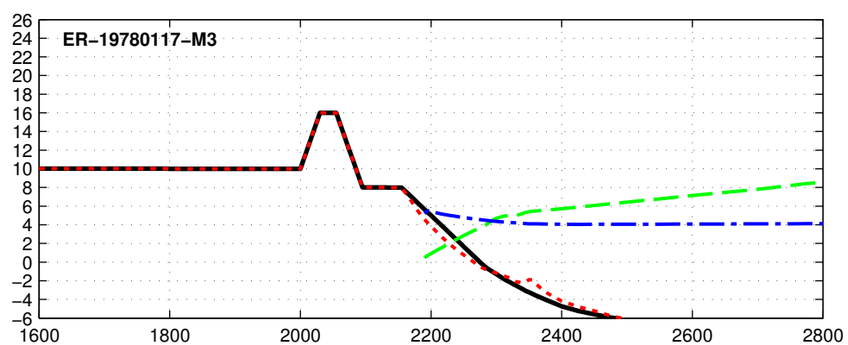


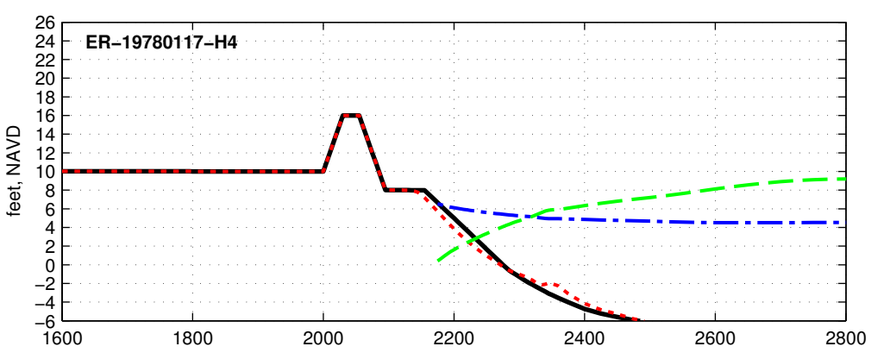


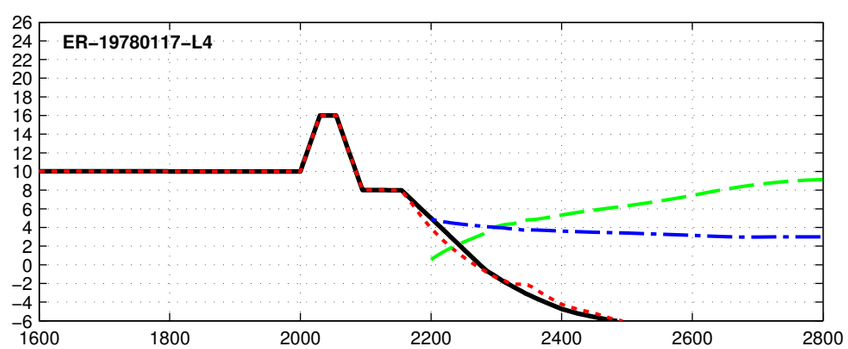


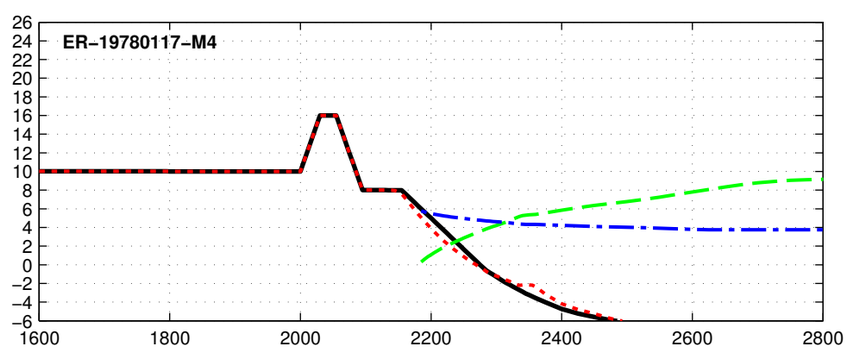






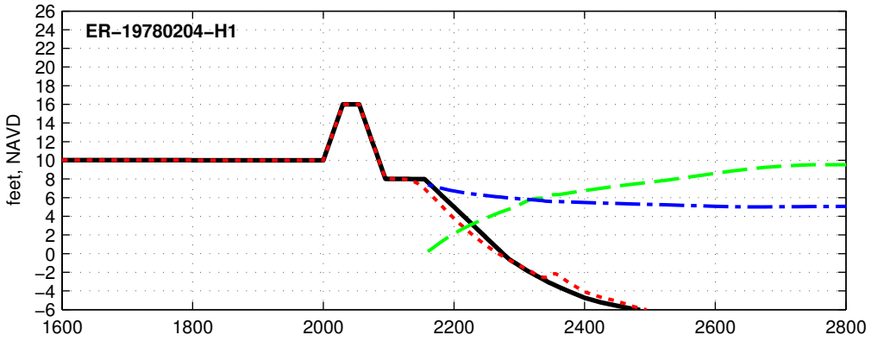


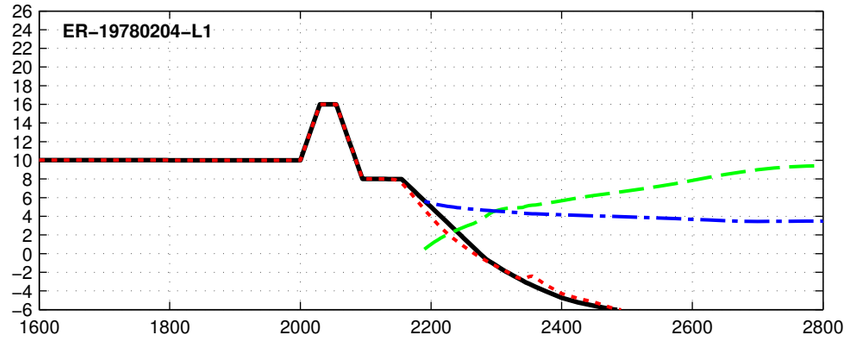


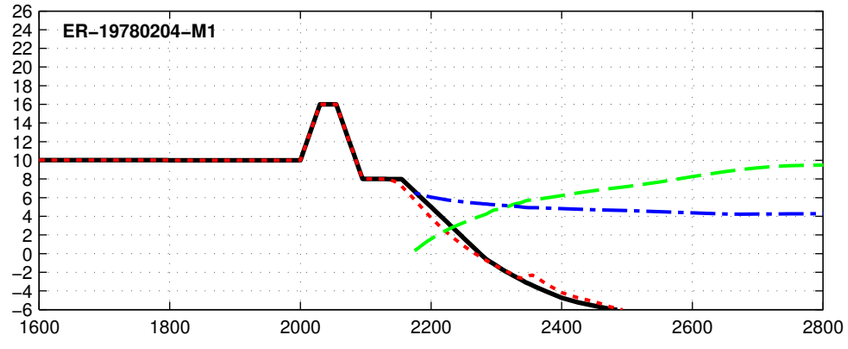


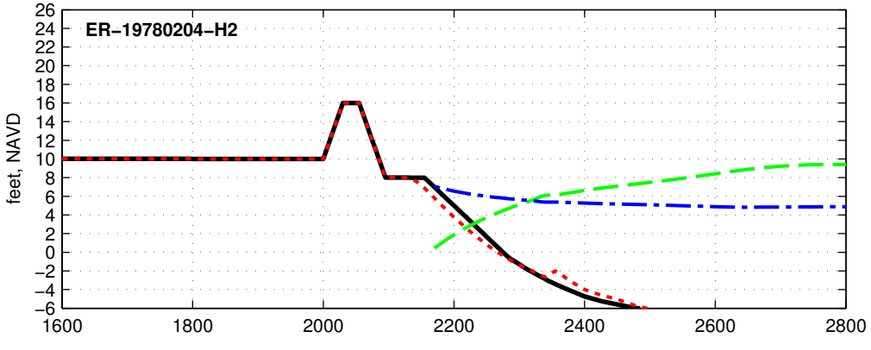
Initial Final Max Hs Max Water Level

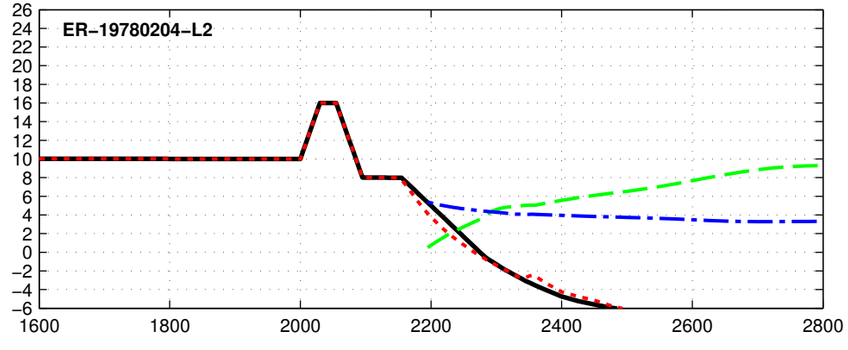
Dune R2T1-16.0-025-060 - ER-19780204

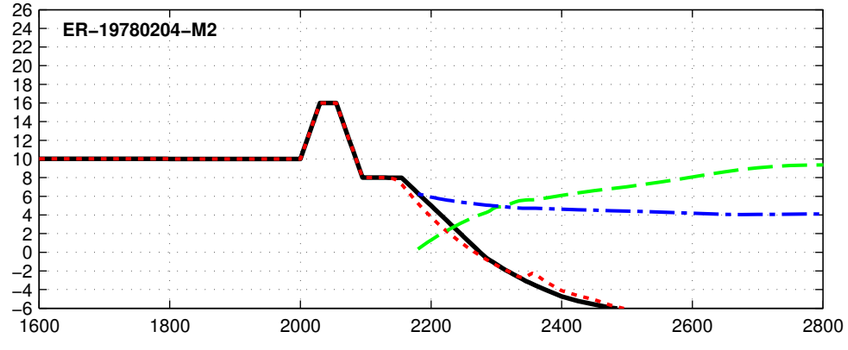


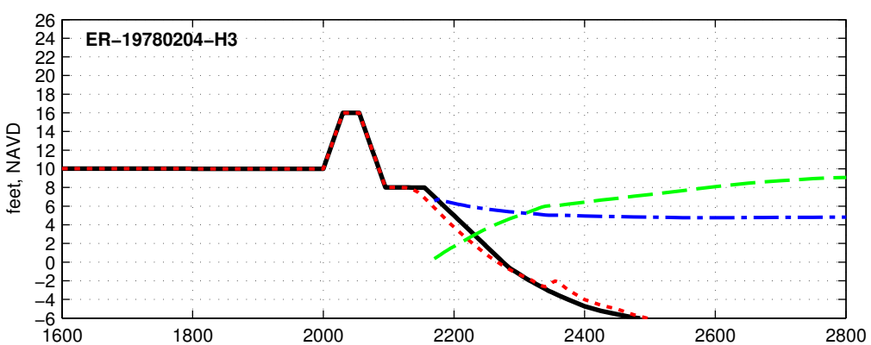


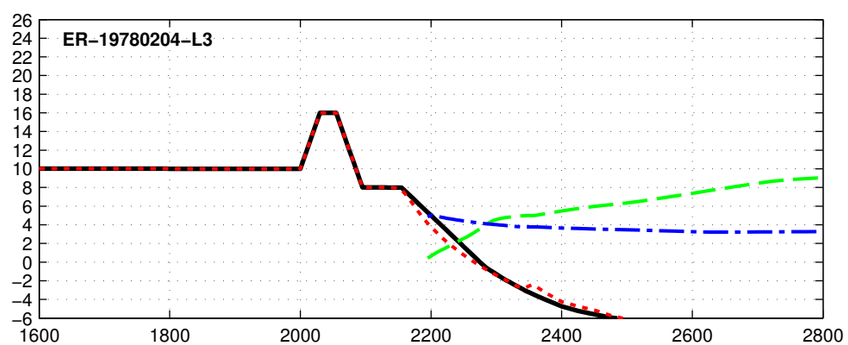


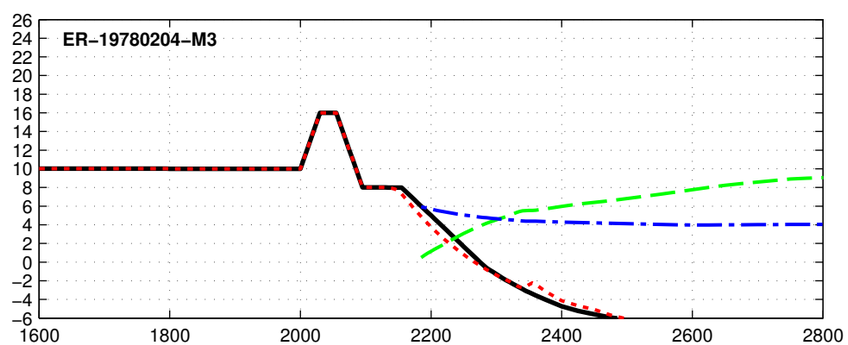


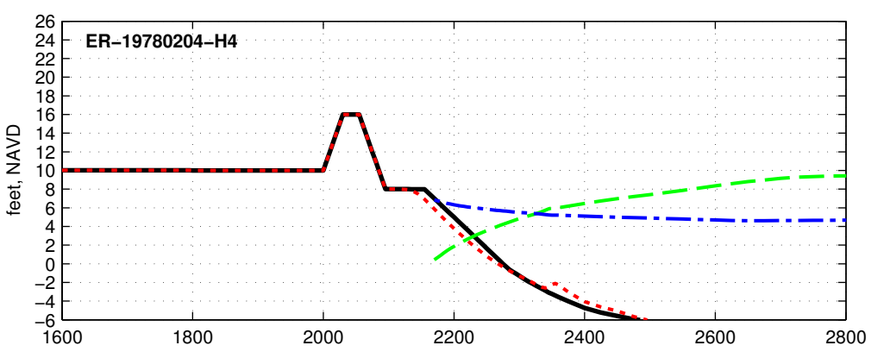


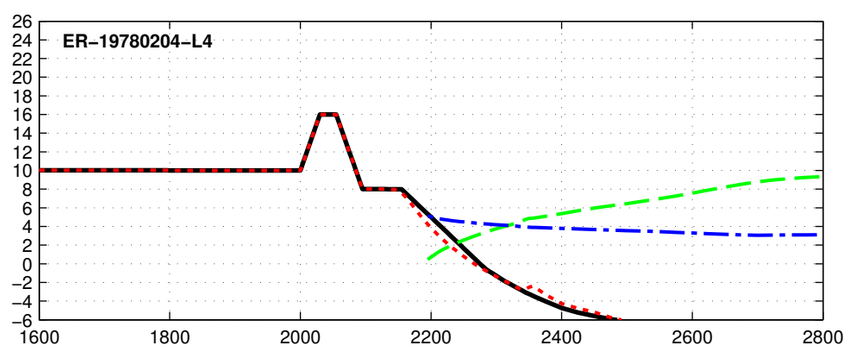


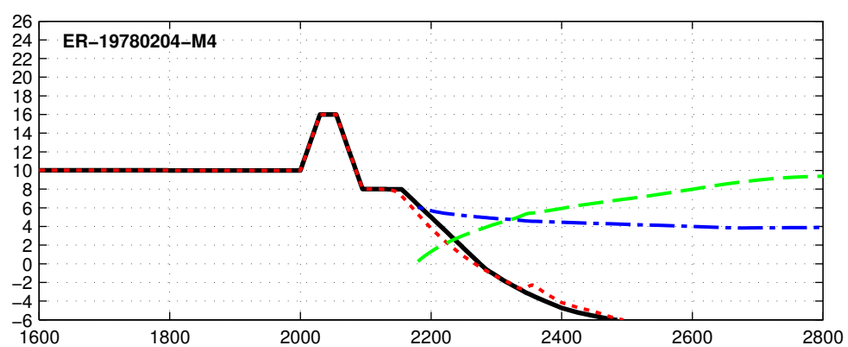






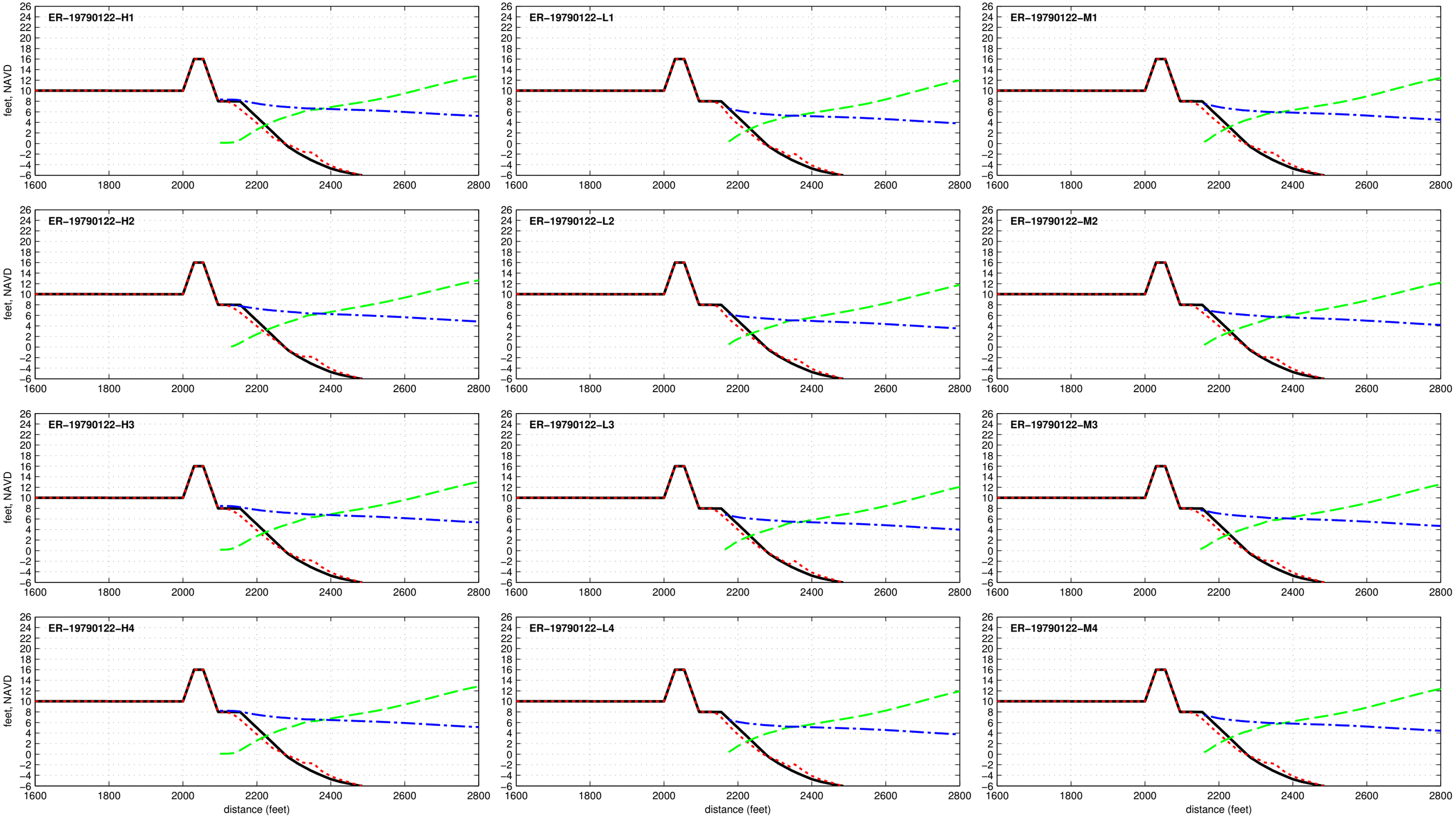




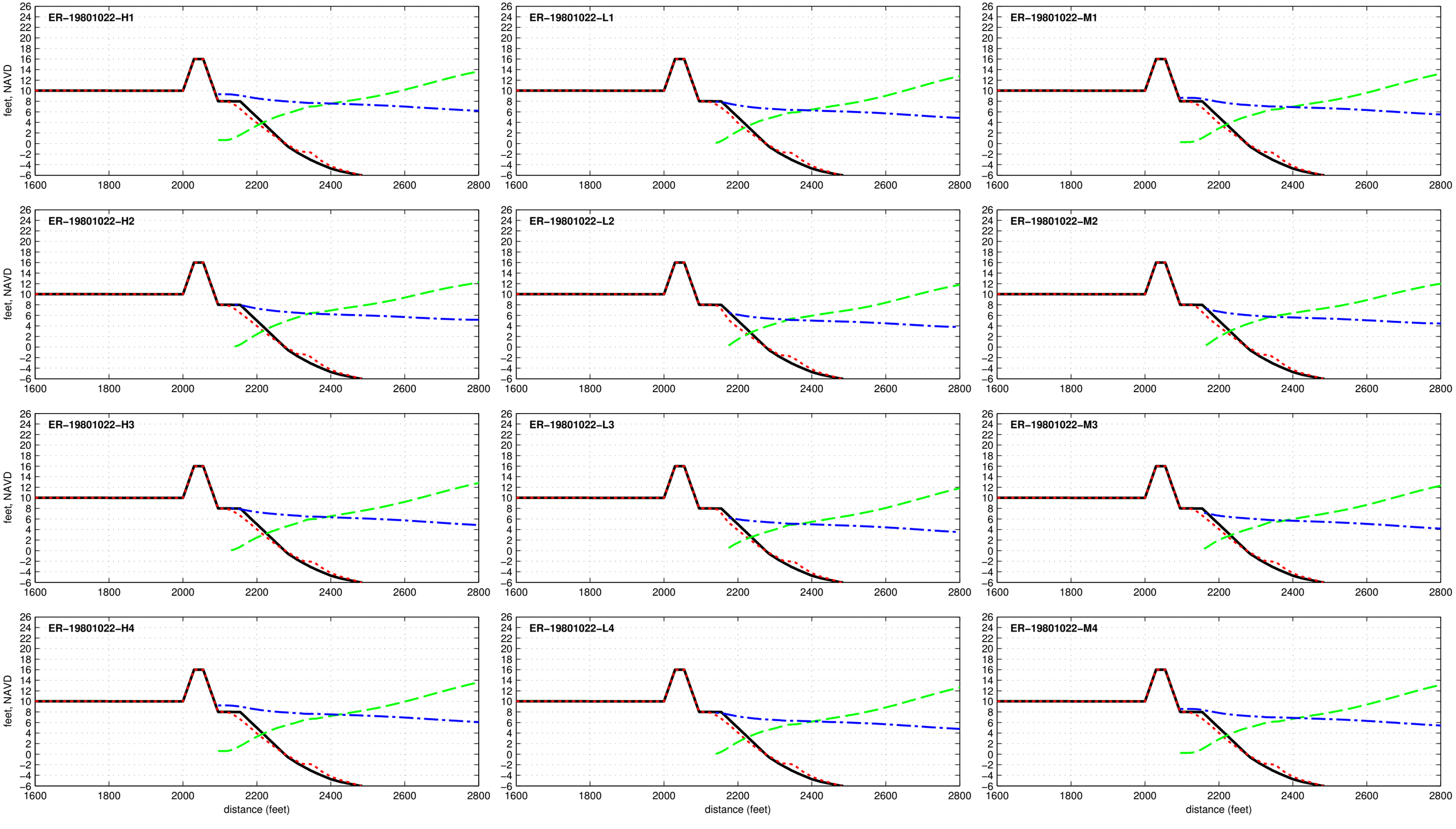


Initial Final Max Hs Max Water Level

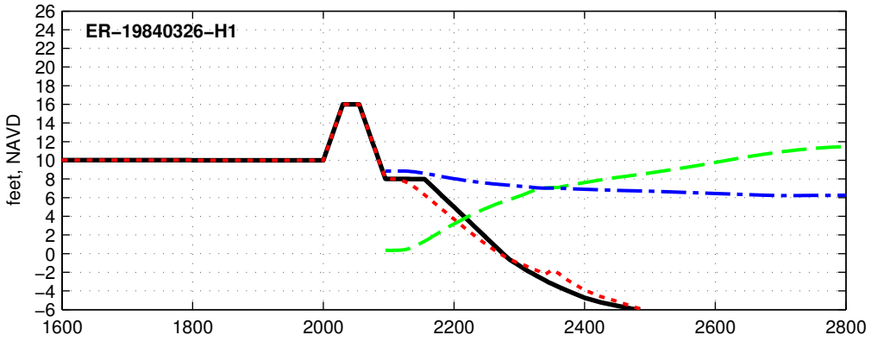
Dune R2T1-16.0-025-060 - ER-19790122

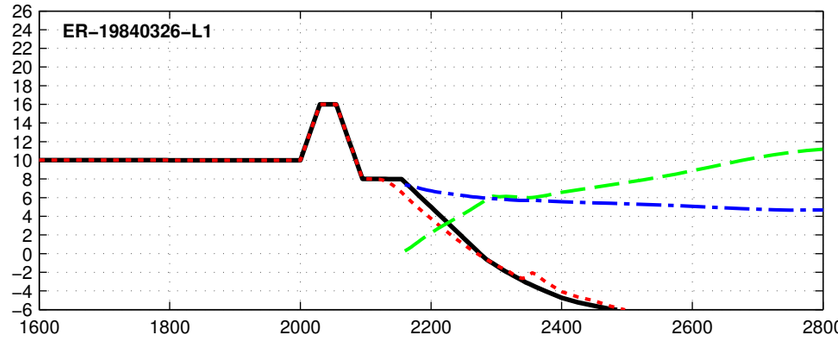


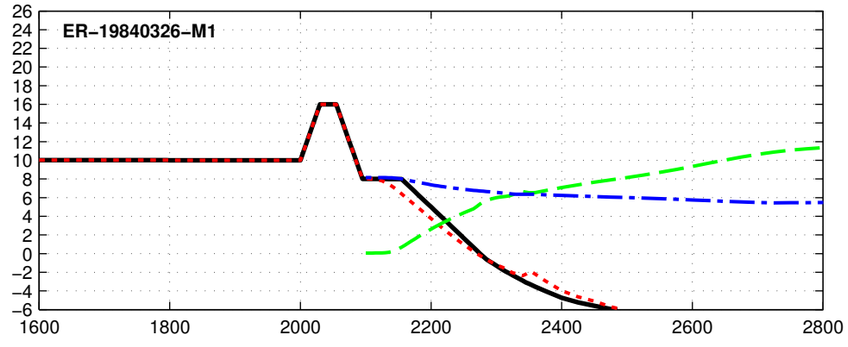
Dune R2T1-16.0-025-060 - ER-19801022

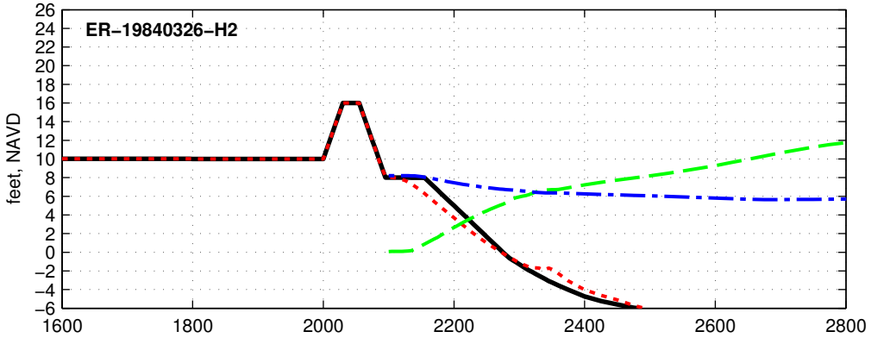


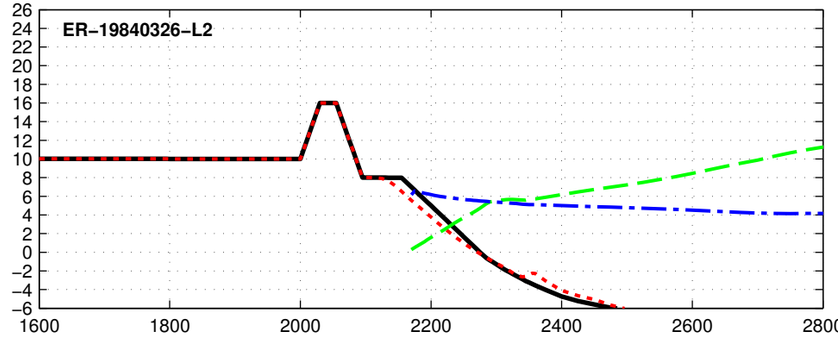
Dune R2T1-16.0-025-060 - ER-19840326

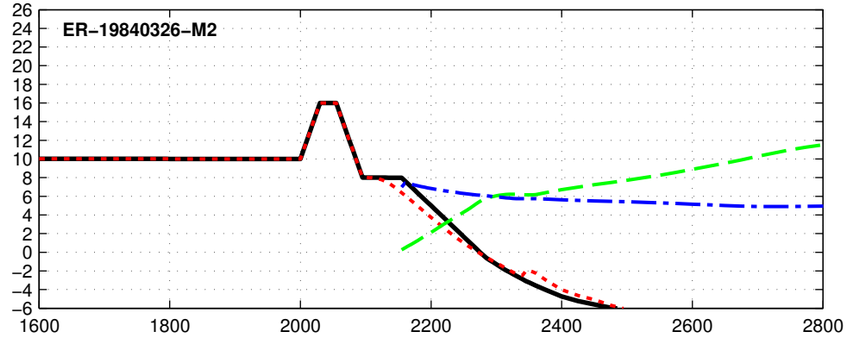


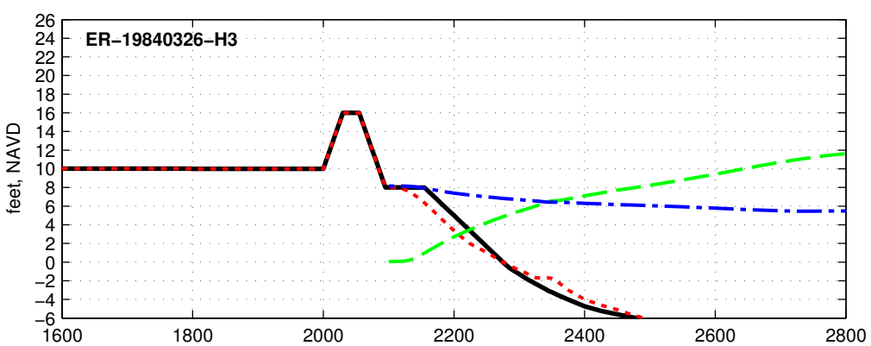


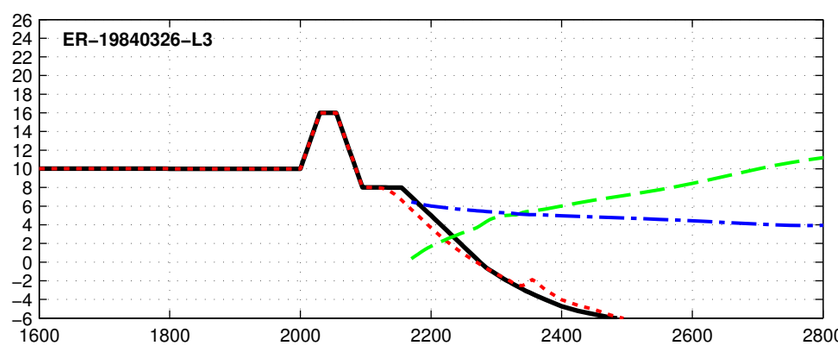


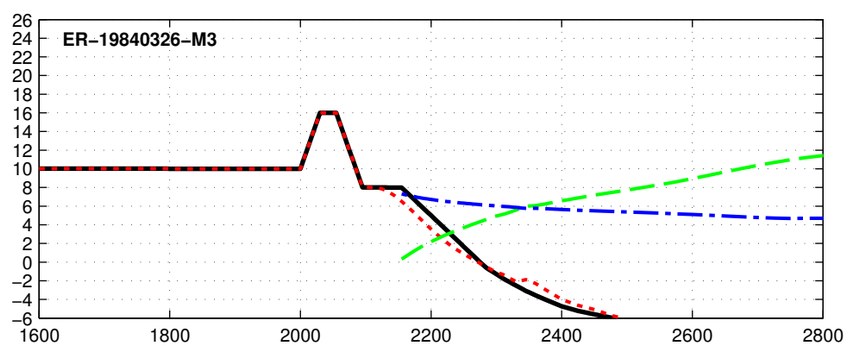


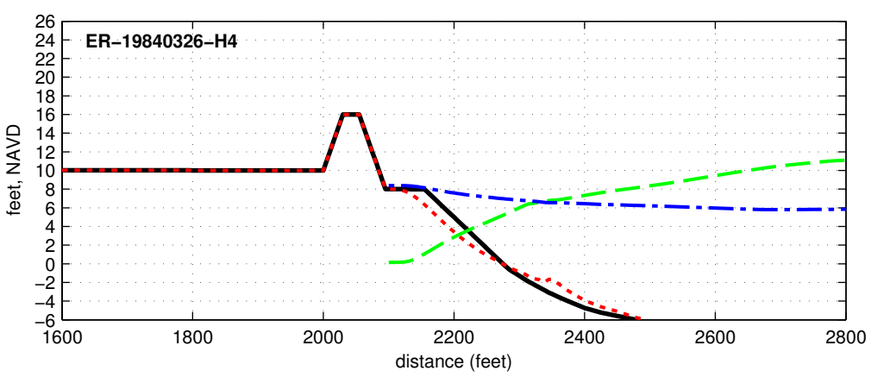


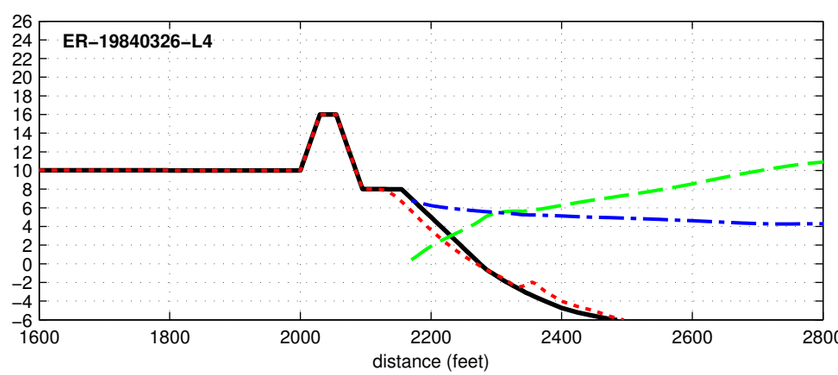


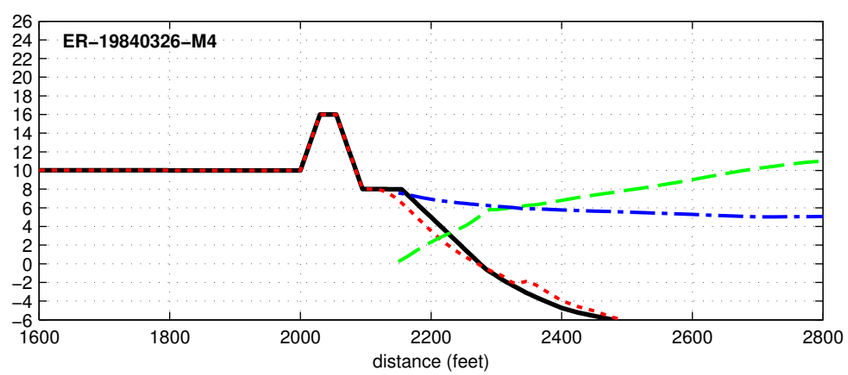






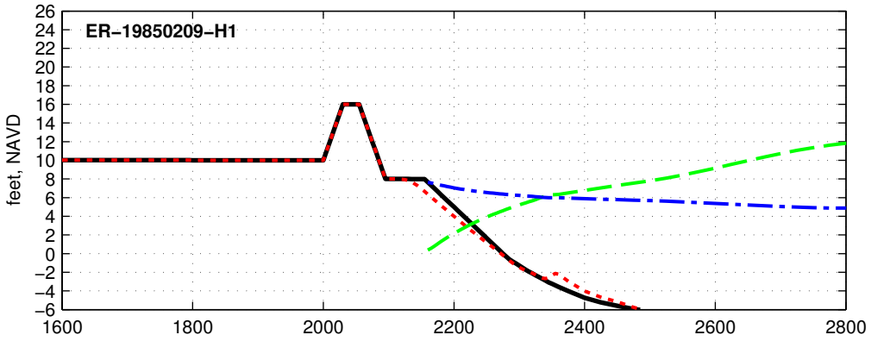


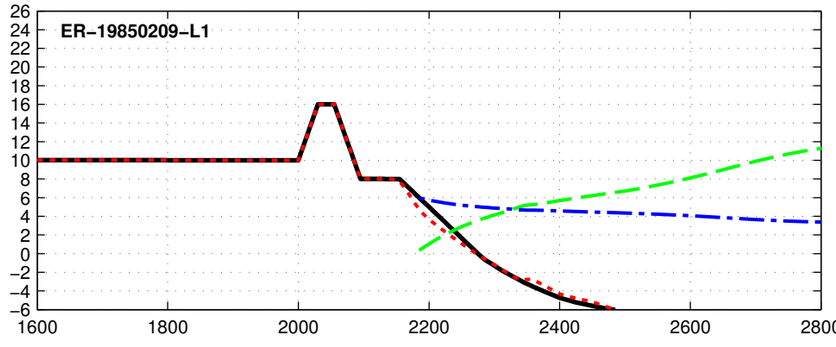


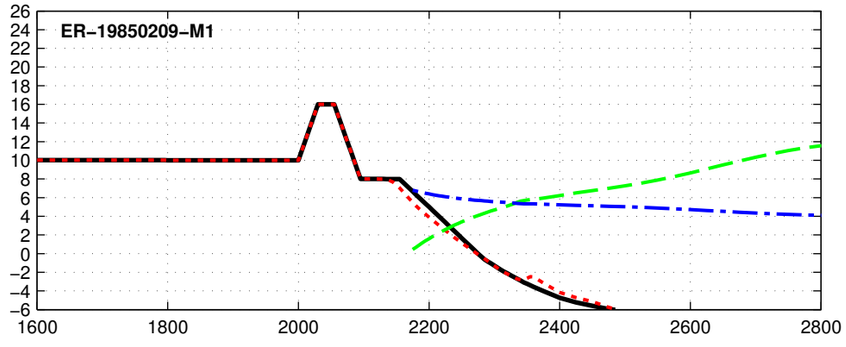


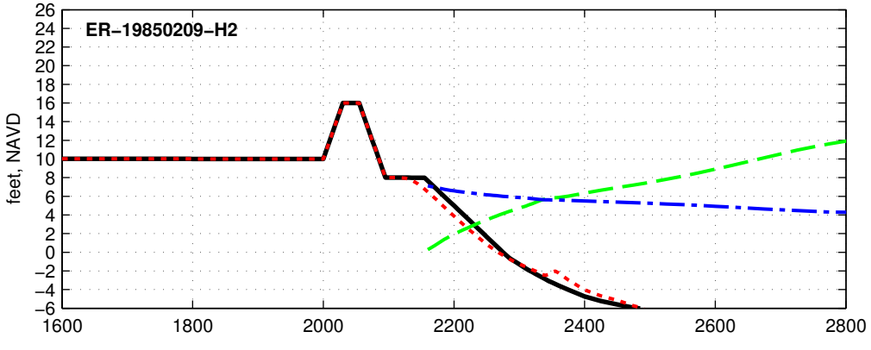
Initial Final Max Hs Max Water Level

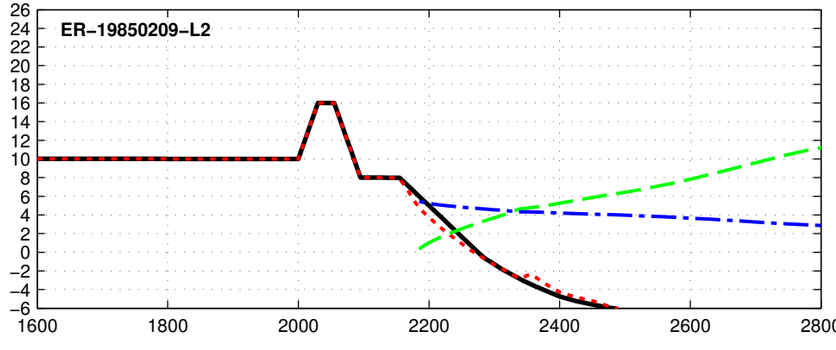
Dune R2T1-16.0-025-060 - ER-19850209

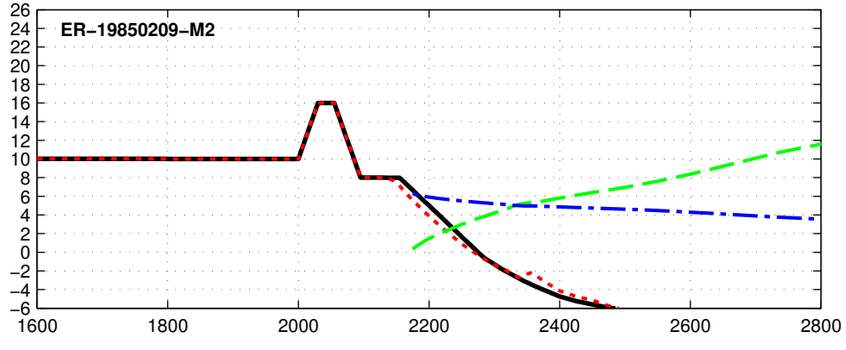


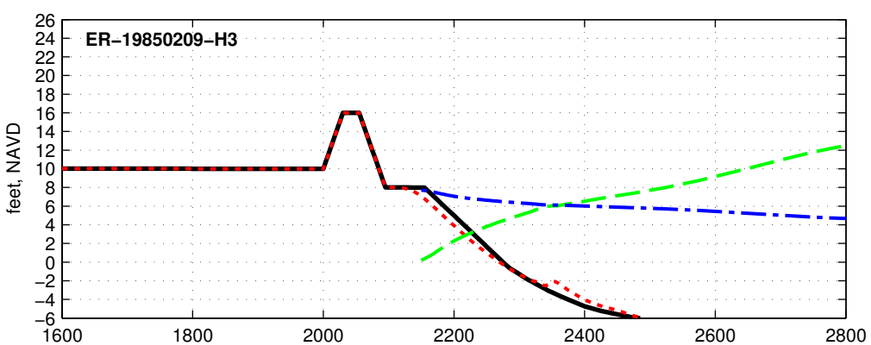


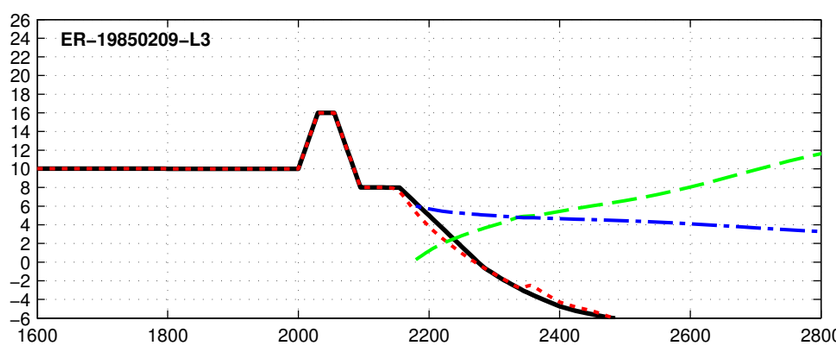


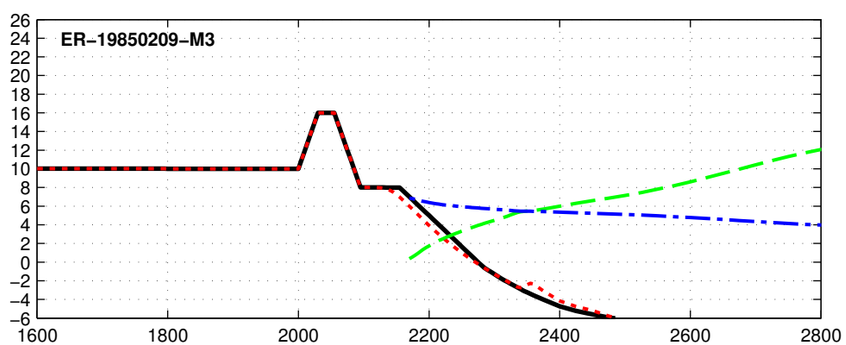


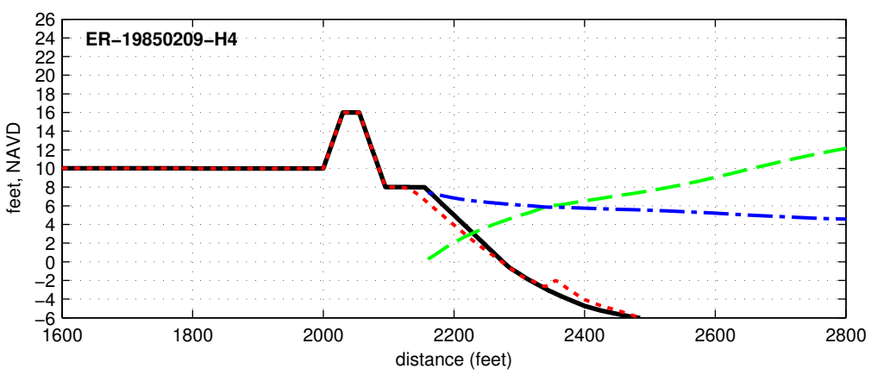


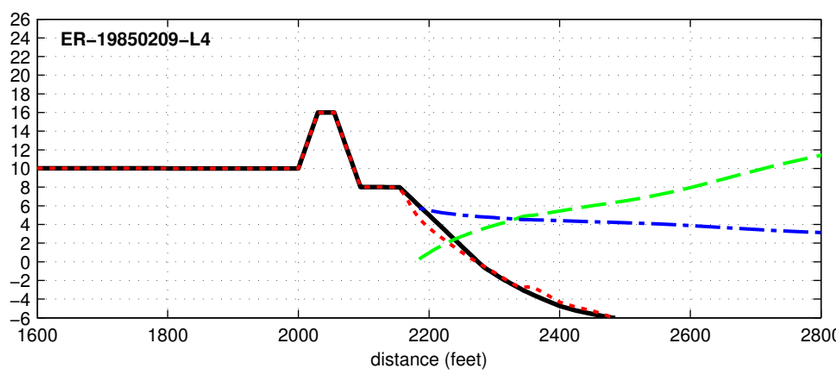


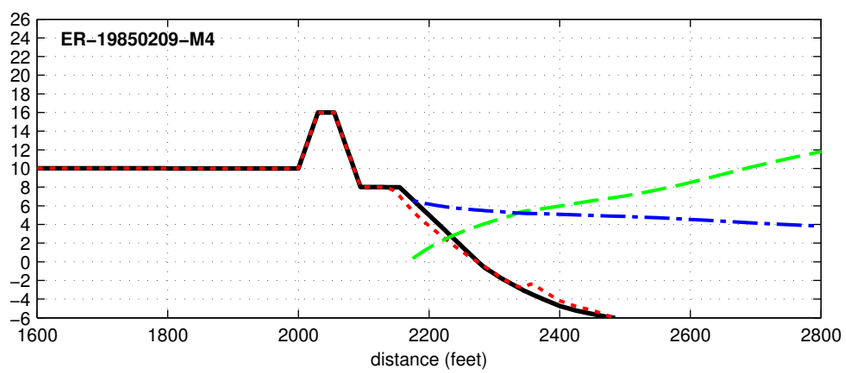






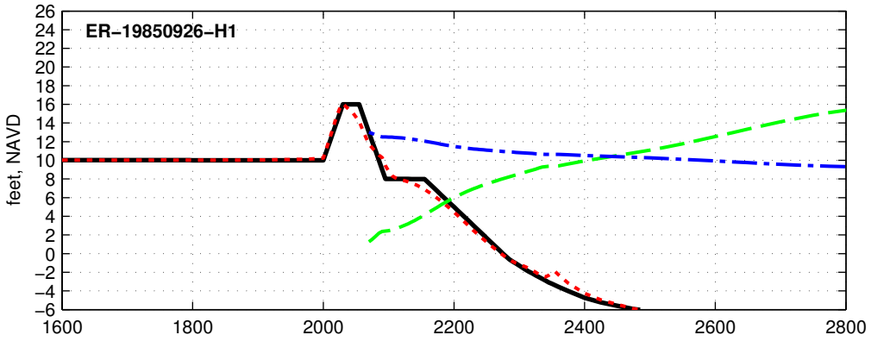


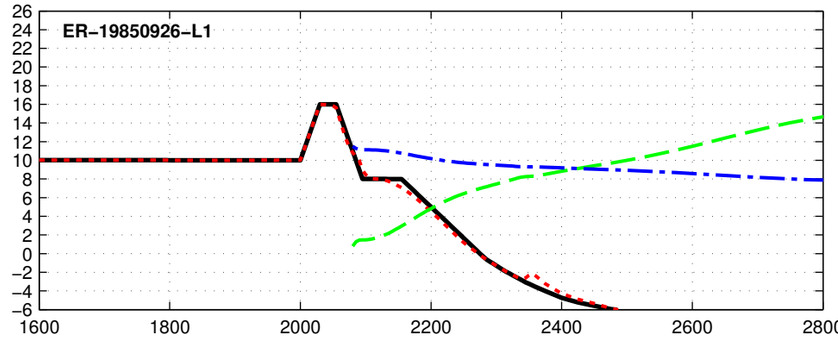


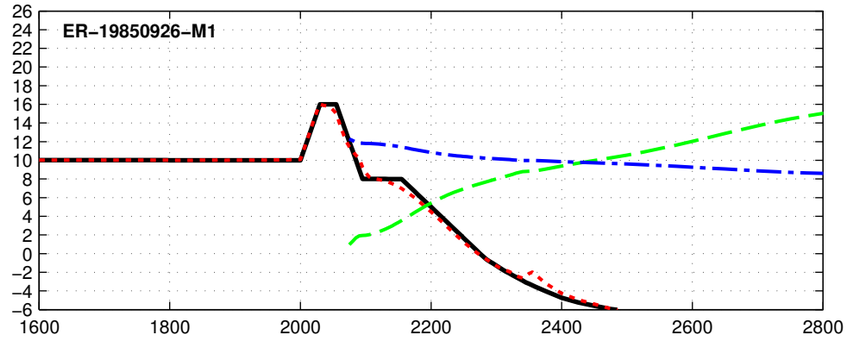


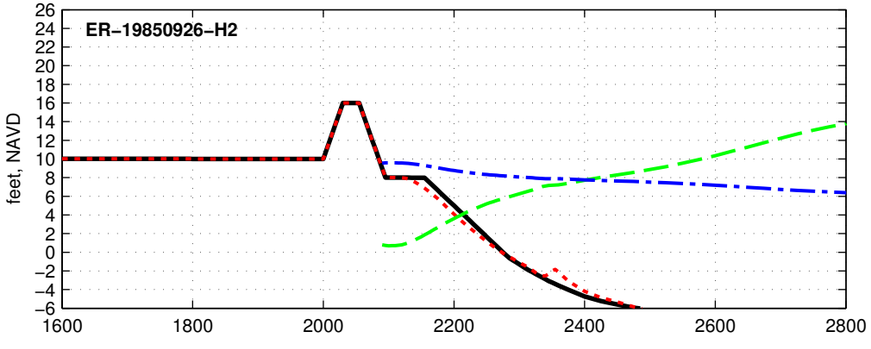
Initial Final Max Hs Max Water Level

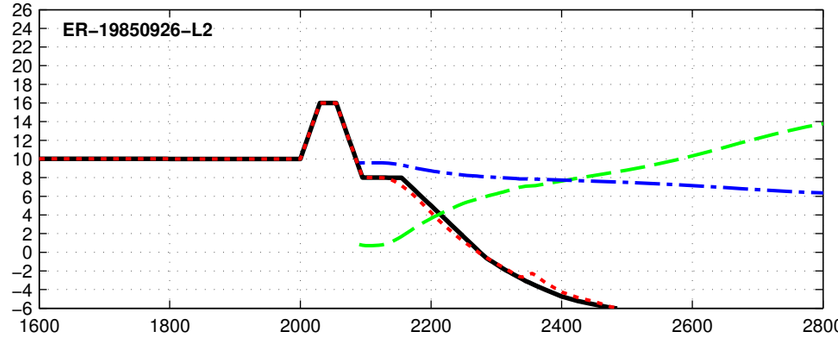
Dune R2T1-16.0-025-060 - ER-19850926

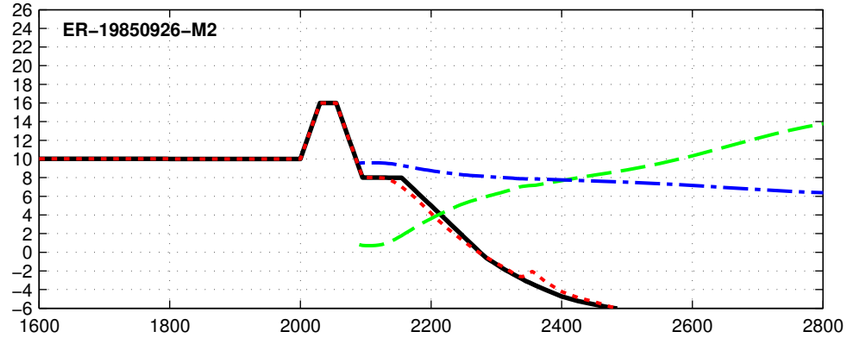


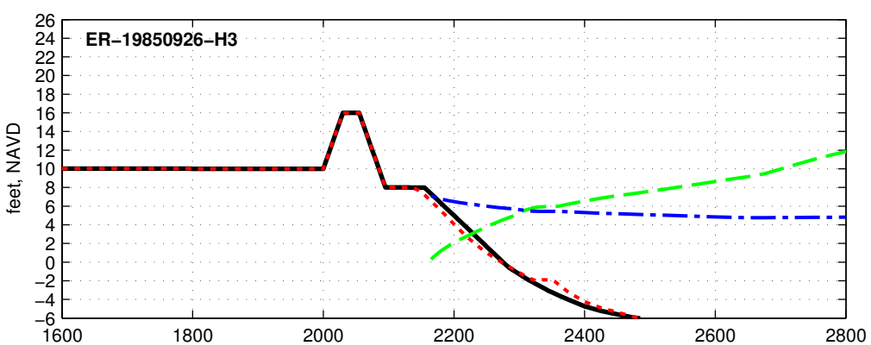


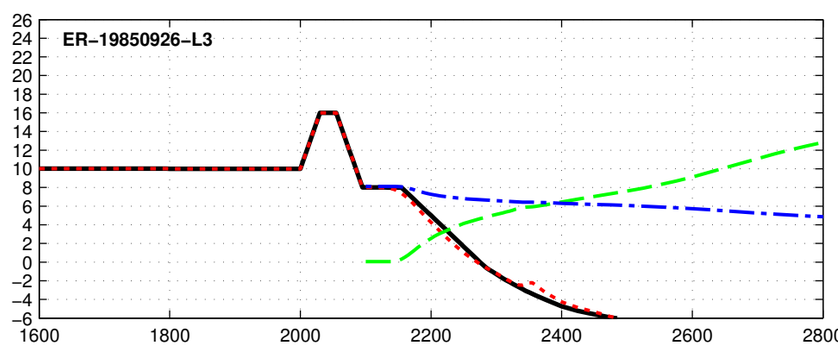


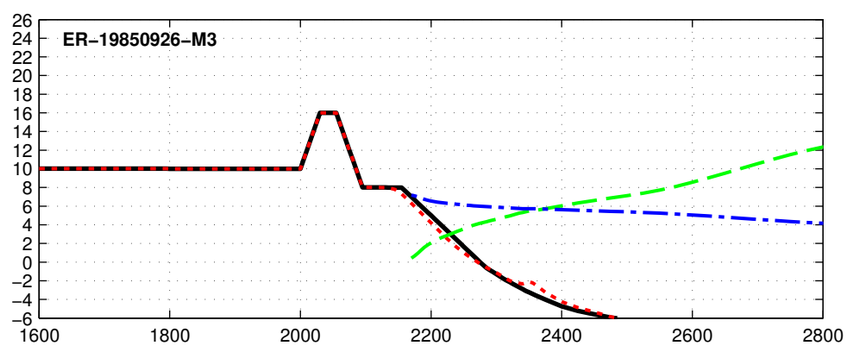


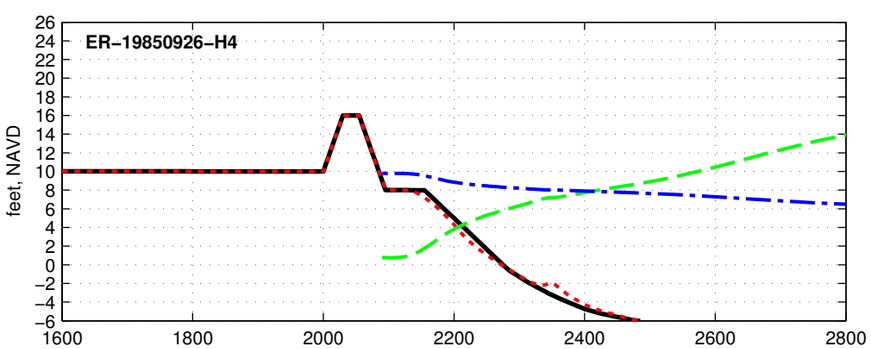


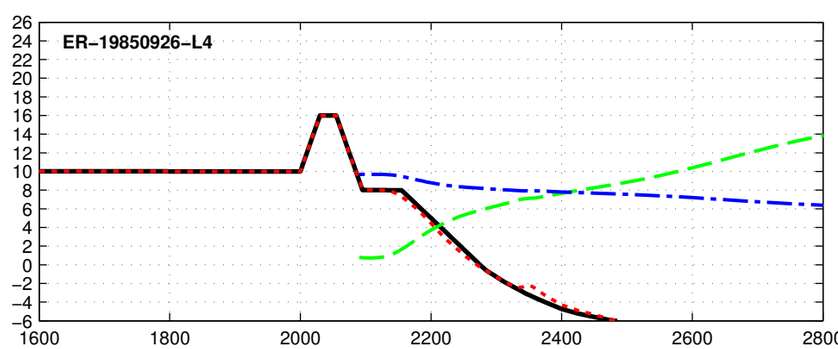


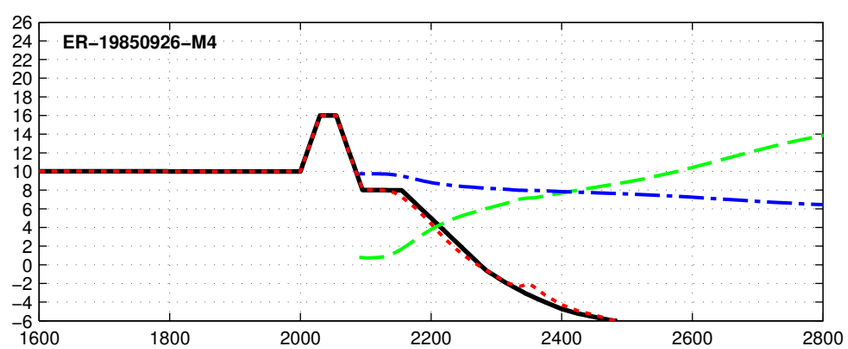






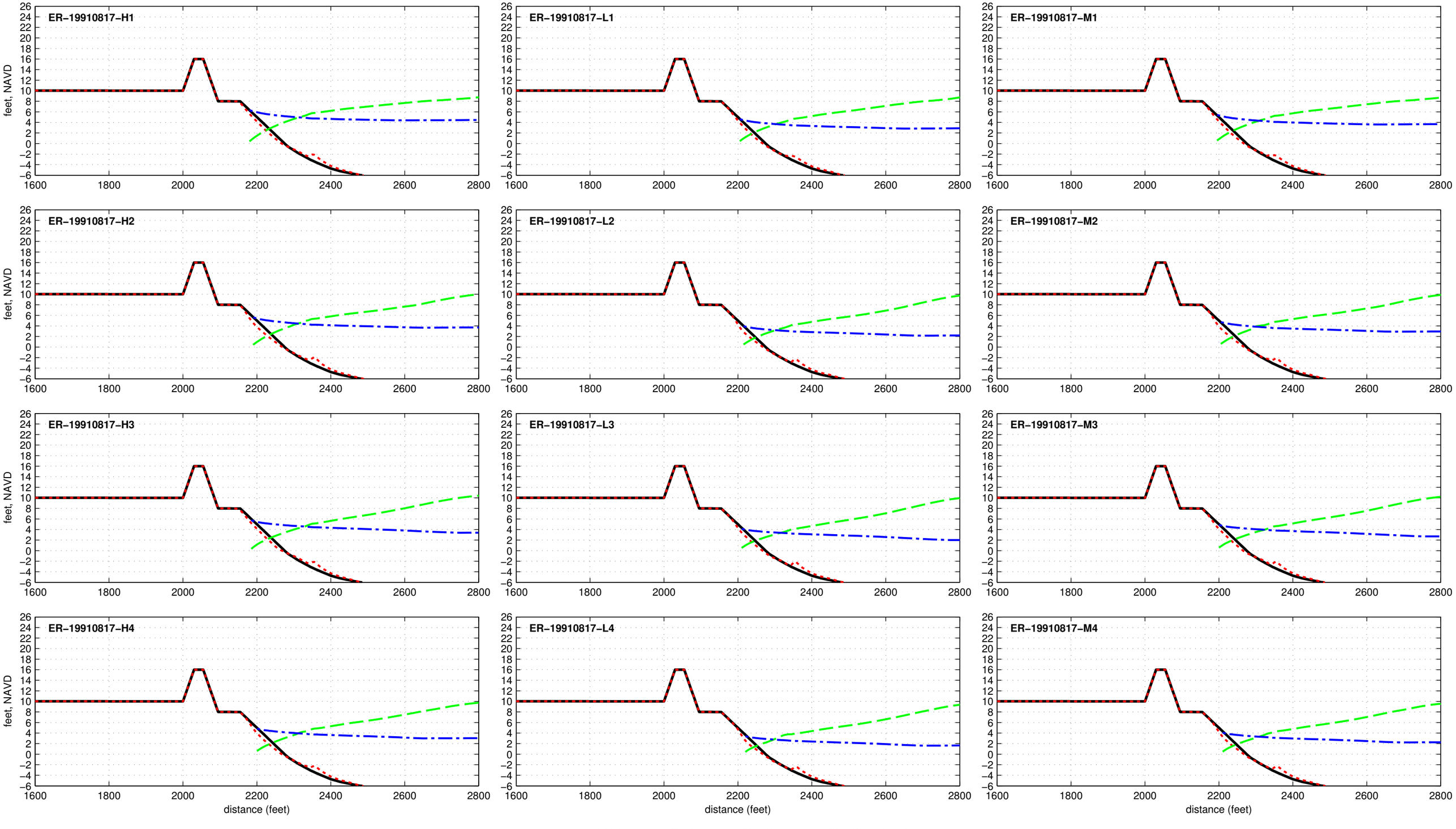




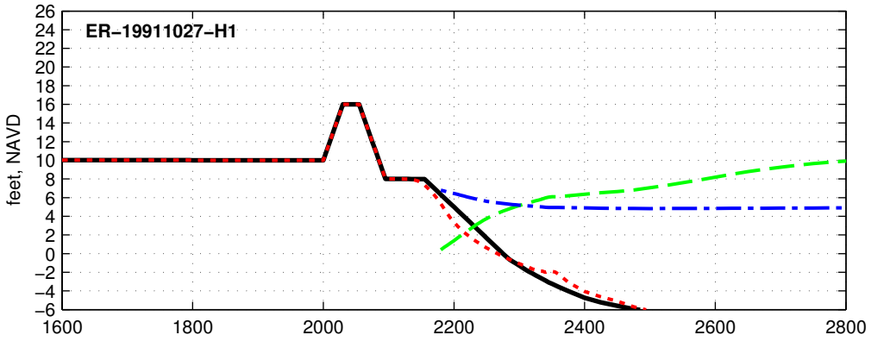


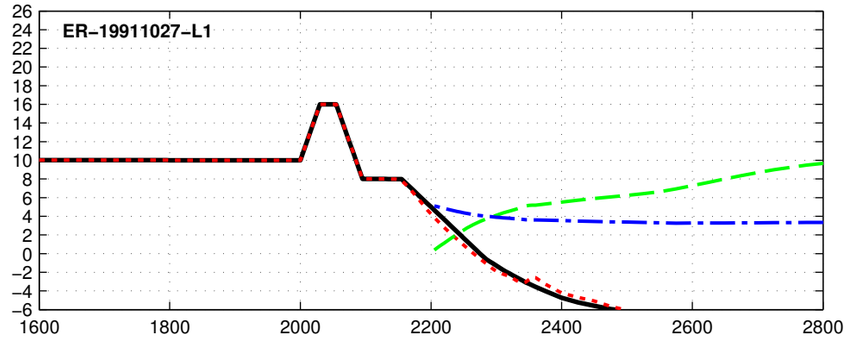
Initial Final Max Hs Max Water Level

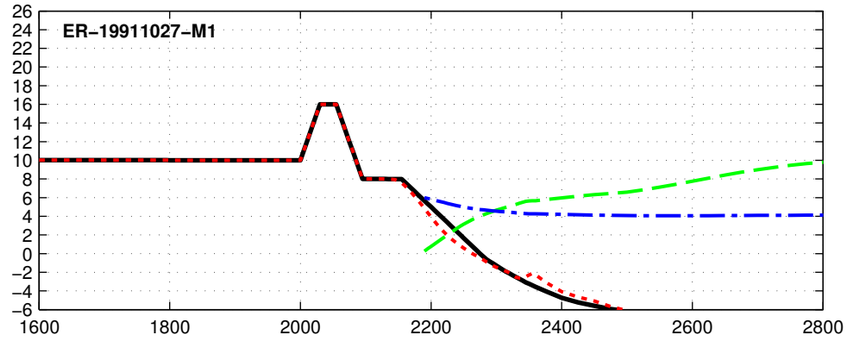
Dune R2T1-16.0-025-060 - ER-19910817

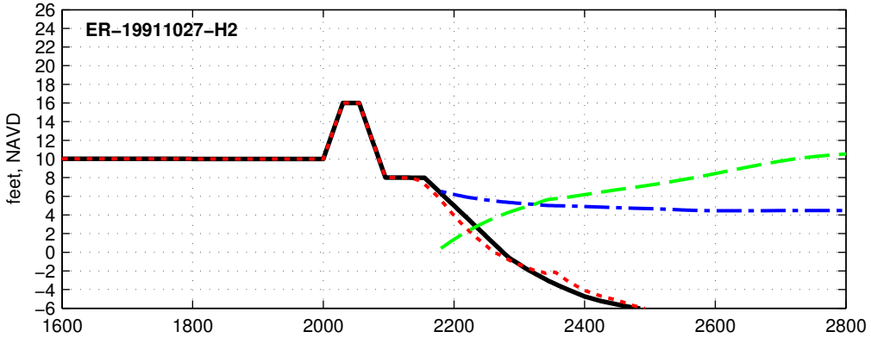


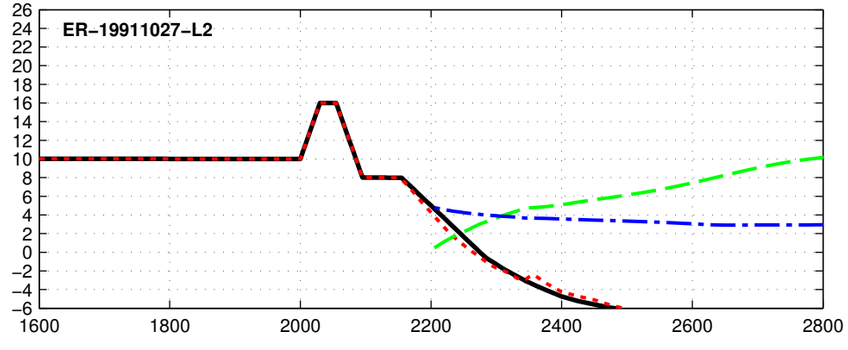
Dune R2T1-16.0-025-060 - ER-19911027

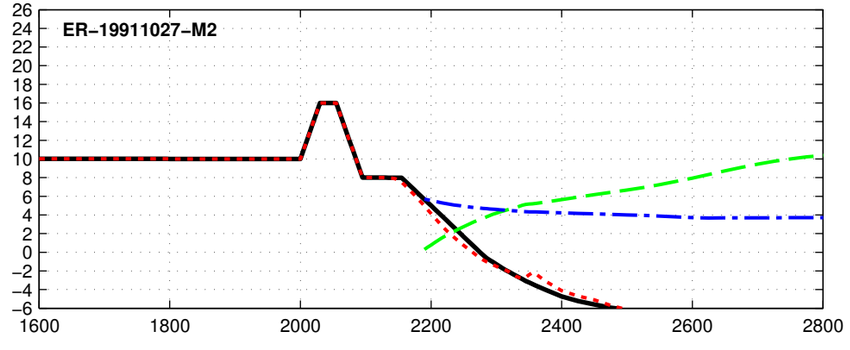


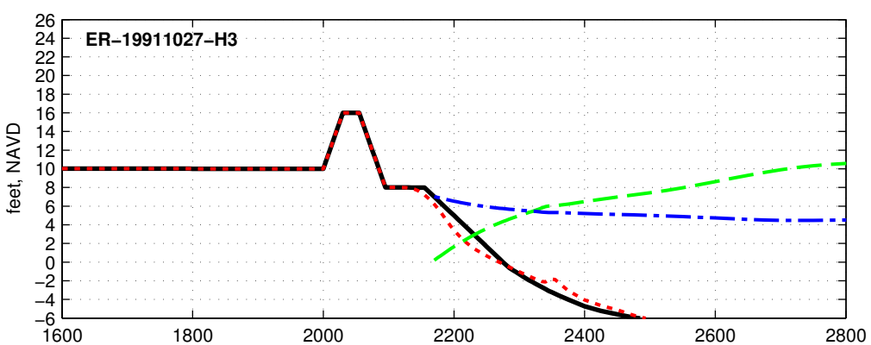


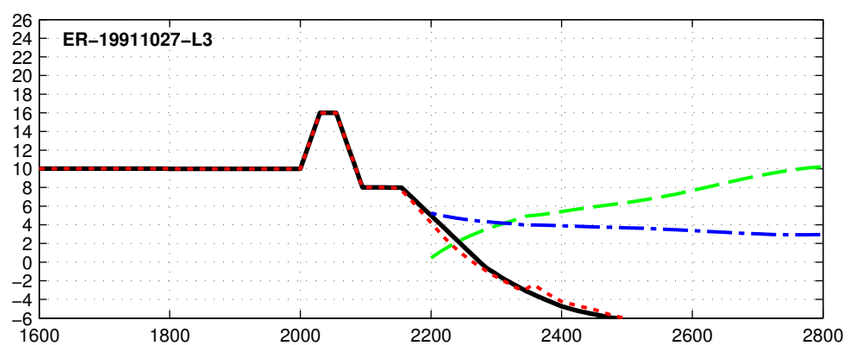


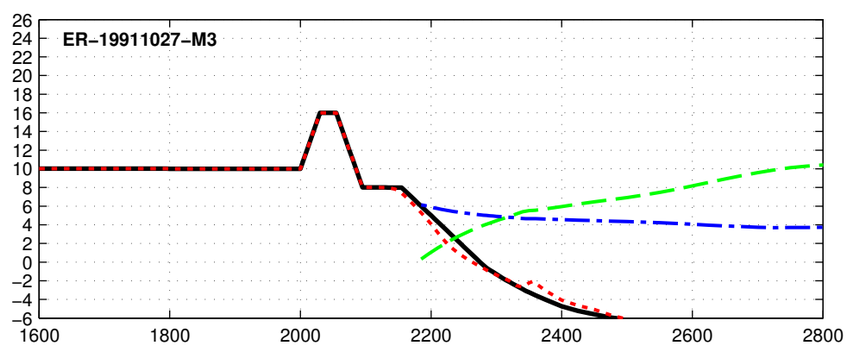


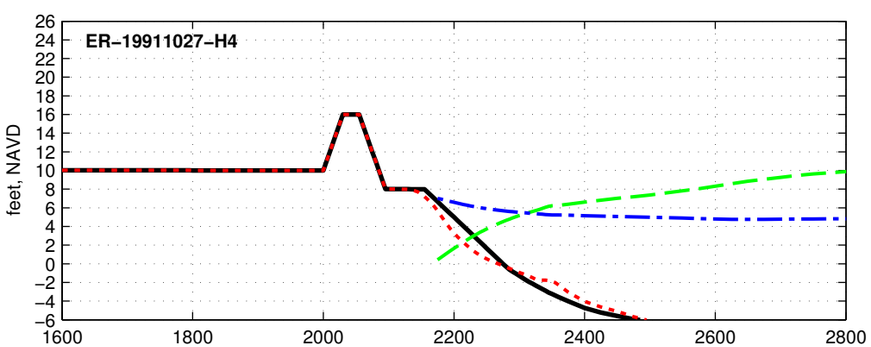


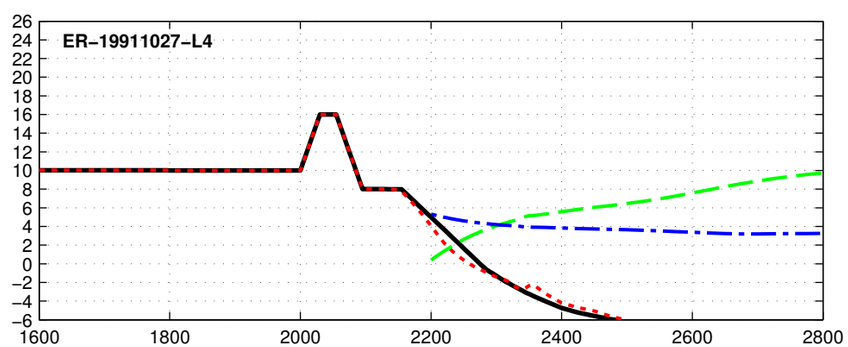


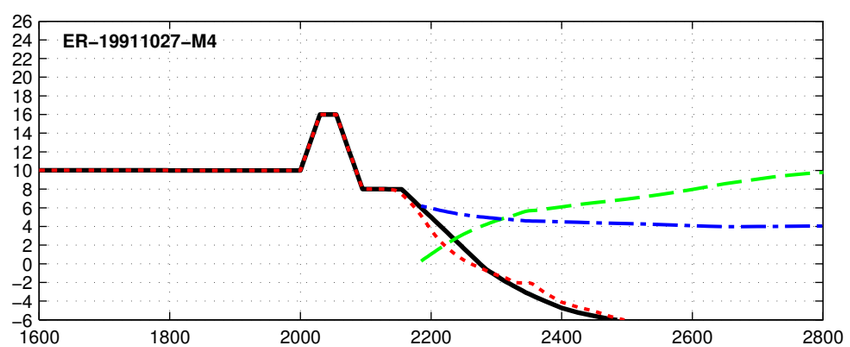






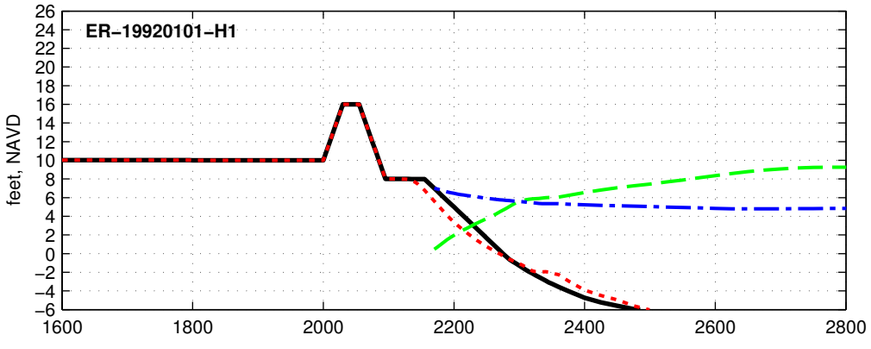


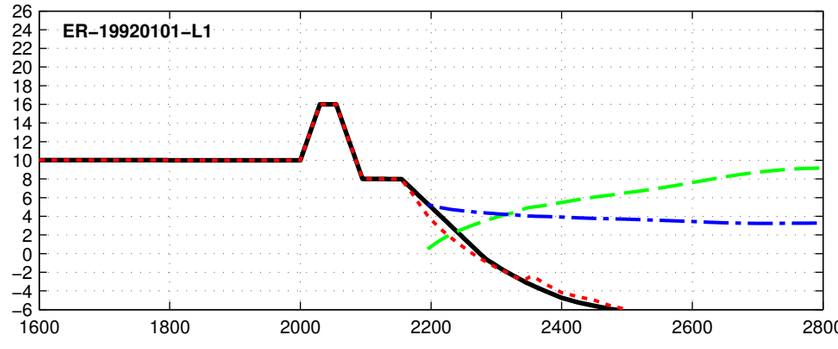


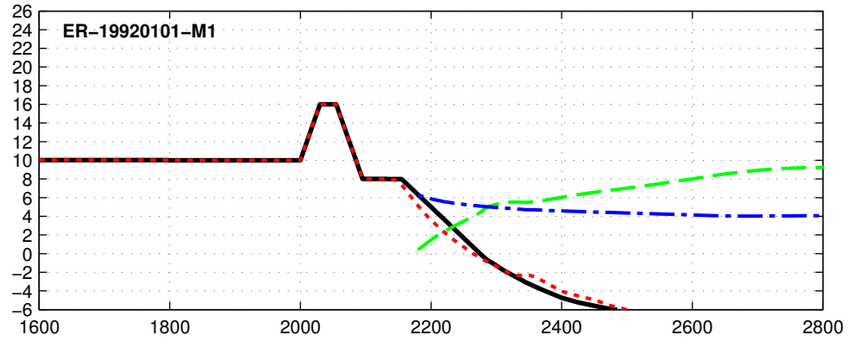


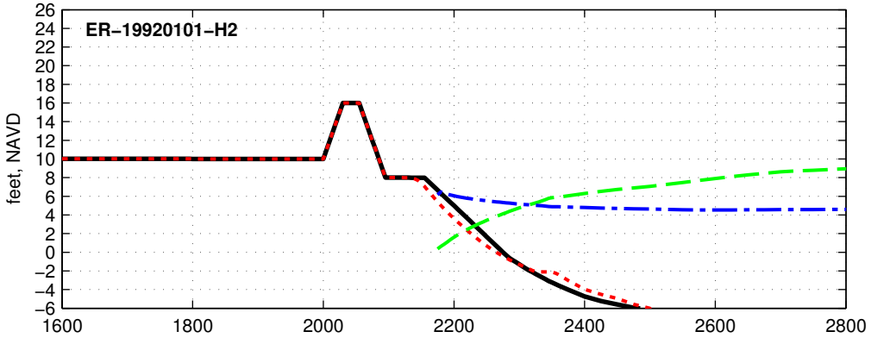
Initial Final Max Hs Max Water Level

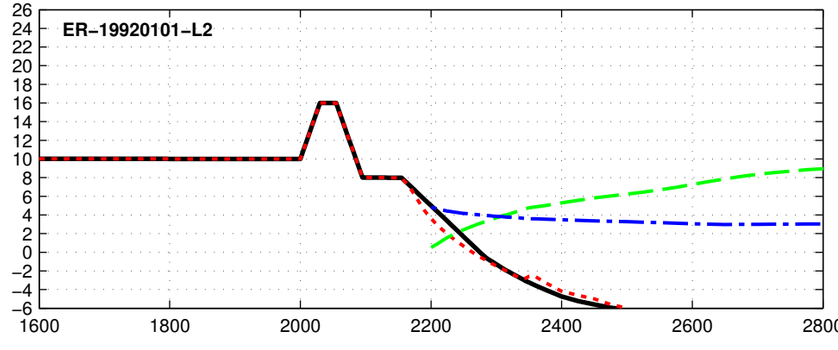
Dune R2T1-16.0-025-060 - ER-19920101

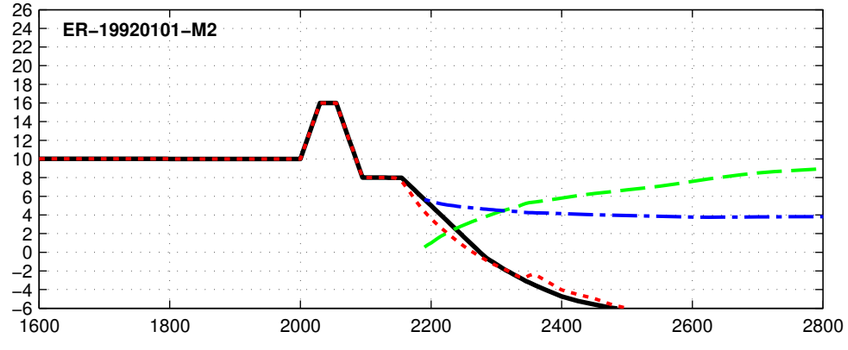


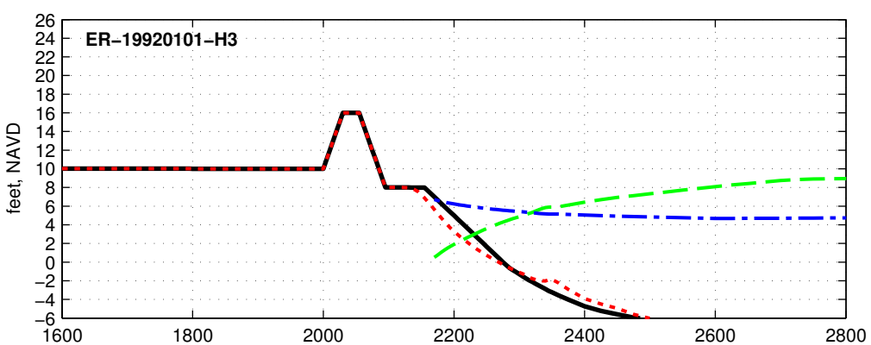


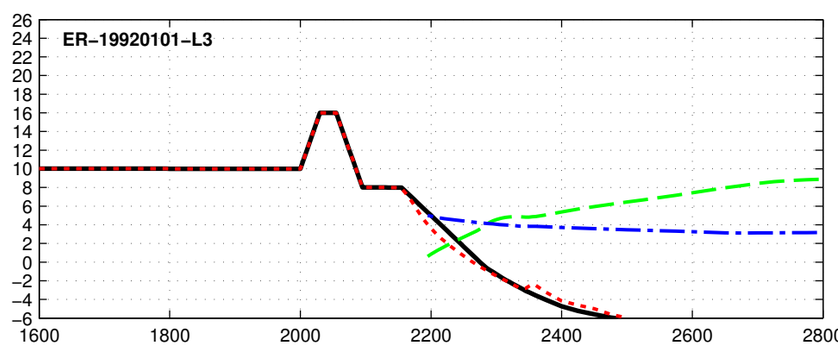


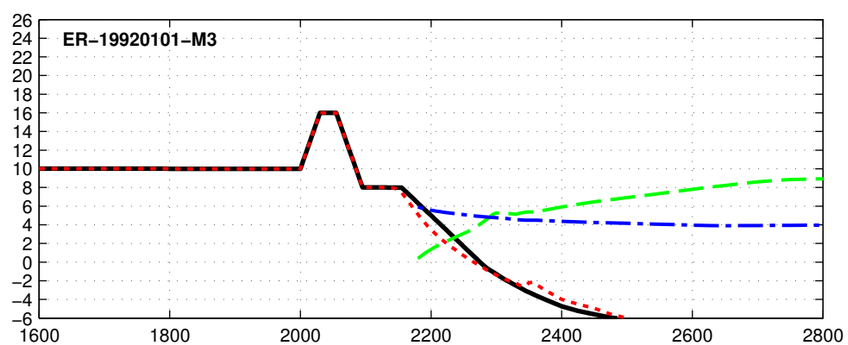


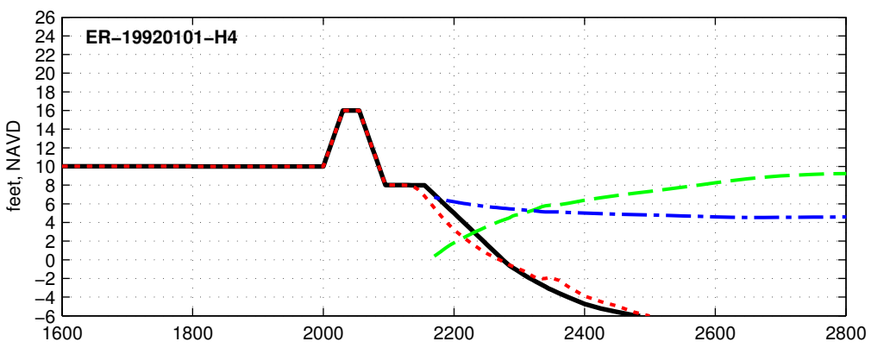


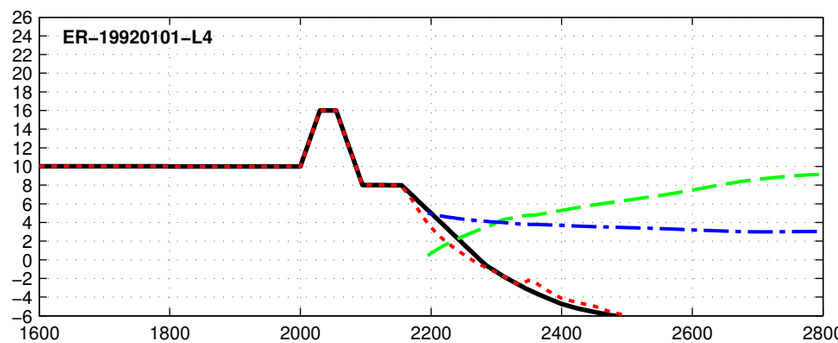


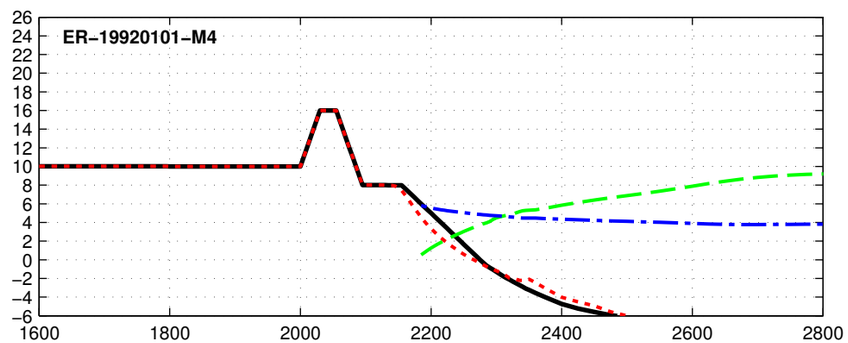






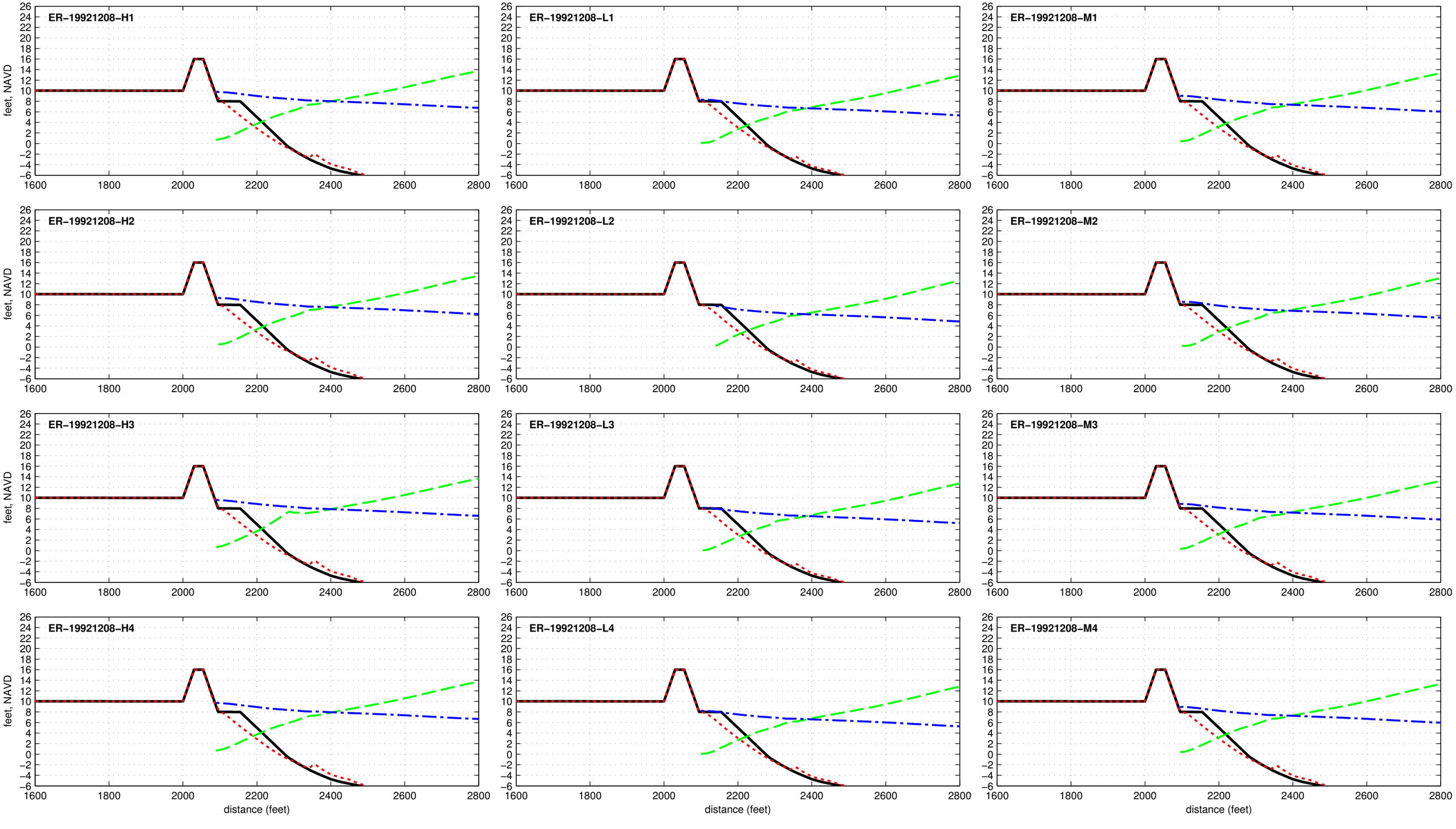




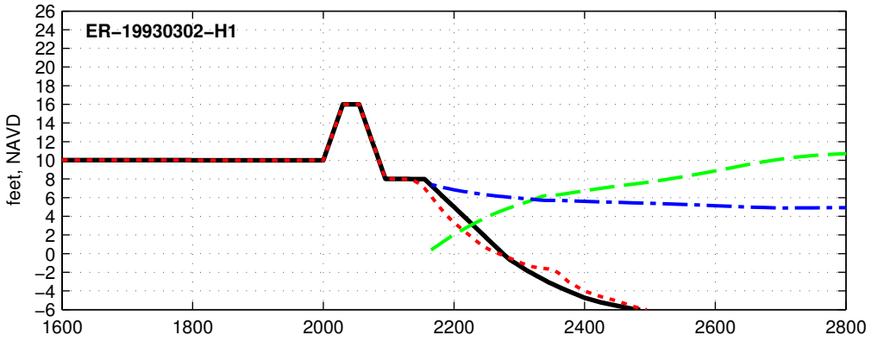


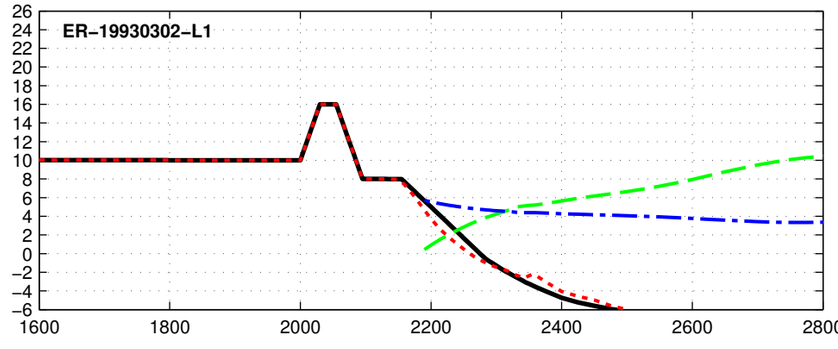
Initial Final Max Hs Max Water Level

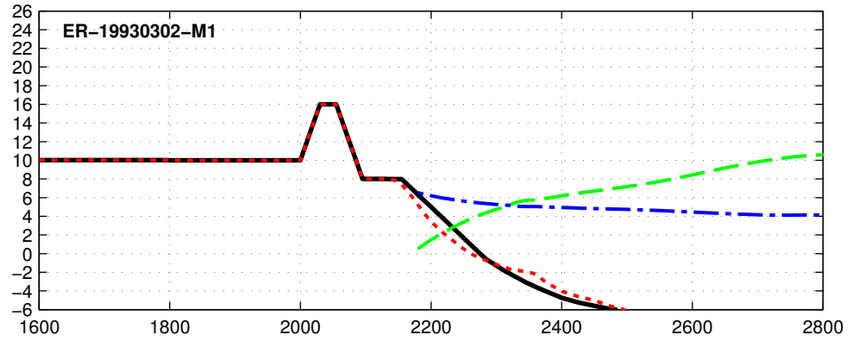
Dune R2T1-16.0-025-060 - ER-19921208

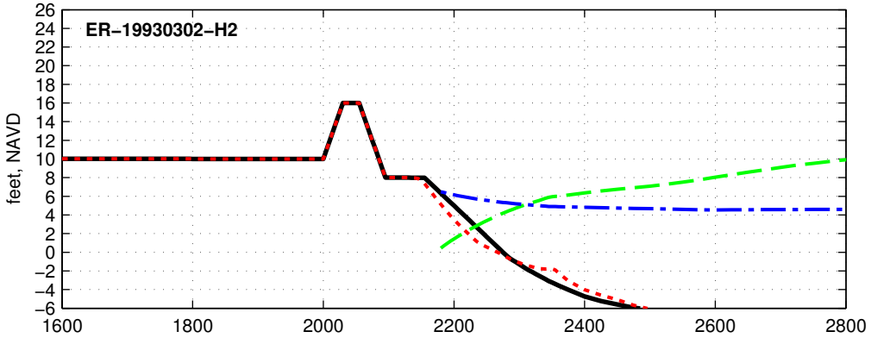


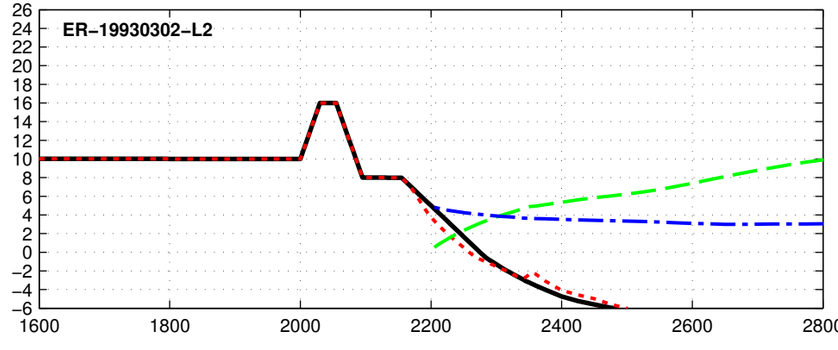
Dune R2T1-16.0-025-060 - ER-19930302

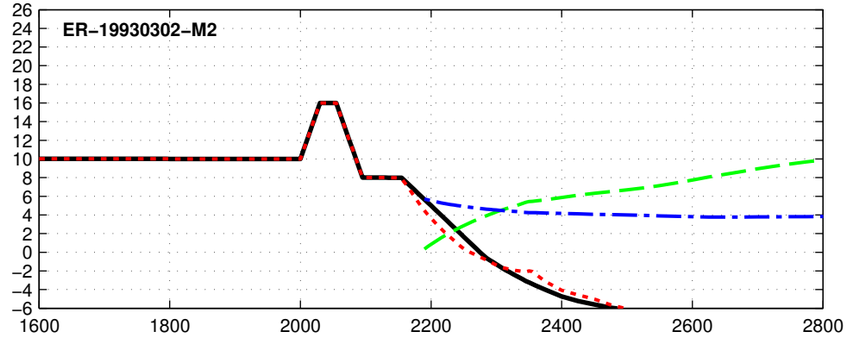


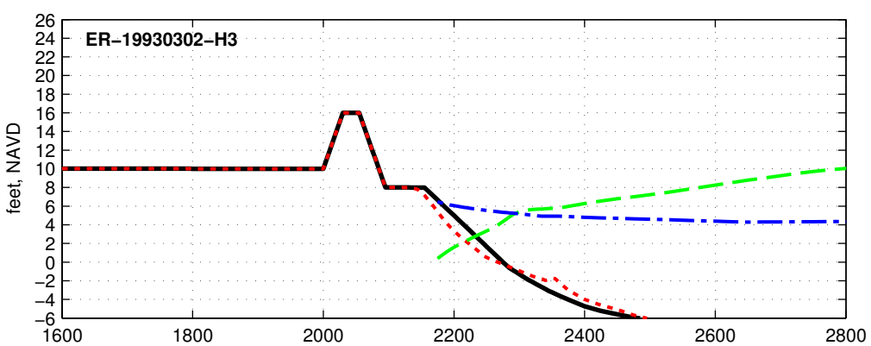


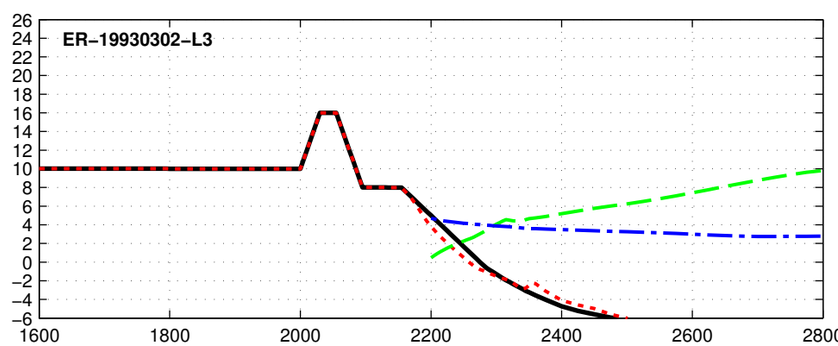


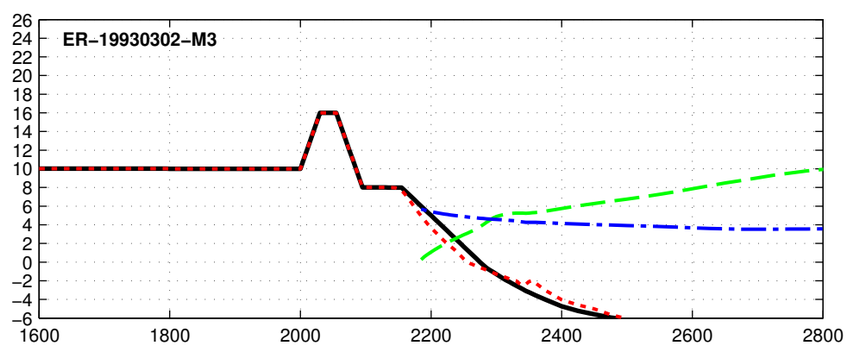


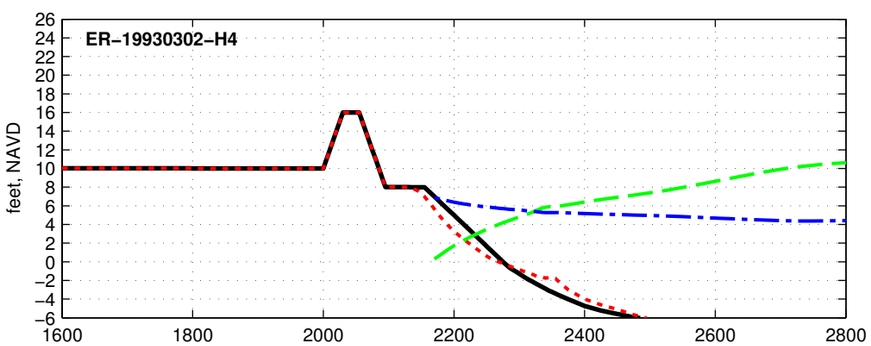


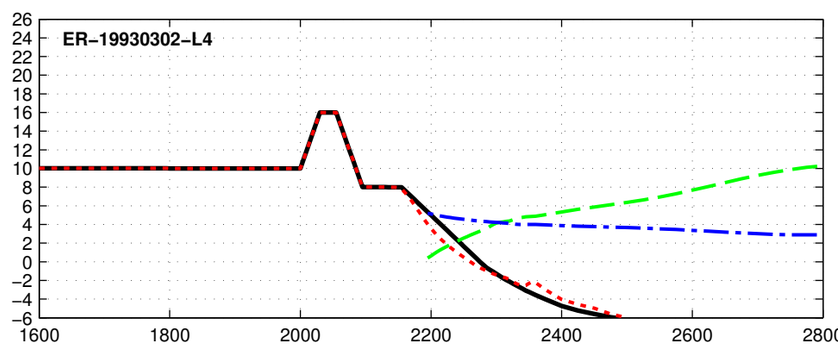


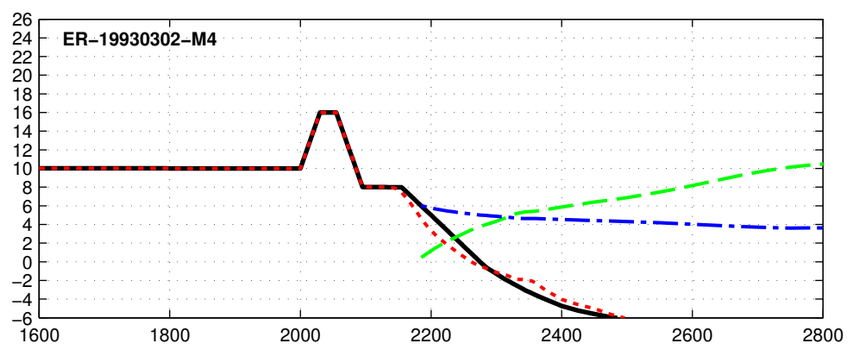










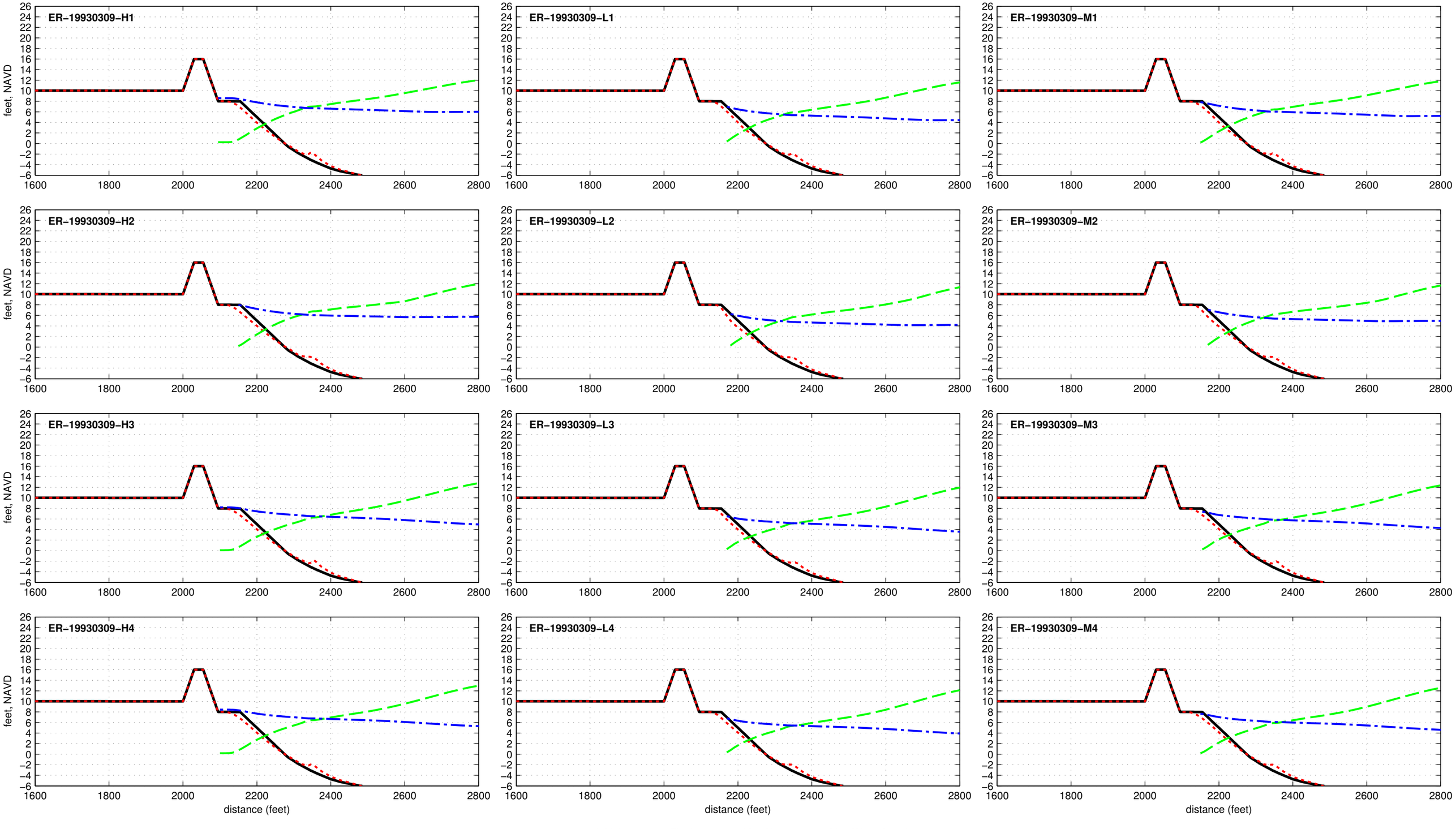


feet, NAVD

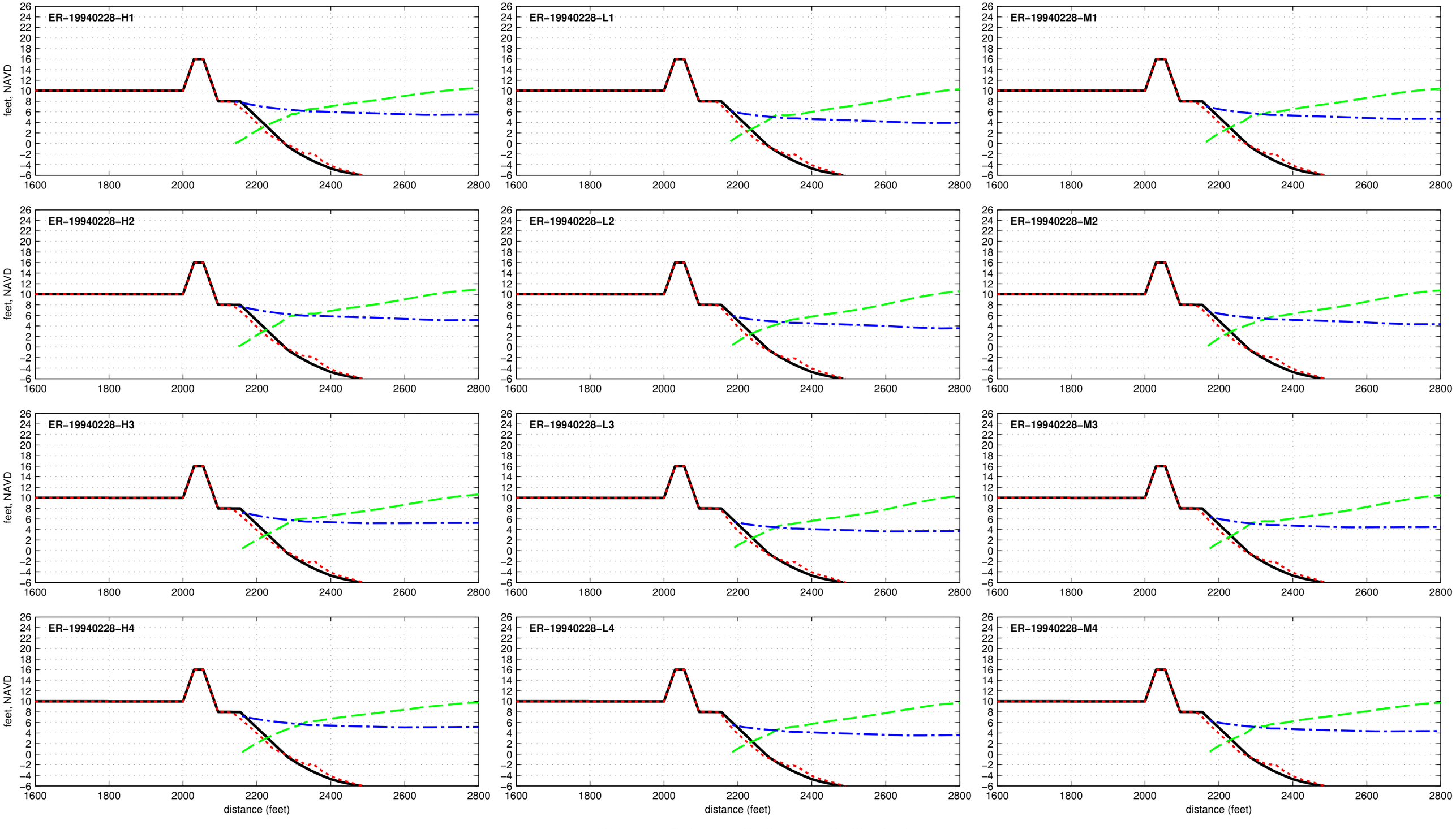
distance (feet)

Initial Final Max Hs Max Water Level

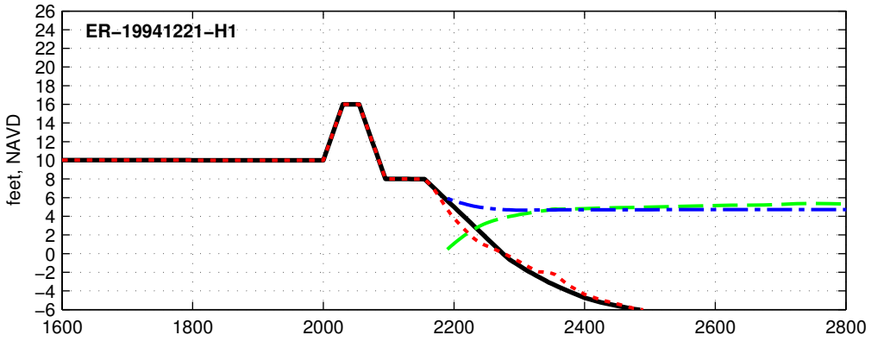
Dune R2T1-16.0-025-060 - ER-19930309

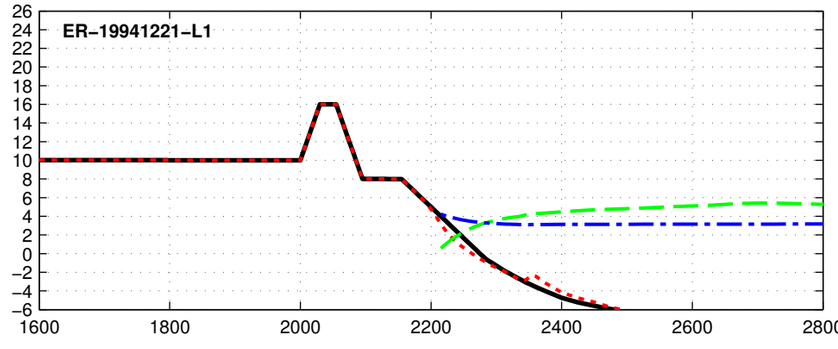


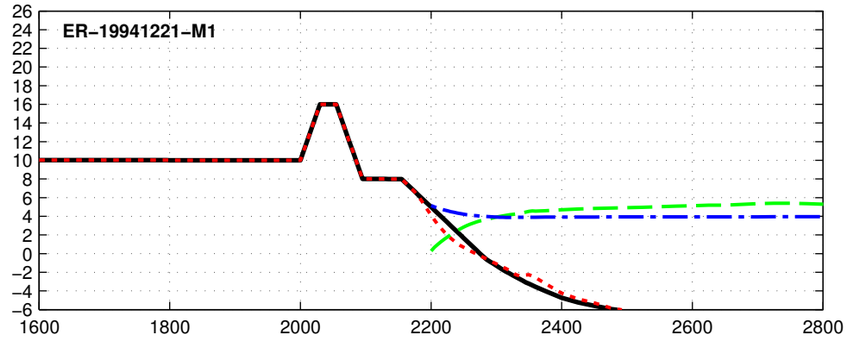
Dune R2T1-16.0-025-060 - ER-19940228

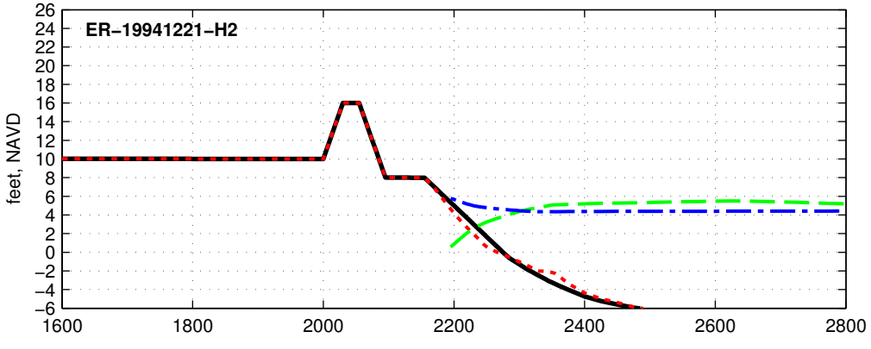


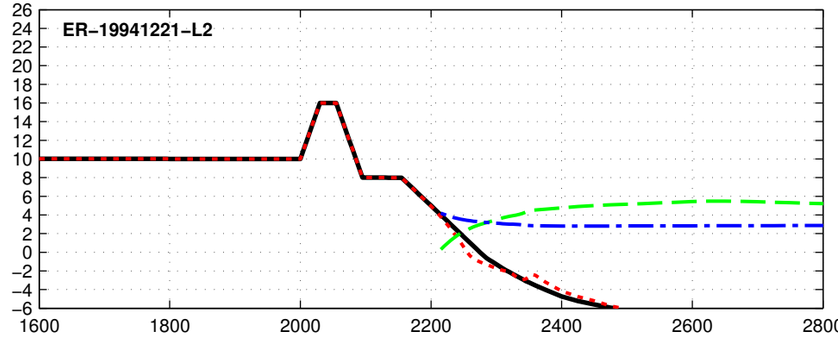
Dune R2T1-16.0-025-060 - ER-19941221

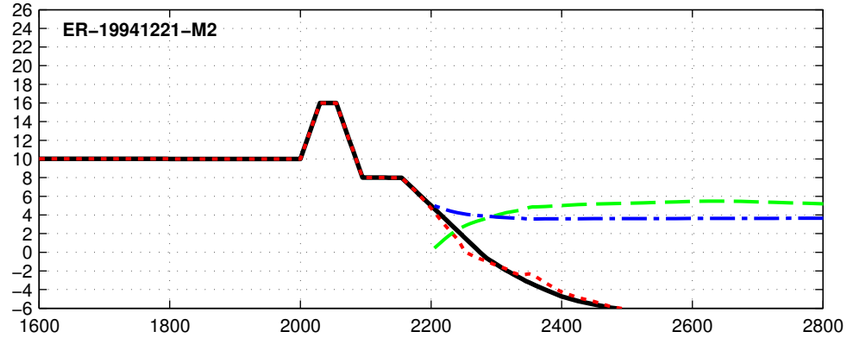


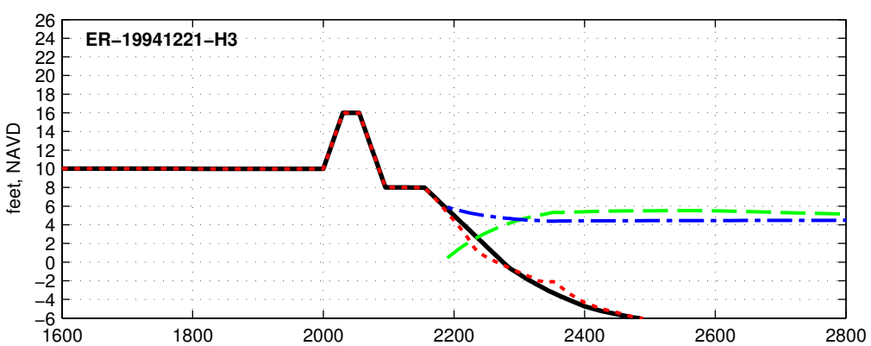


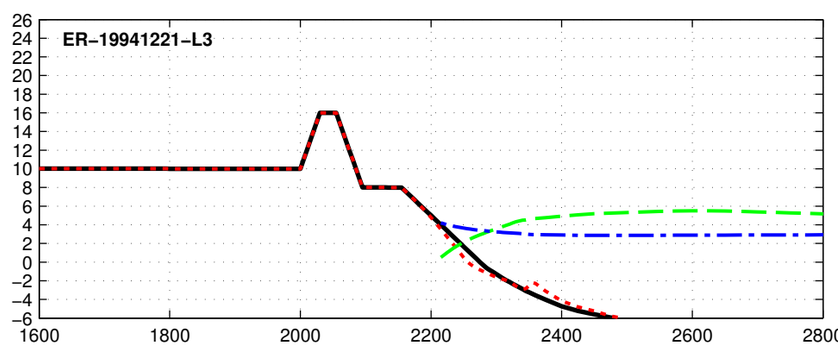


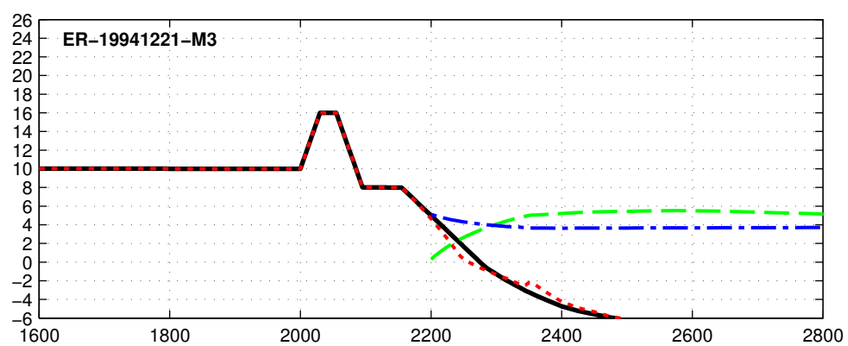


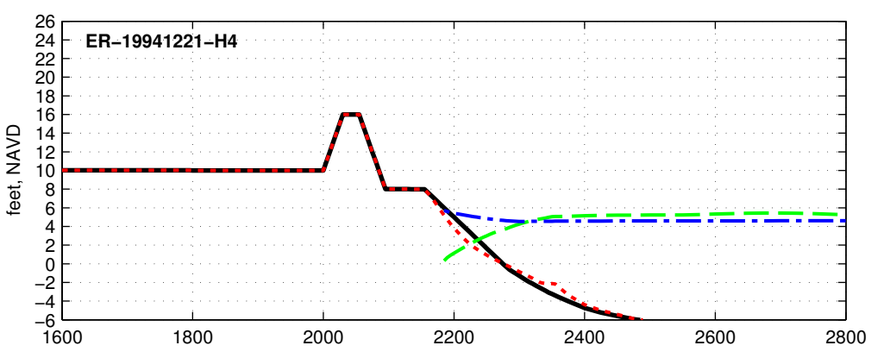


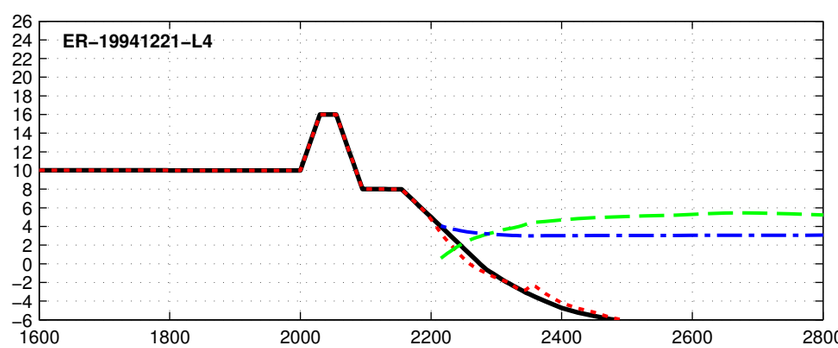


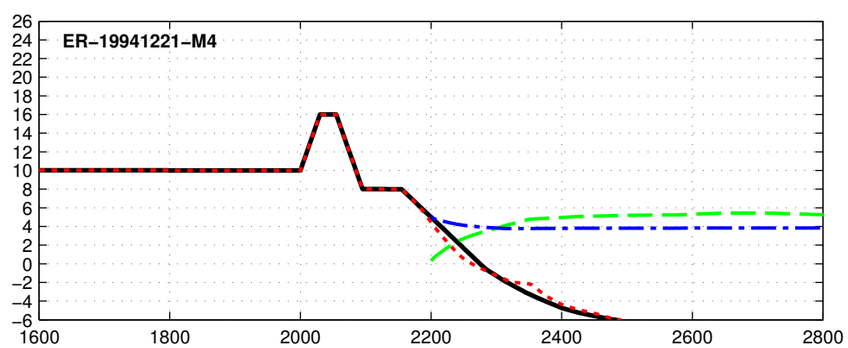






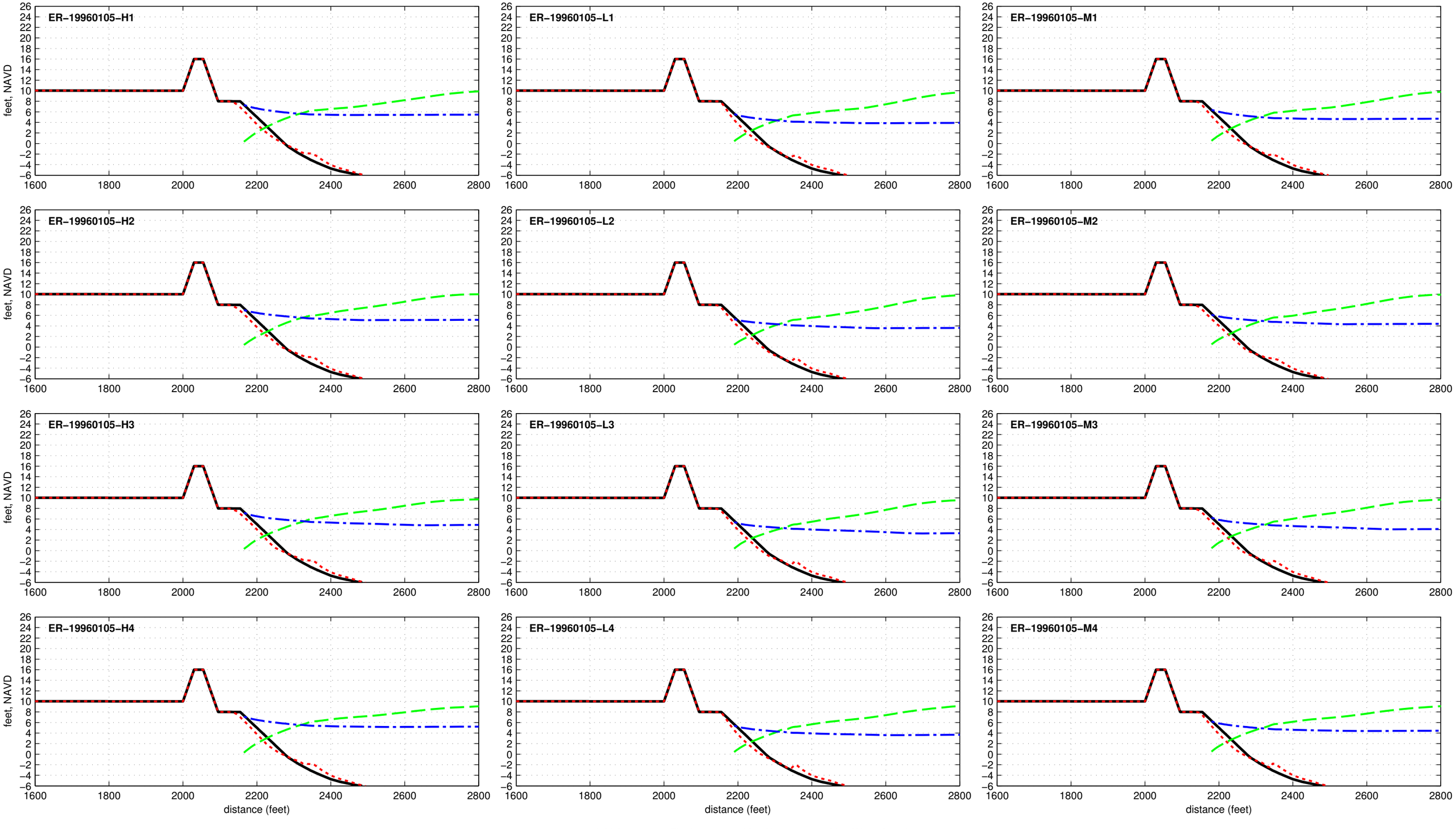




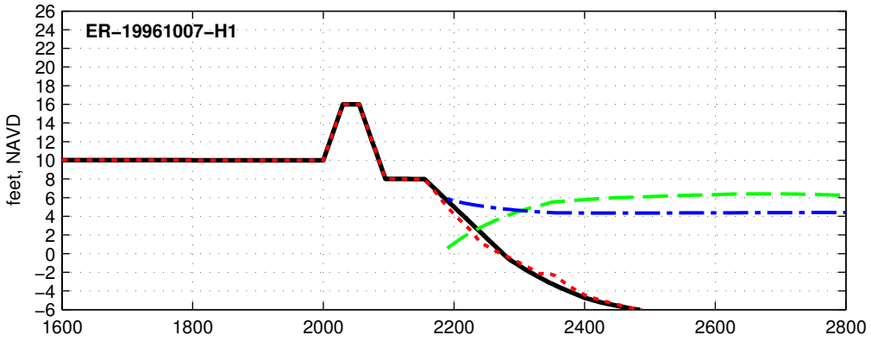


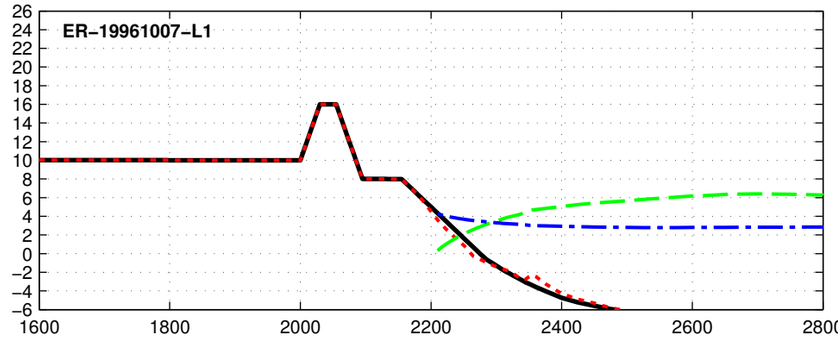
Initial Final Max Hs Max Water Level

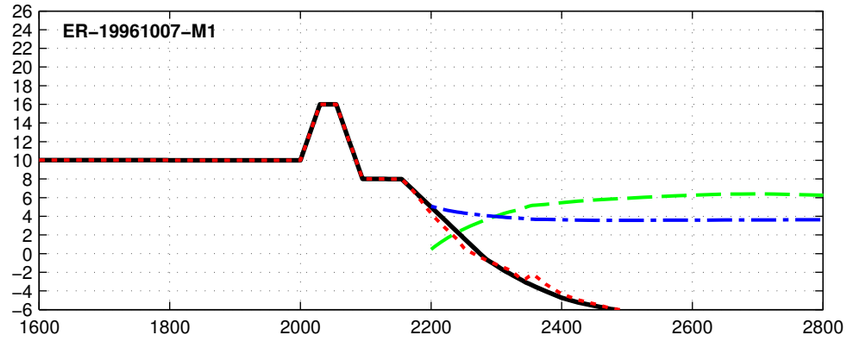
Dune R2T1-16.0-025-060 - ER-19960105

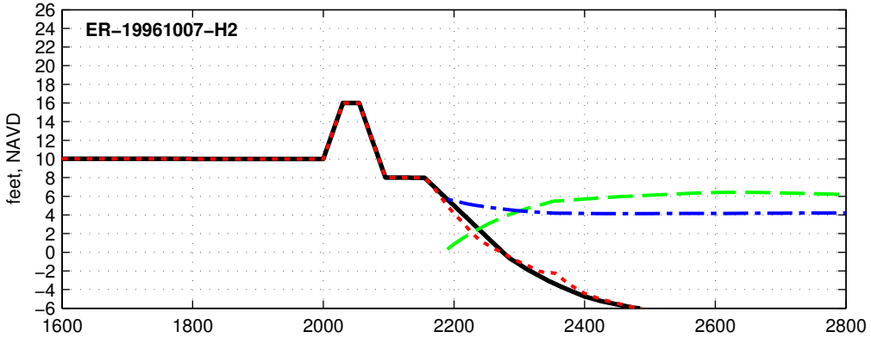


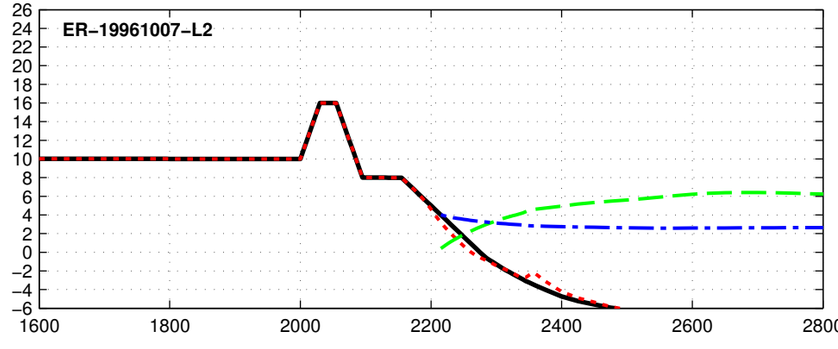
Dune R2T1-16.0-025-060 - ER-19961007

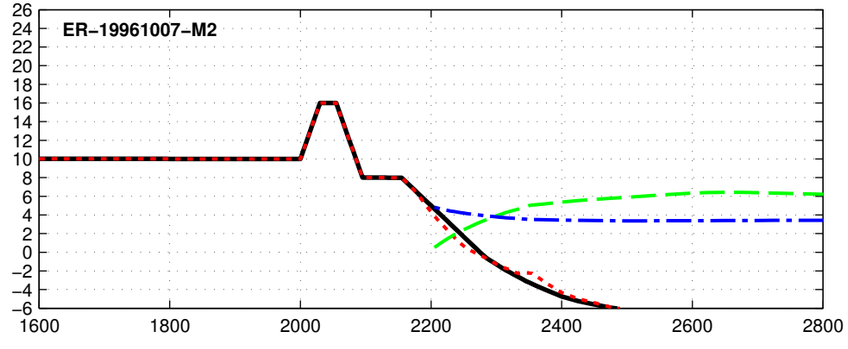


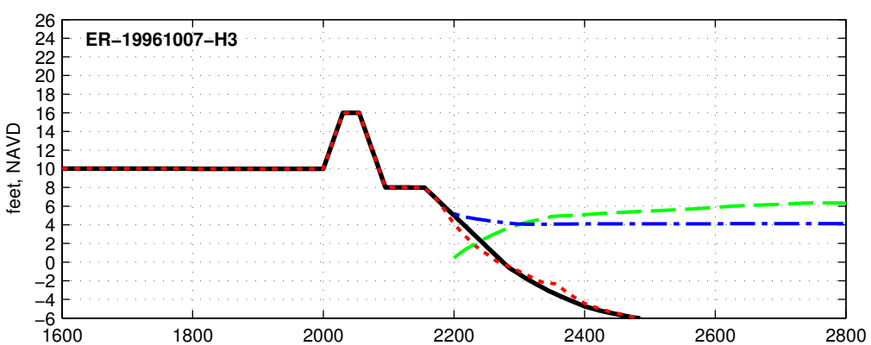


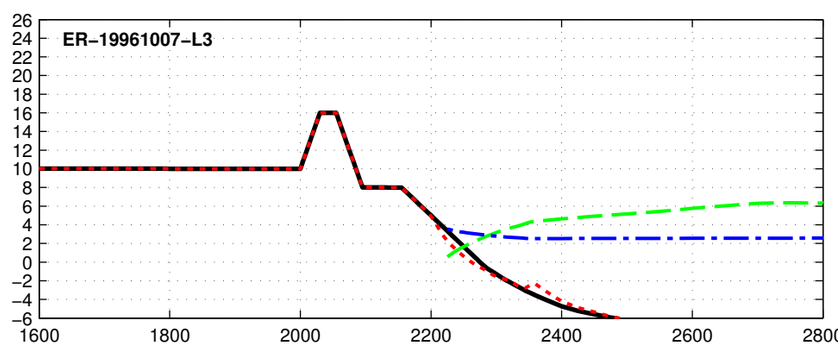


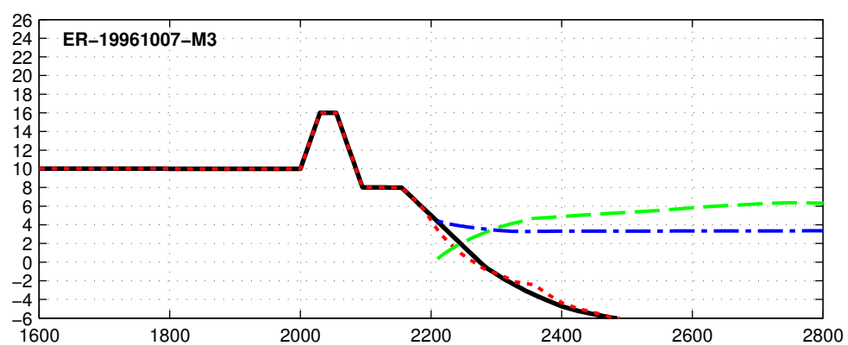


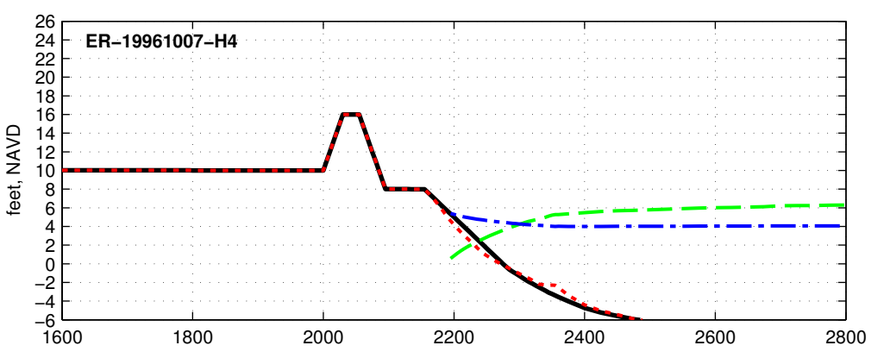


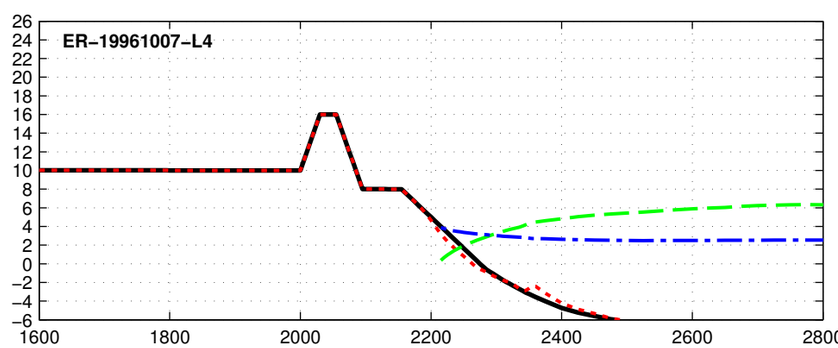


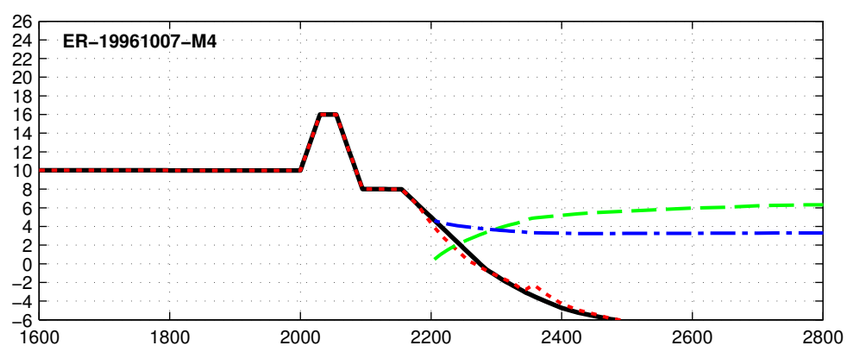






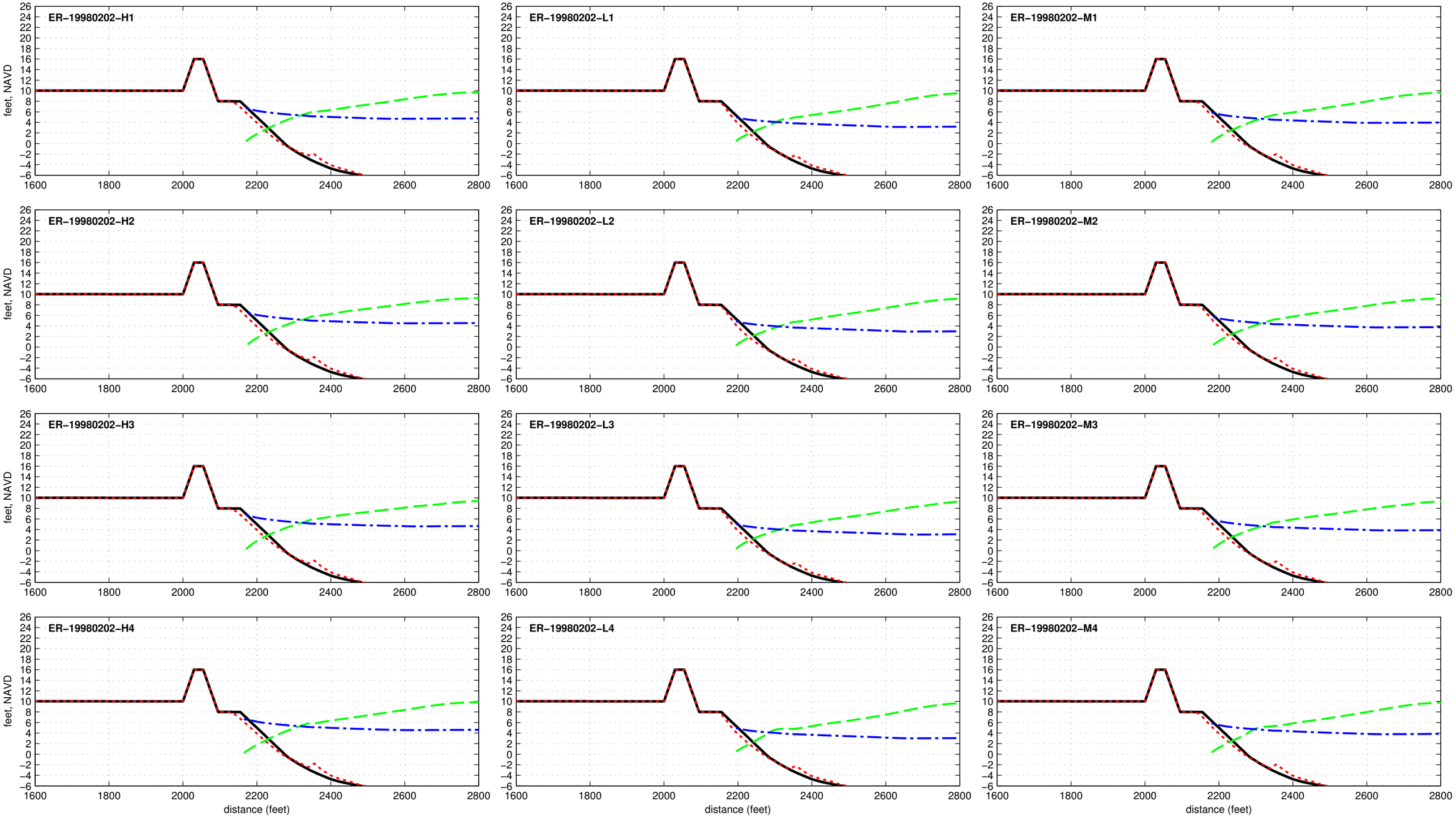




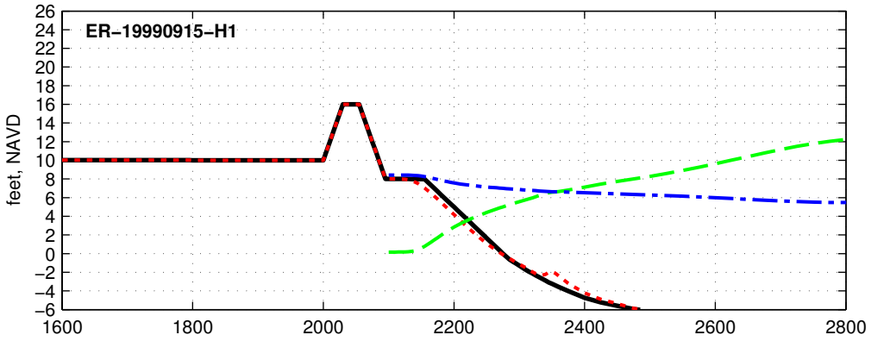


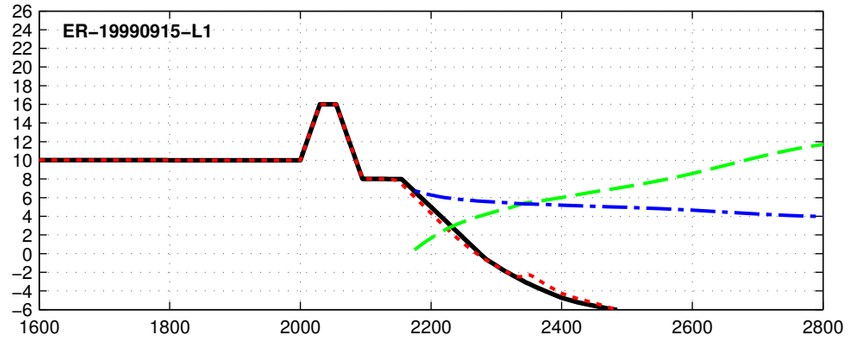
Initial Final Max Hs Max Water Level

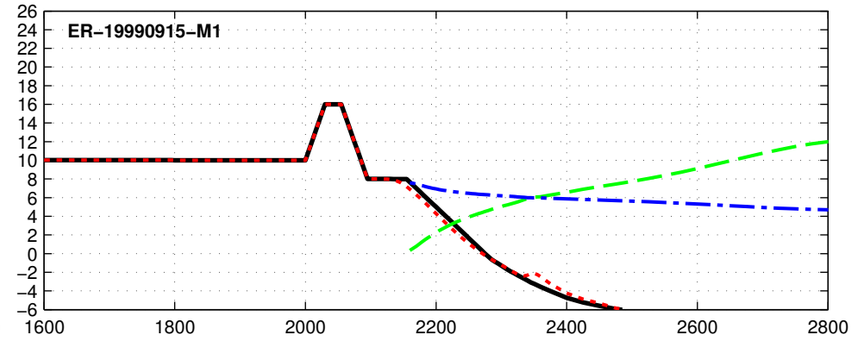
Dune R2T1-16.0-025-060 - ER-19980202

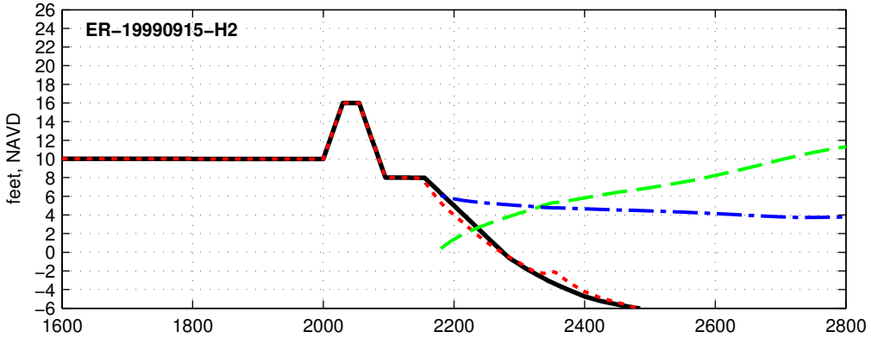


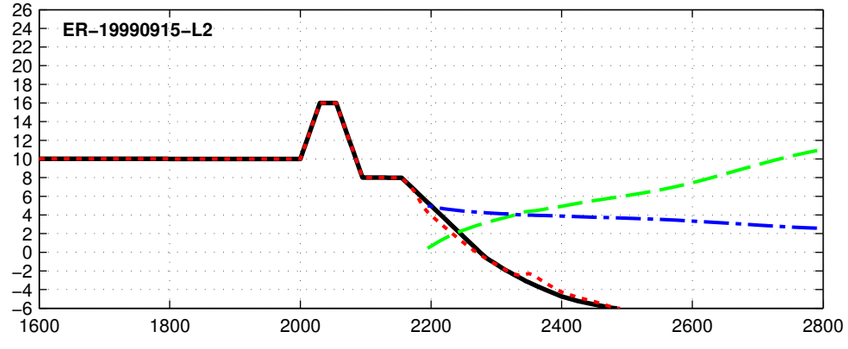
Dune R2T1-16.0-025-060 - ER-19990915

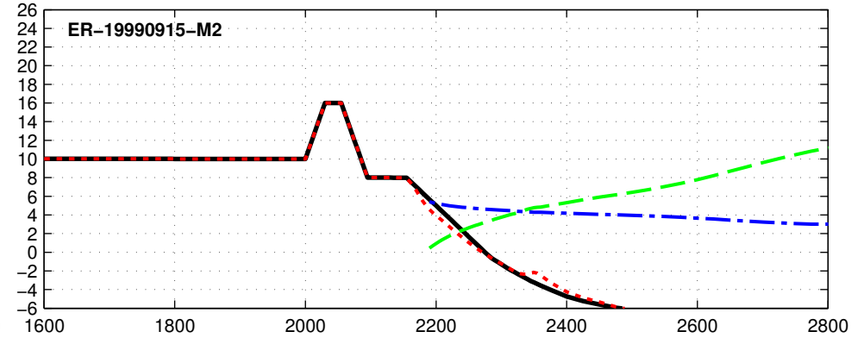


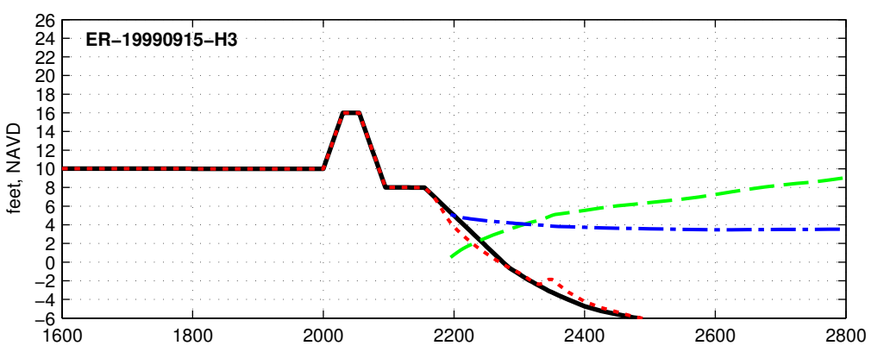


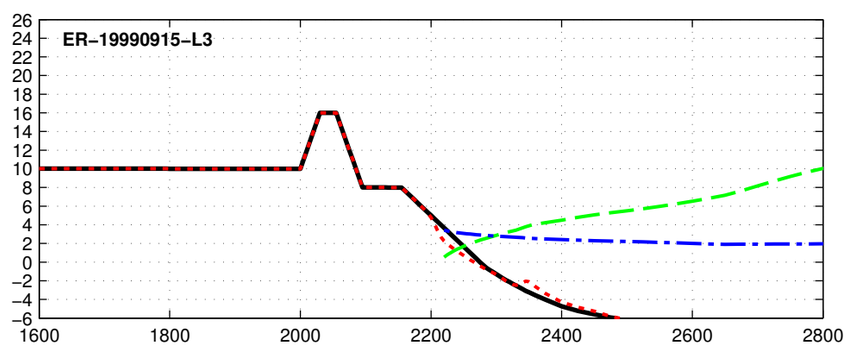


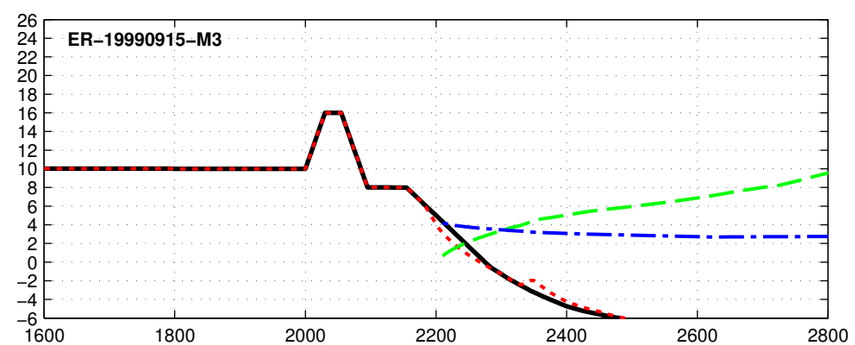


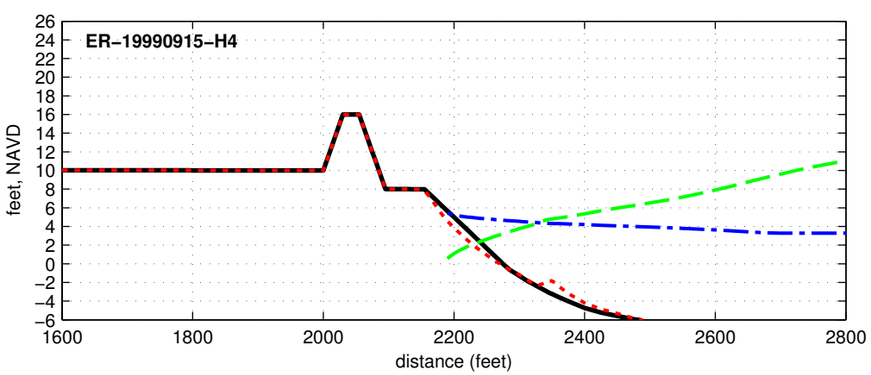


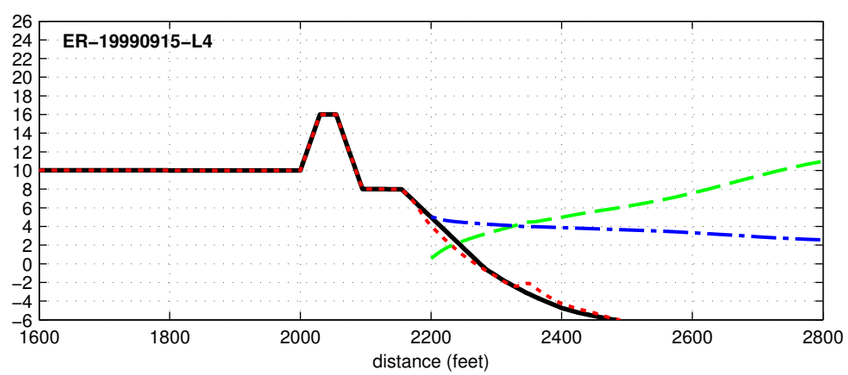


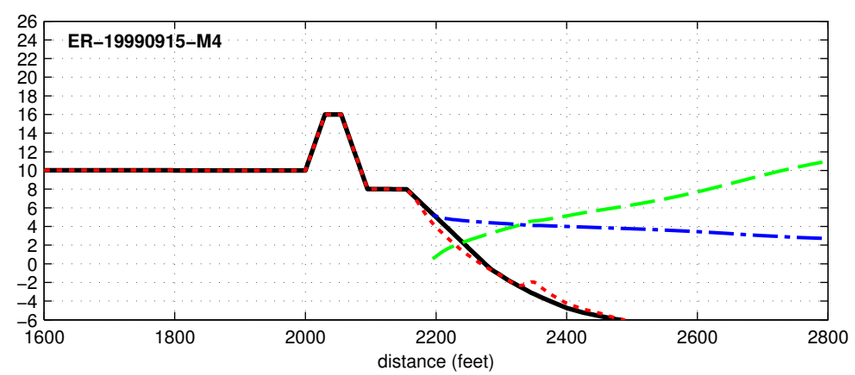






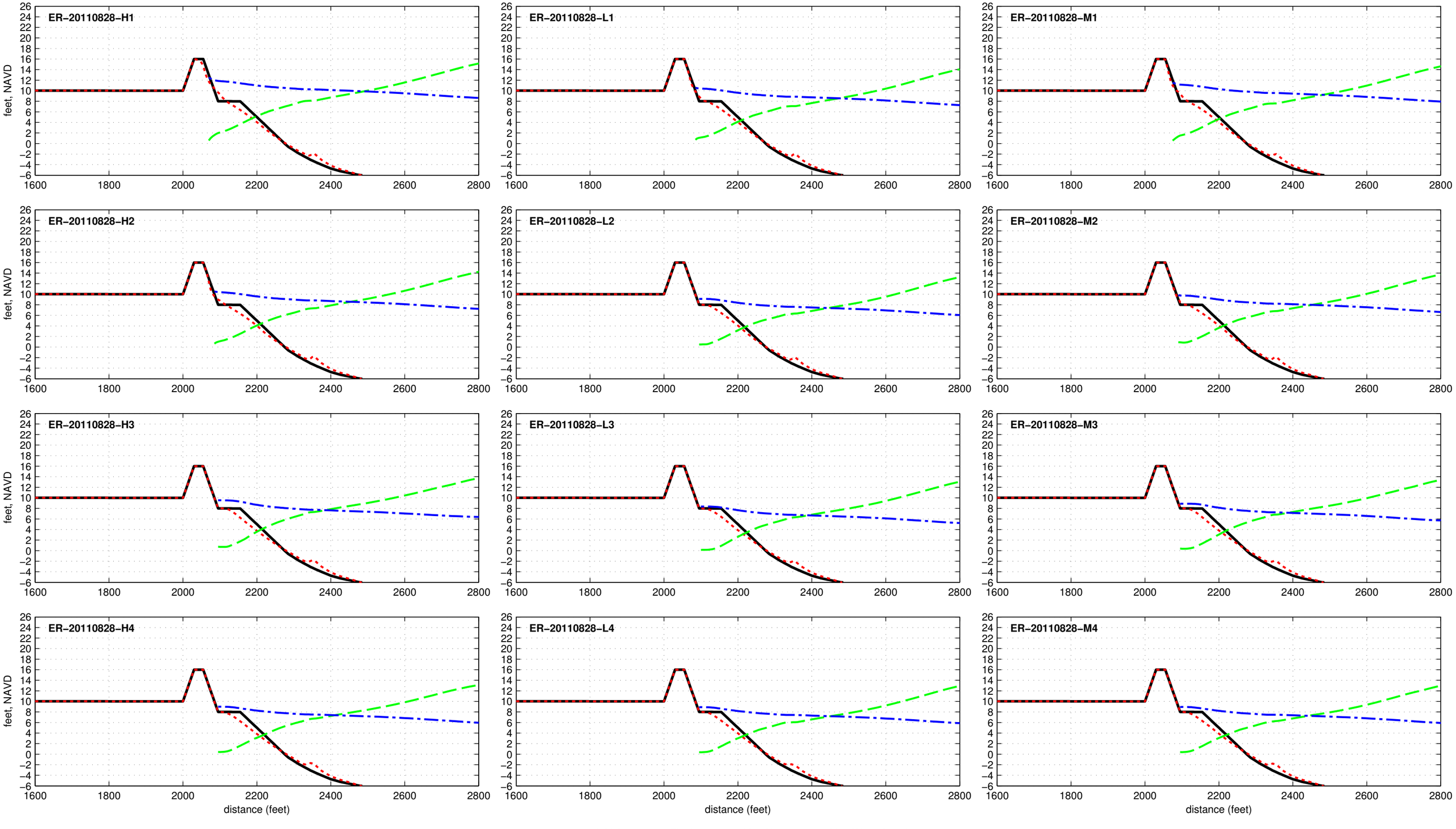




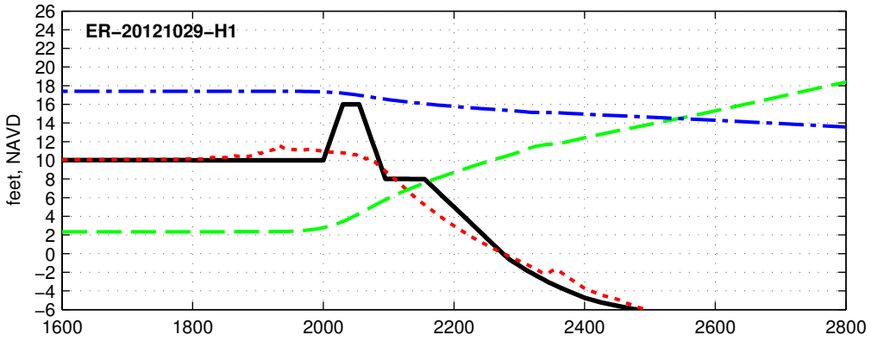


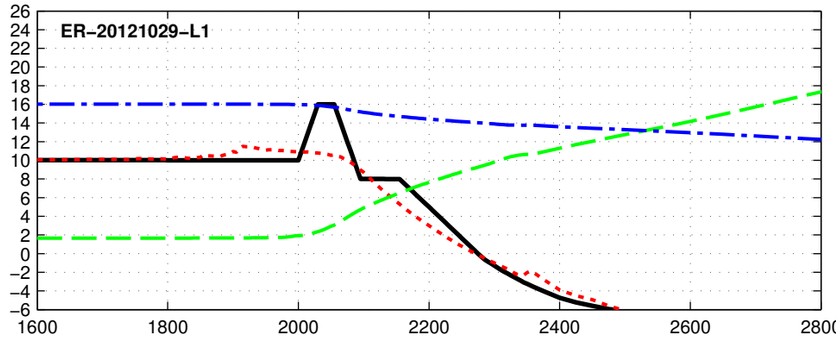
Initial Final Max Hs Max Water Level

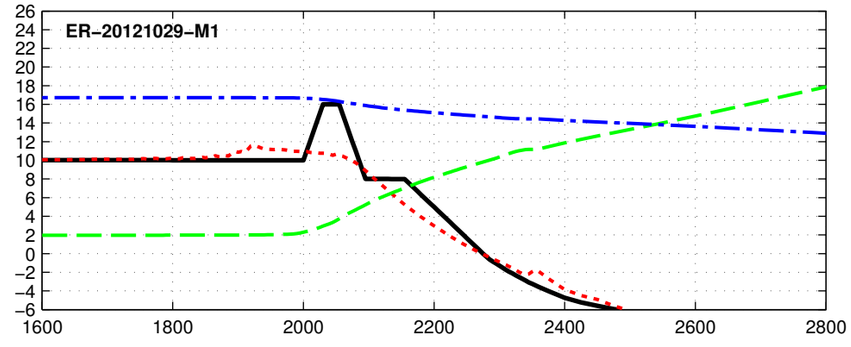
Dune R2T1-16.0-025-060 - ER-20110828

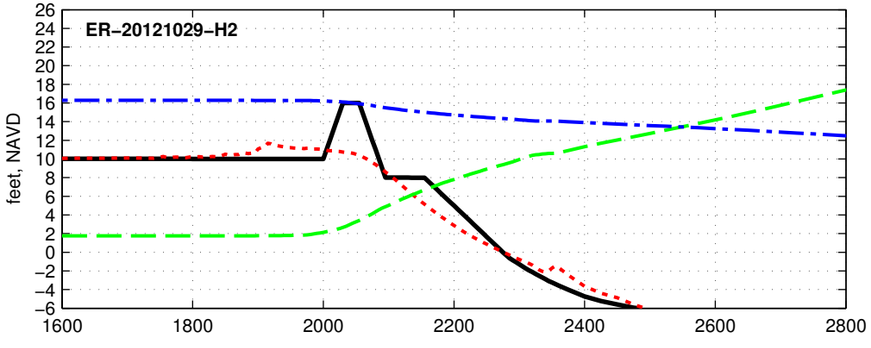


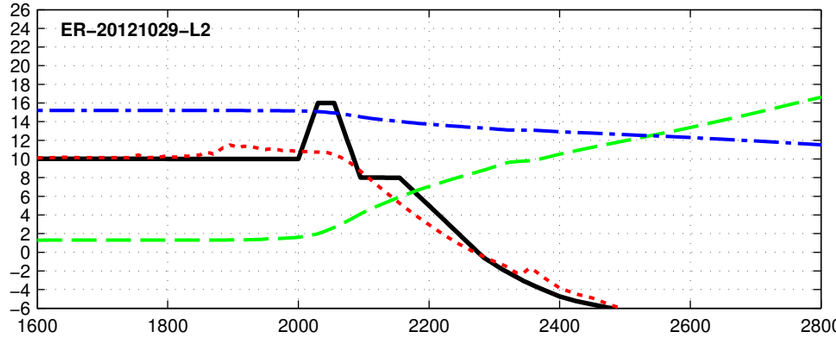
Dune R2T1-16.0-025-060 - ER-20121029

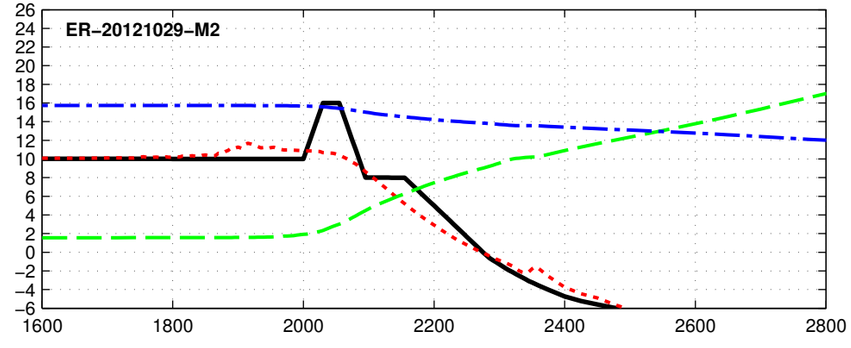


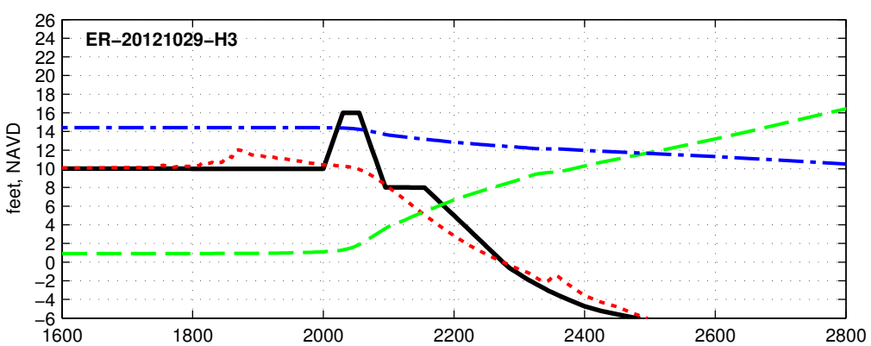


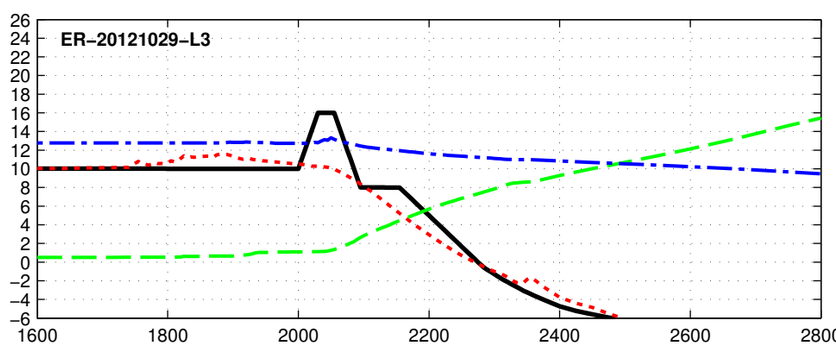


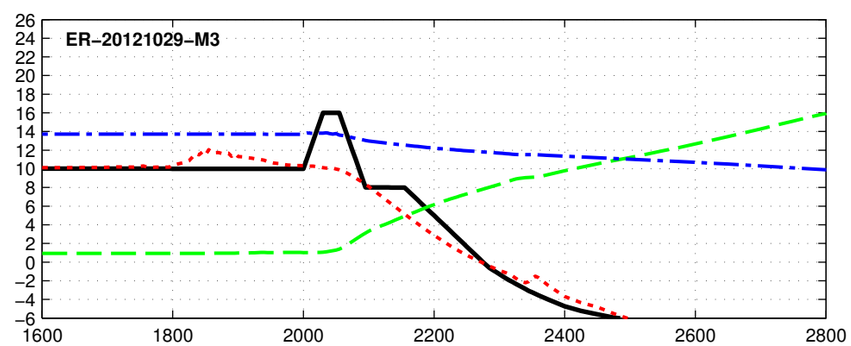


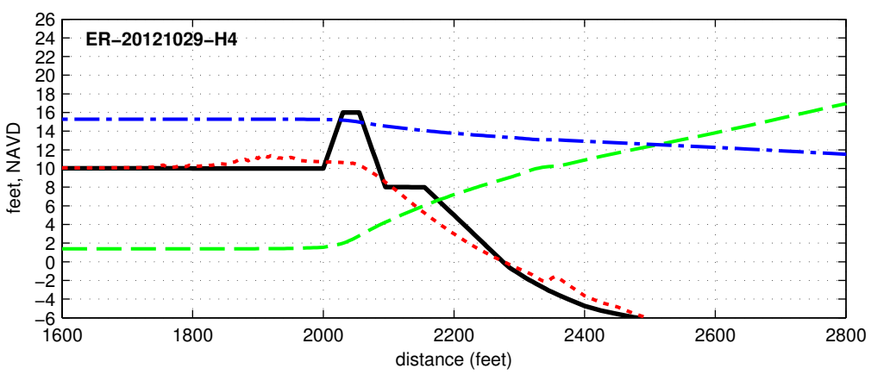


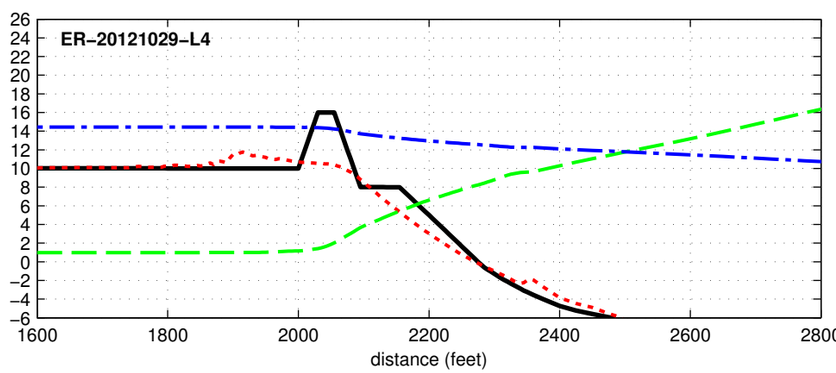


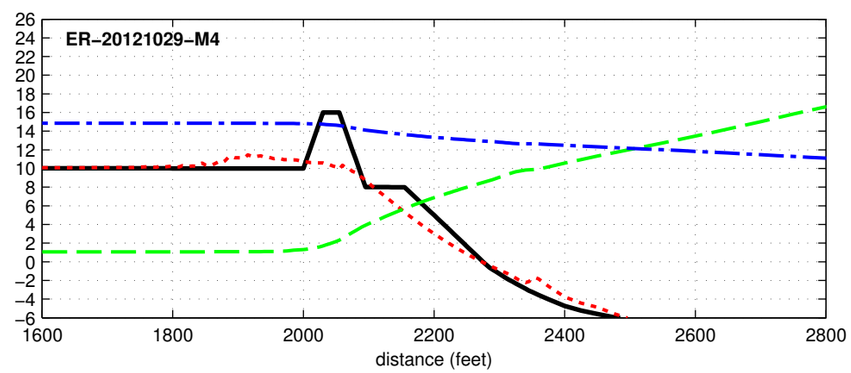








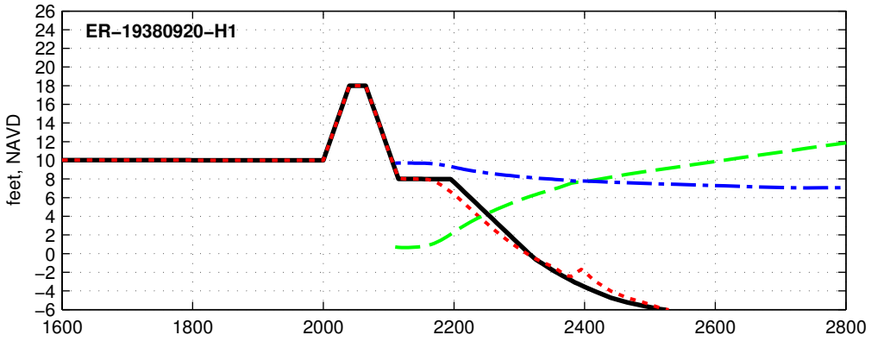


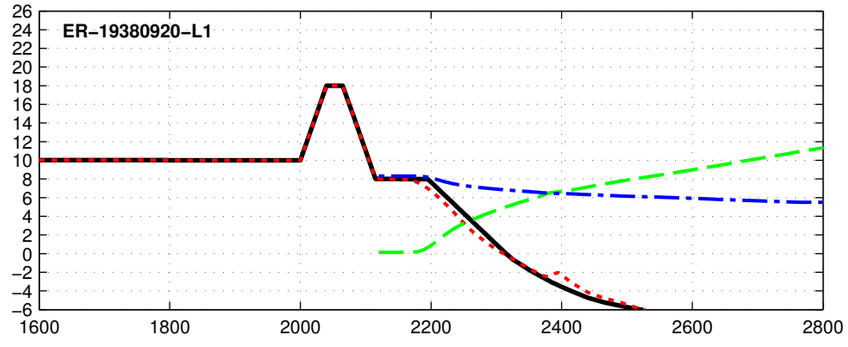


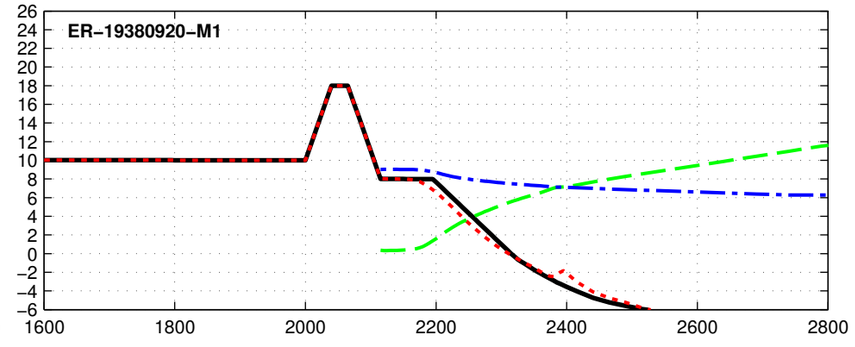
Initial Final Max Hs Max Water Level

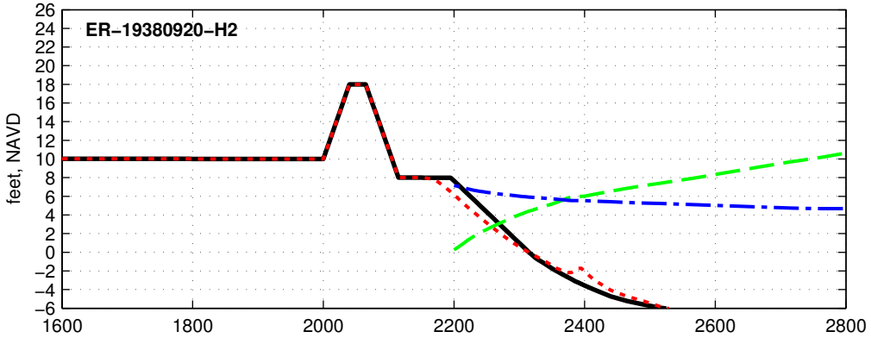
R2T1_18.0_025_080 - DUNE

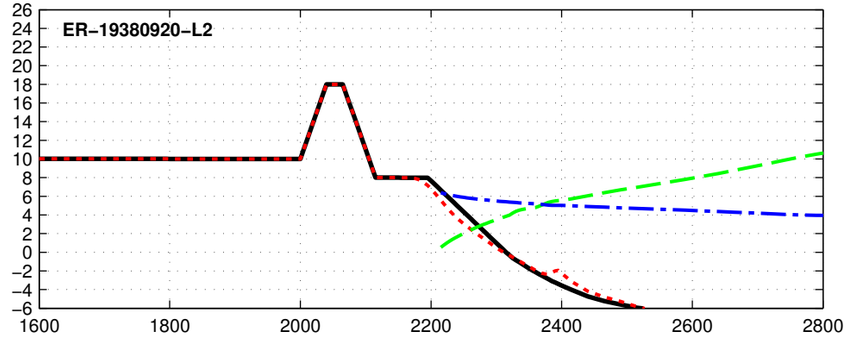
Dune R2T1-18.0-025-080 - ER-19380920

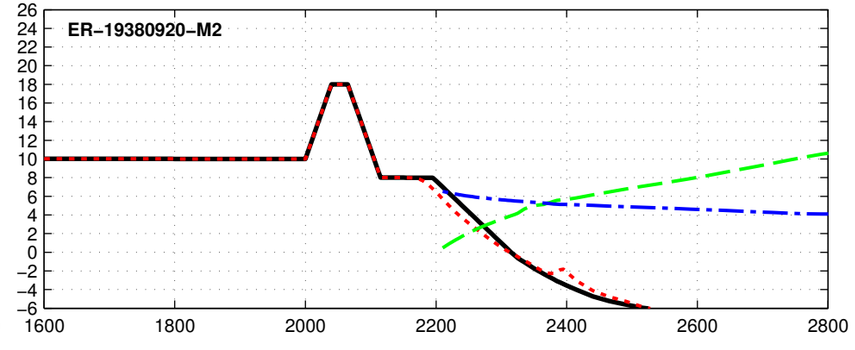


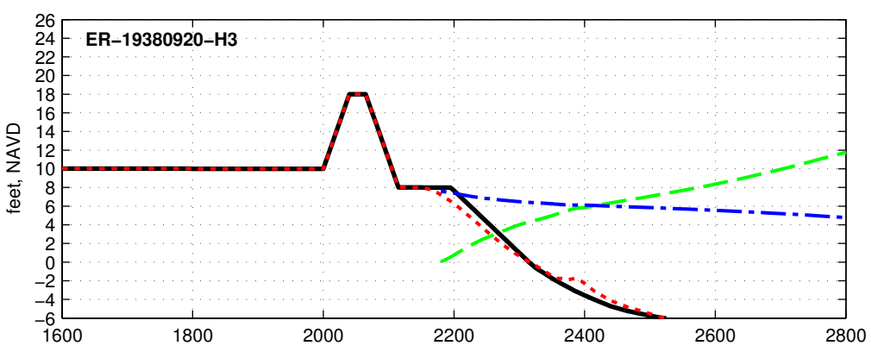


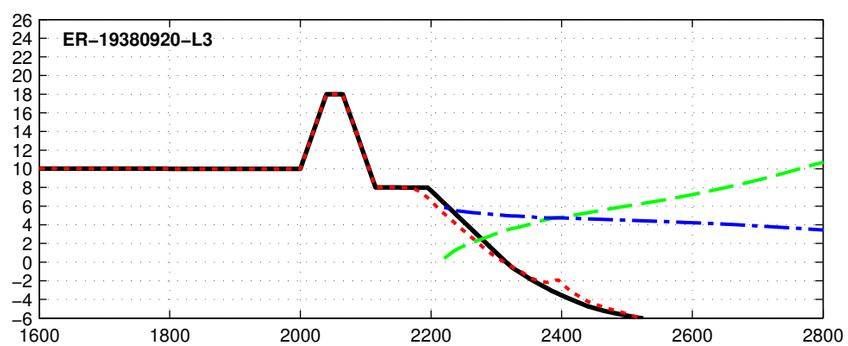


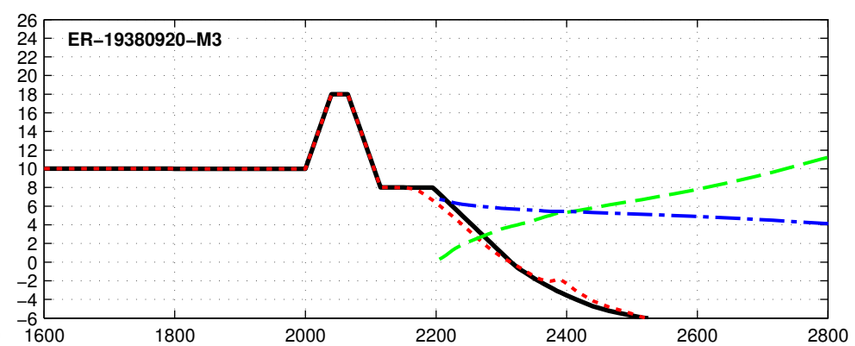


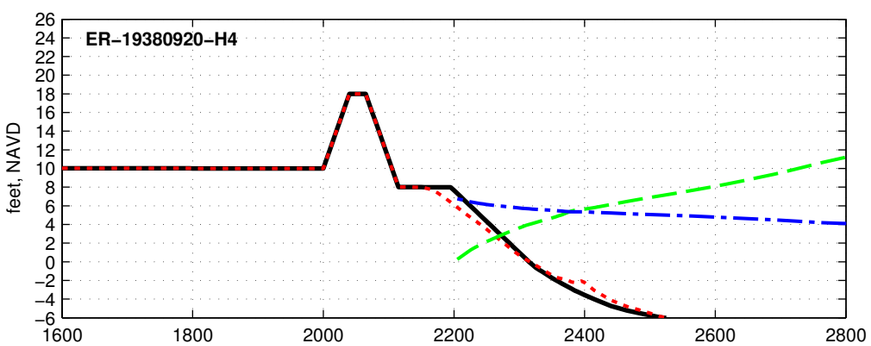


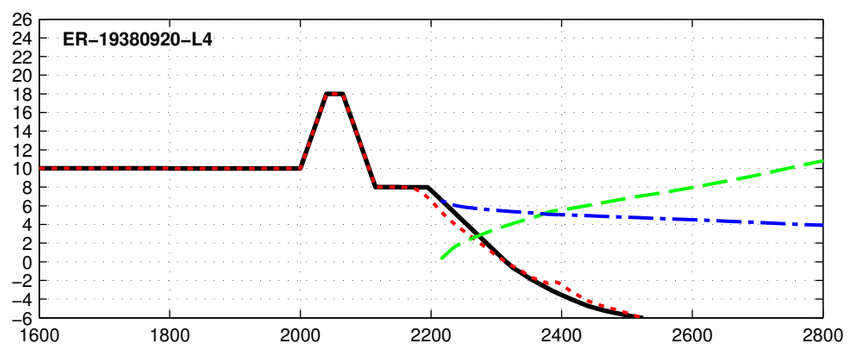


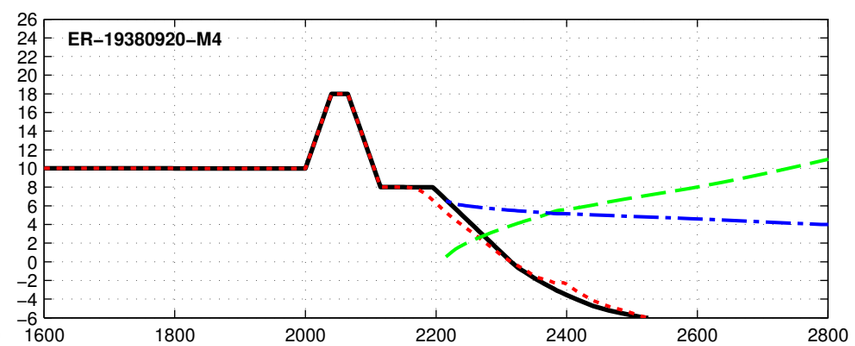






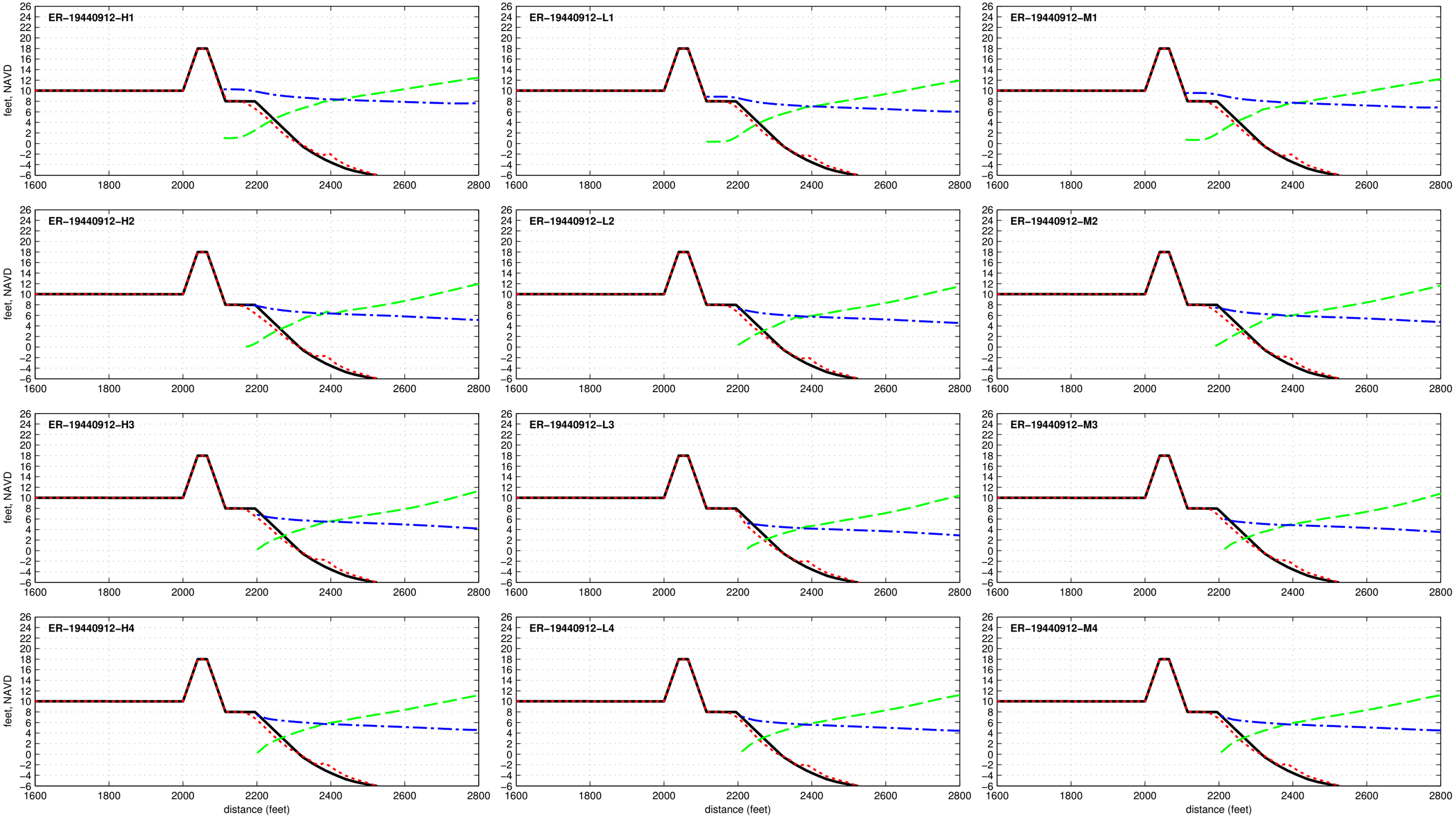






Initial Final Max Hs Max Water Level

Dune R2T1-18.0-025-080 - ER-19440912



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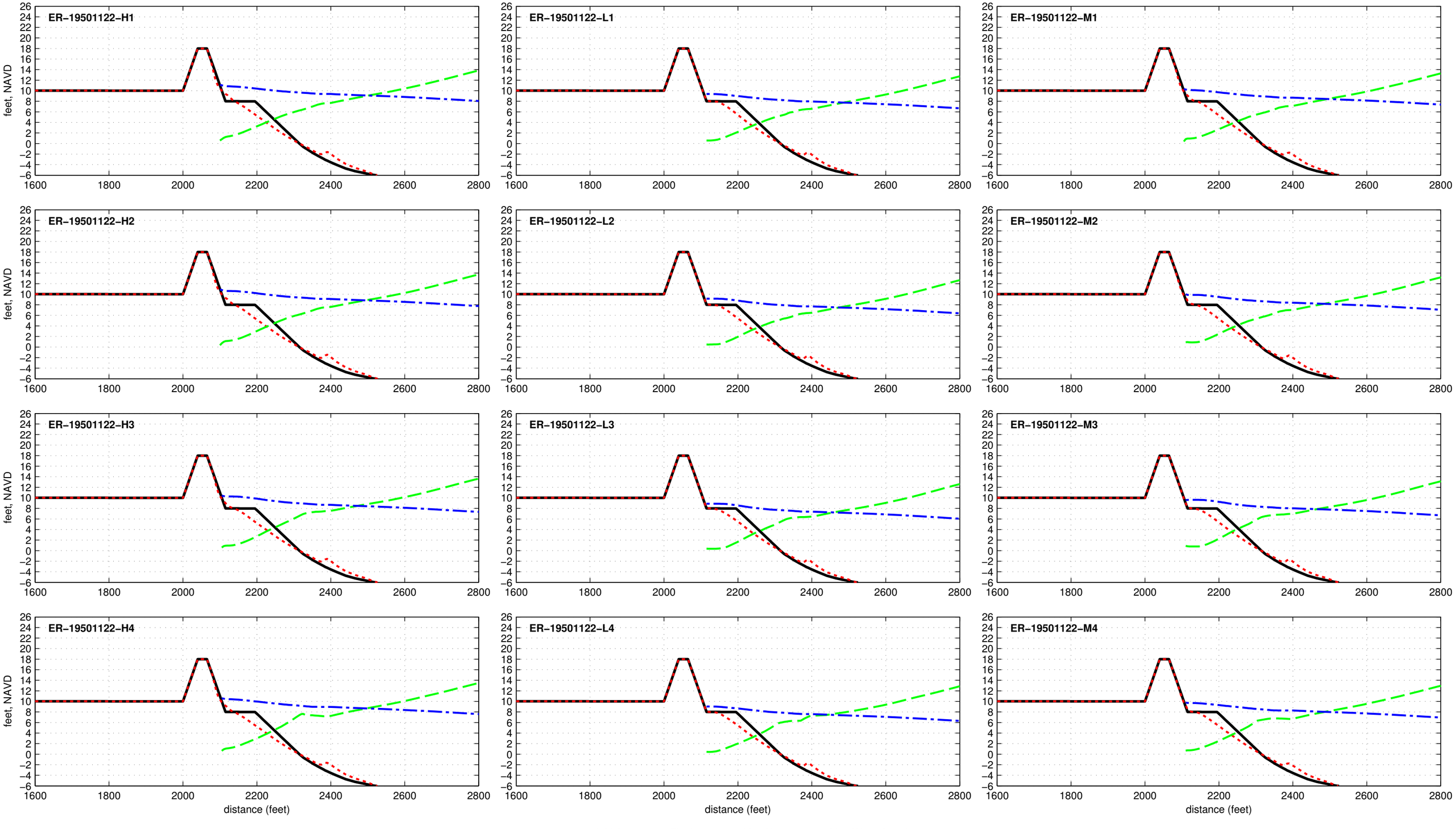
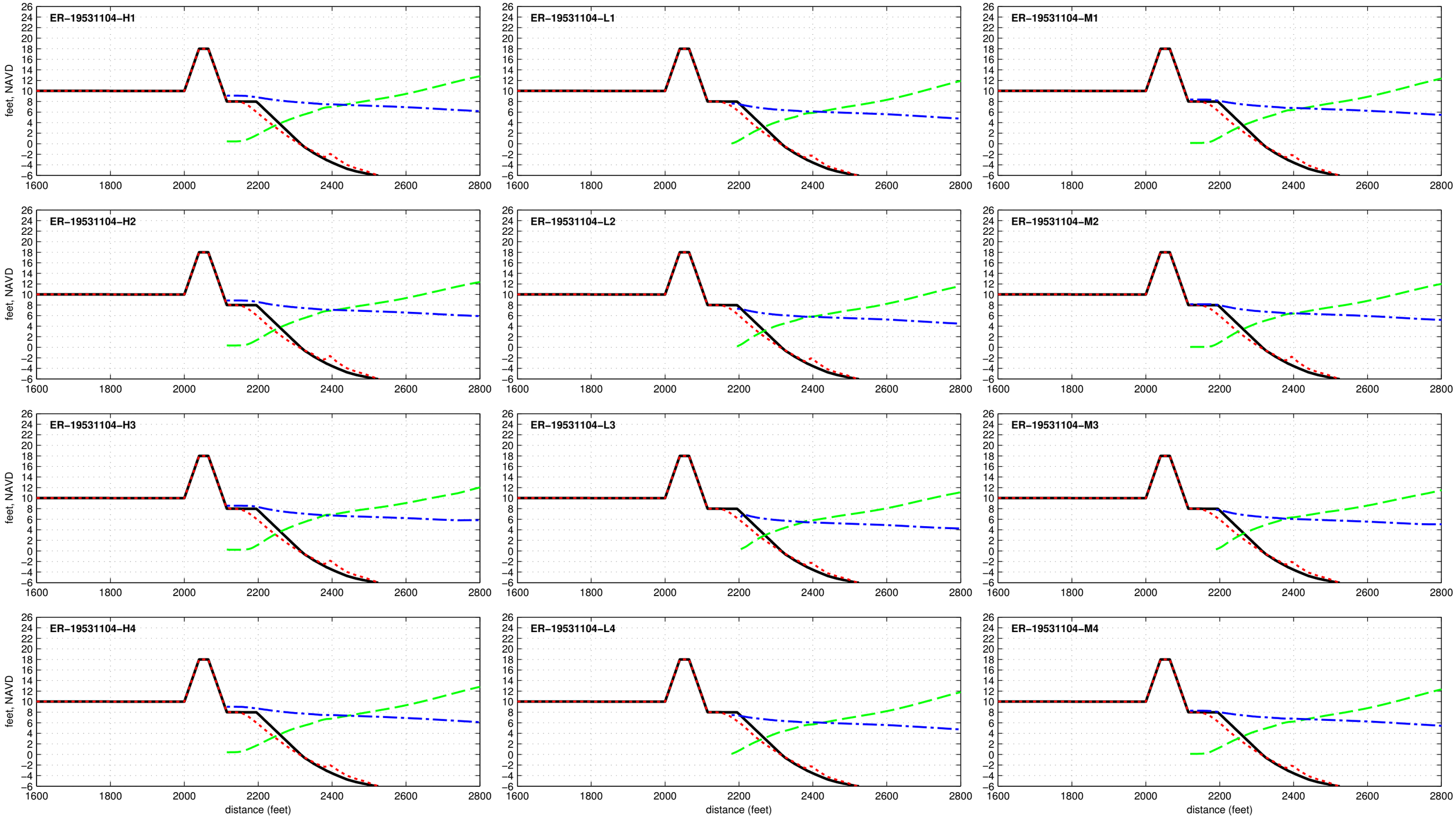


Dune R2T1-18.0-025-080 - ER-19531104



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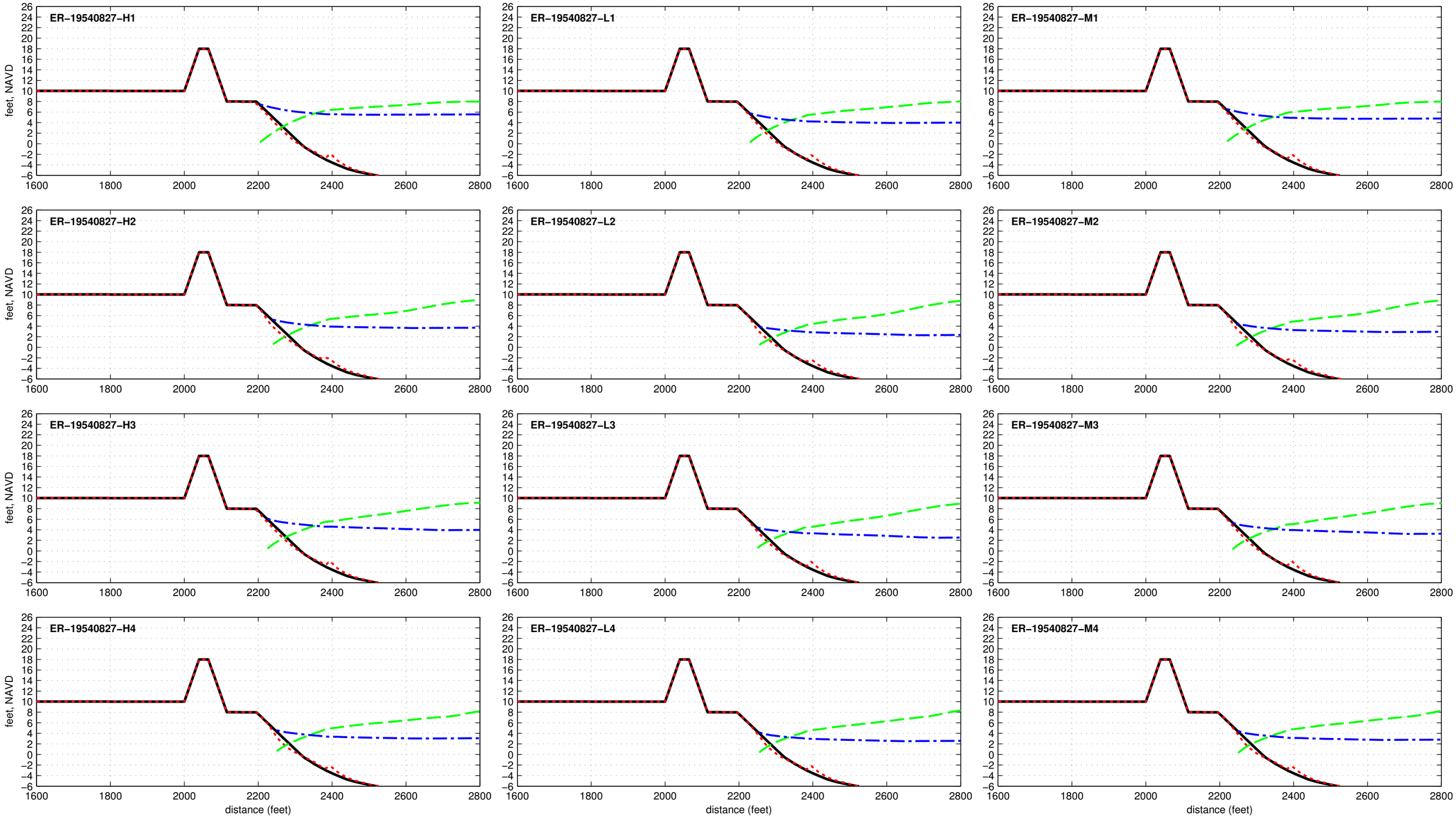
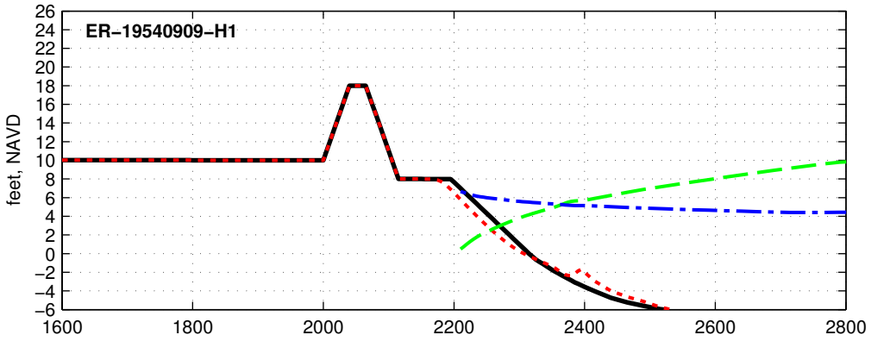
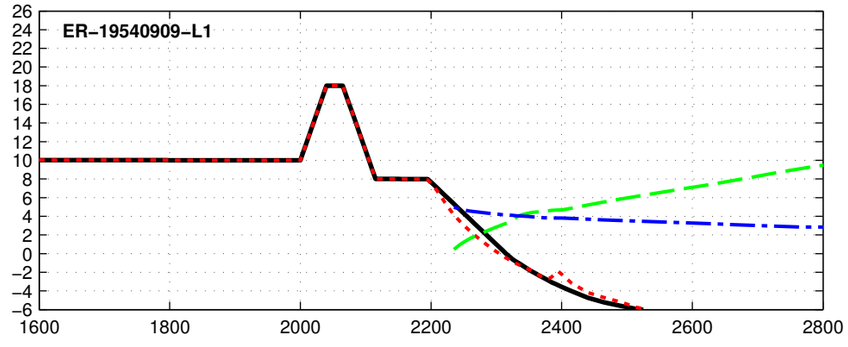
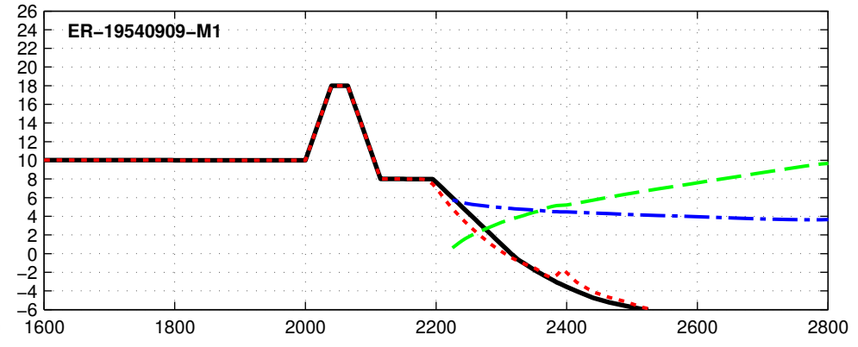
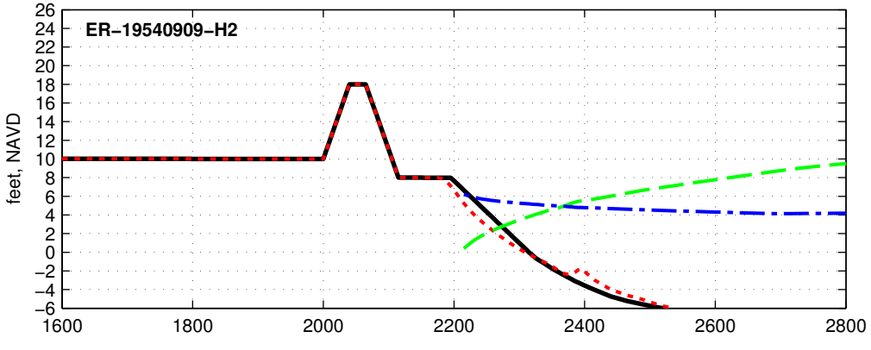


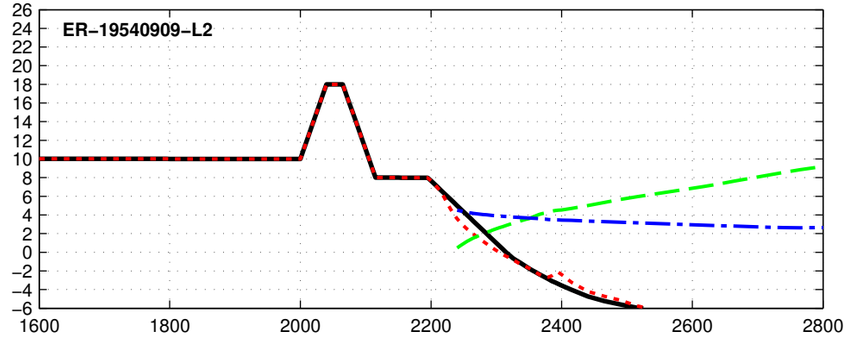
Dune R2T1-18.0-025-080 - ER-19540909

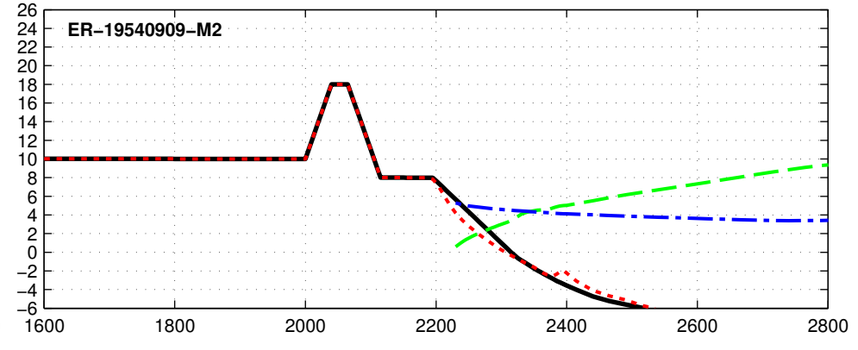


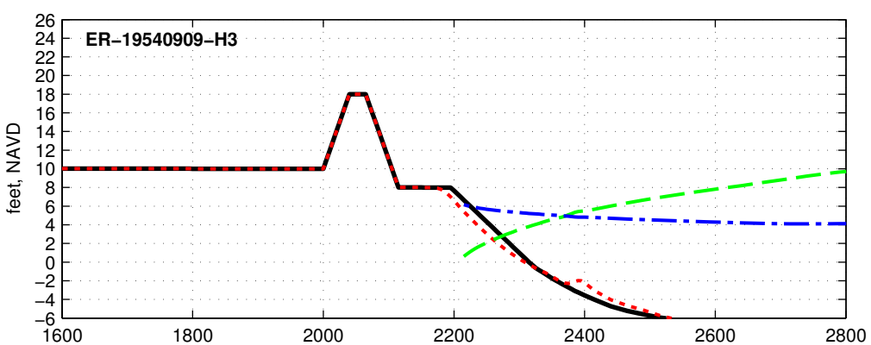


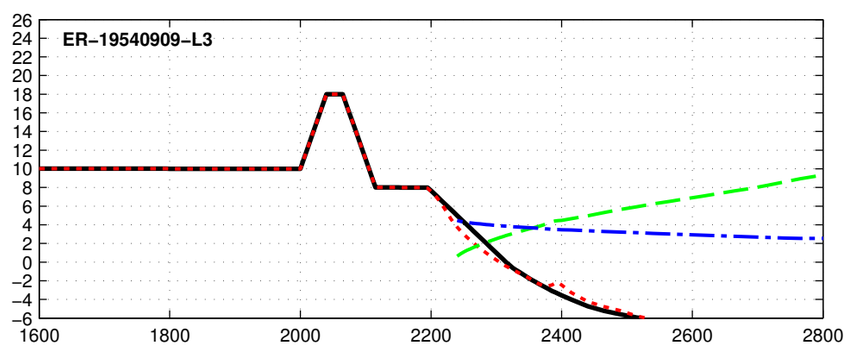


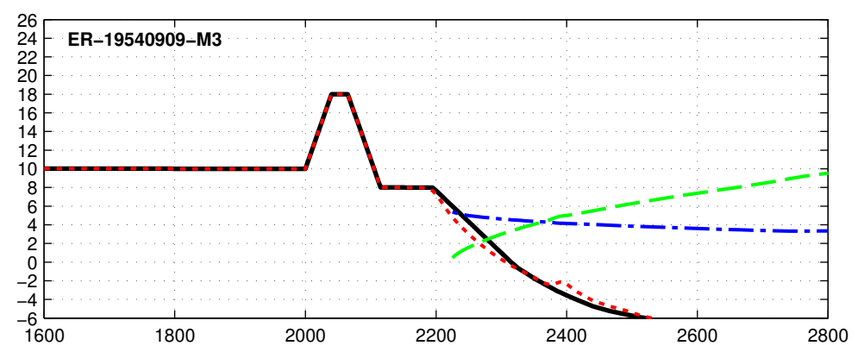


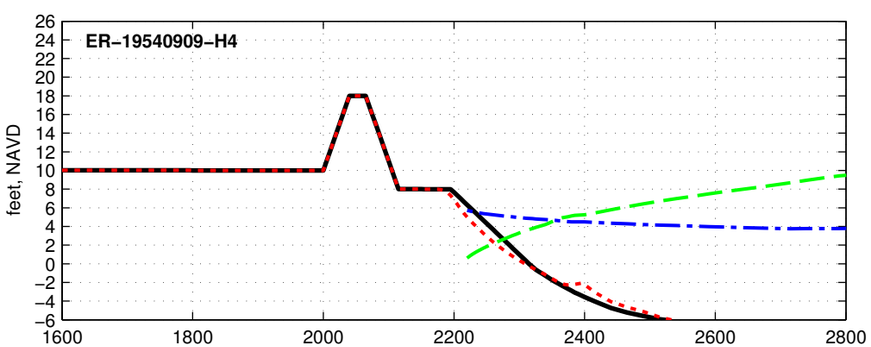


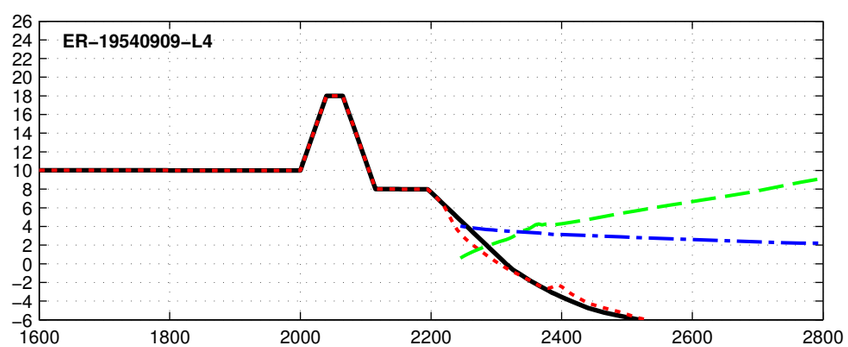


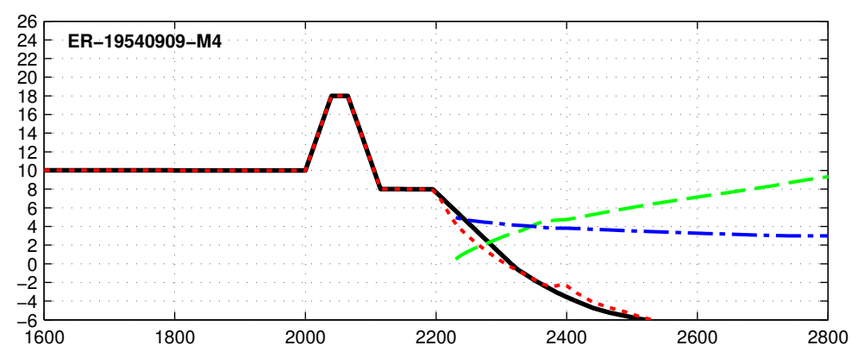






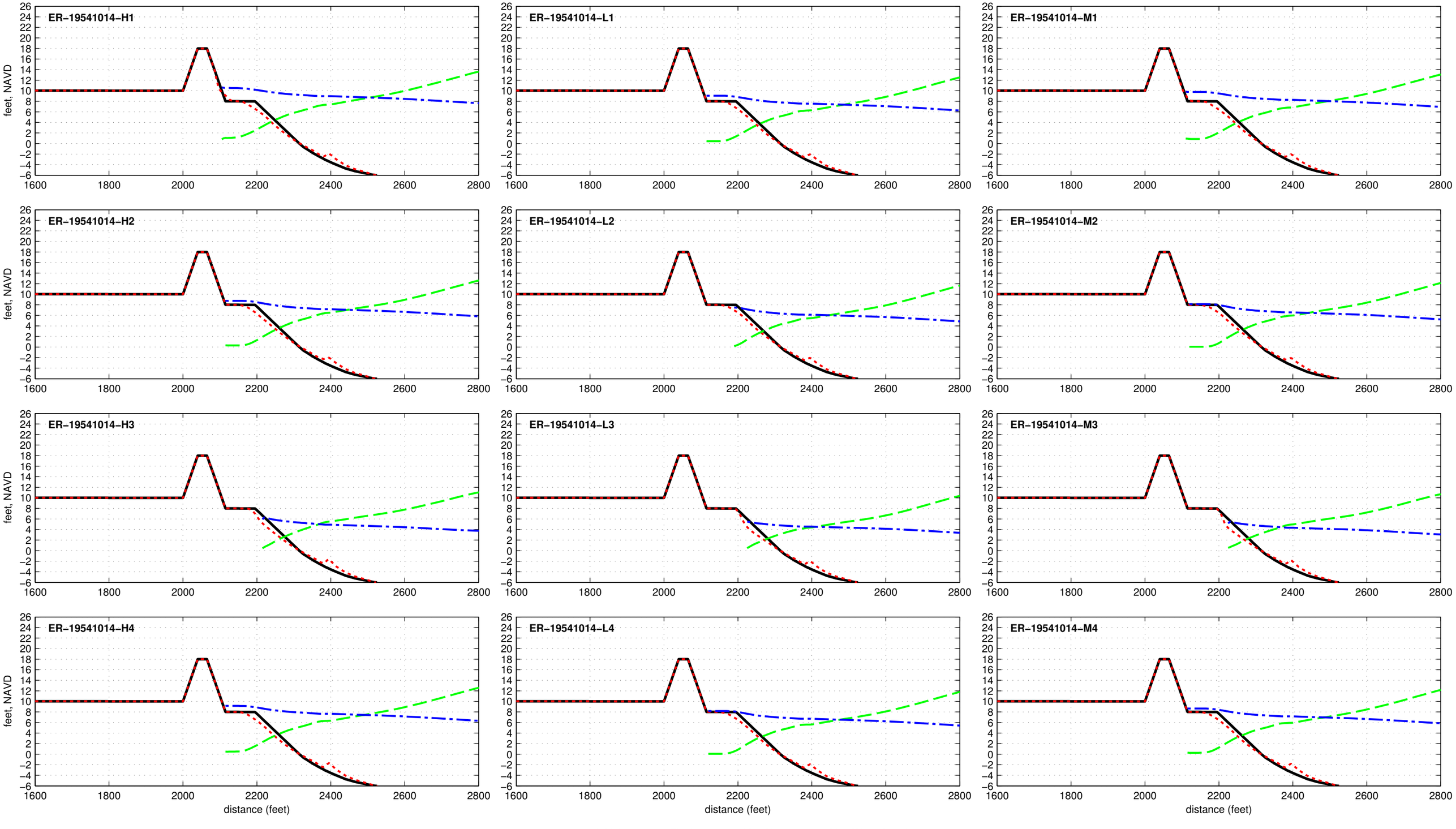




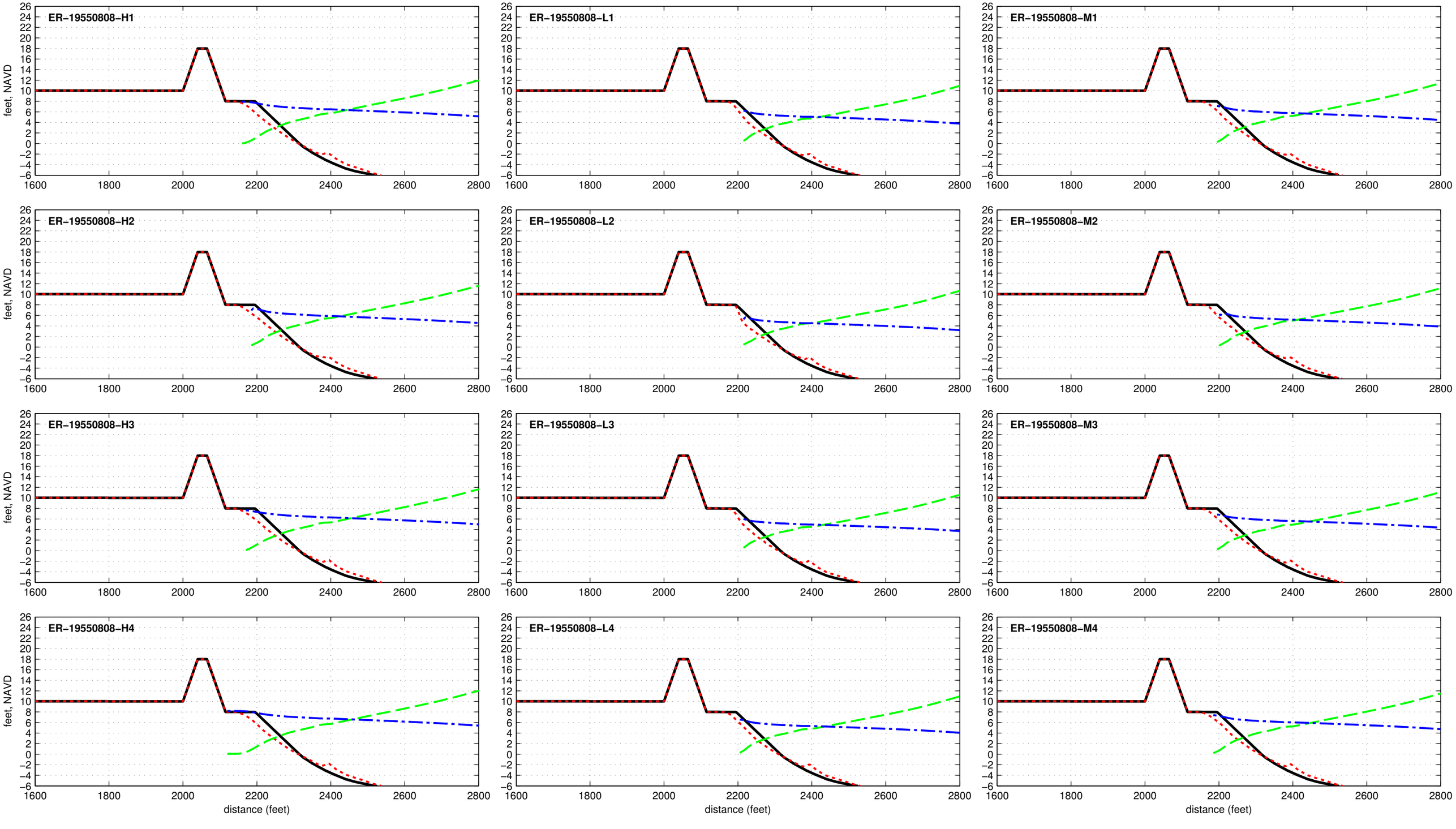


Initial Final Max Hs Max Water Level

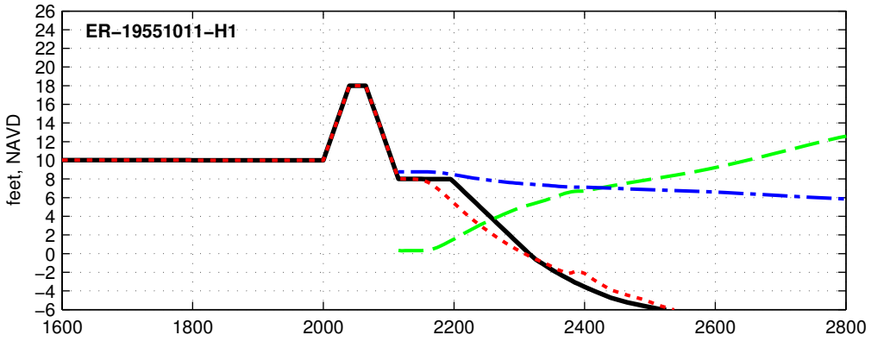
Dune R2T1-18.0-025-080 - ER-19541014

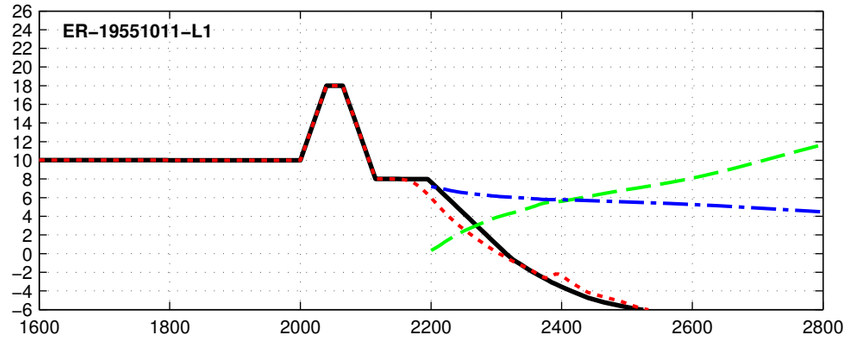


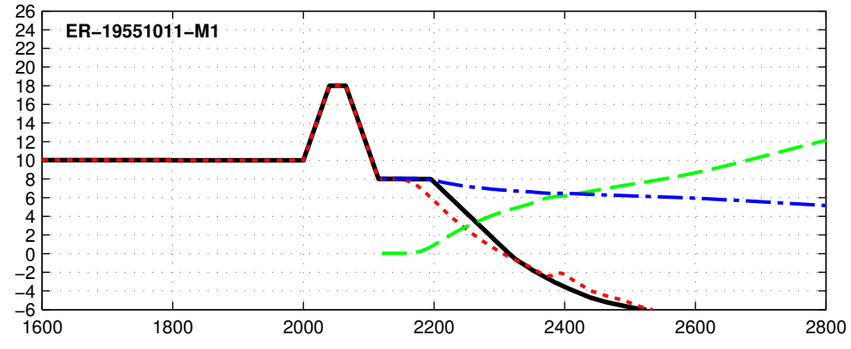
Dune R2T1-18.0-025-080 - ER-19550808

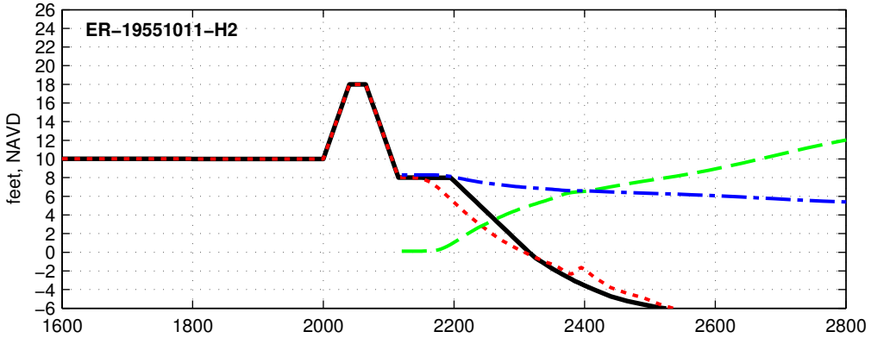


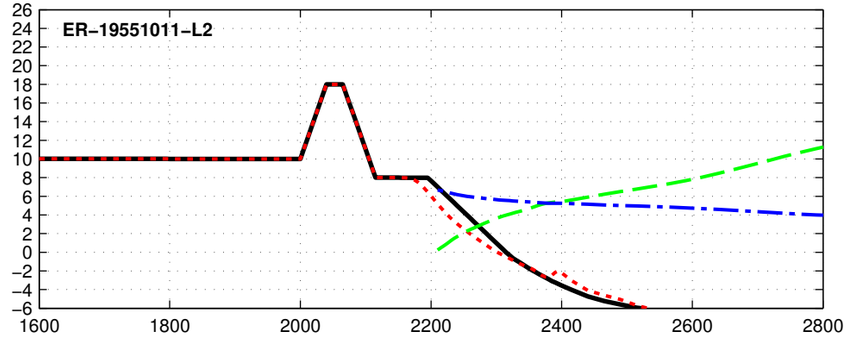
Dune R2T1-18.0-025-080 - ER-19551011

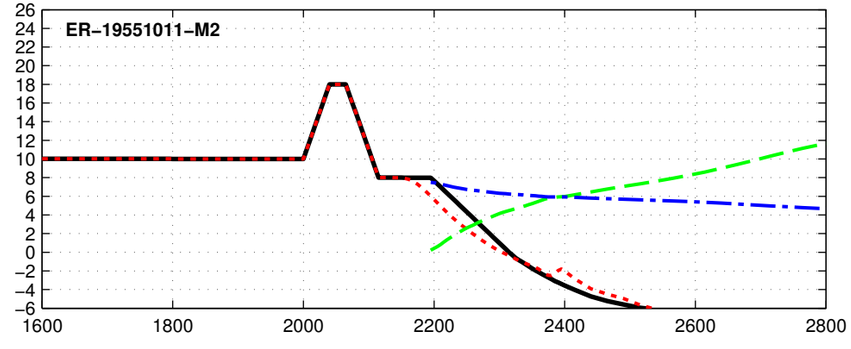


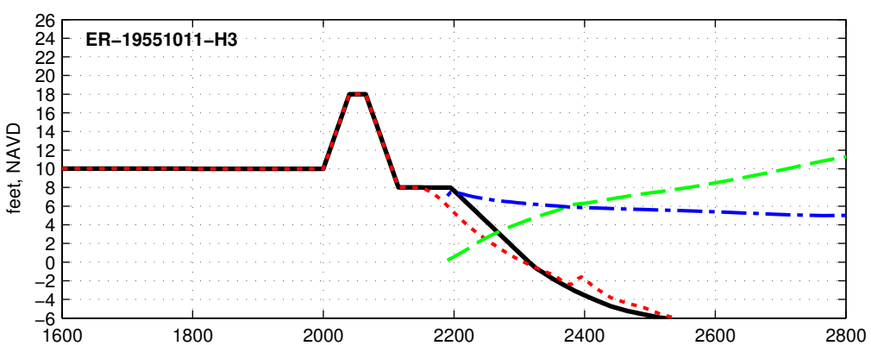


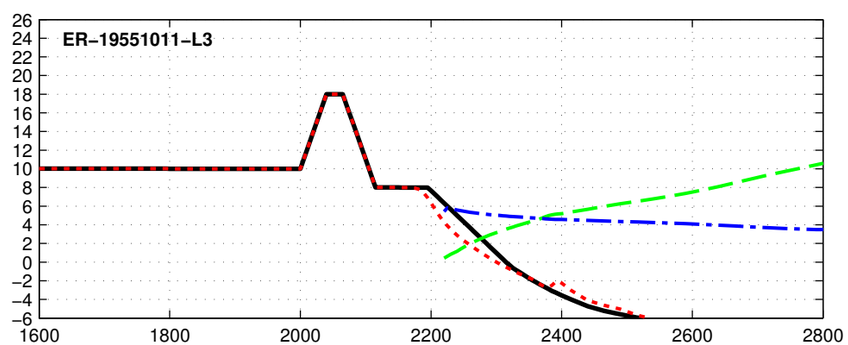


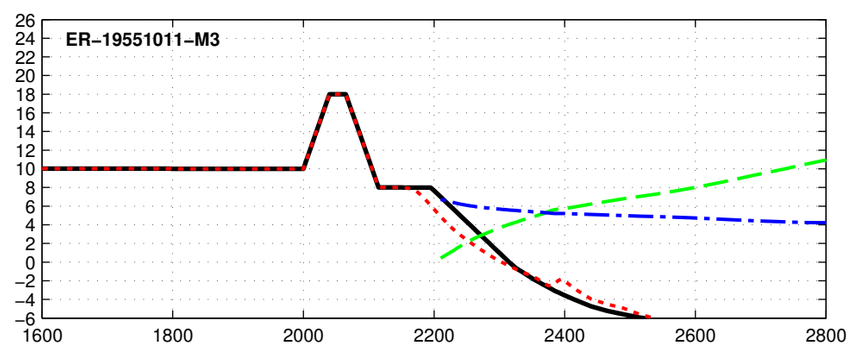


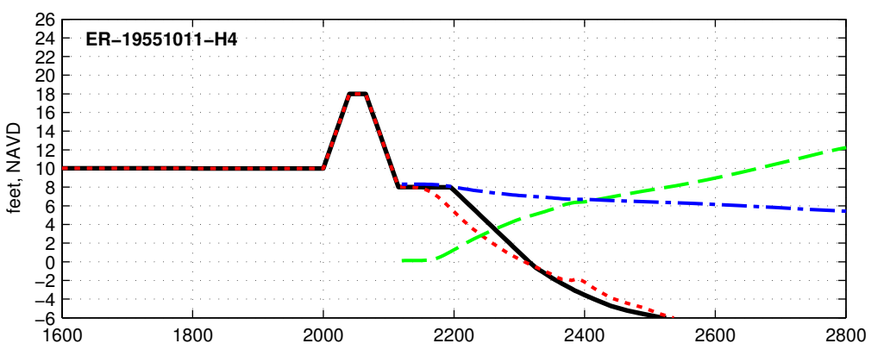


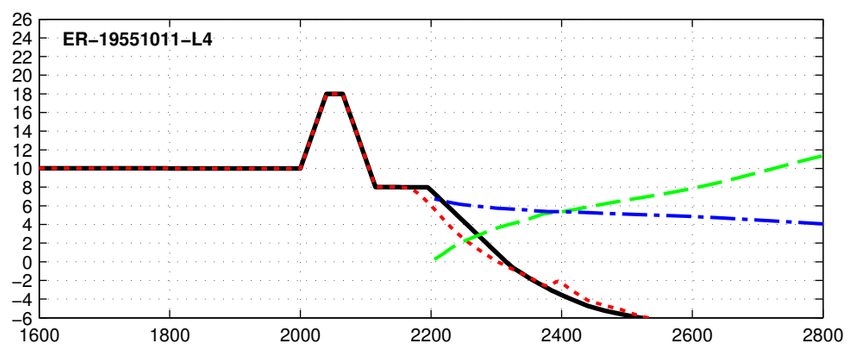


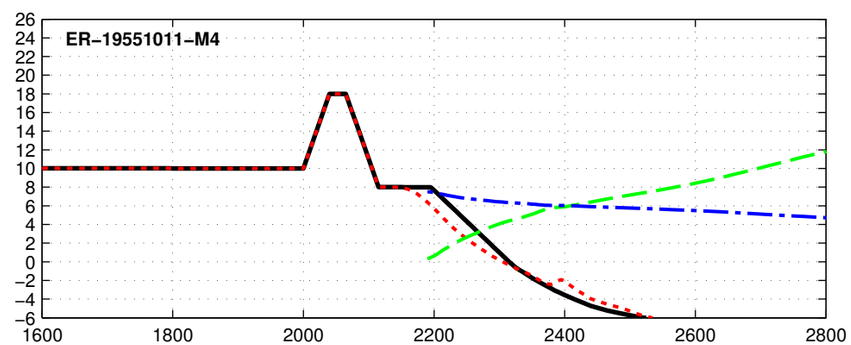






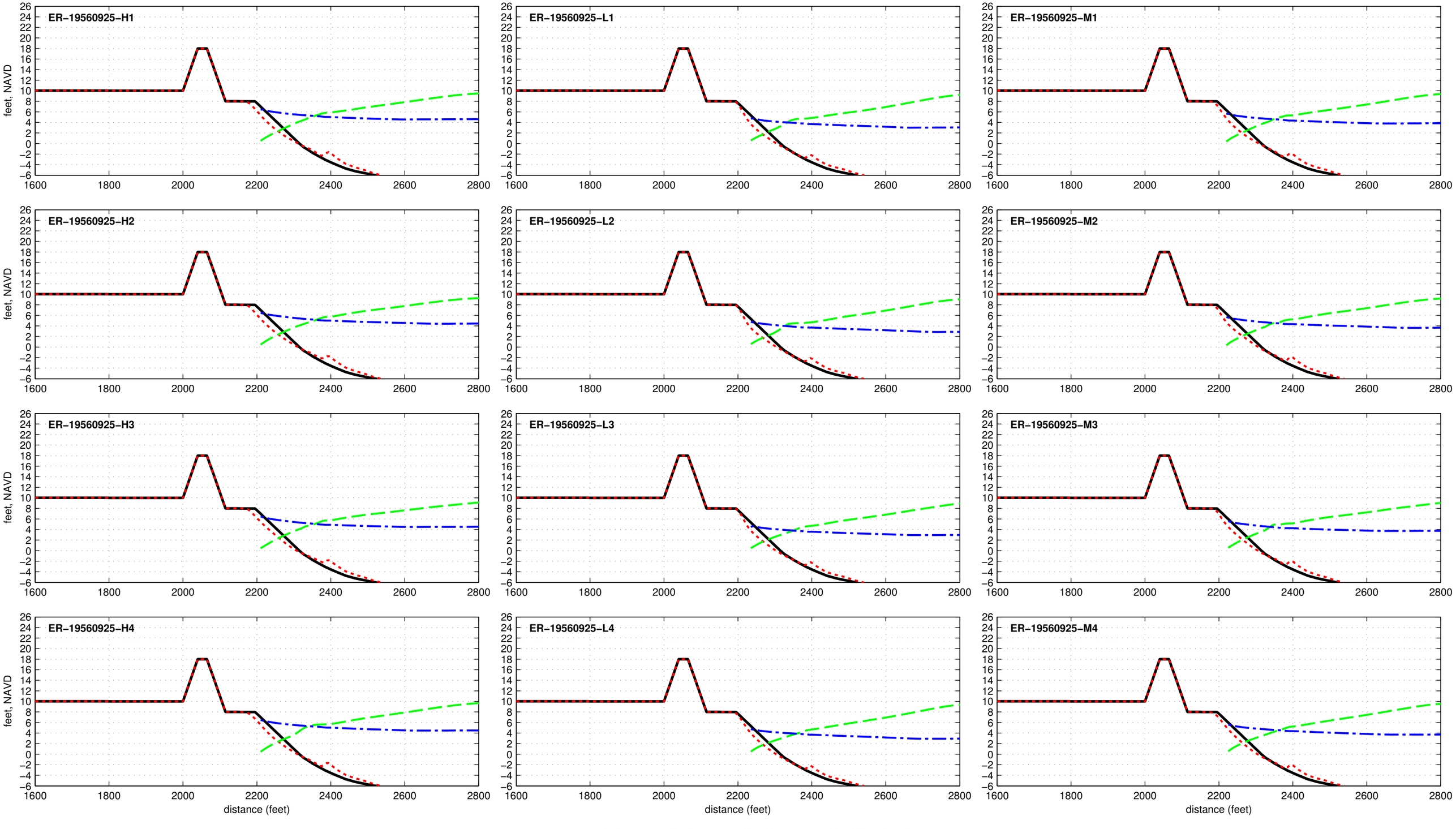




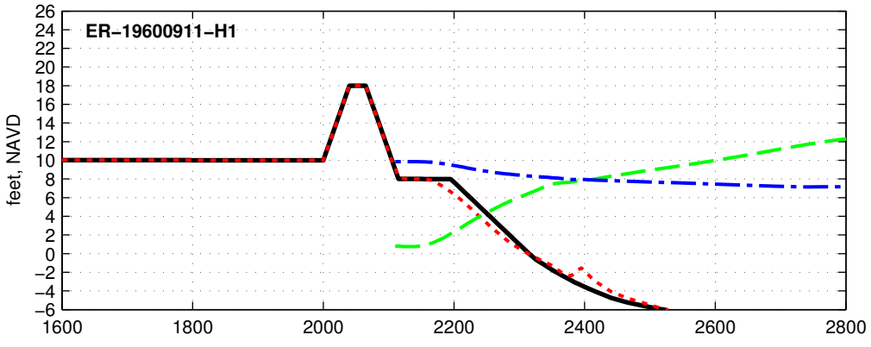


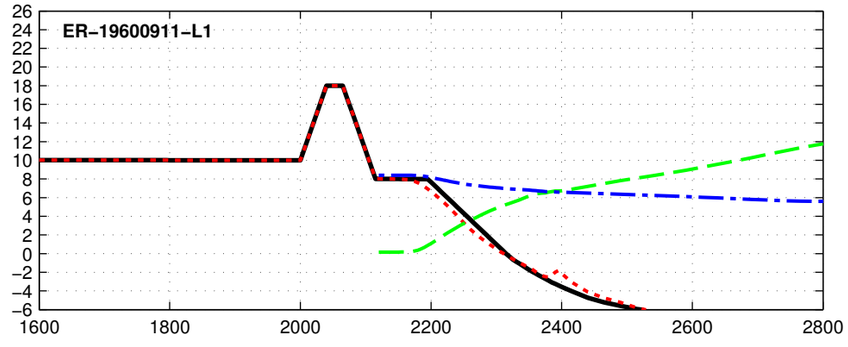
Initial Final Max Hs Max Water Level

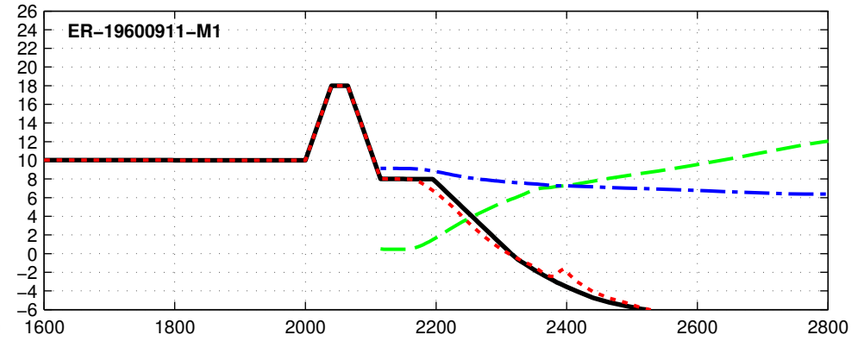
Dune R2T1-18.0-025-080 - ER-19560925

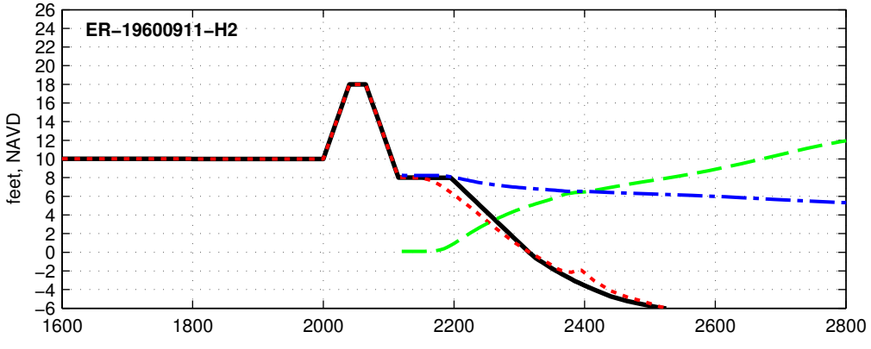


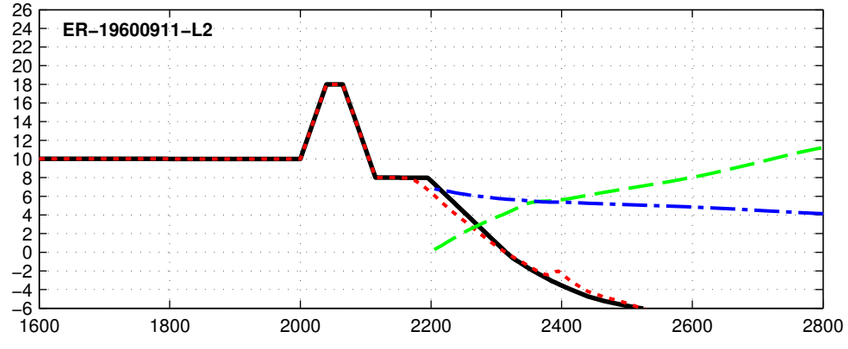
Dune R2T1-18.0-025-080 - ER-19600911

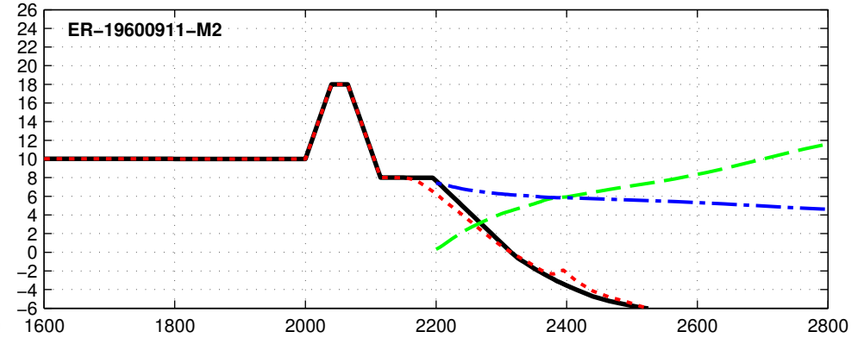


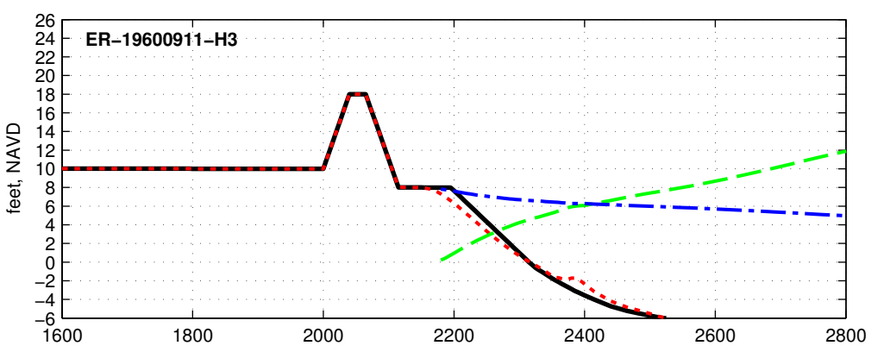


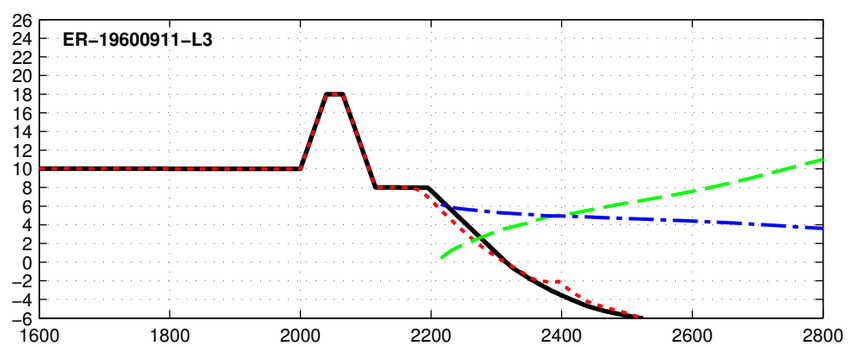


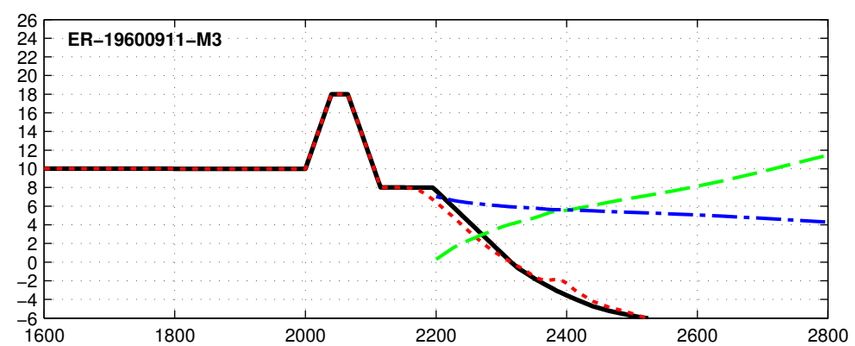


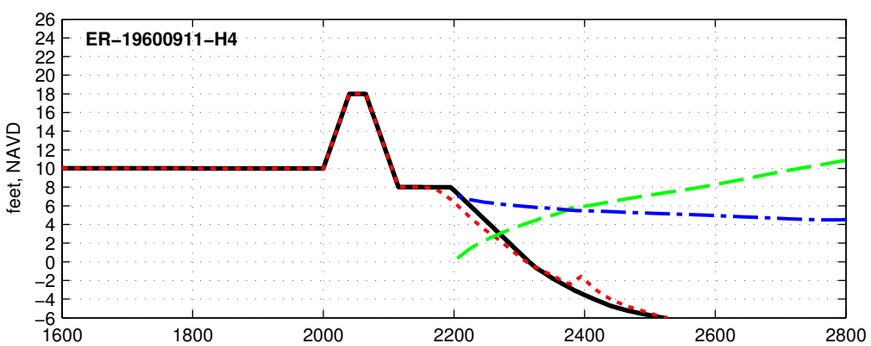


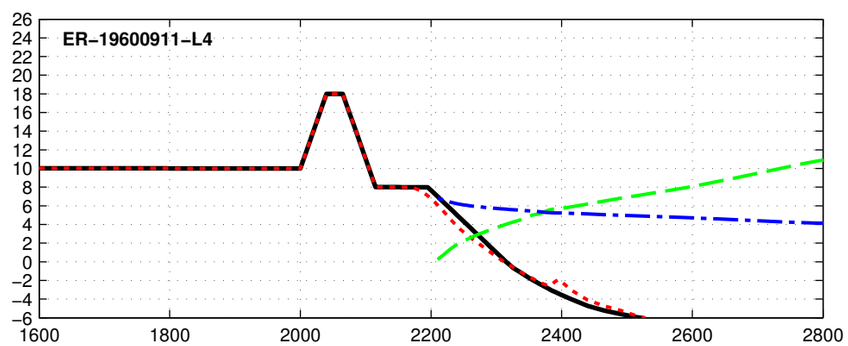


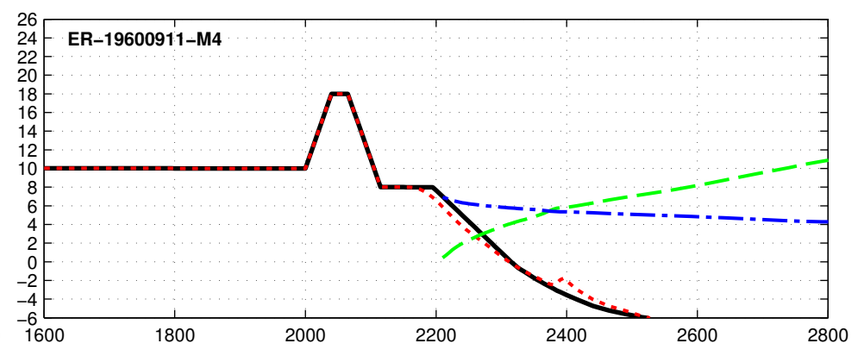






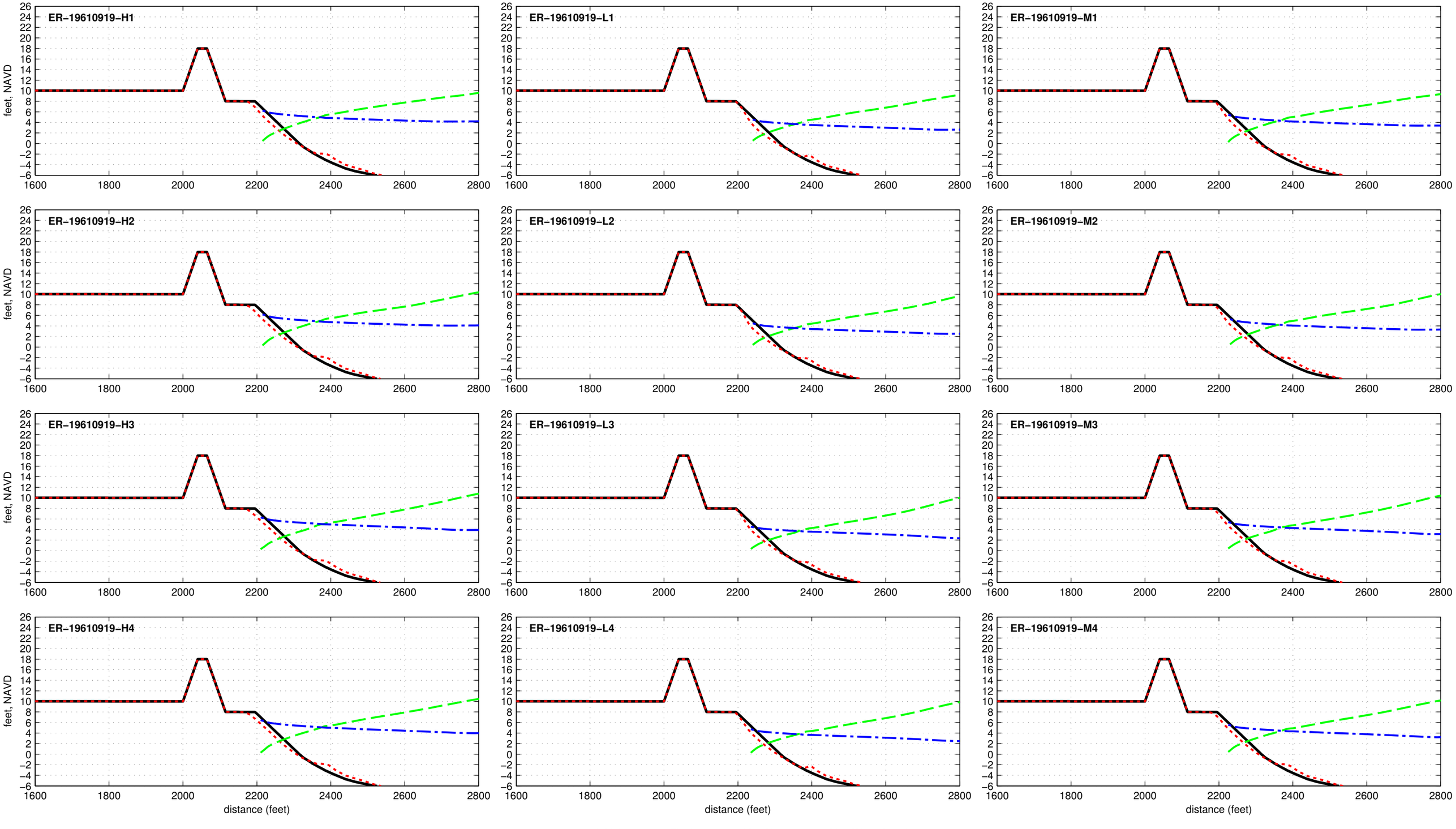




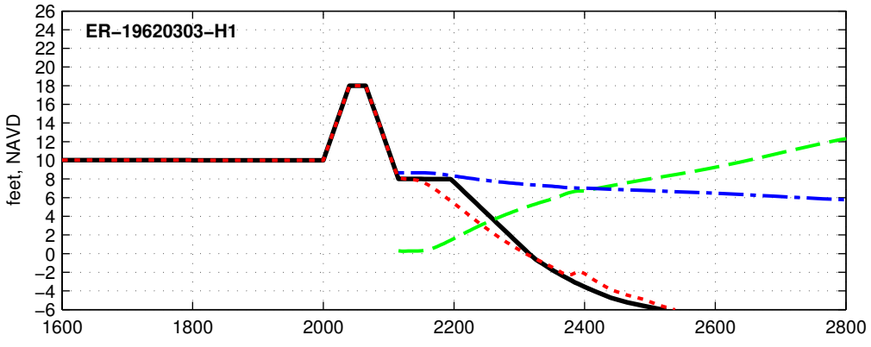


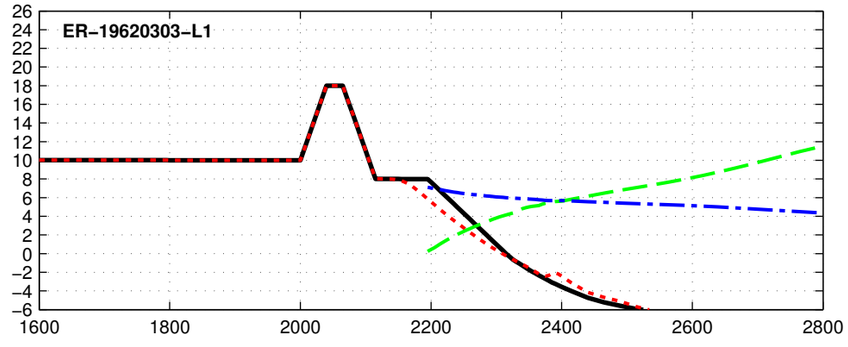
Initial Final Max Hs Max Water Level

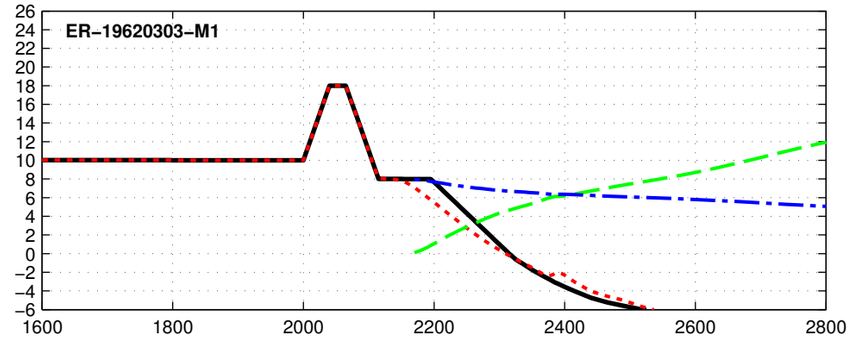
Dune R2T1-18.0-025-080 - ER-19610919

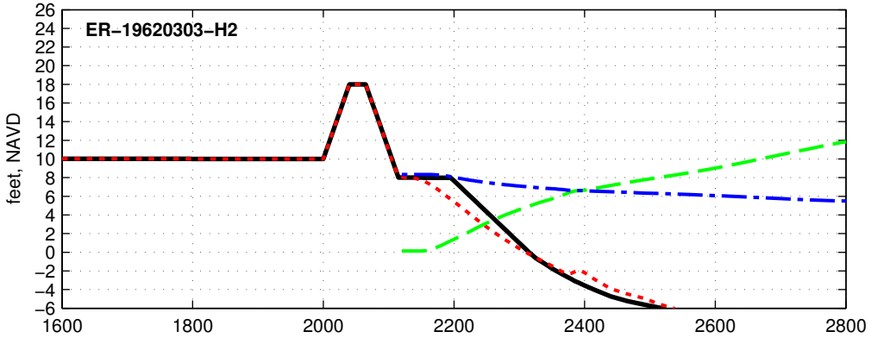


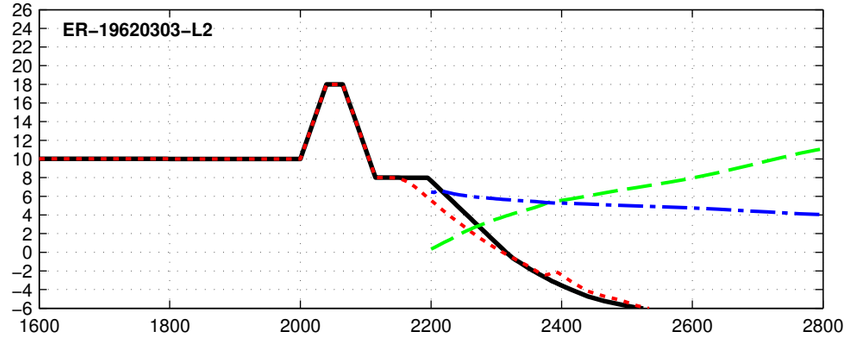
Dune R2T1-18.0-025-080 - ER-19620303

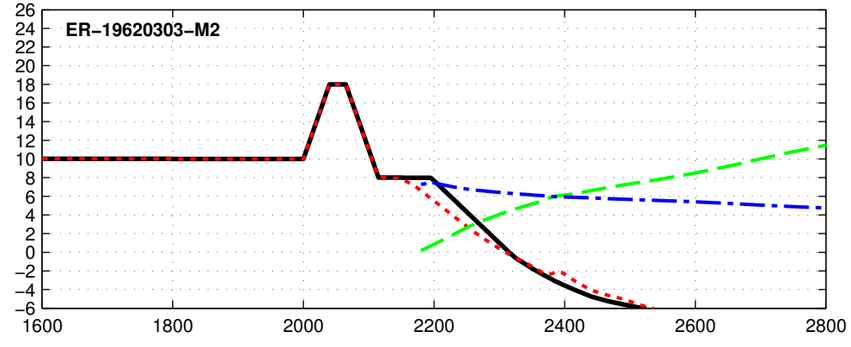


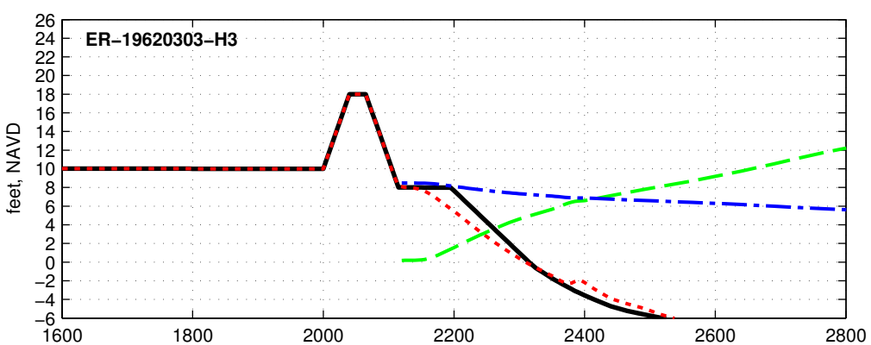


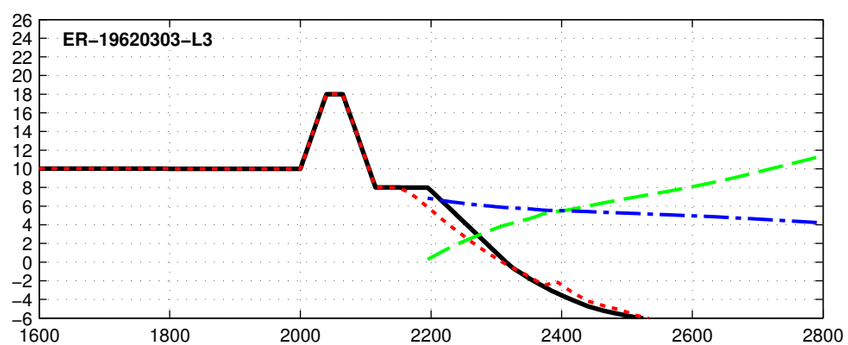


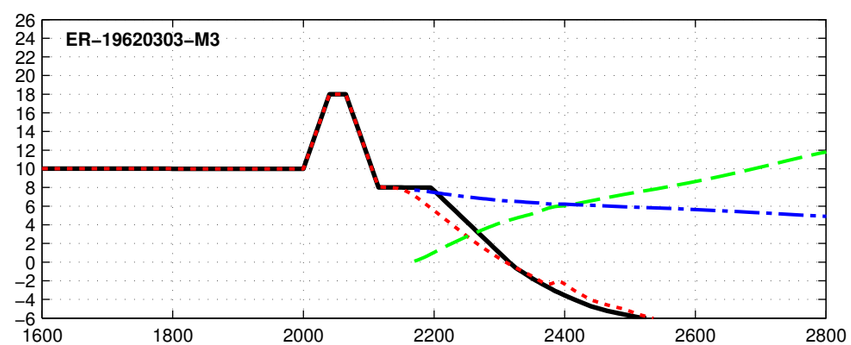


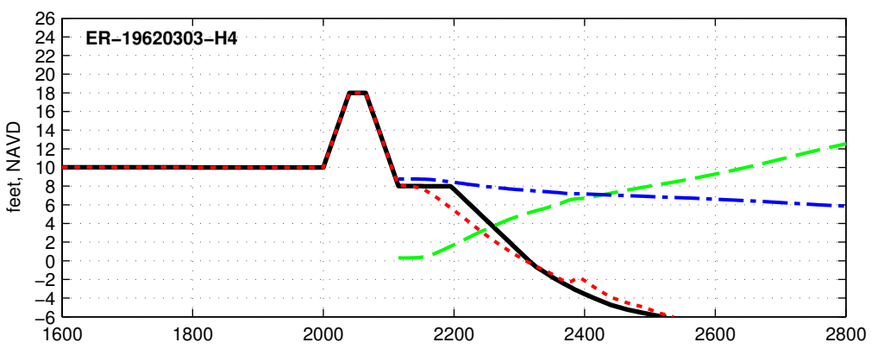


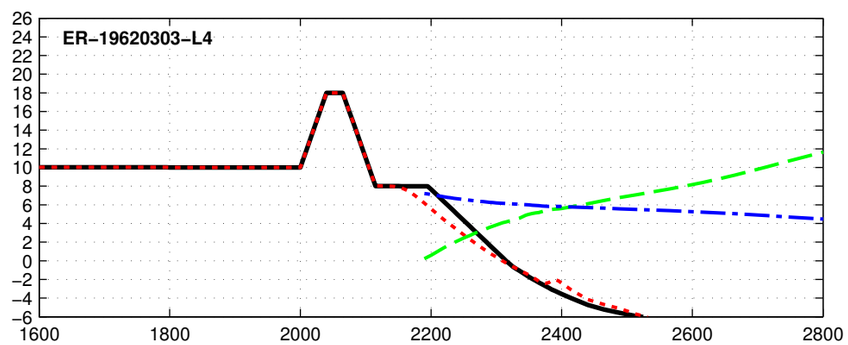


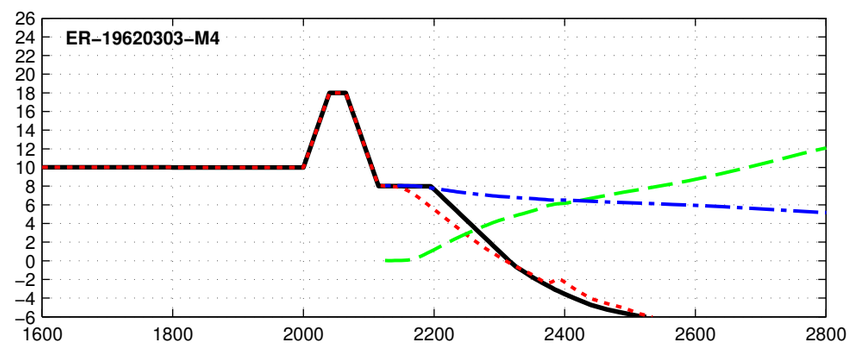






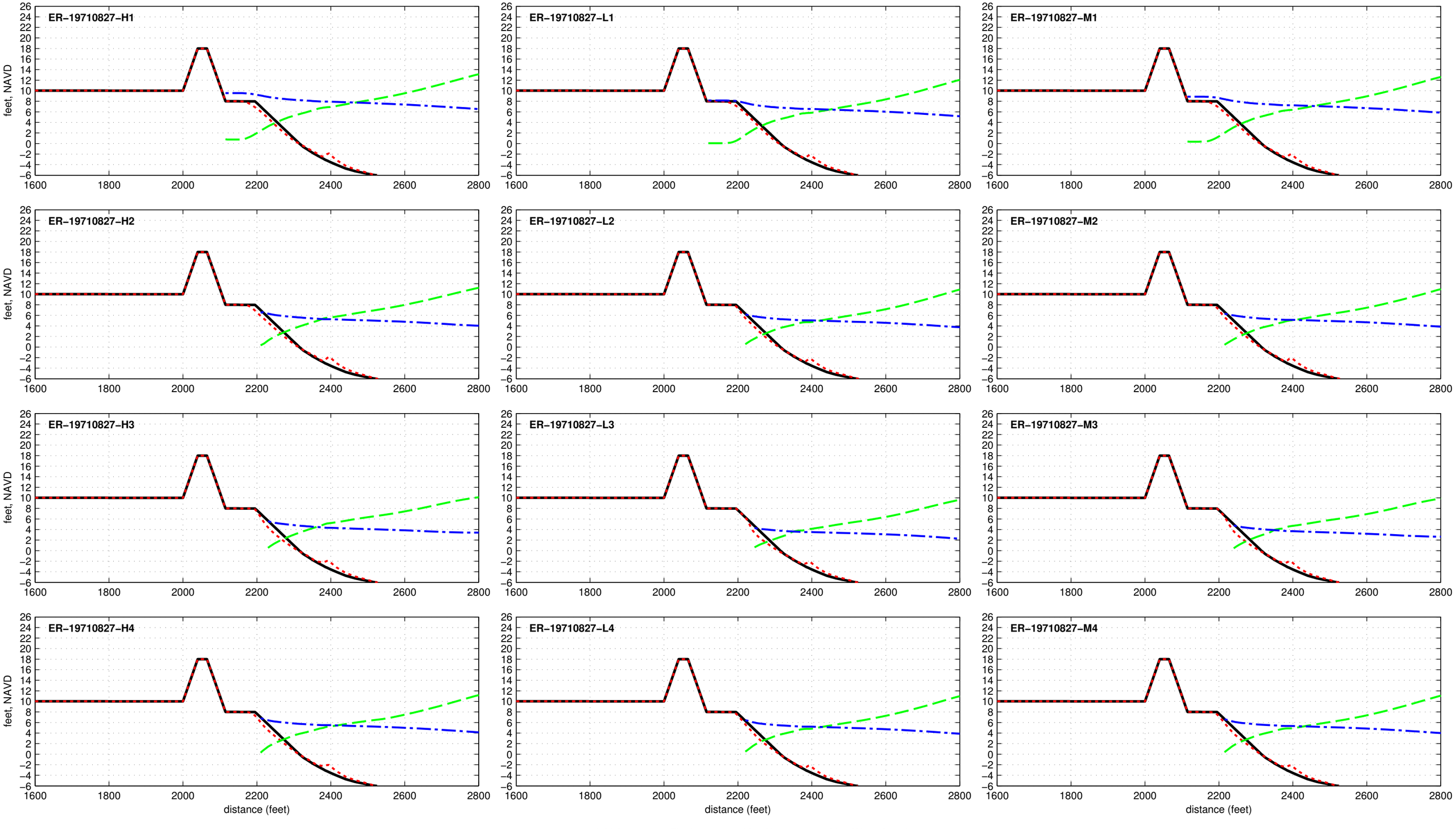




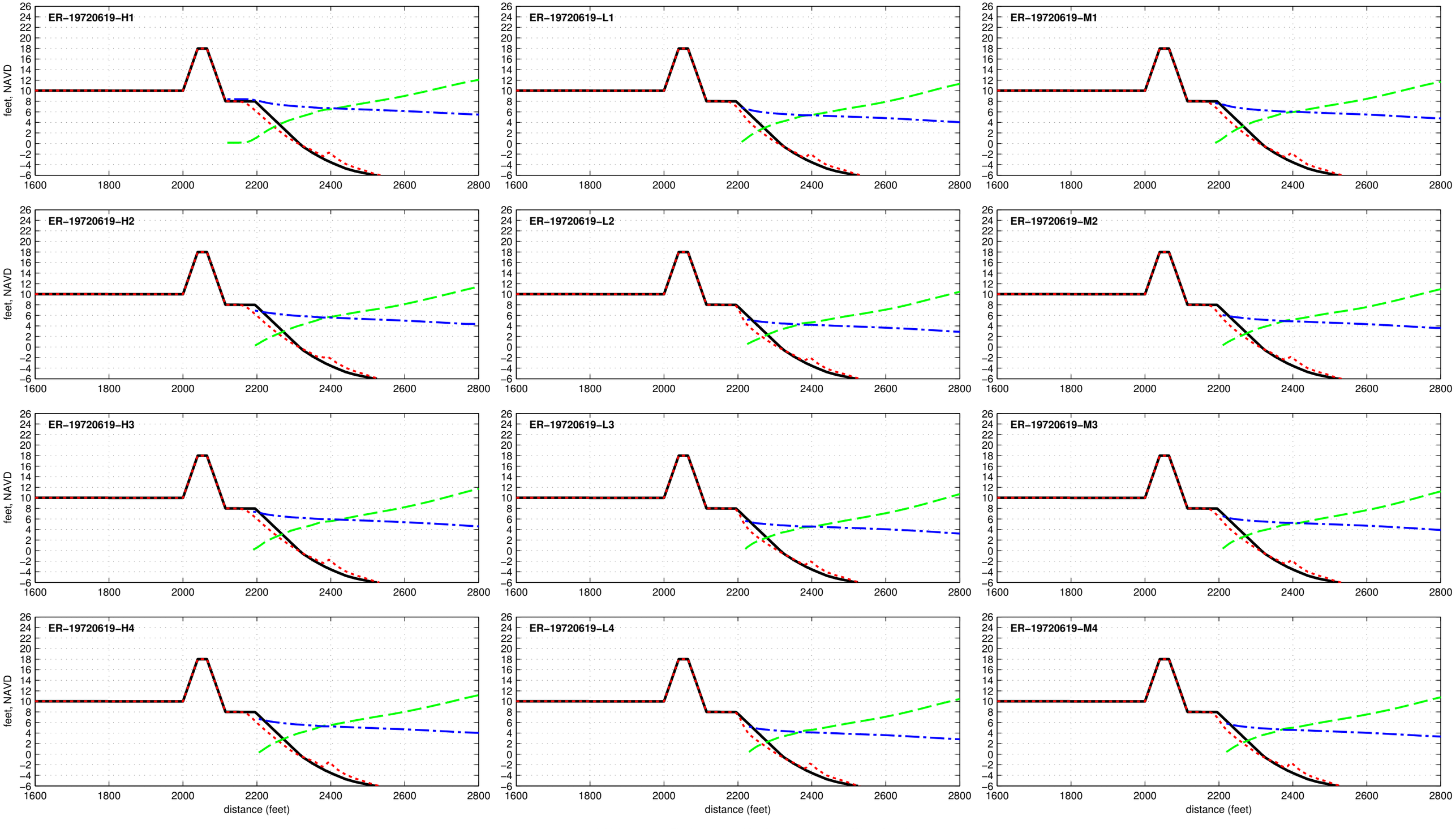


Initial Final Max Hs Max Water Level

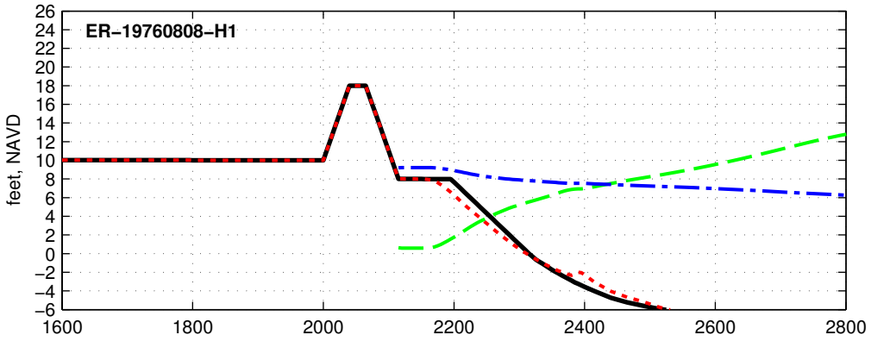
Dune R2T1-18.0-025-080 - ER-19710827

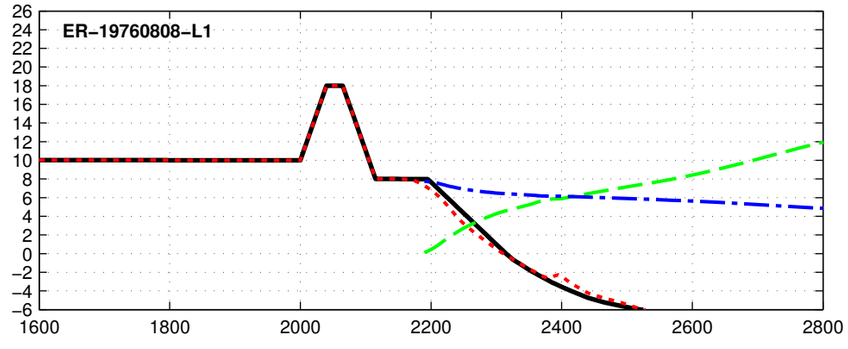


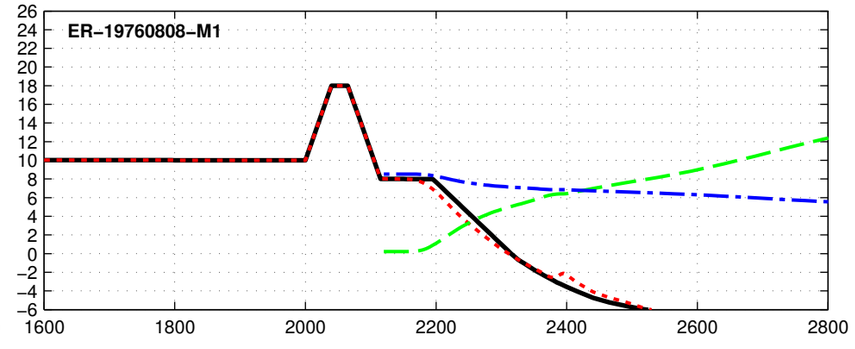
Dune R2T1-18.0-025-080 - ER-19720619

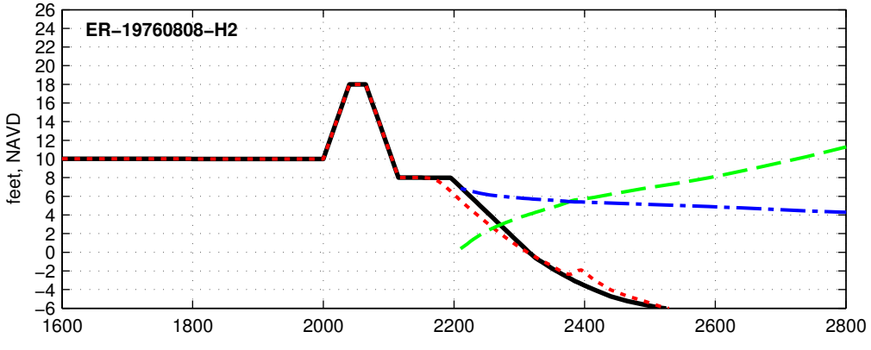


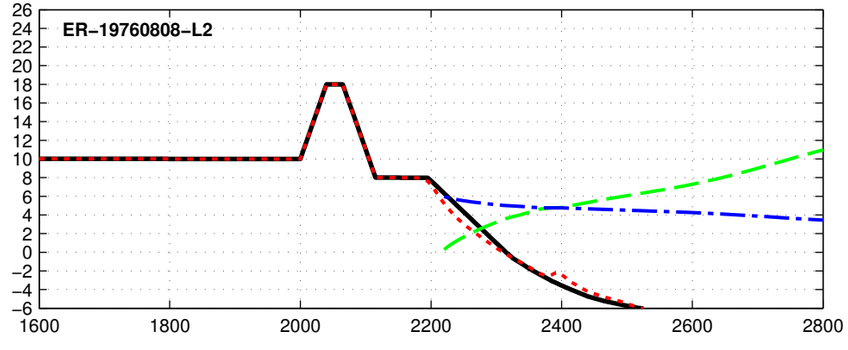
Dune R2T1-18.0-025-080 - ER-19760808

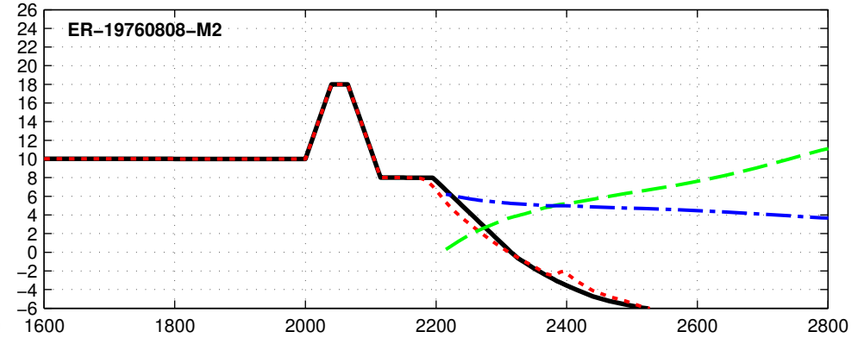


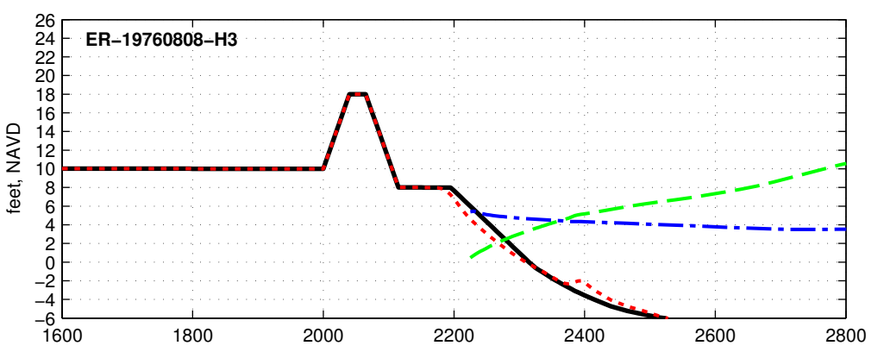


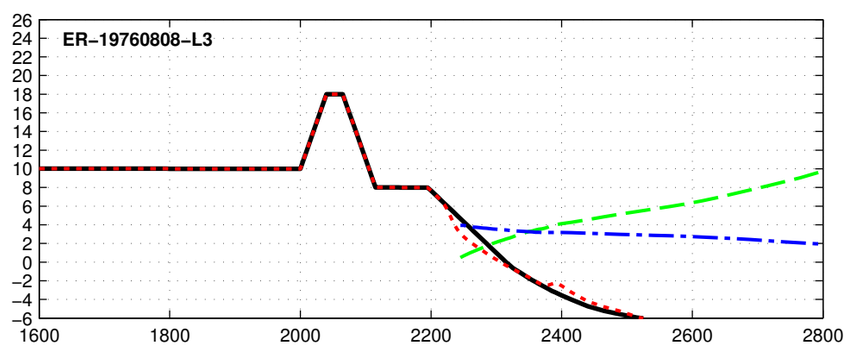


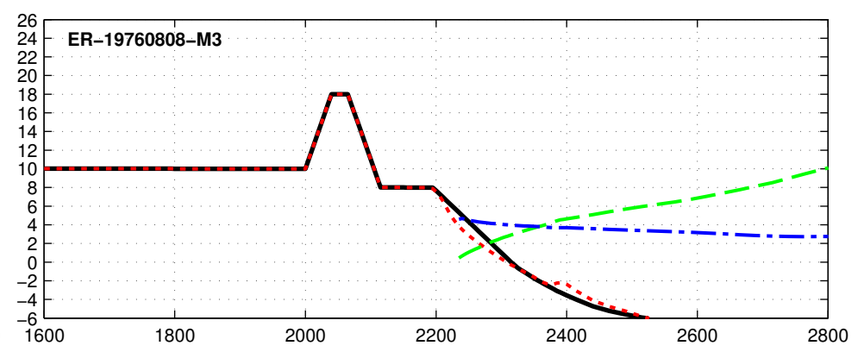


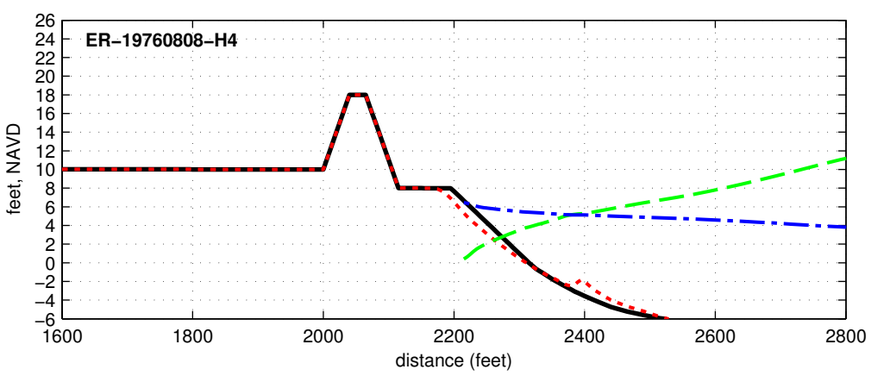


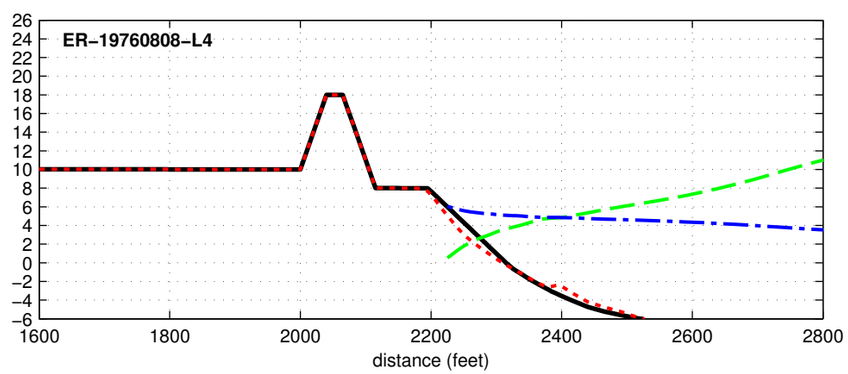


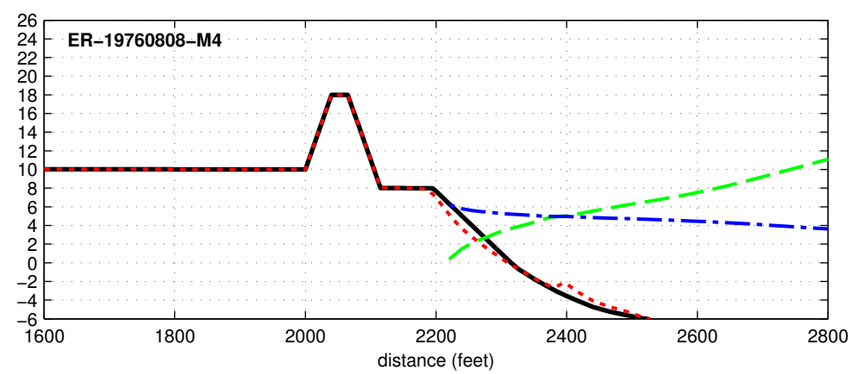






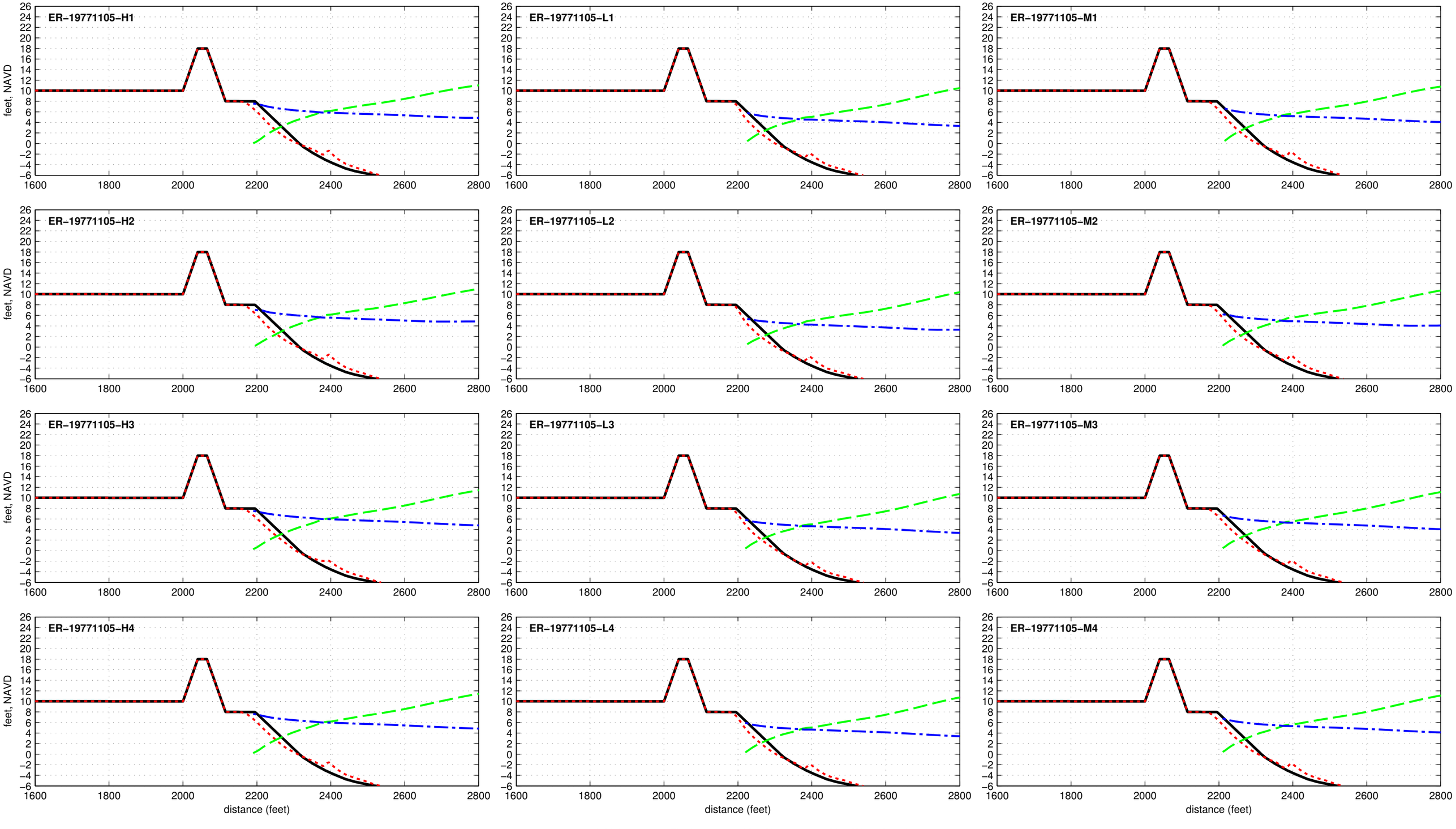






Initial Final Max Hs Max Water Level

Dune R2T1-18.0-025-080 - ER-19771105



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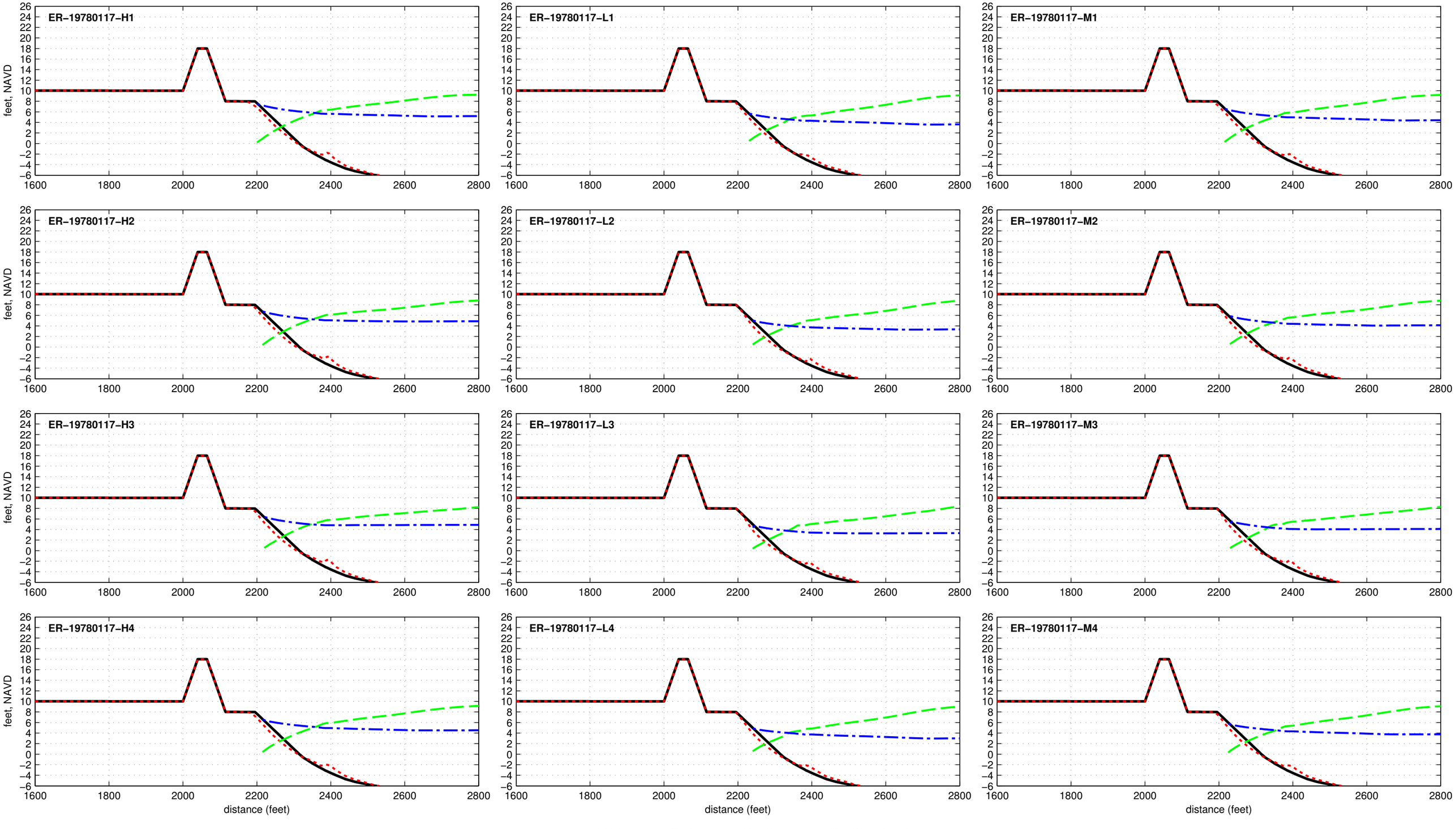

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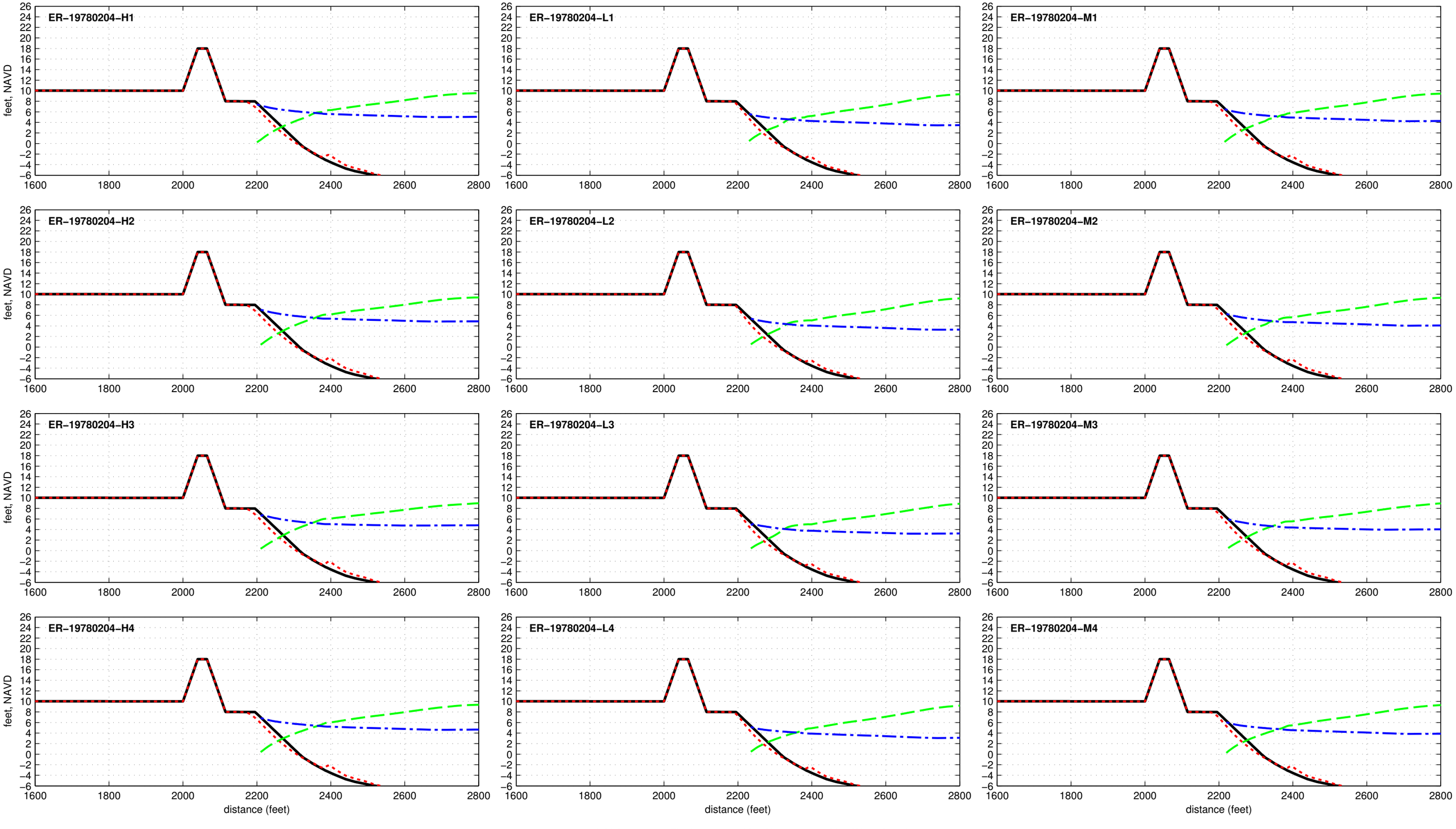

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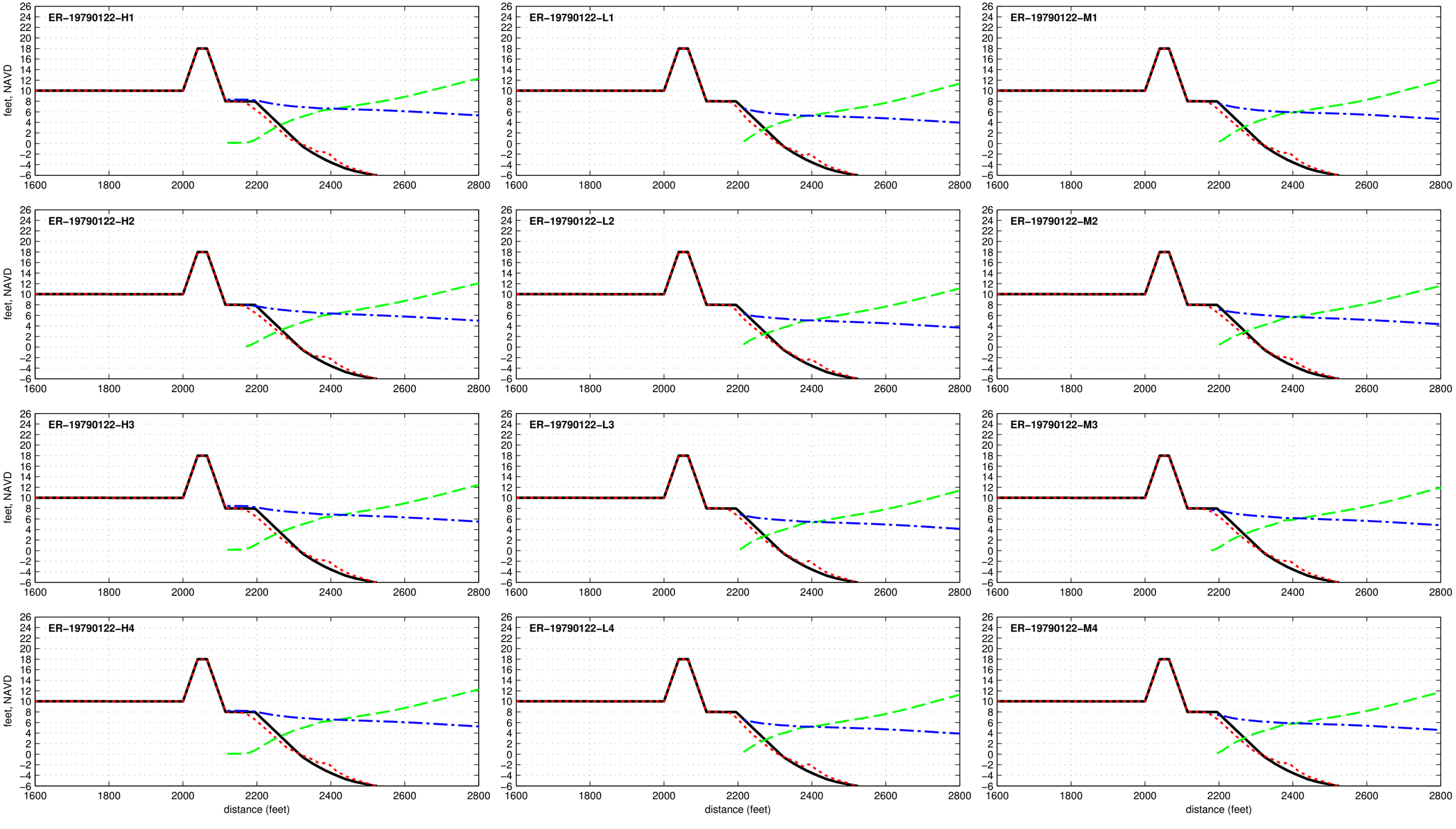

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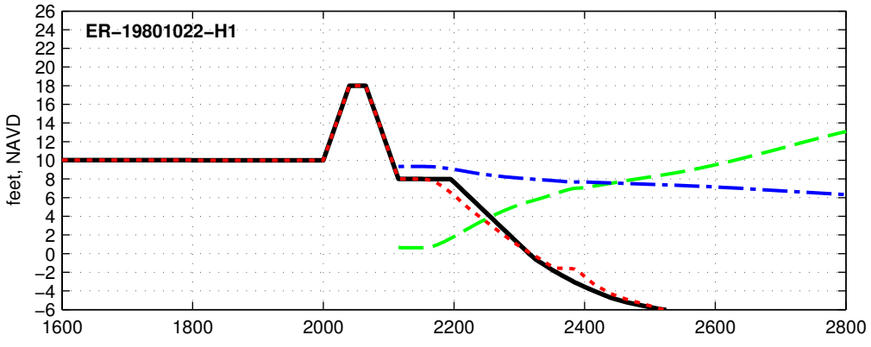
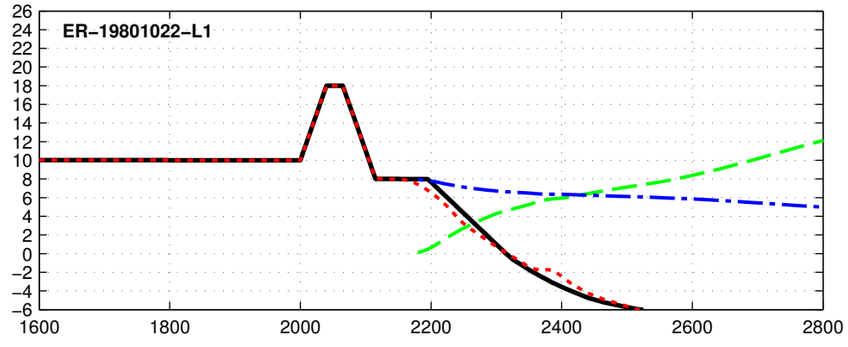
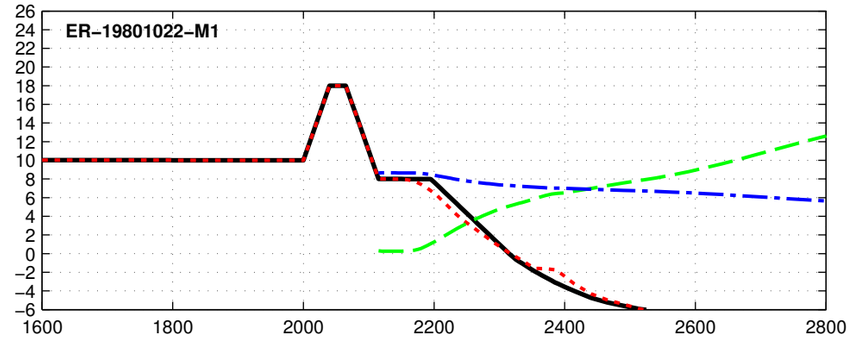
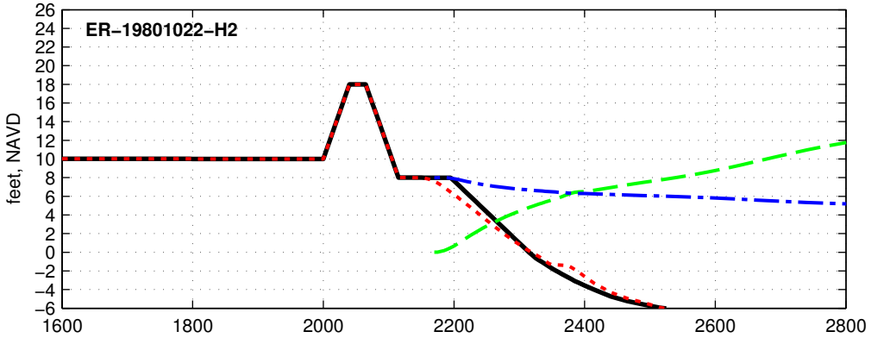
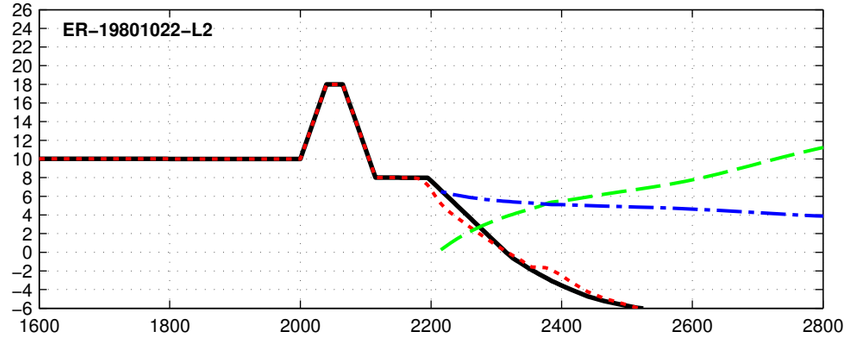
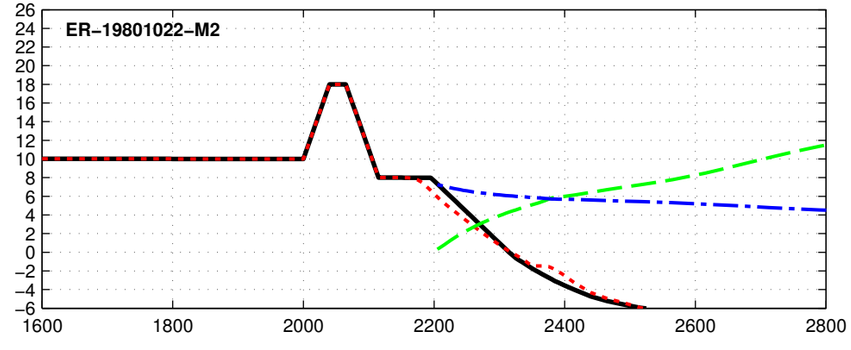


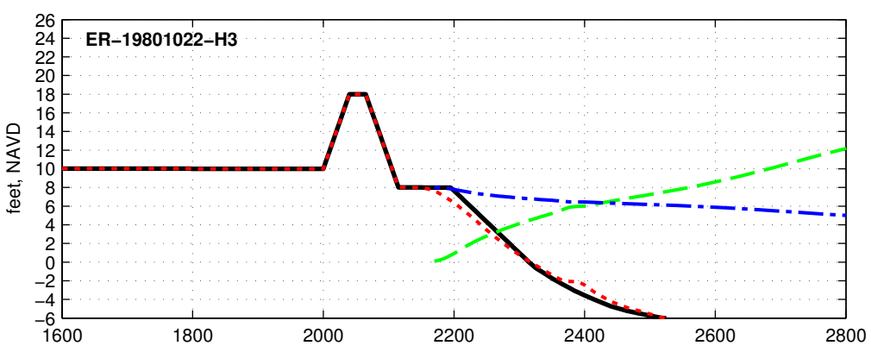


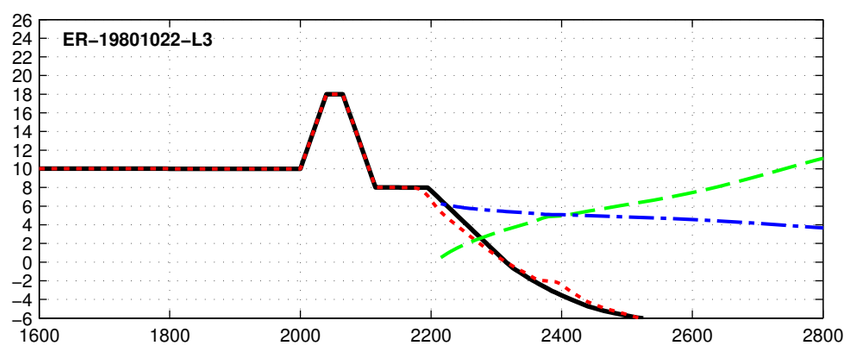


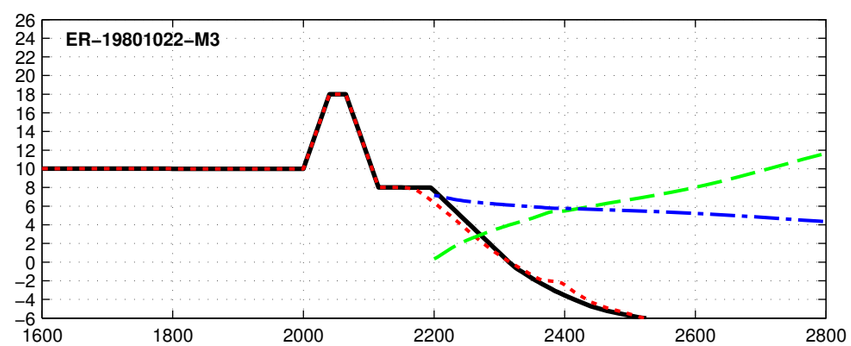


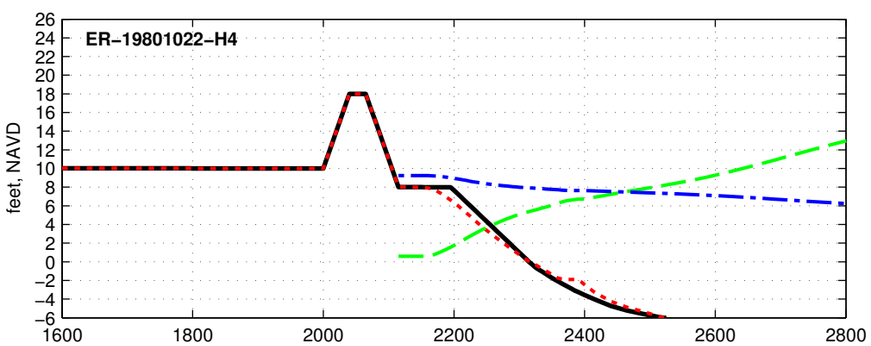


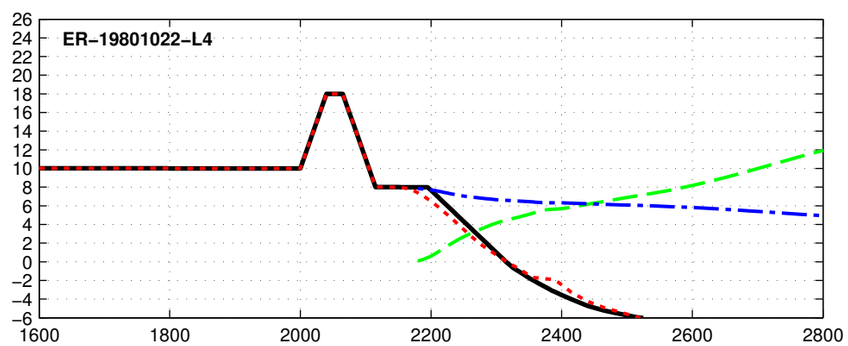


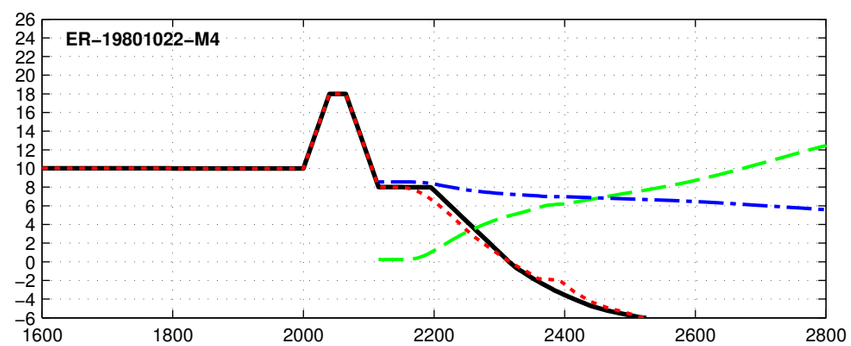






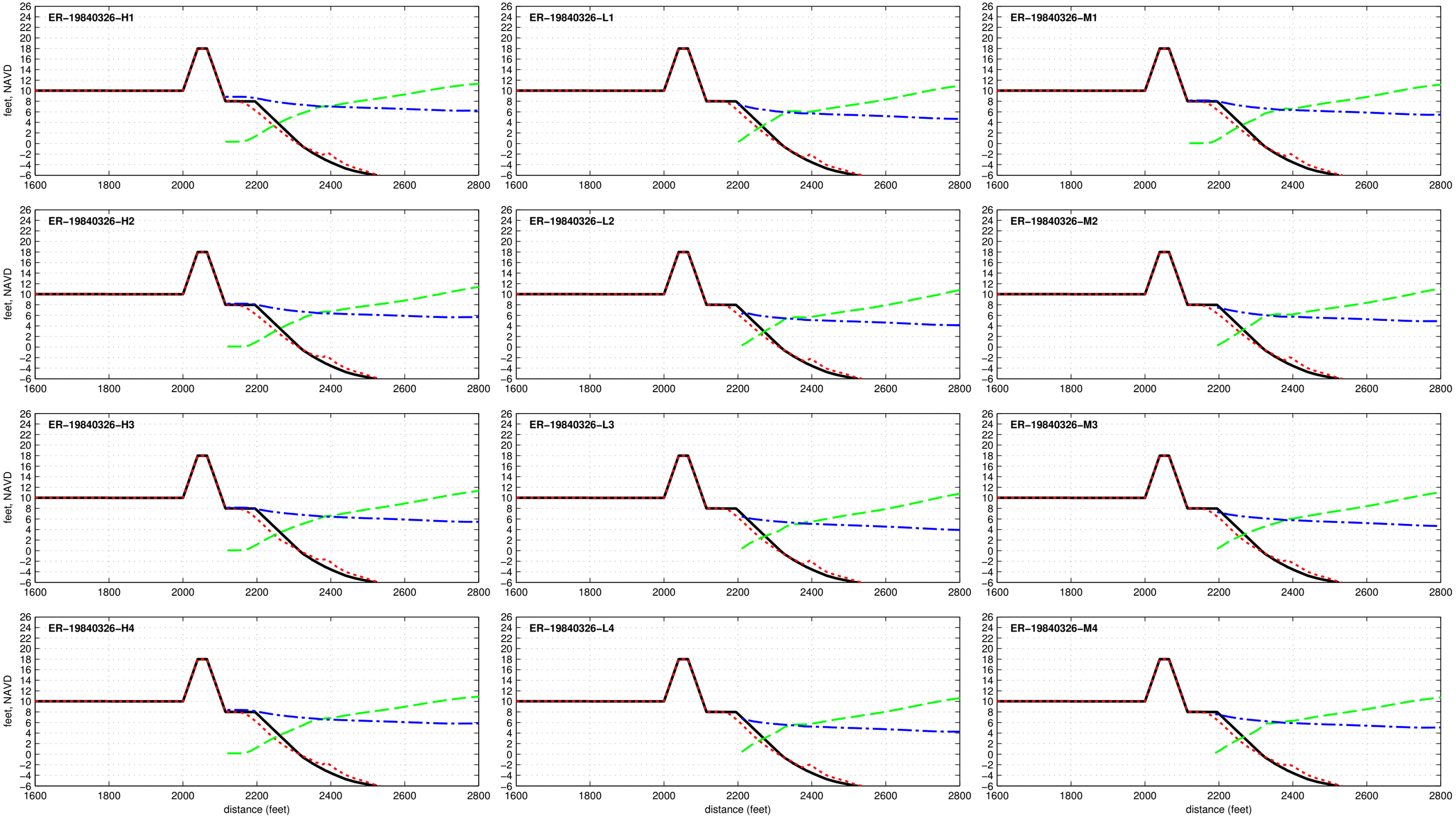




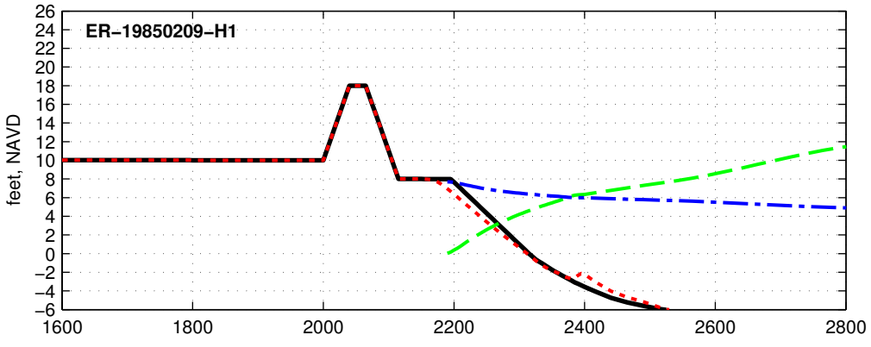


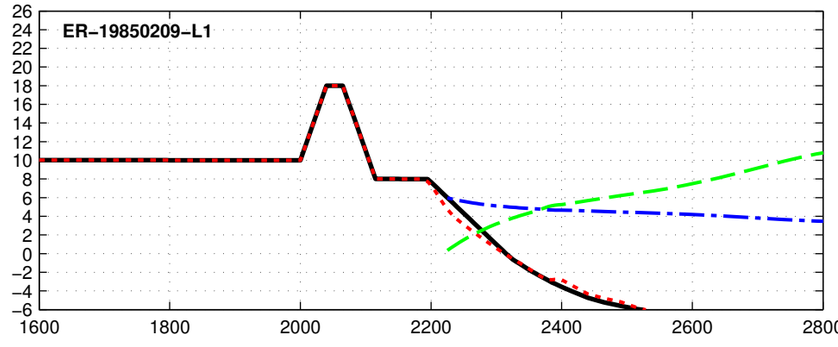
Initial Final Max Hs Max Water Level

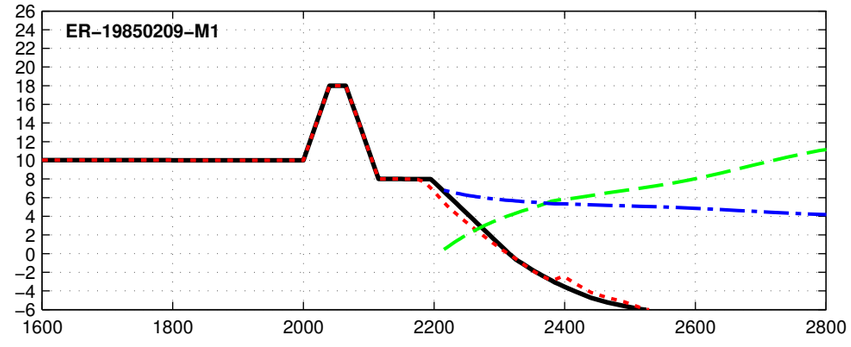
Dune R2T1-18.0-025-080 - ER-19840326

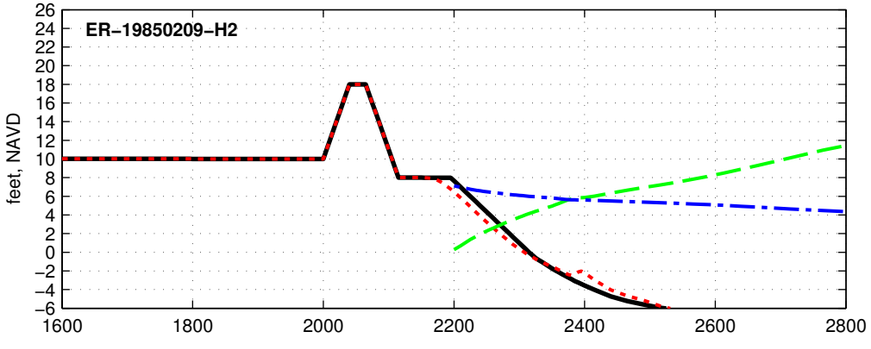


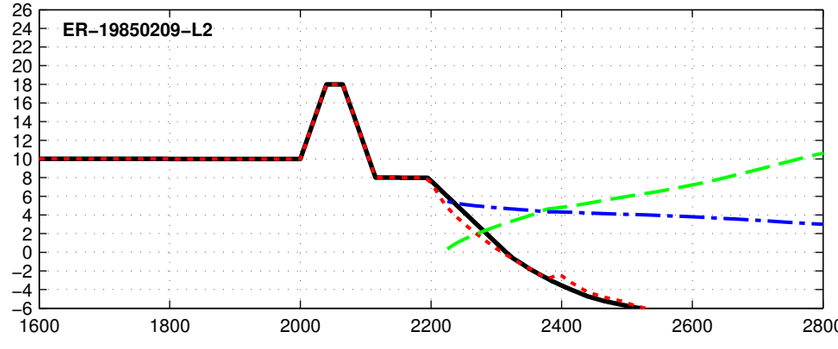
Dune R2T1-18.0-025-080 - ER-19850209

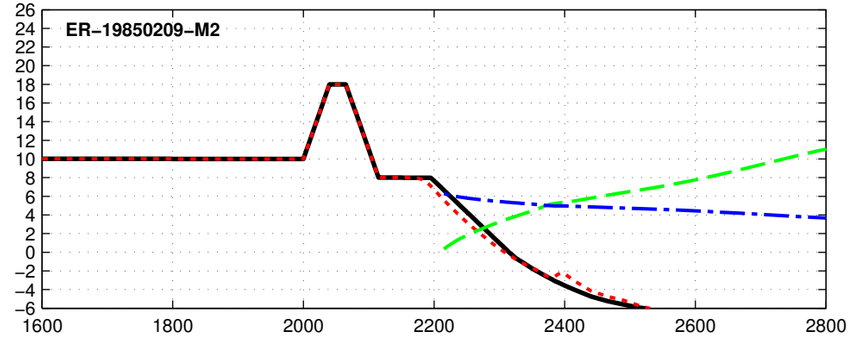


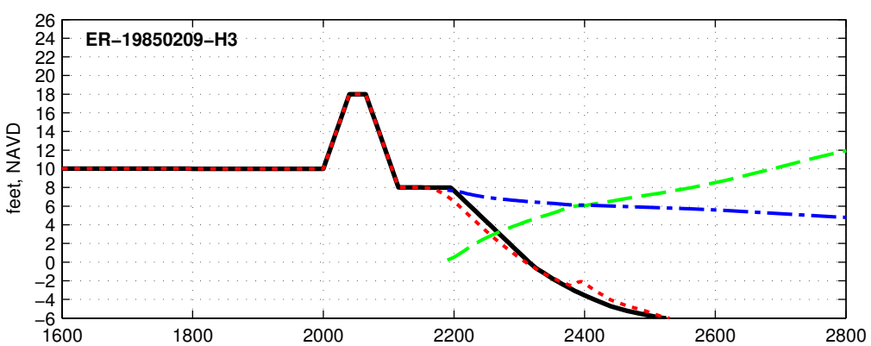


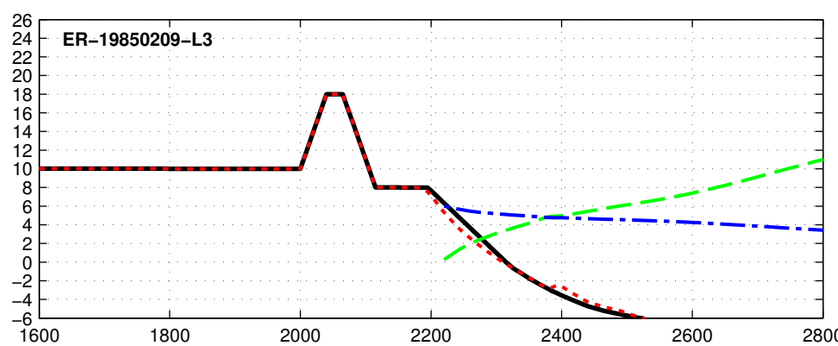


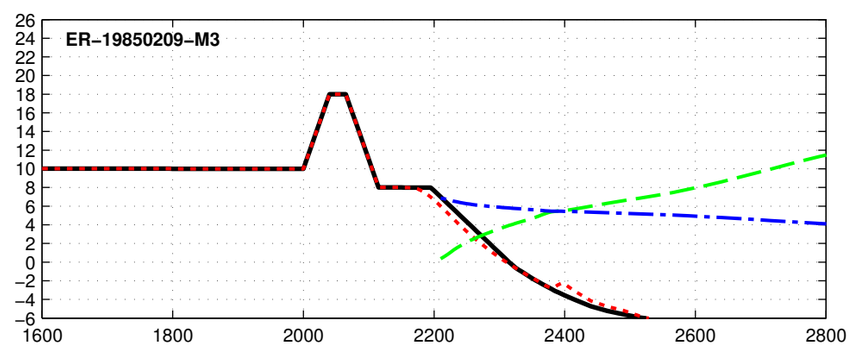


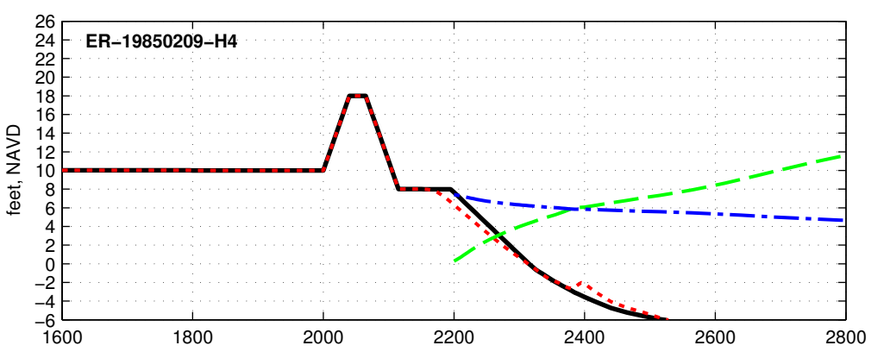


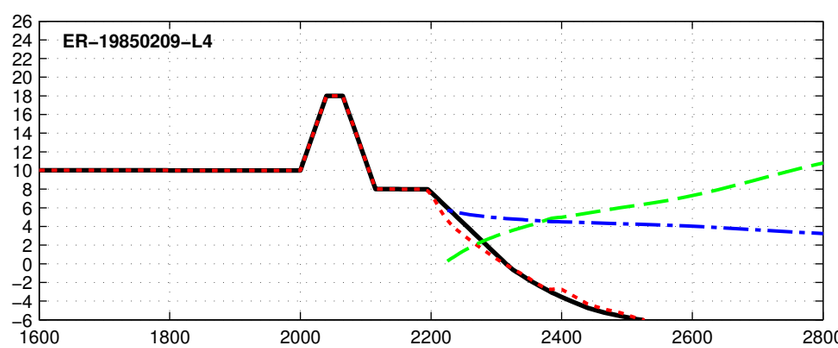


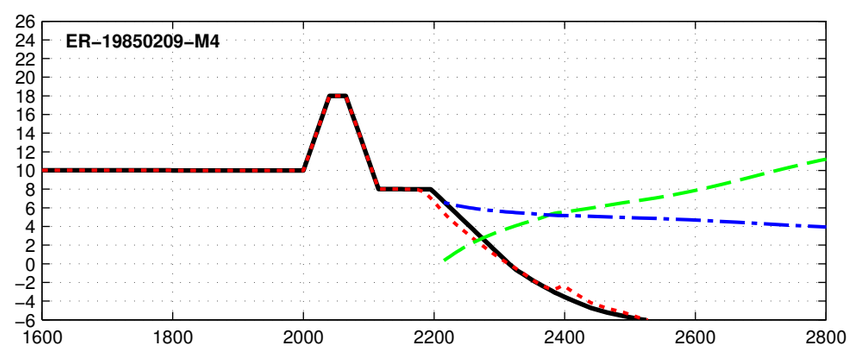






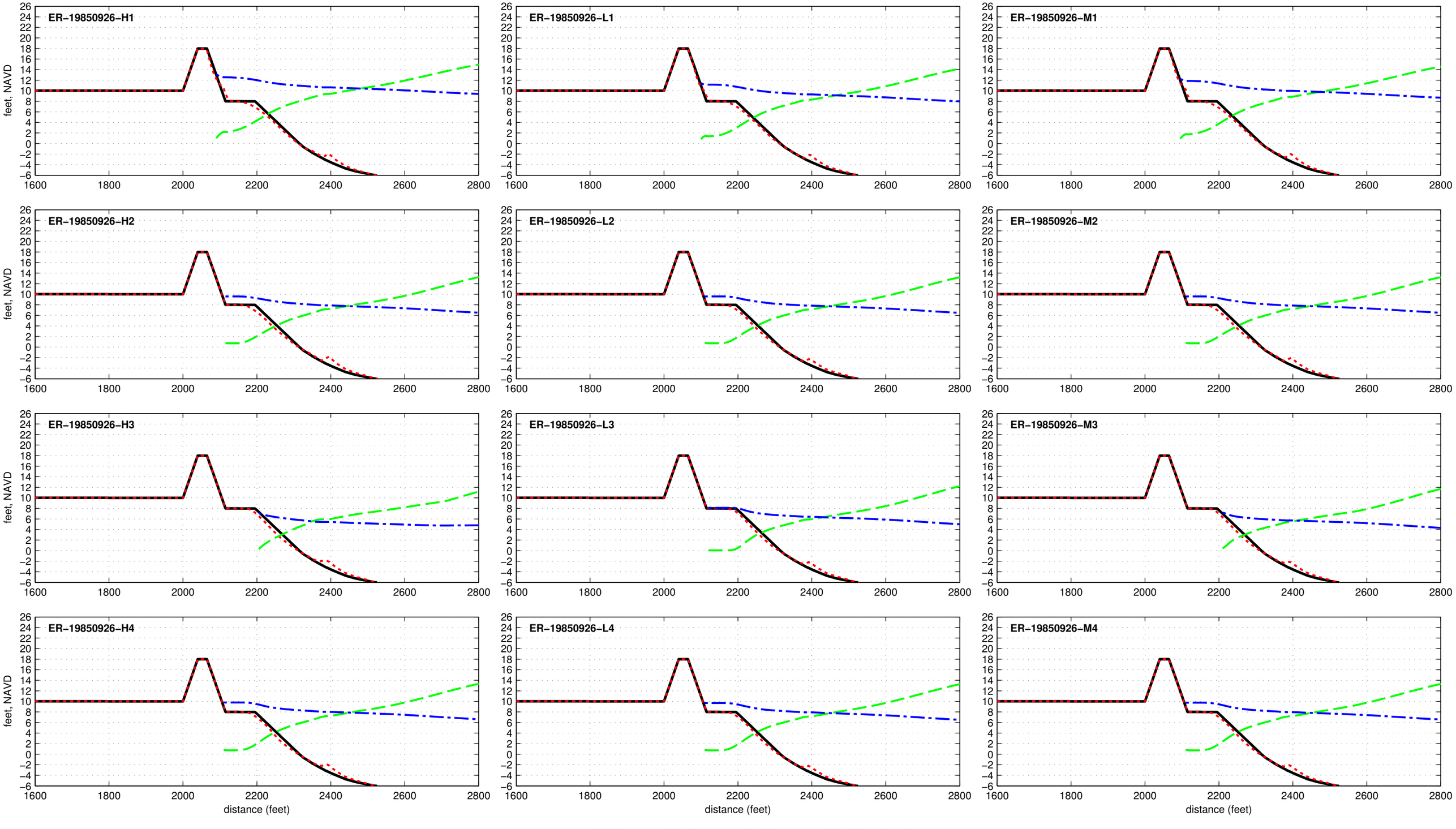




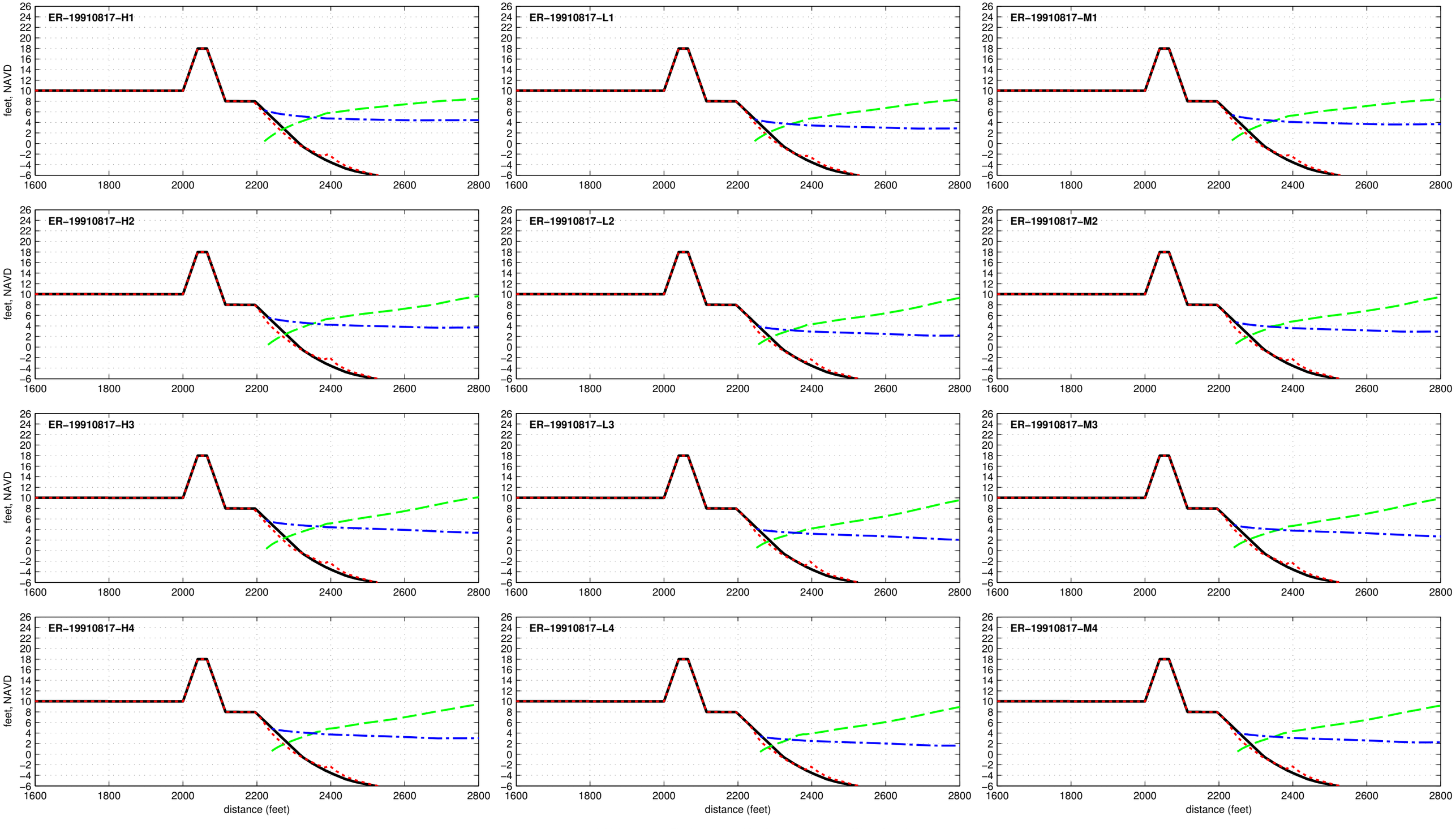


Initial Final Max Hs Max Water Level

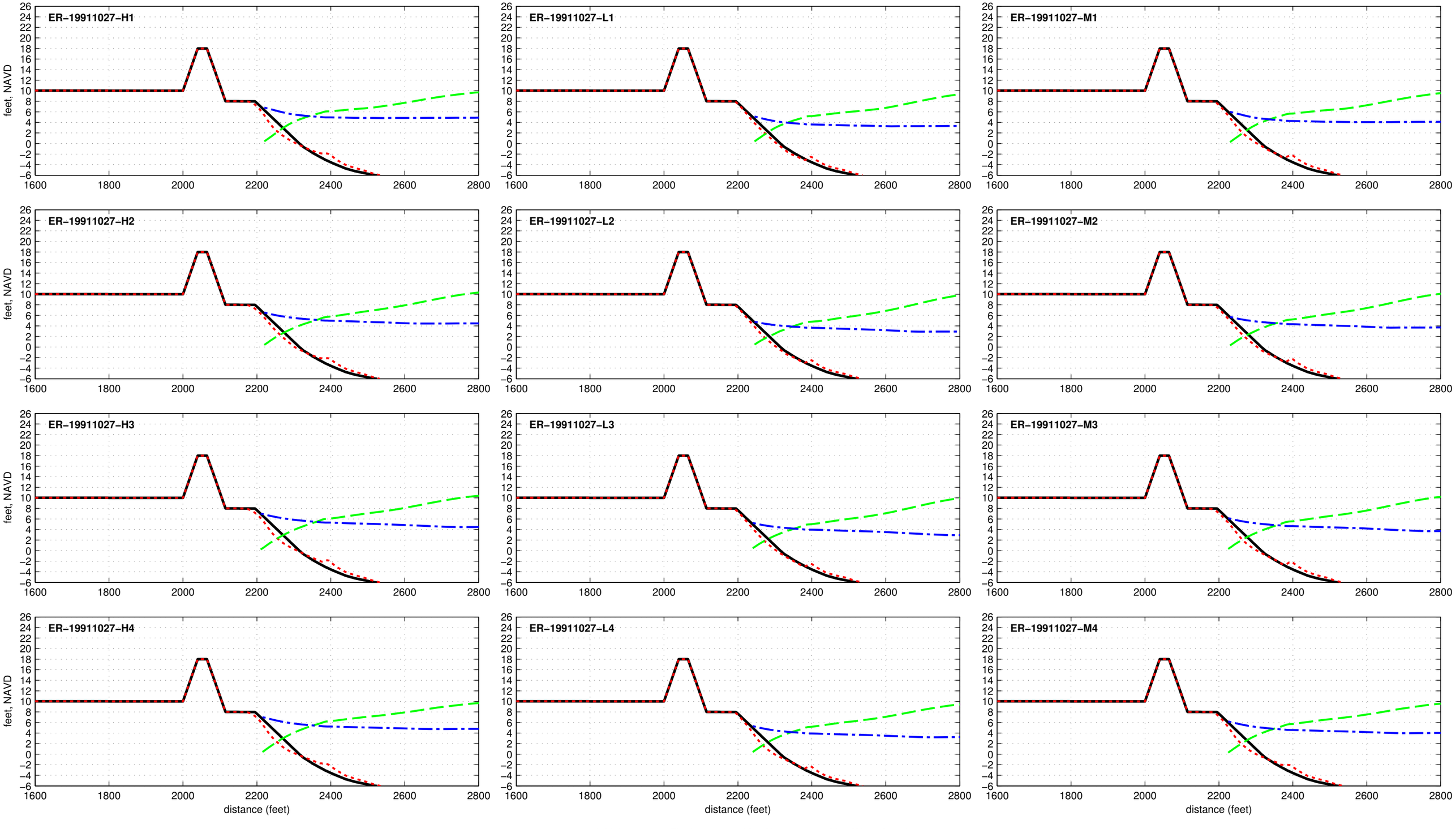
Dune R2T1-18.0-025-080 - ER-19850926



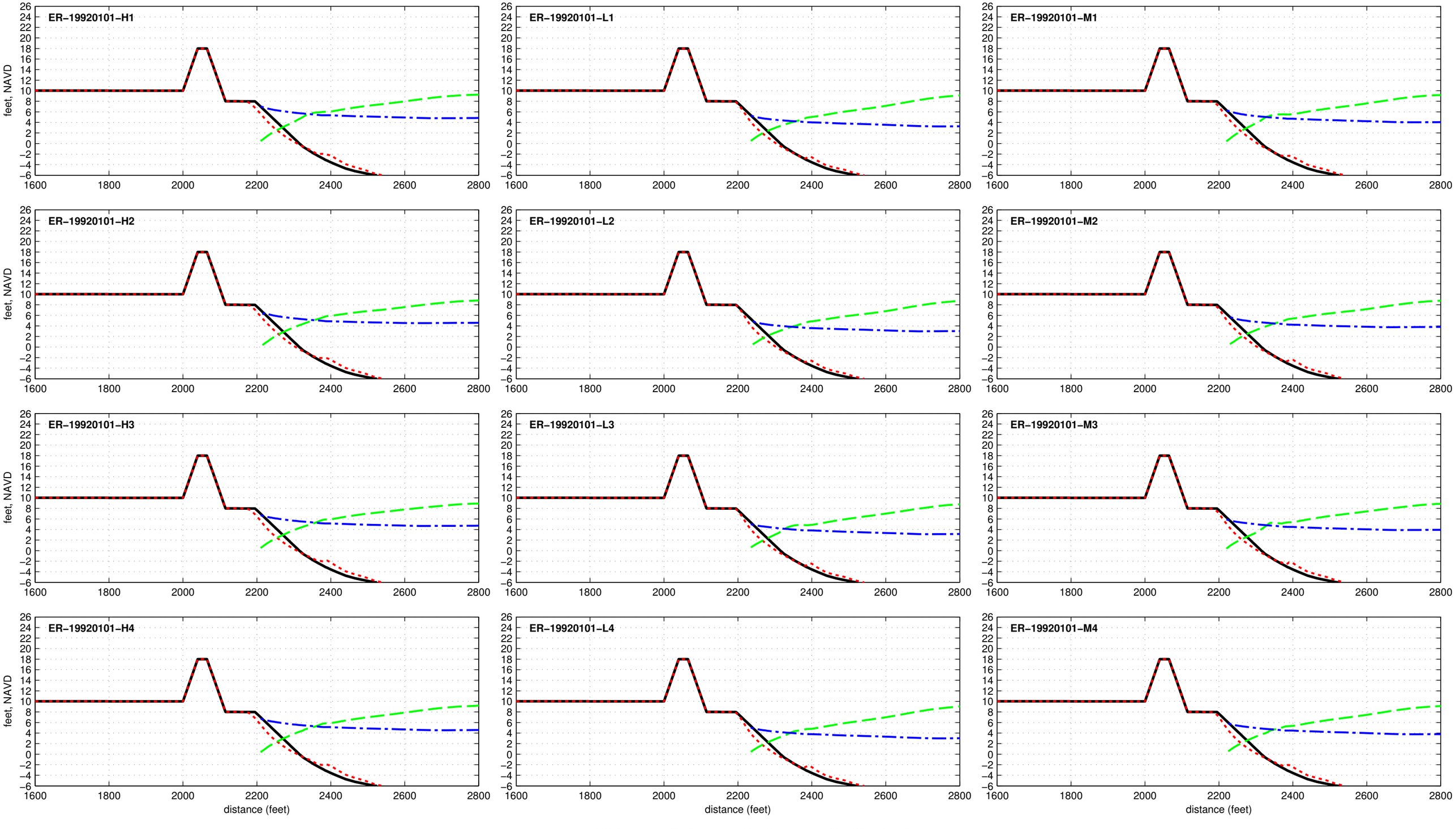
Dune R2T1-18.0-025-080 - ER-19910817



Dune R2T1-18.0-025-080 - ER-19911027



Dune R2T1-18.0-025-080 - ER-19920101



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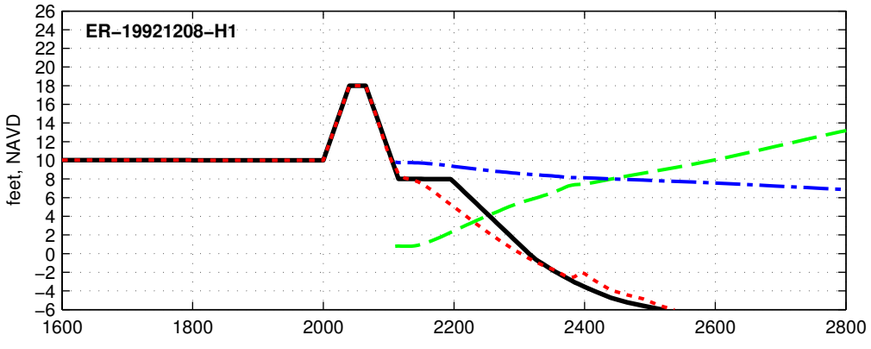
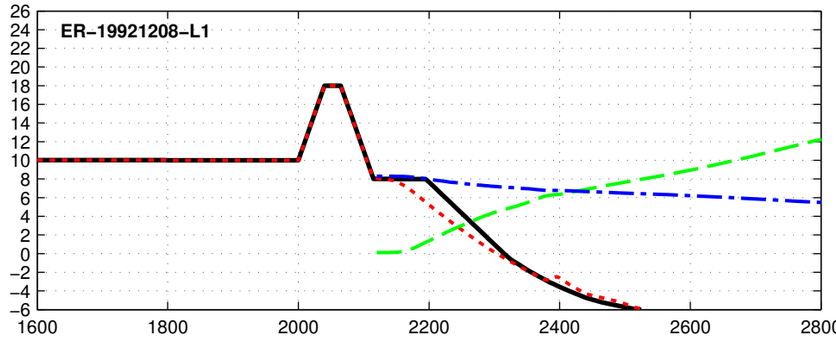
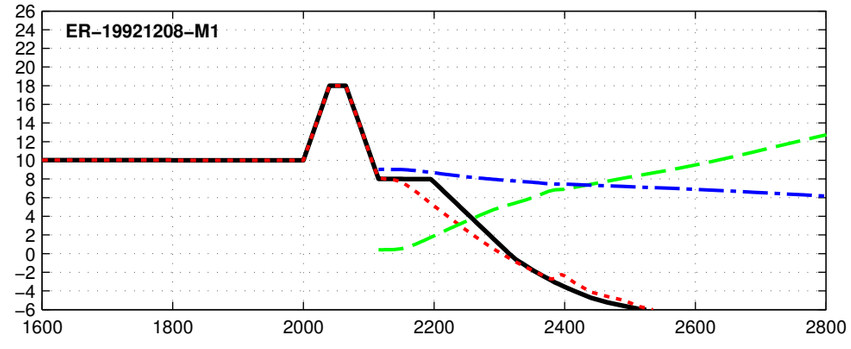
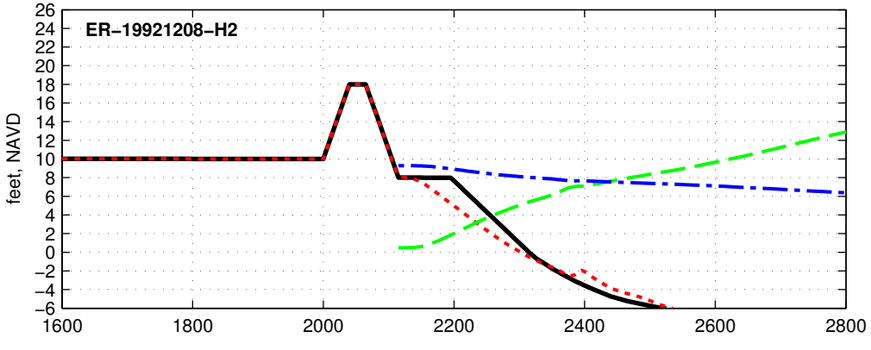
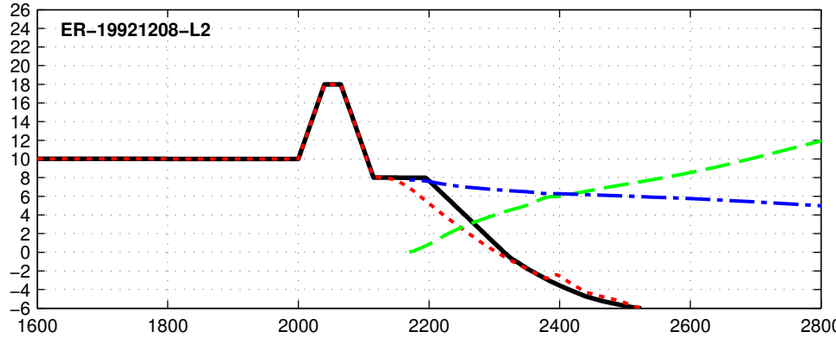
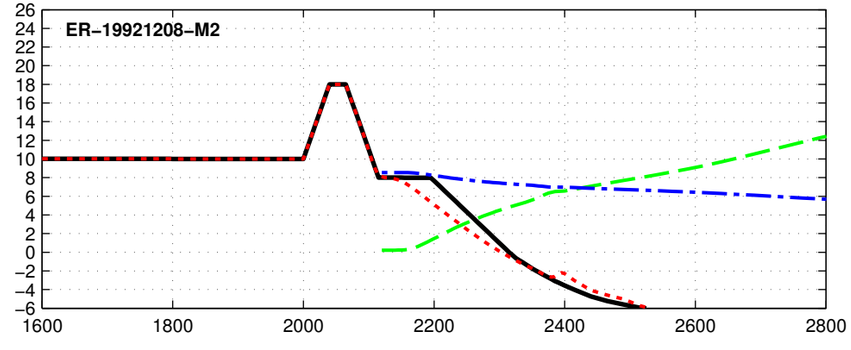


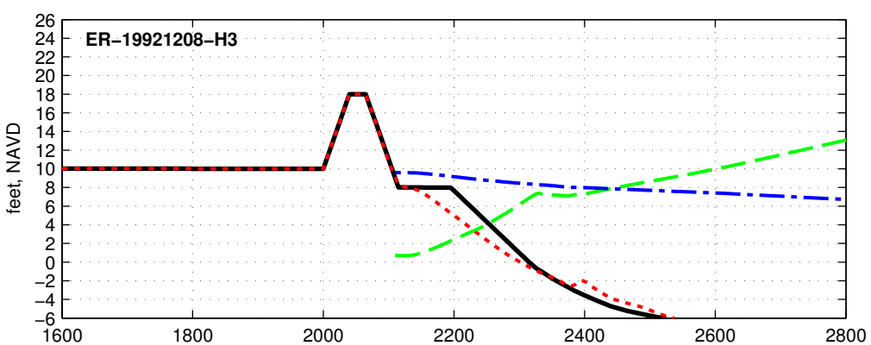


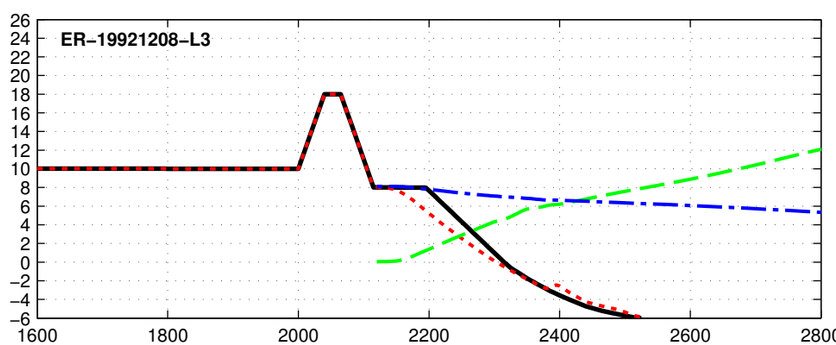


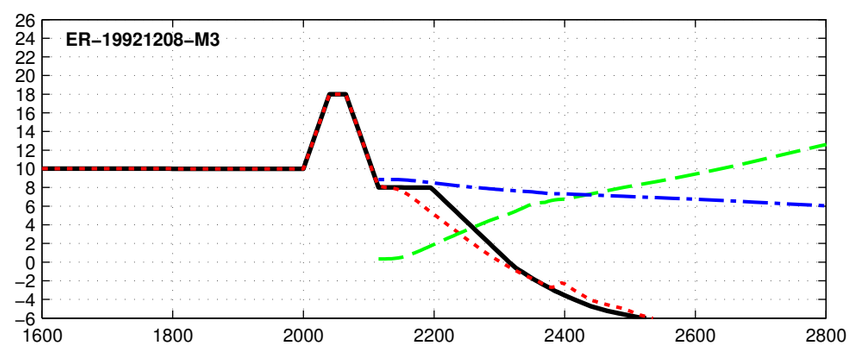


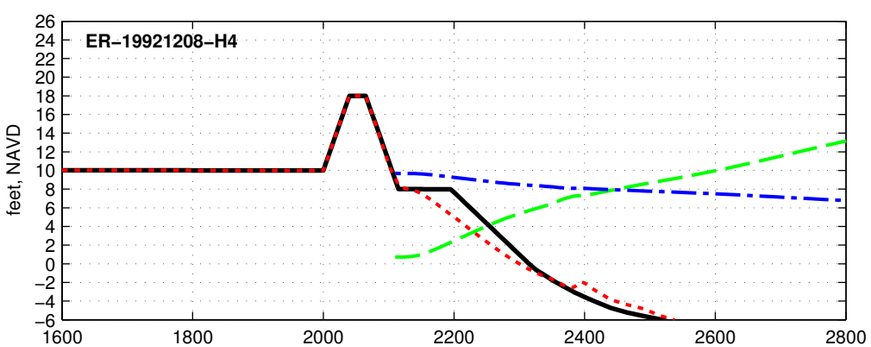


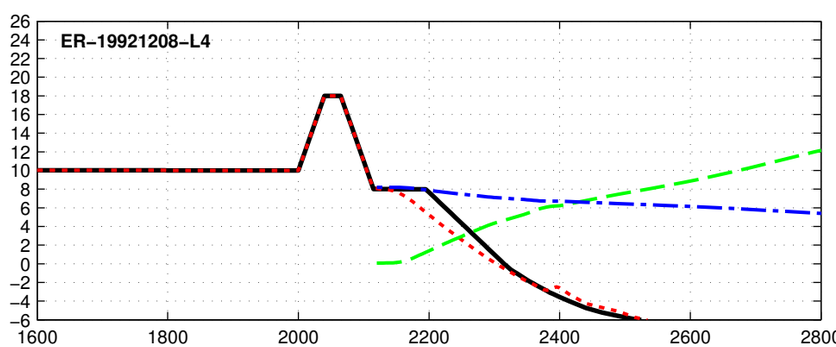


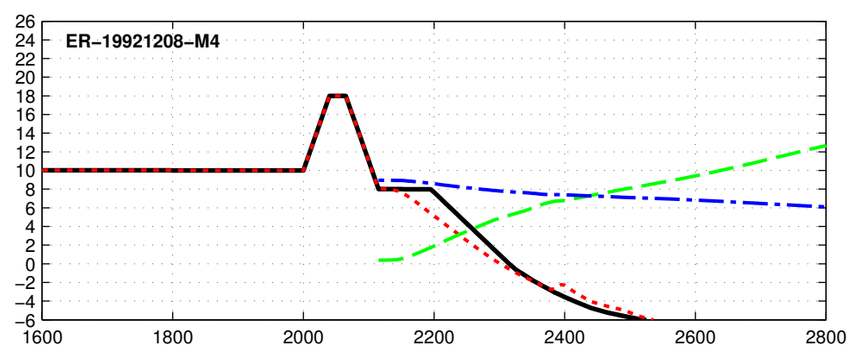










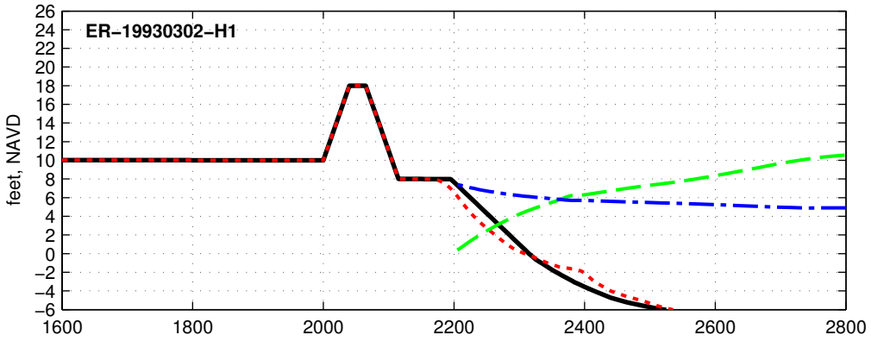


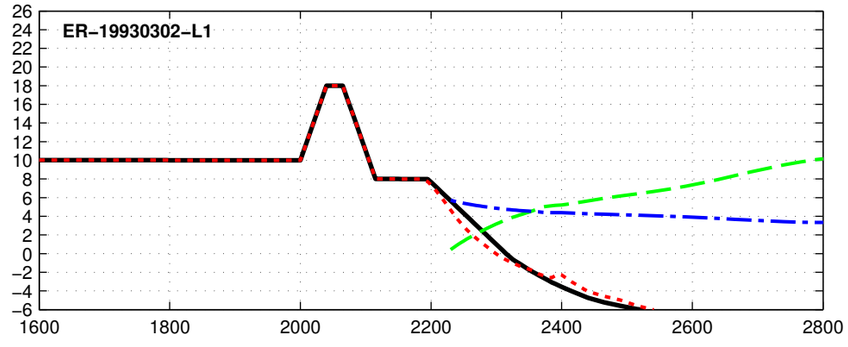
feet, NAVD

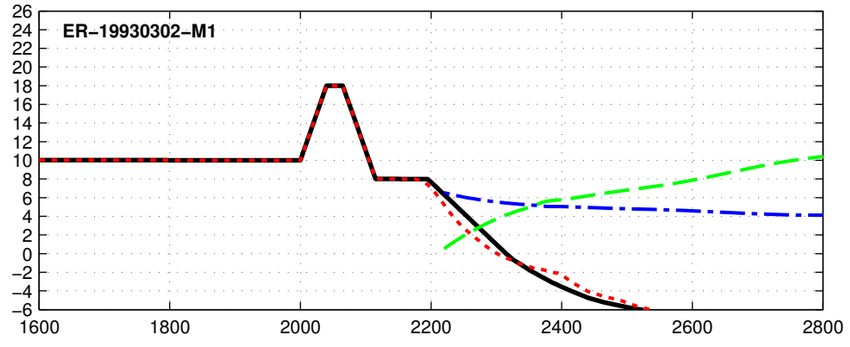
distance (feet)

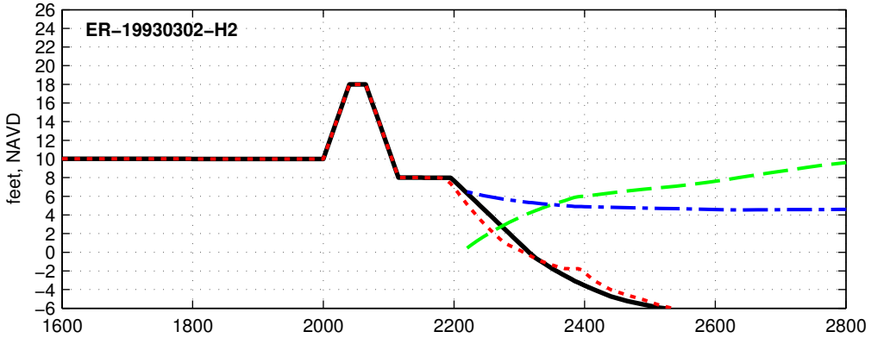
Initial Final Max Hs Max Water Level

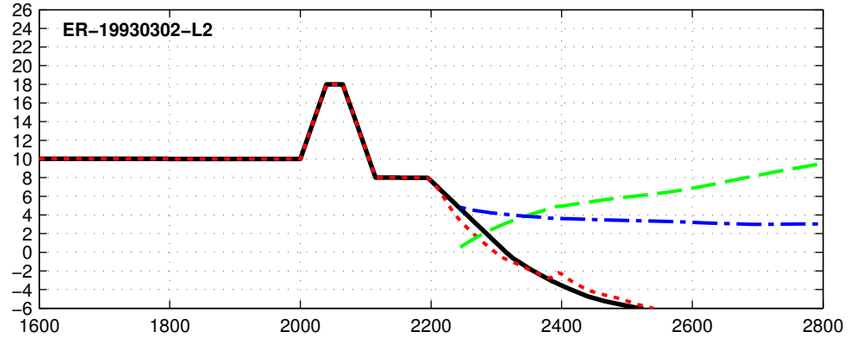
Dune R2T1–18.0–025–080 – ER–19930302

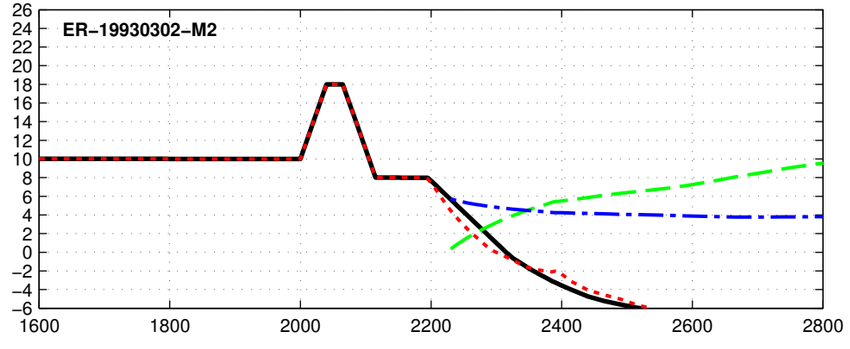


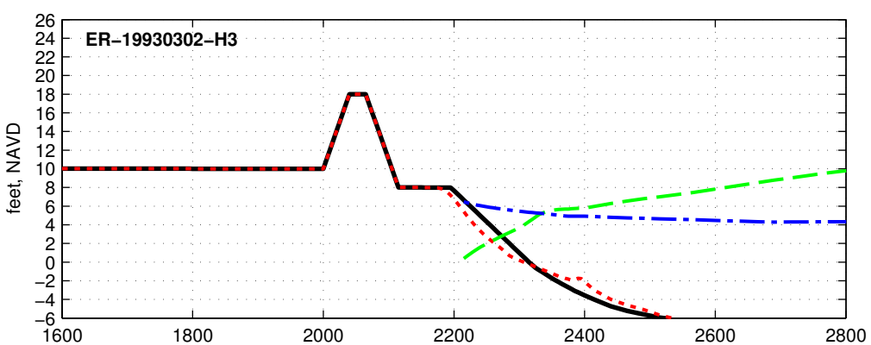


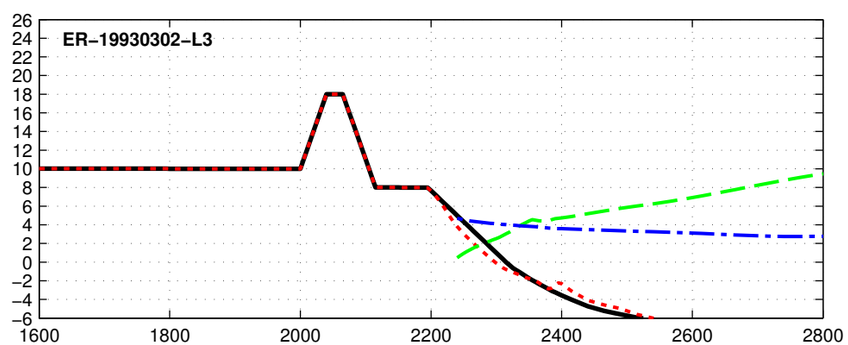


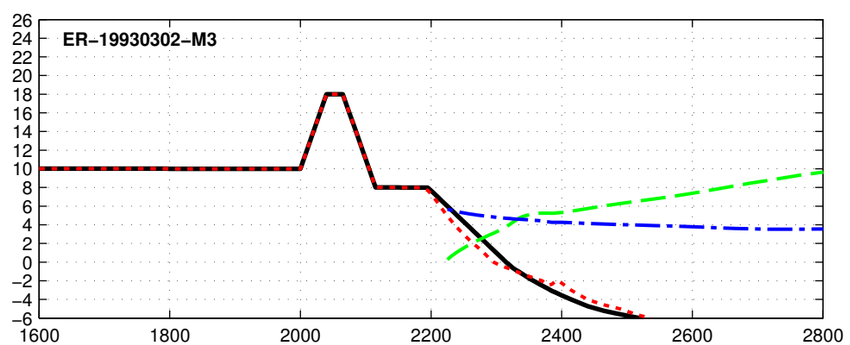


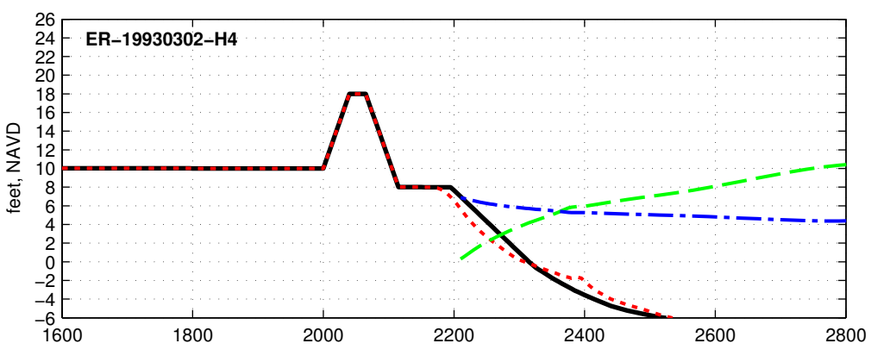


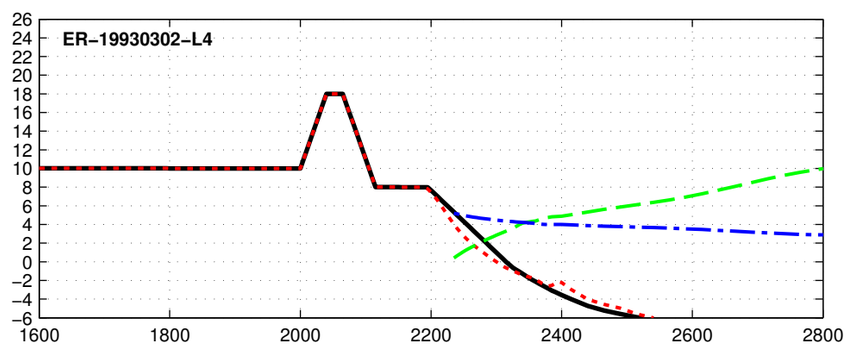


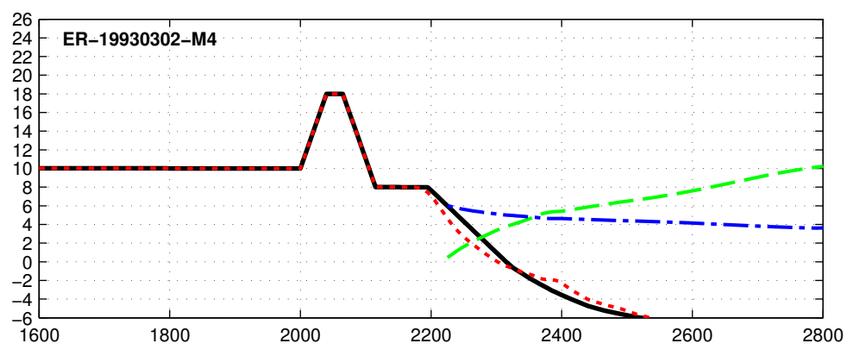






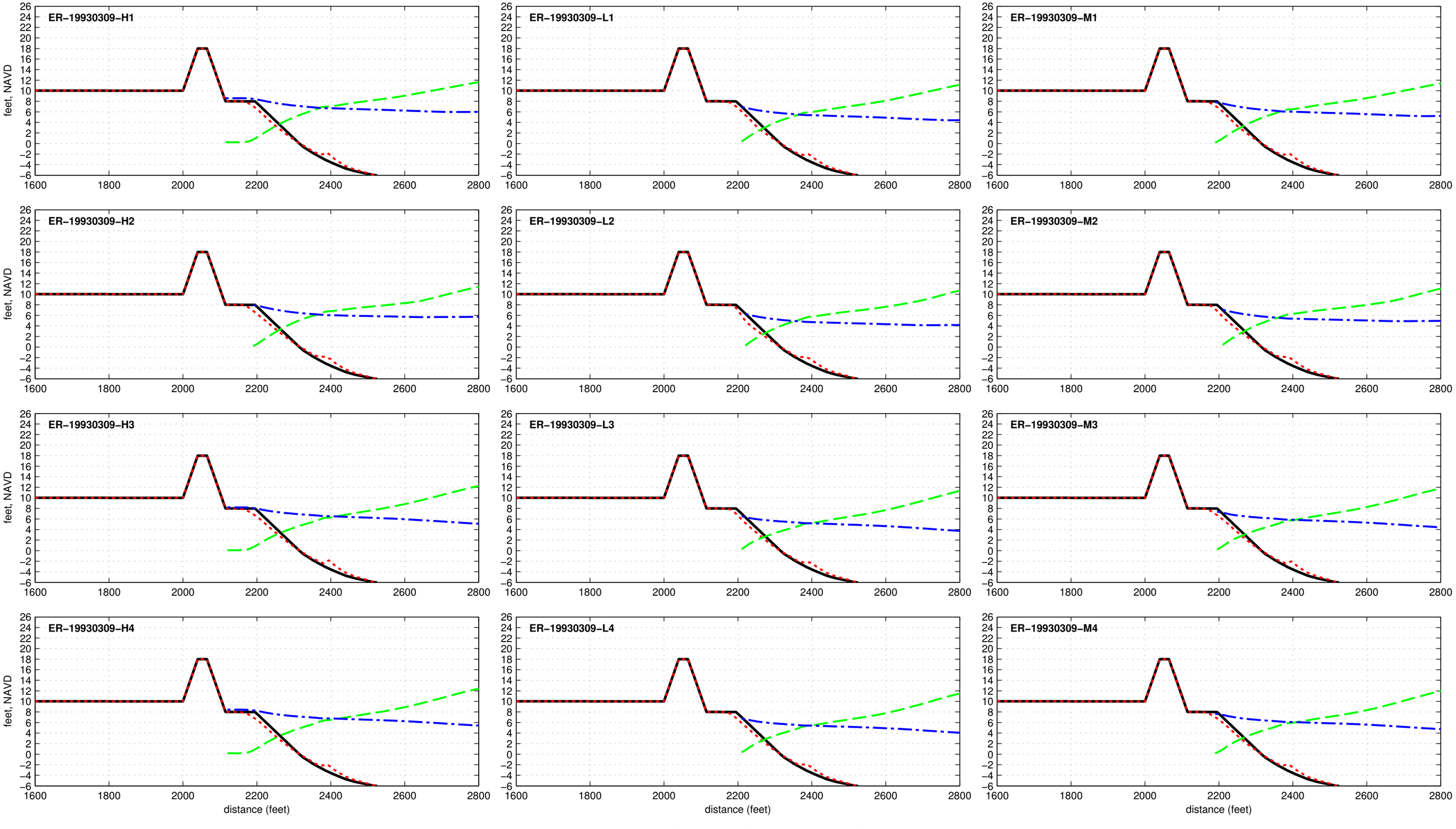






Initial Final Max Hs Max Water Level

Dune R2T1-18.0-025-080 - ER-19930309



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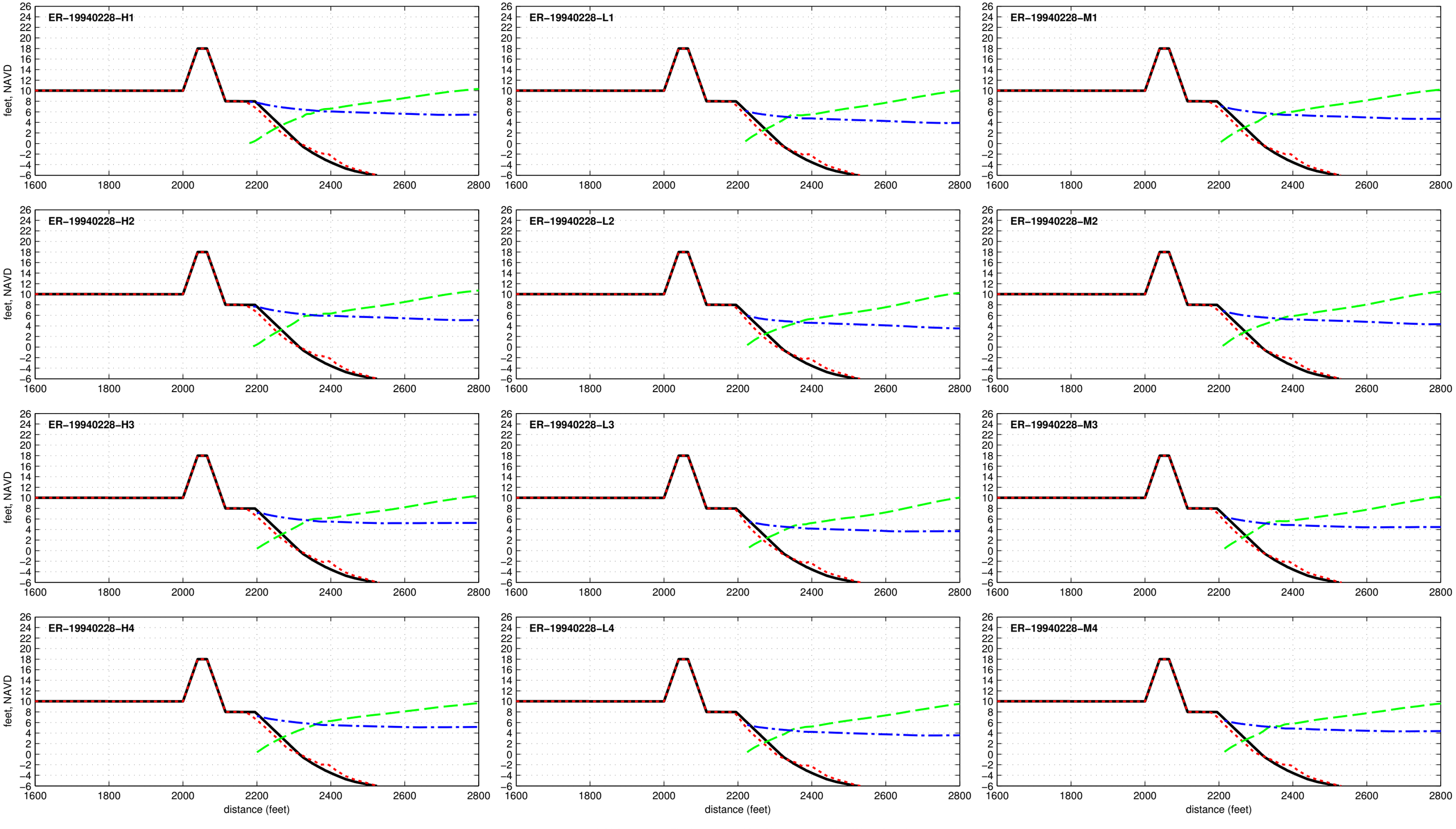

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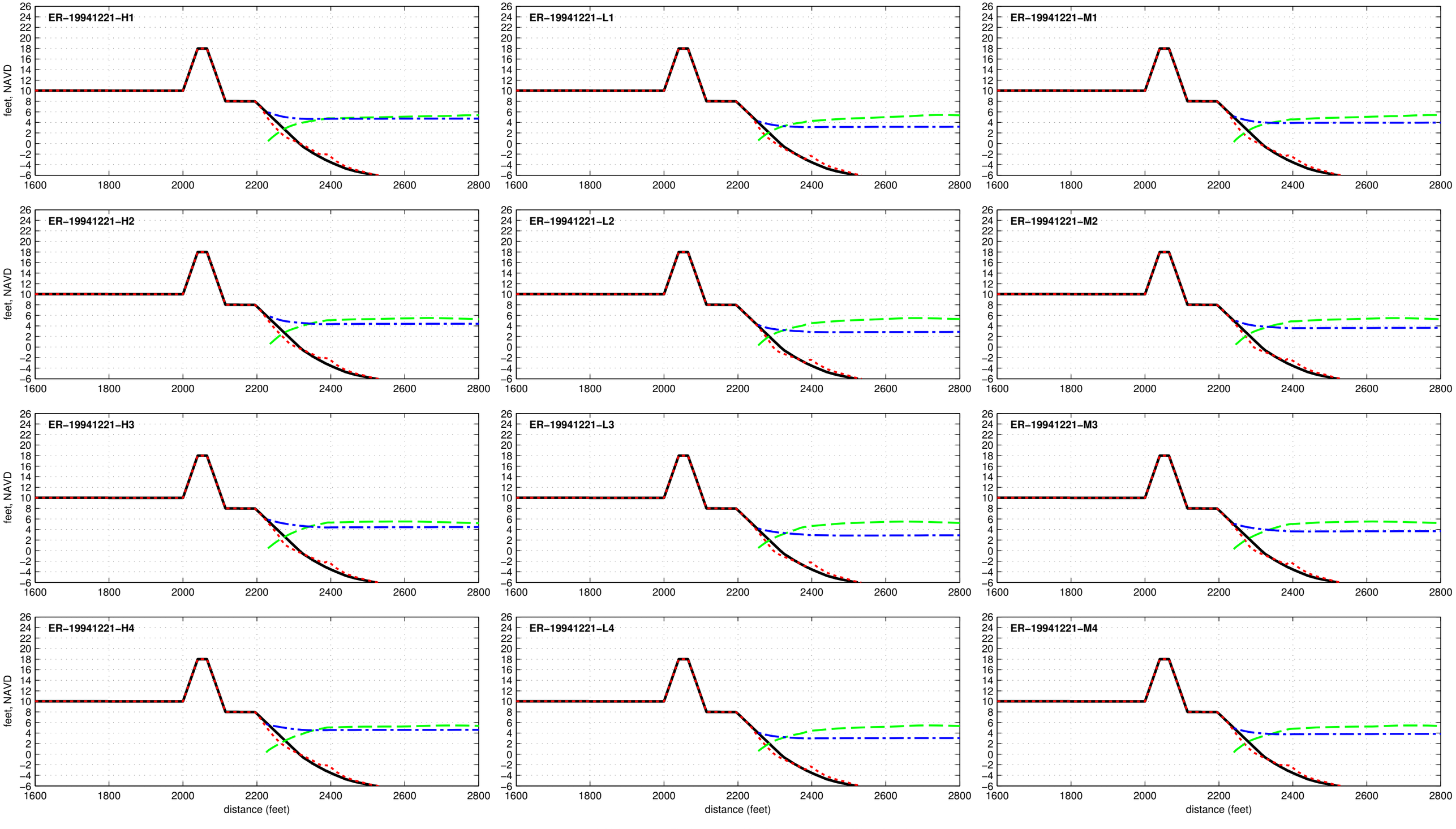

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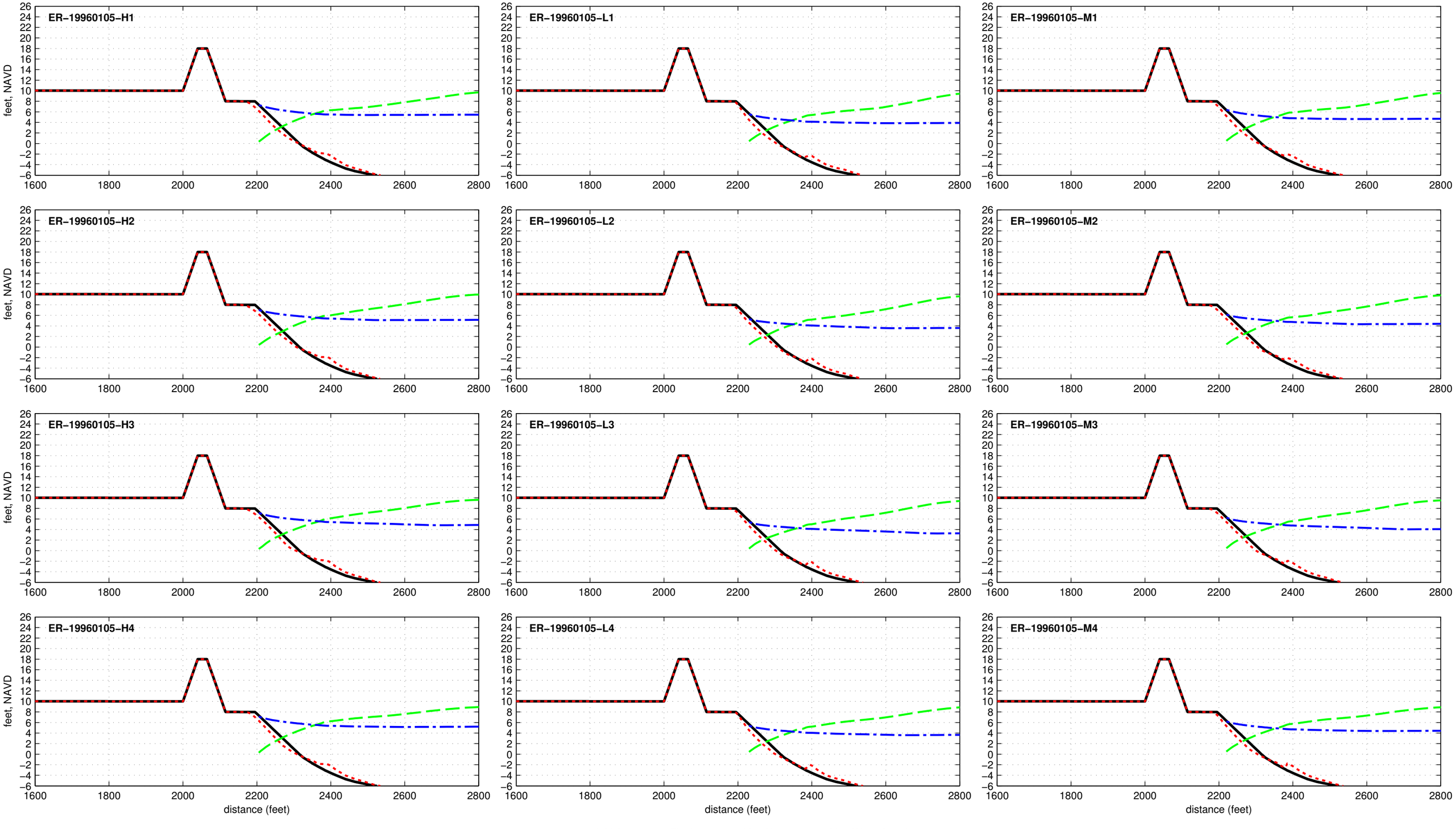

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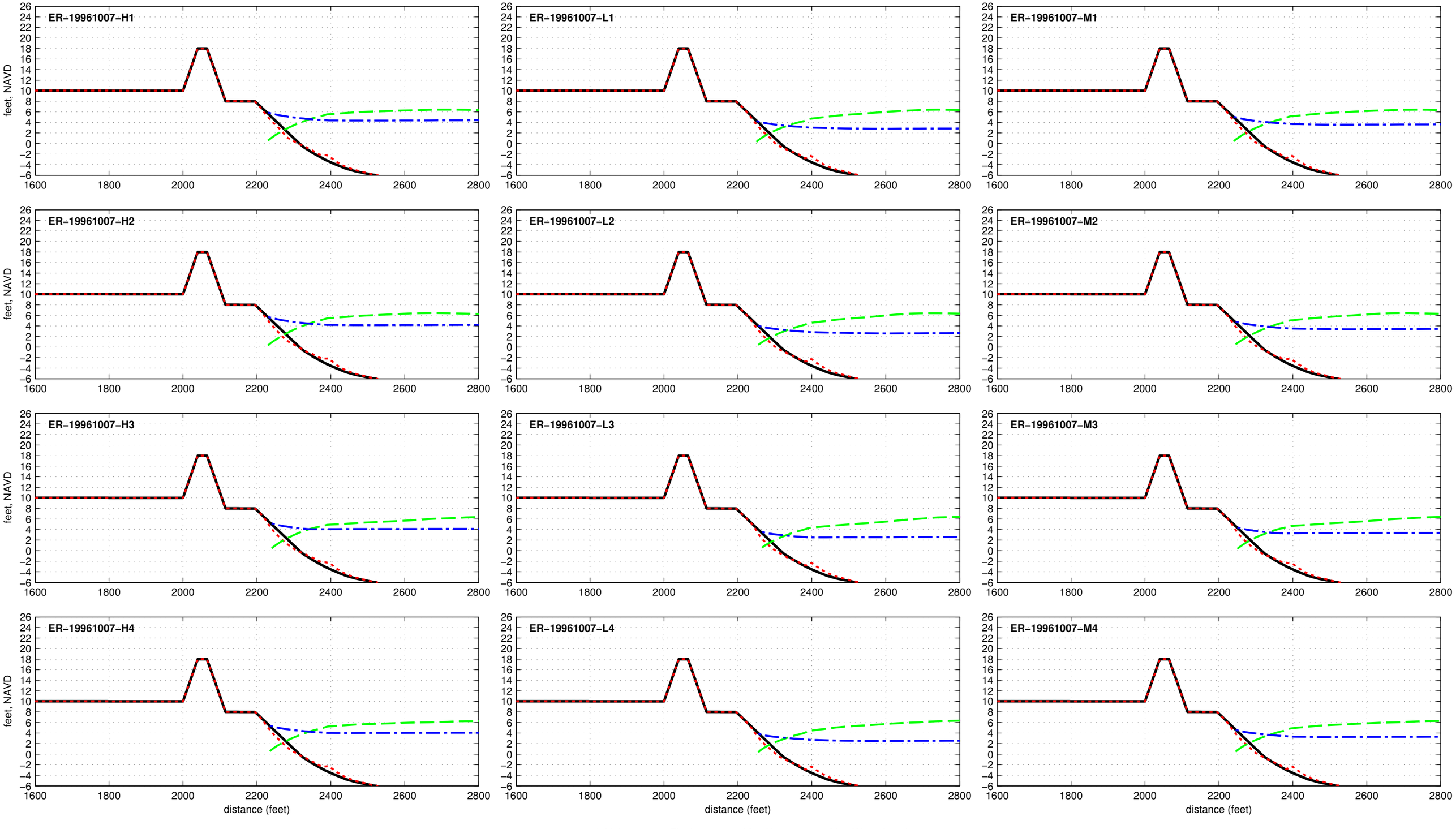

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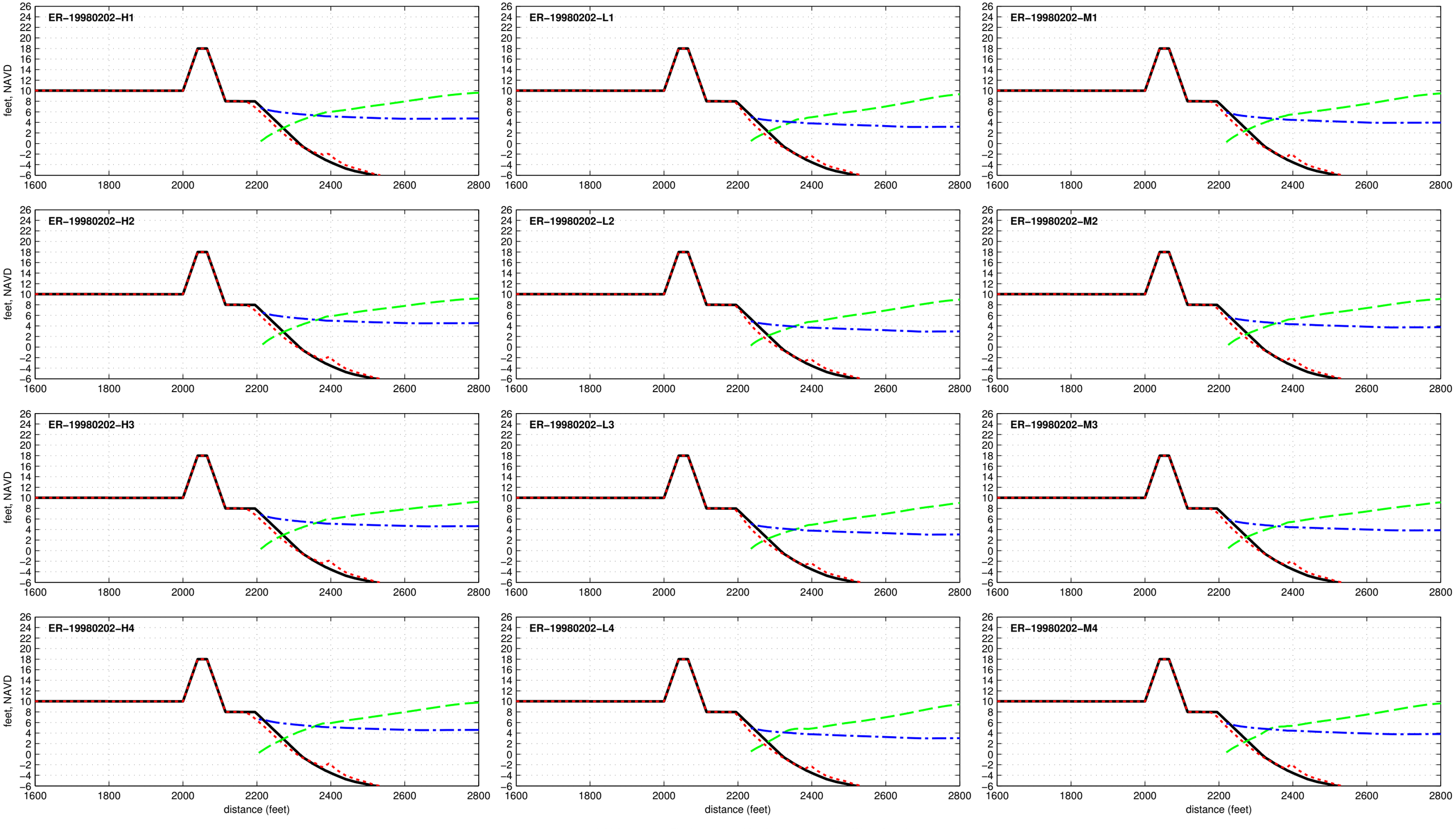

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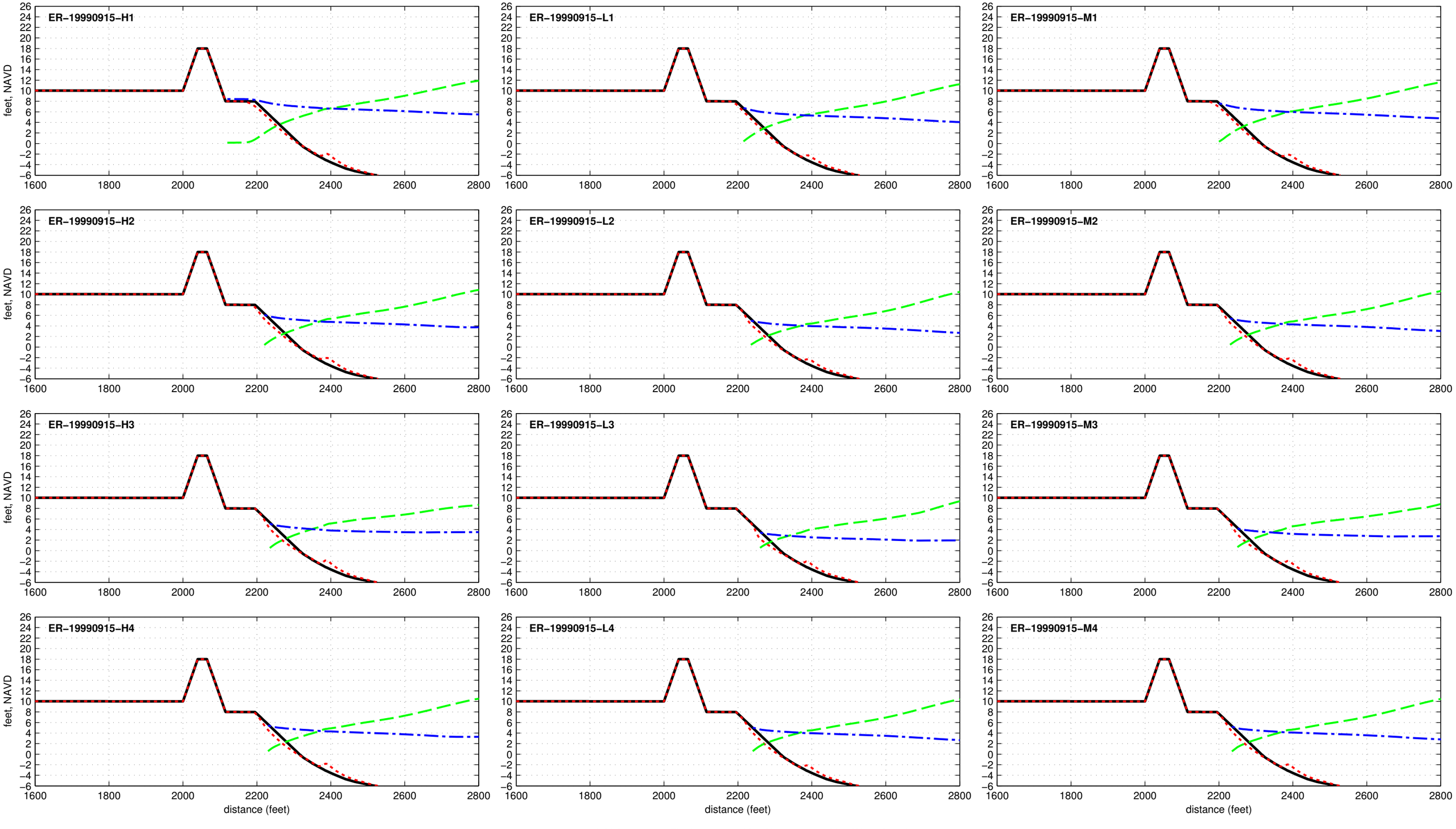
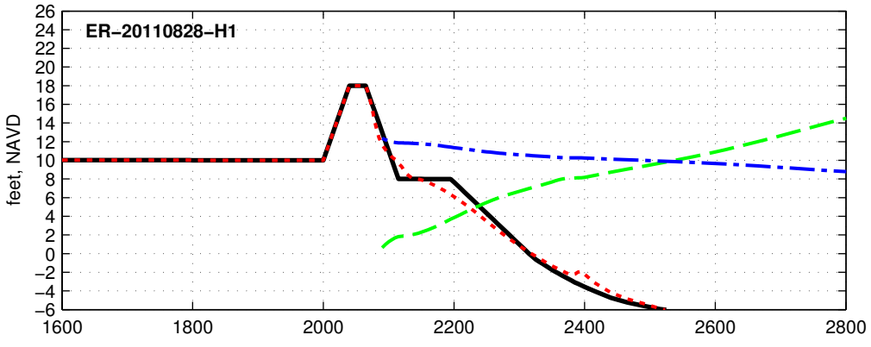
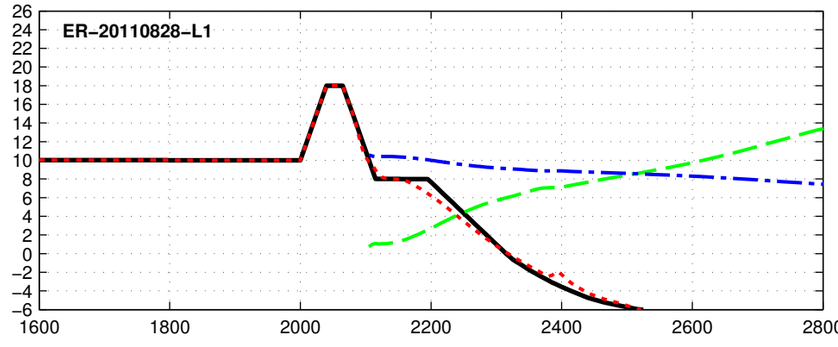
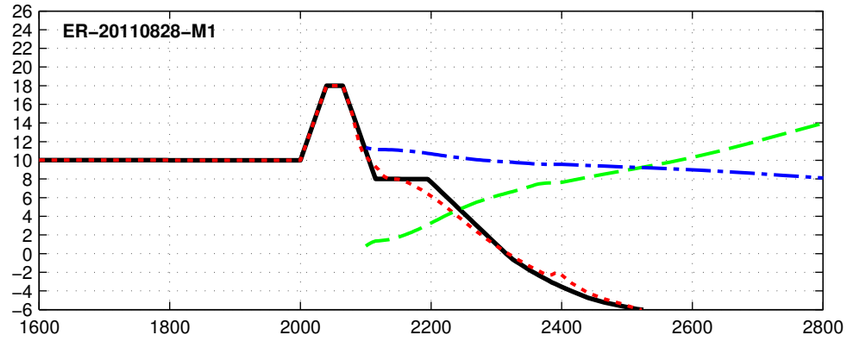
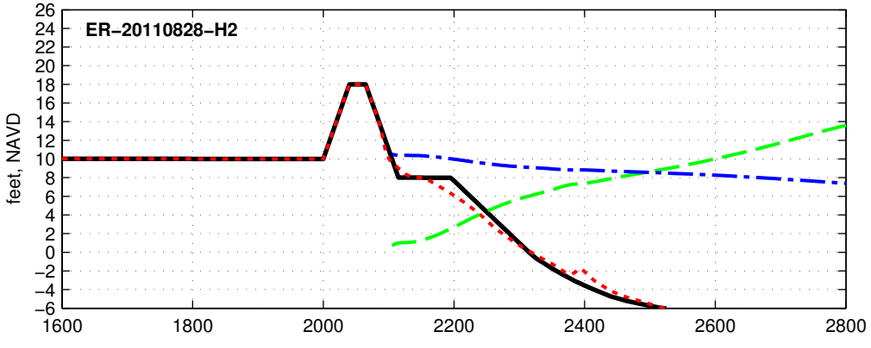


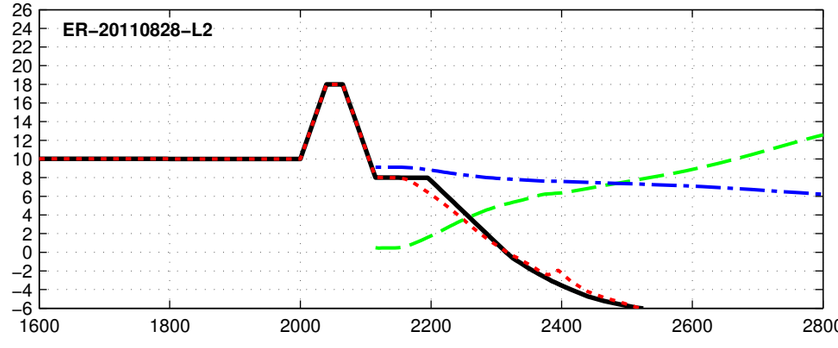
Dune R2T1-18.0-025-080 - ER-20110828

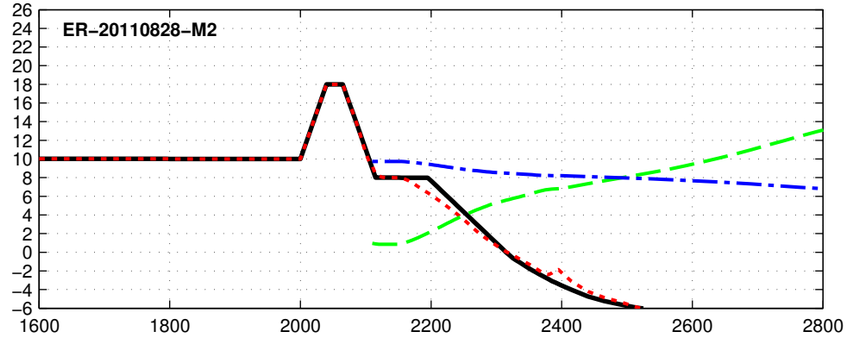


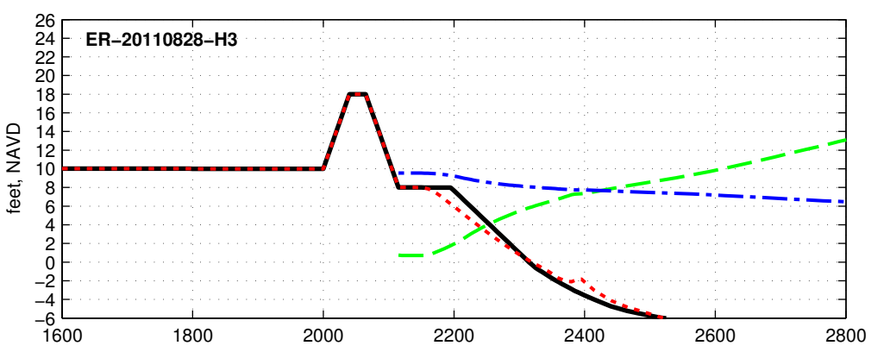


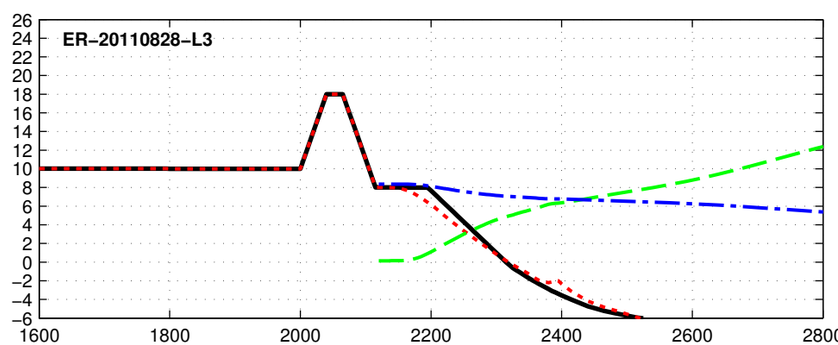


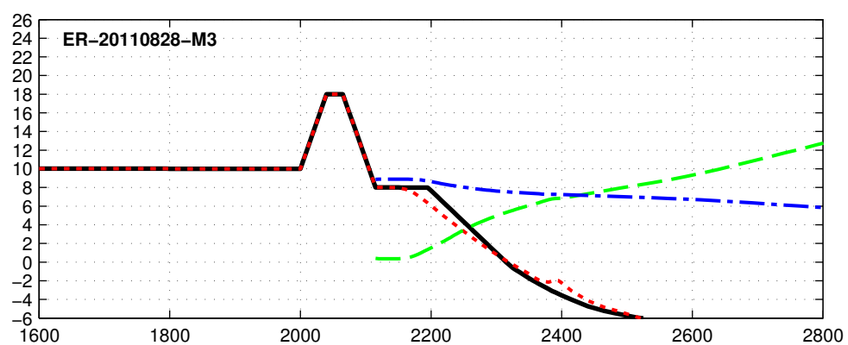


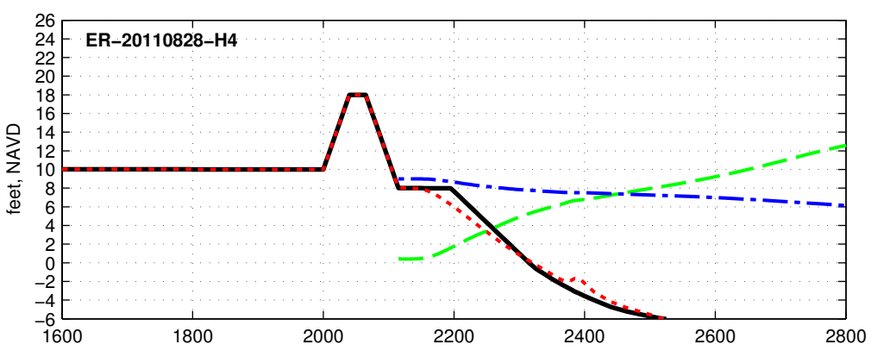


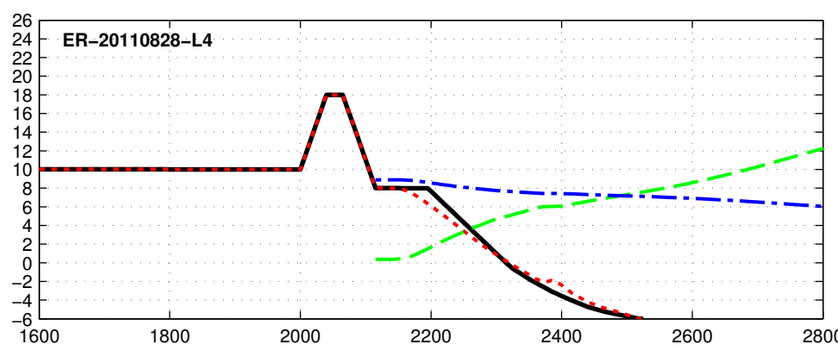


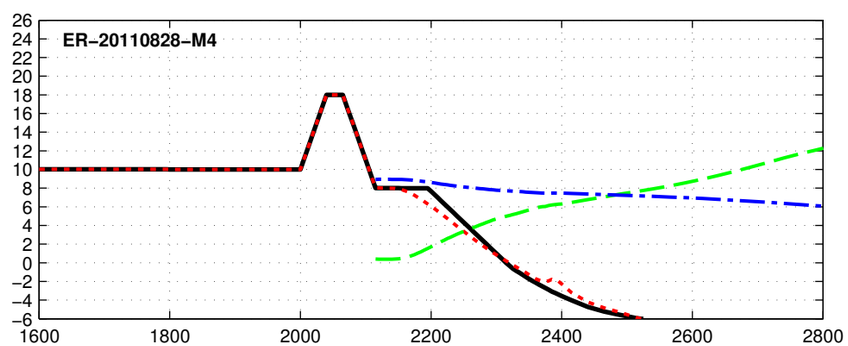






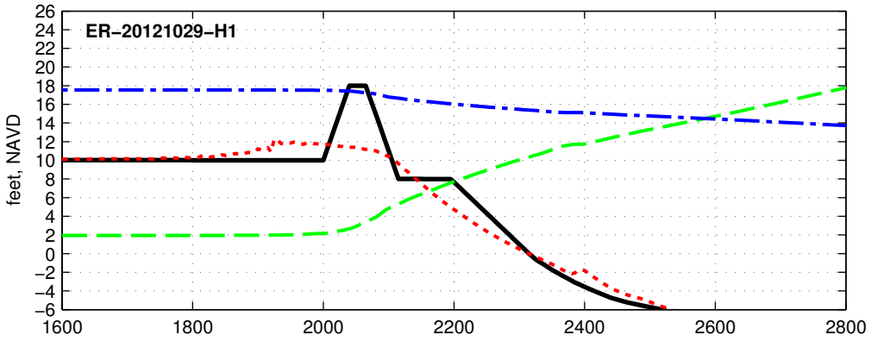


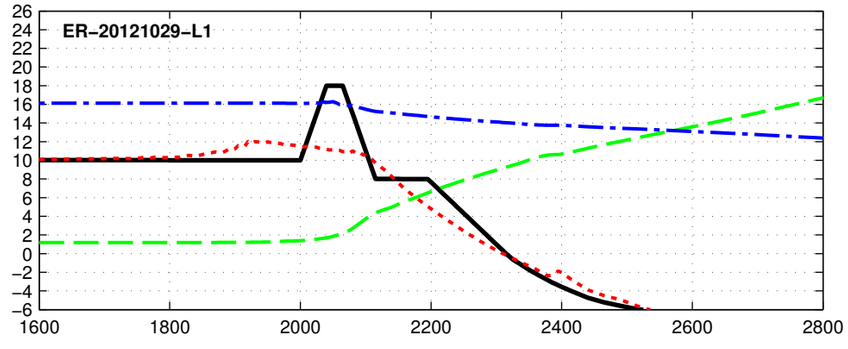


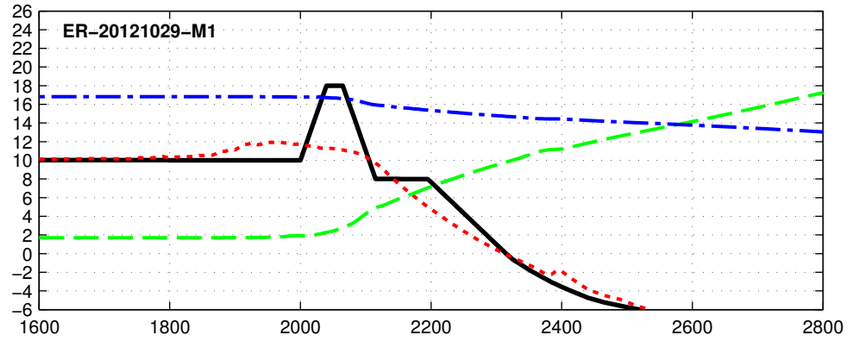


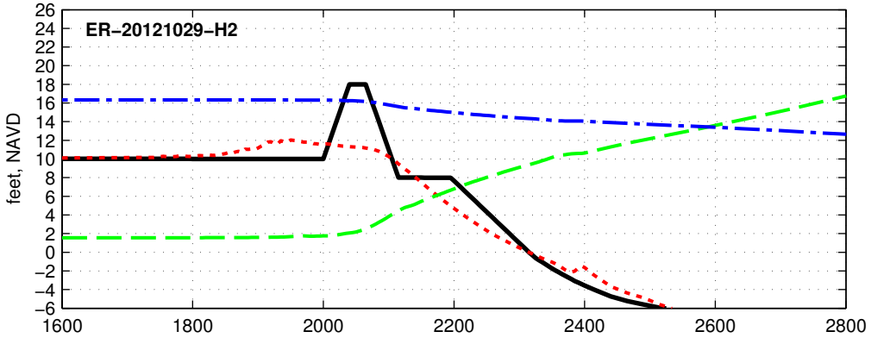
Initial Final Max Hs Max Water Level

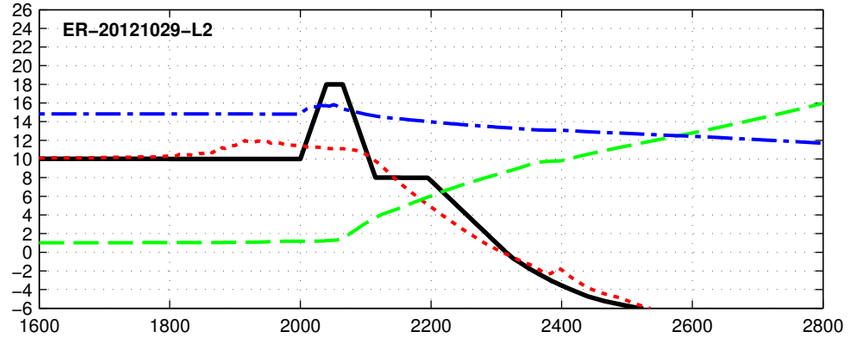
Dune R2T1-18.0-025-080 - ER-20121029

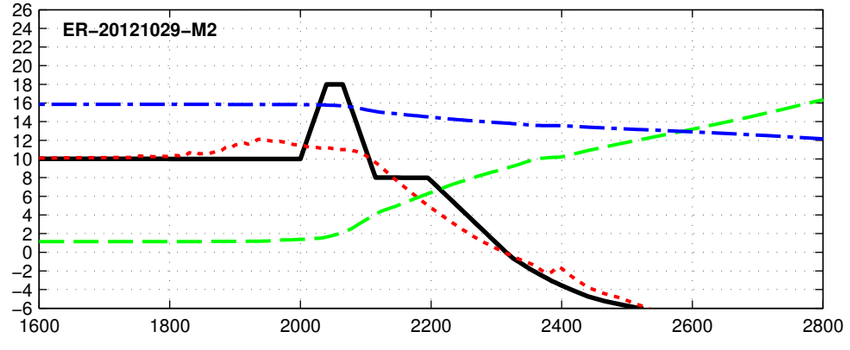


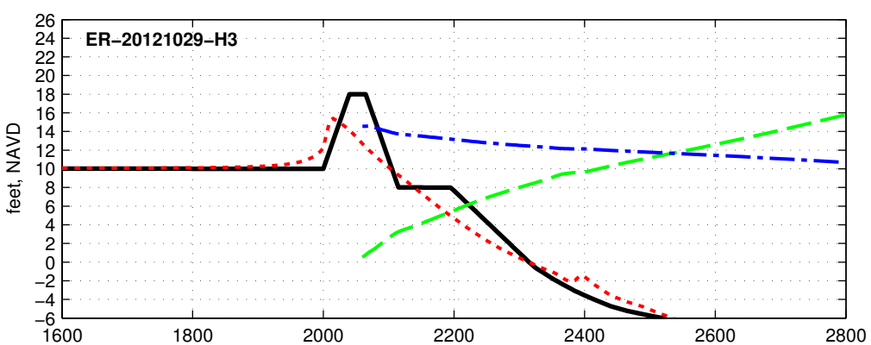


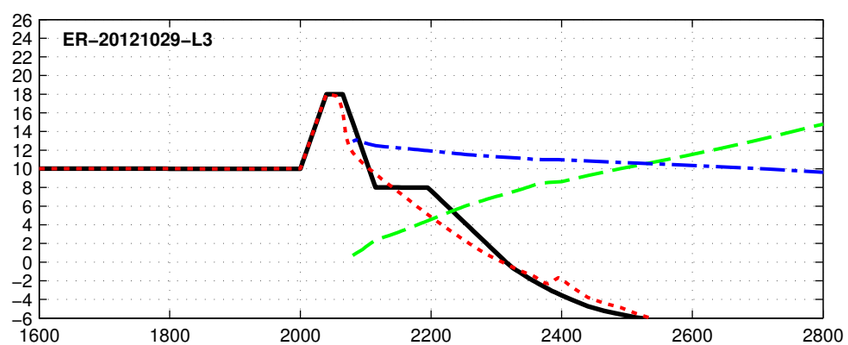


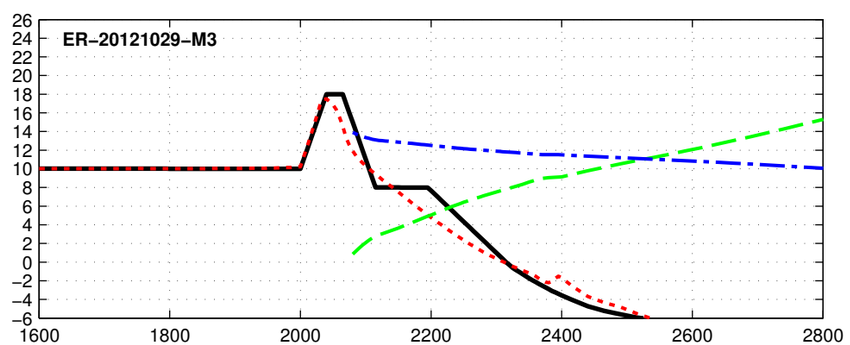


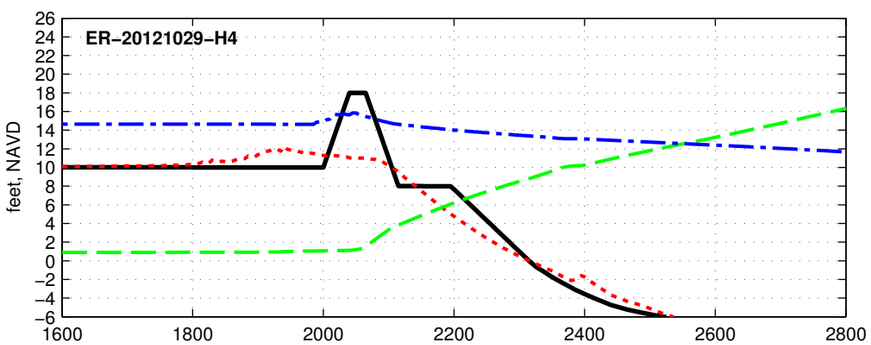


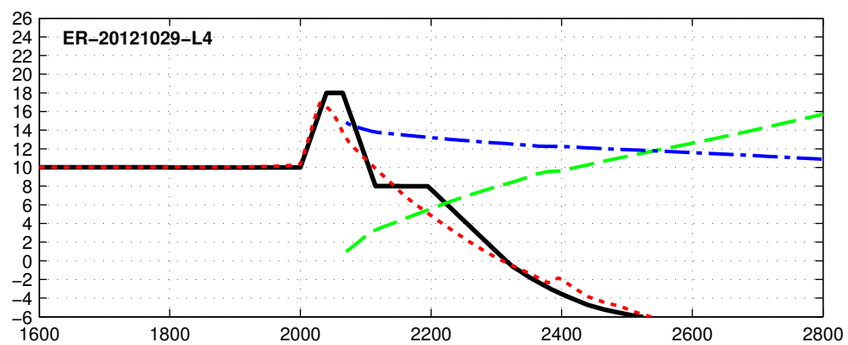


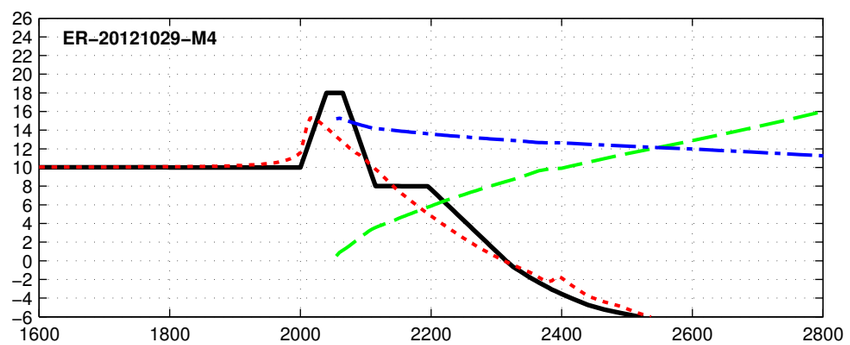








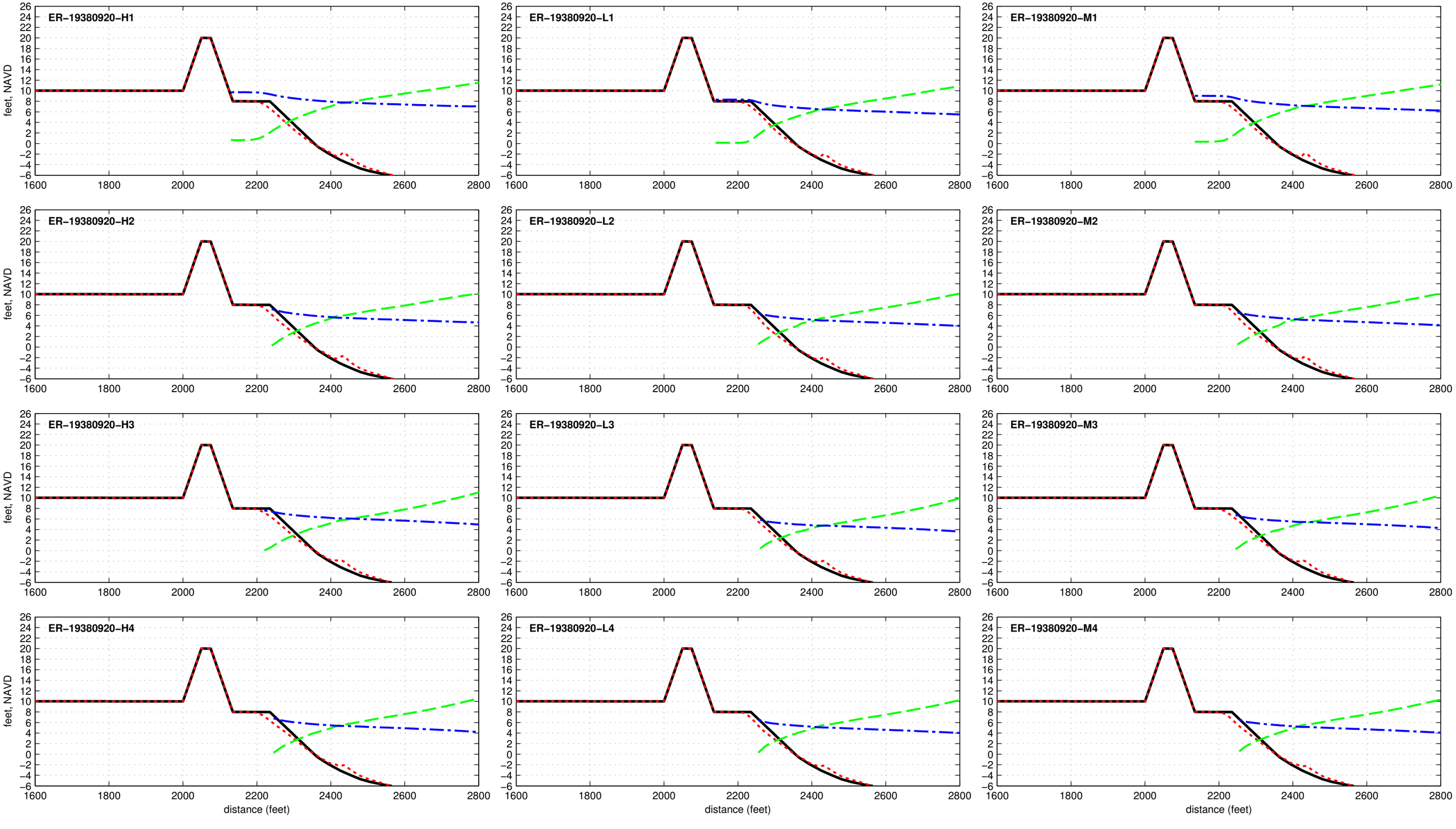




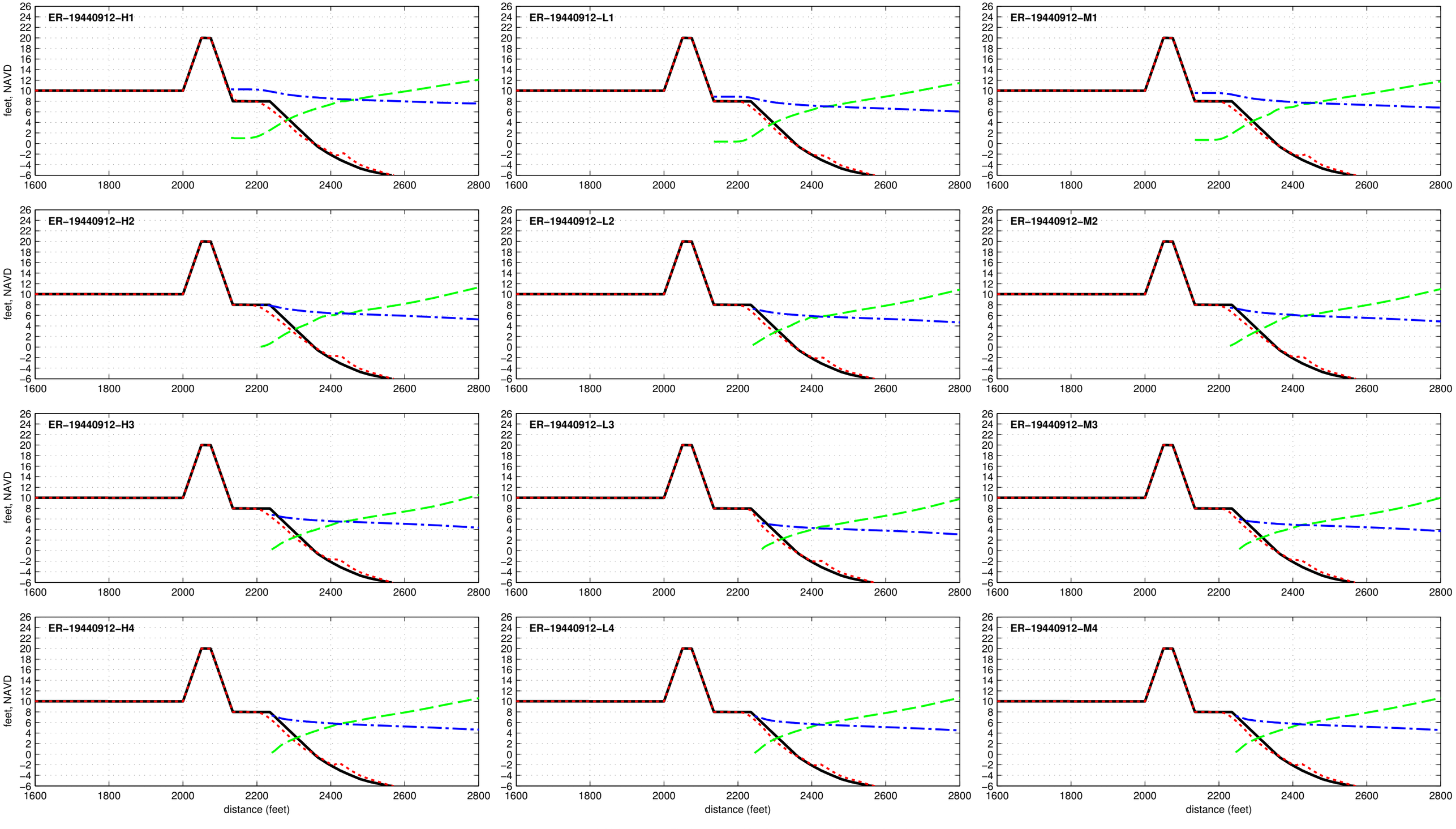
Initial Final Max Hs Max Water Level

R2T1_20.0_025_100 - DUNE

Dune R2T1-20.0-025-100 - ER-19380920



Dune R2T1-20.0-025-100 - ER-19440912



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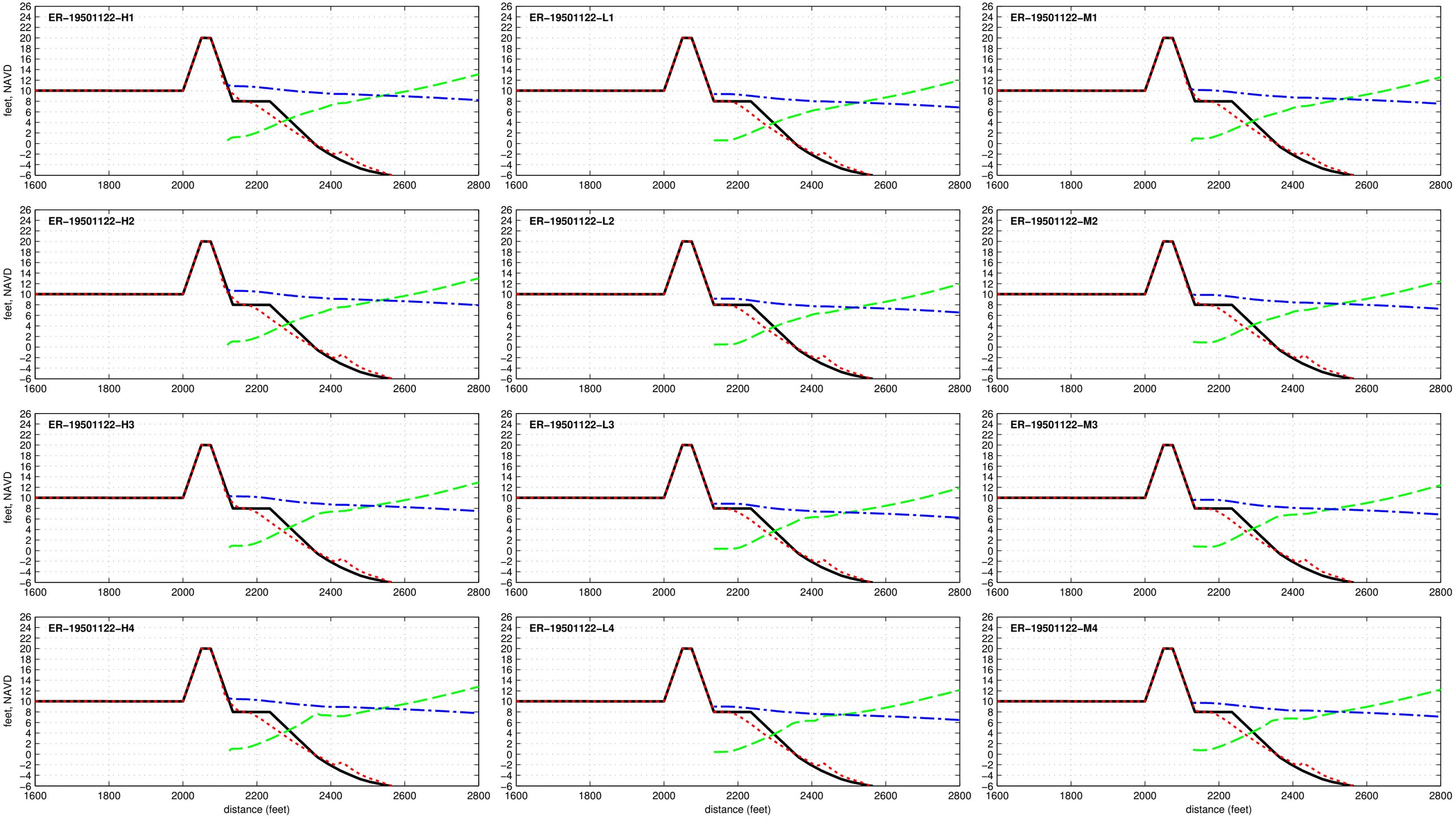

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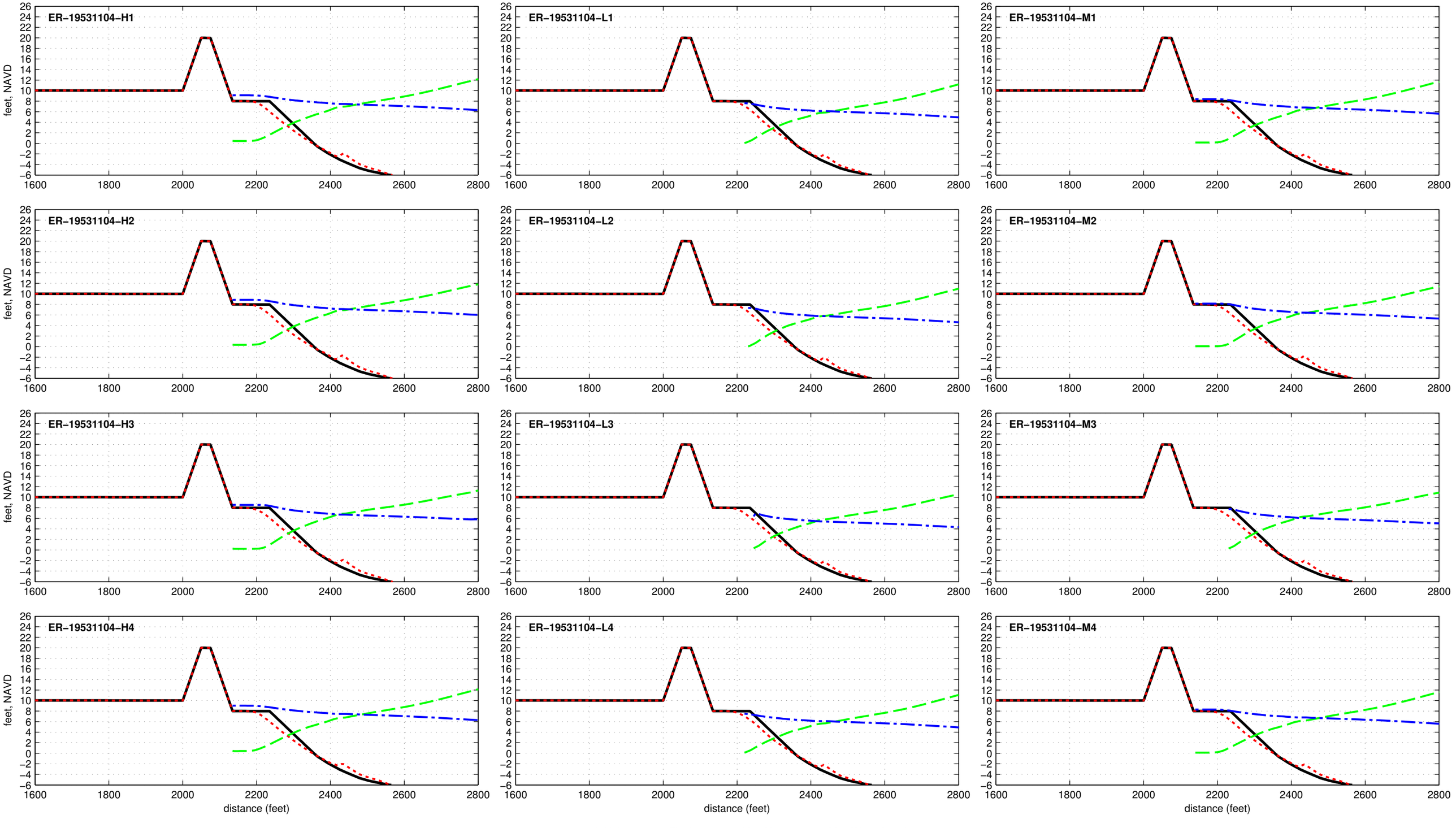

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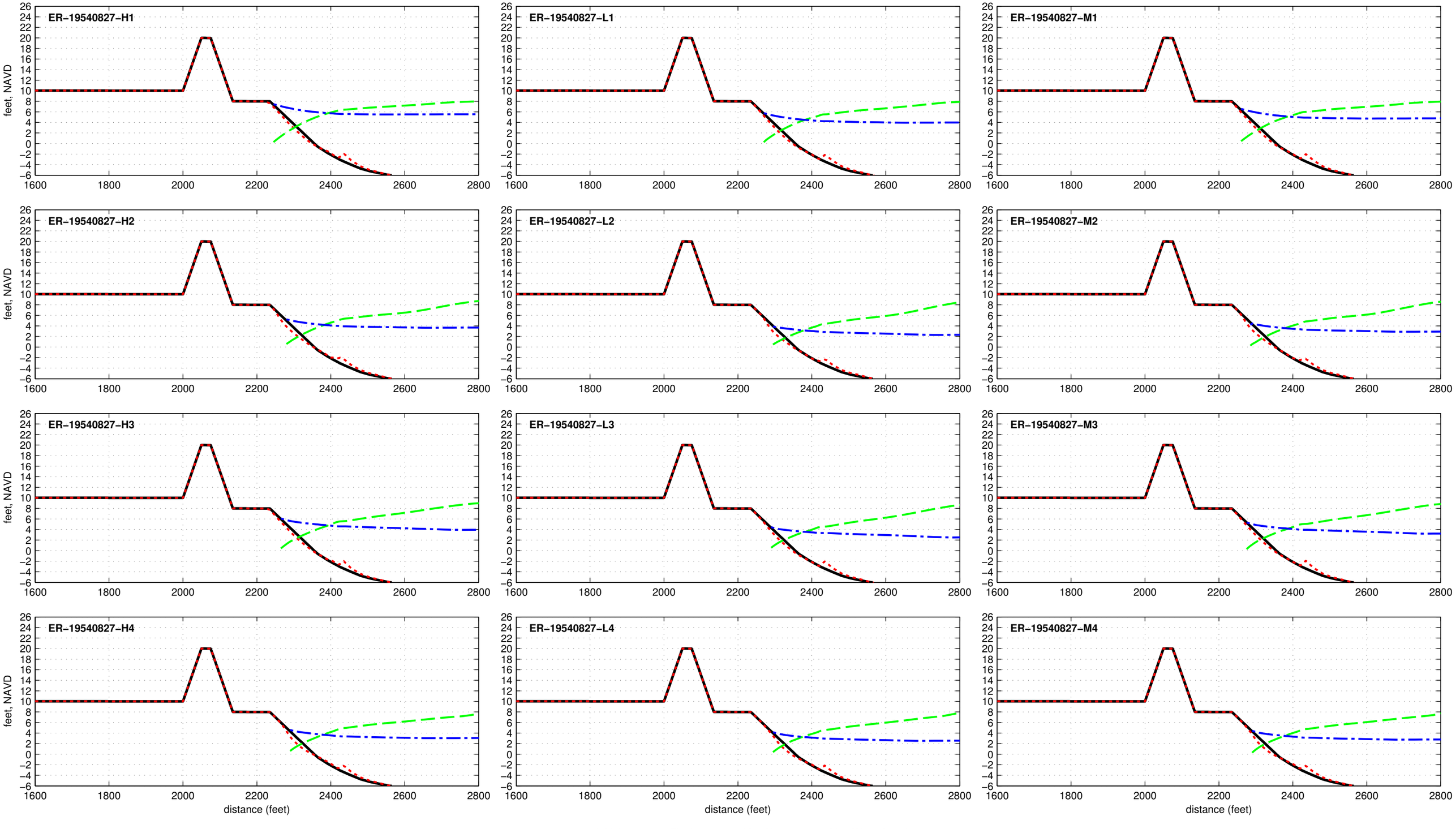

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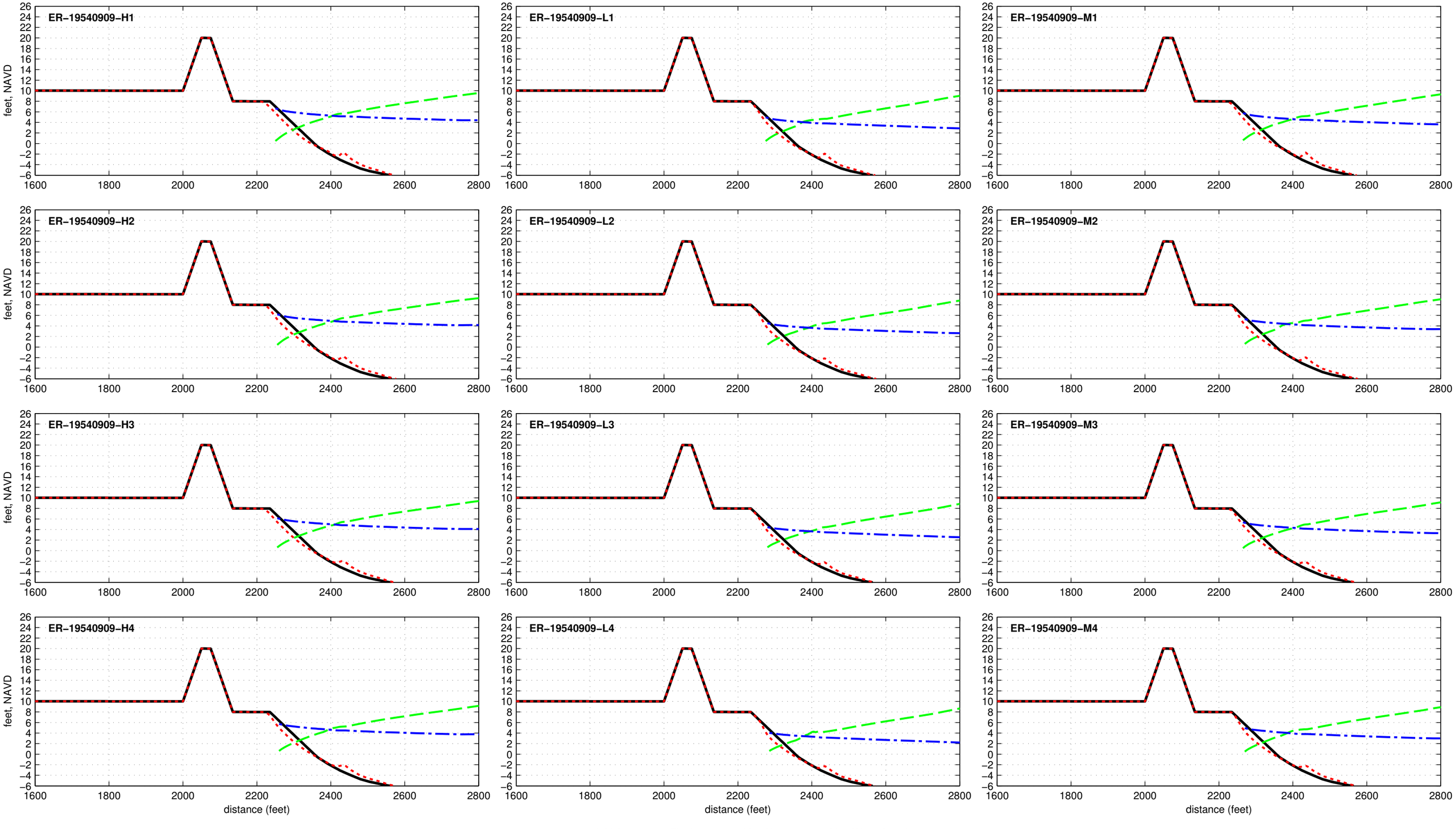

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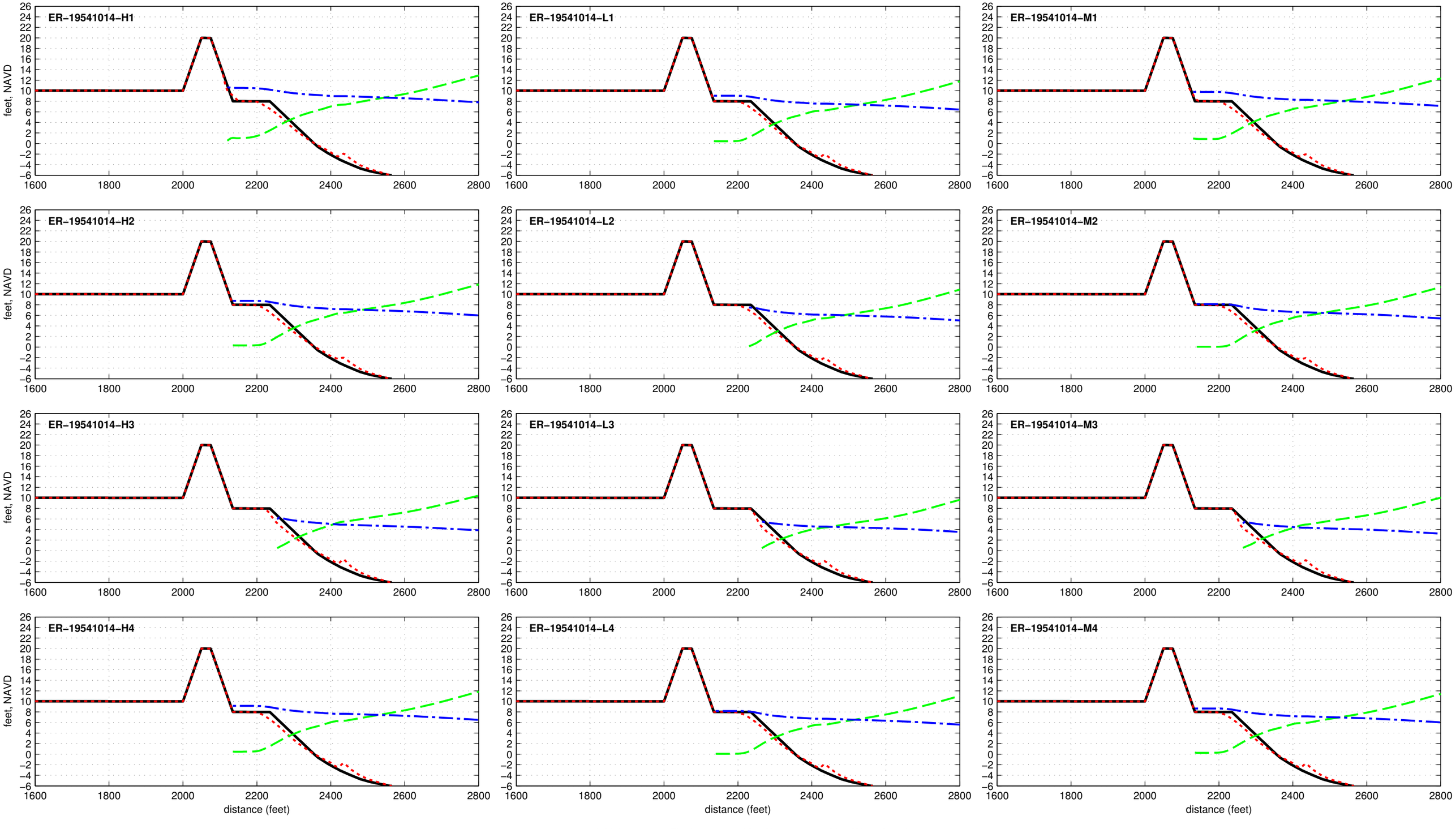

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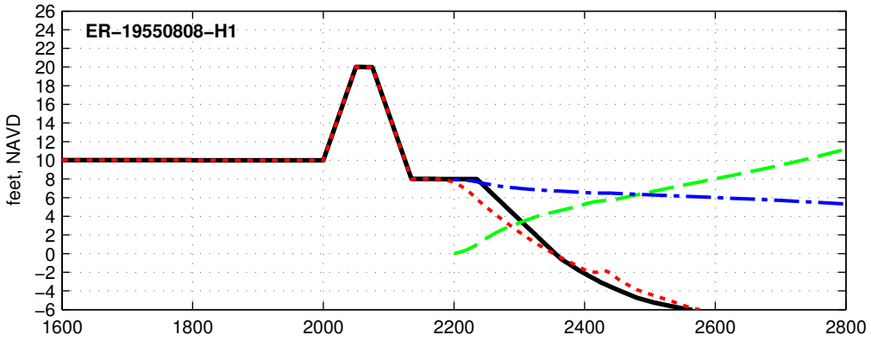
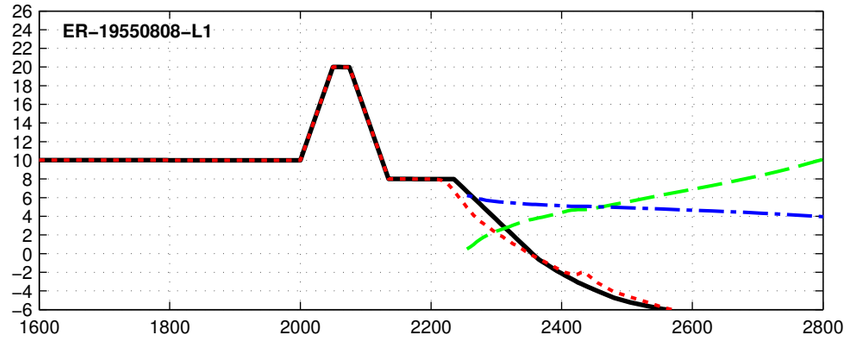
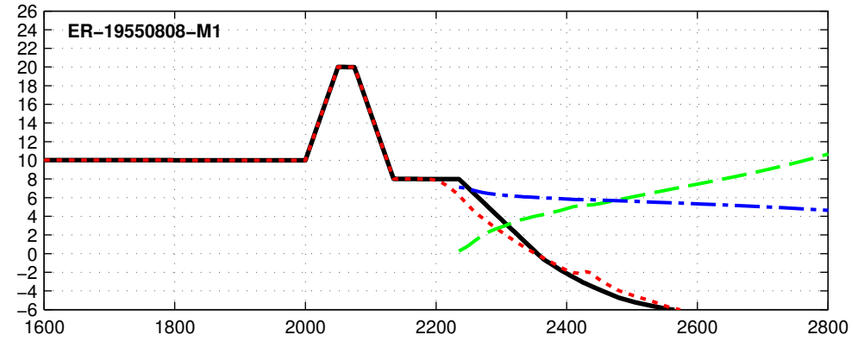
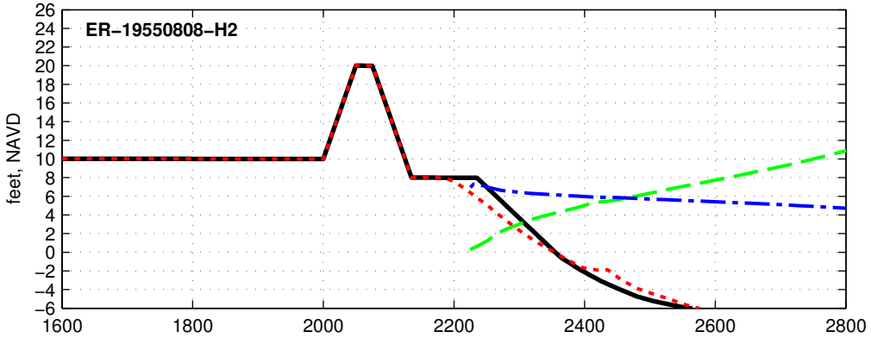
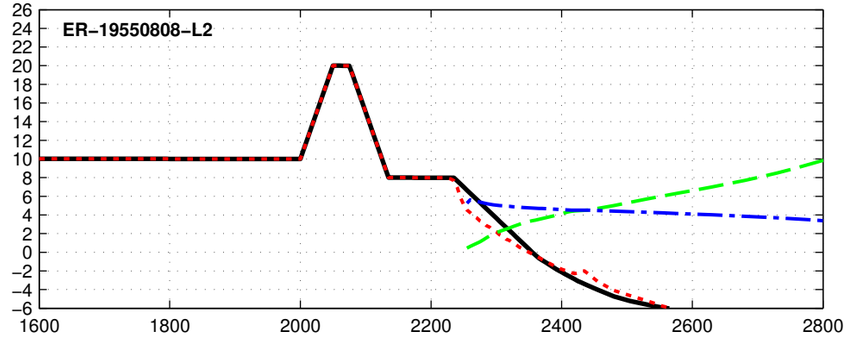
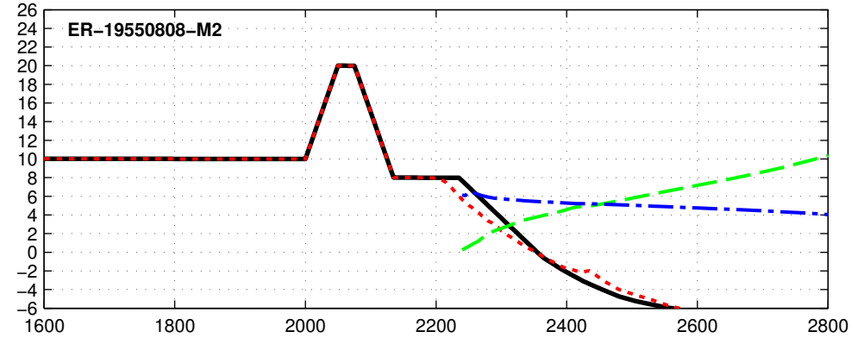


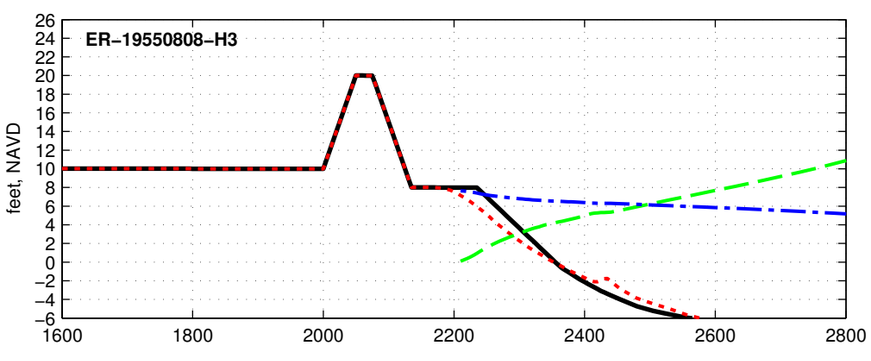


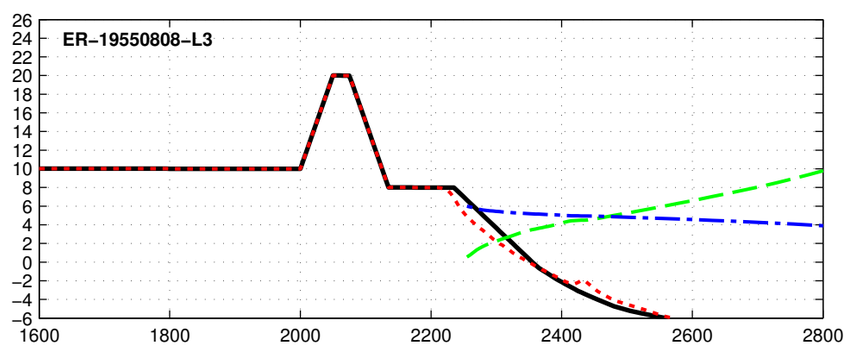


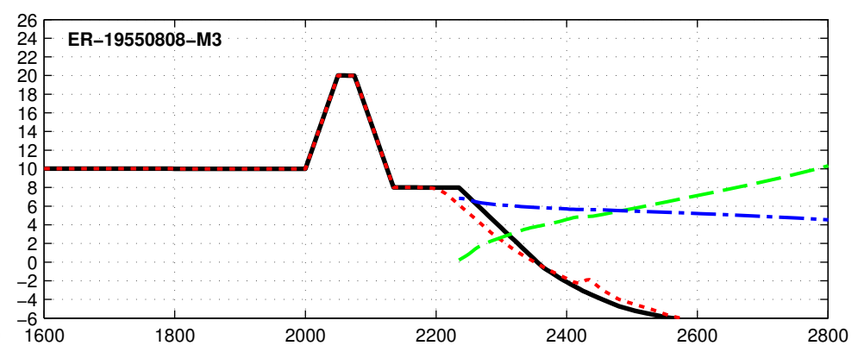


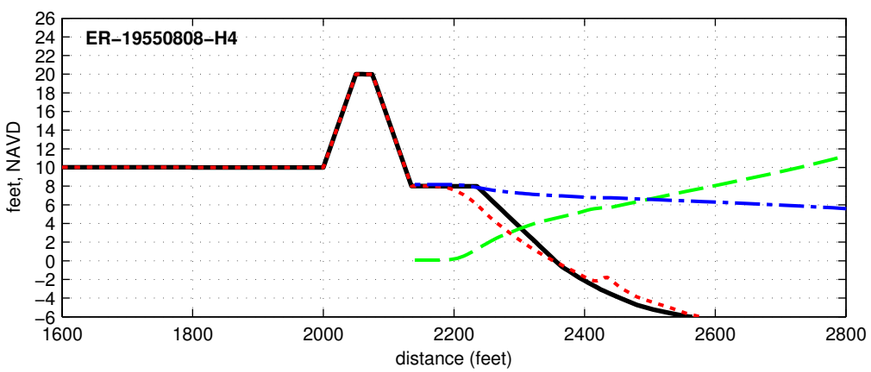


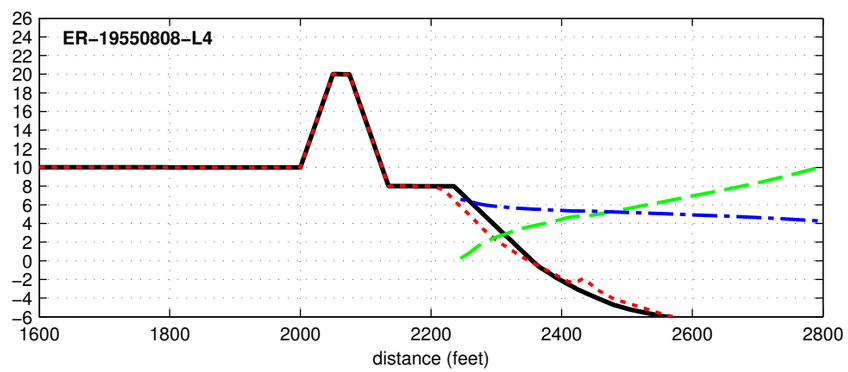


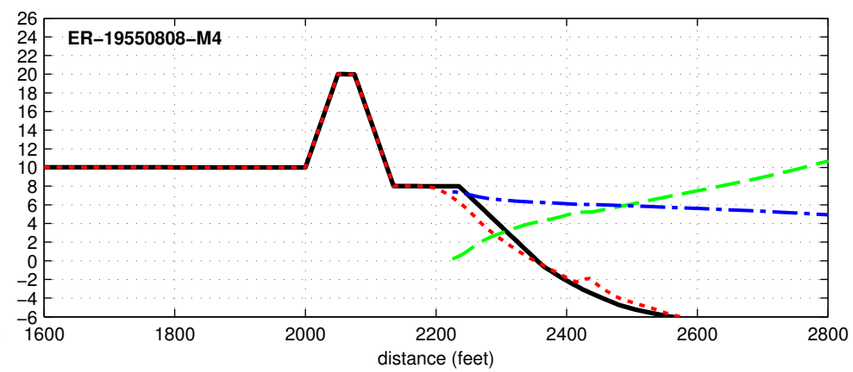






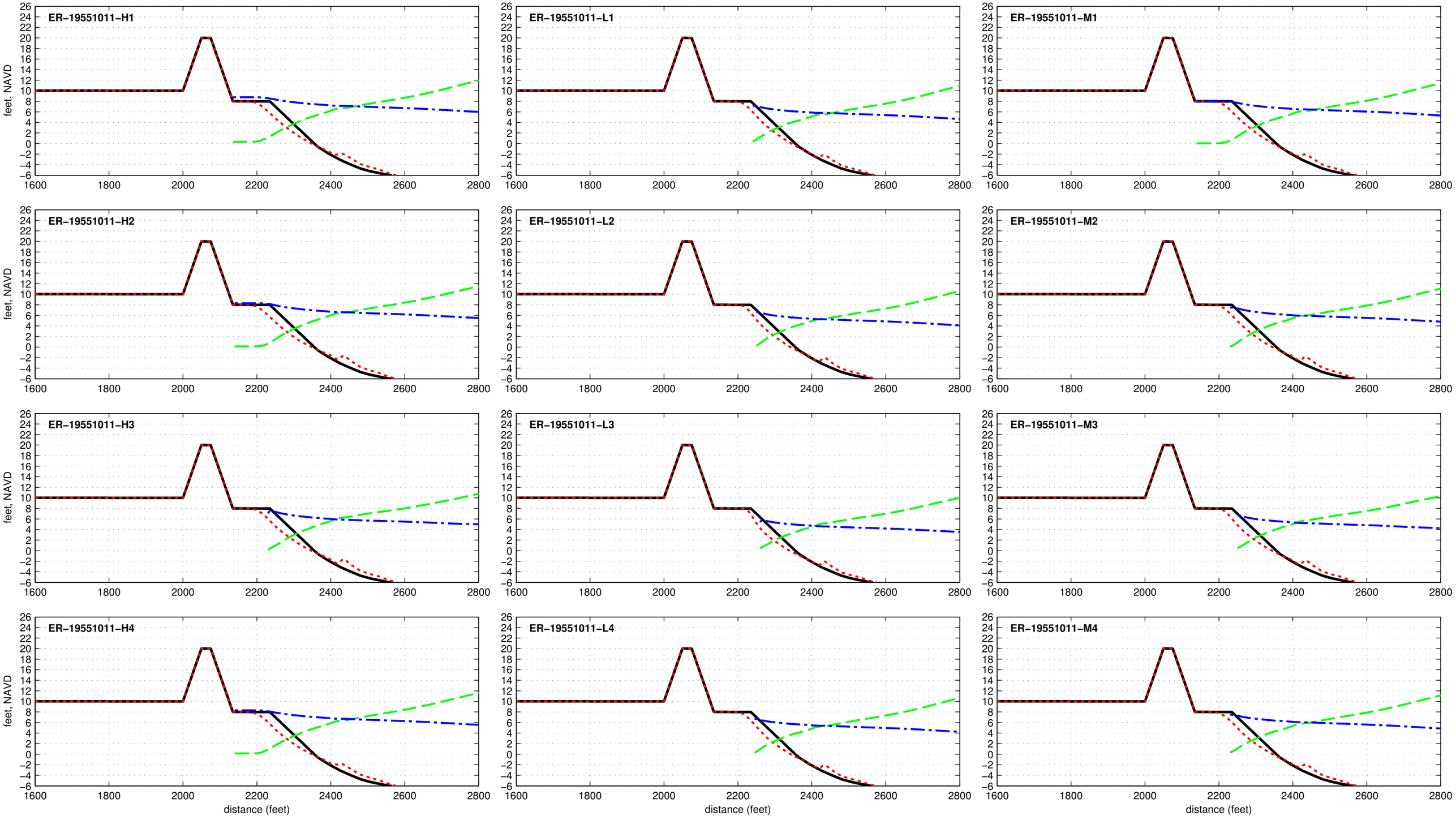






Initial Final Max Hs Max Water Level

Dune R2T1-20.0-025-100 - ER-19551011



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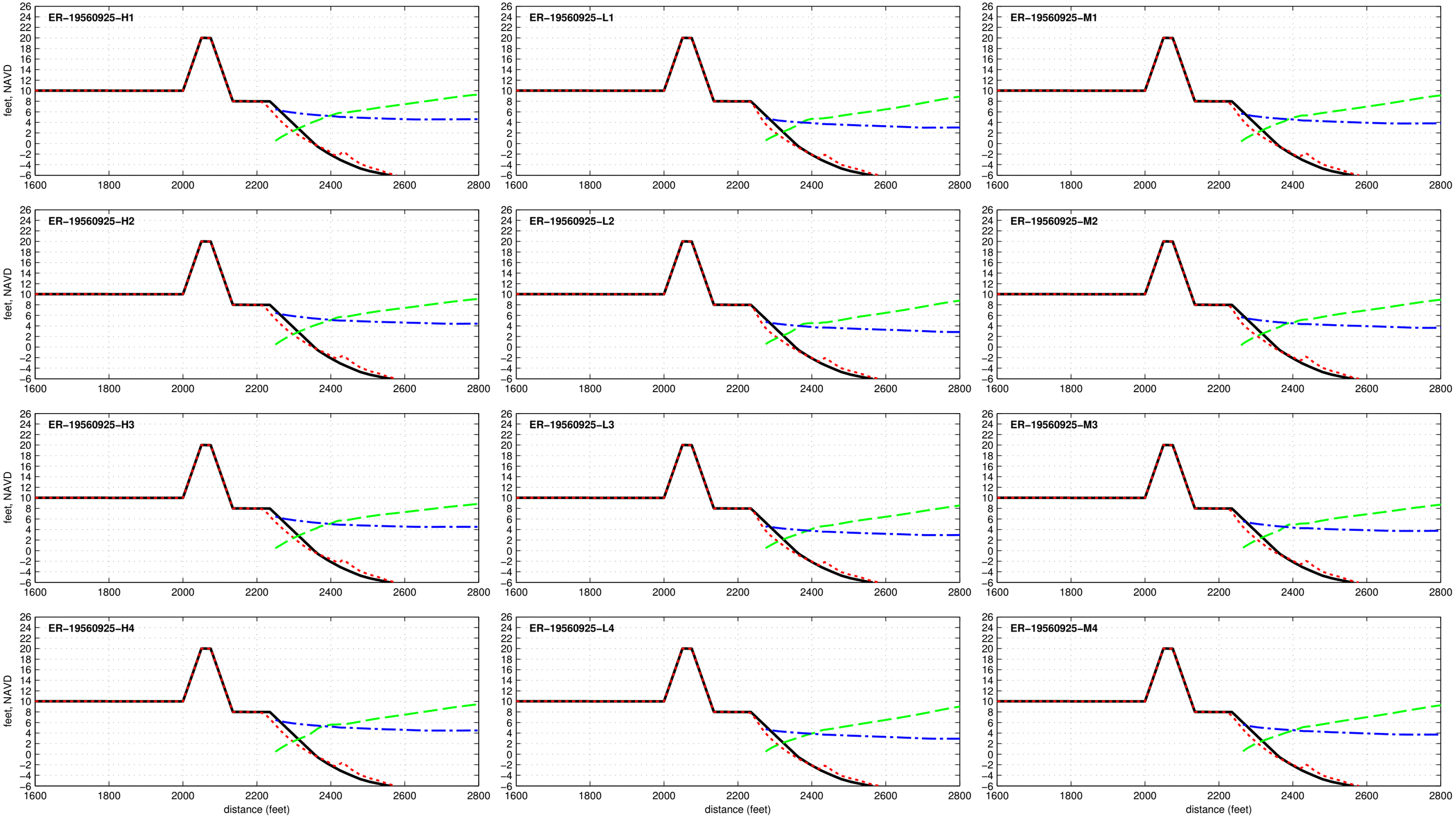

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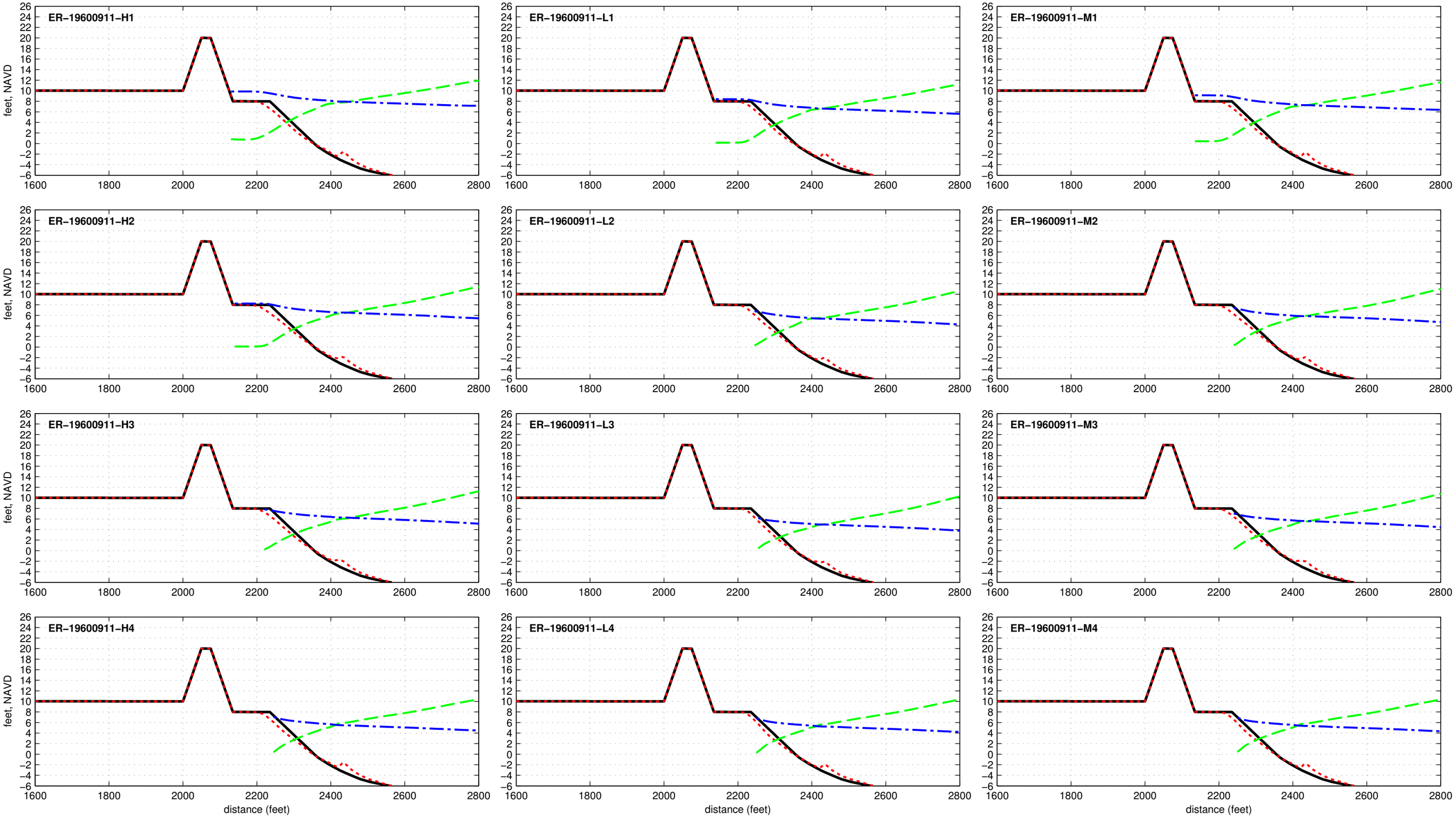

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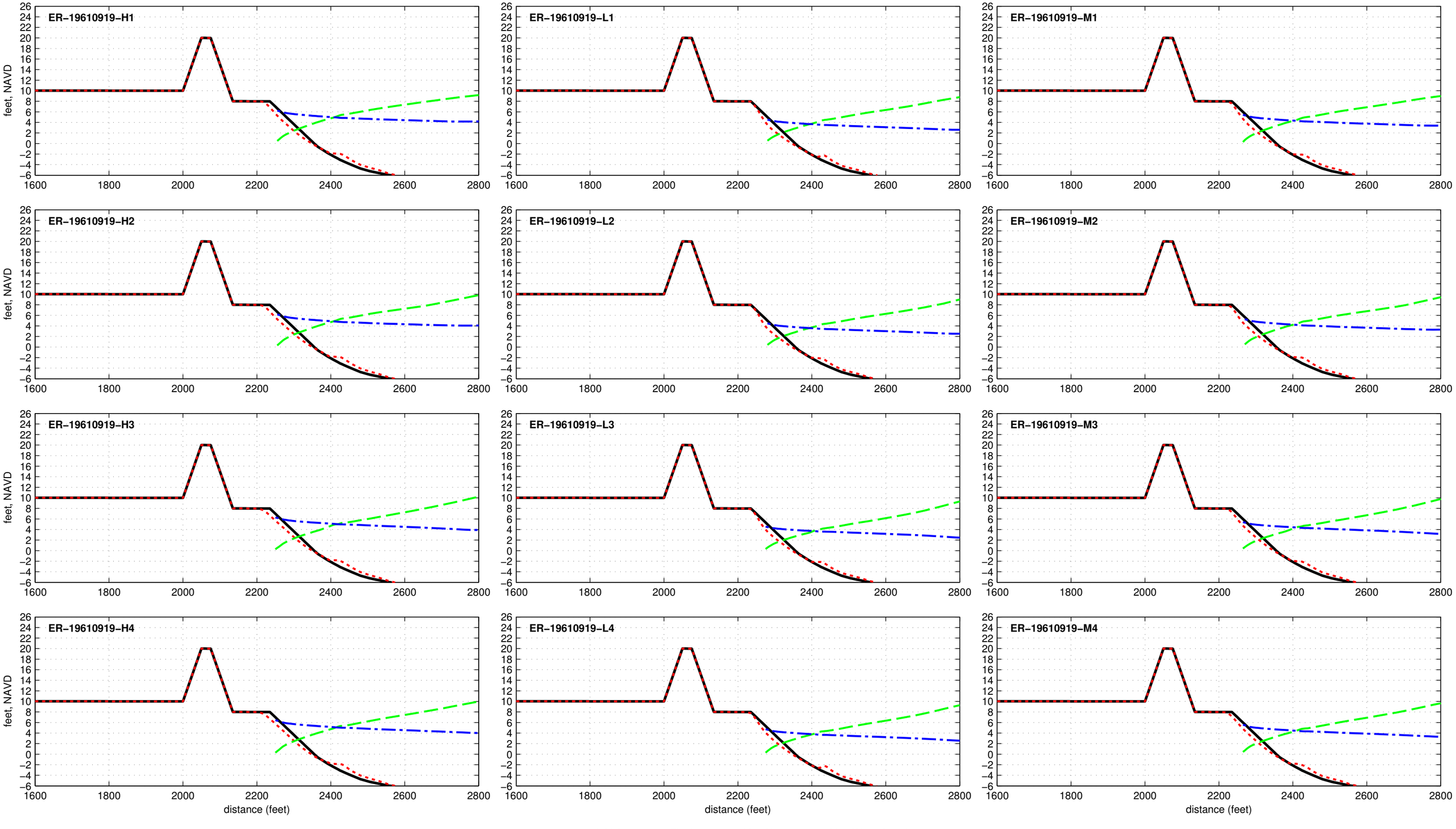

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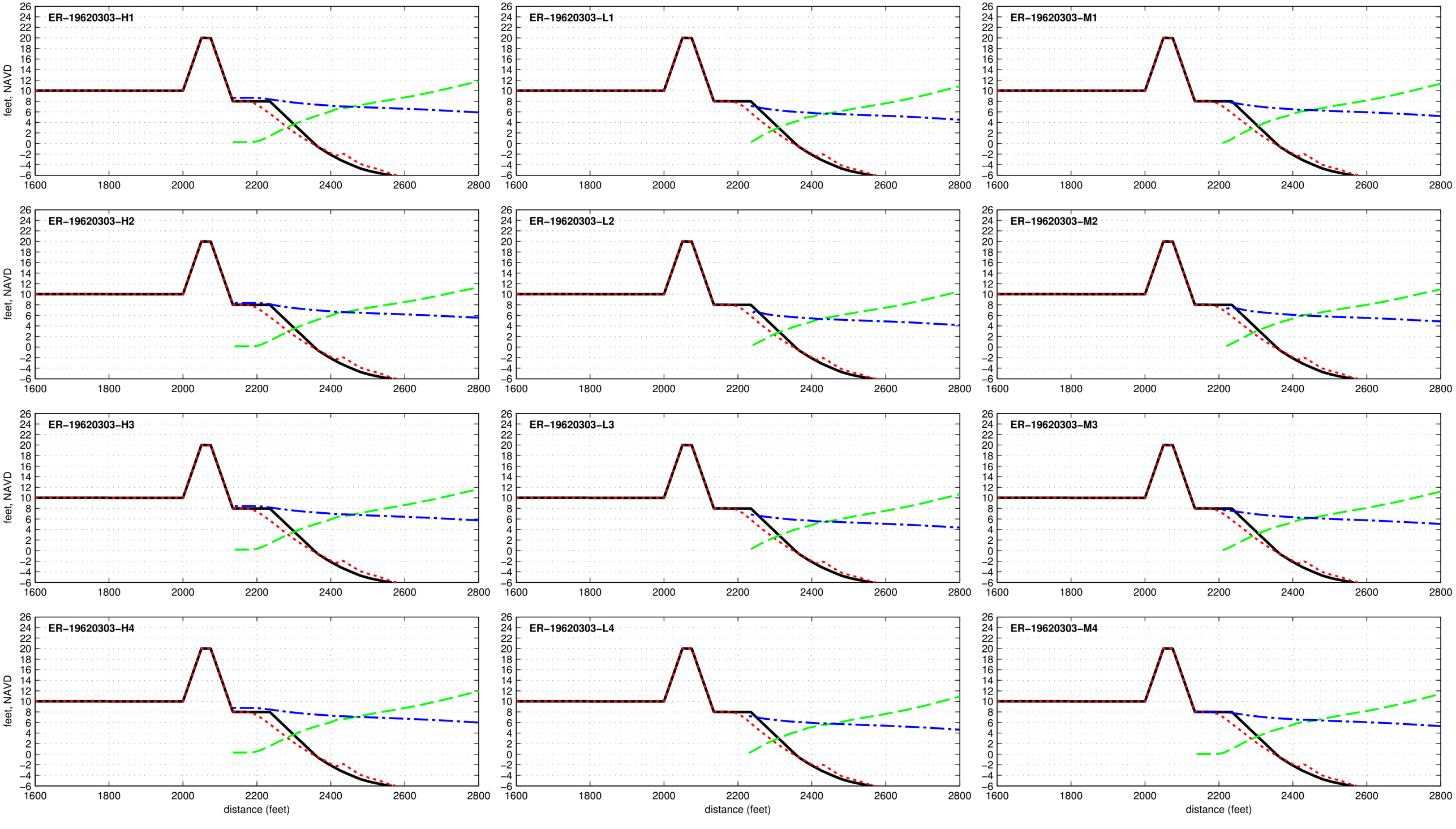

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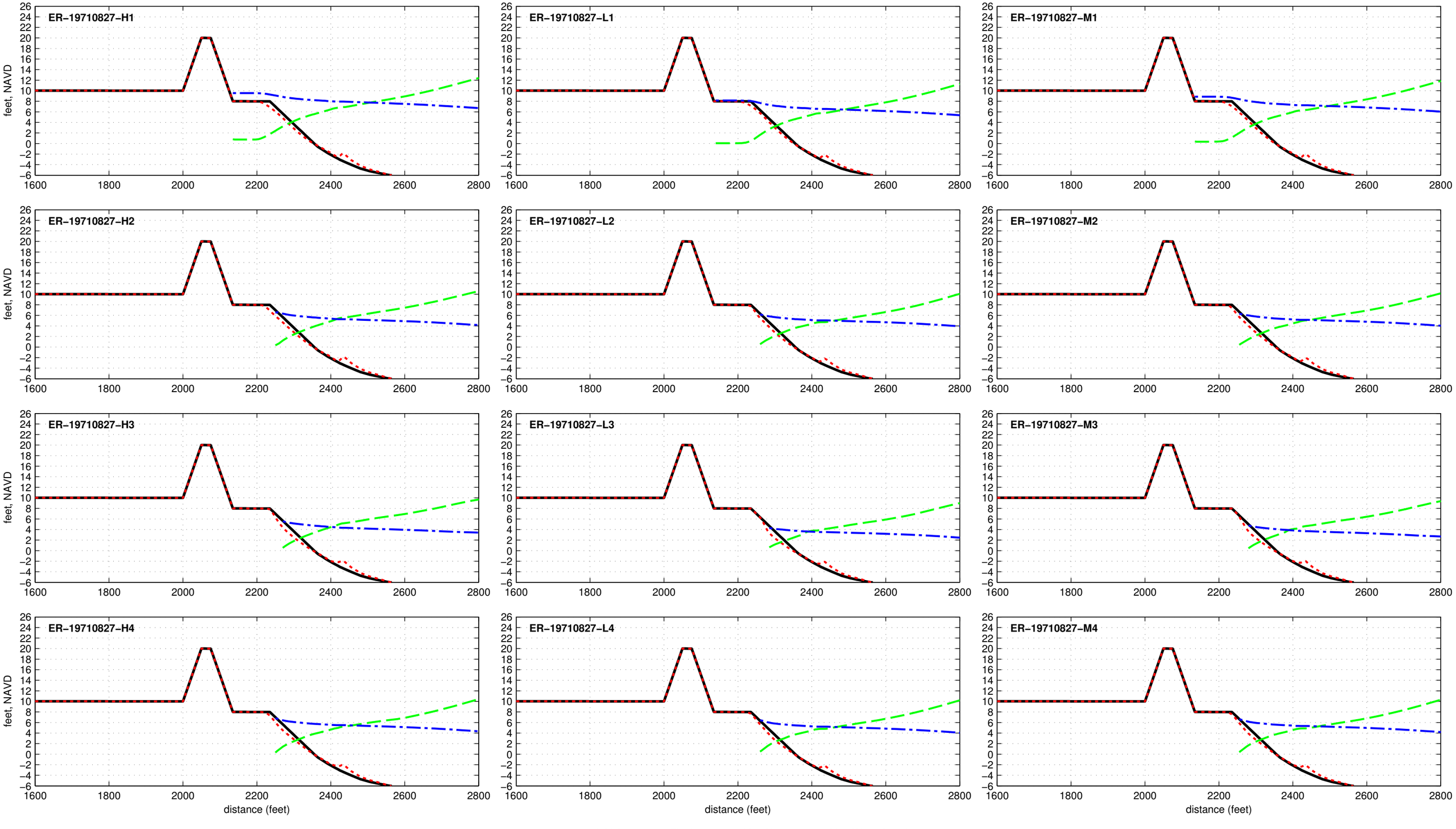

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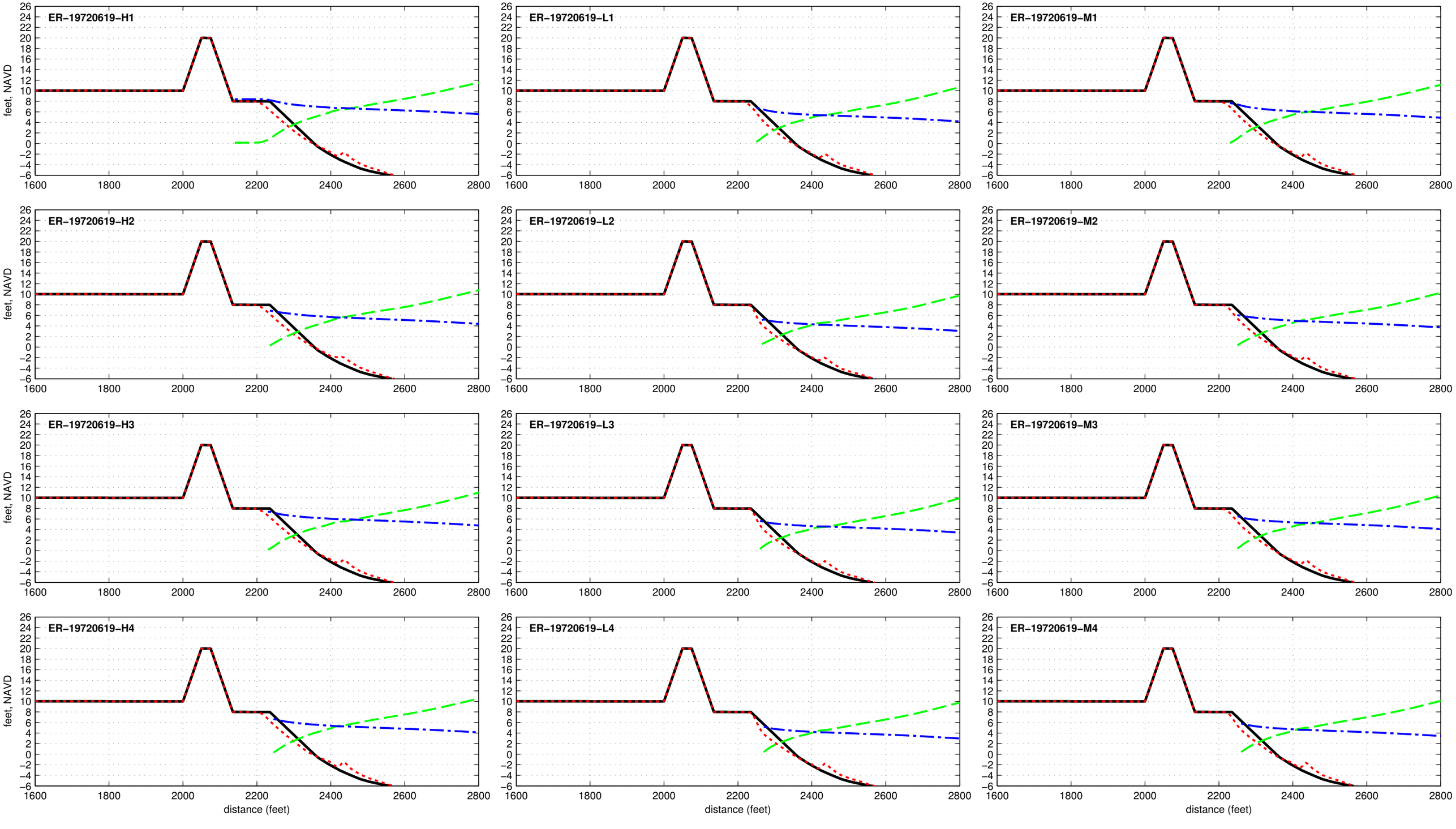

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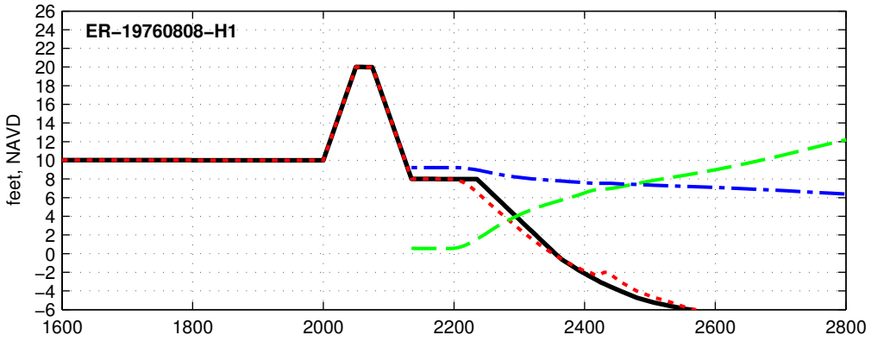
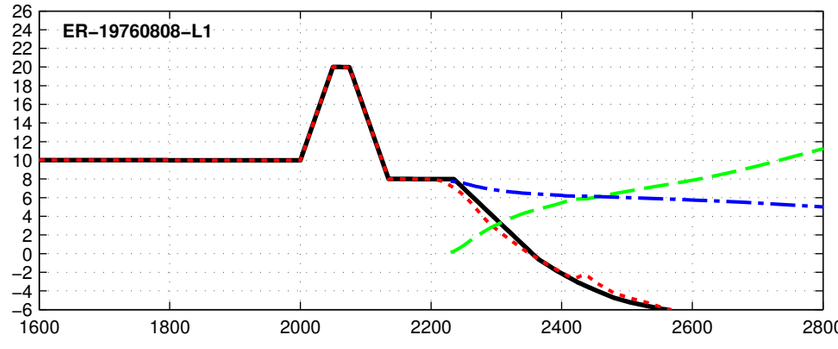
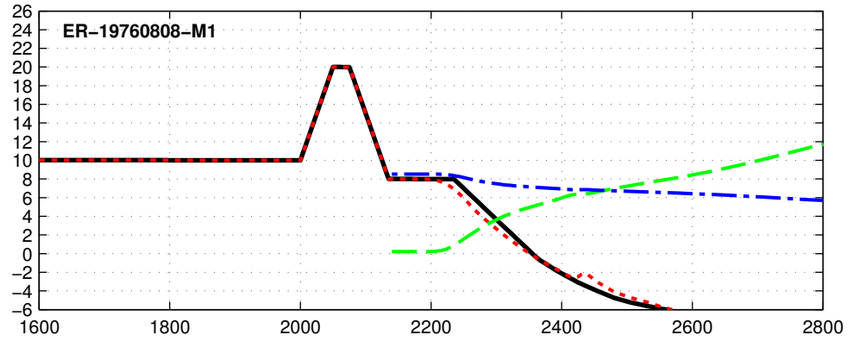
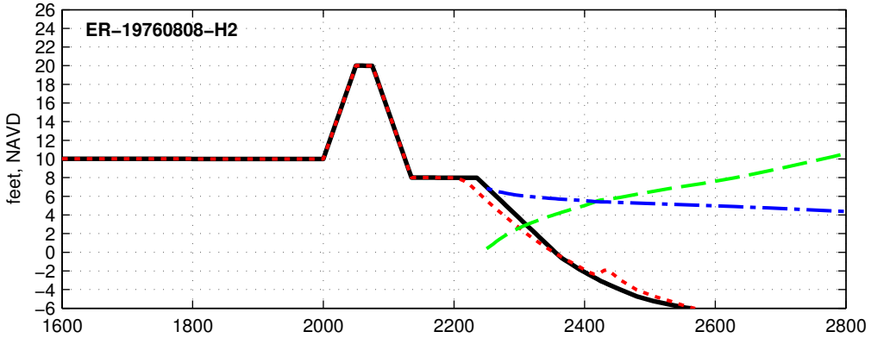
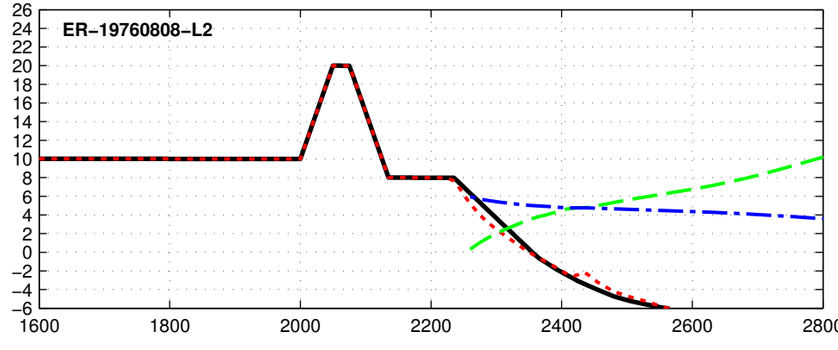
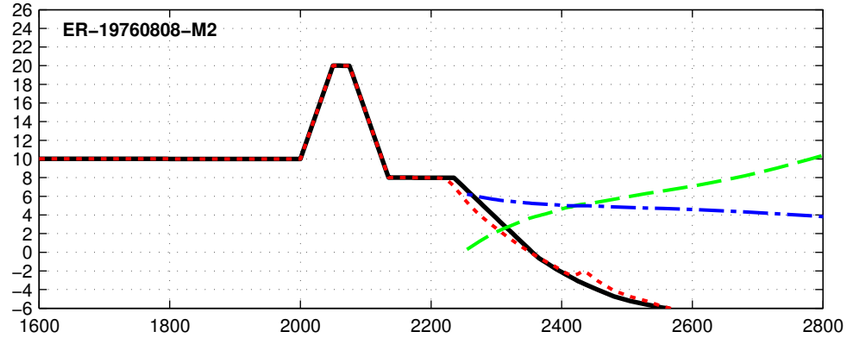


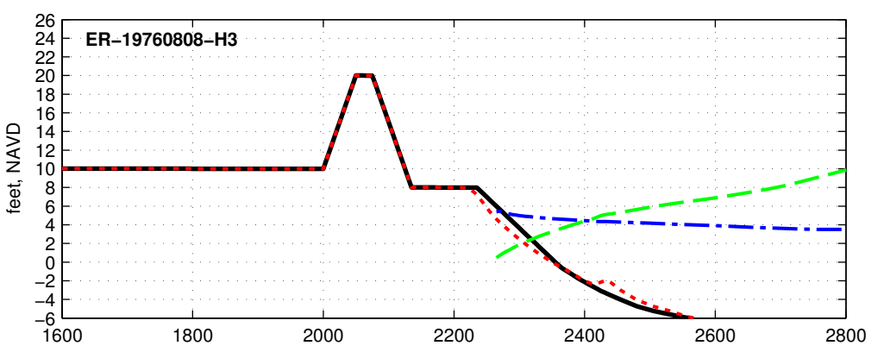


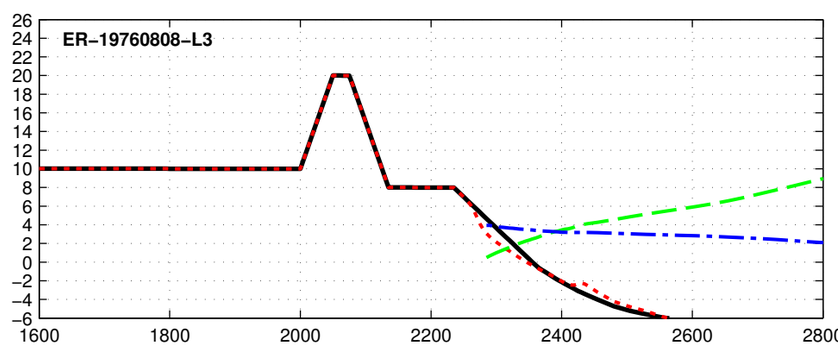


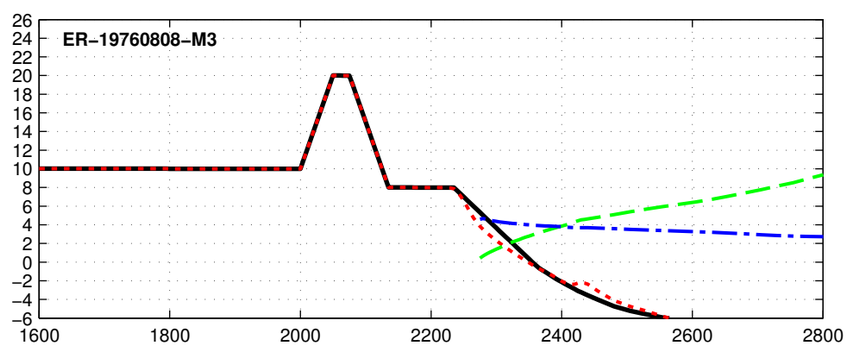


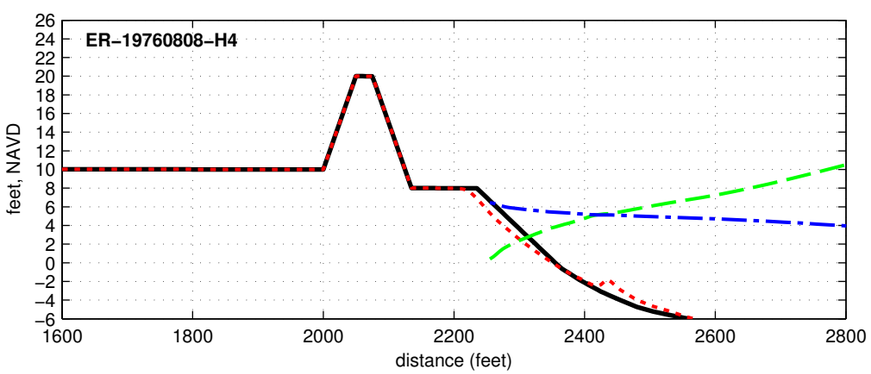


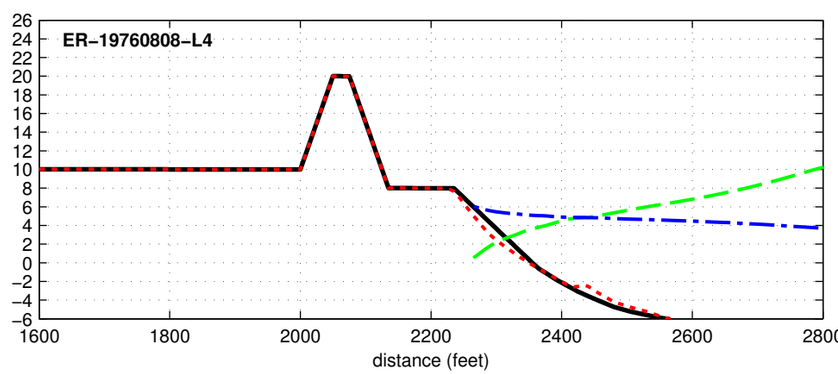


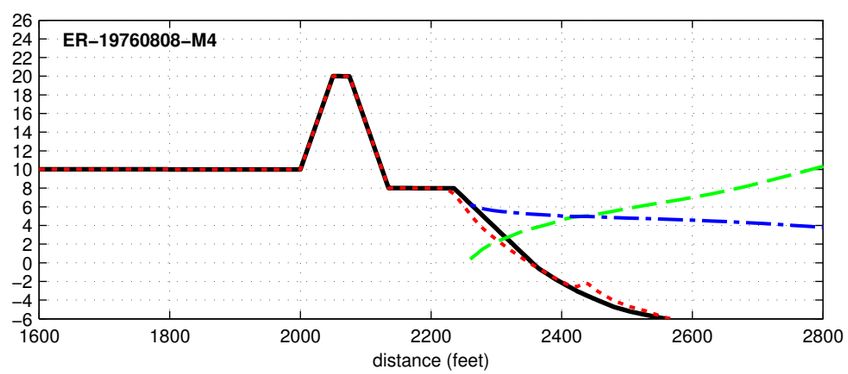






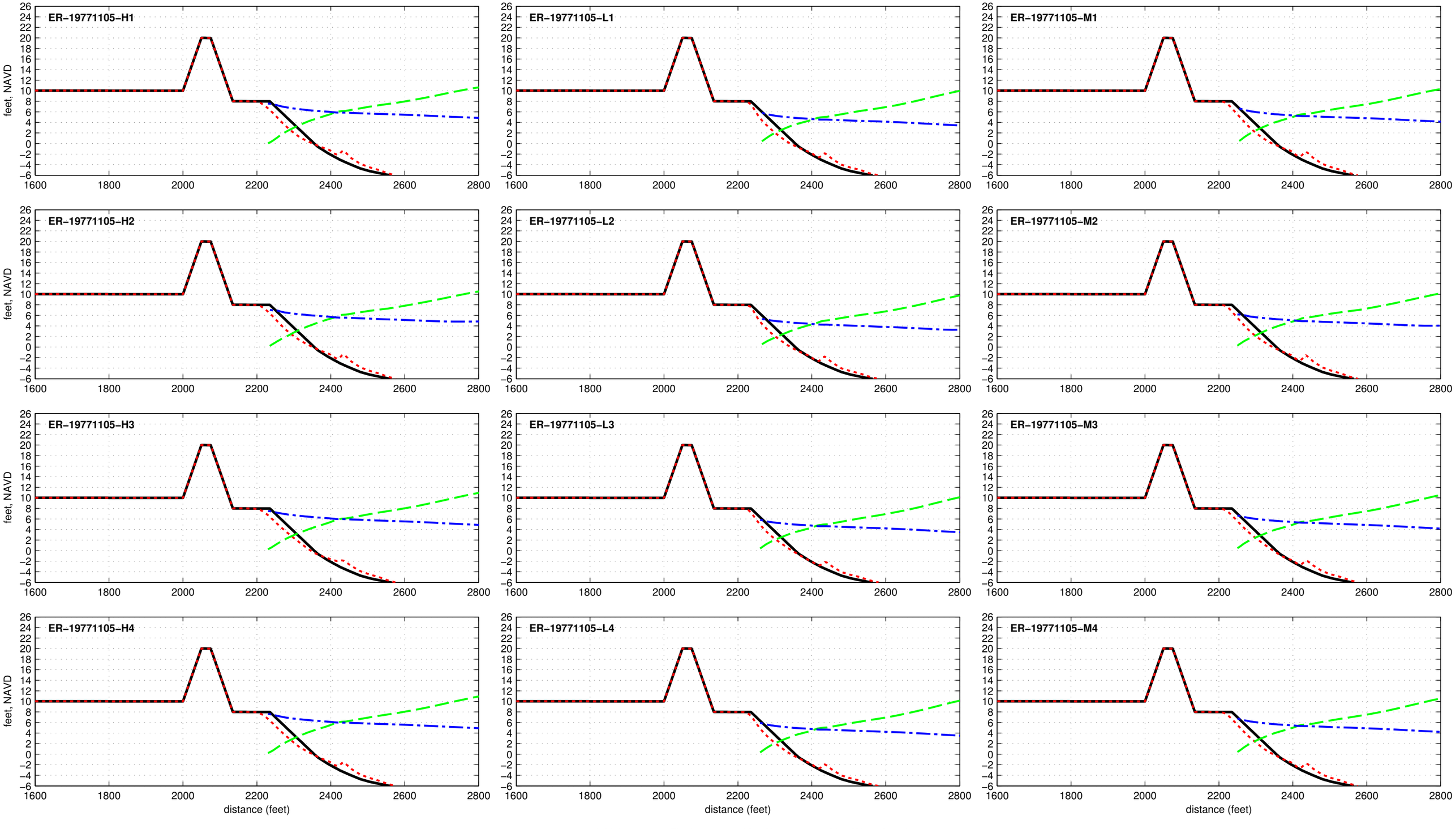




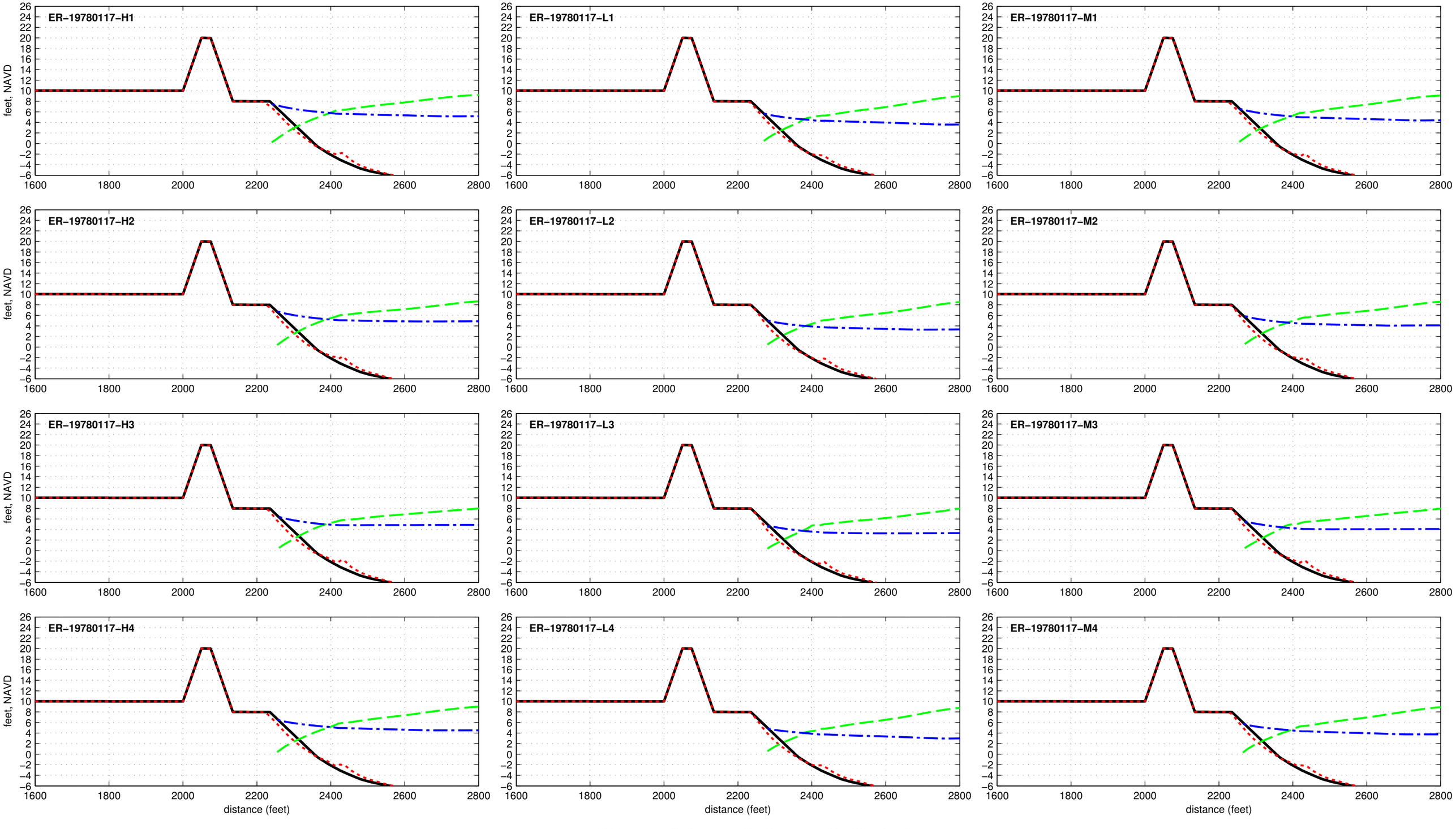


Initial Final Max Hs Max Water Level

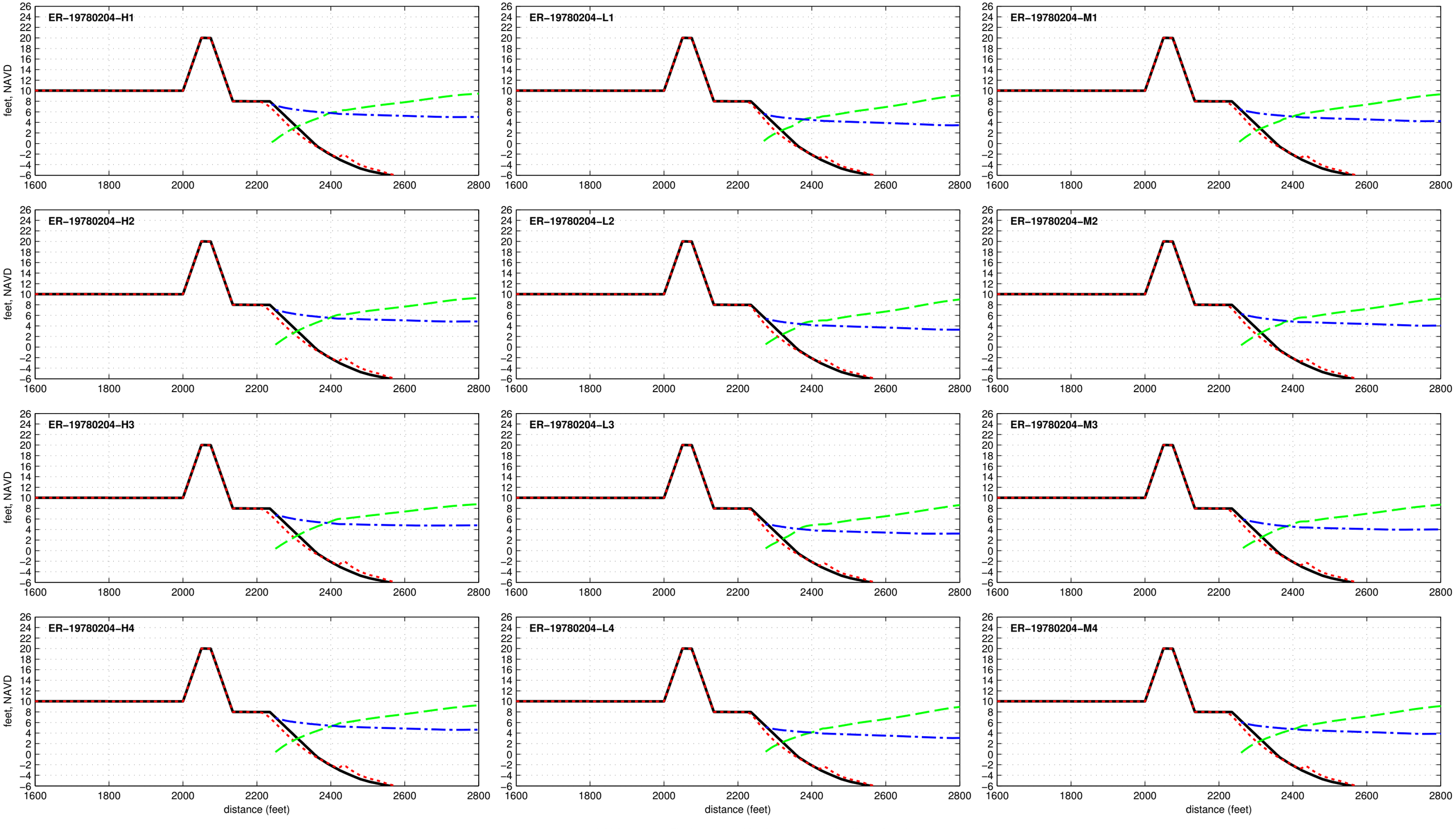
Dune R2T1-20.0-025-100 - ER-19771105



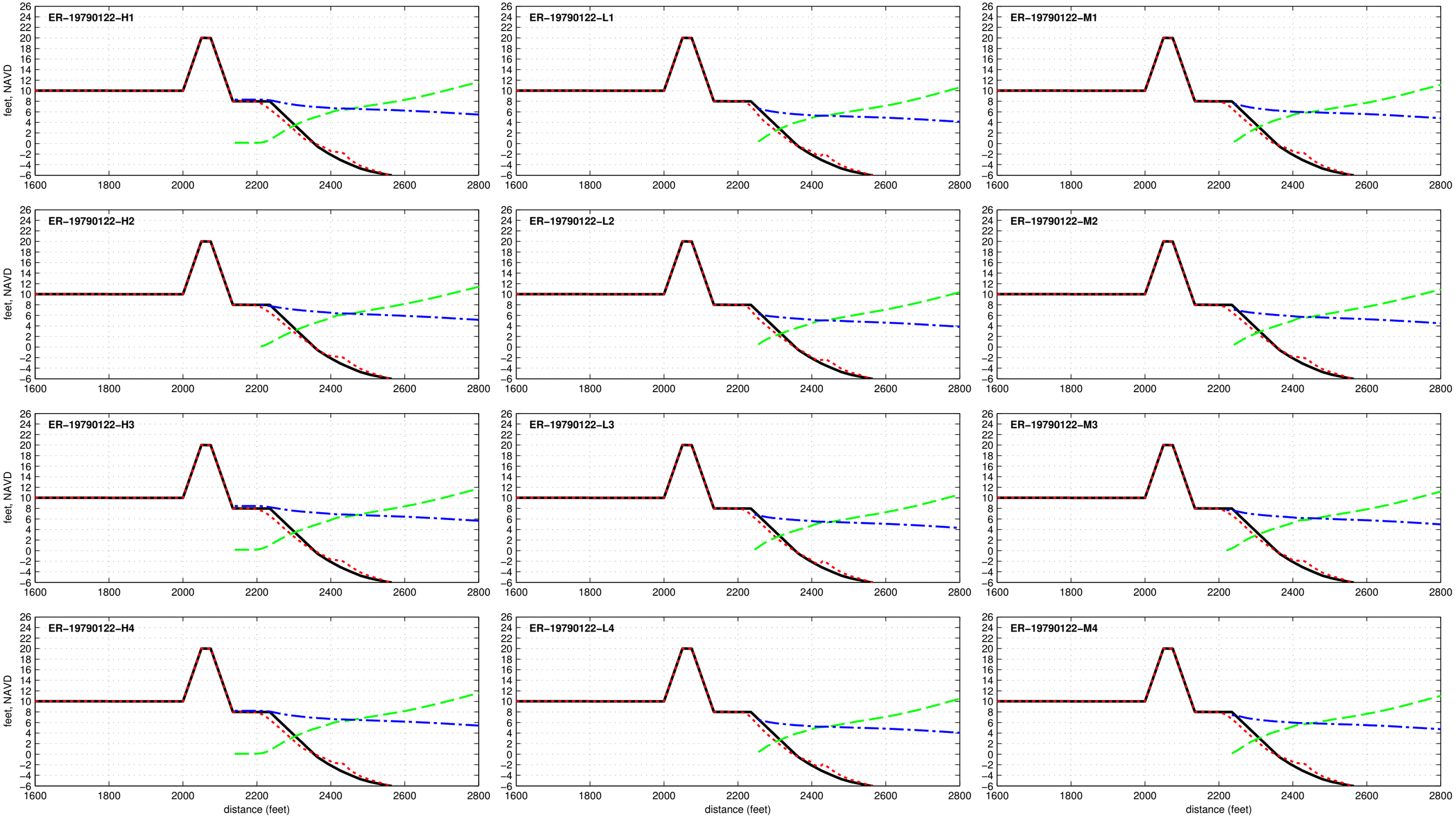
Dune R2T1–20.0–025–100 – ER–19780117



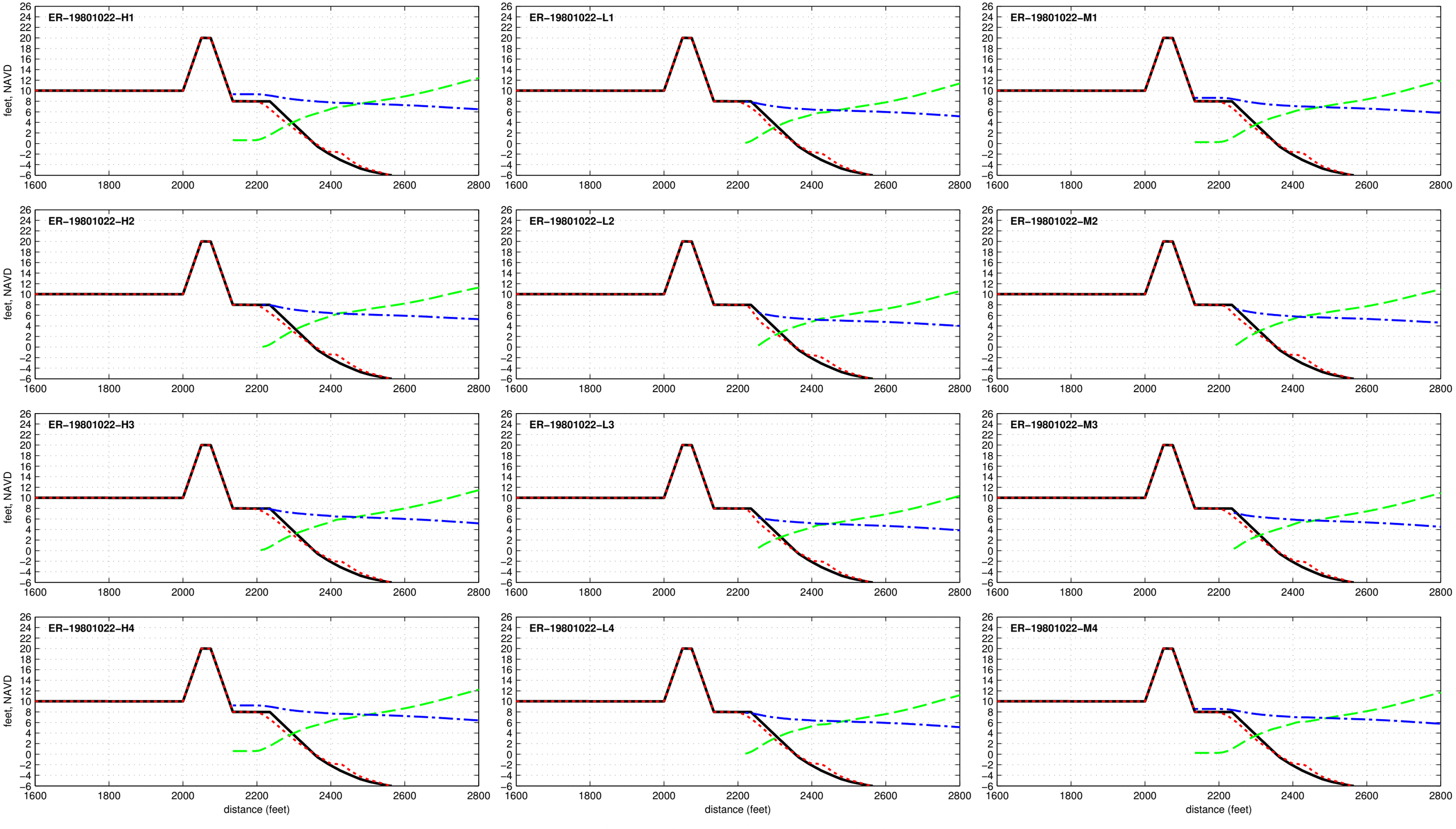
Dune R2T1–20.0–025–100 – ER–19780204



Dune R2T1-20.0-025-100 - ER-19790122



Dune R2T1-20.0-025-100 - ER-19801022



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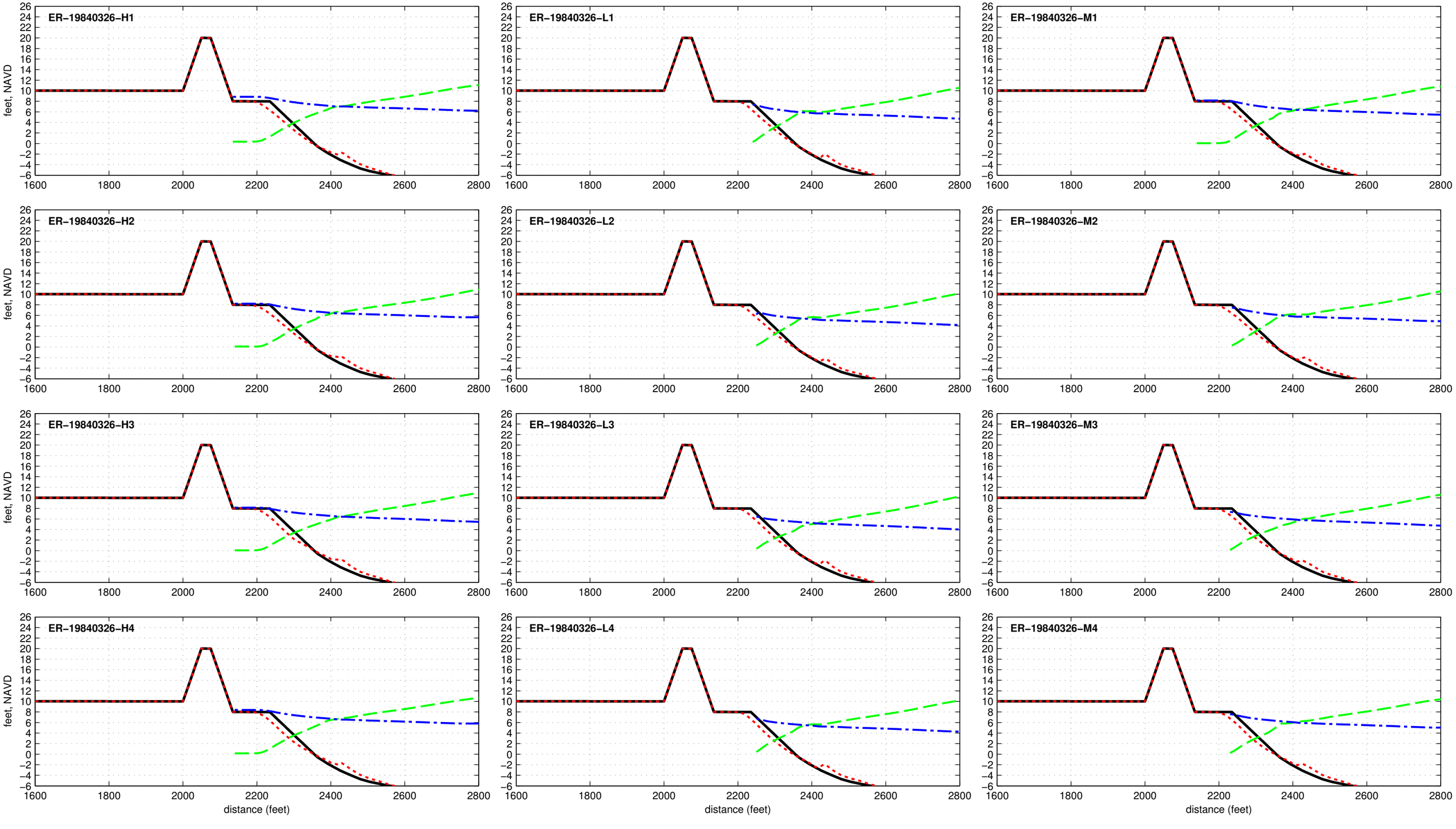

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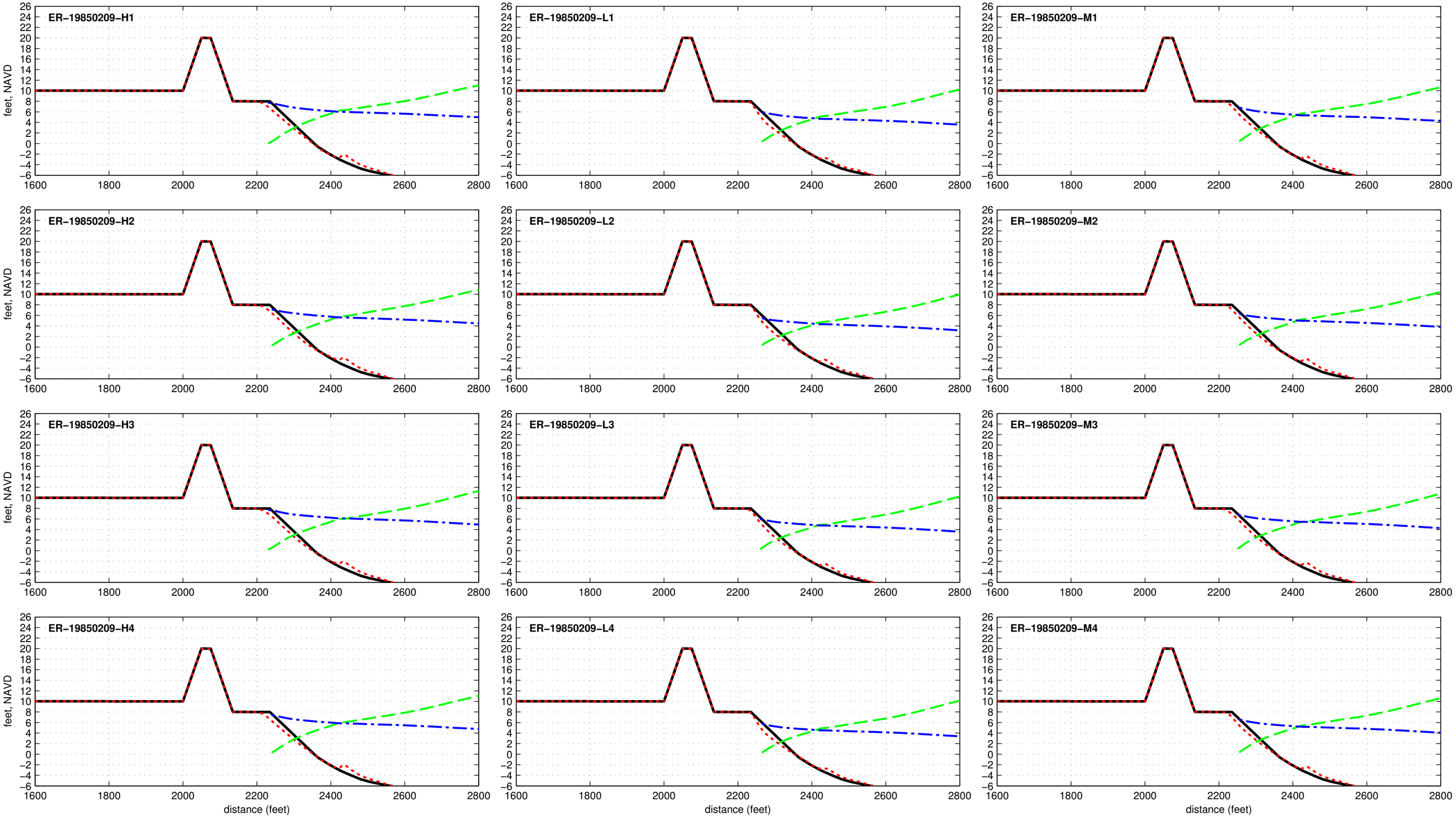

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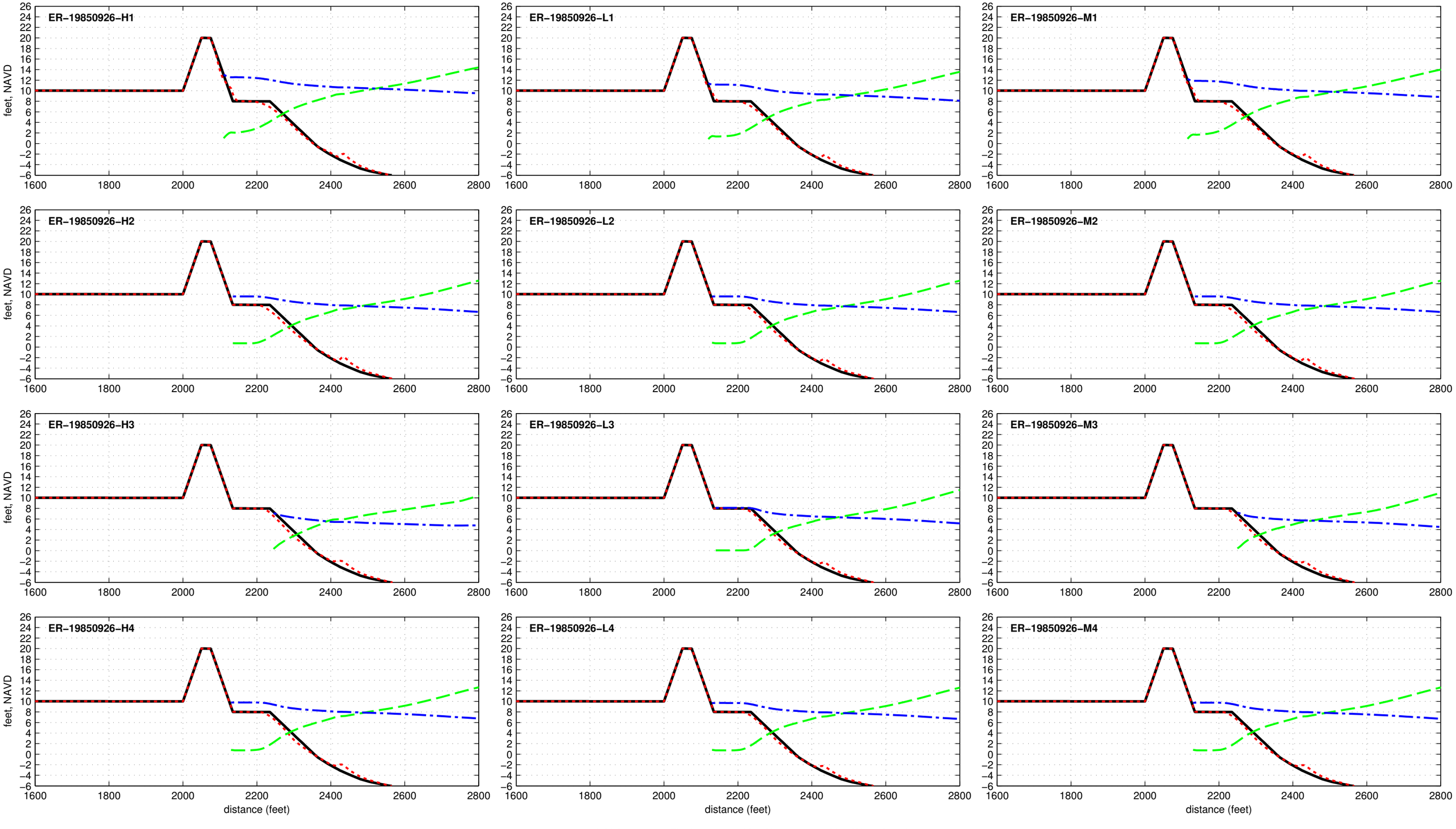

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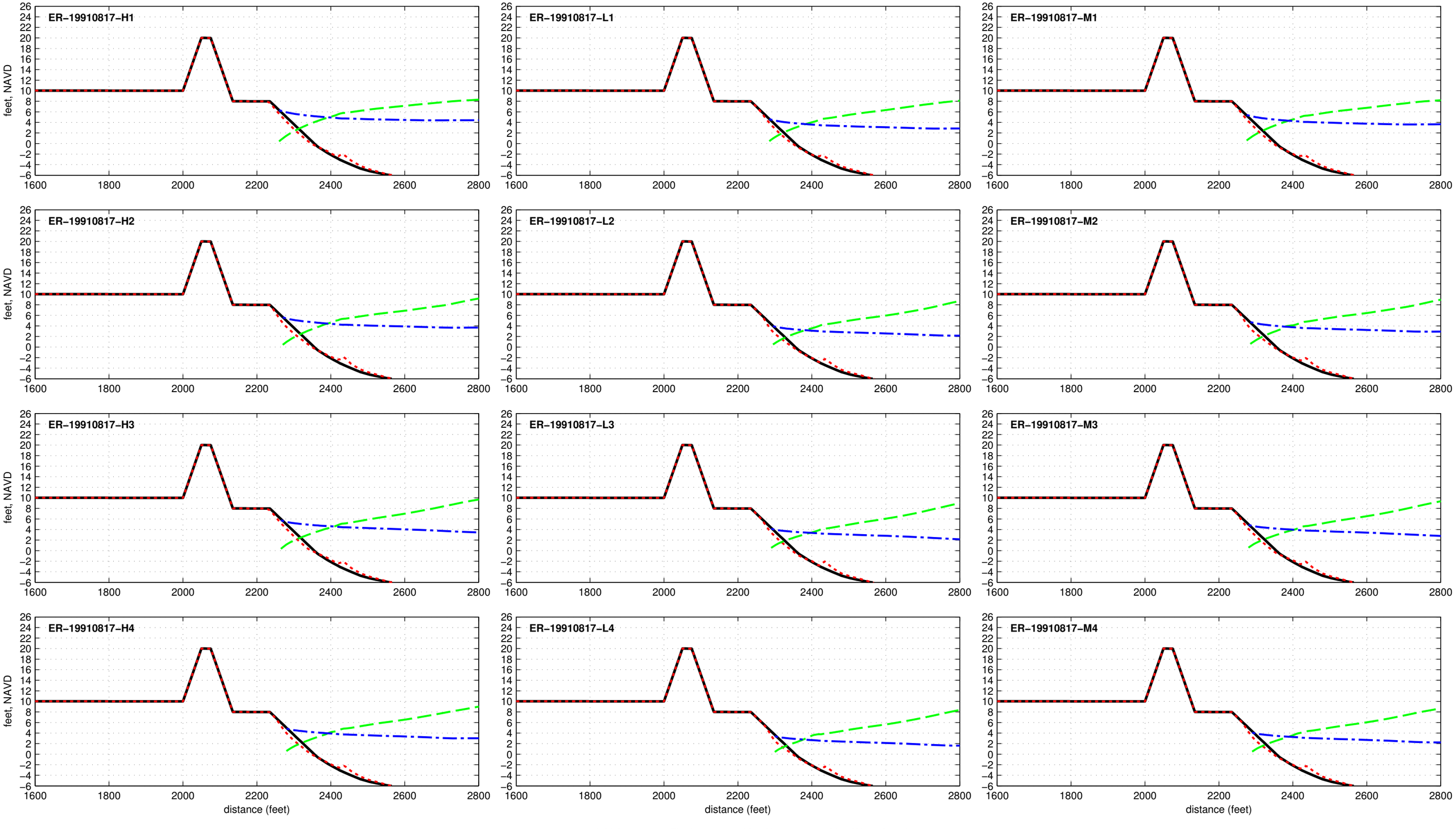

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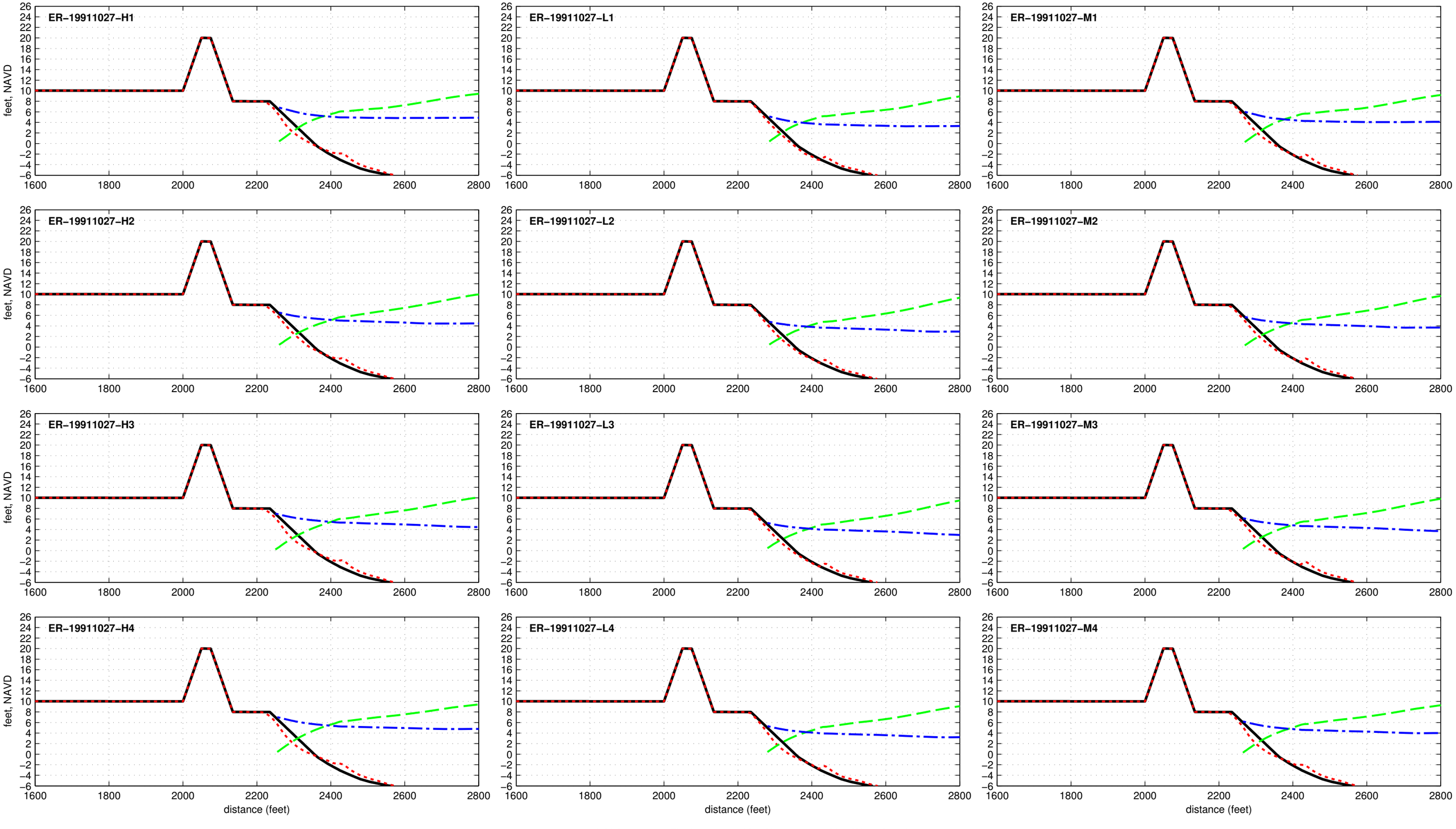

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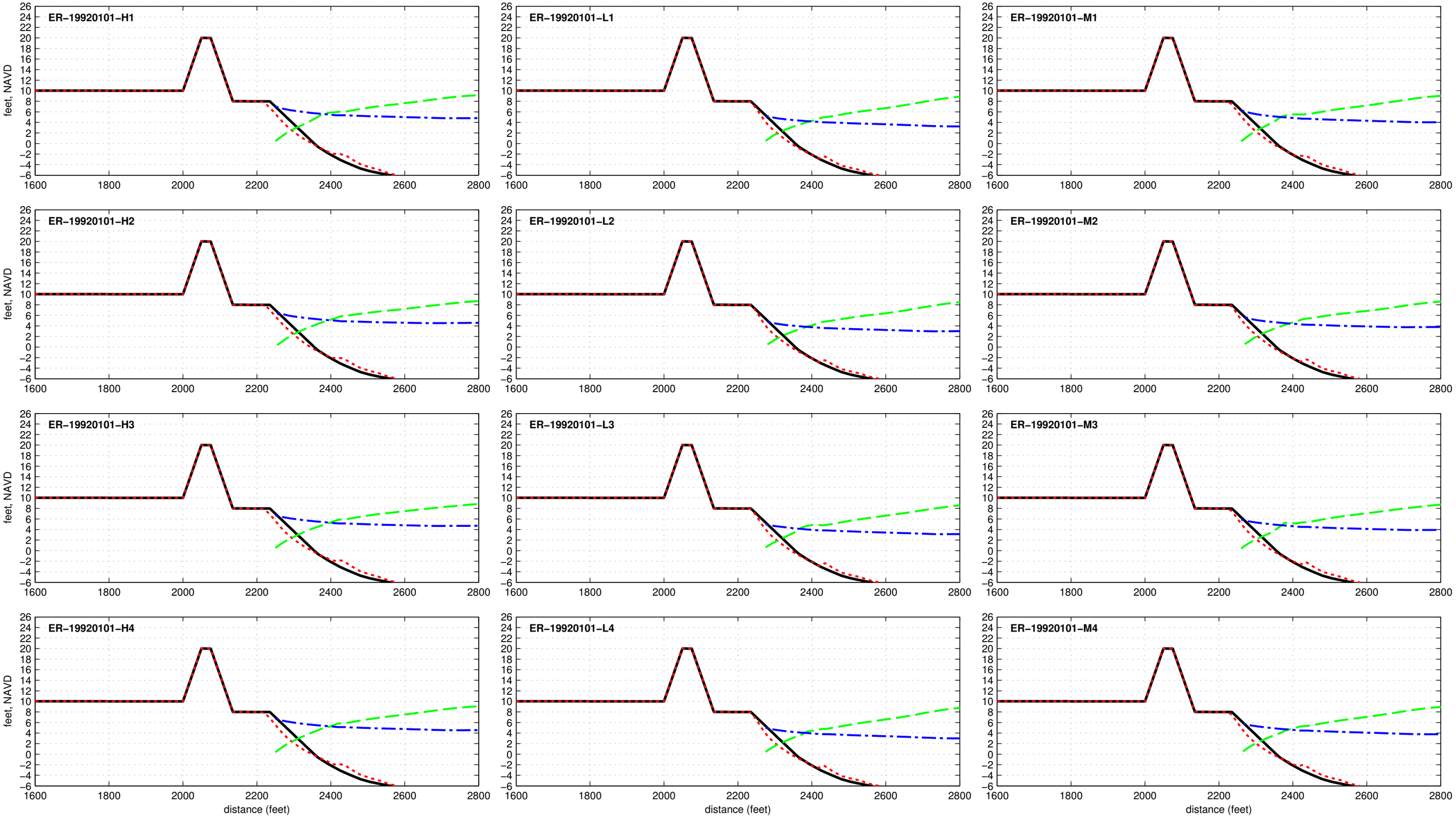

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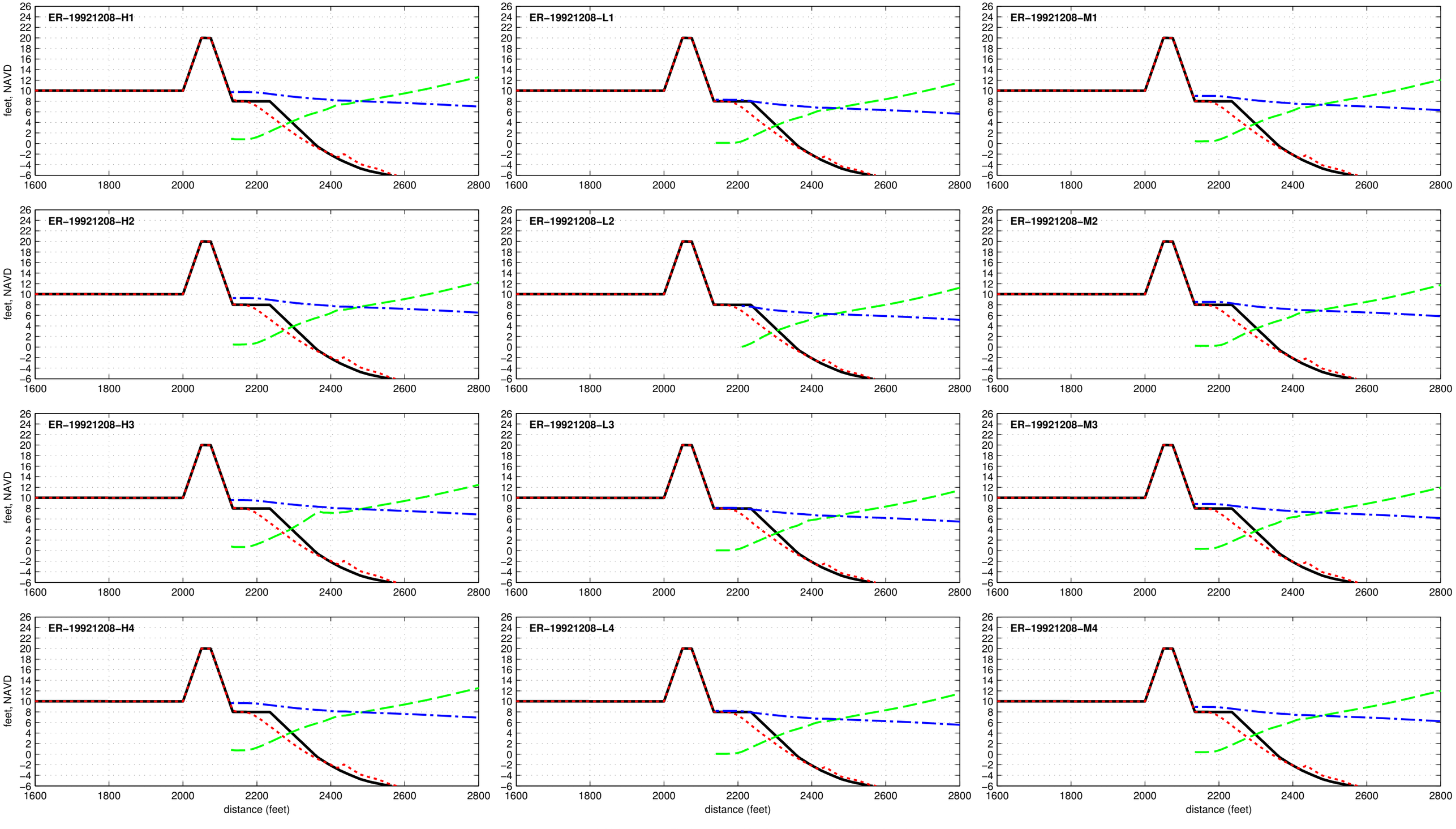

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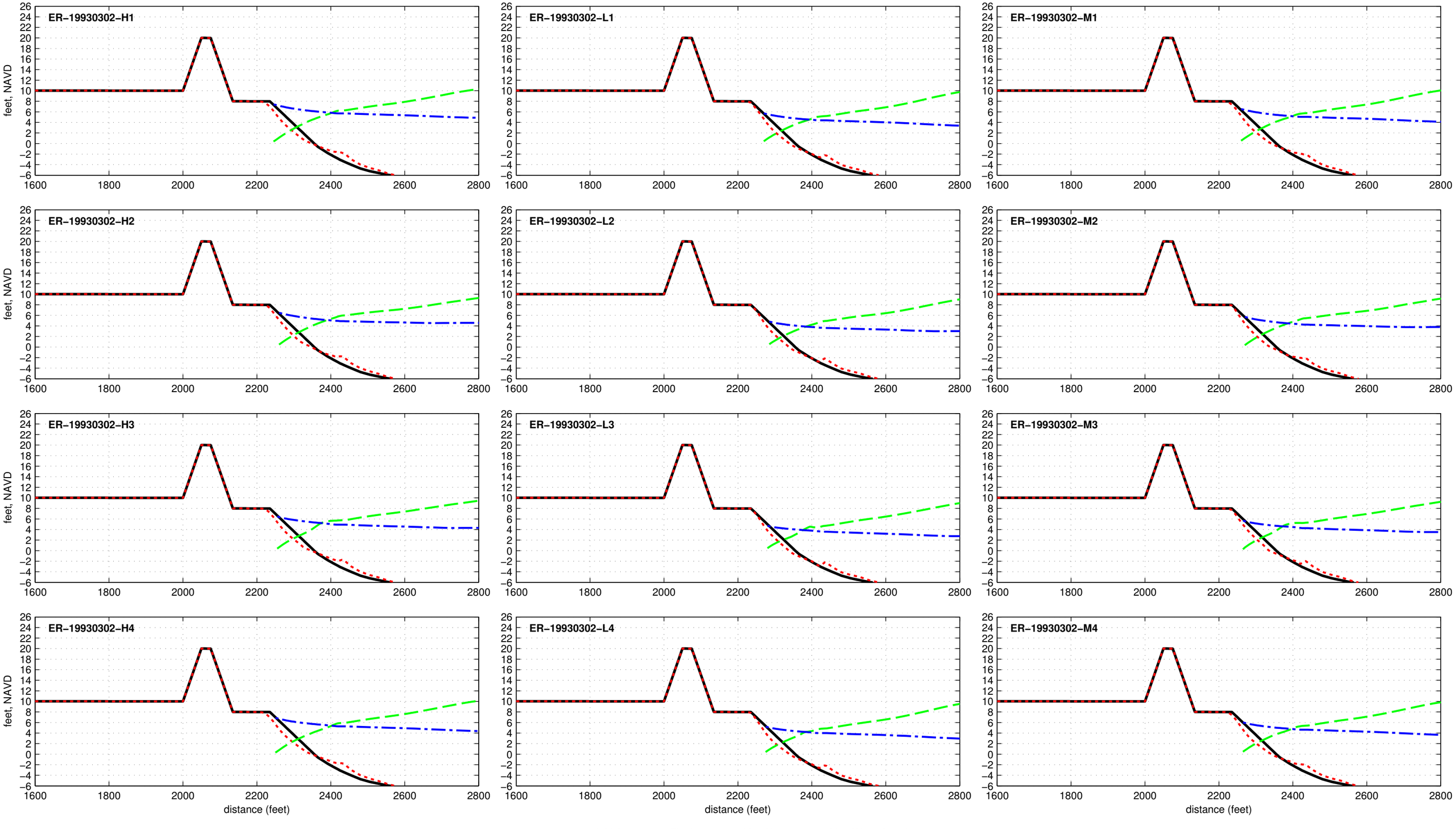

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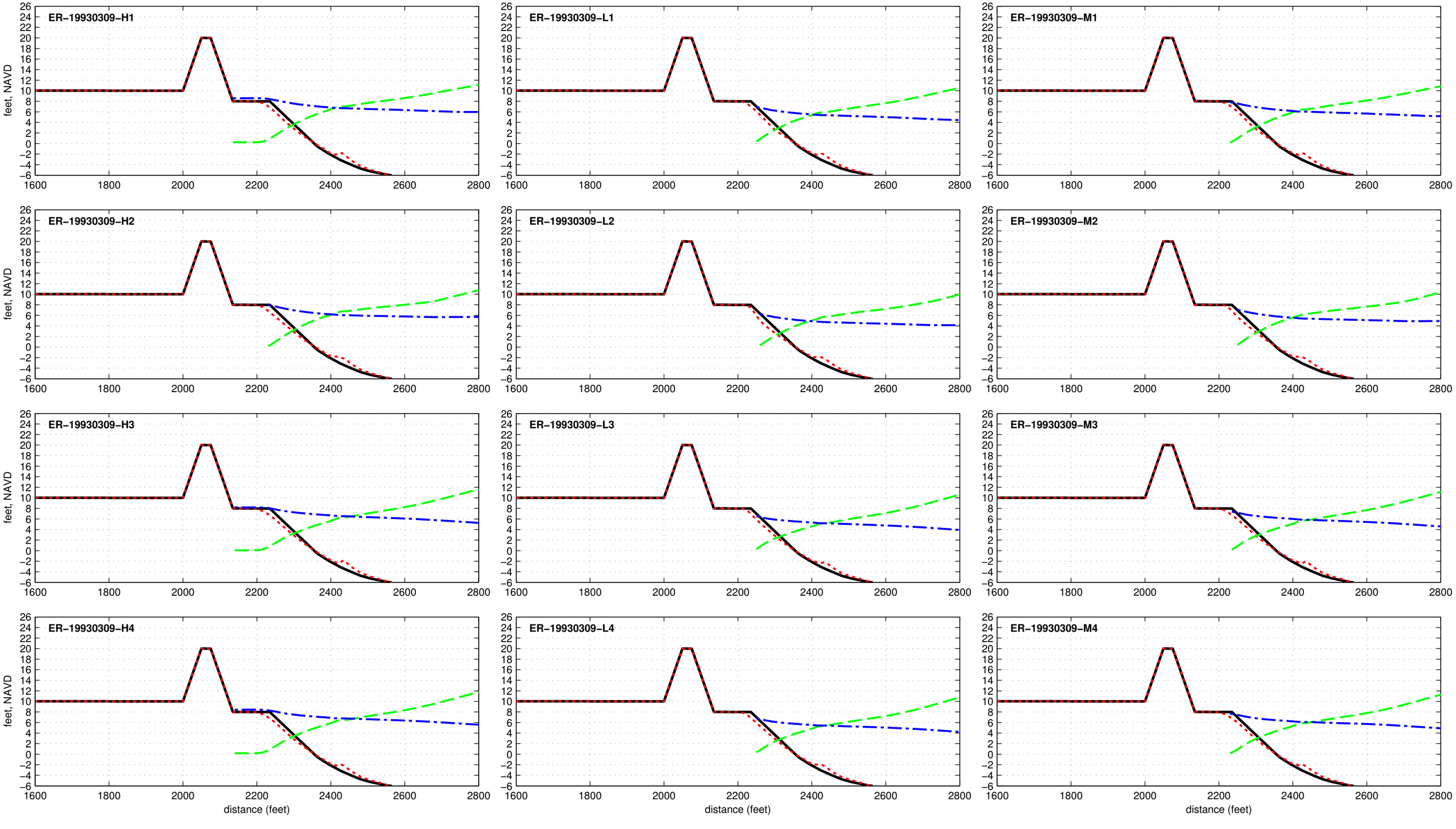

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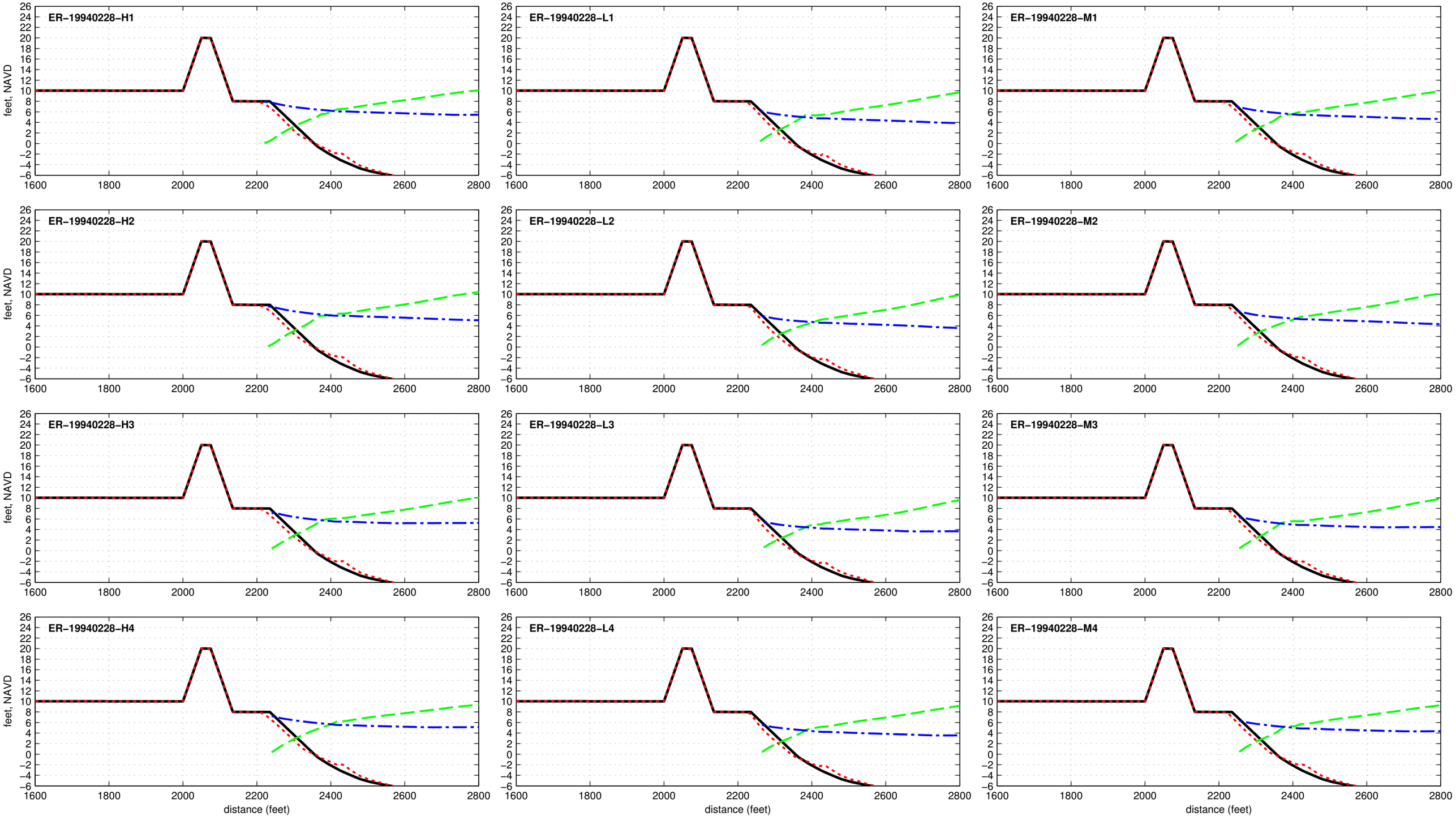

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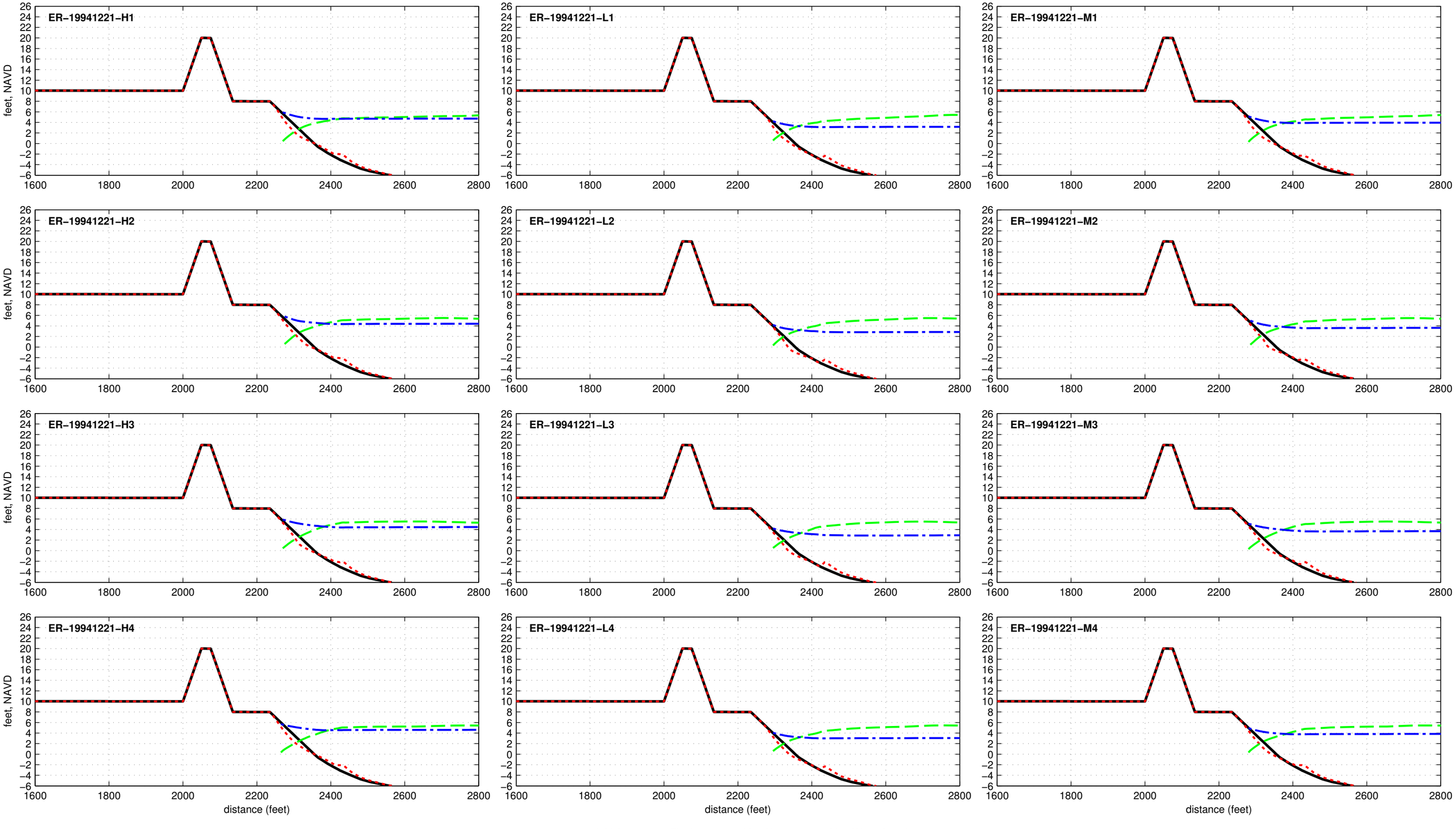

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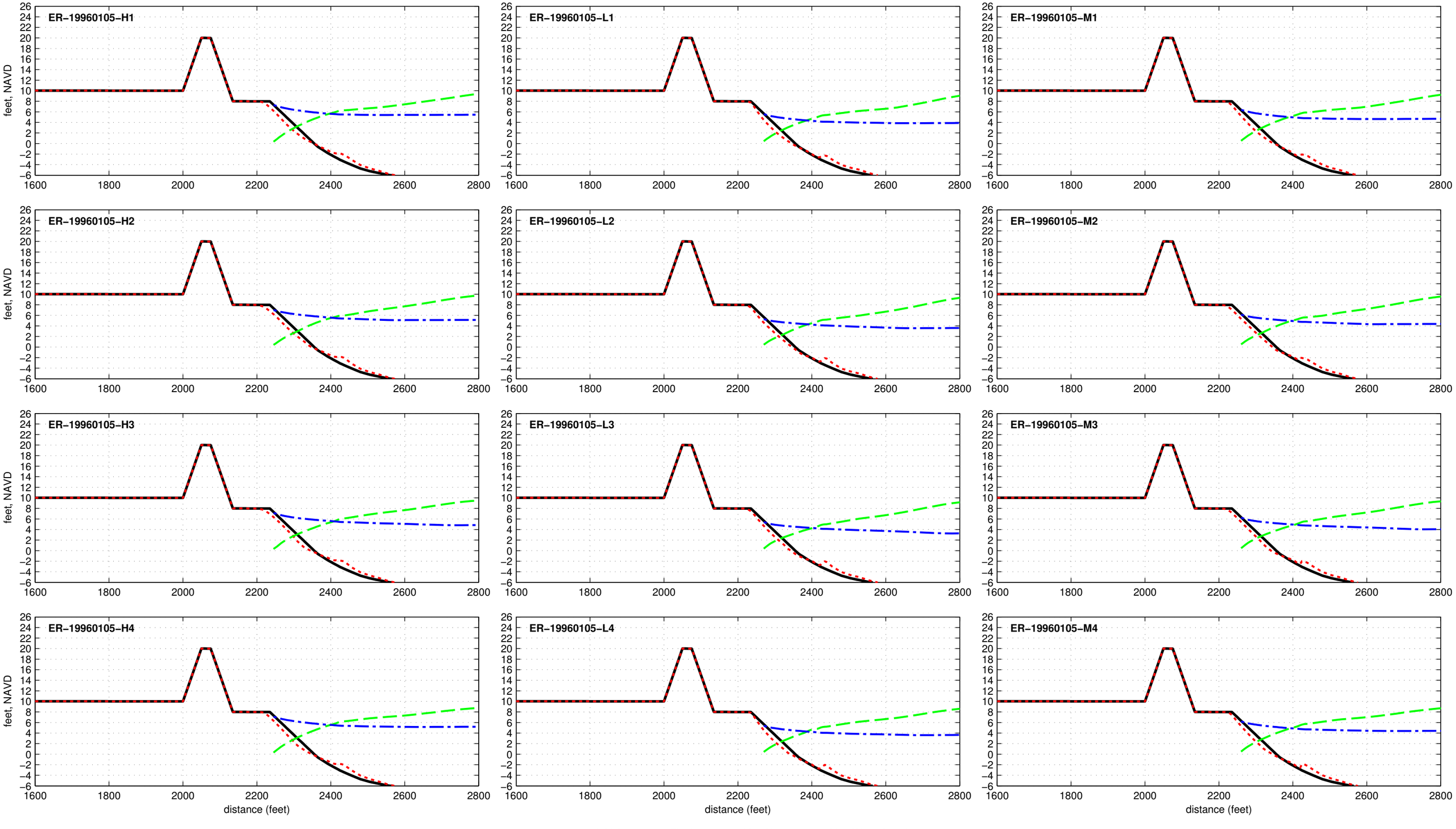

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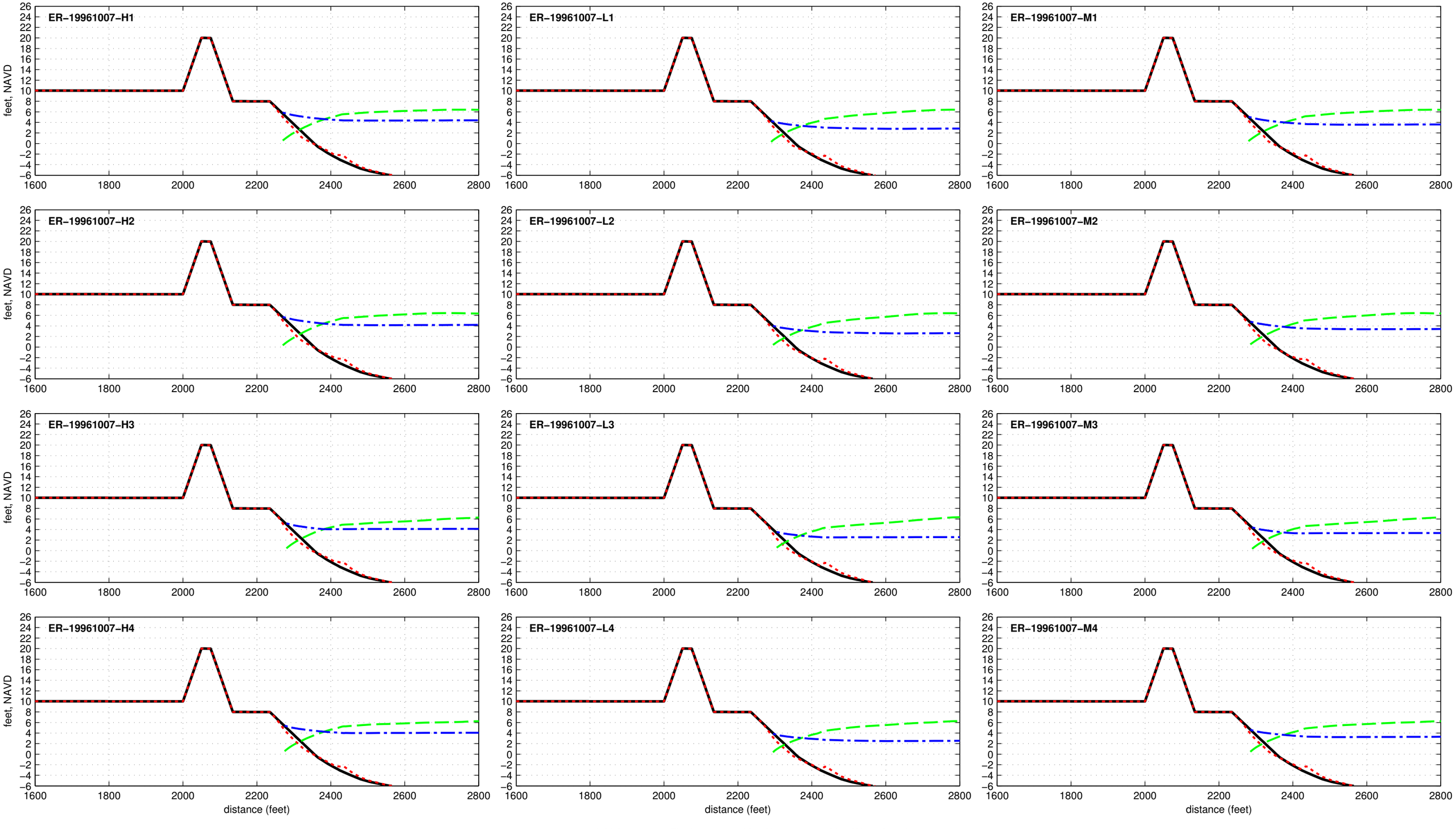

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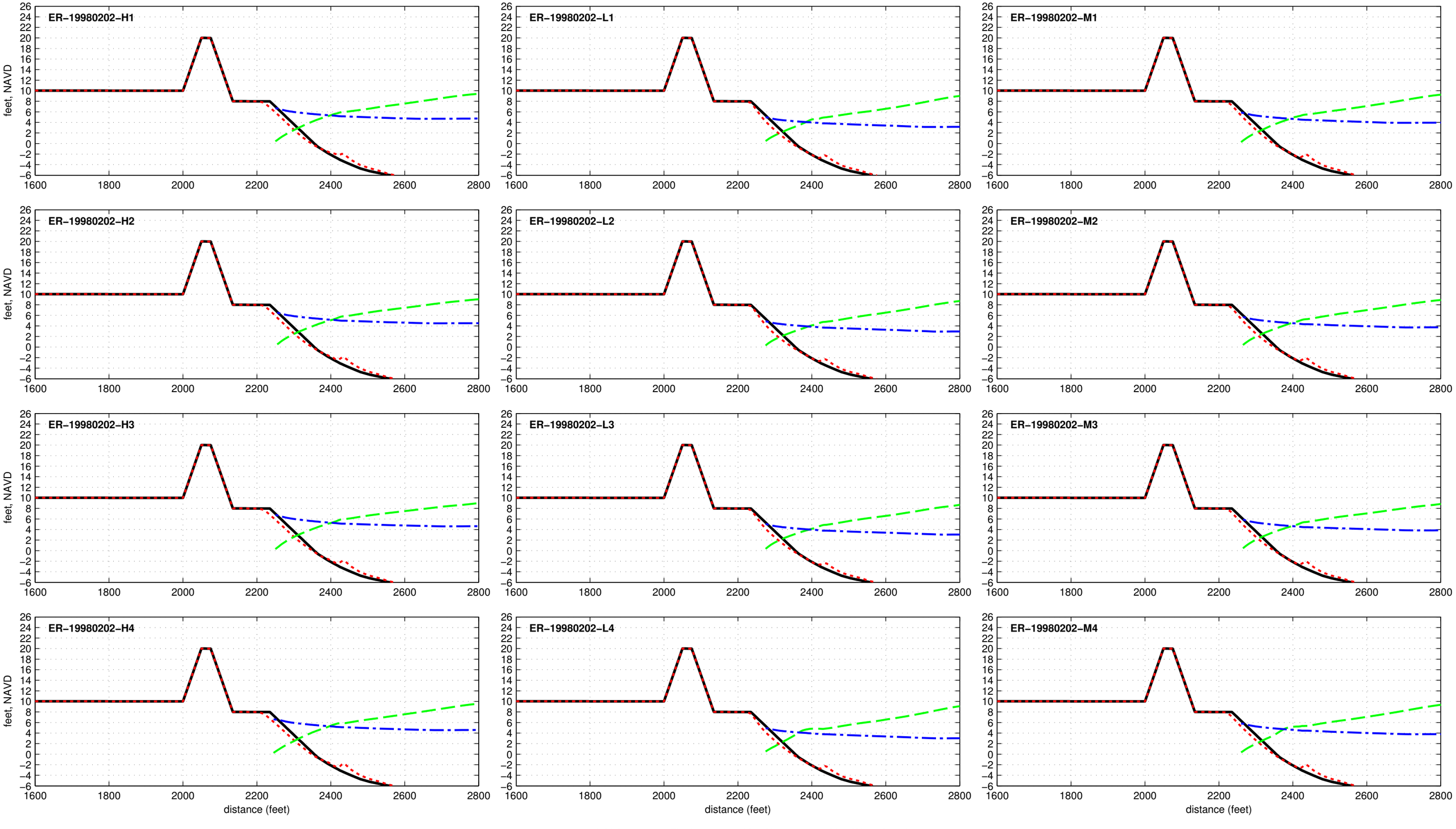

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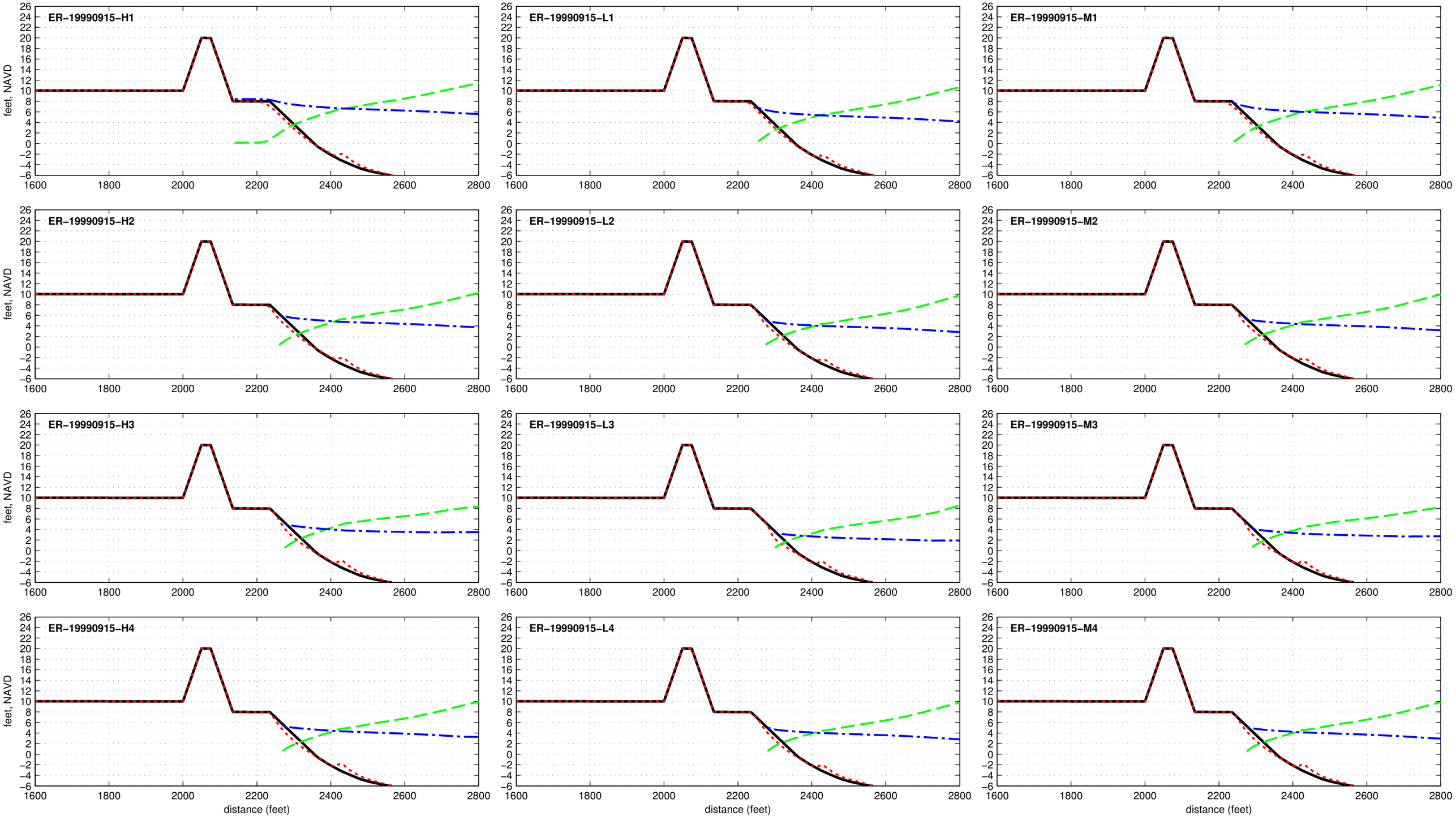

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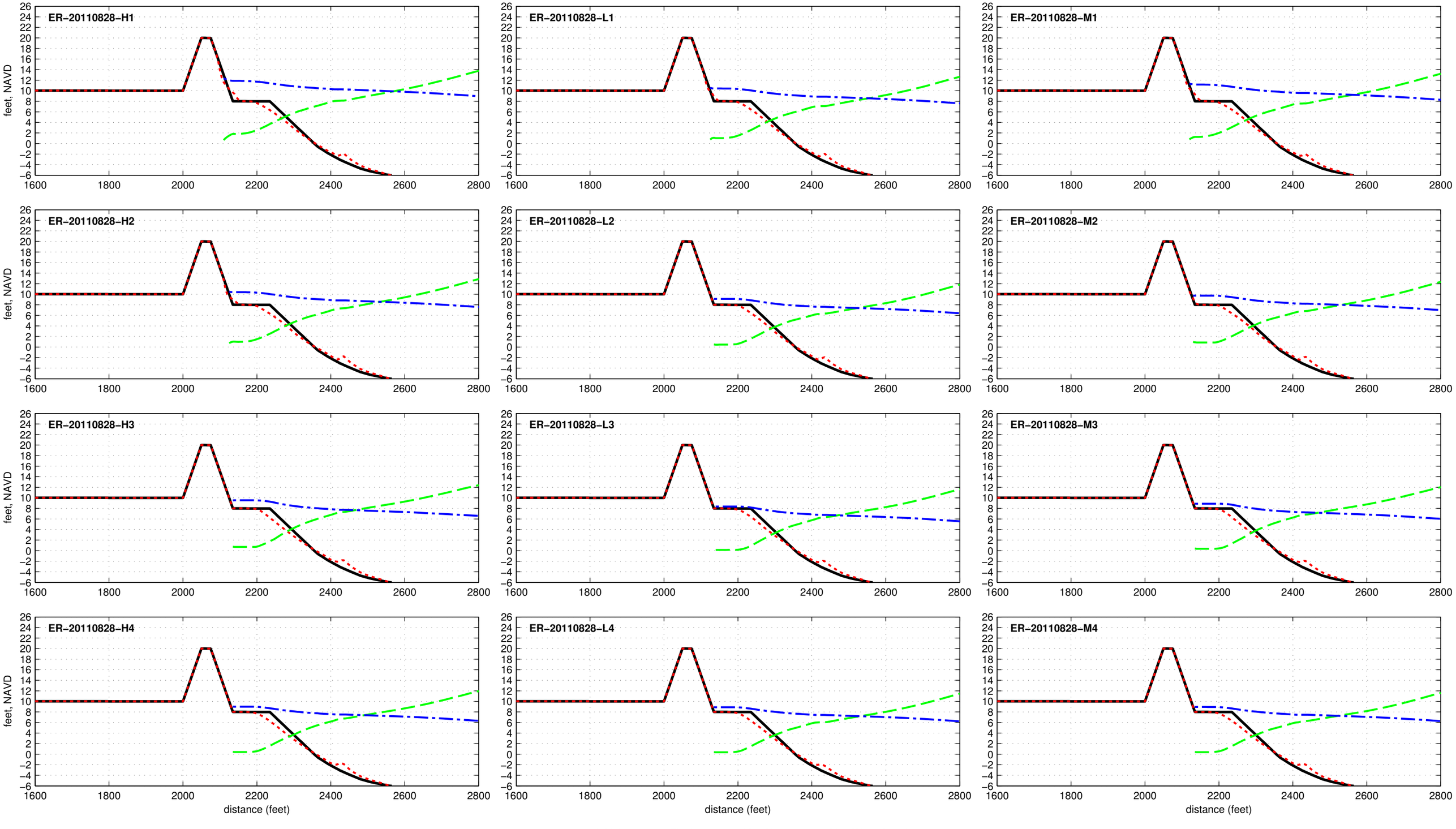

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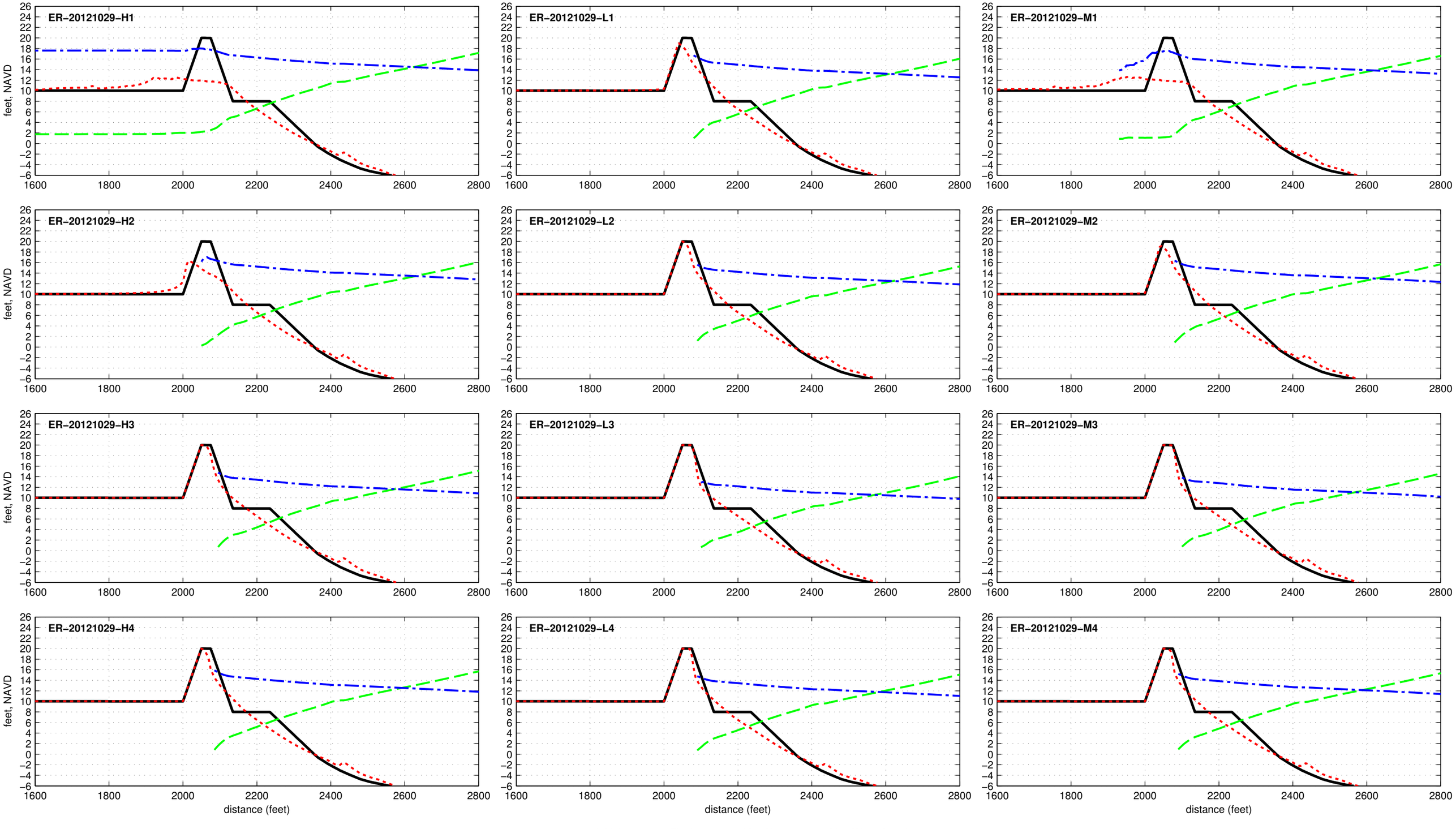
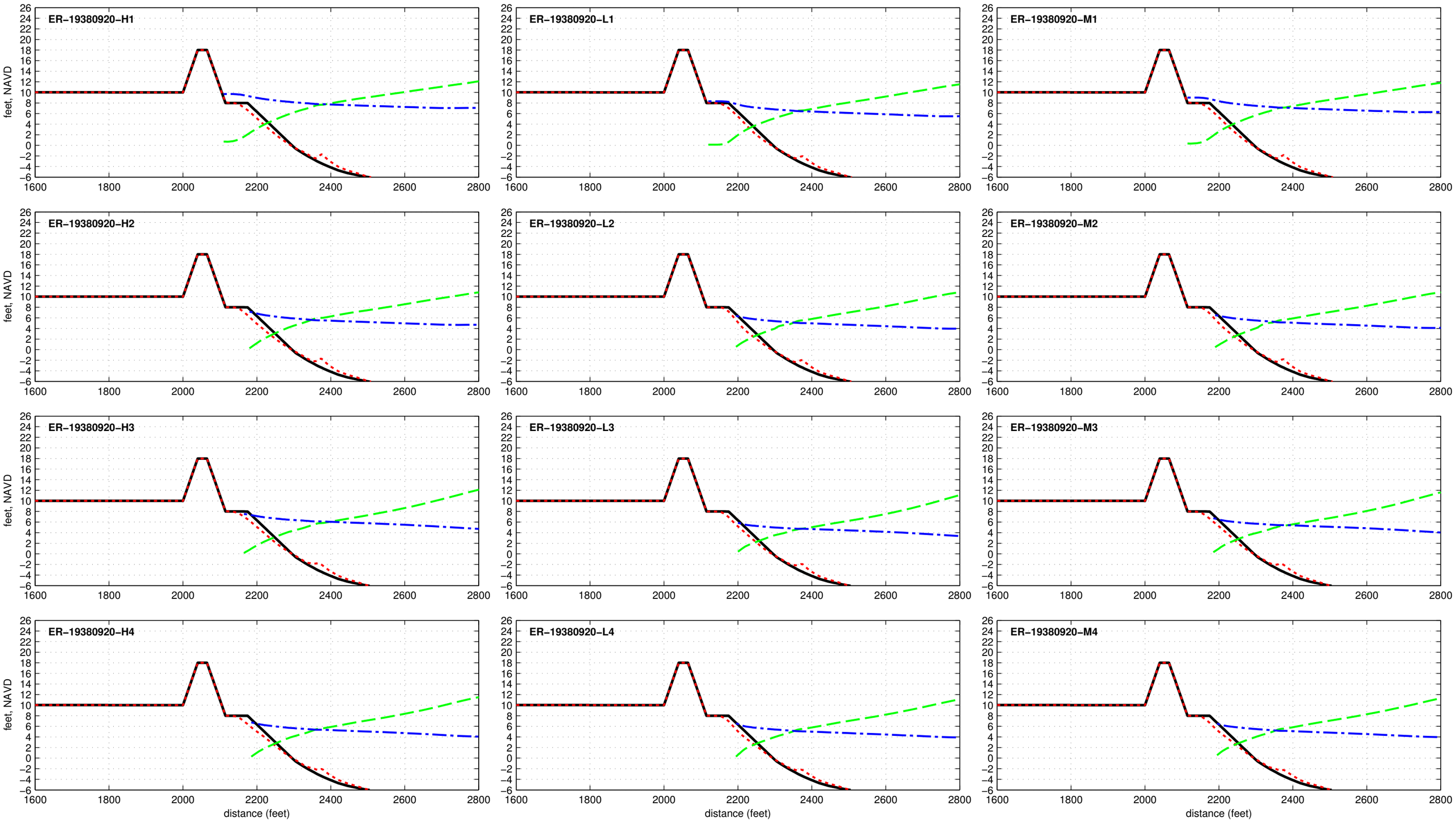


R2T1_18.0_025_060 – BURIED SEAWALL

Buried Seawall R2T1-18.0-025-060 - ER-19380920



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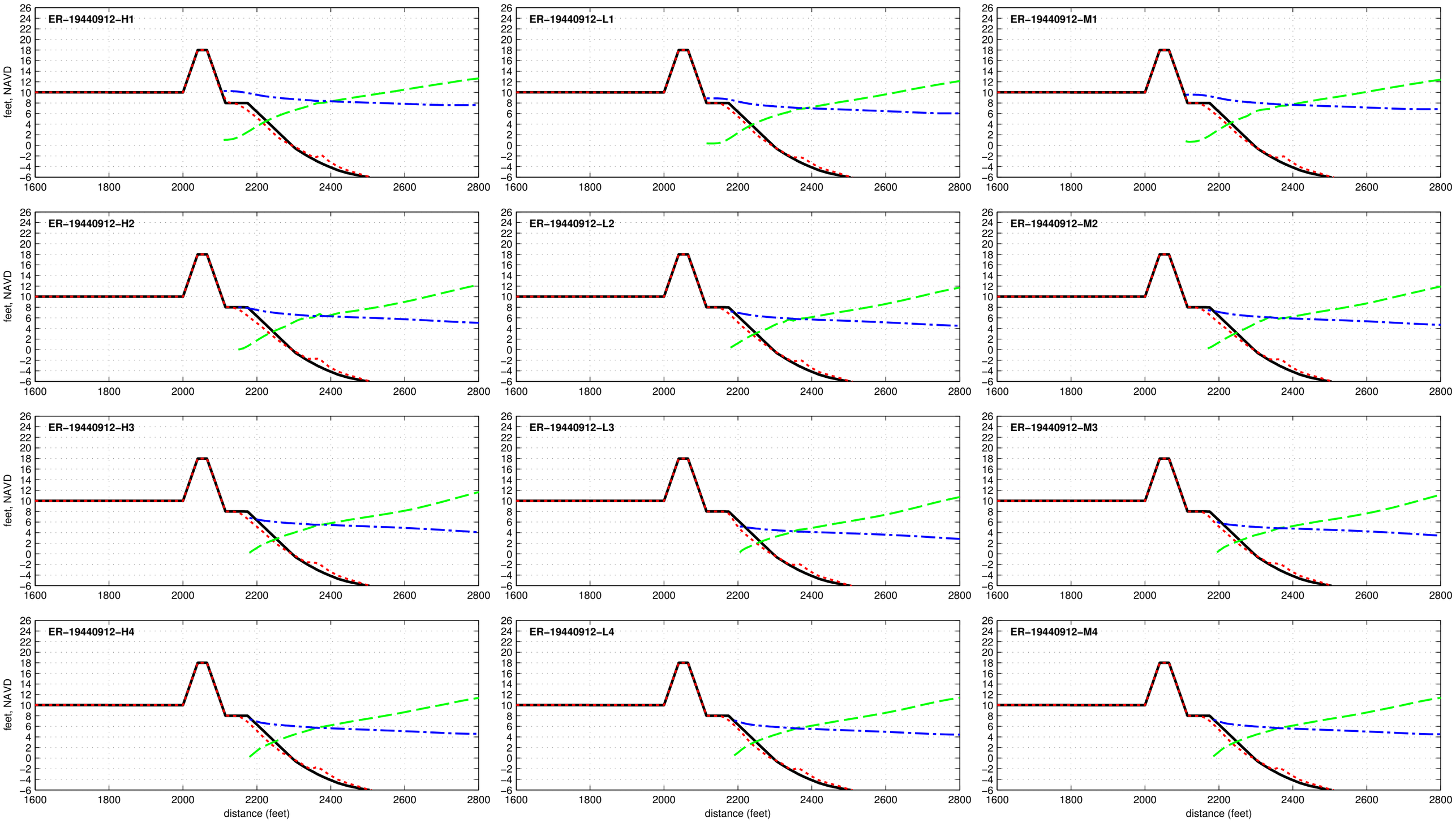

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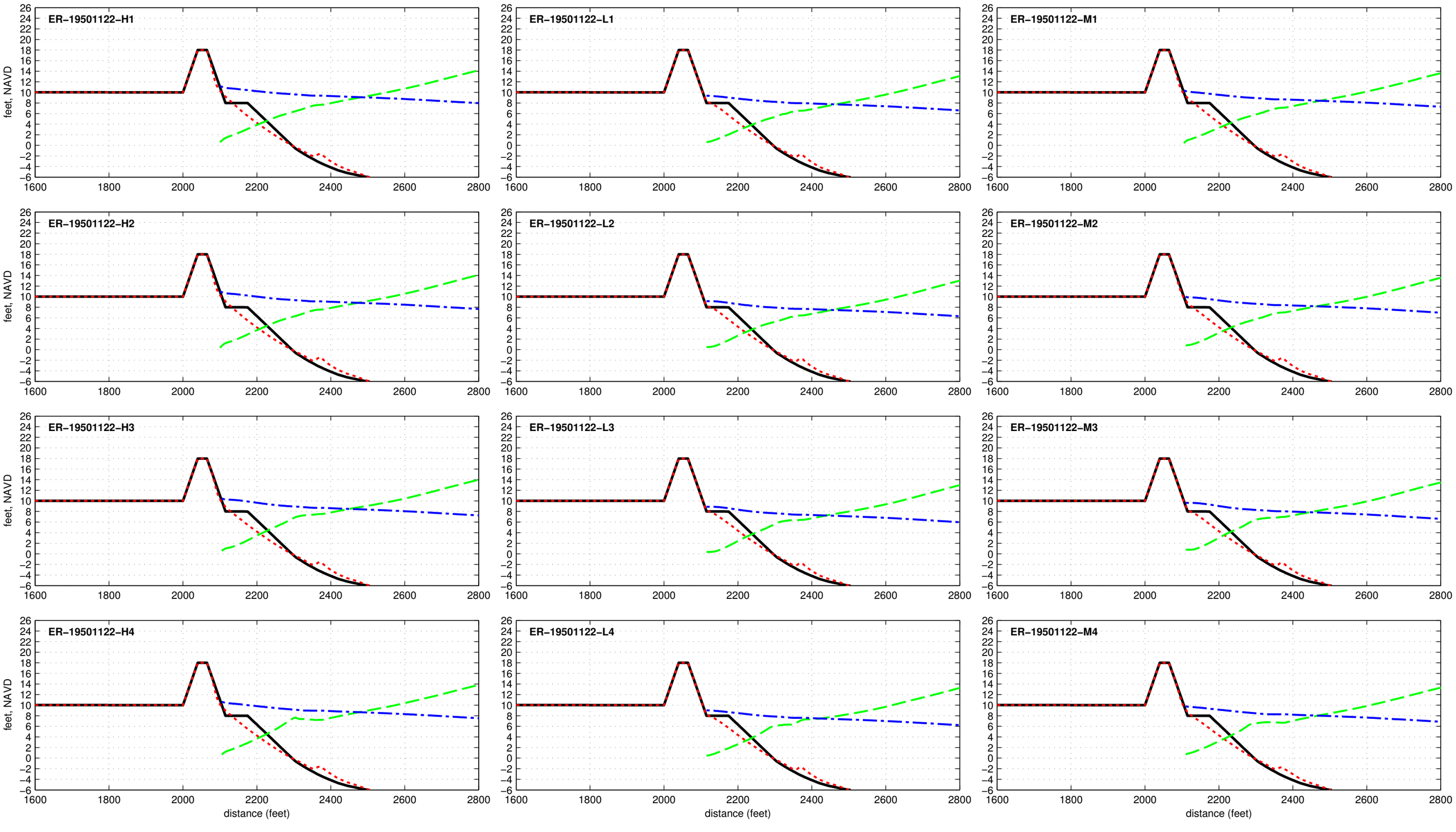

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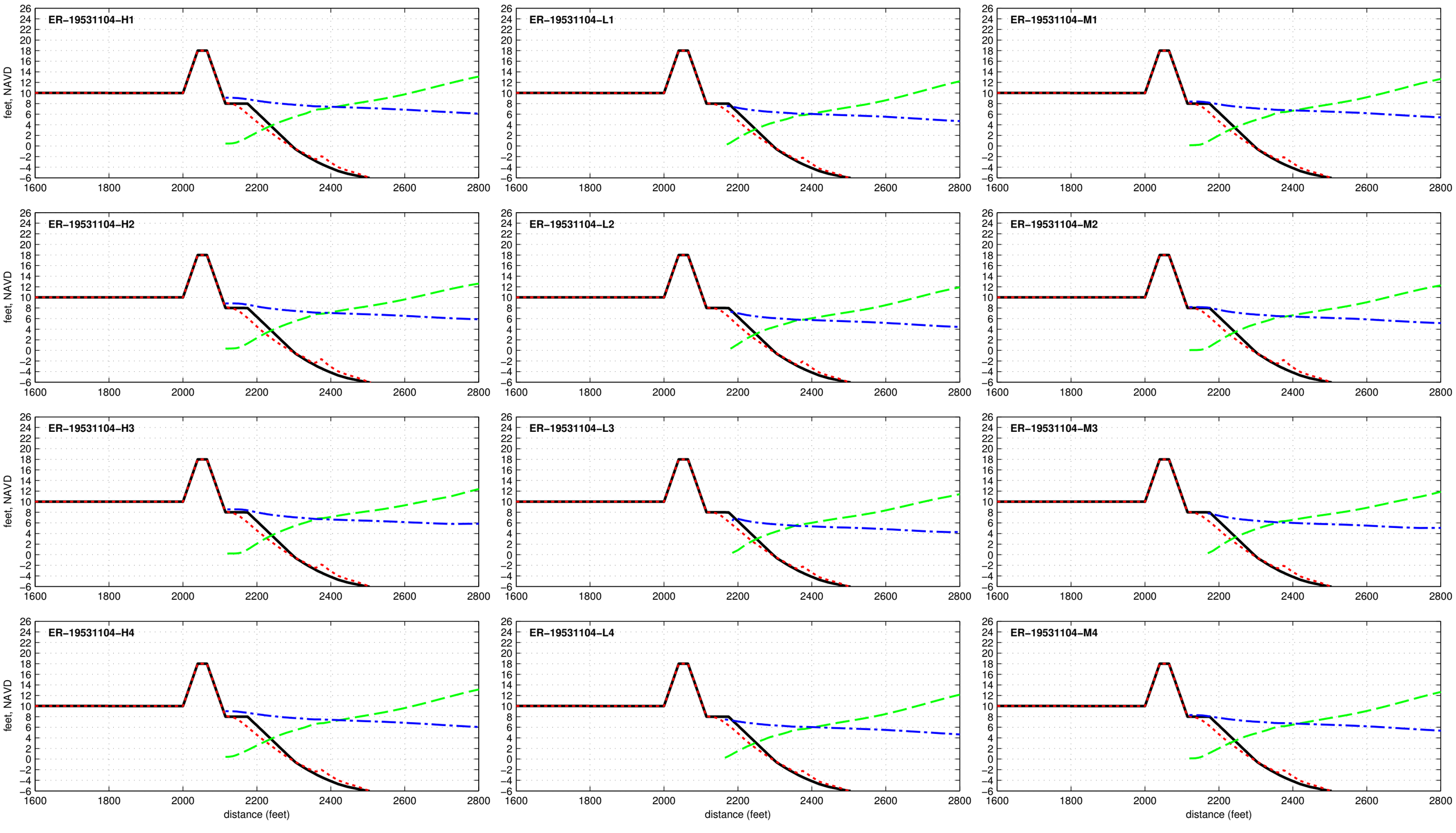

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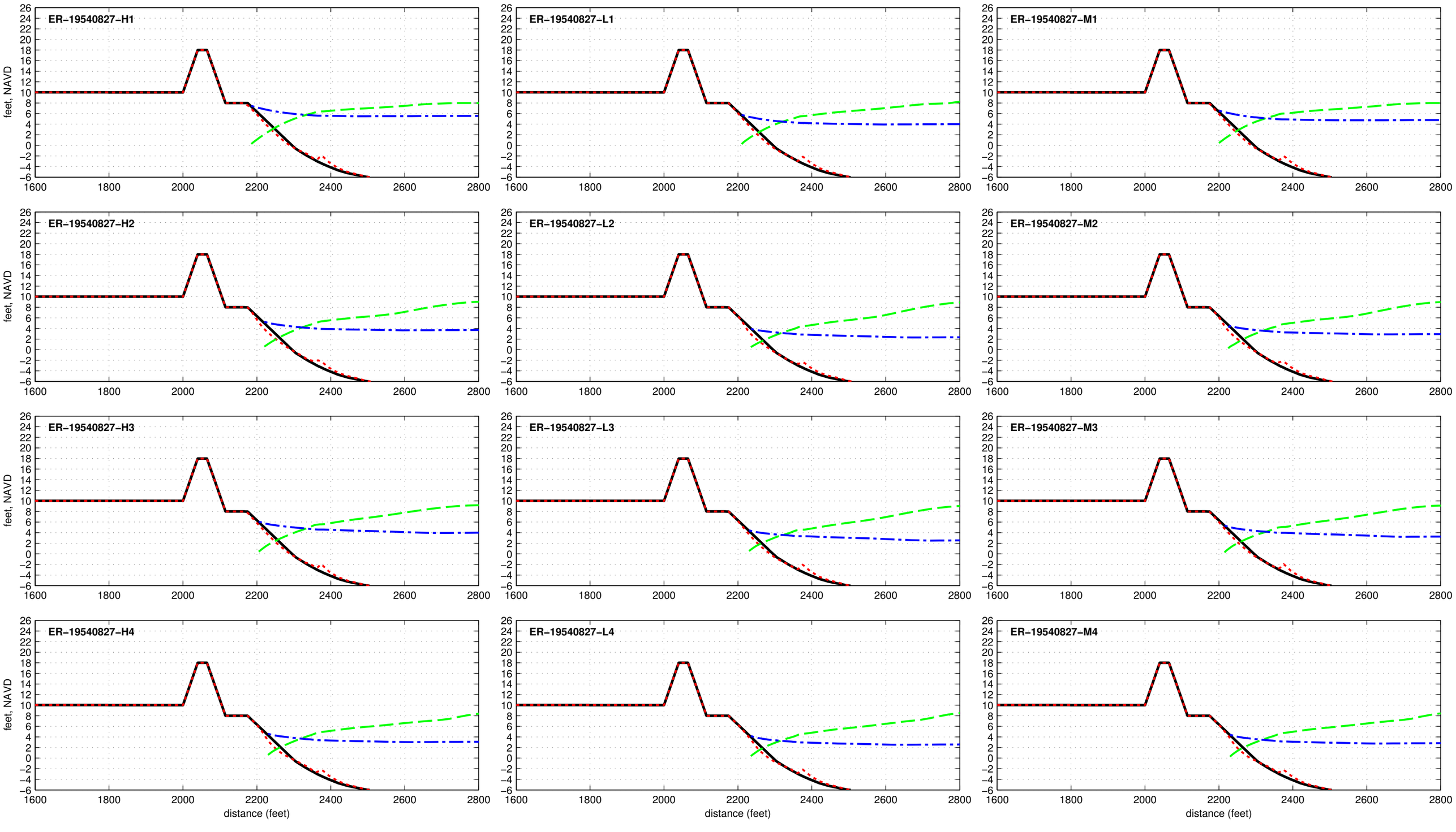

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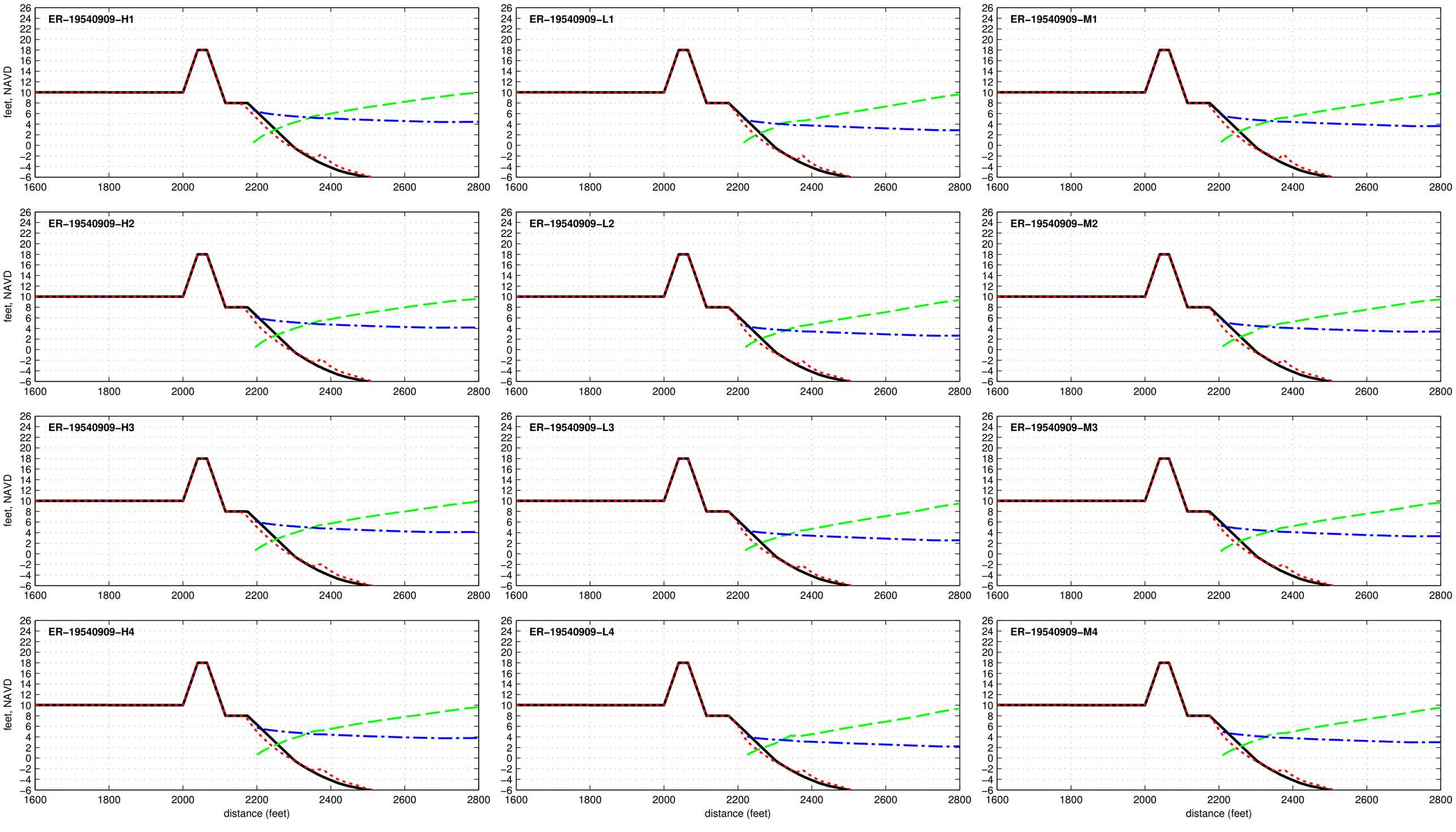

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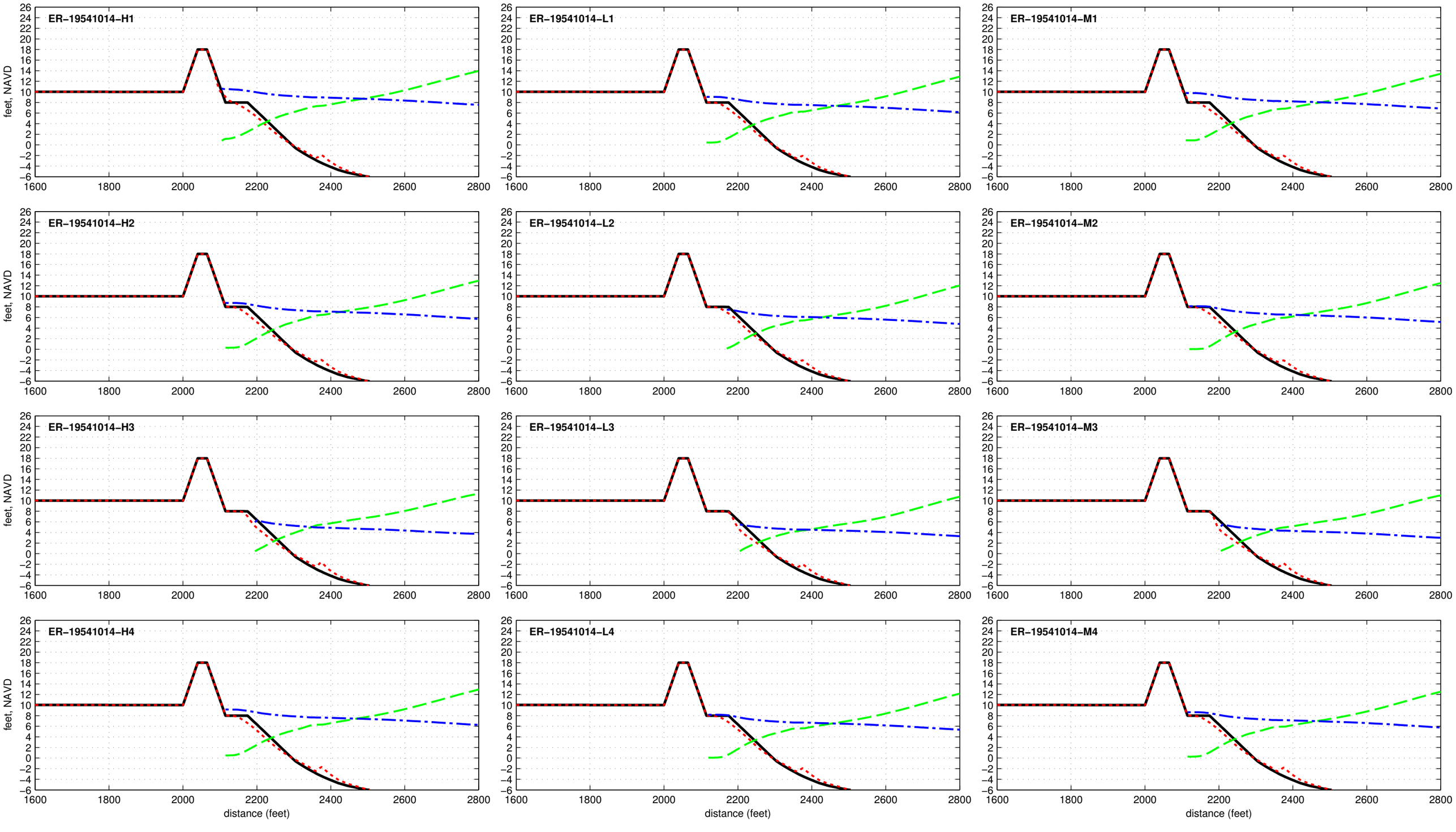

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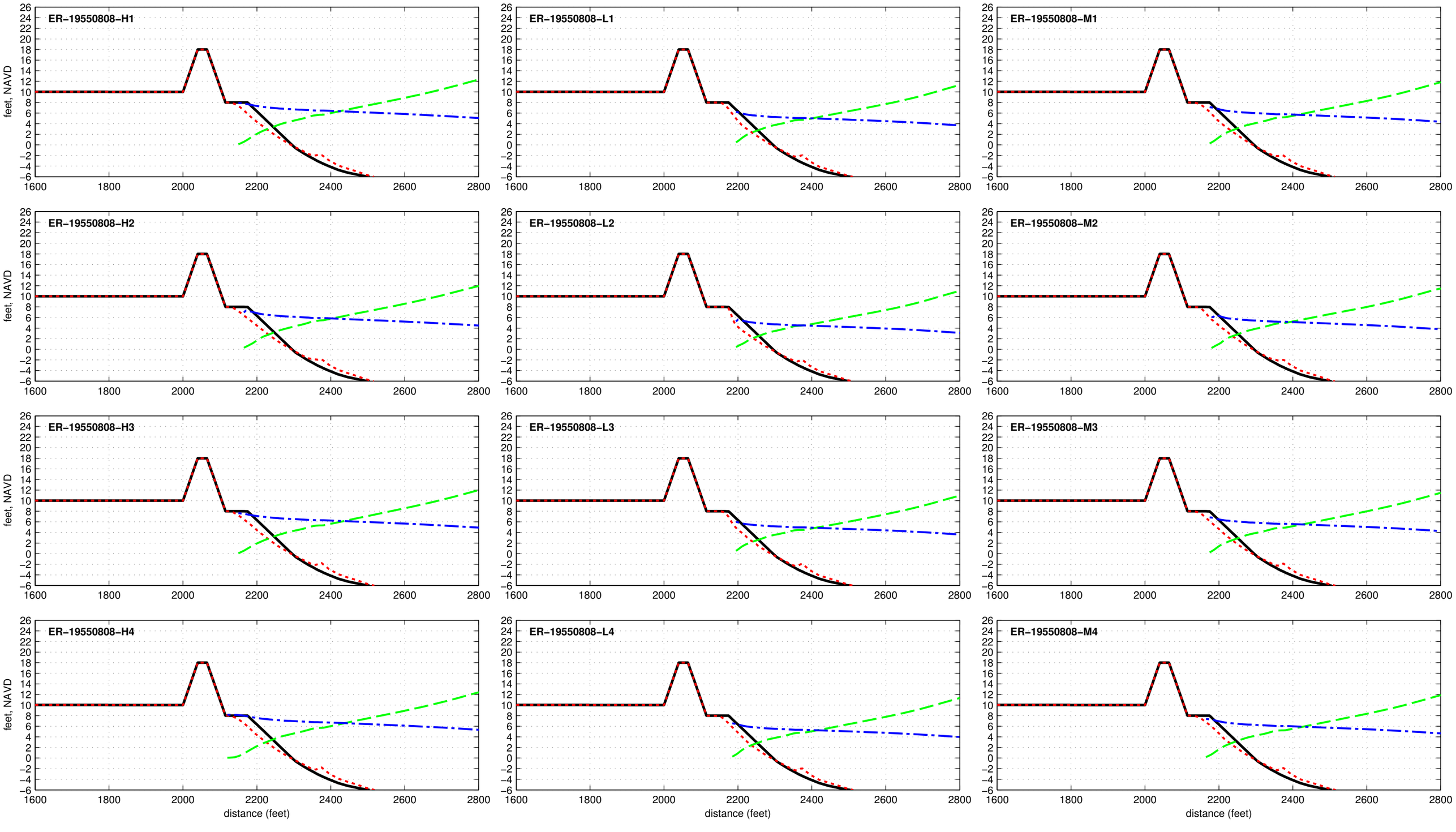

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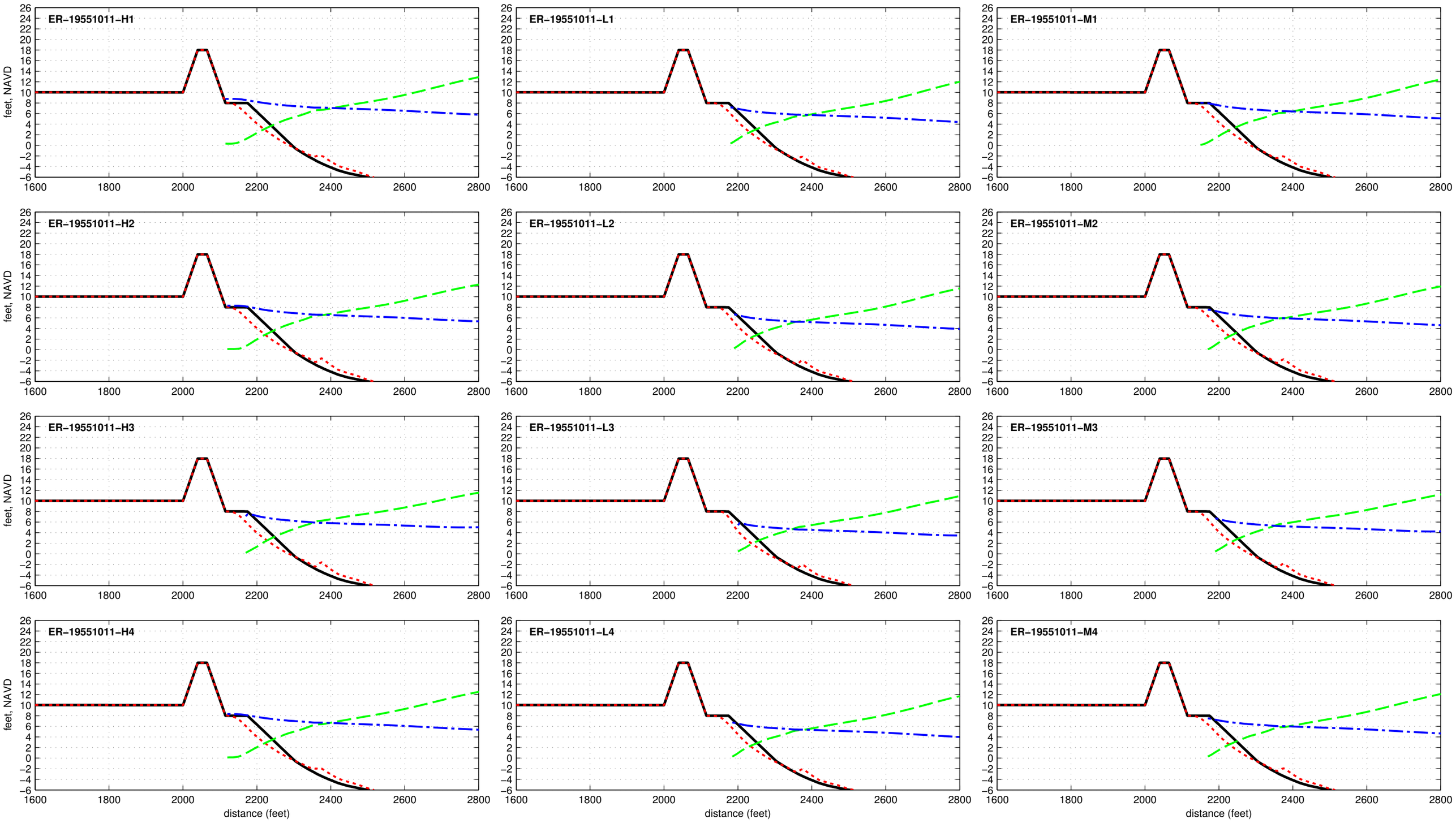

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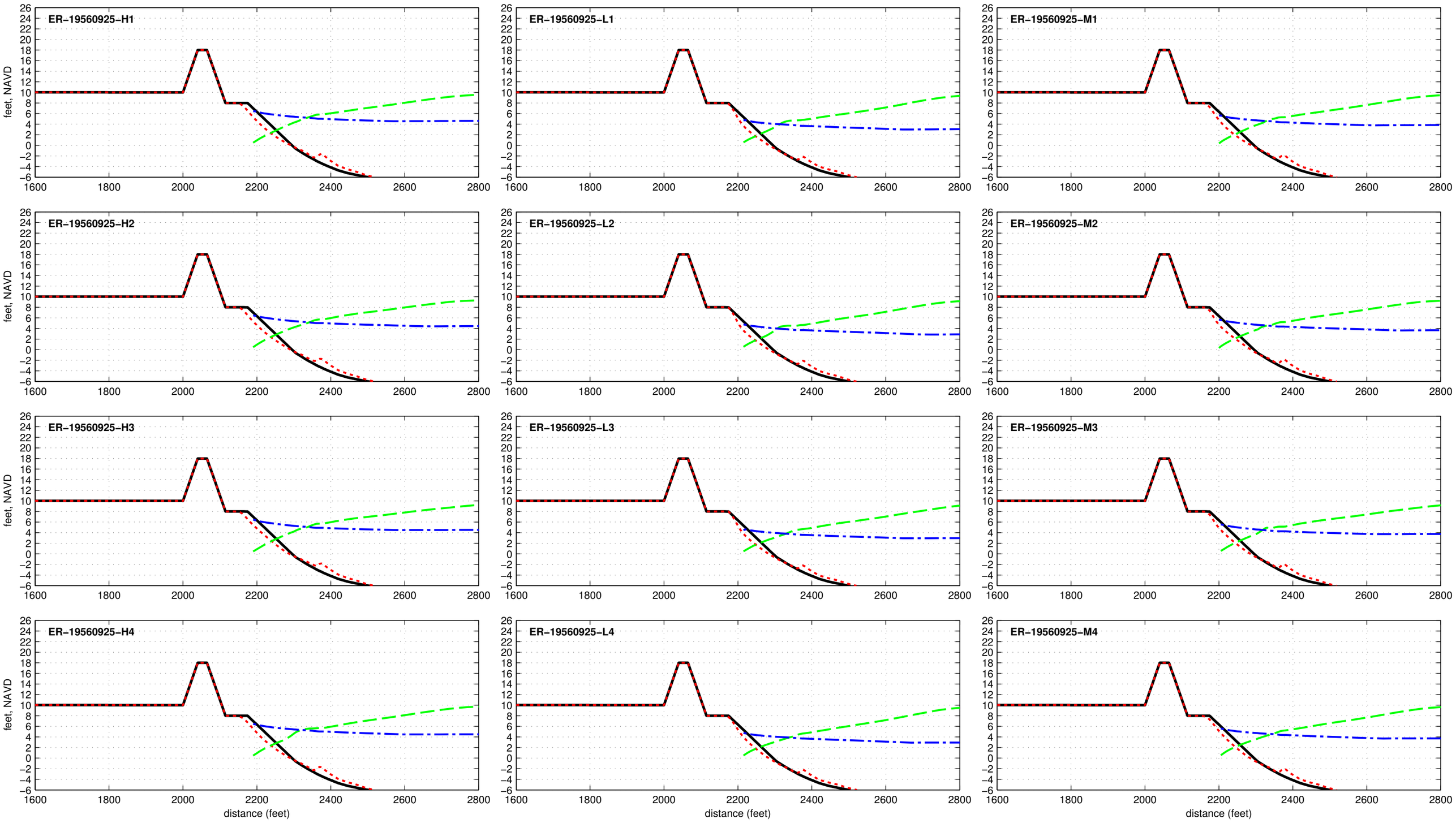

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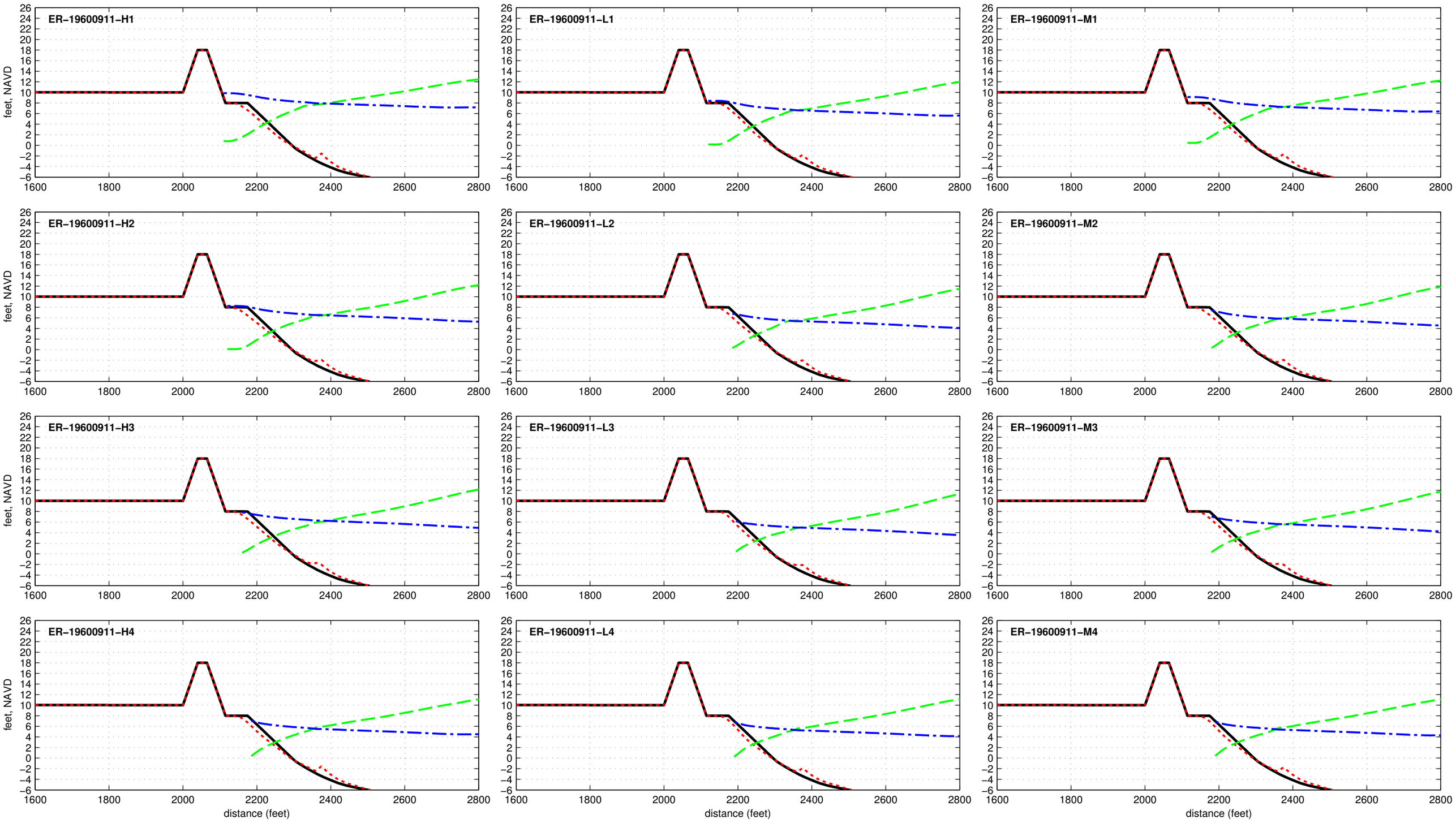

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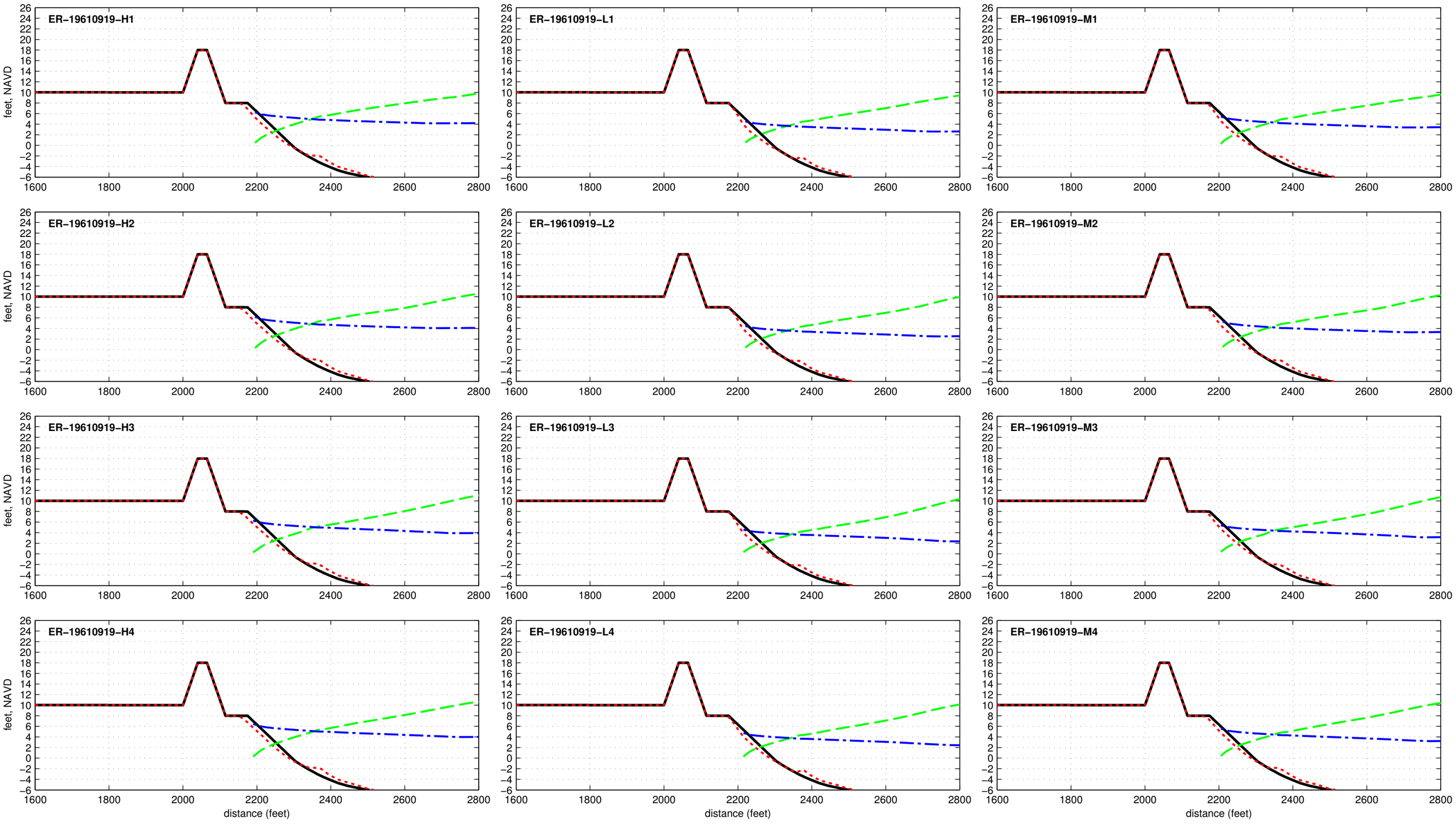

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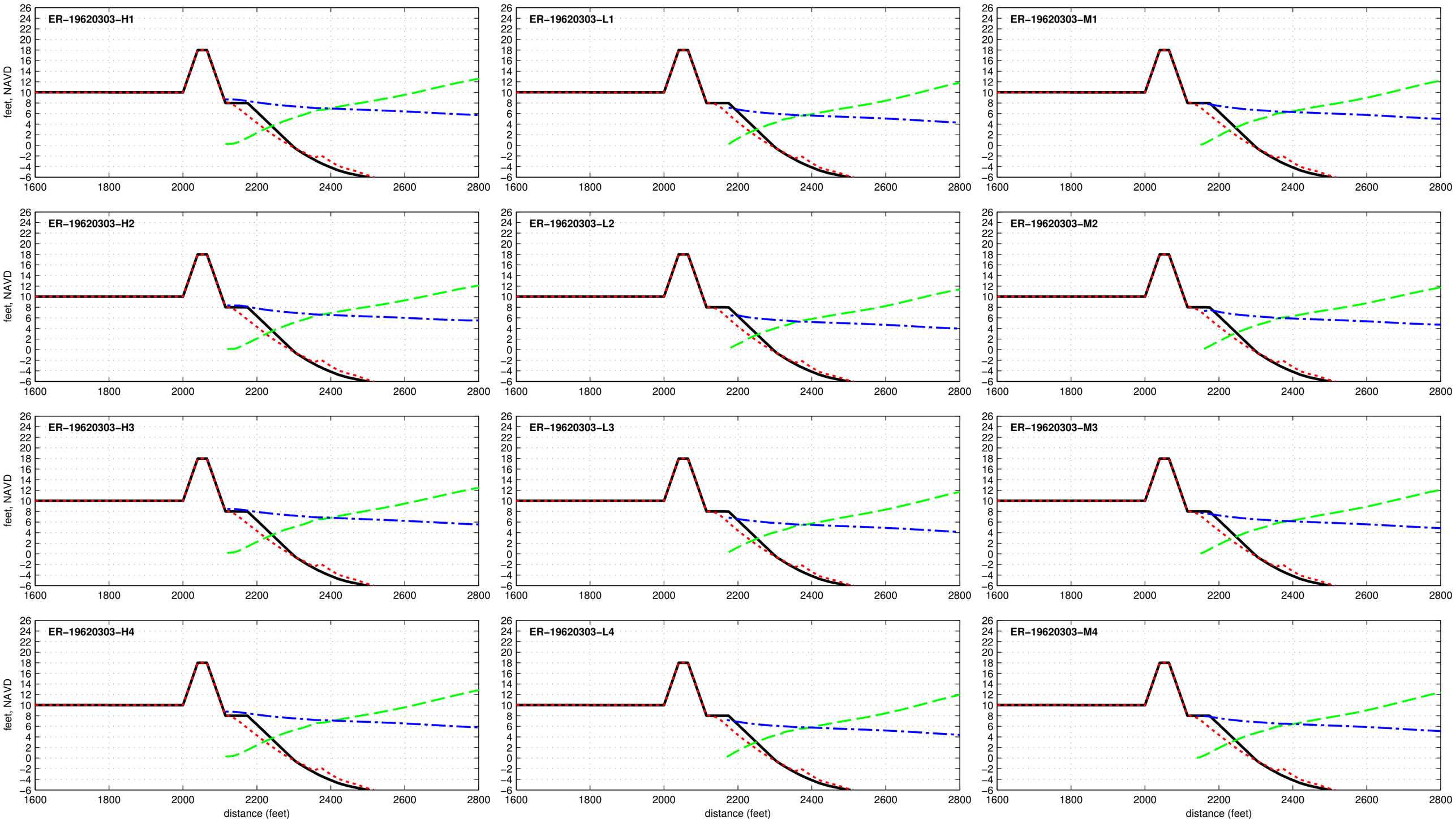

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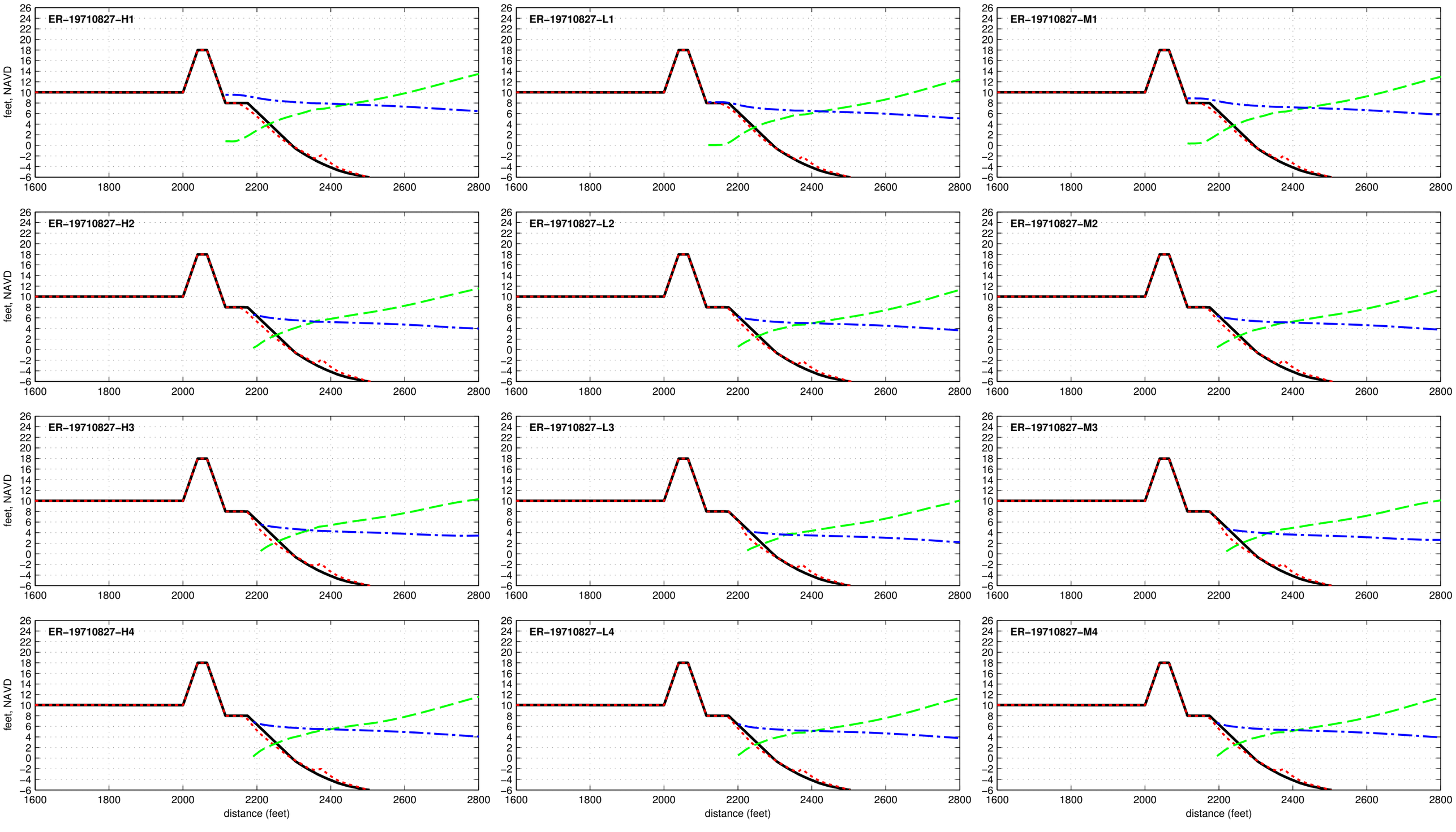

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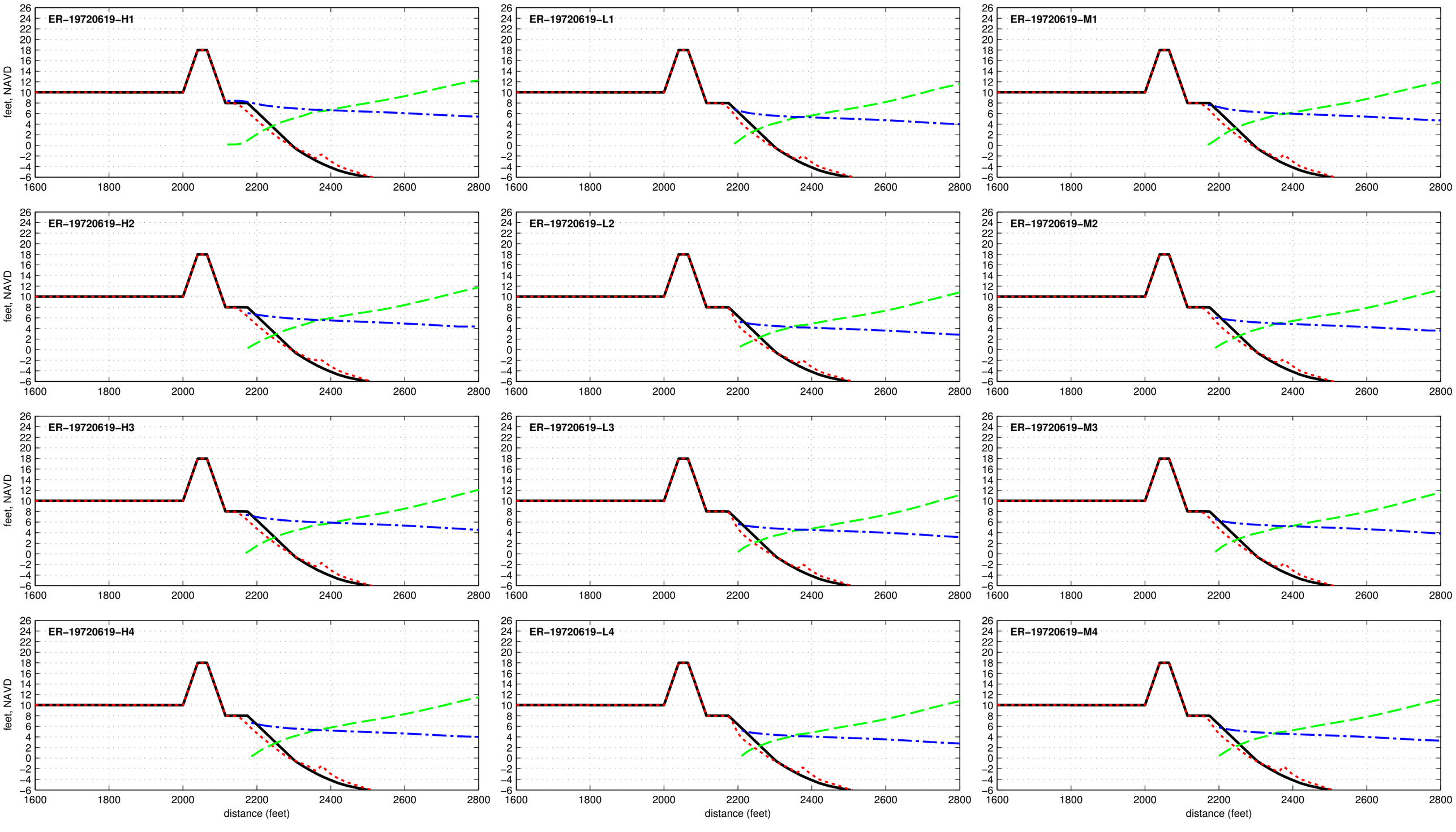

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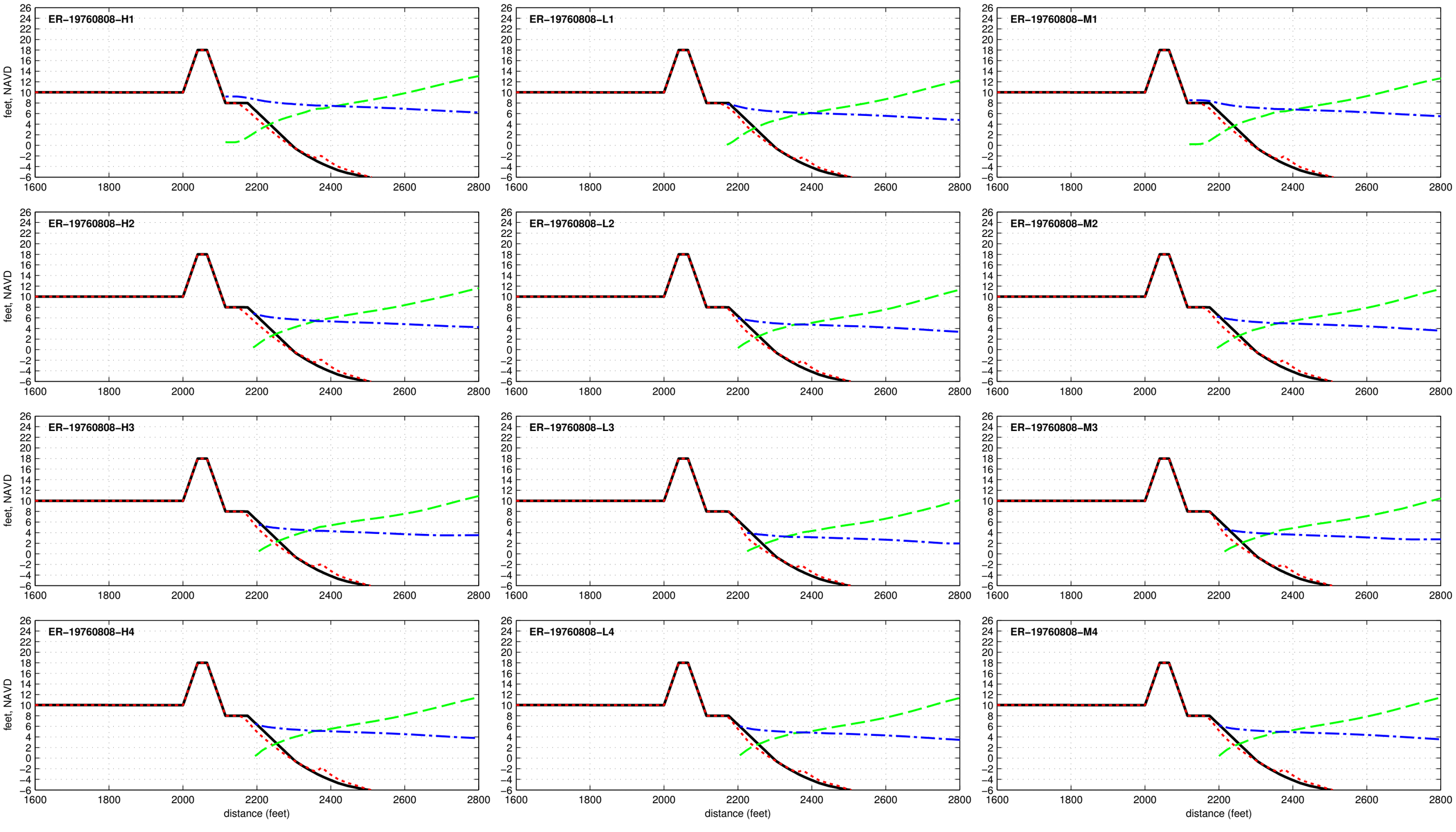

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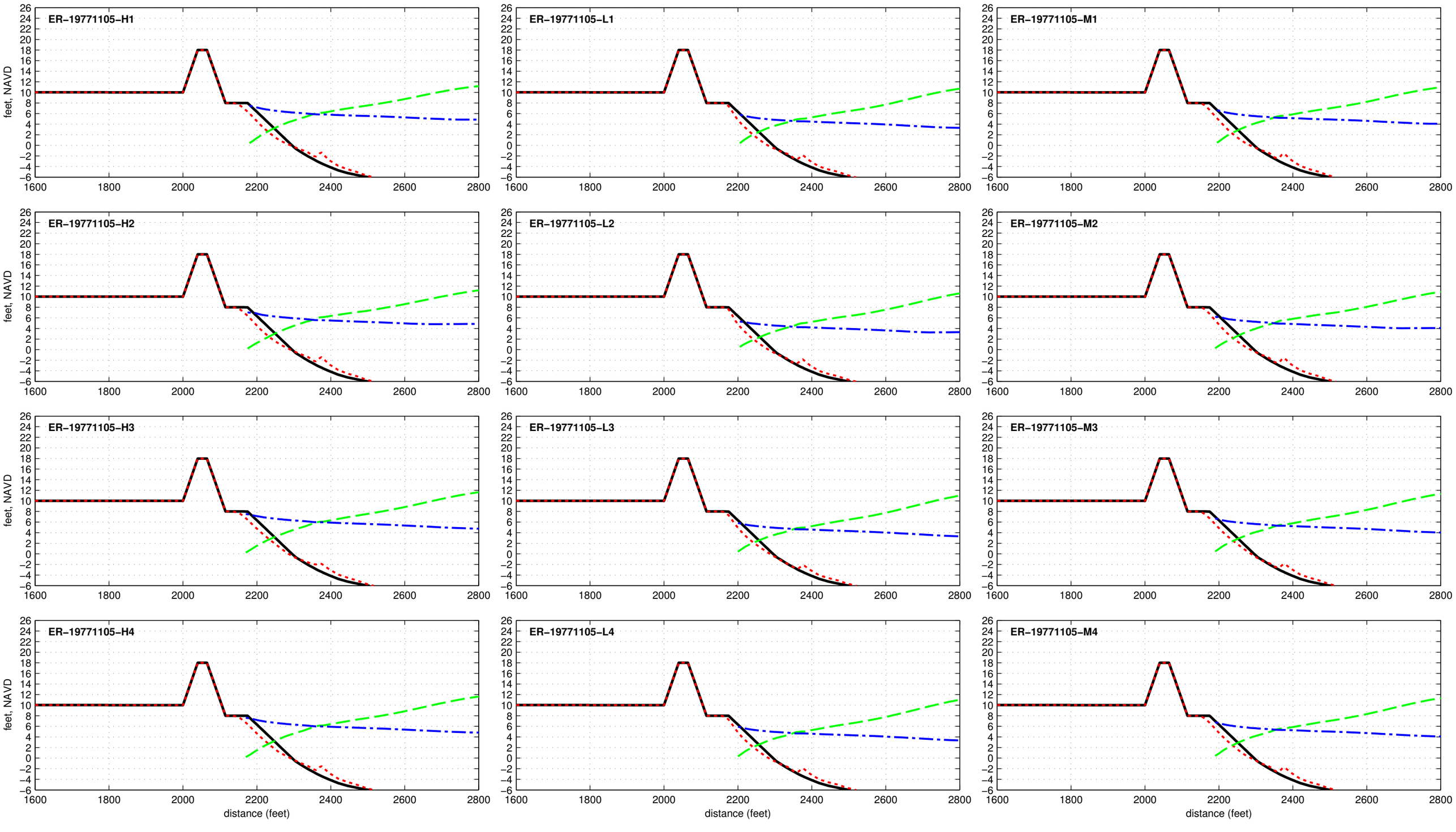

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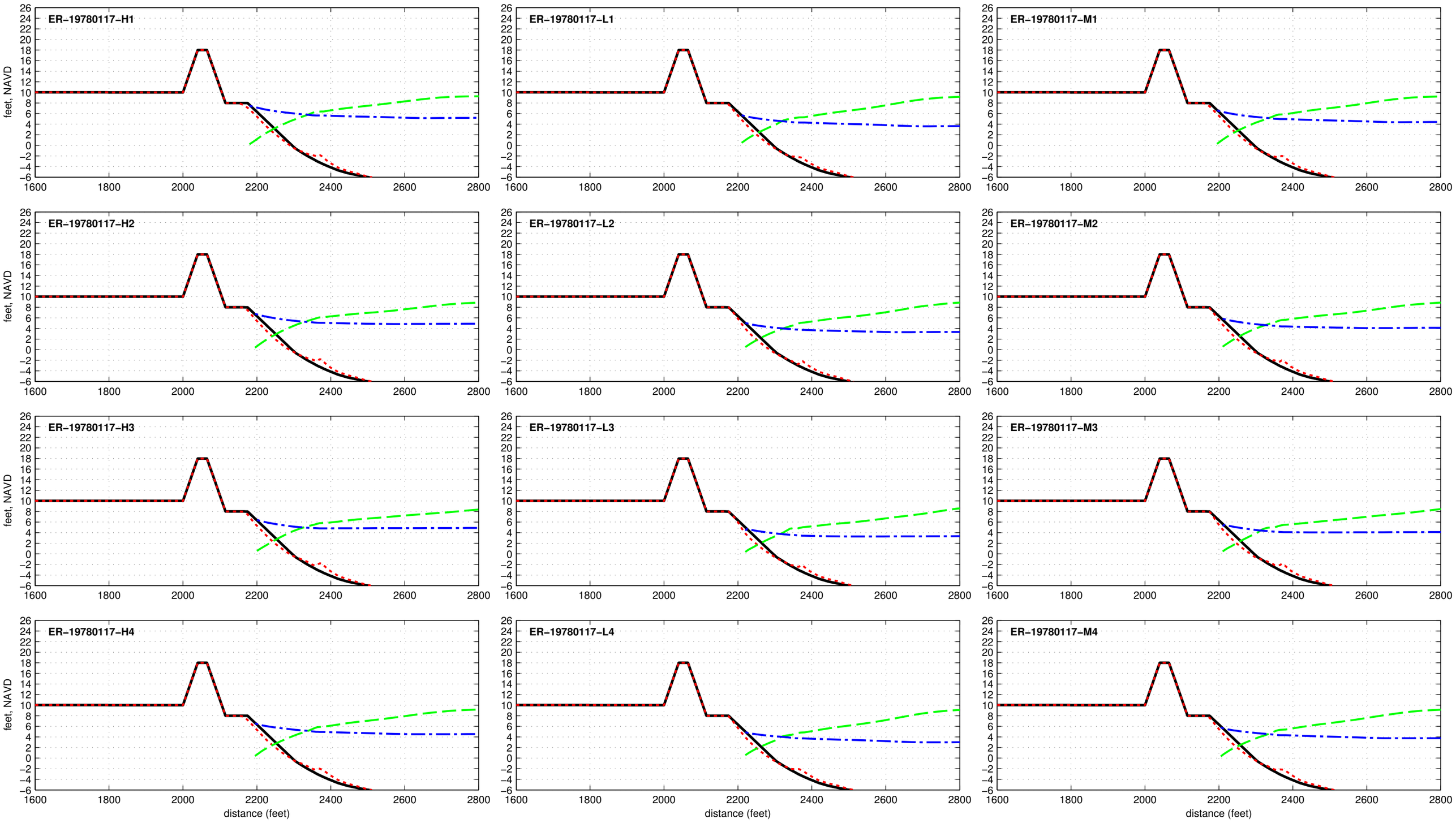

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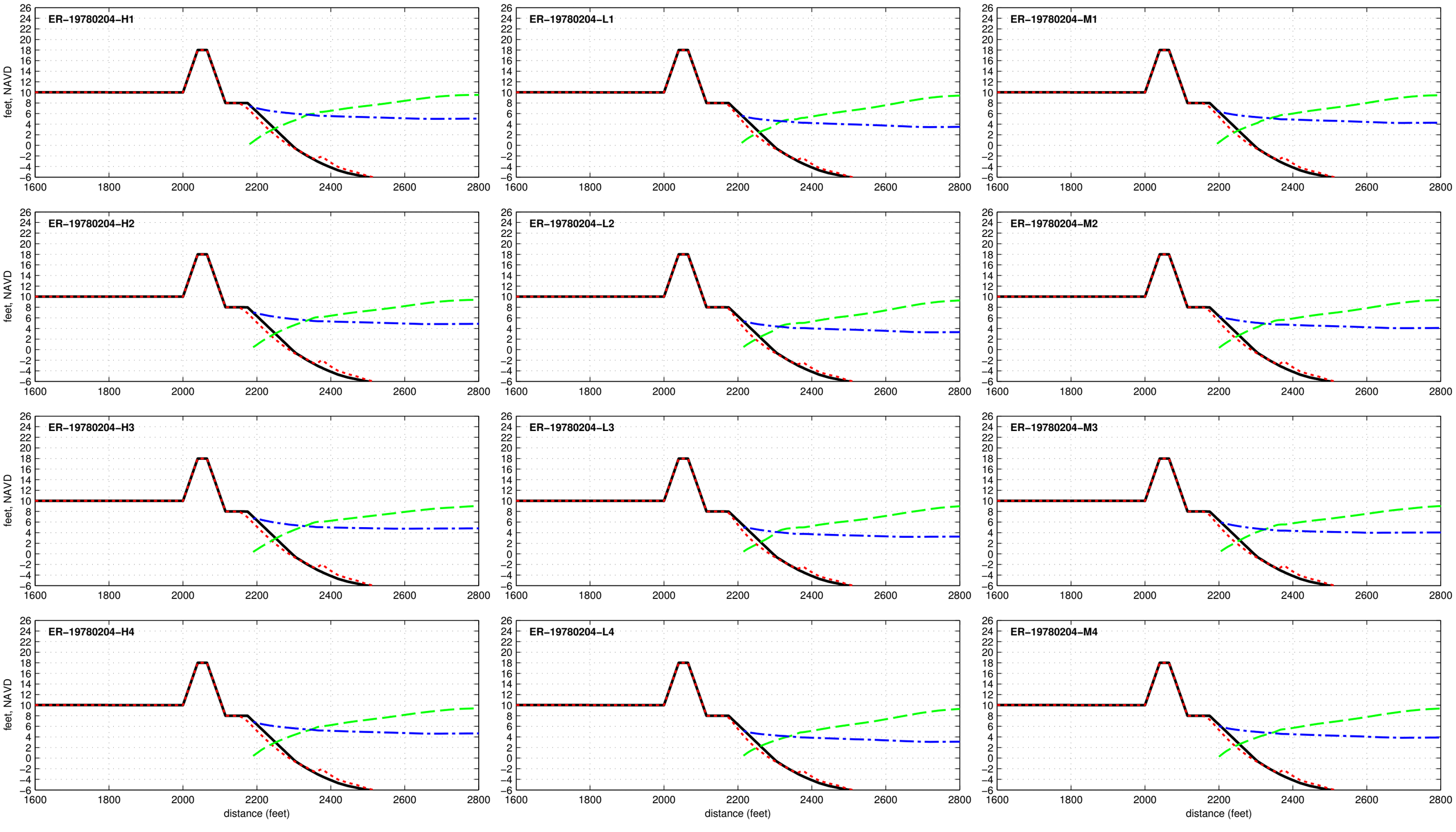

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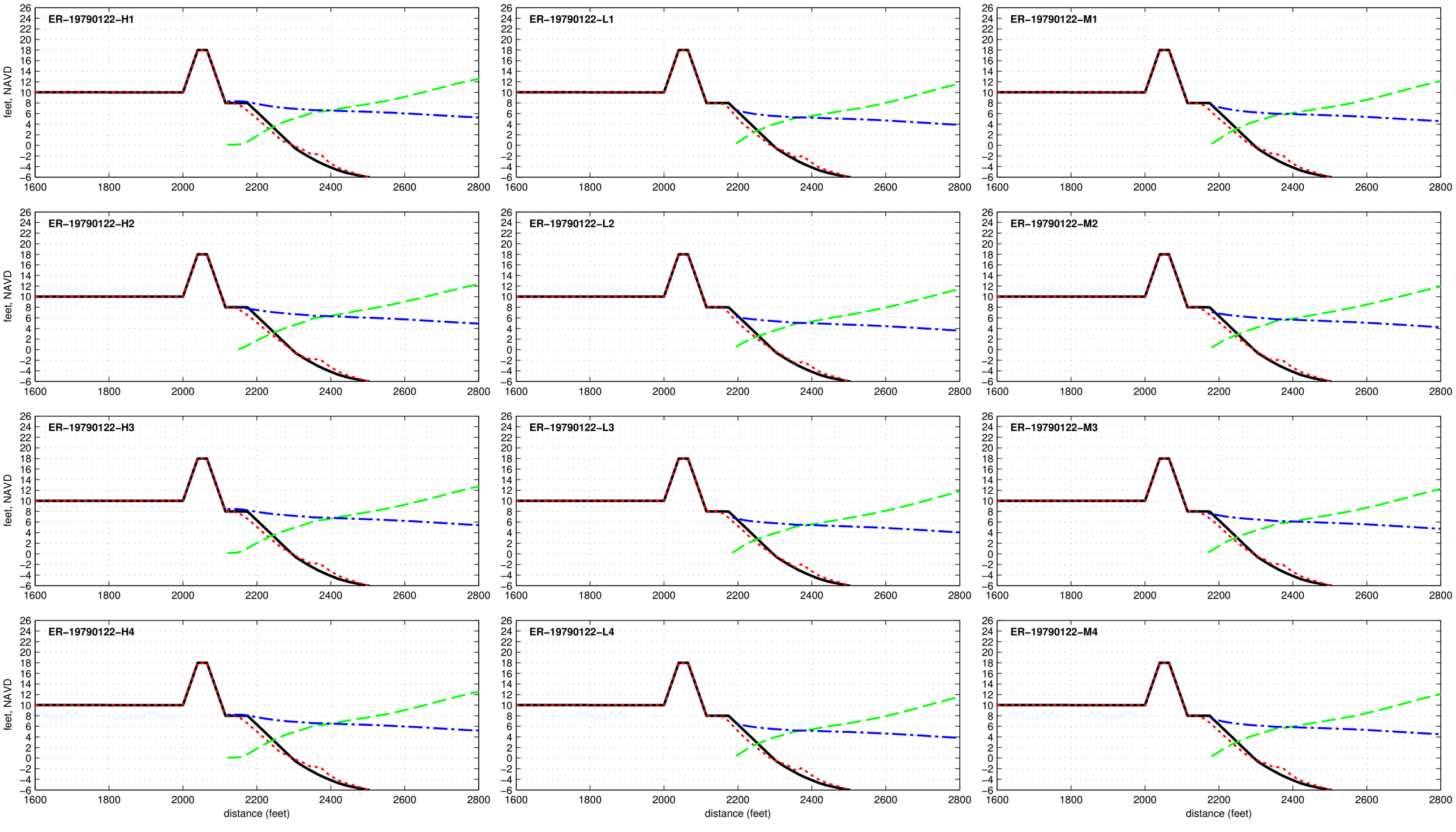

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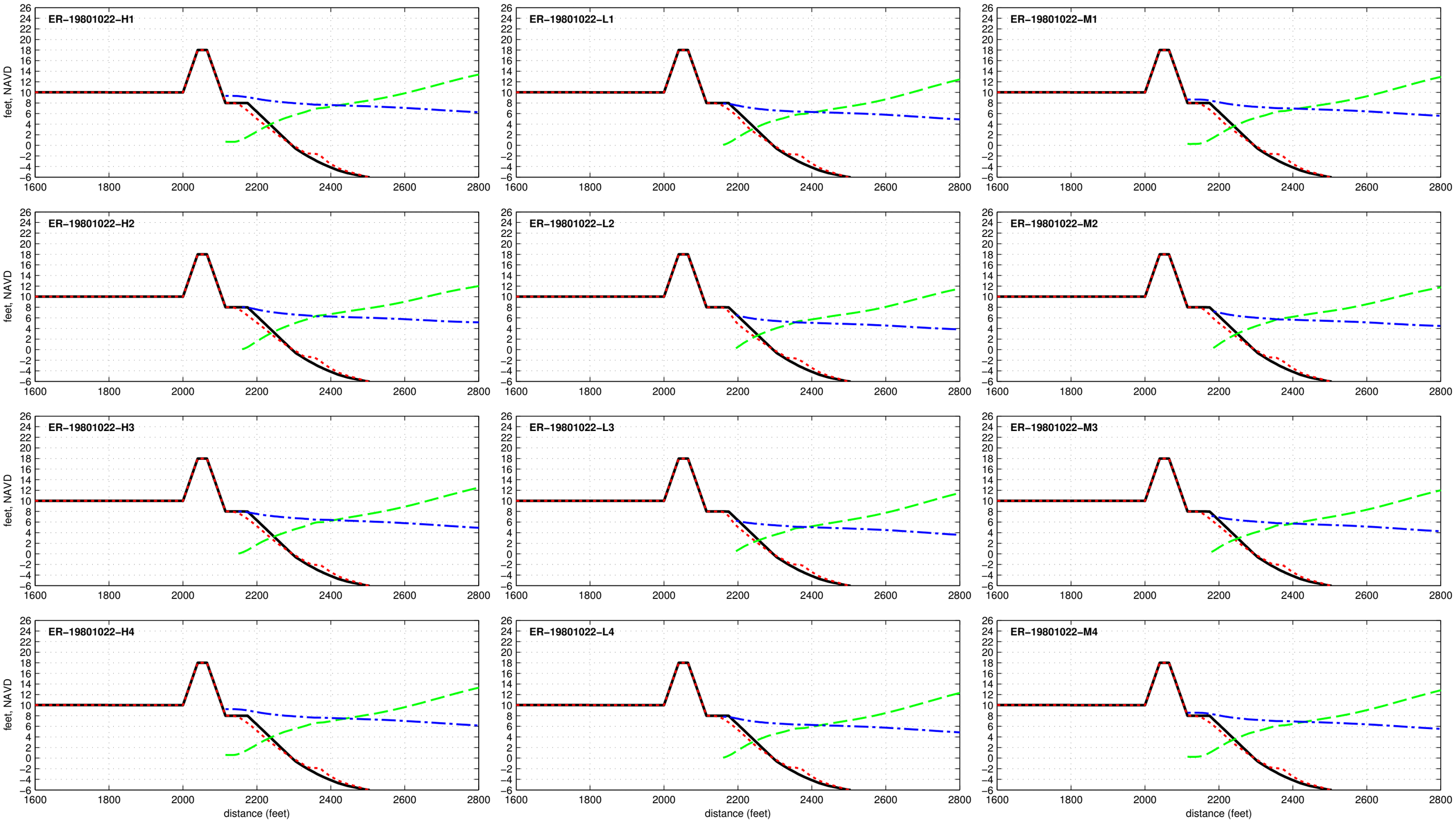

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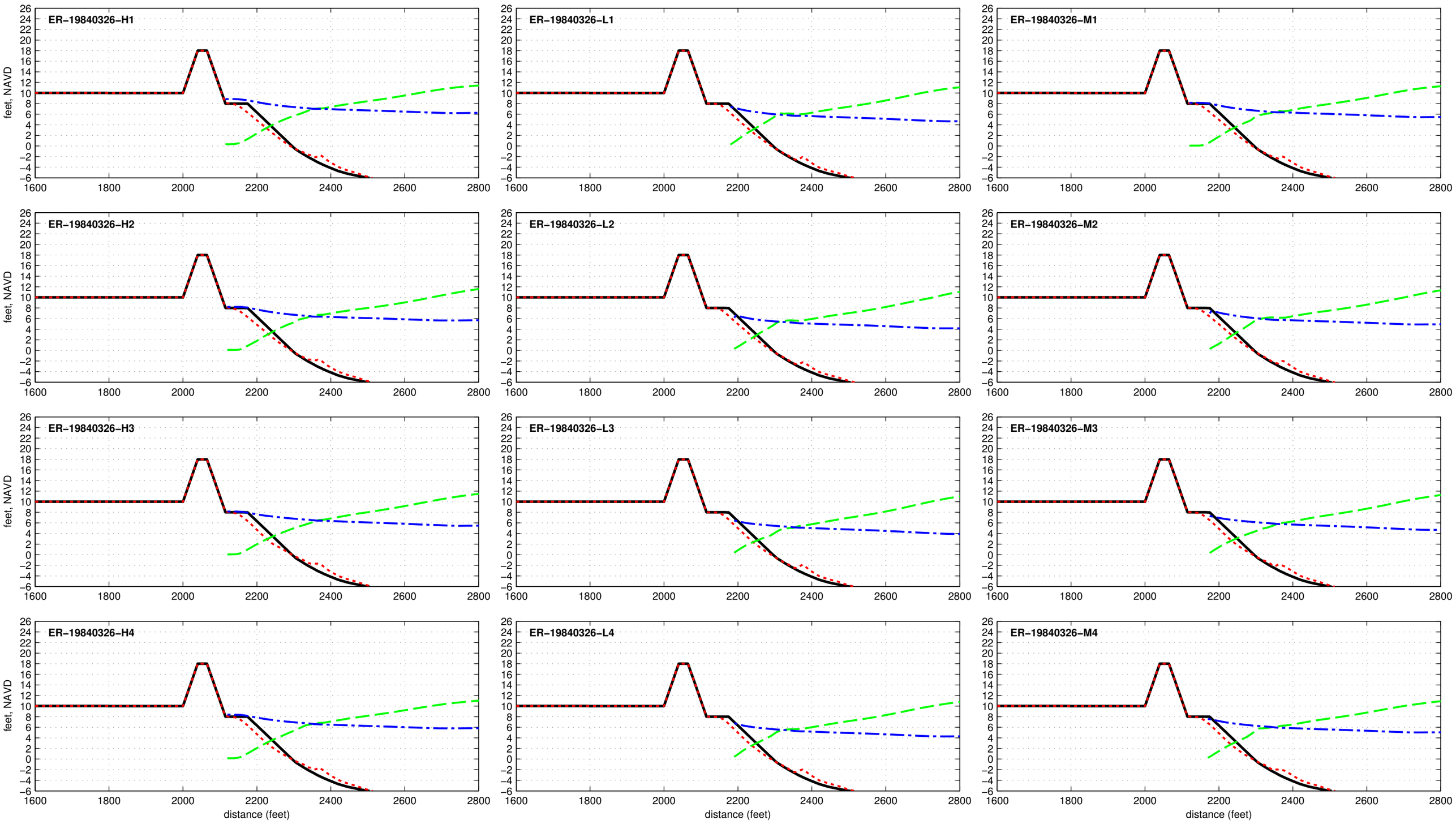

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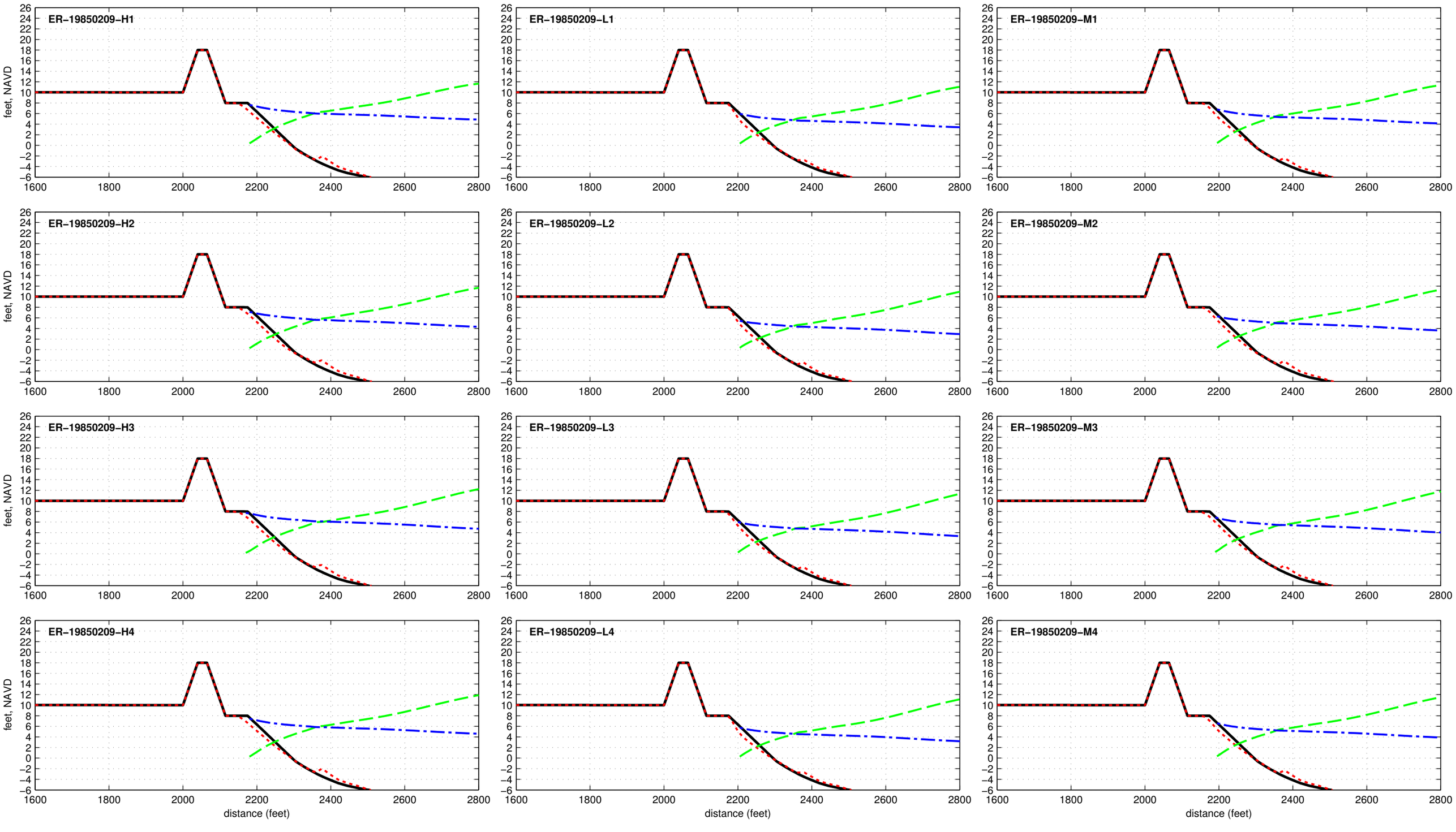

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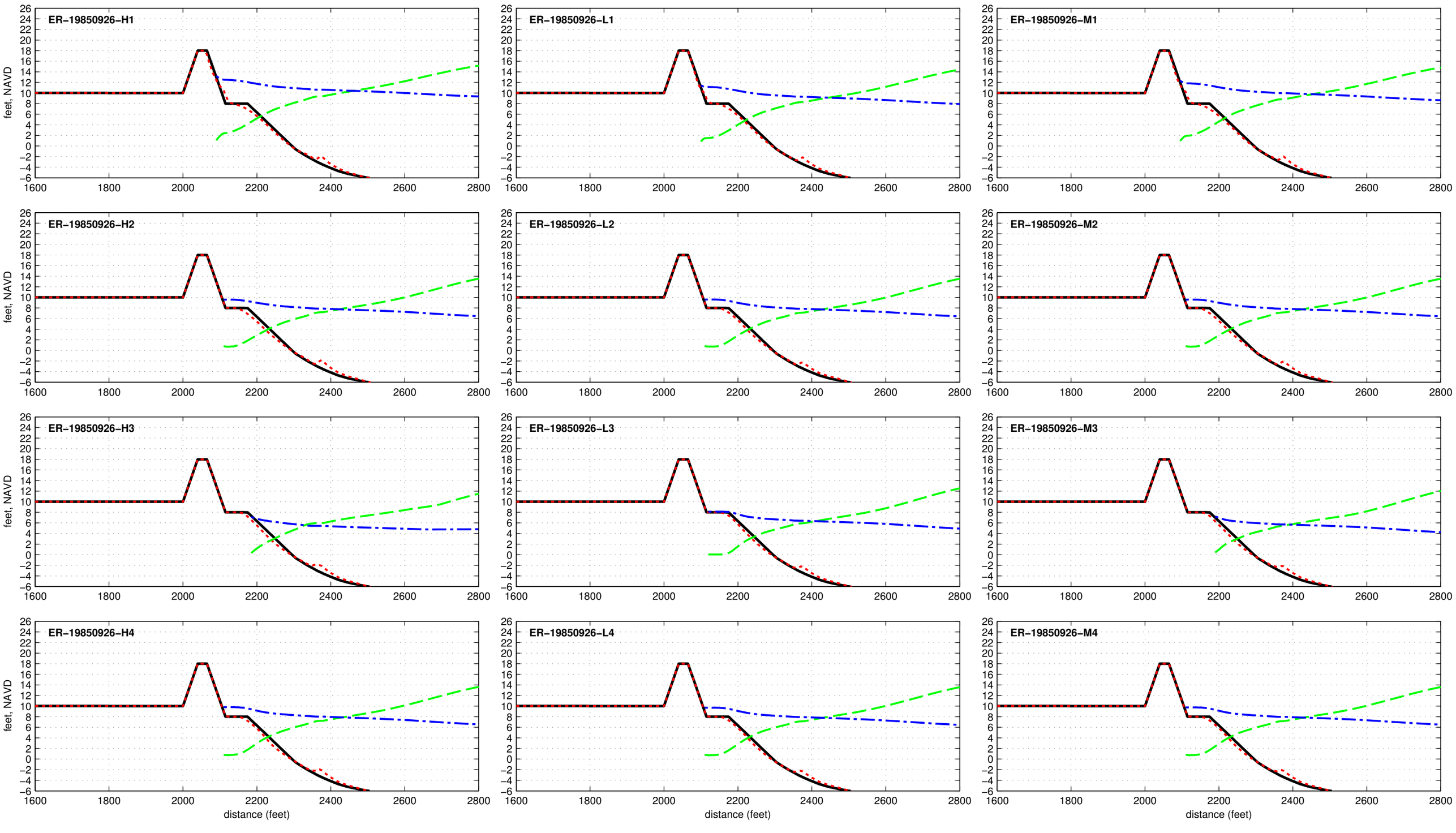

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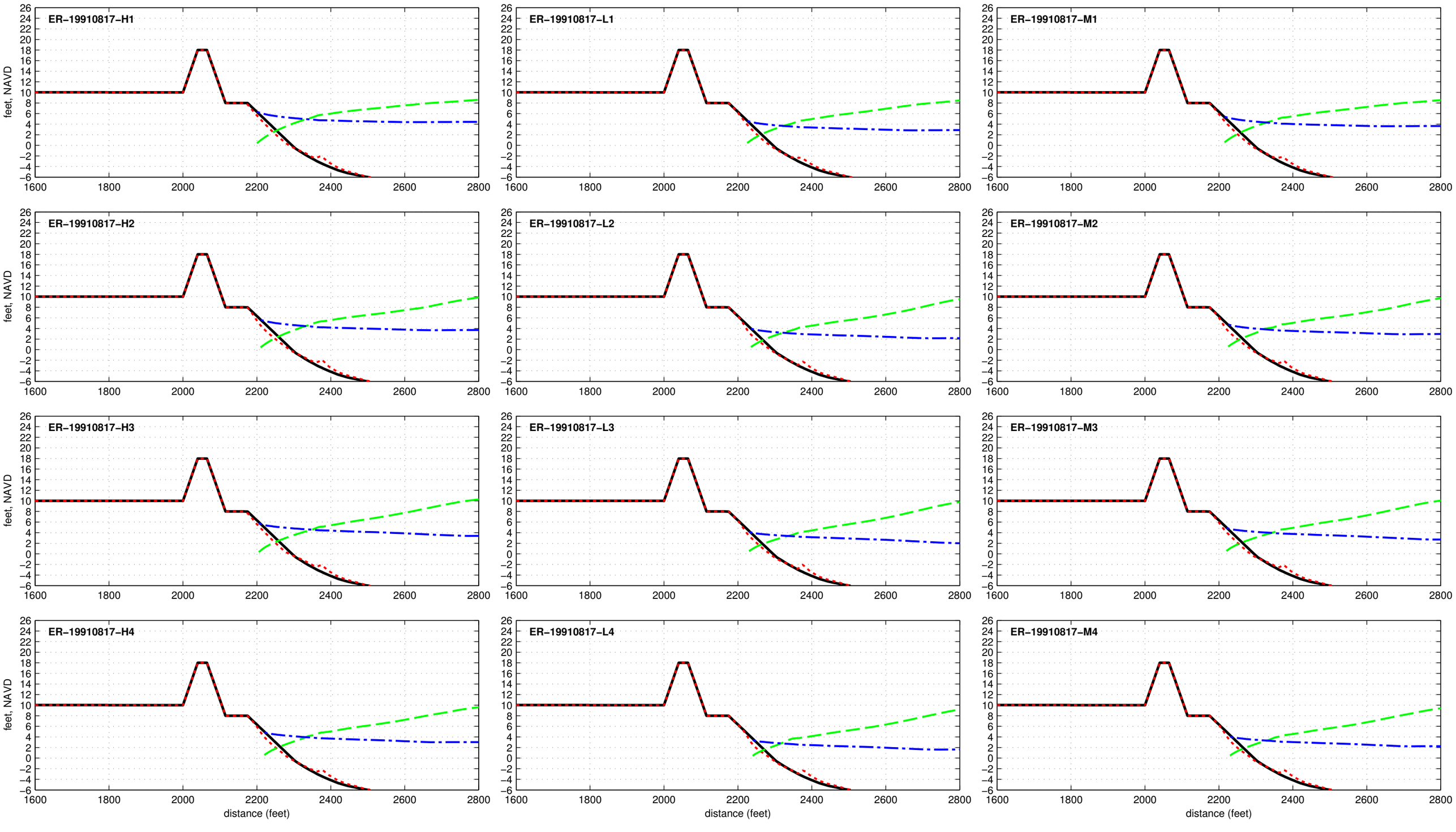

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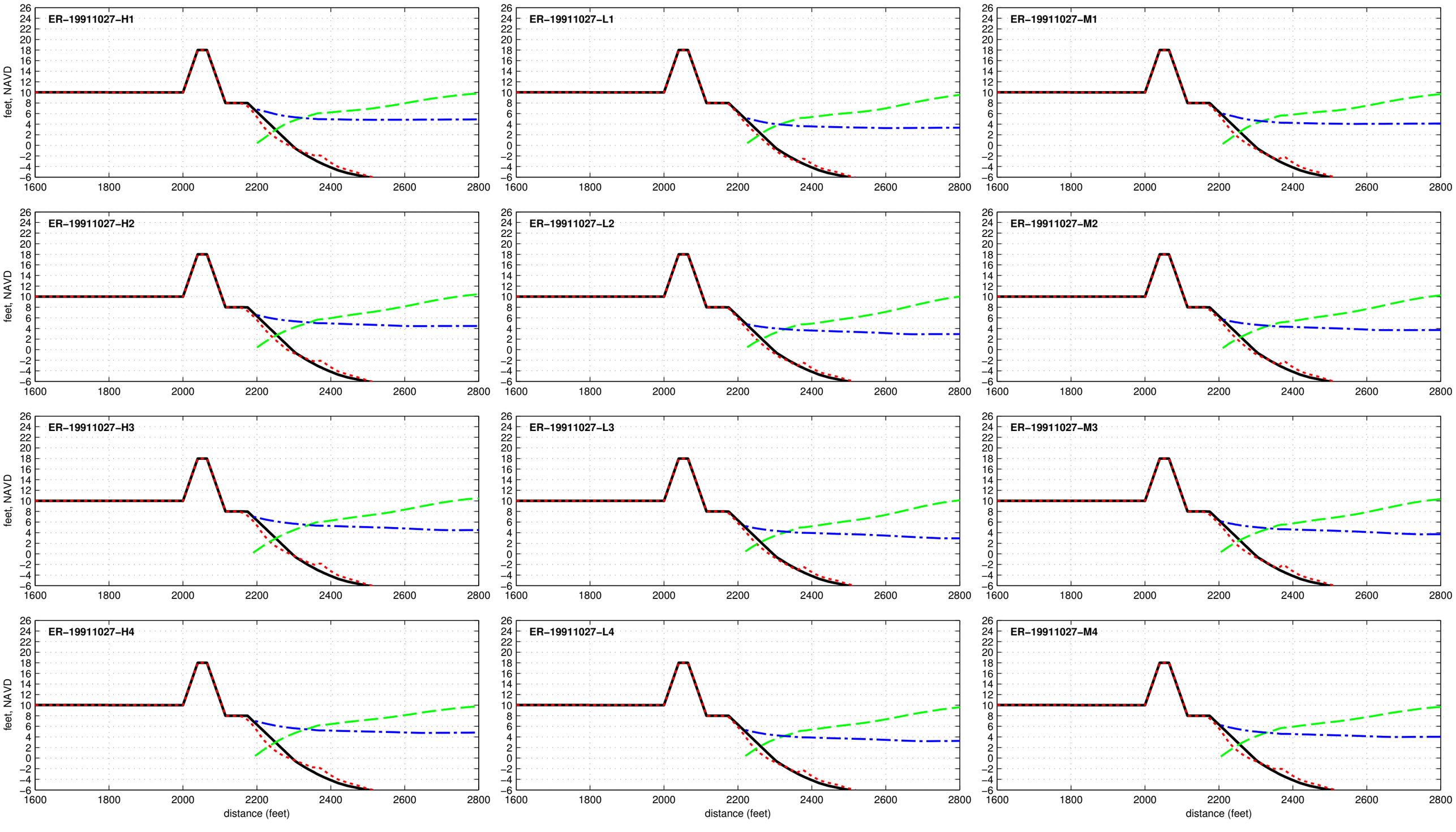

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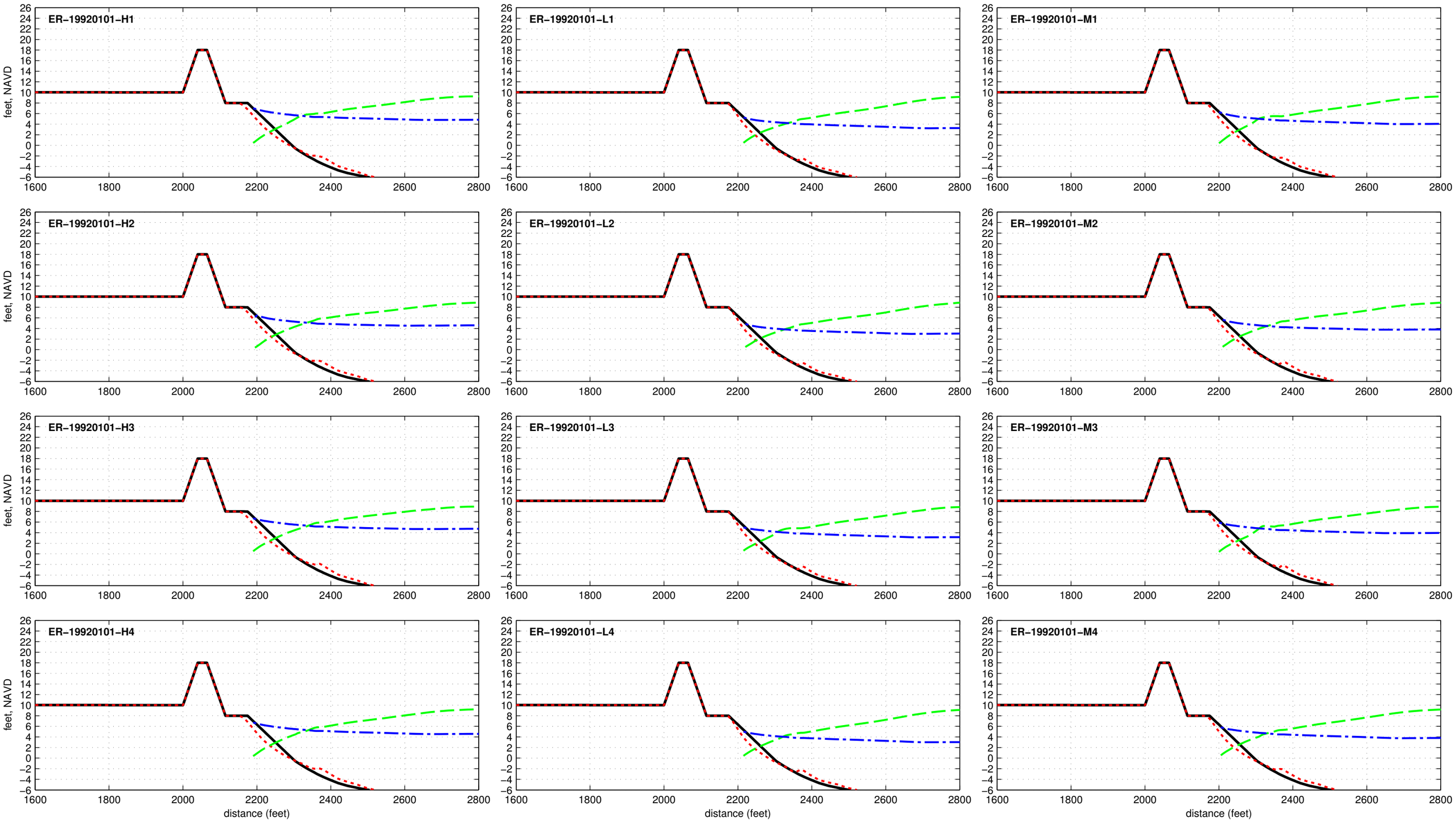

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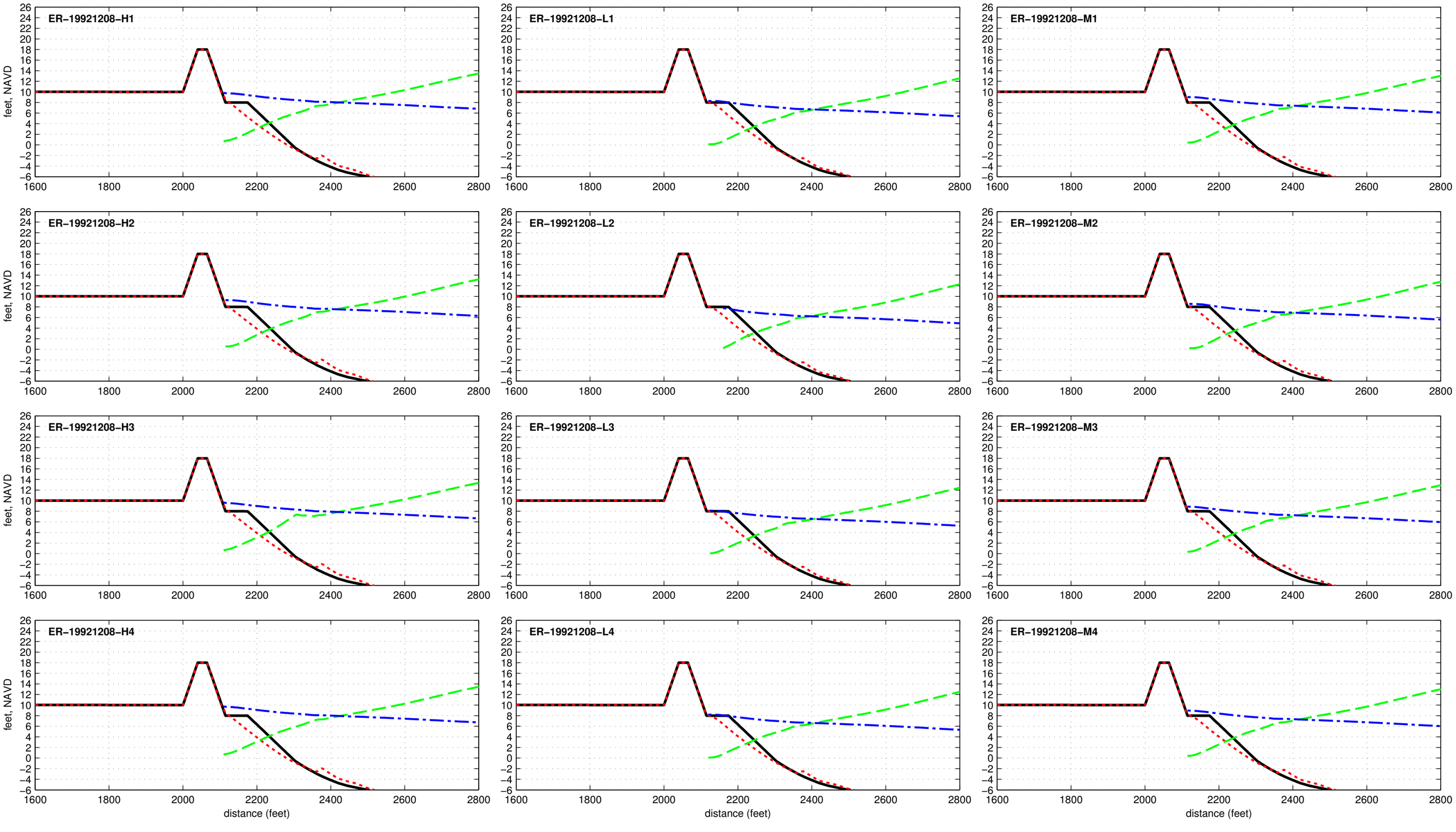

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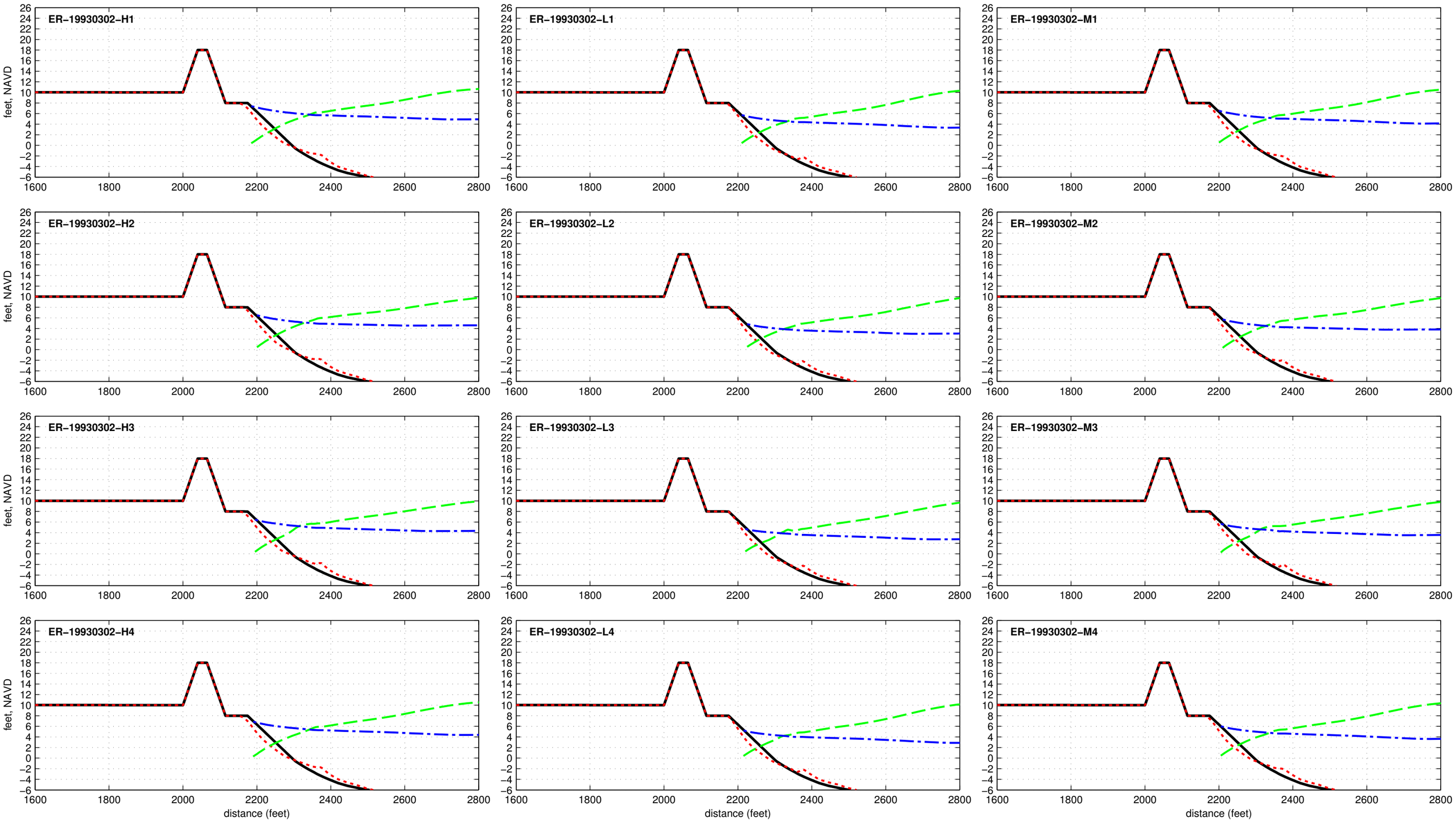

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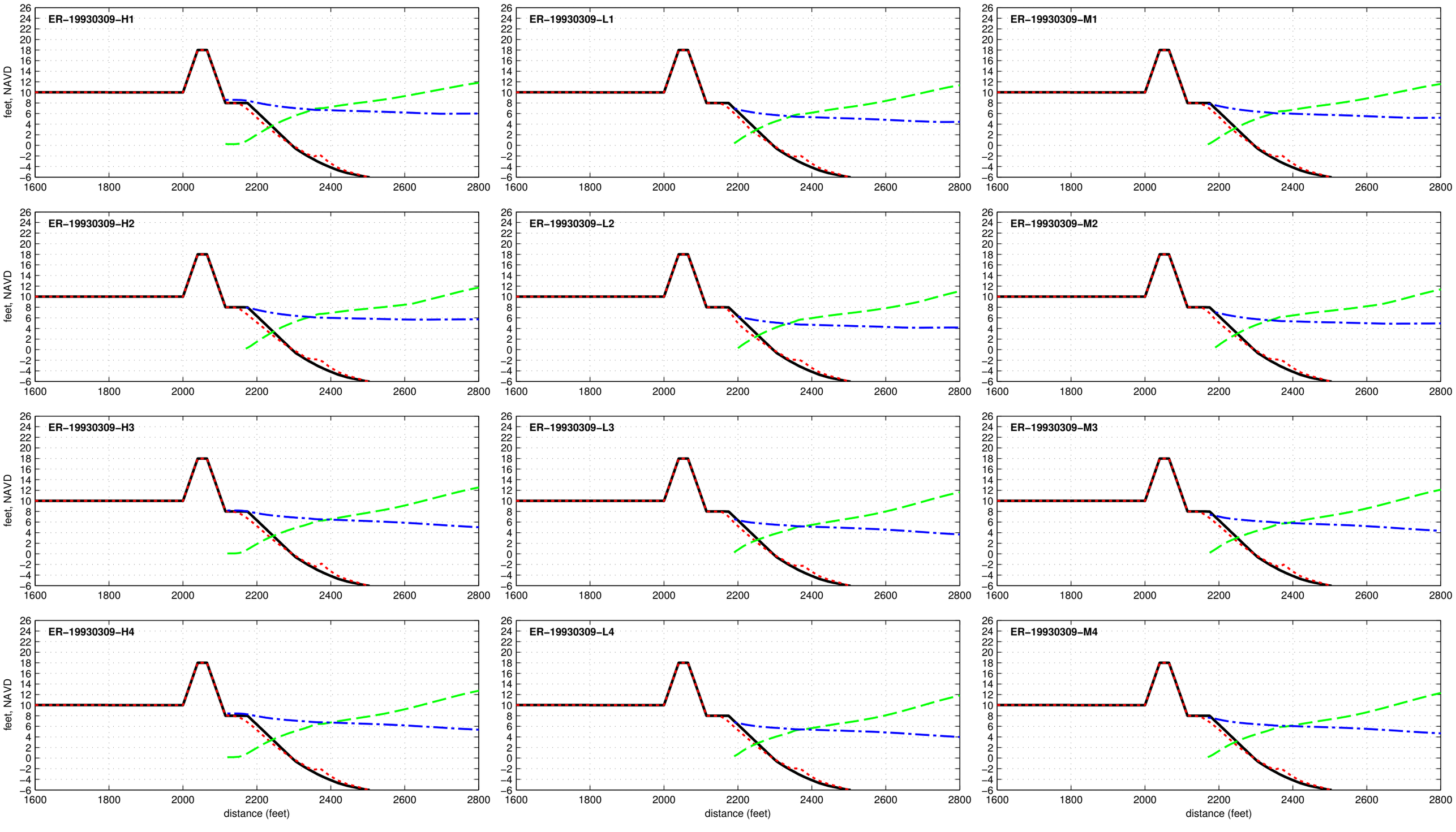

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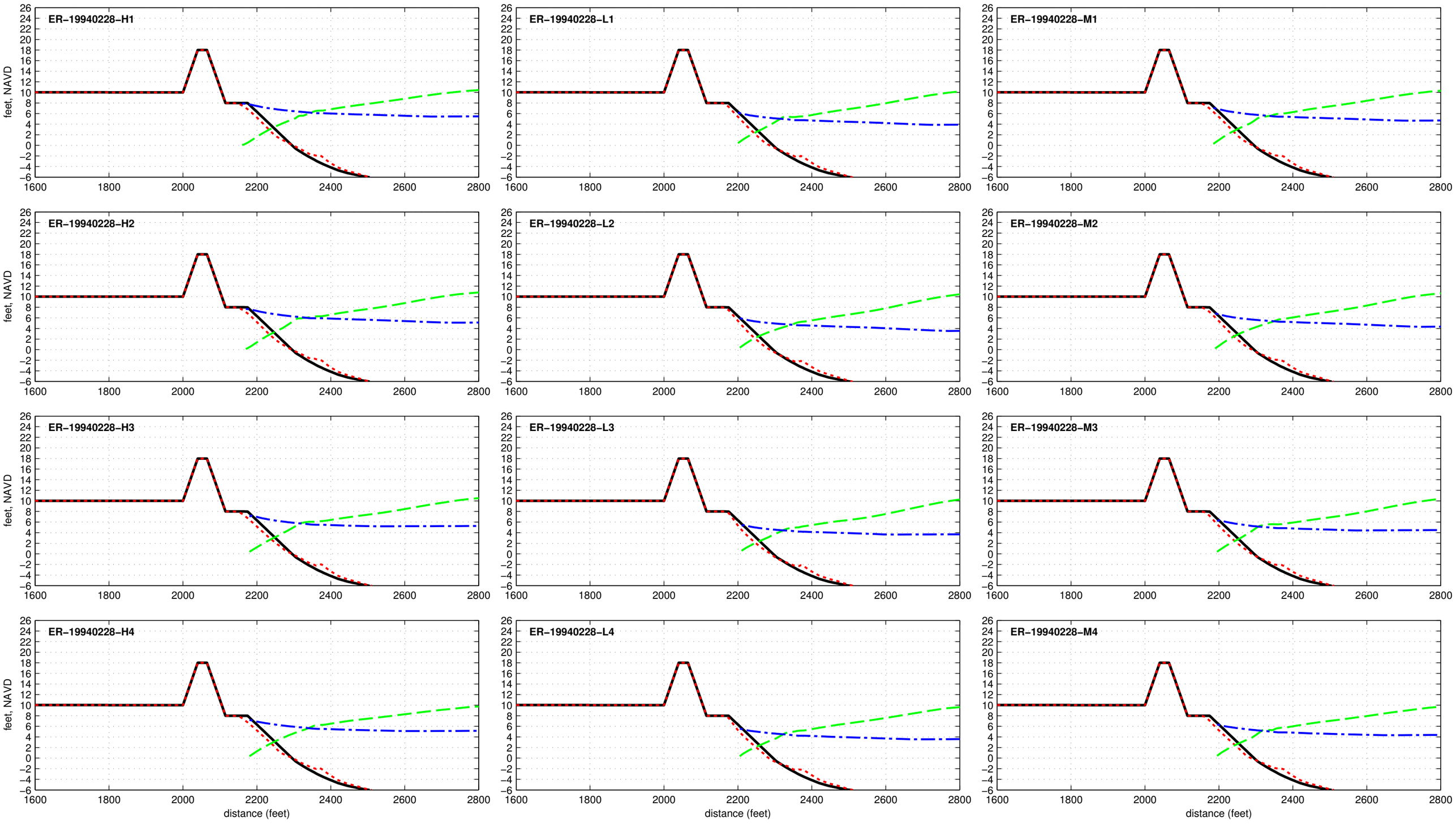

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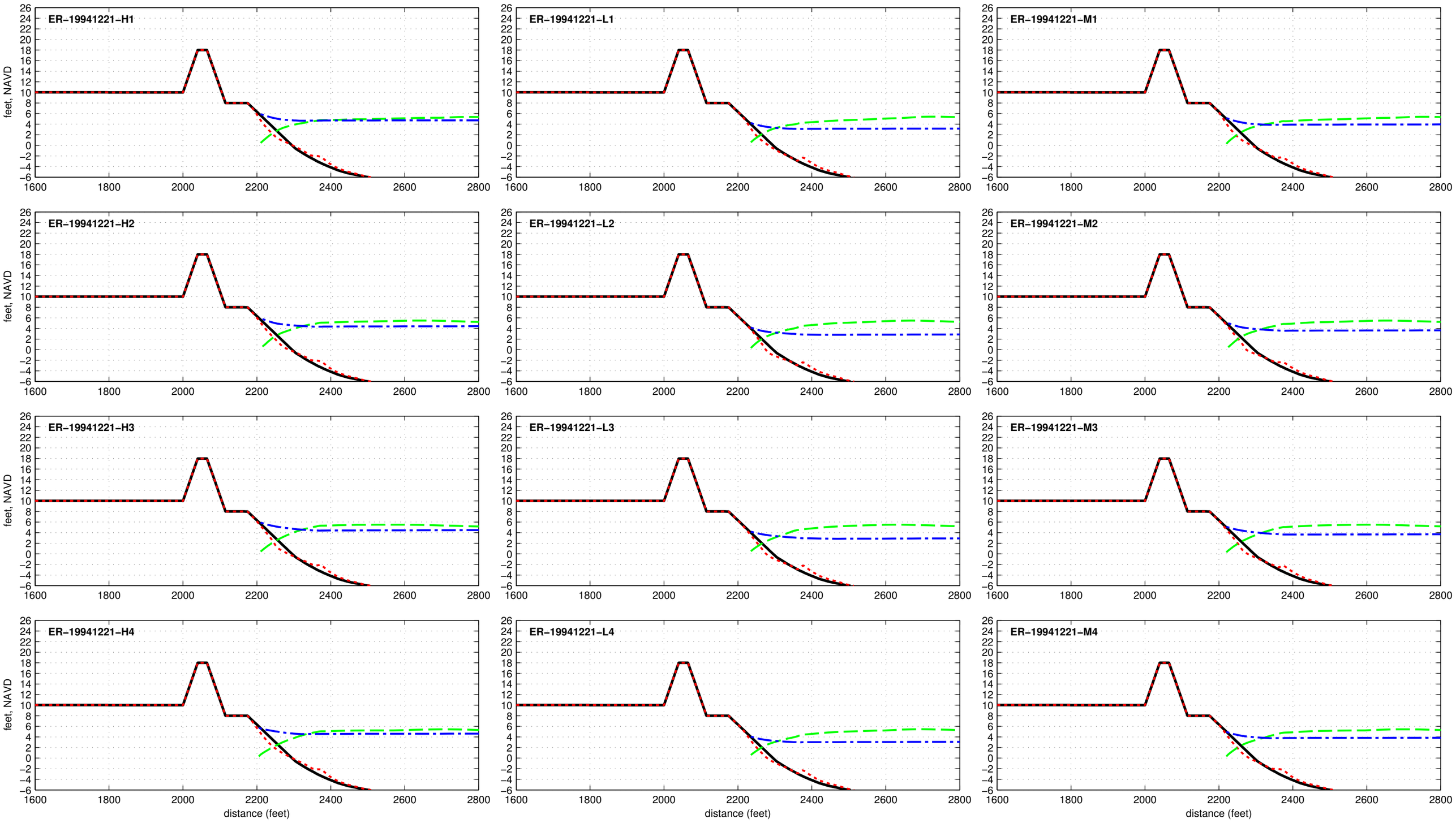

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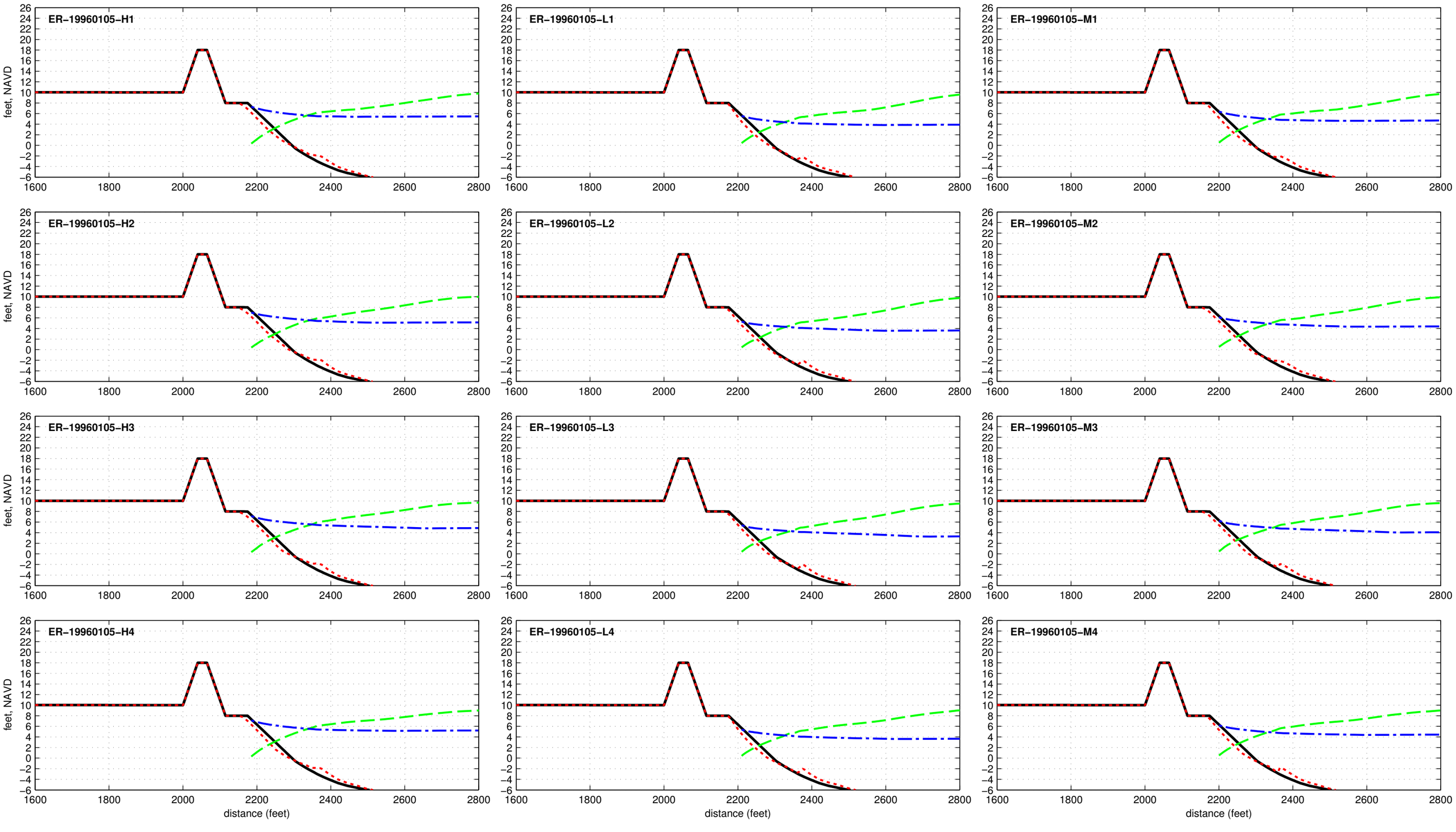

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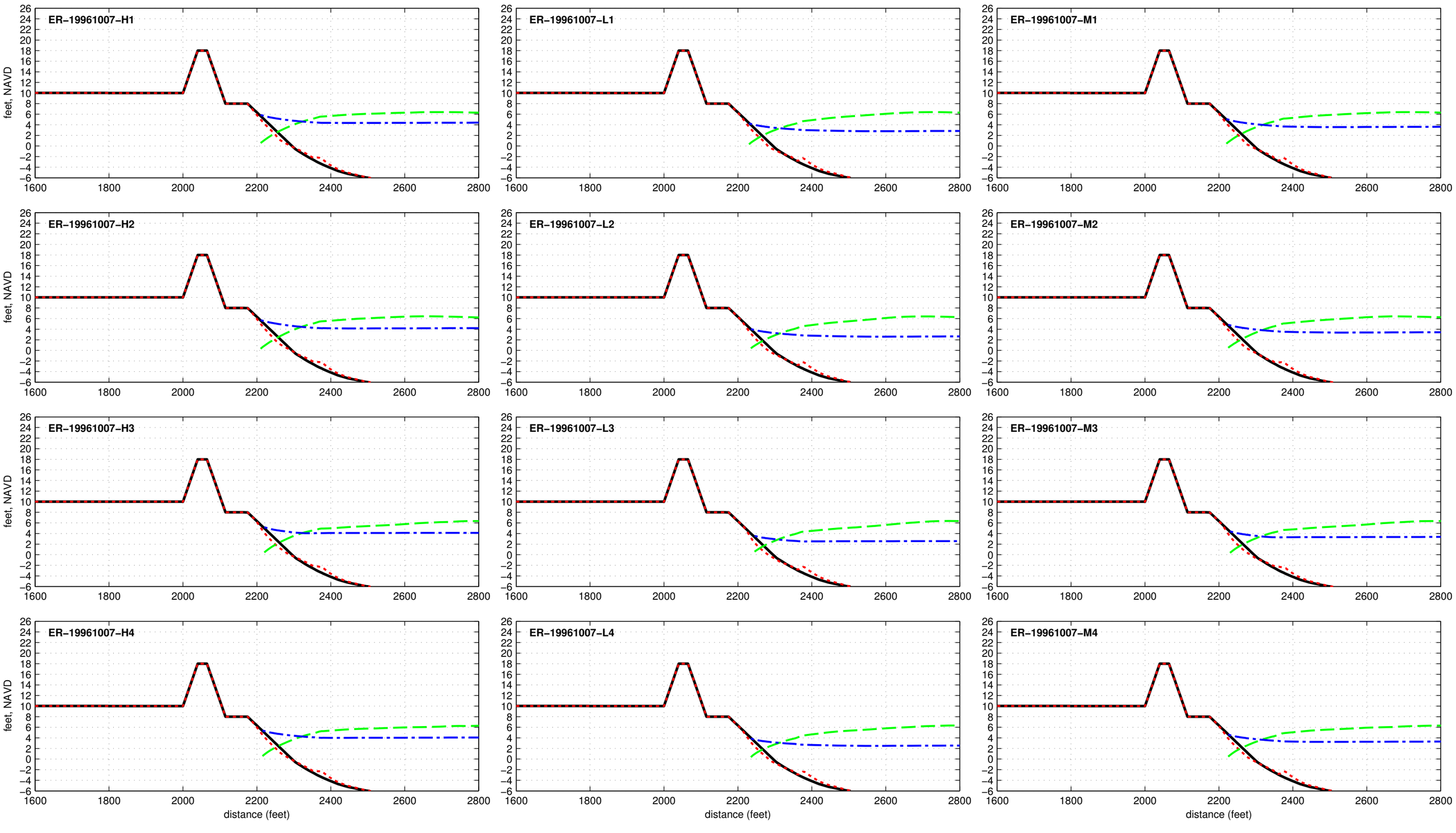

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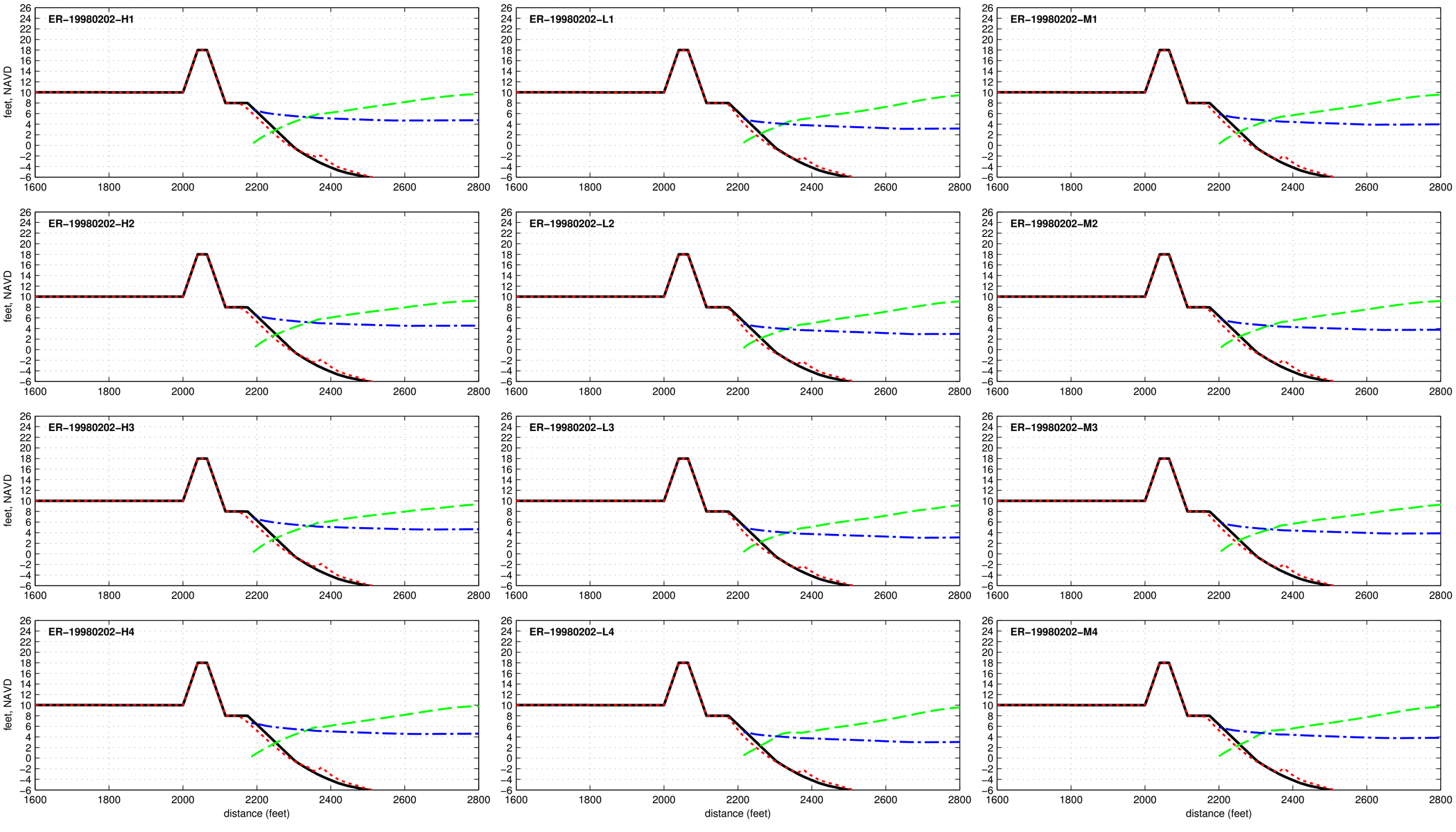

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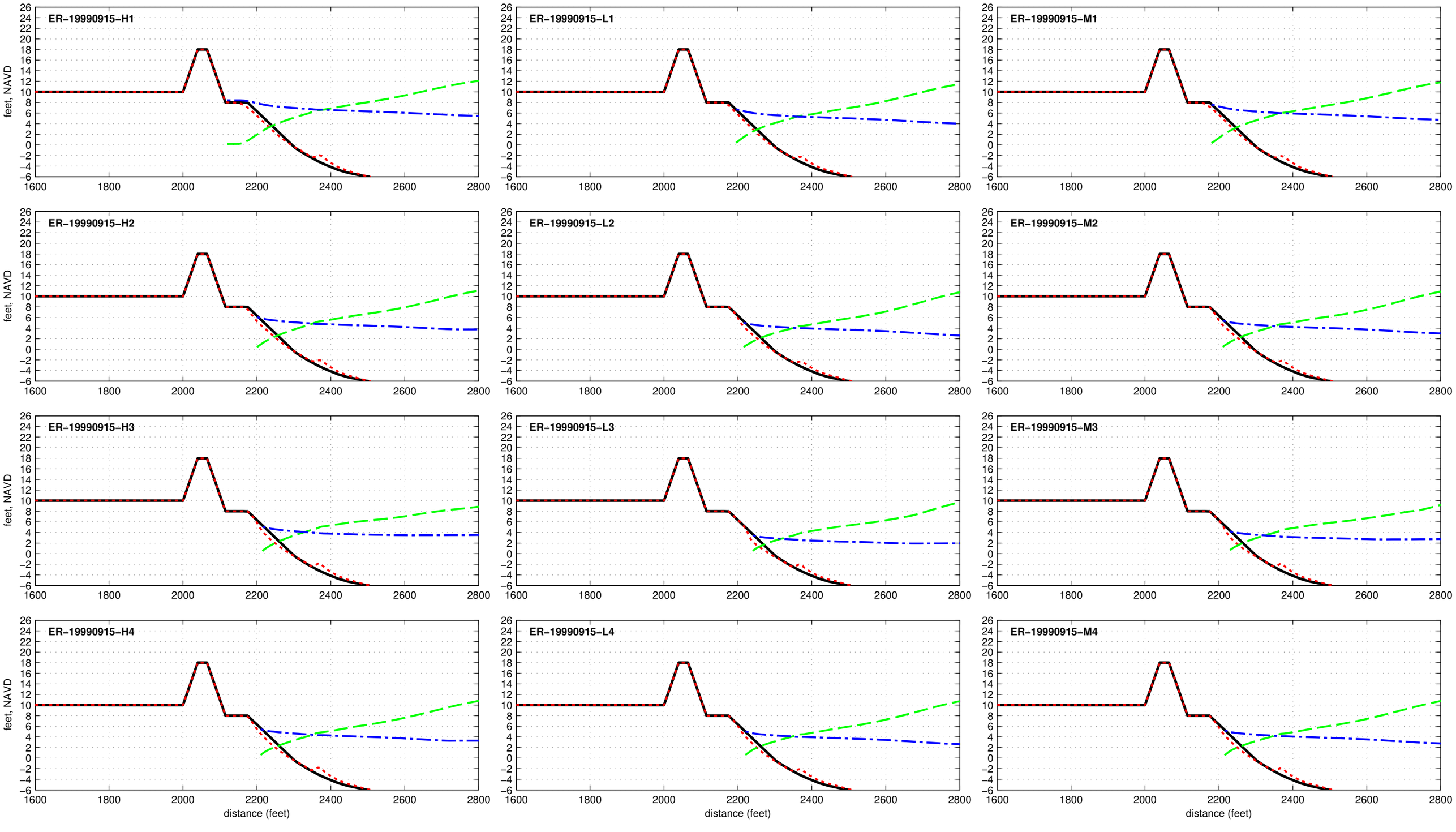

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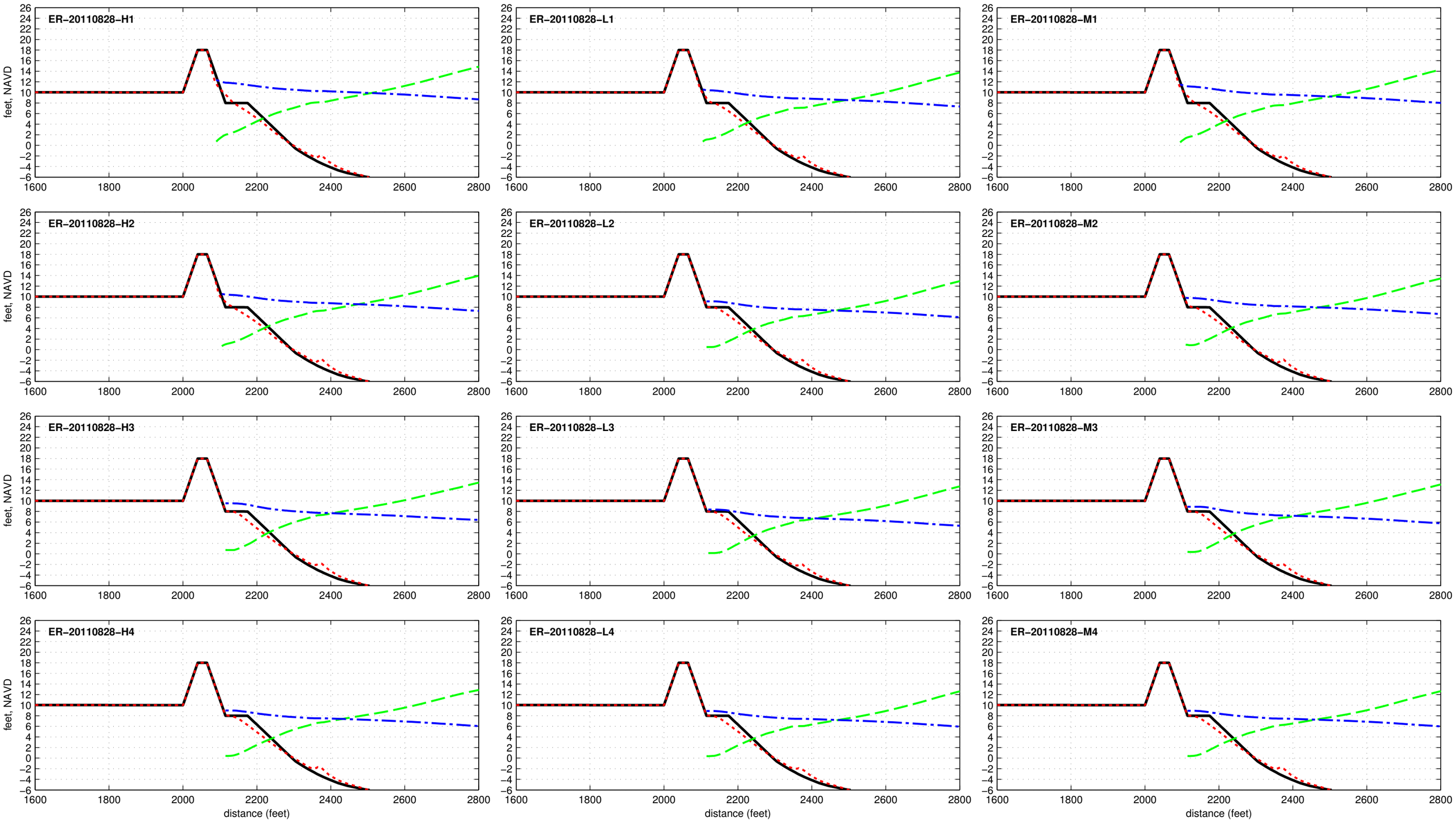

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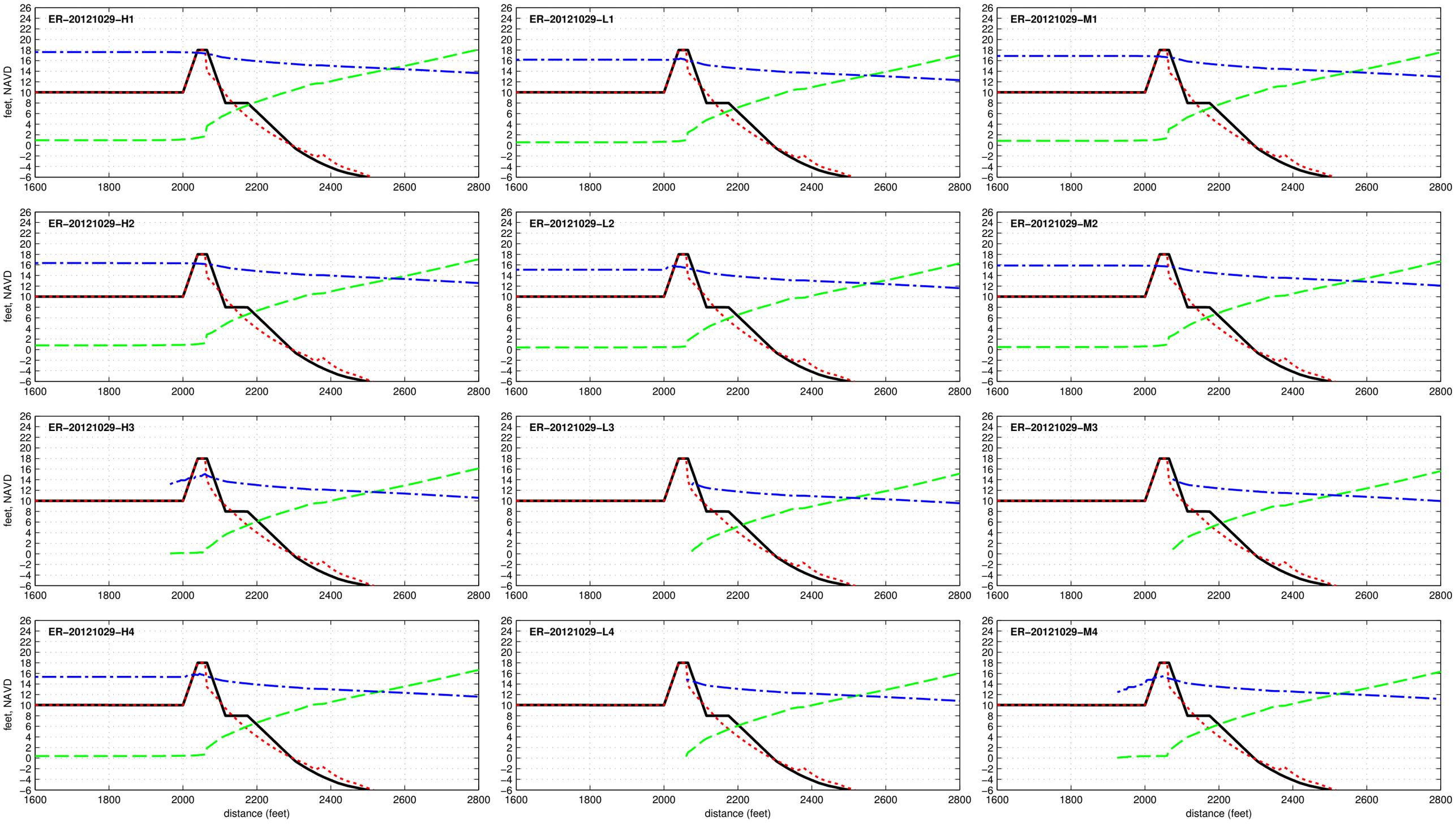
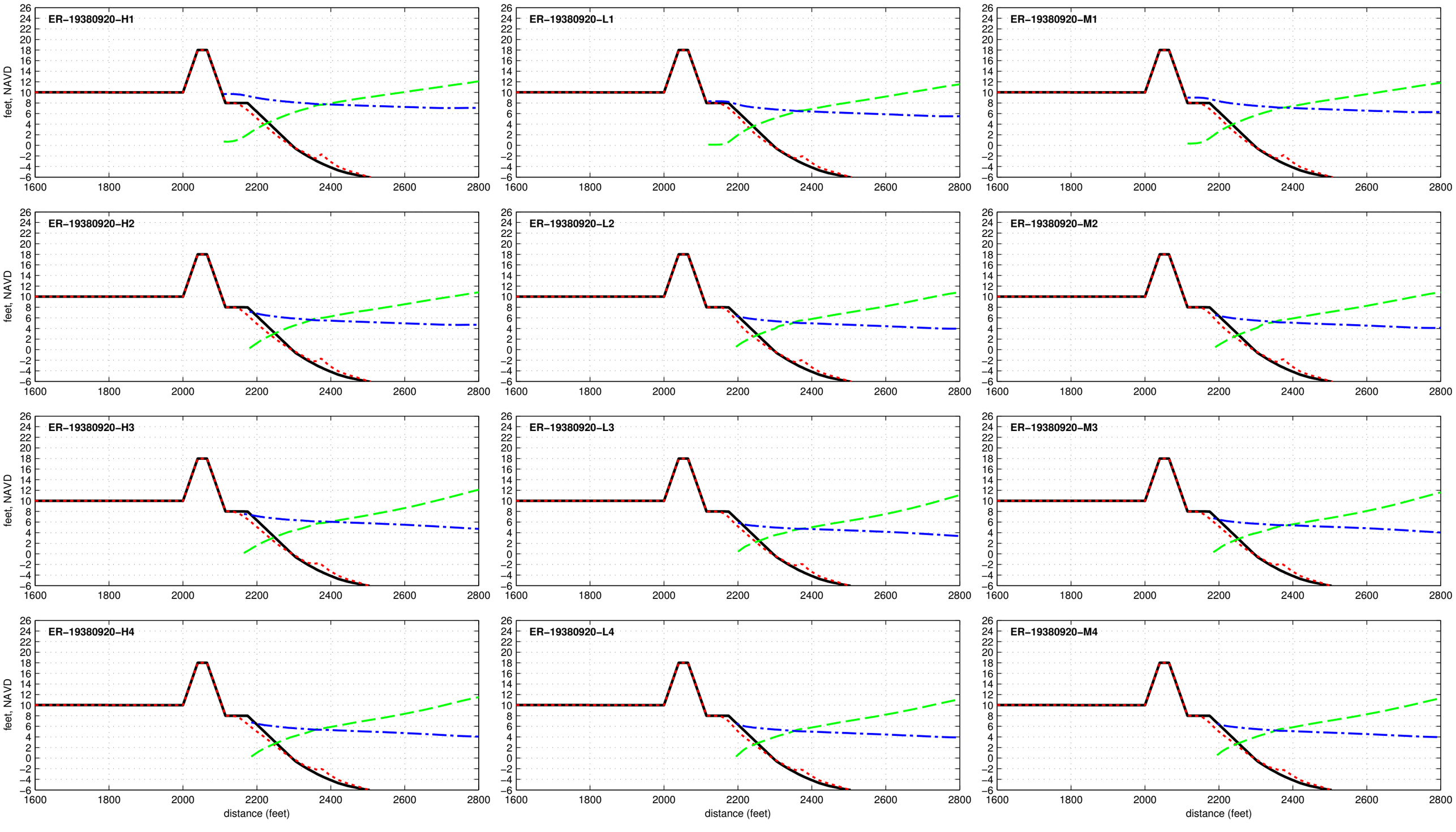


R2T1_18.0_025_060 – COMPOSITE SEAWALL

Composite Seawall R2T1-18.0-025-060 - ER-19380920



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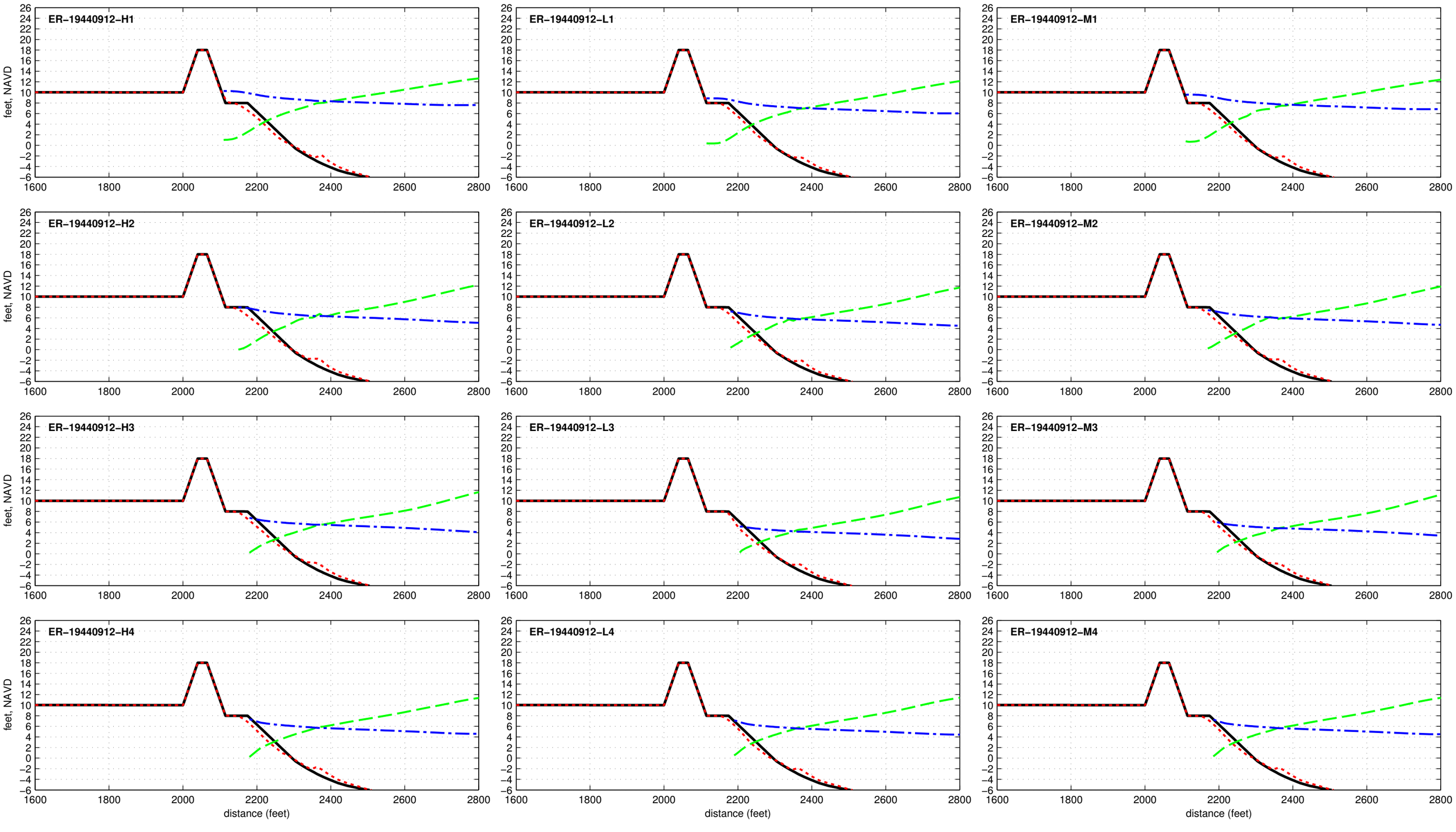

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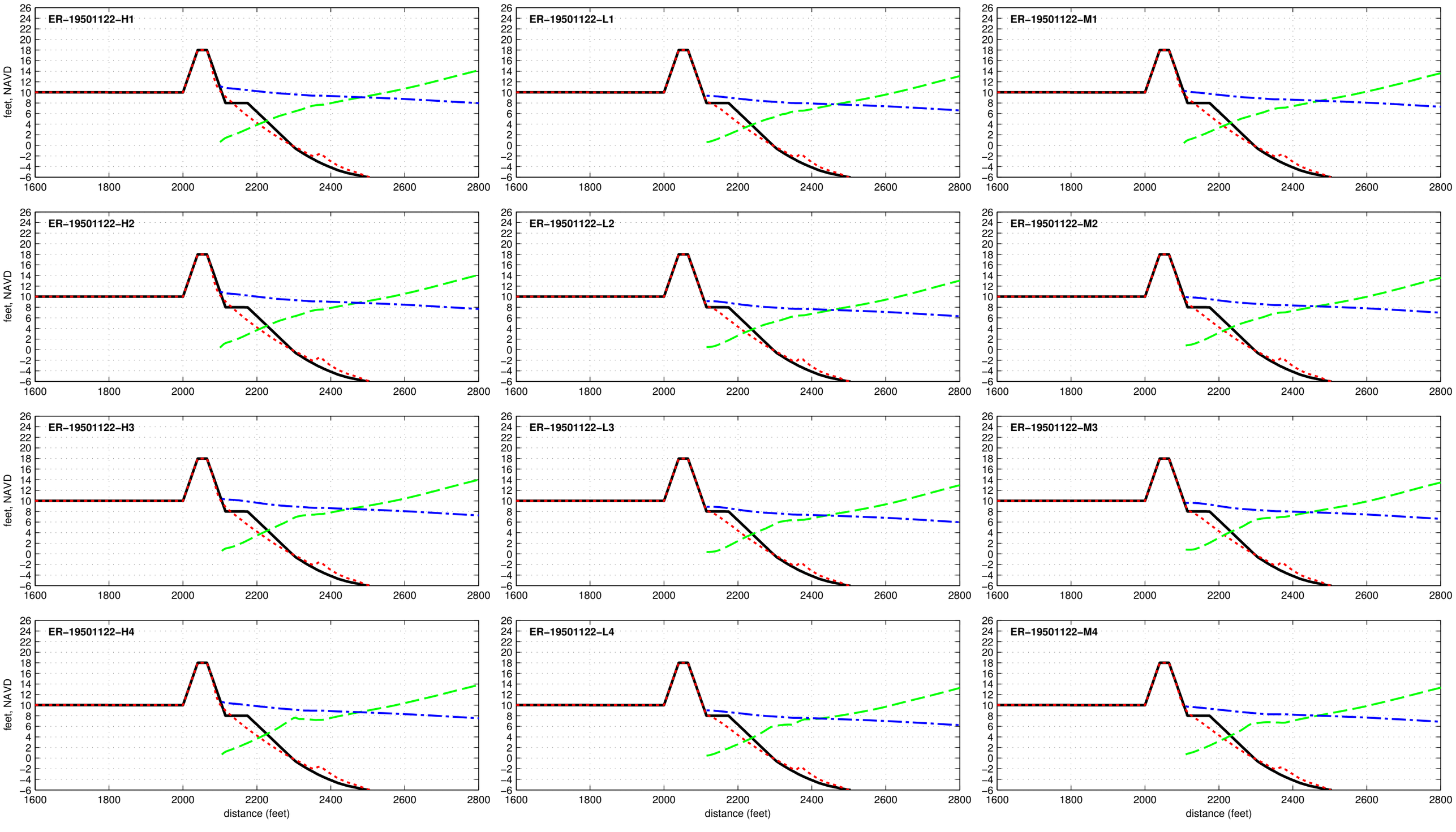

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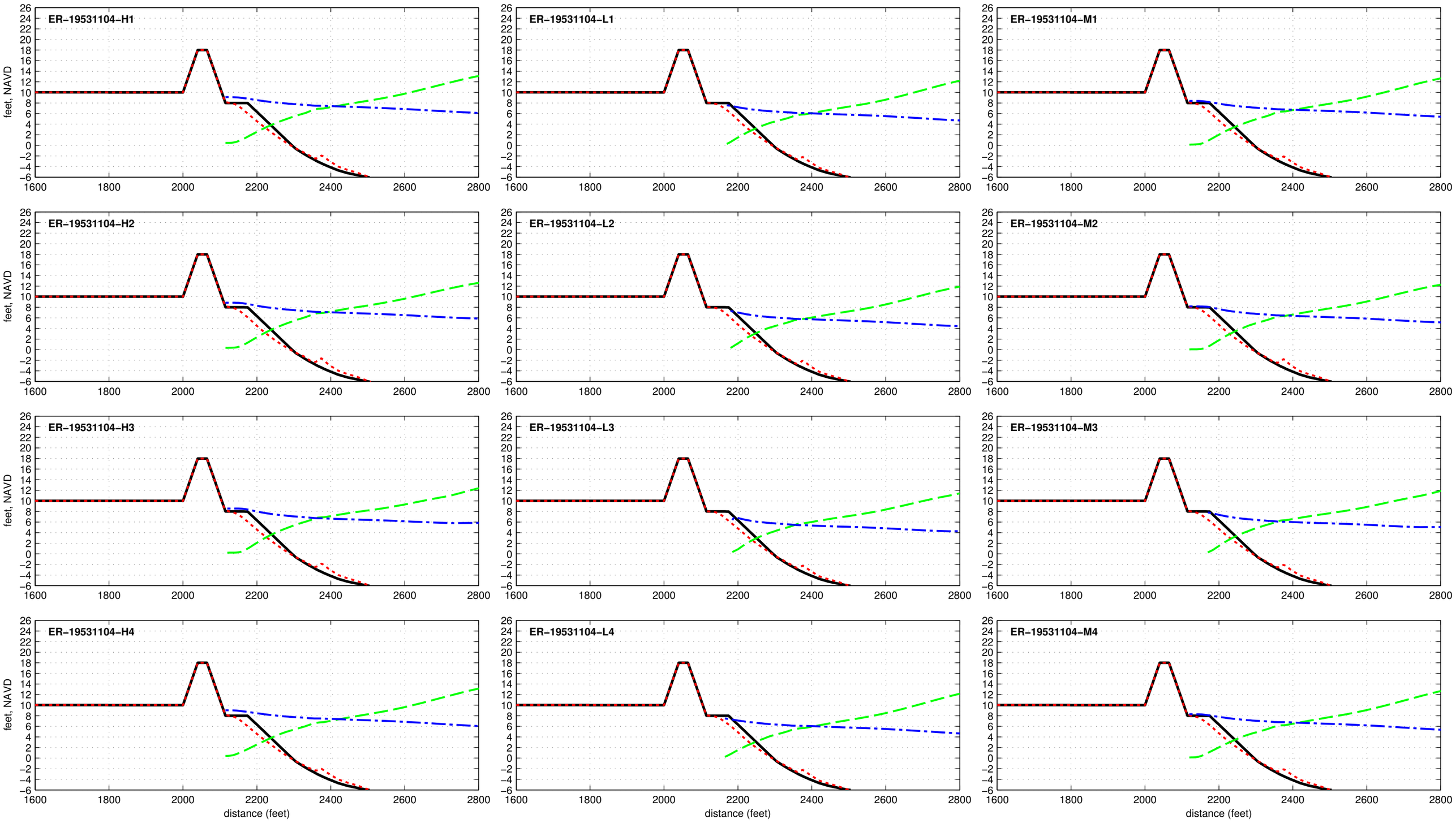

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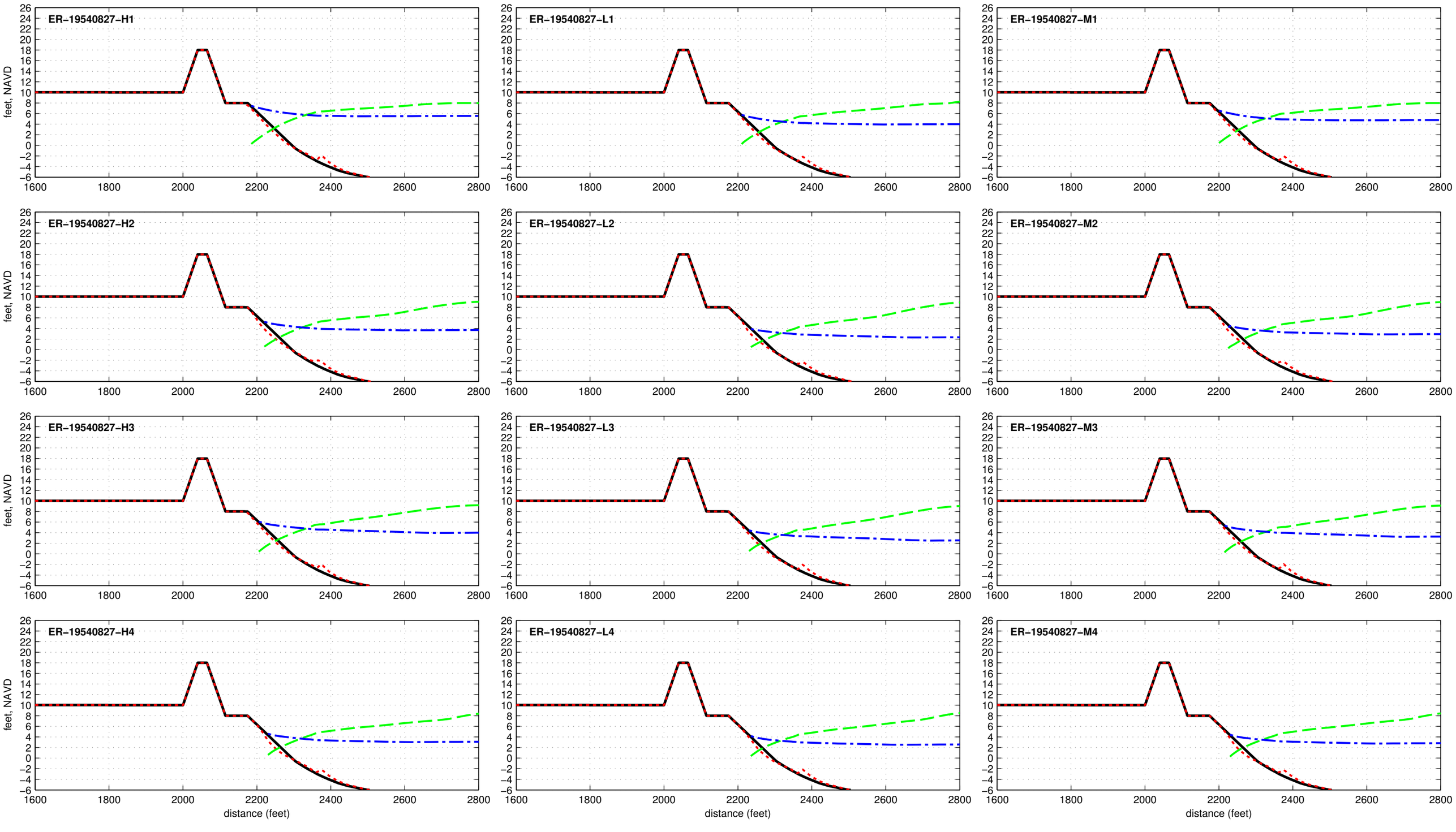

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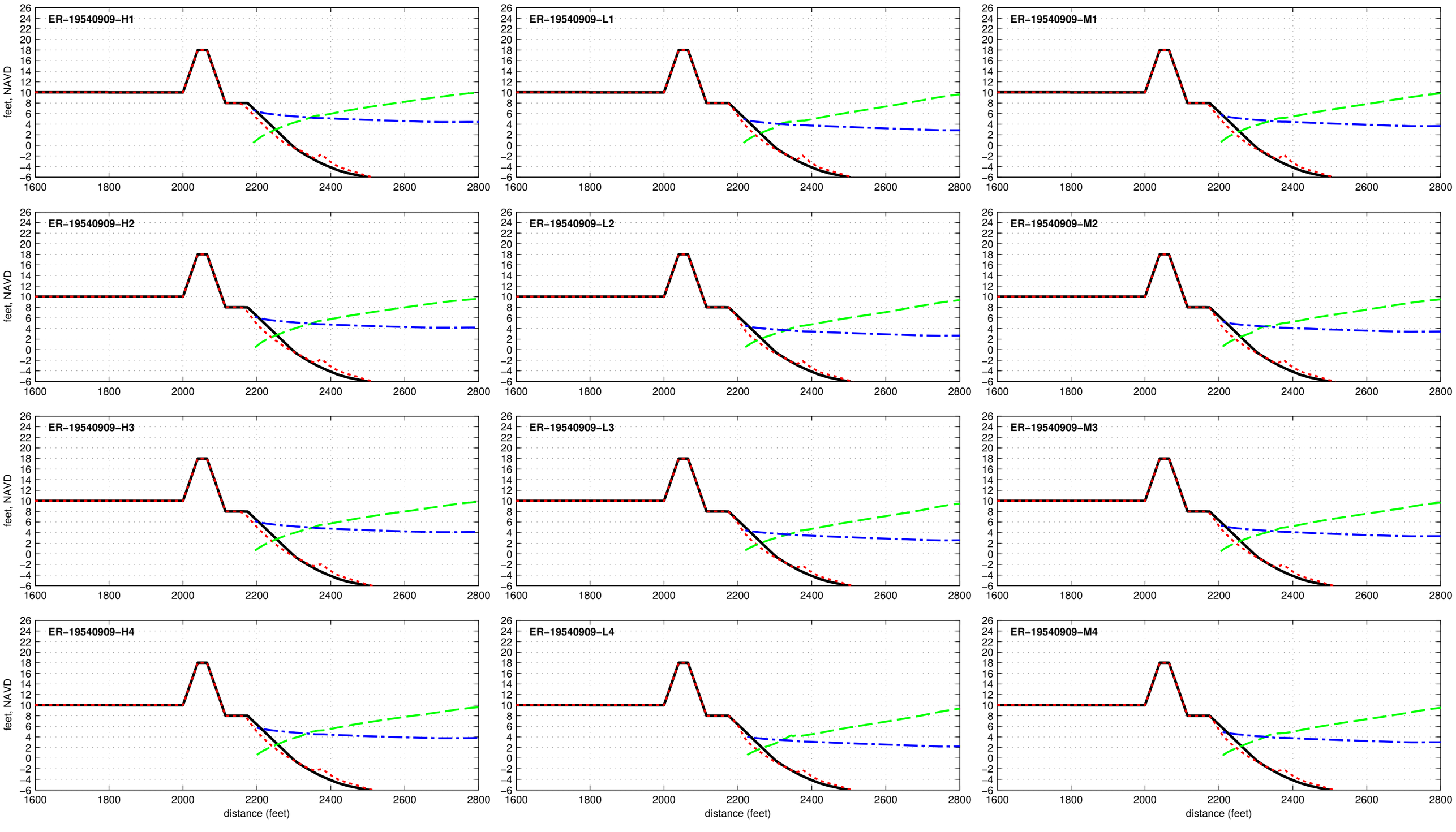

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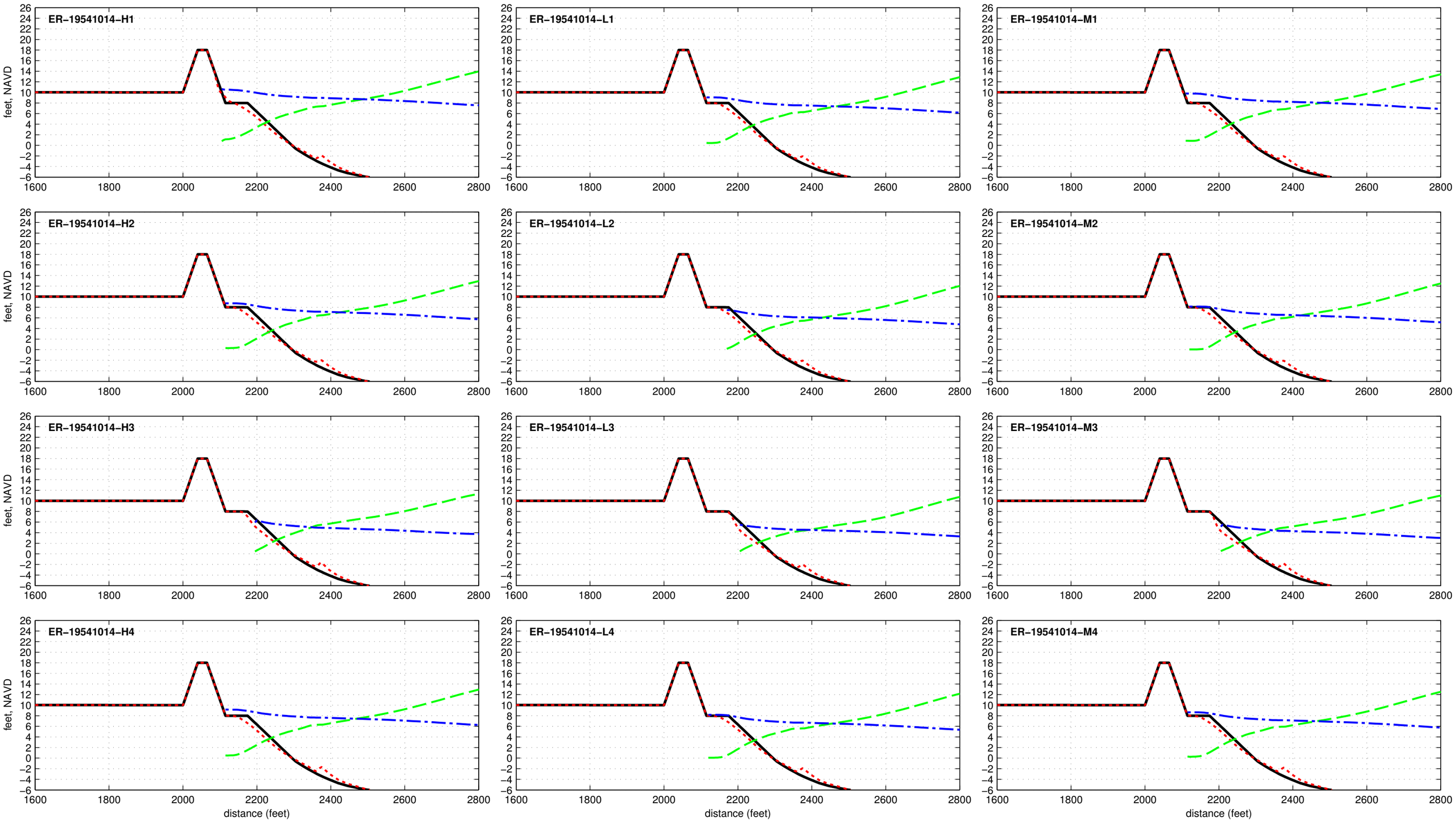

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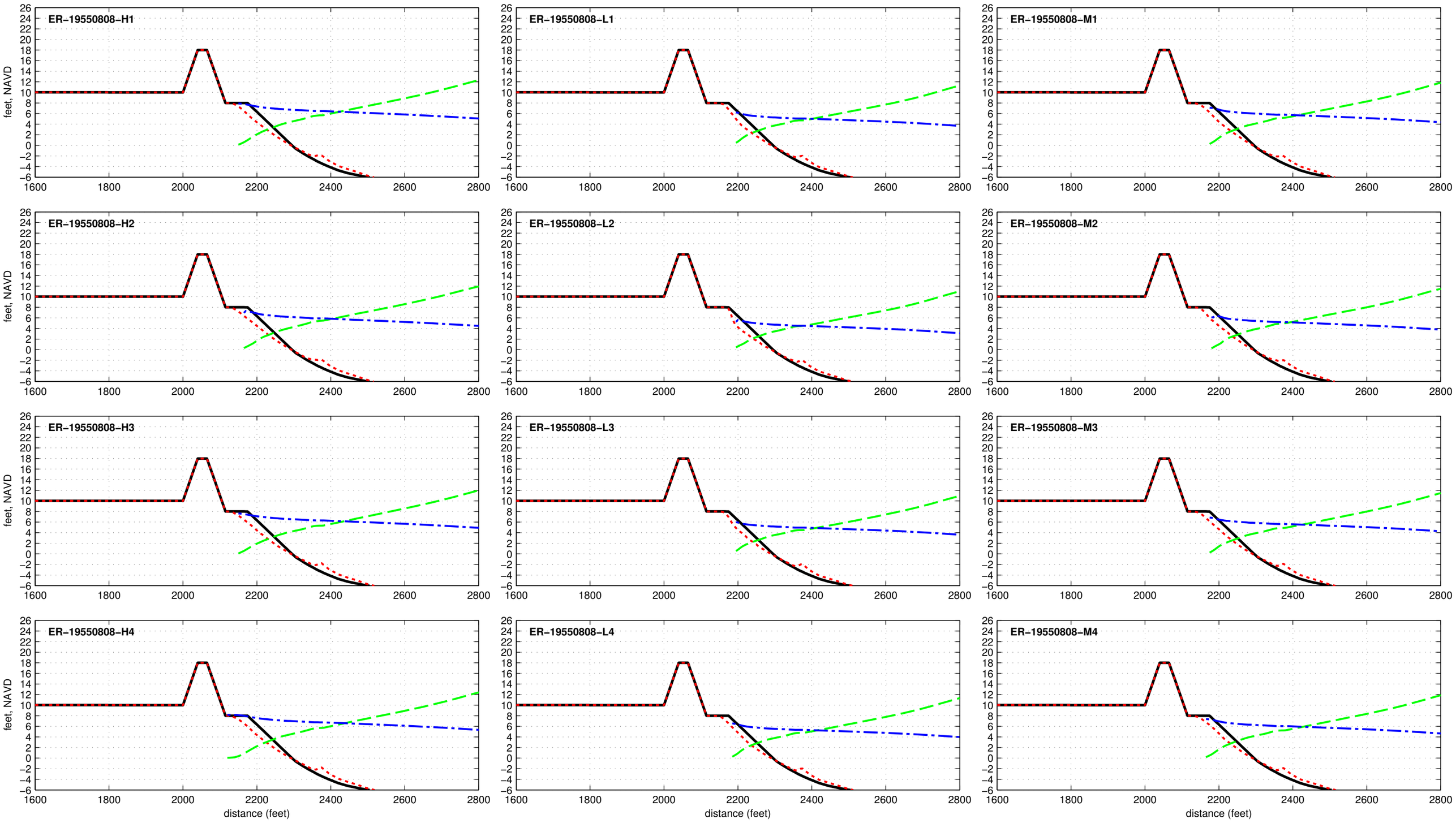

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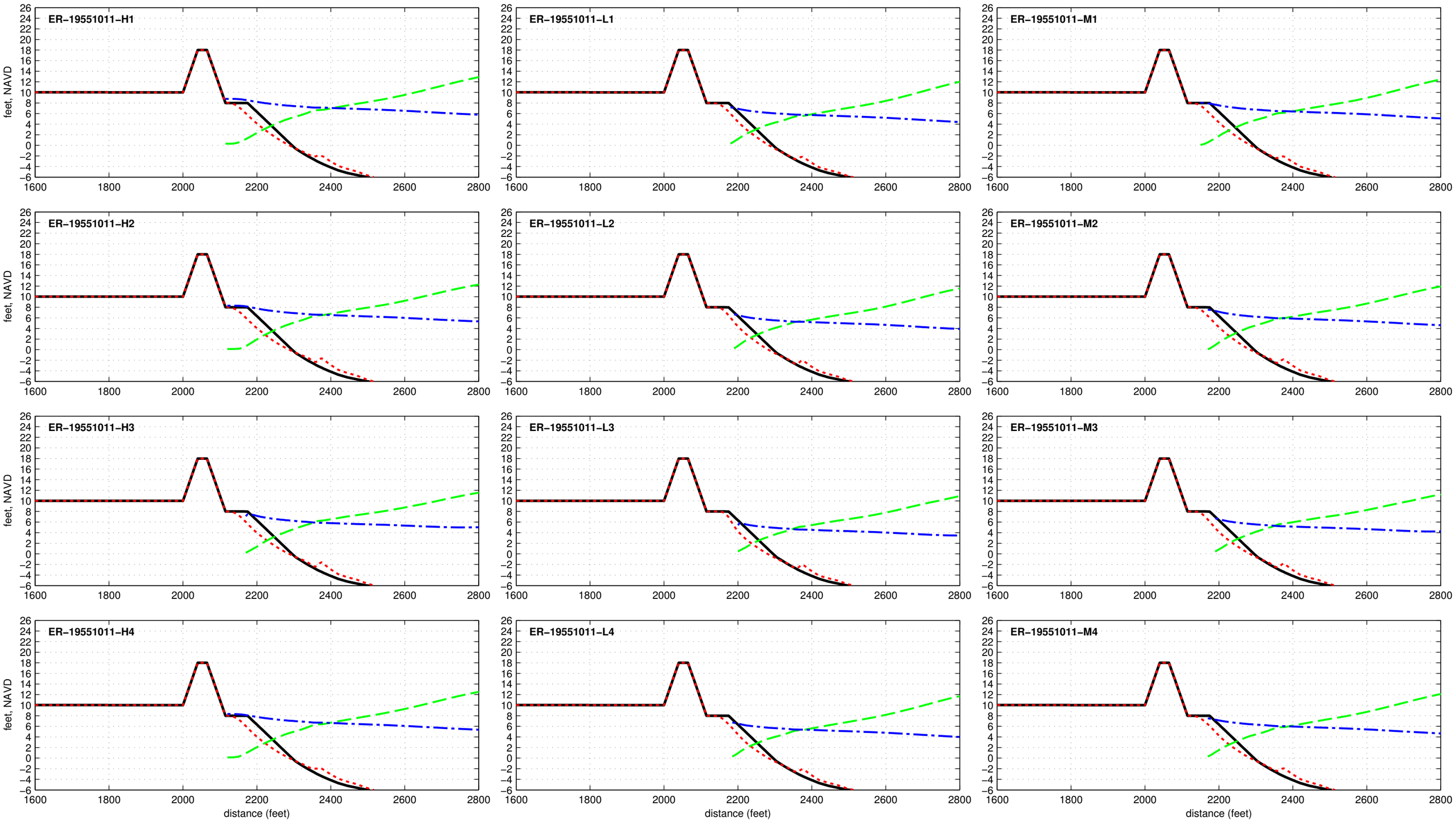

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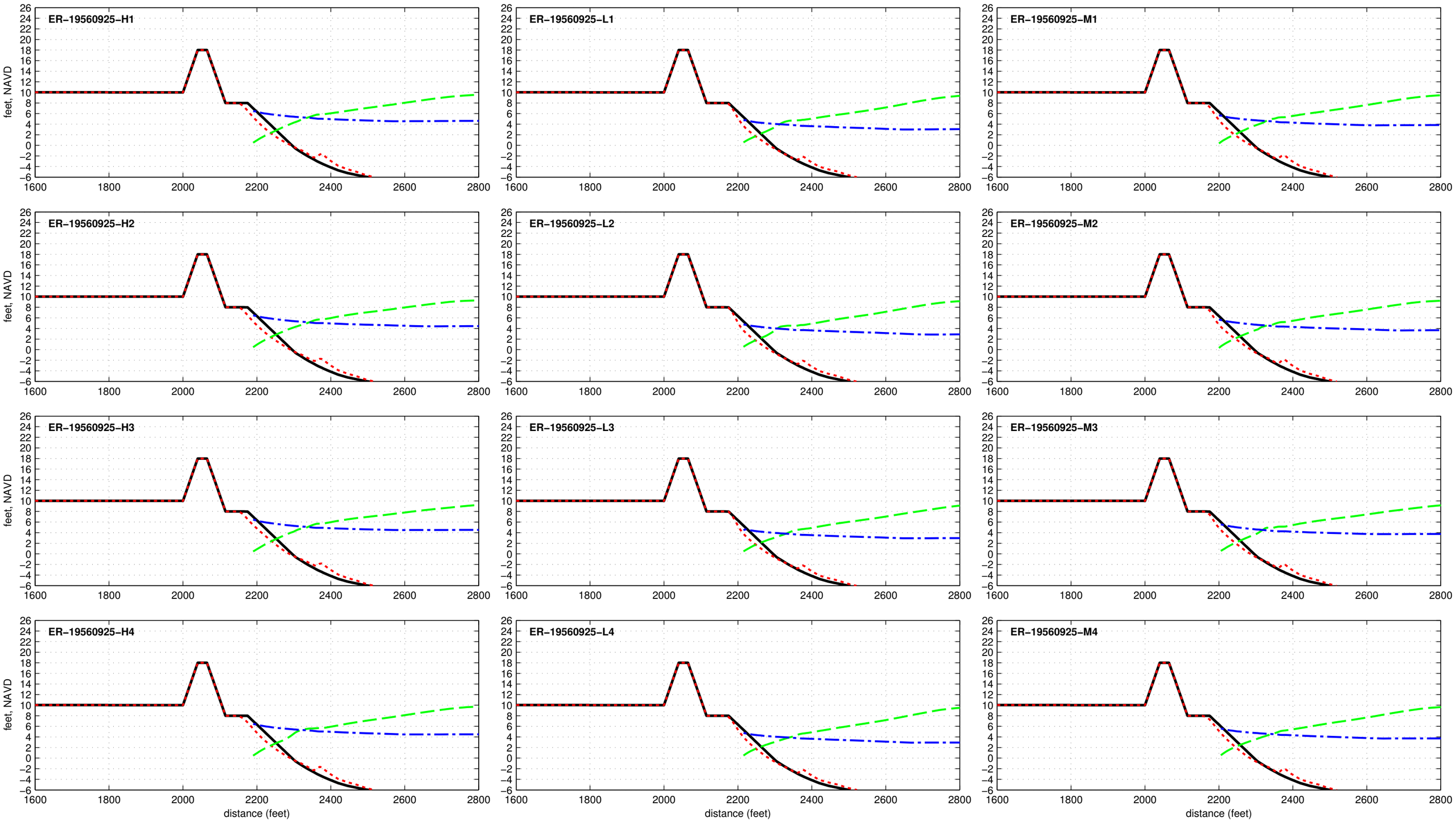

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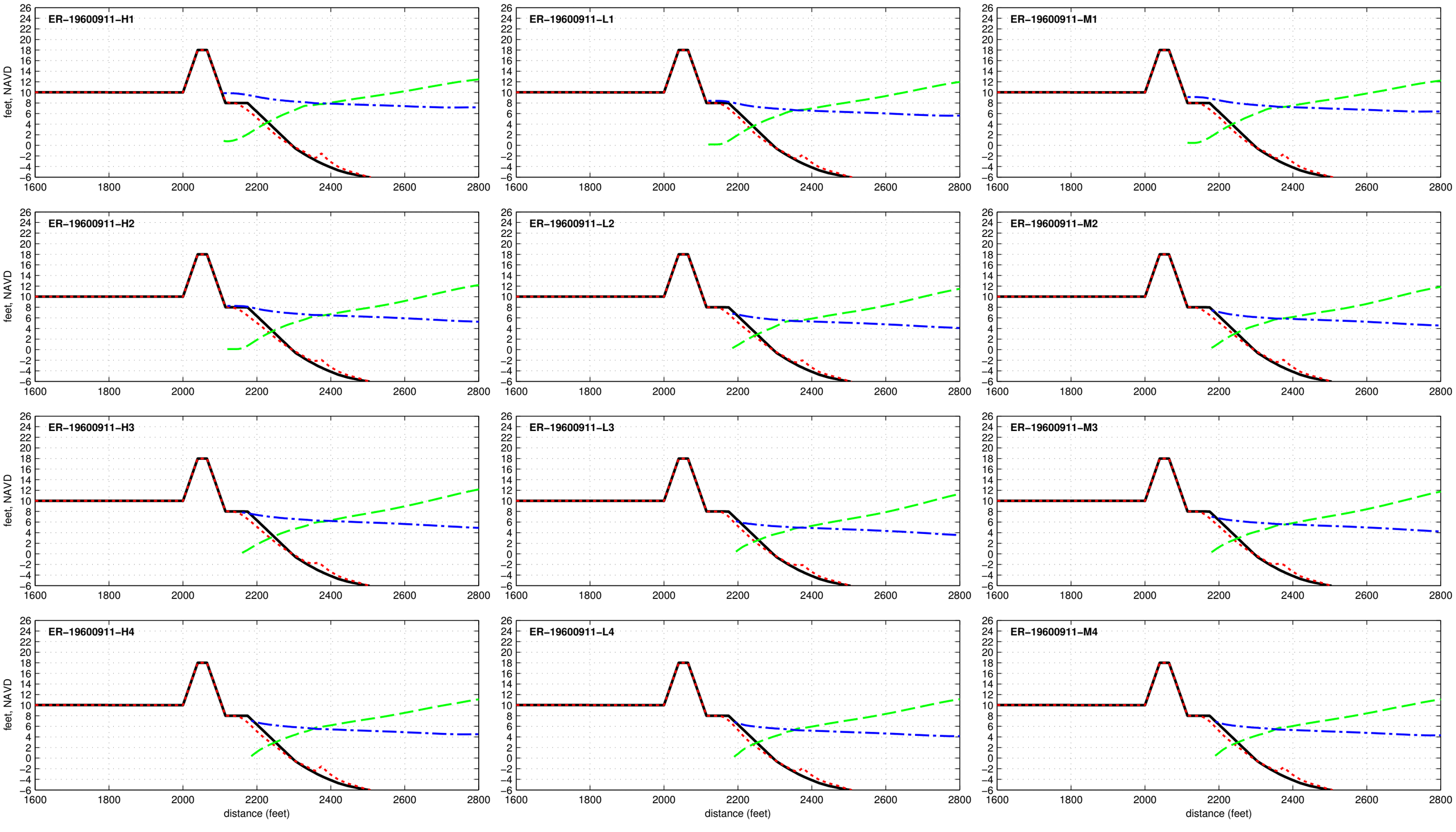

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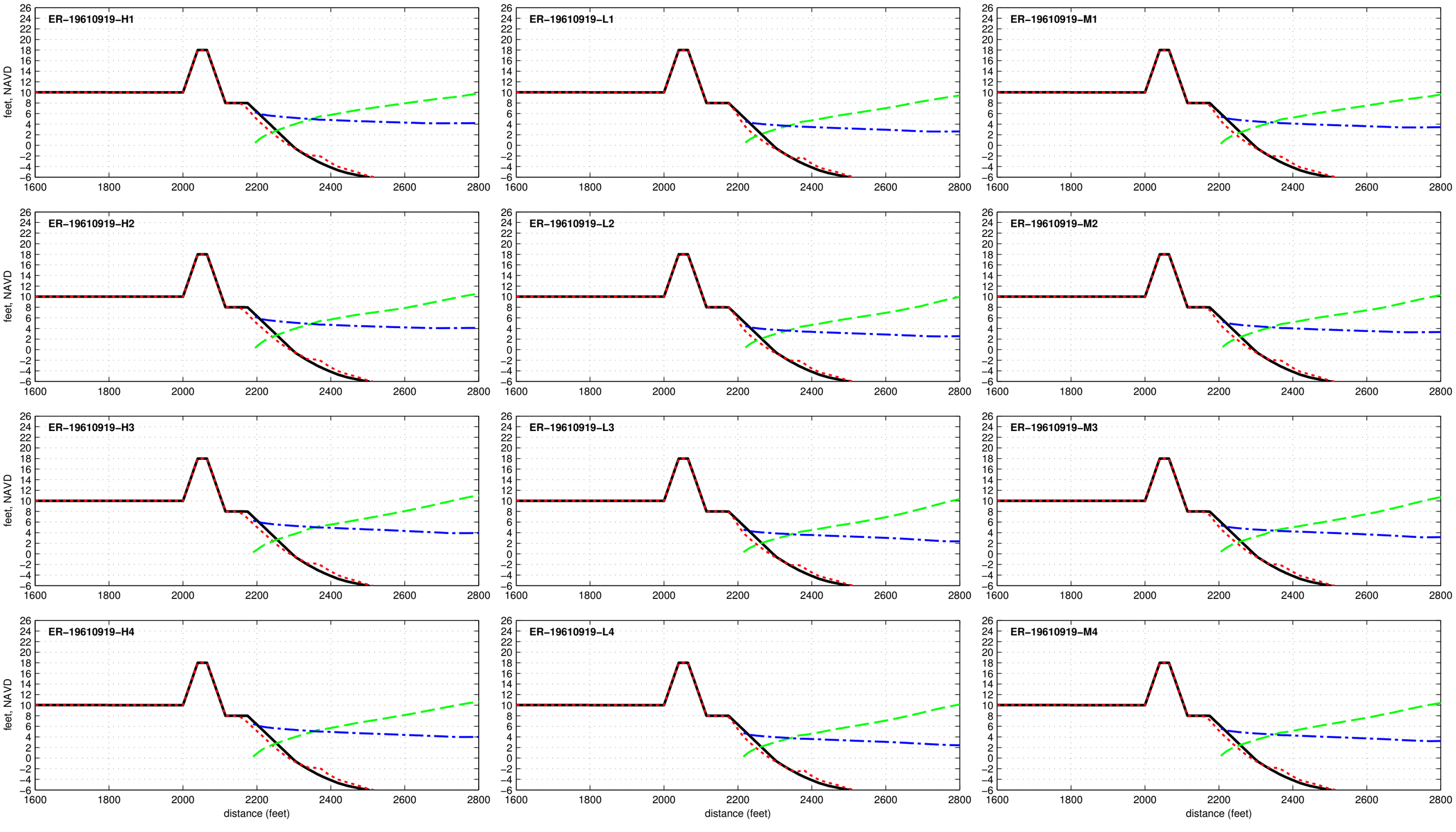
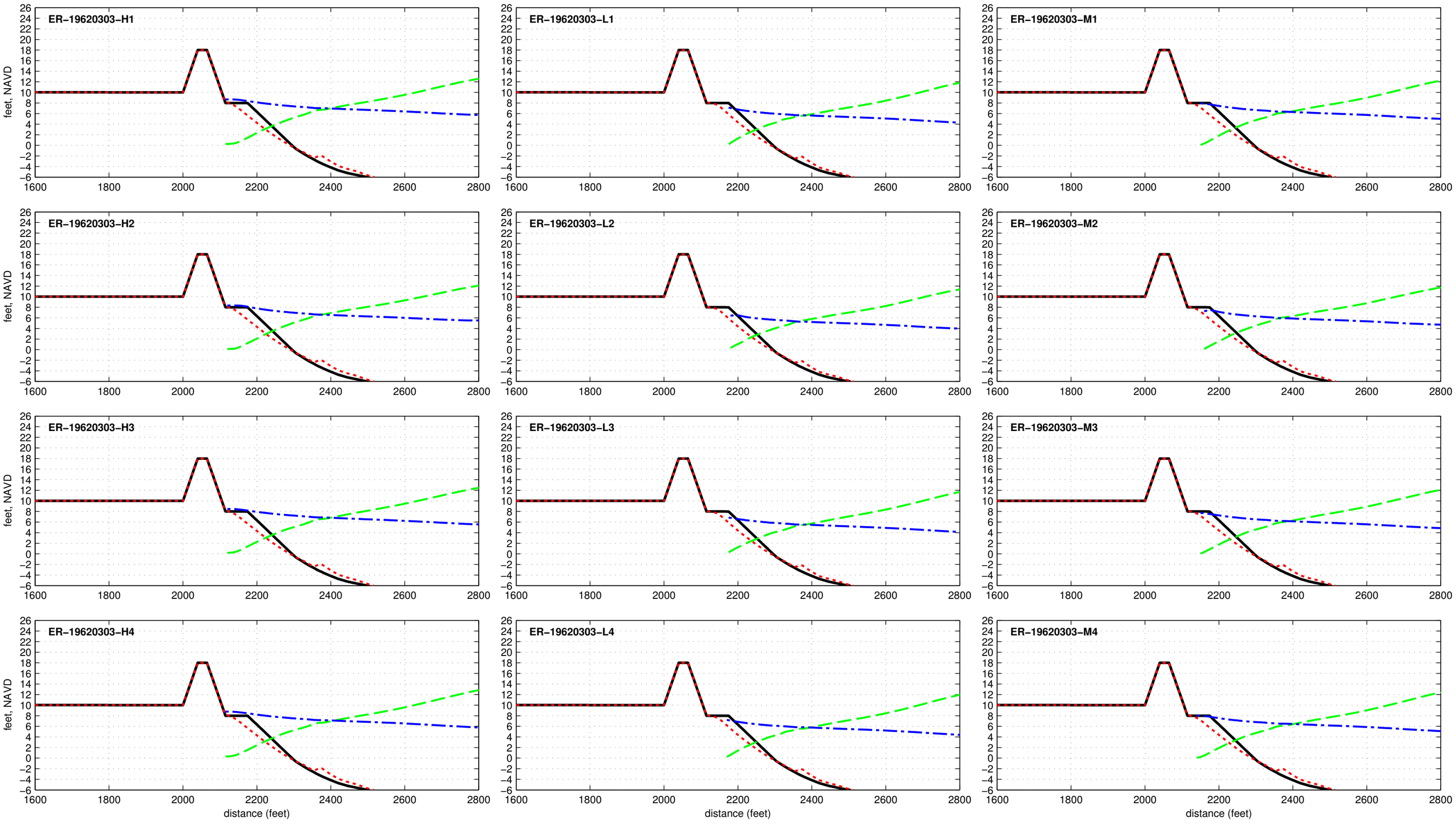


Composite Seawall R2T1-18.0-025-060 - ER-19620303



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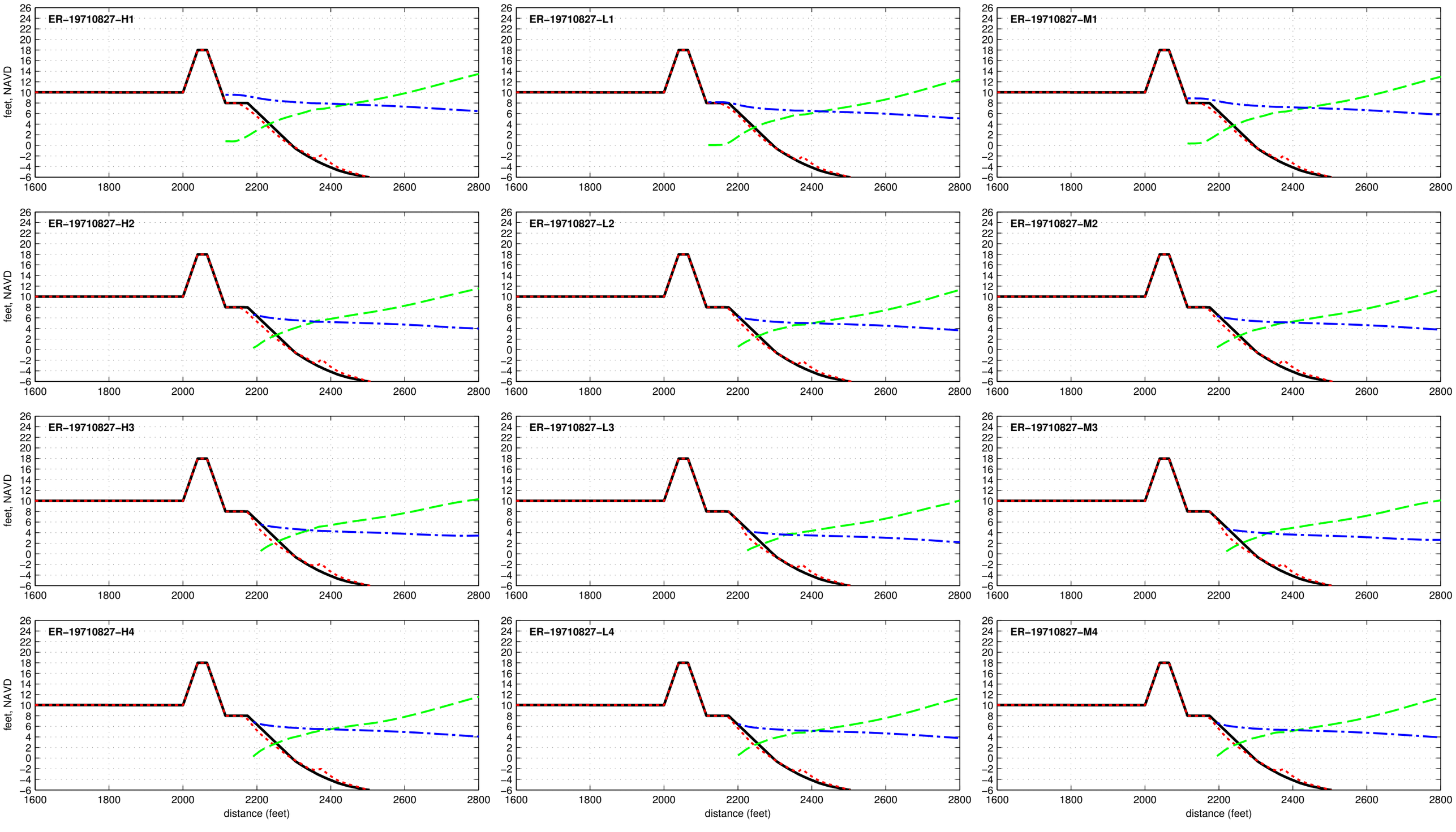

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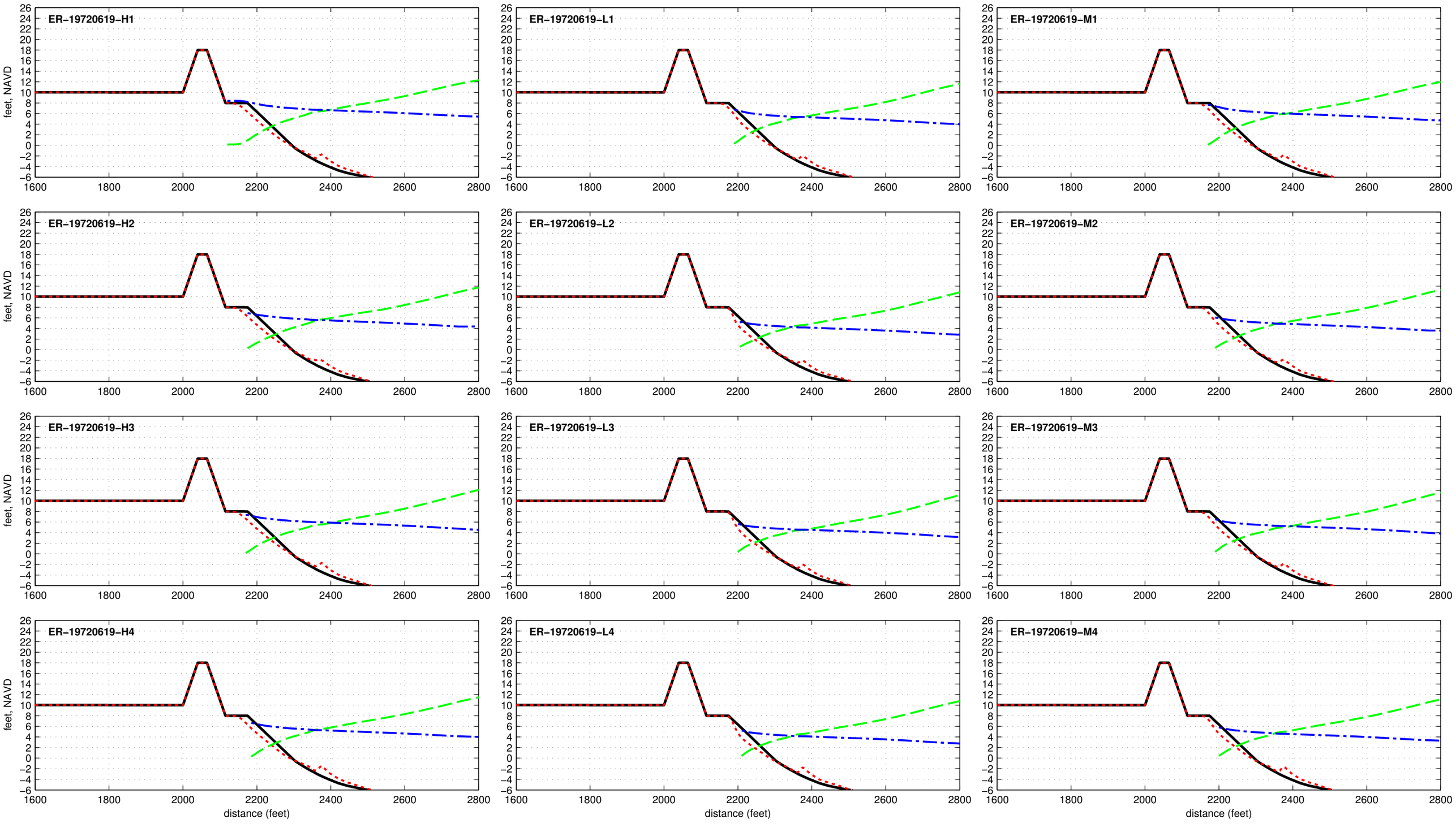

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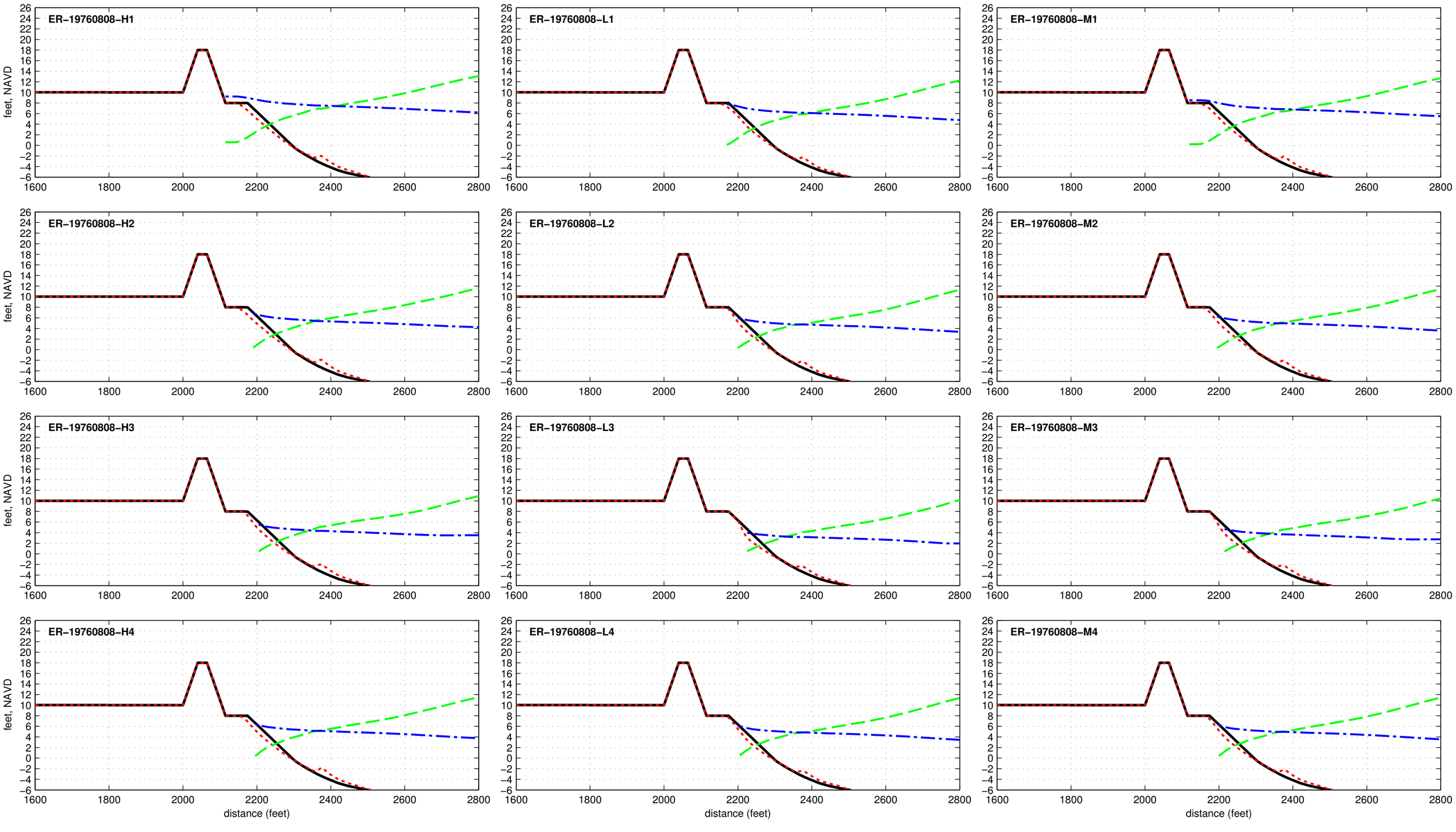

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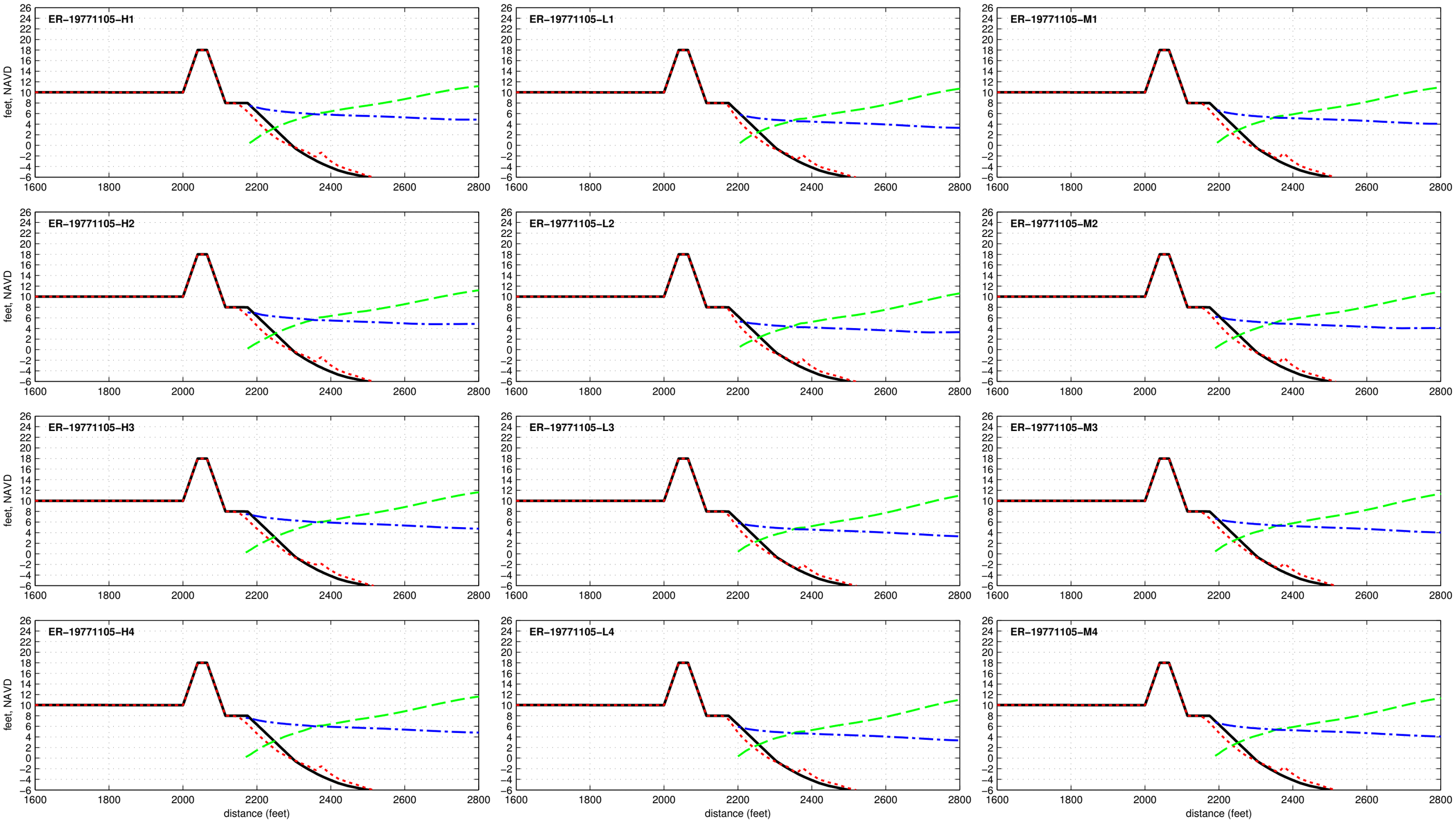

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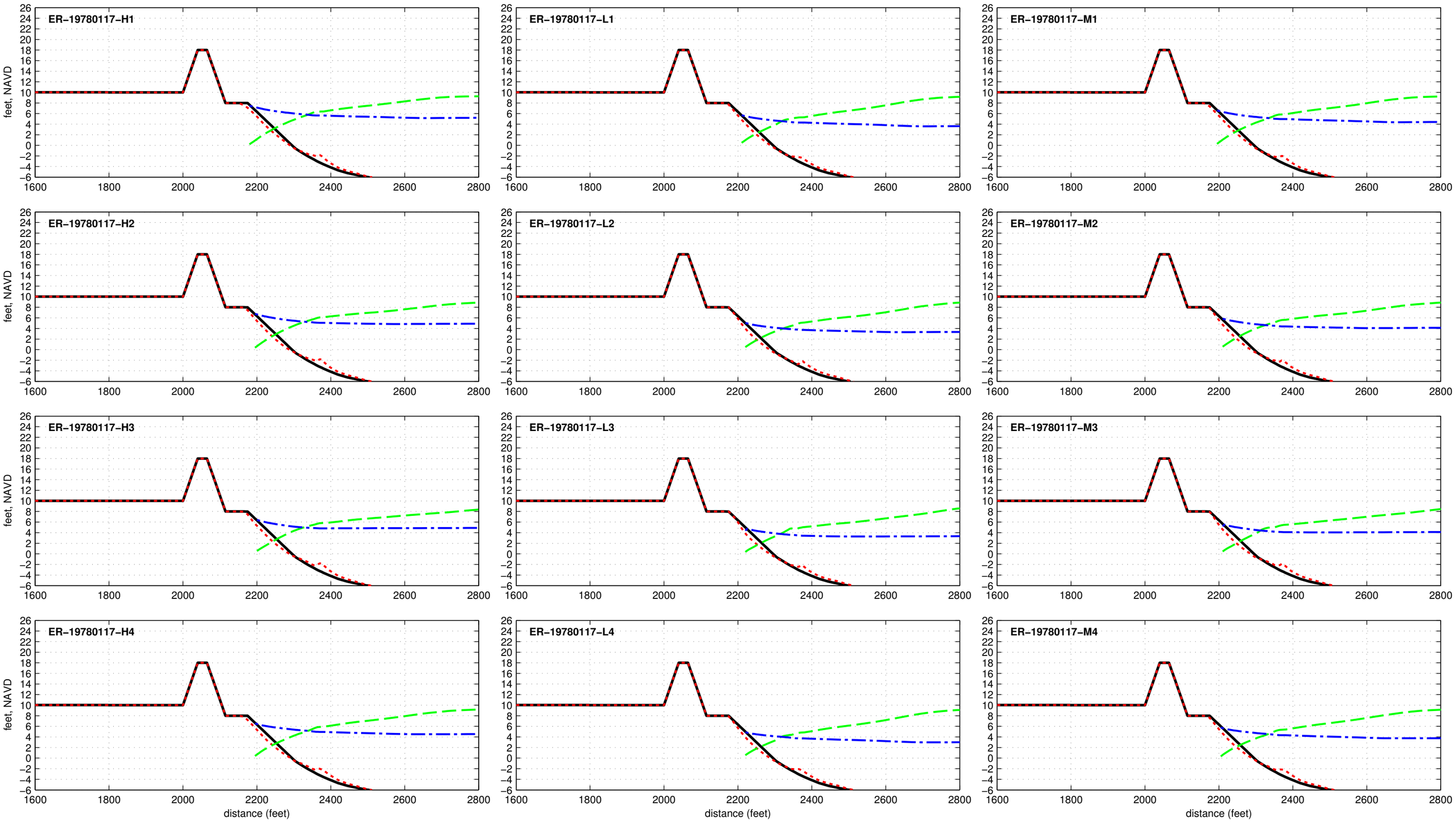

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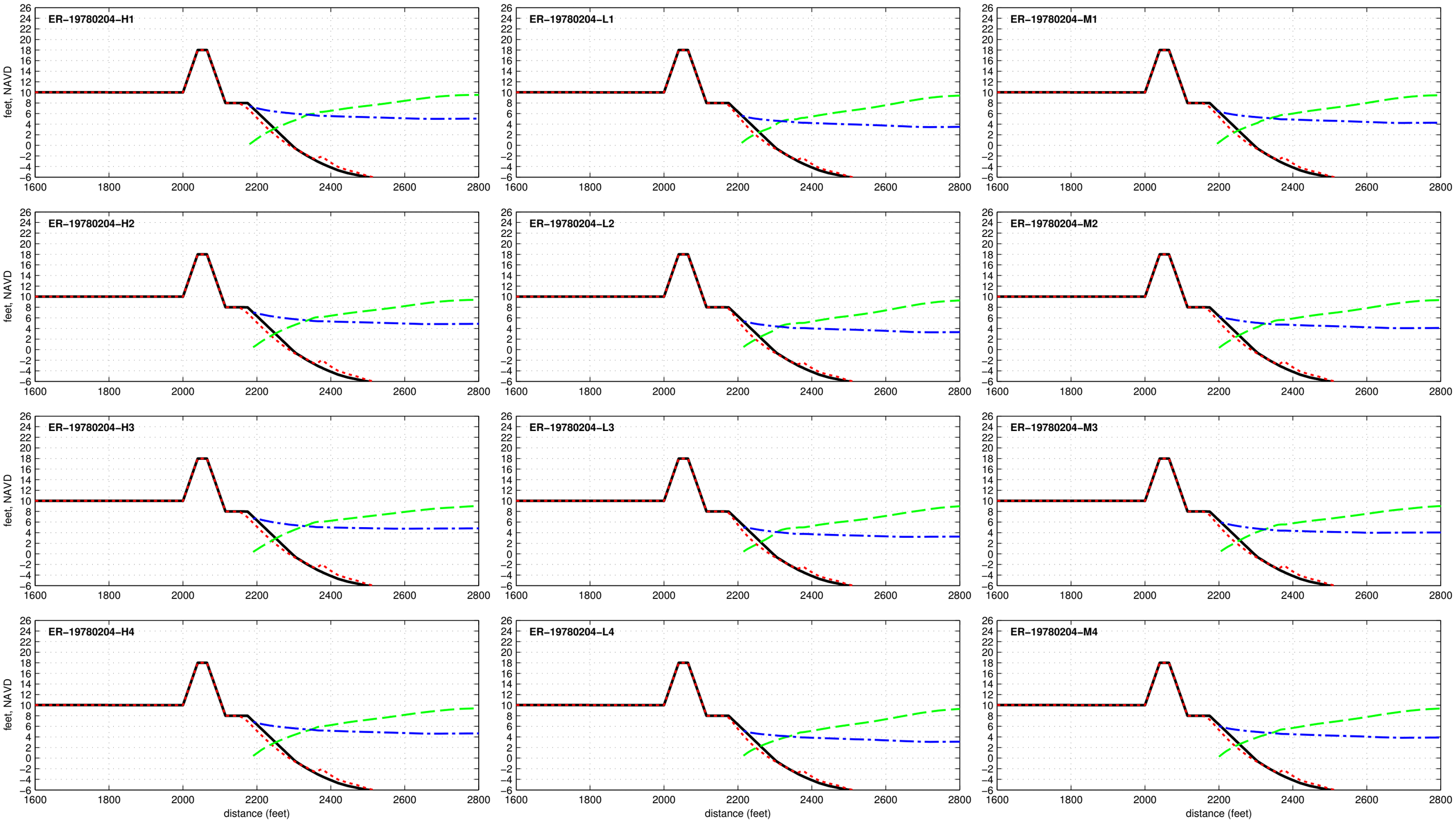

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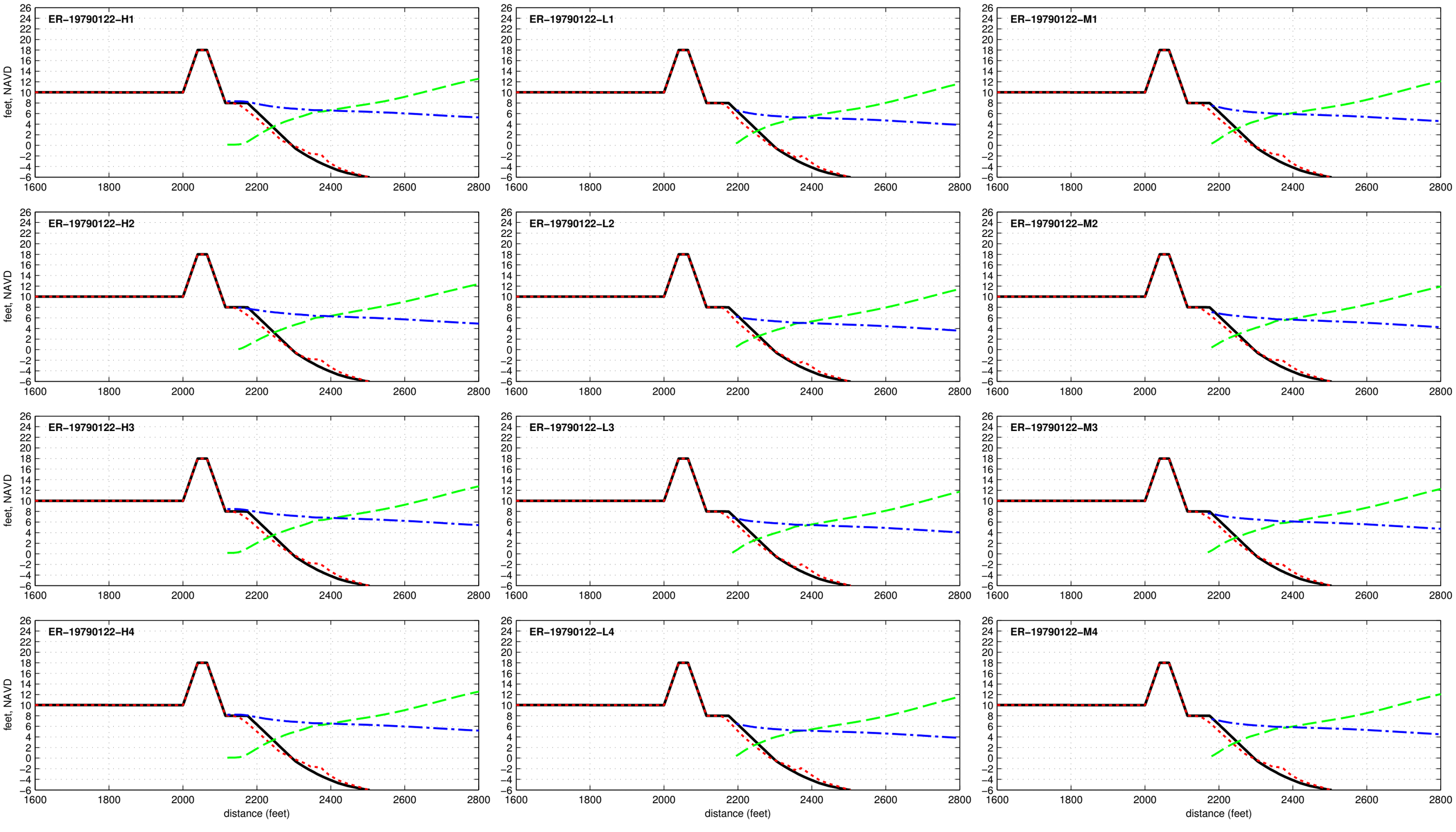

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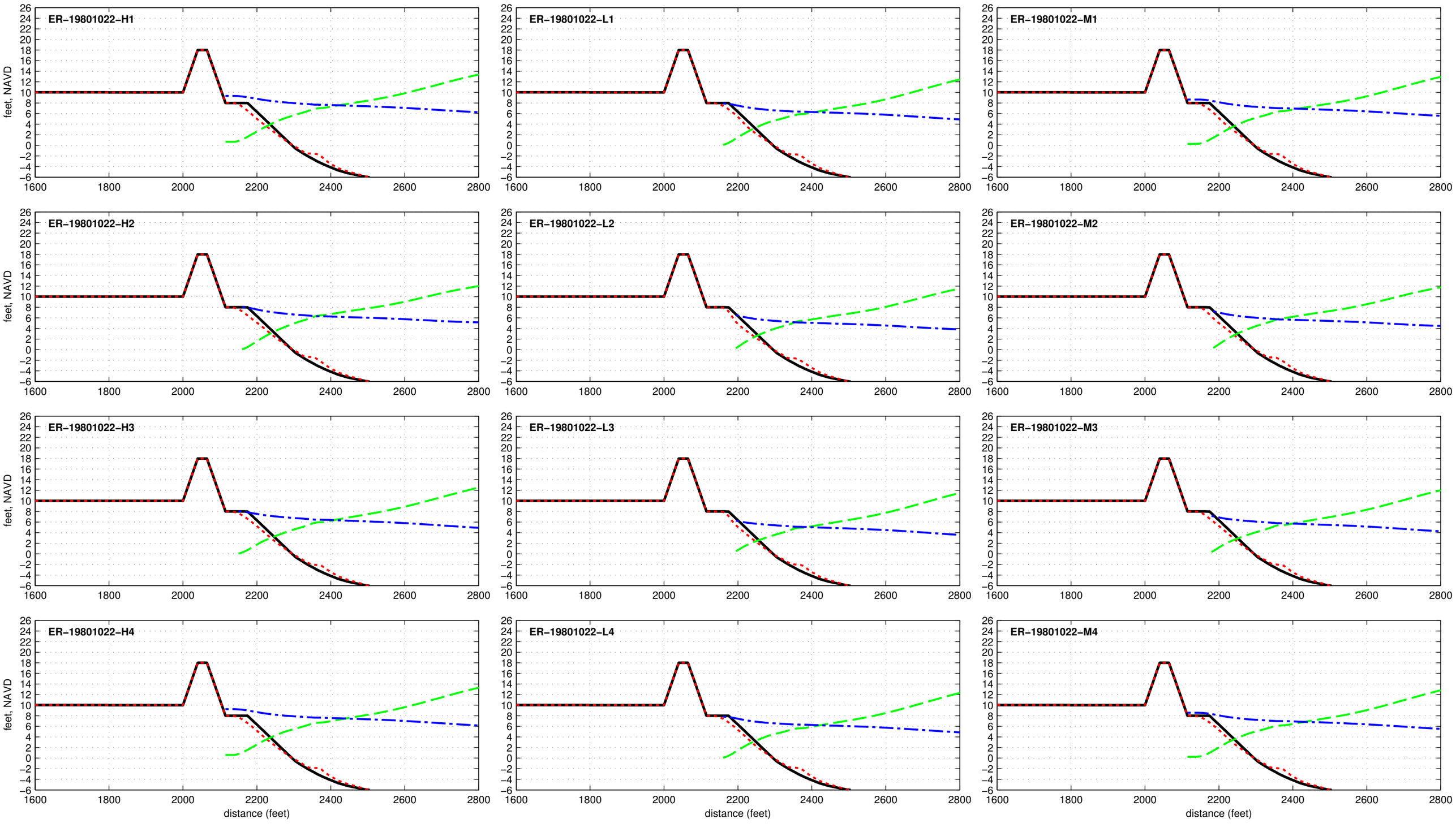

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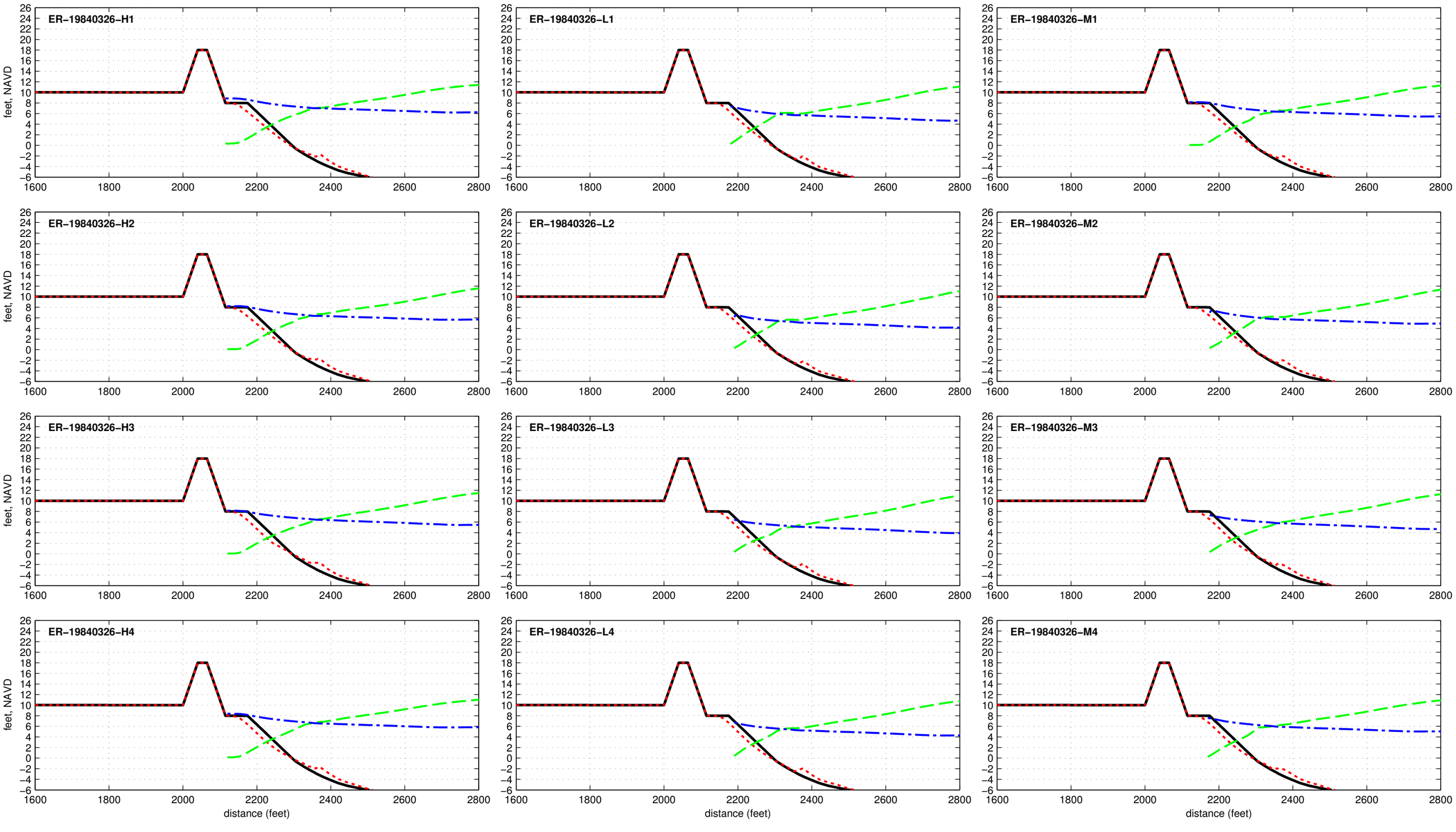

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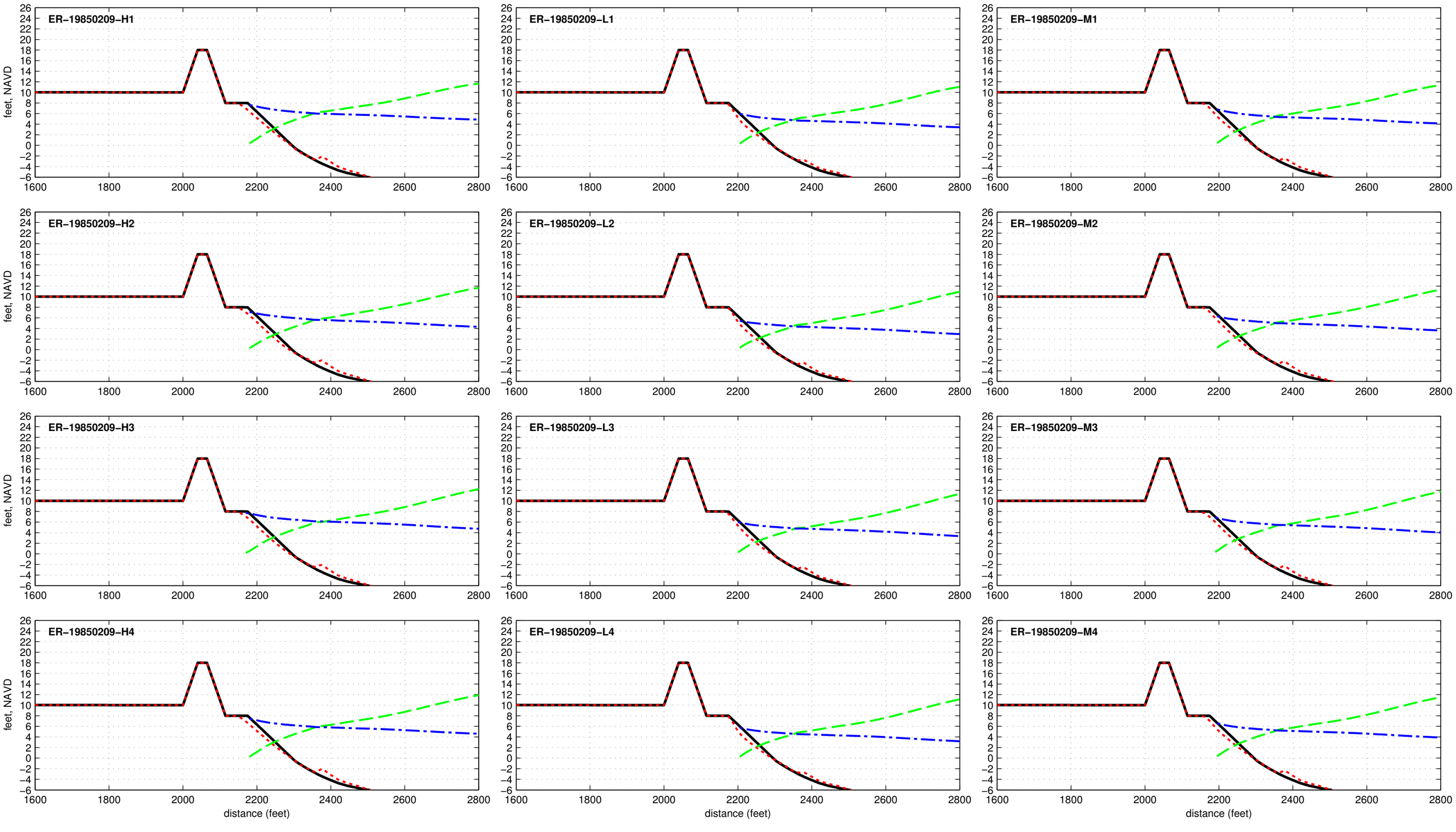

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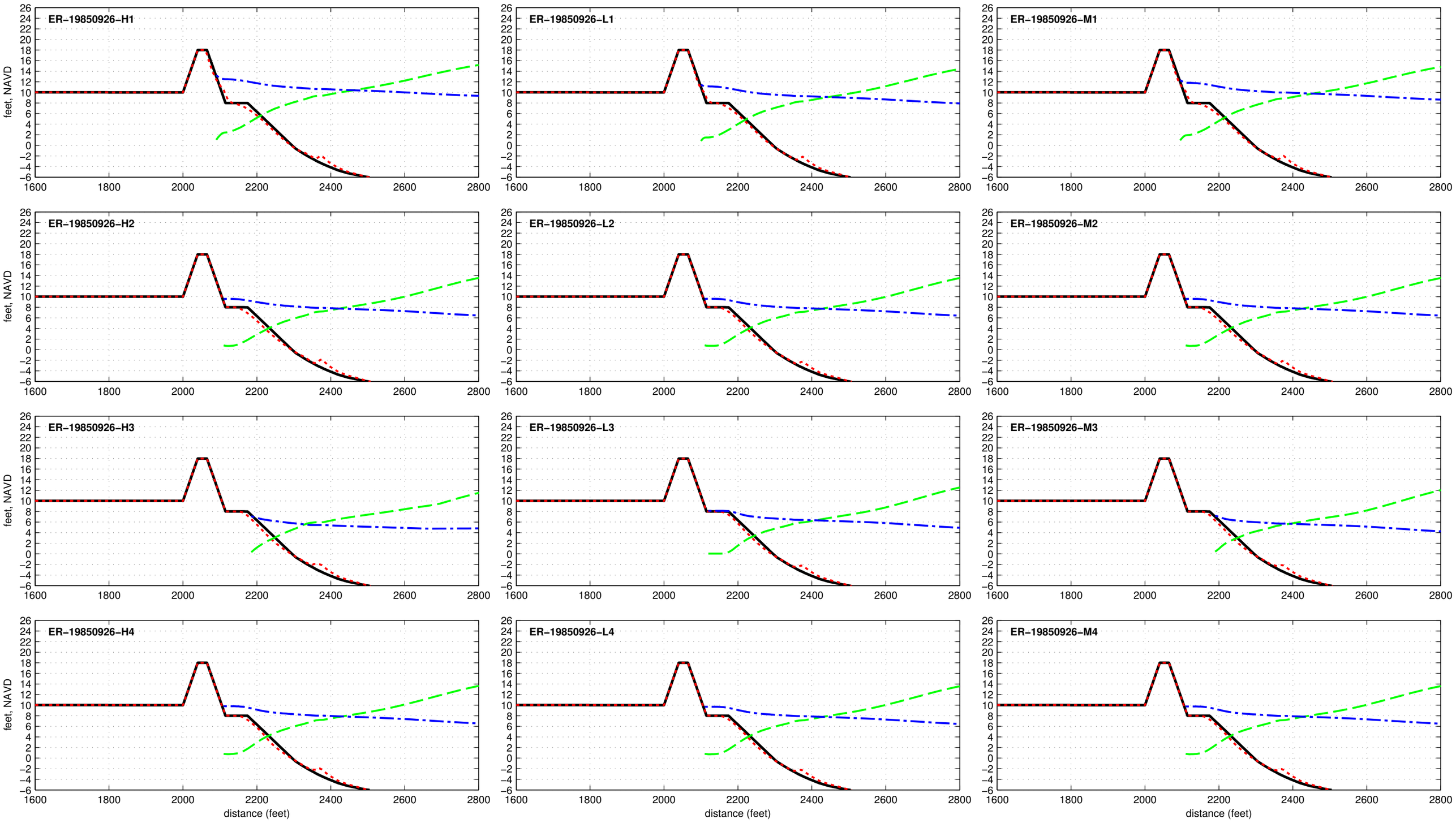

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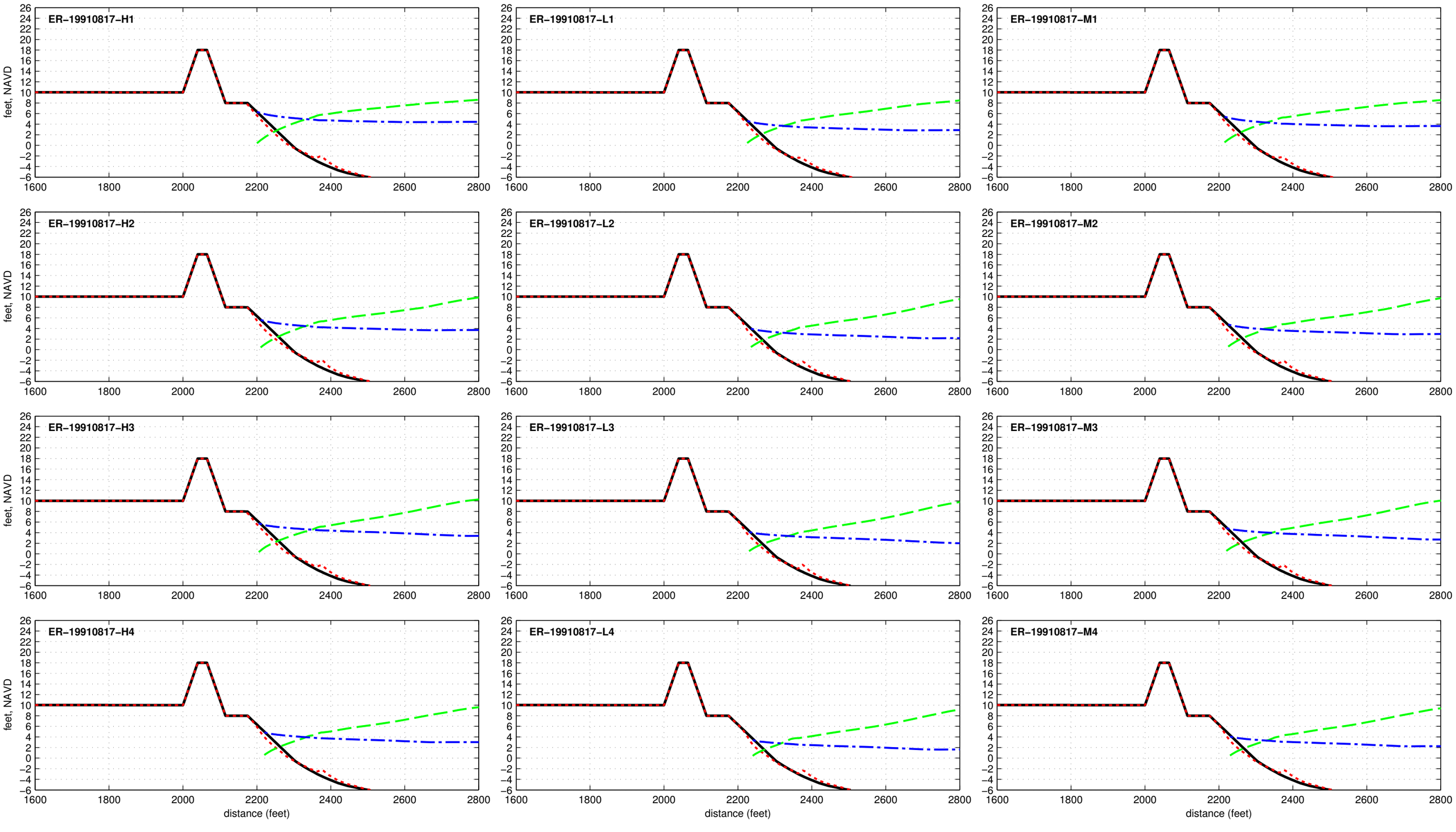

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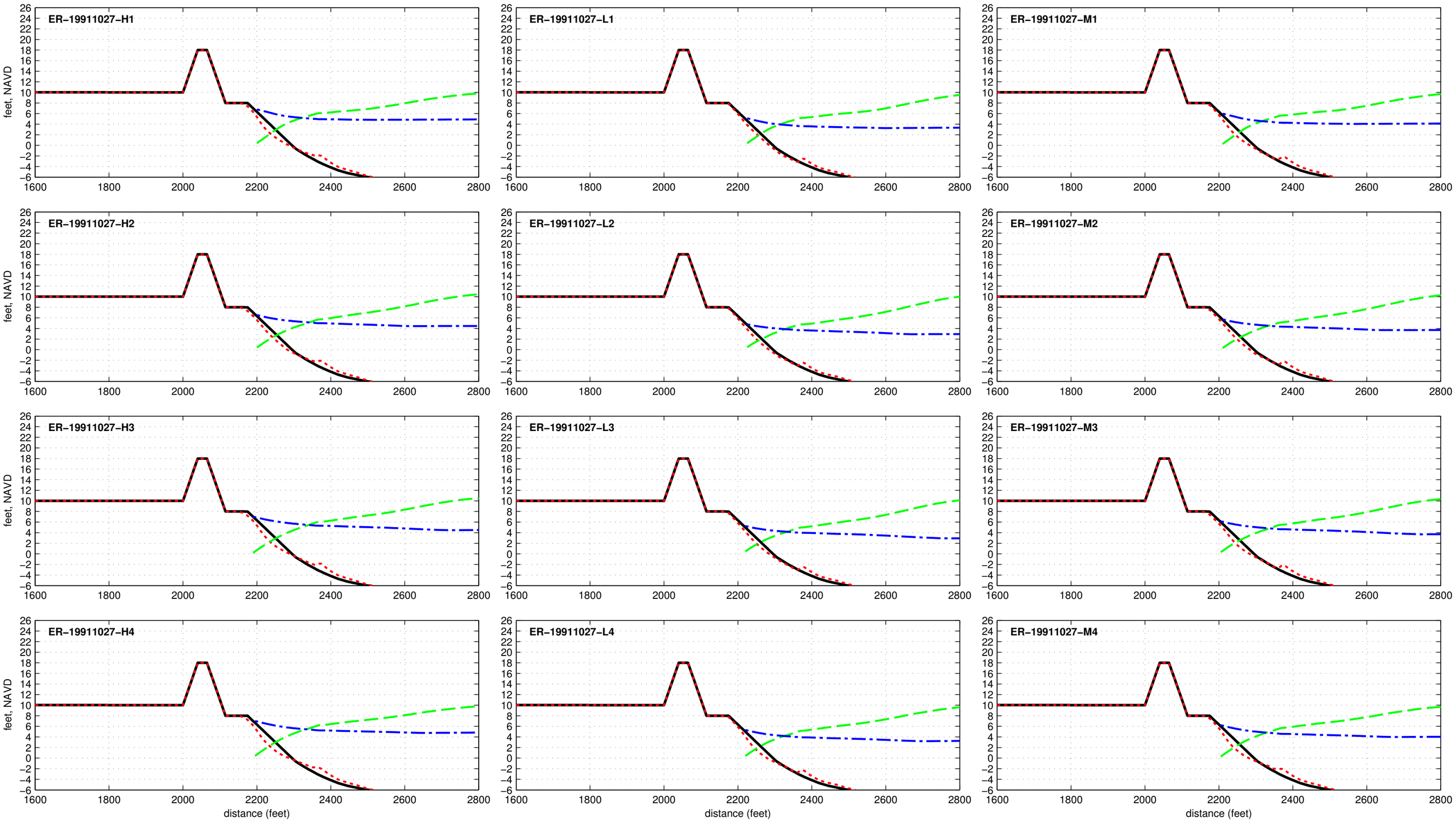

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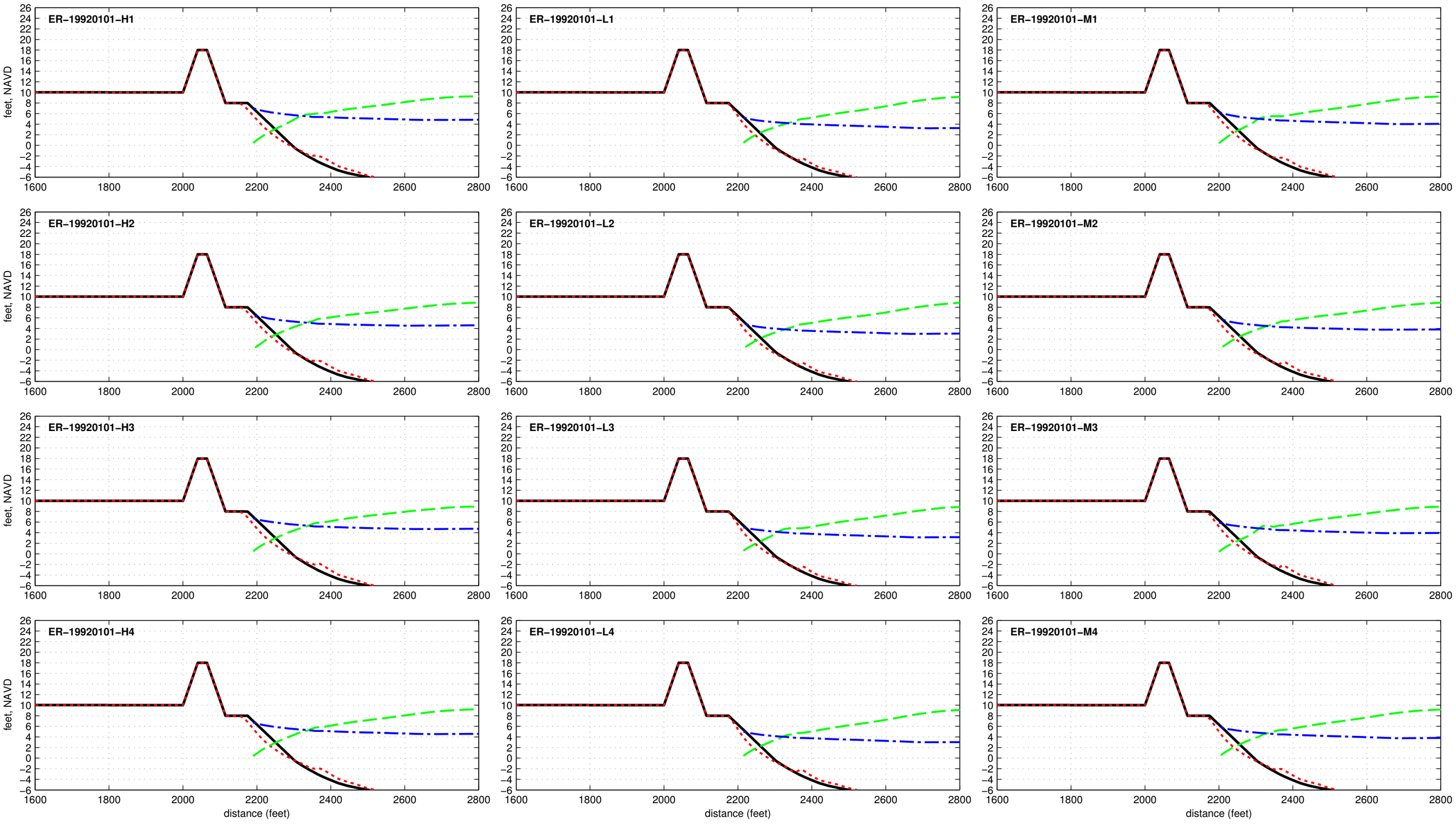

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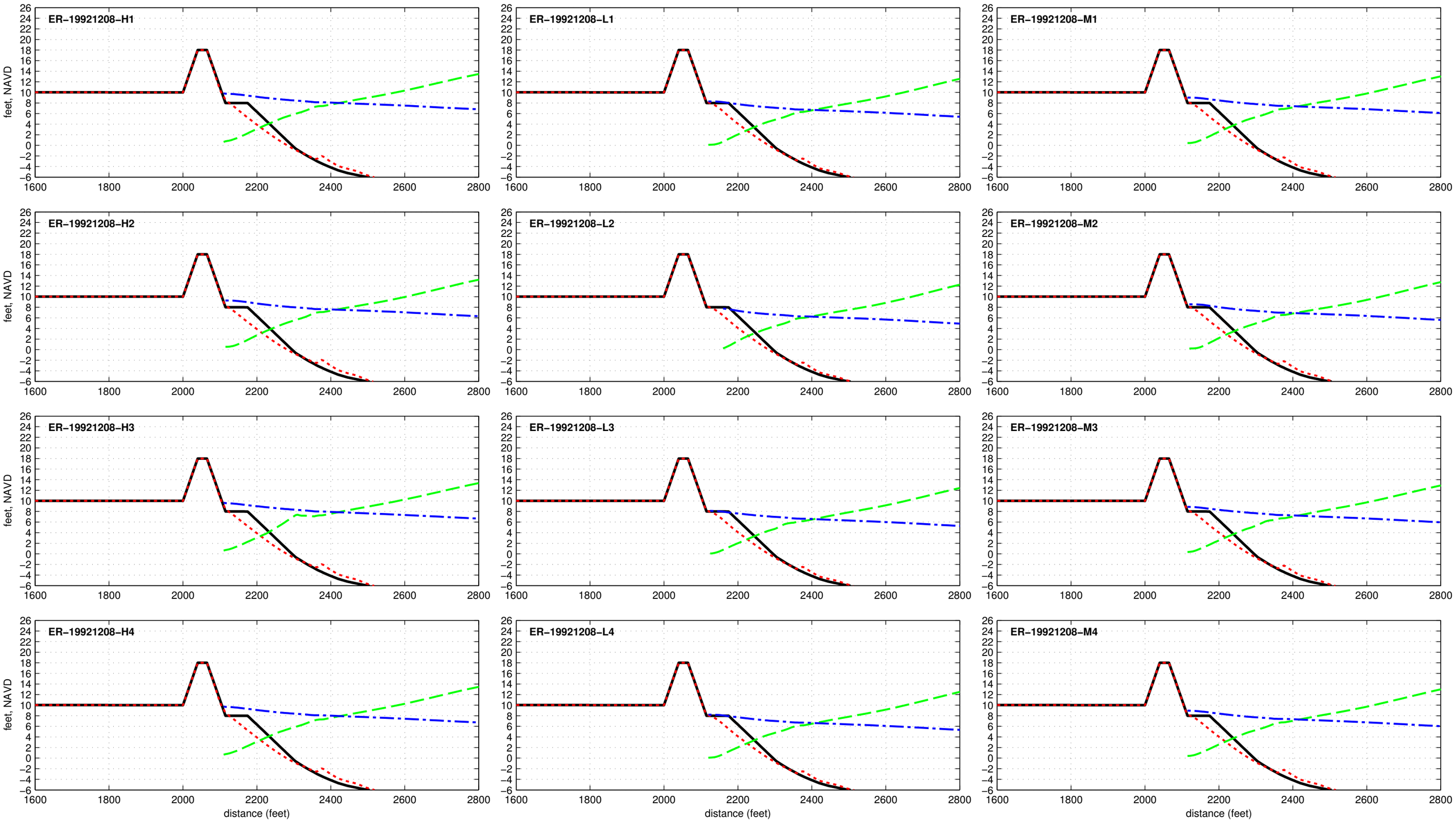

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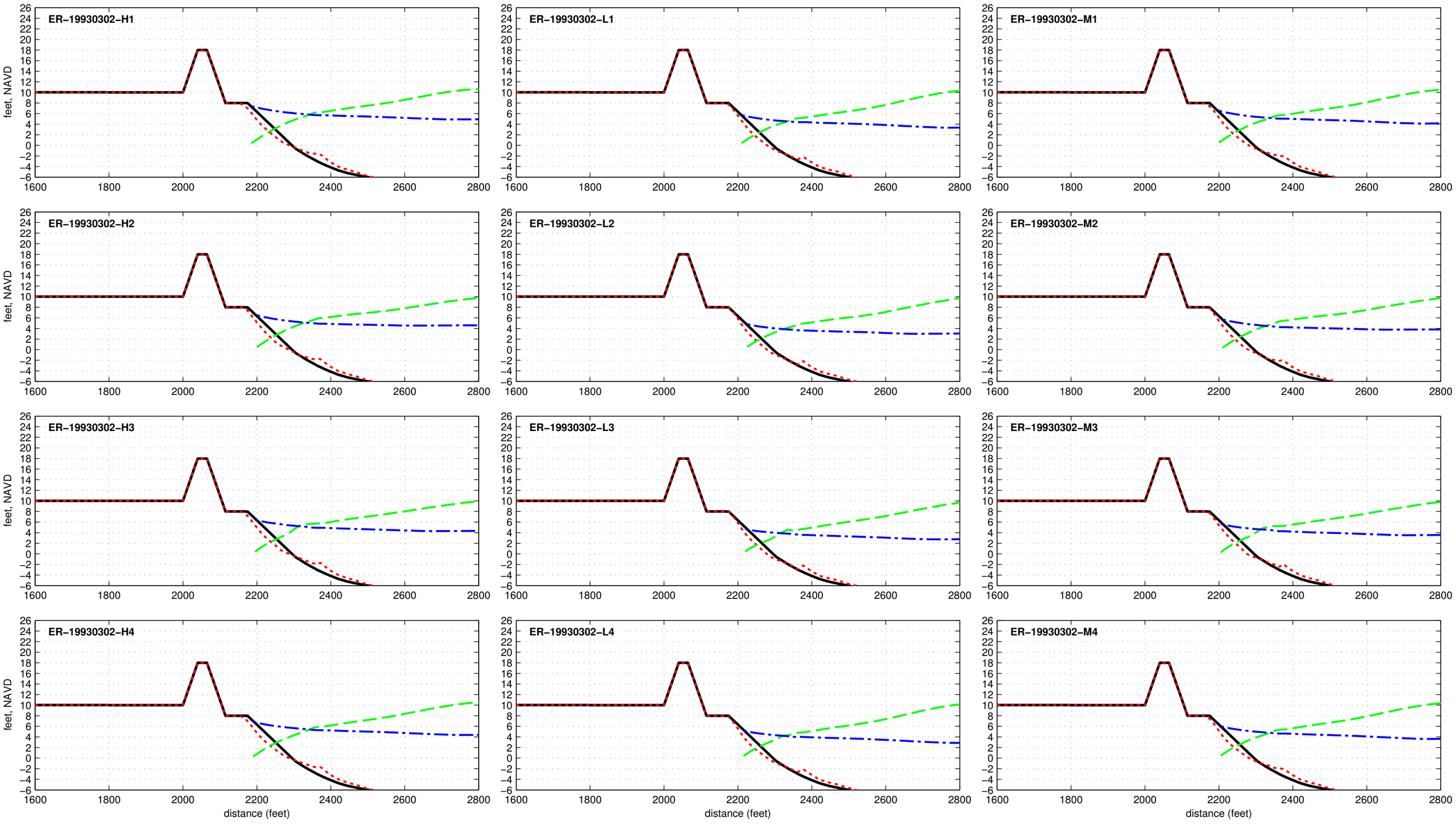

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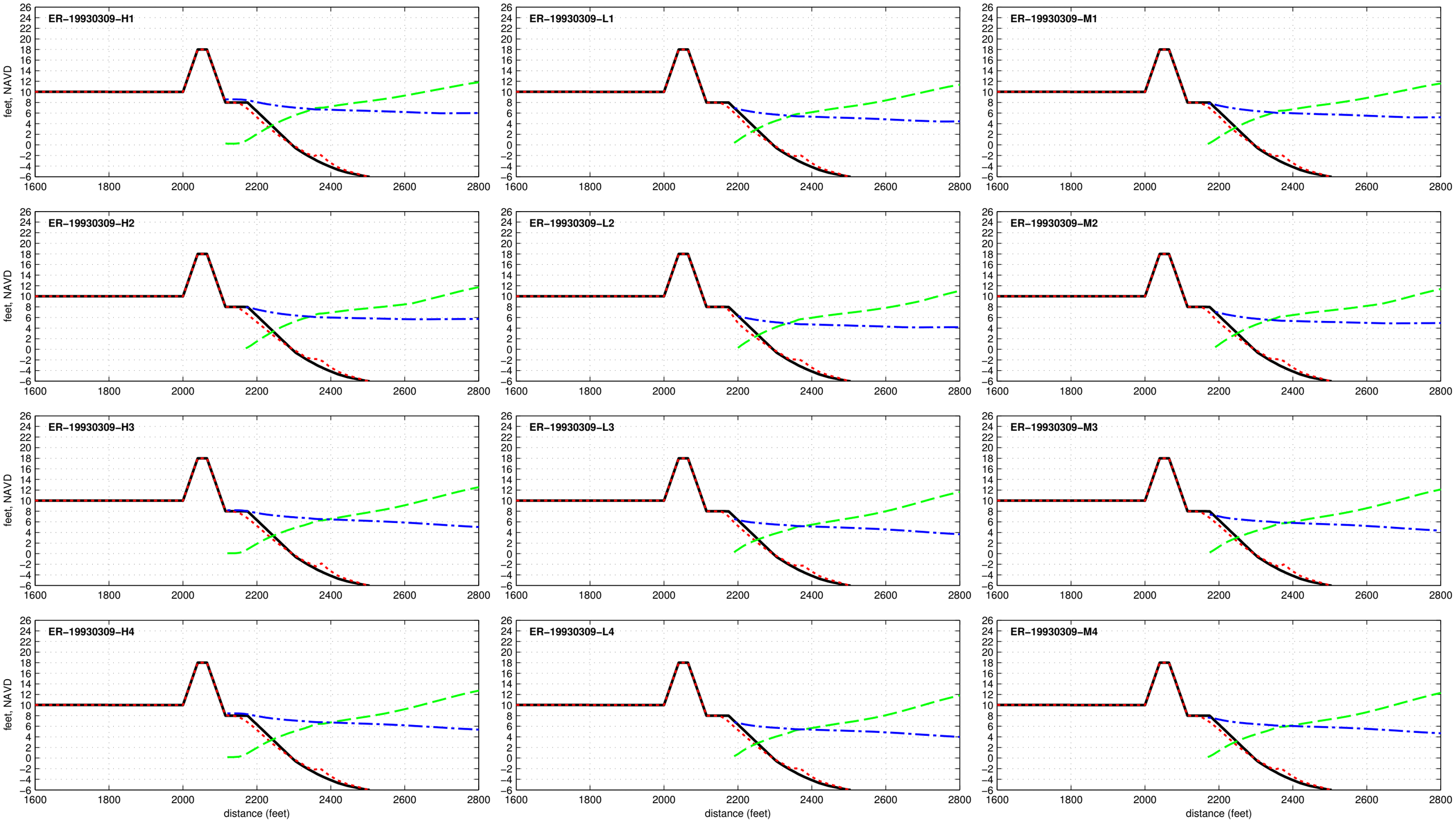

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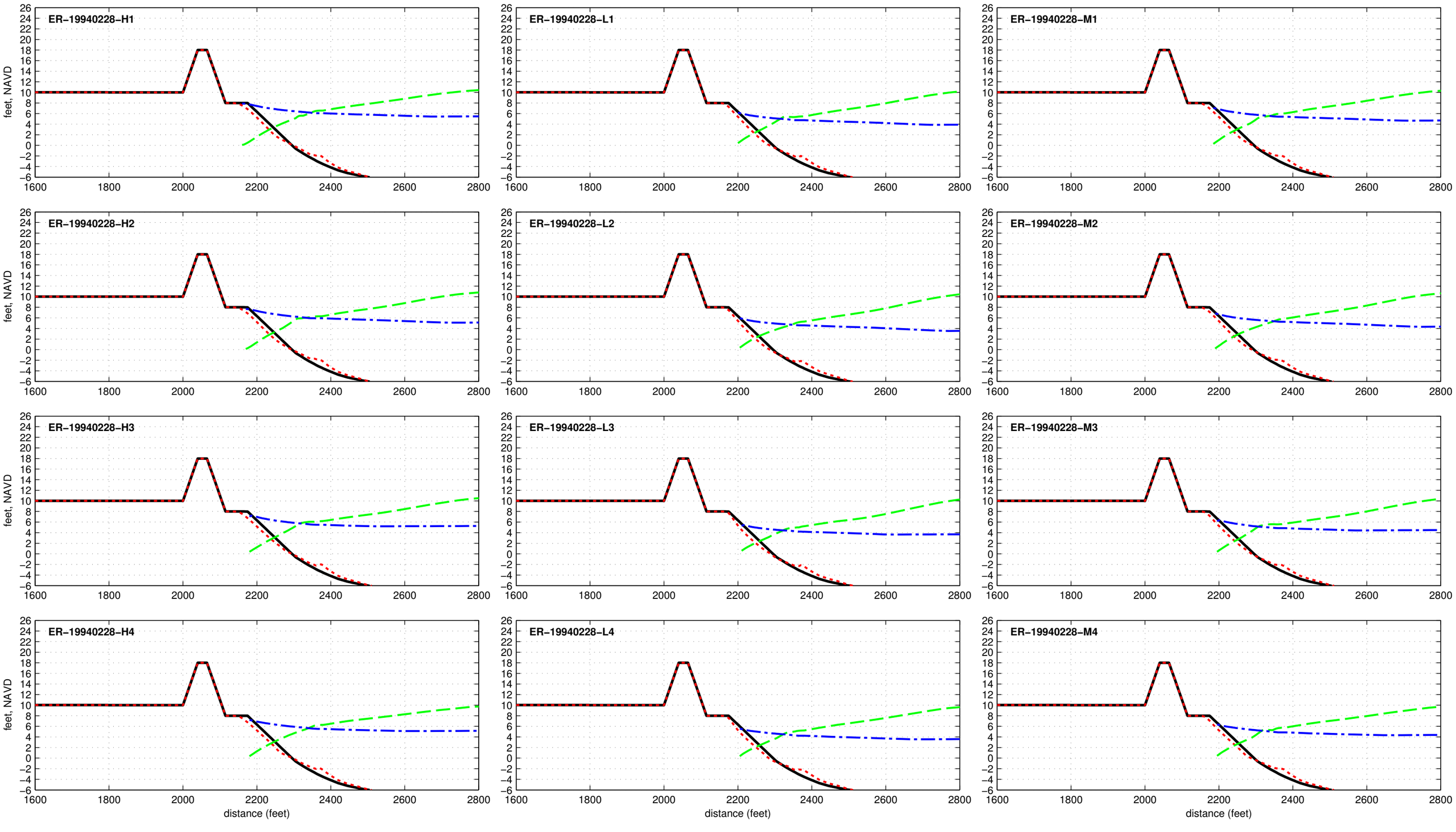

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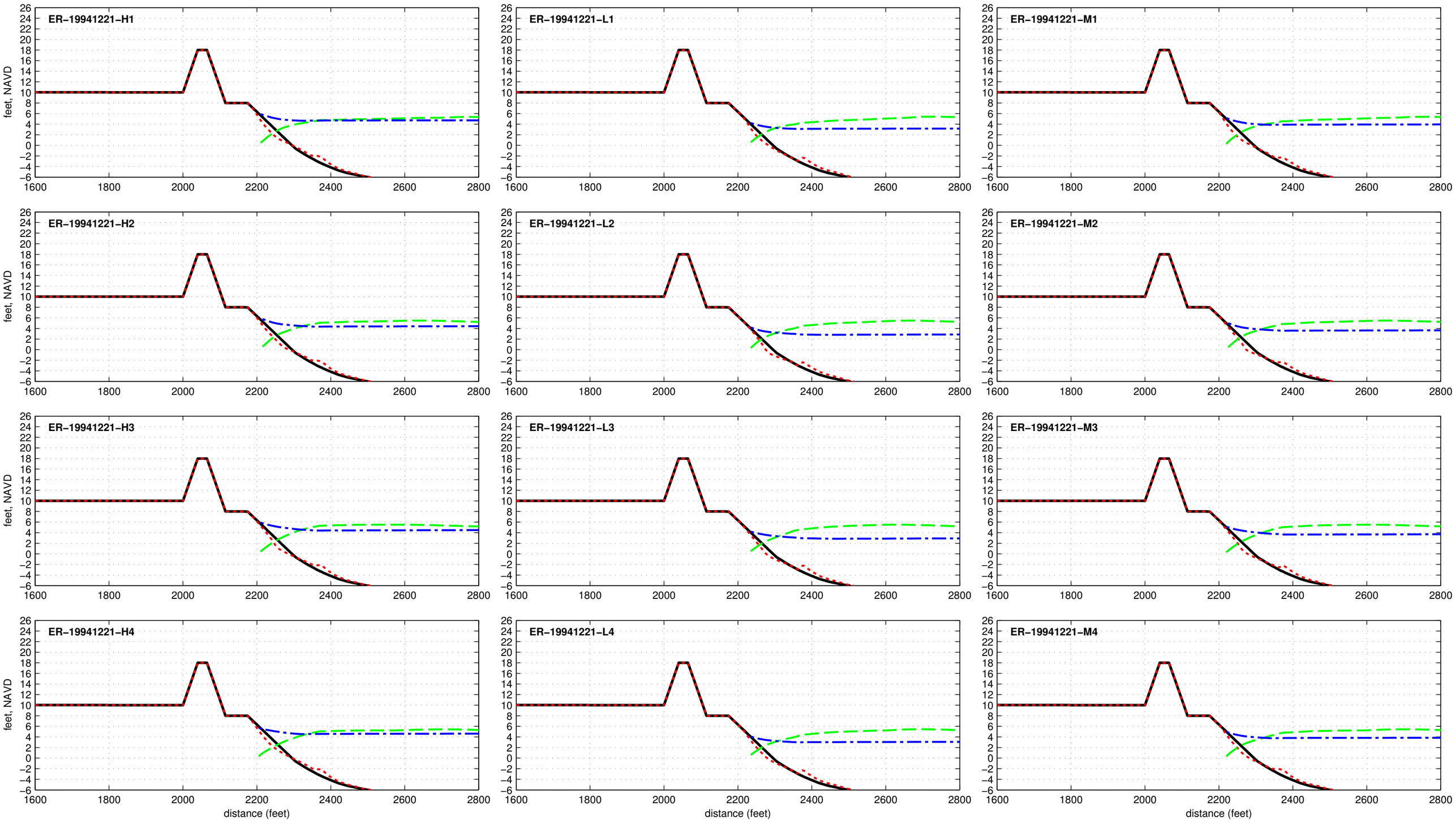

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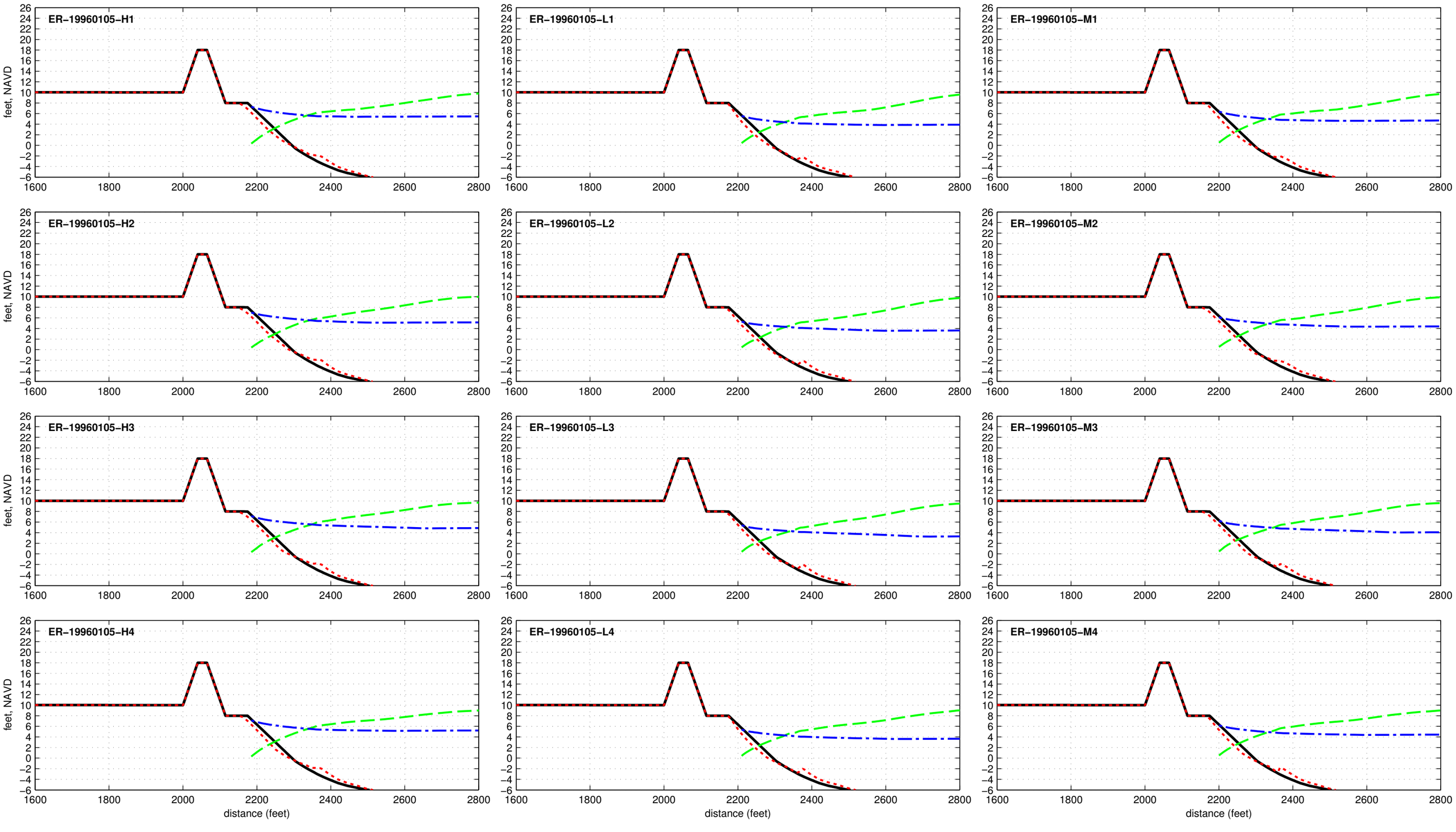

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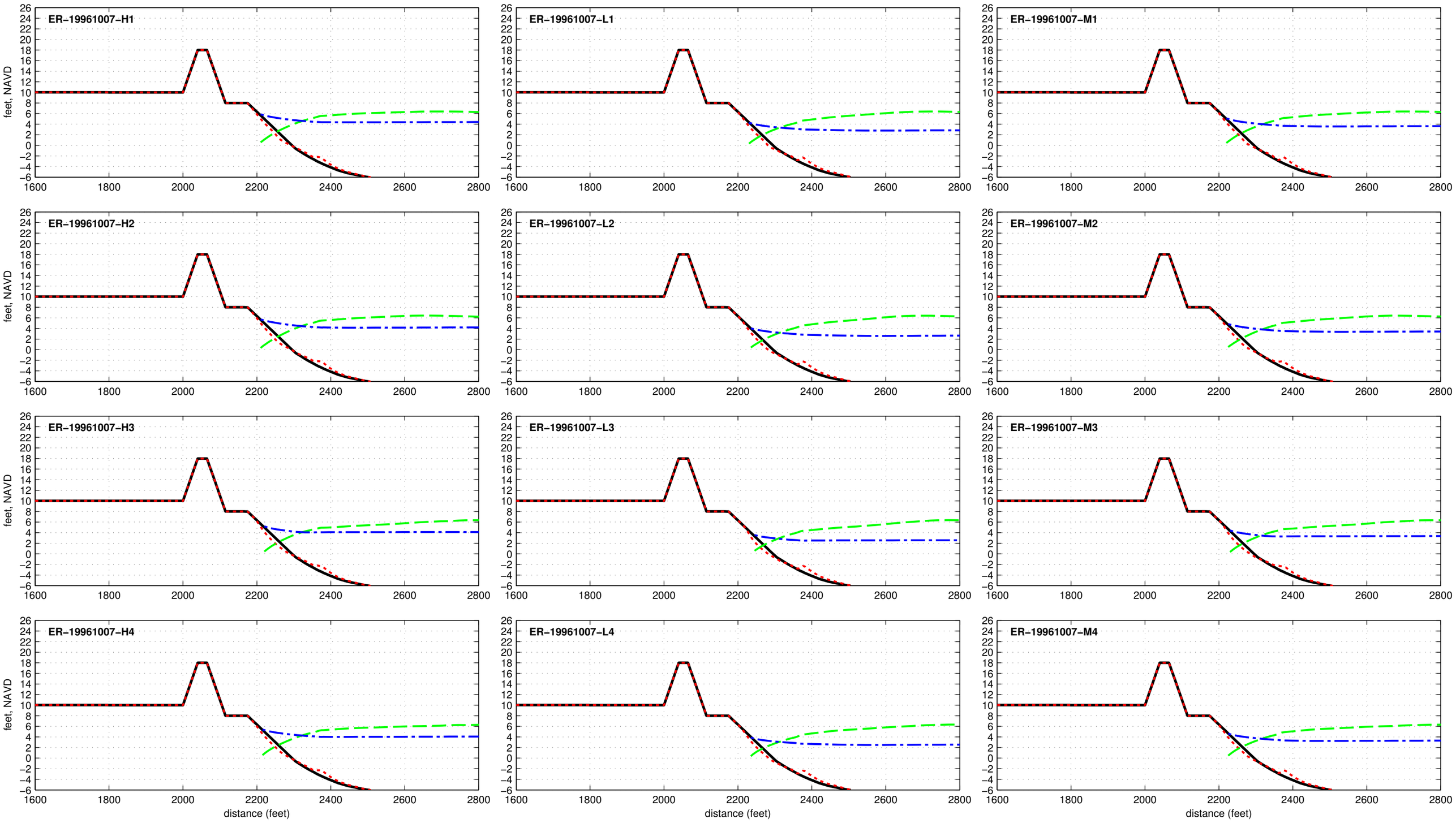

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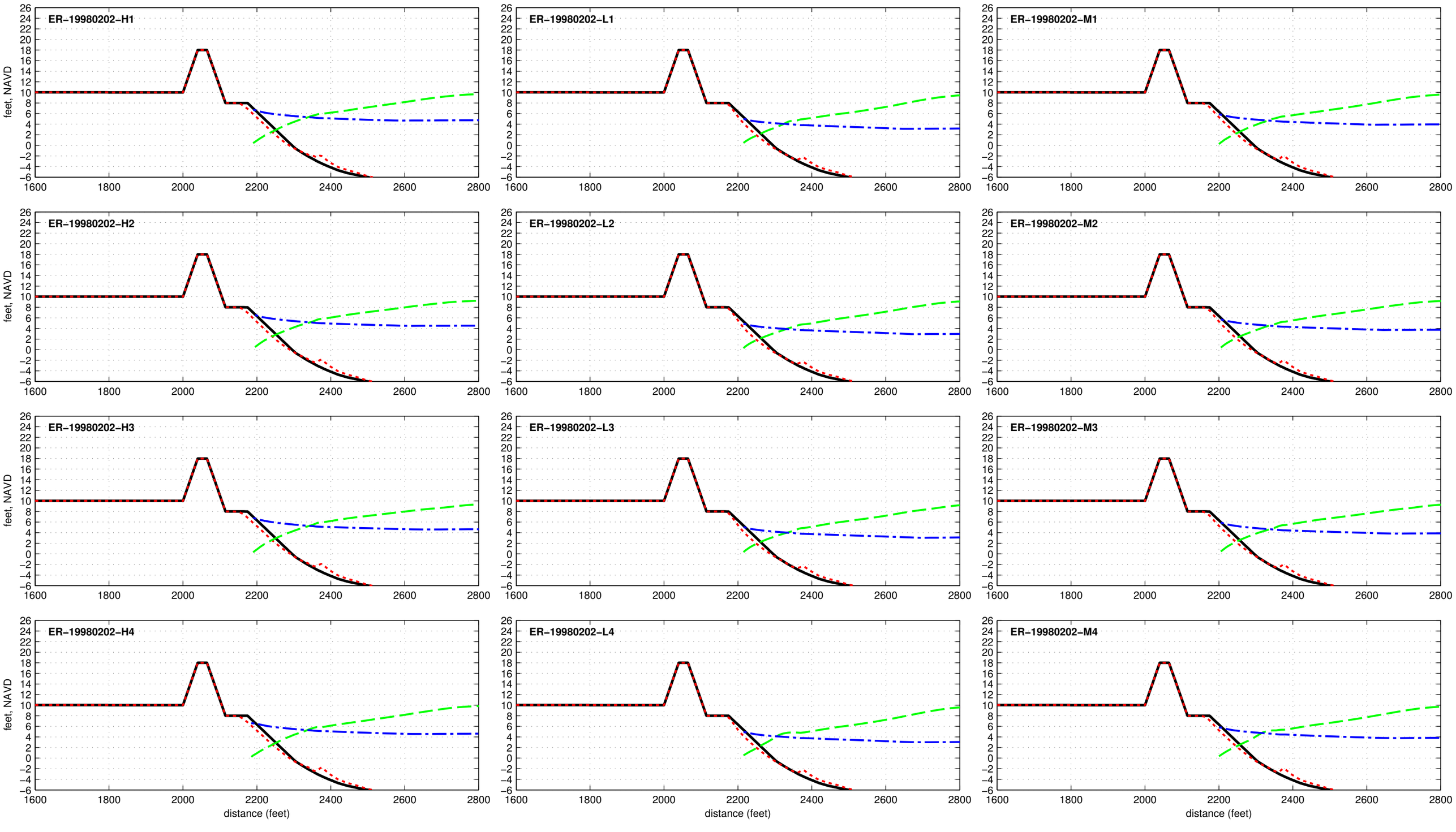

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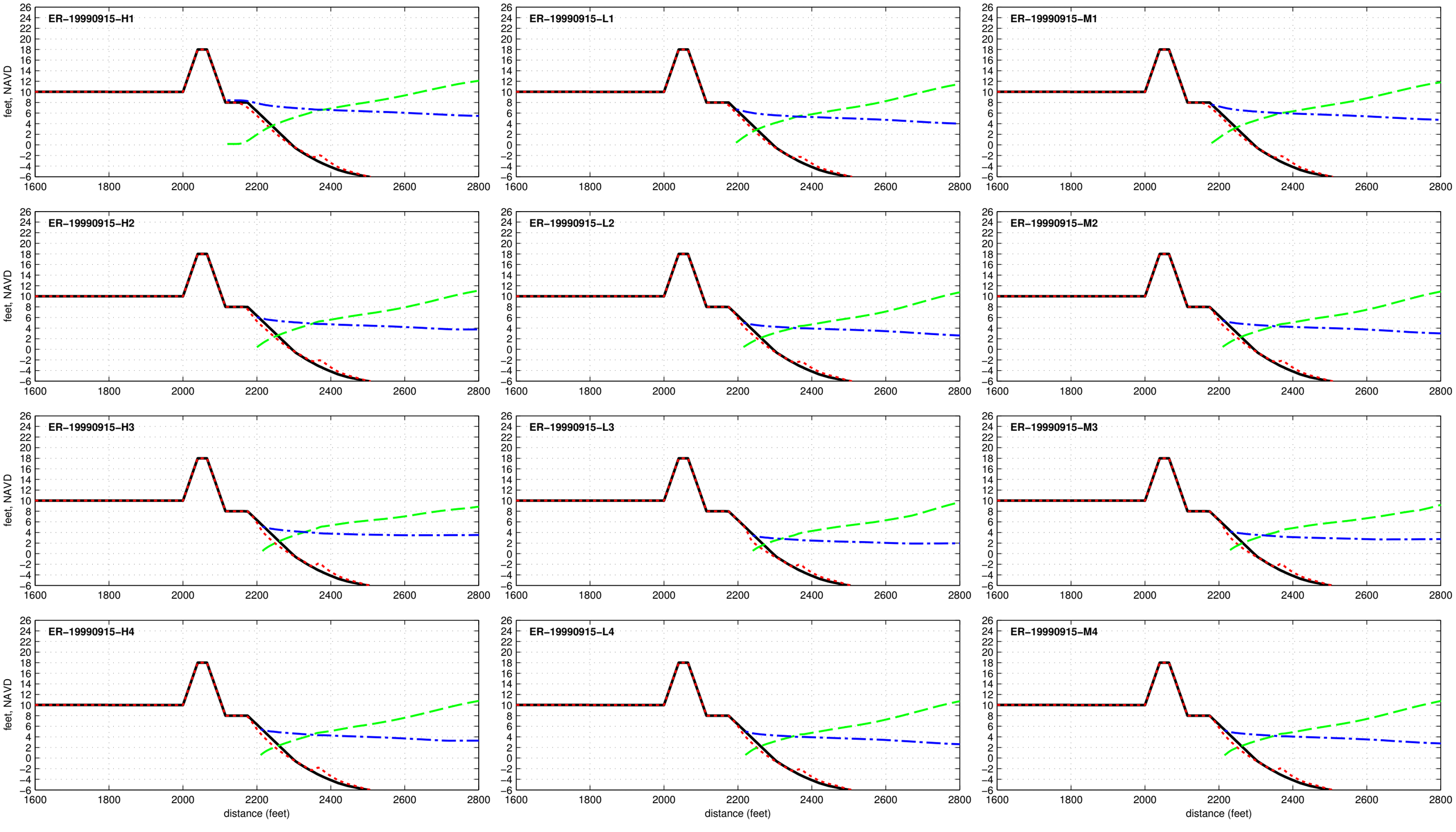

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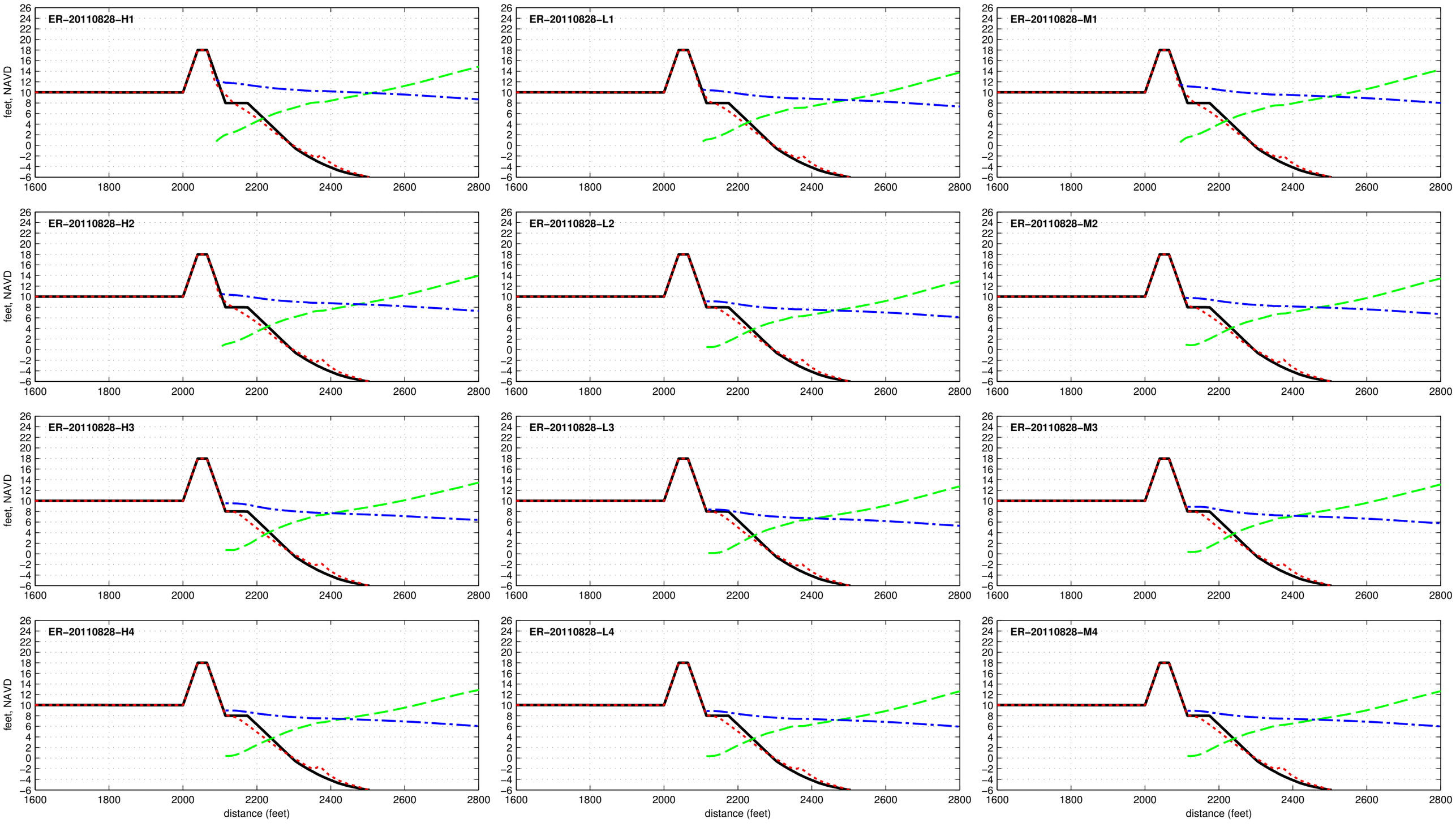

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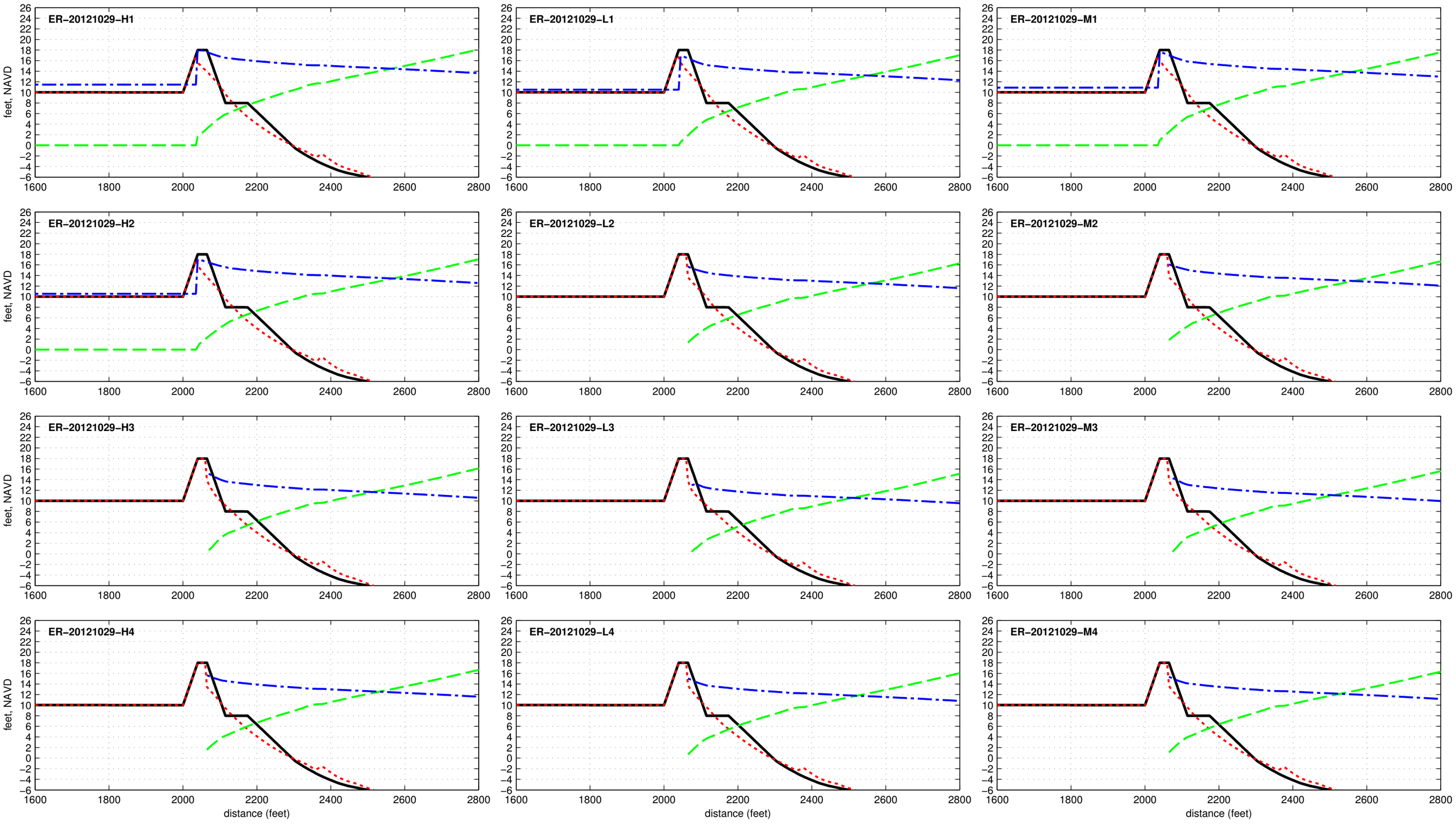


ENCLOSURE B

GENESIS-T MODEL RESULTS

SUB APPENDIX B

GENESIS-T MODEL RESULTS

GENESIS-T MODEL CALIBRAITON

Reach 5

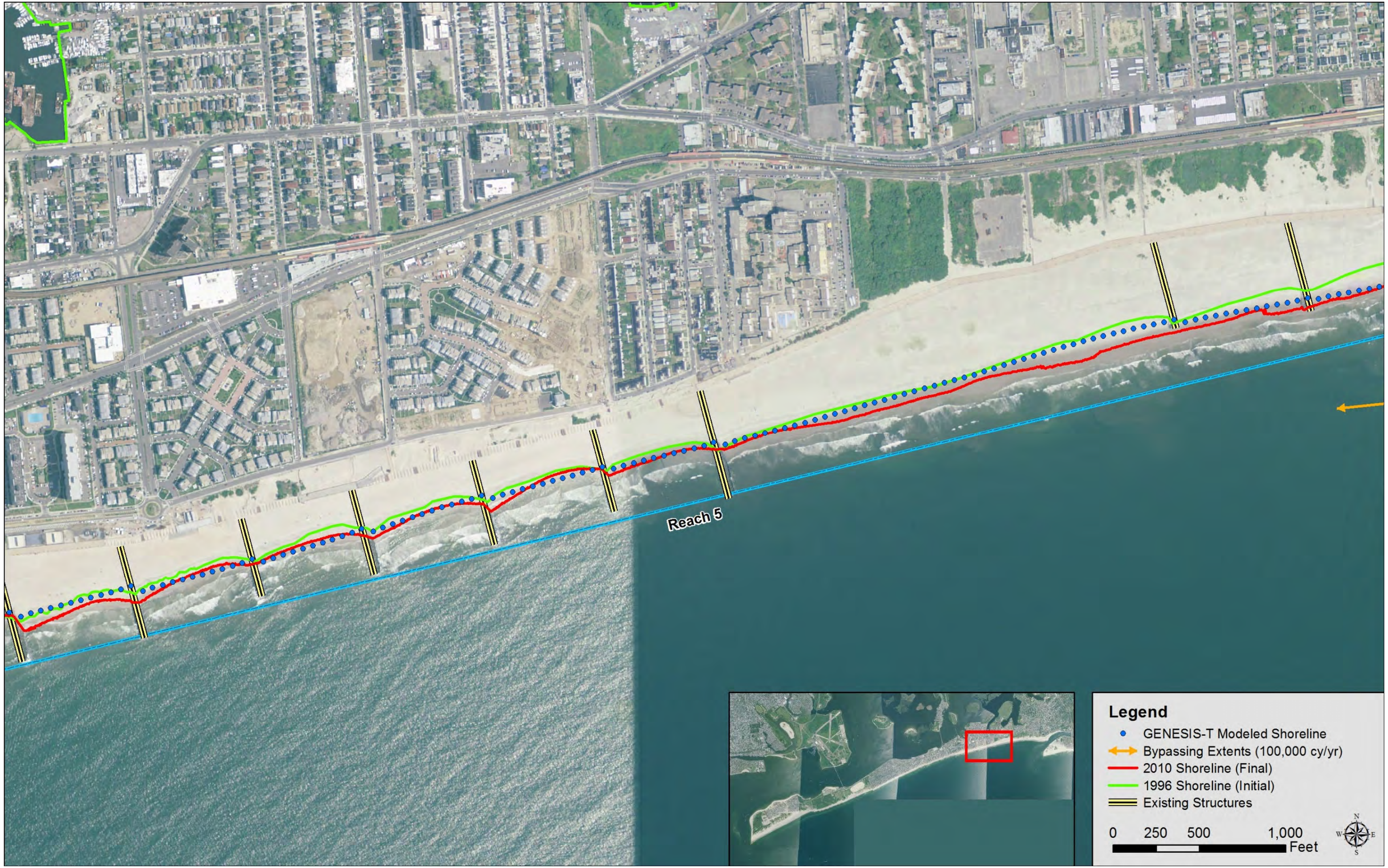
Reach 6

Legend

- GENESIS-T Modeled Shoreline
- Bypassing Extents (100,000 cy/yr)
- 2010 Shoreline (Final)
- 1996 Shoreline (Initial)
- Existing Structures

0 250 500 1,000 Feet

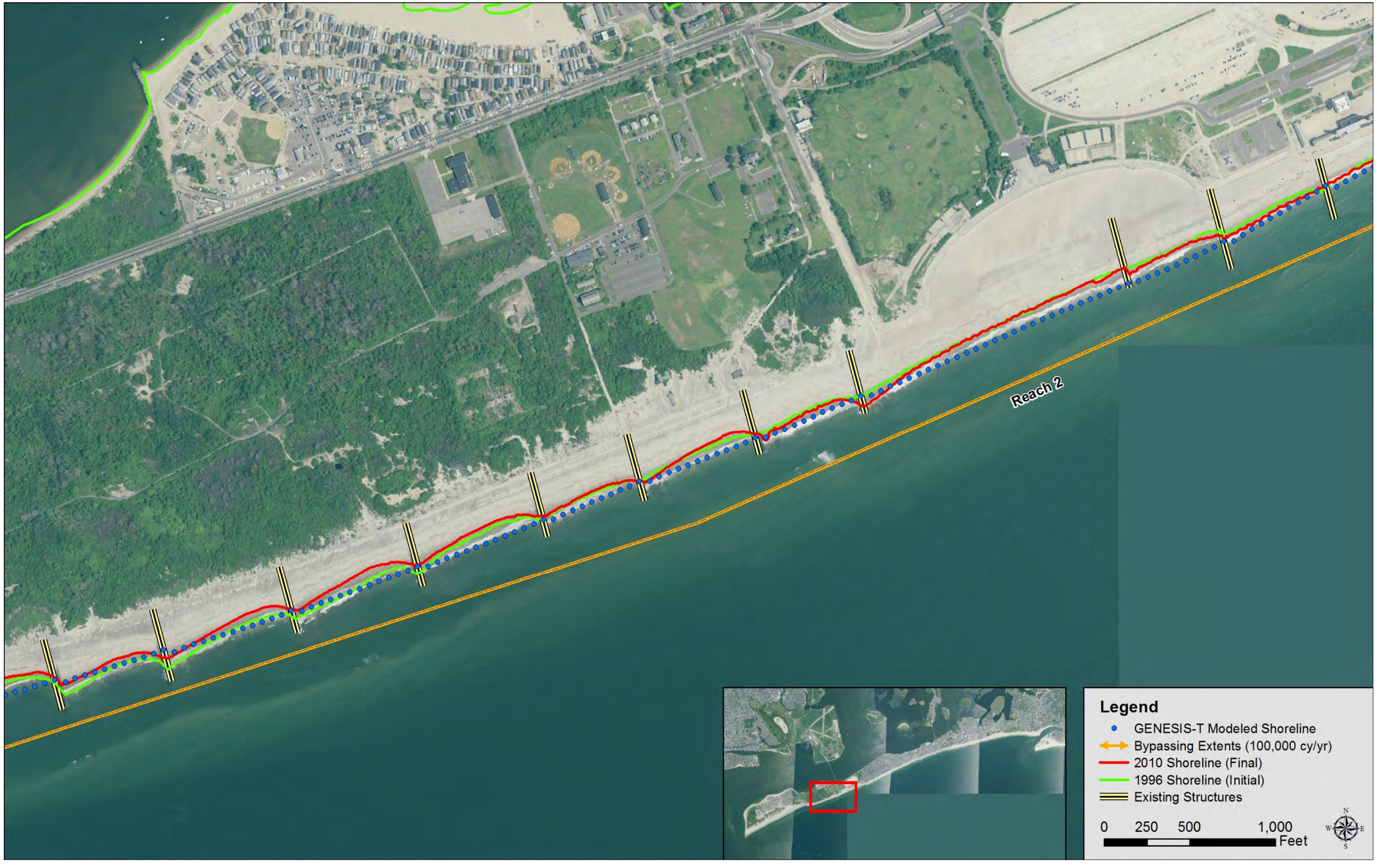
N W E S


Reach 5
Legend
GENESIS-T Modeled Shoreline
Bypassing Extents (100,000 cy/yr)
2010 Shoreline (Final)
1996 Shoreline (Initial)
Existing Structures
0 250 500 1,000
Feet
N
W
E
S

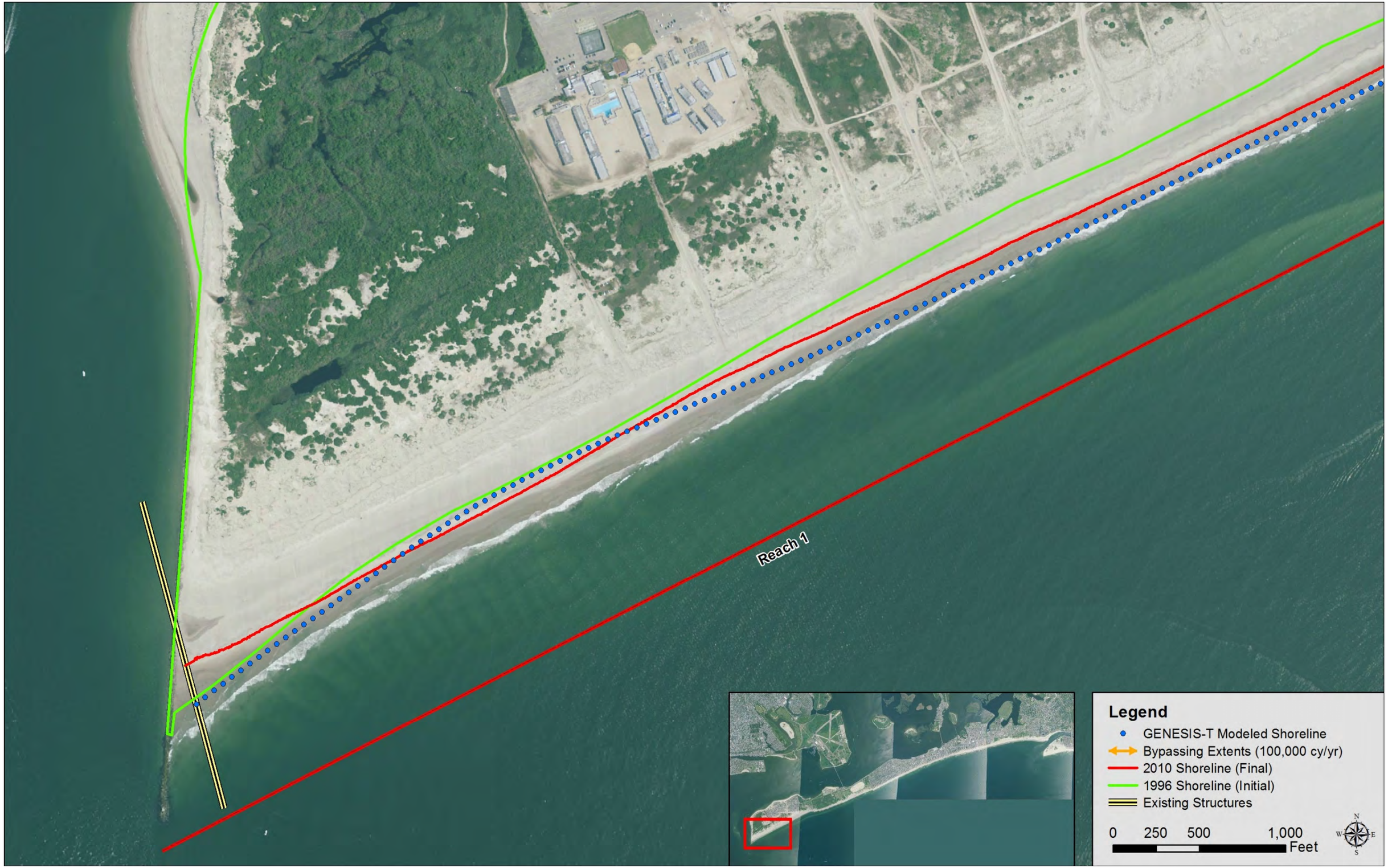

Reach 5
Reach 4
Legend
GENESIS-T Modeled Shoreline
Bypassing Extents (100,000 cy/yr)
2010 Shoreline (Final)
1996 Shoreline (Initial)
Existing Structures
0 250 500 1,000
Feet
N
W
E
S


Reach 4
Reach 3
Legend
GENESIS-T Modeled Shoreline
Bypassing Extents (100,000 cy/yr)
2010 Shoreline (Final)
1996 Shoreline (Initial)
Existing Structures
0
250
500
1,000
Feet
N
W
E
S


Reach 3
Reach 2
Legend
GENESIS-T Modeled Shoreline
Bypassing Extents (100,000 cy/yr)
2010 Shoreline (Final)
1996 Shoreline (Initial)
Existing Structures
0 250 500 1,000
Feet
N
W
E
S


Reach 2
Legend
GENESIS-T Modeled Shoreline
Bypassing Extents (100,000 cy/yr)
2010 Shoreline (Final)
1996 Shoreline (Initial)
Existing Structures
0 250 500 1,000
Feet
N
W
E
S


Reach 2
Reach 1
Legend
GENESIS-T Modeled Shoreline
Bypassing Extents (100,000 cy/yr)
2010 Shoreline (Final)
1996 Shoreline (Initial)
Existing Structures
0
250
500
1,000
Feet
N
W
E
S


Reach 1
Legend
GENESIS-T Modeled Shoreline
Bypassing Extents (100,000 cy/yr)
2010 Shoreline (Final)
1996 Shoreline (Initial)
Existing Structures
0
250
500
1,000
Feet
N
W
E
S

GENESIS-T WOPFC

Reach 5

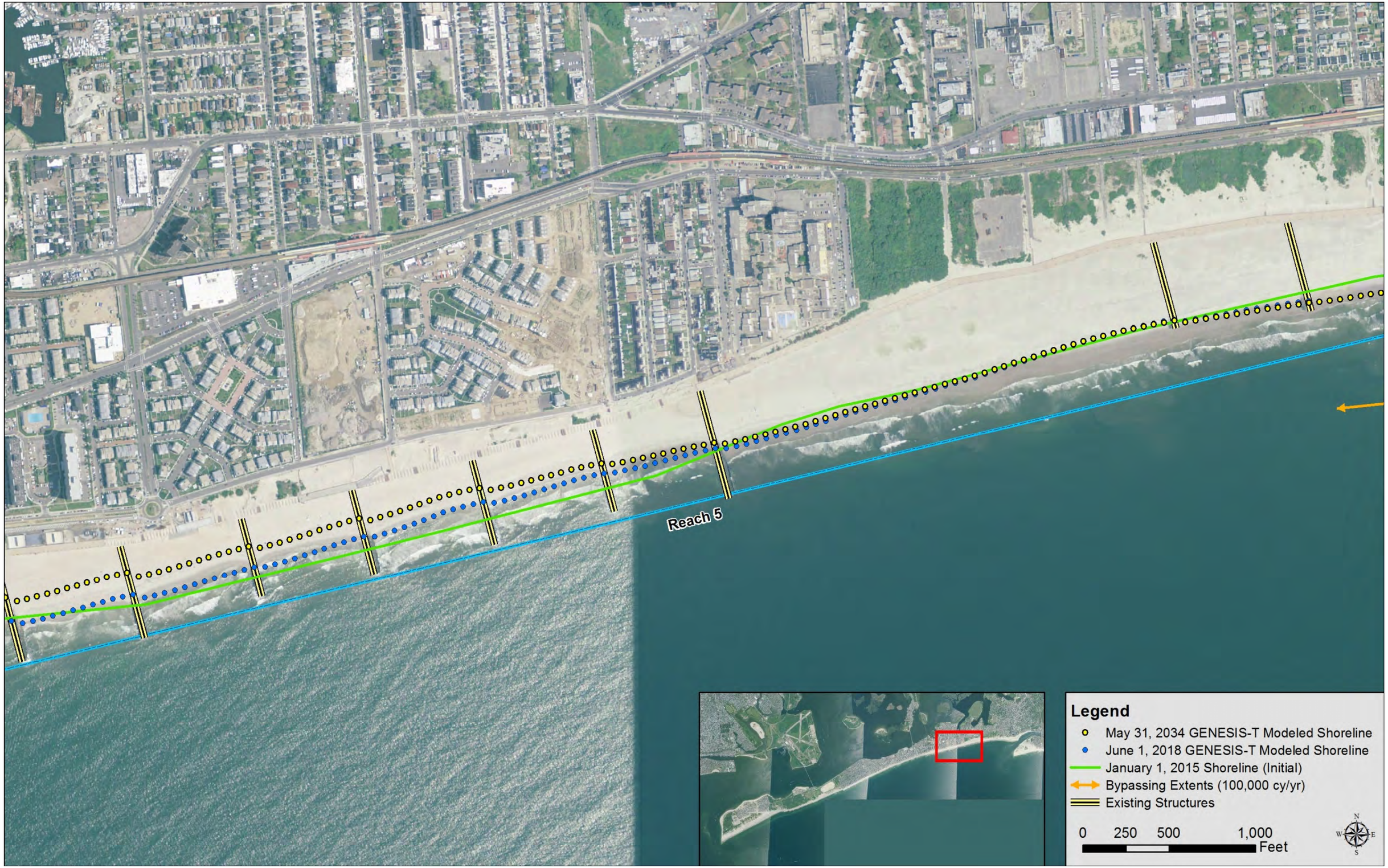
Reach 6

Legend

- May 31, 2034 GENESIS-T Modeled Shoreline
- June 1, 2018 GENESIS-T Modeled Shoreline
- January 1, 2015 Shoreline (Initial)
- Bypassing Extents (100,000 cy/yr)
- Existing Structures

0 250 500 1,000 Feet

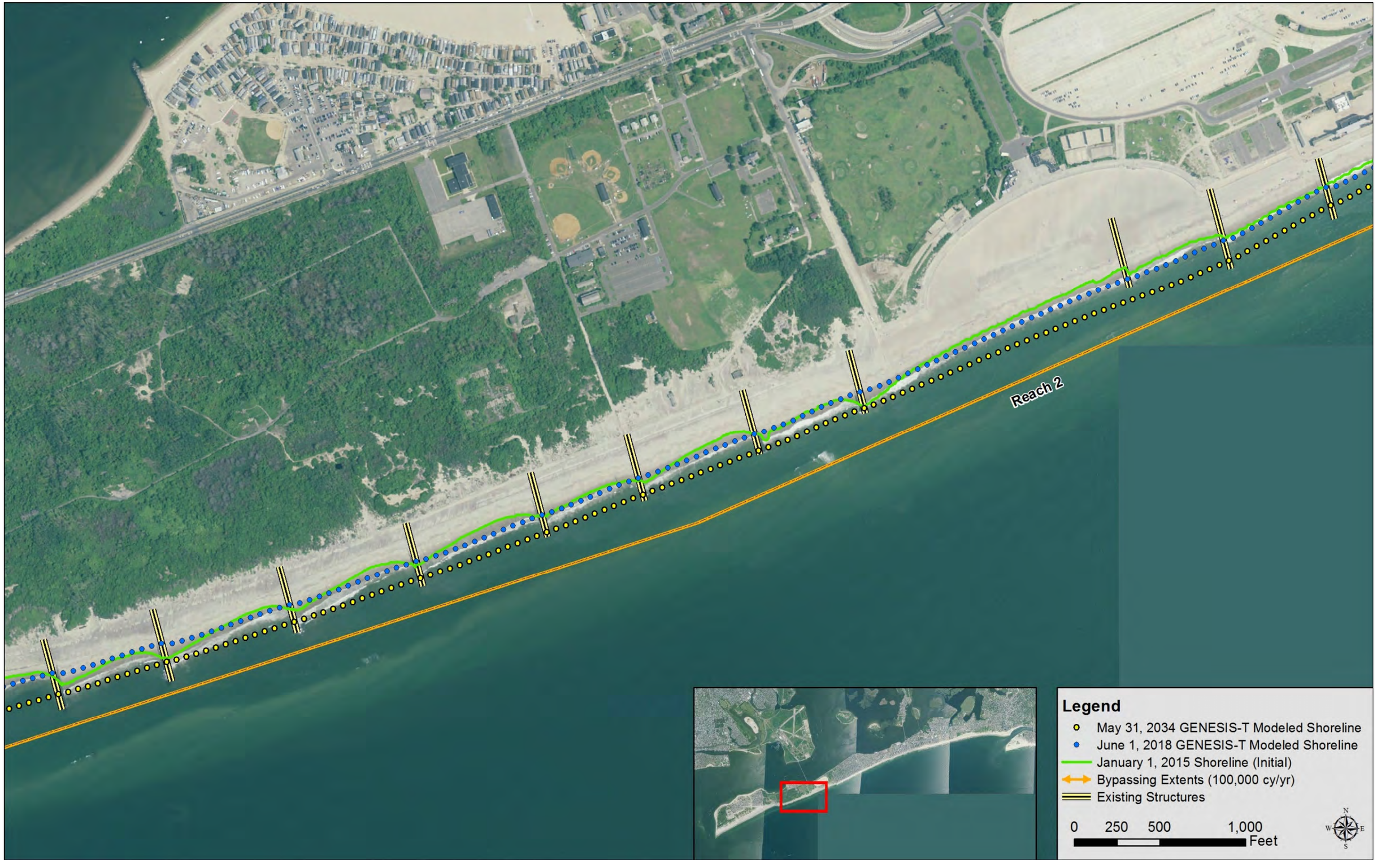
N W E S

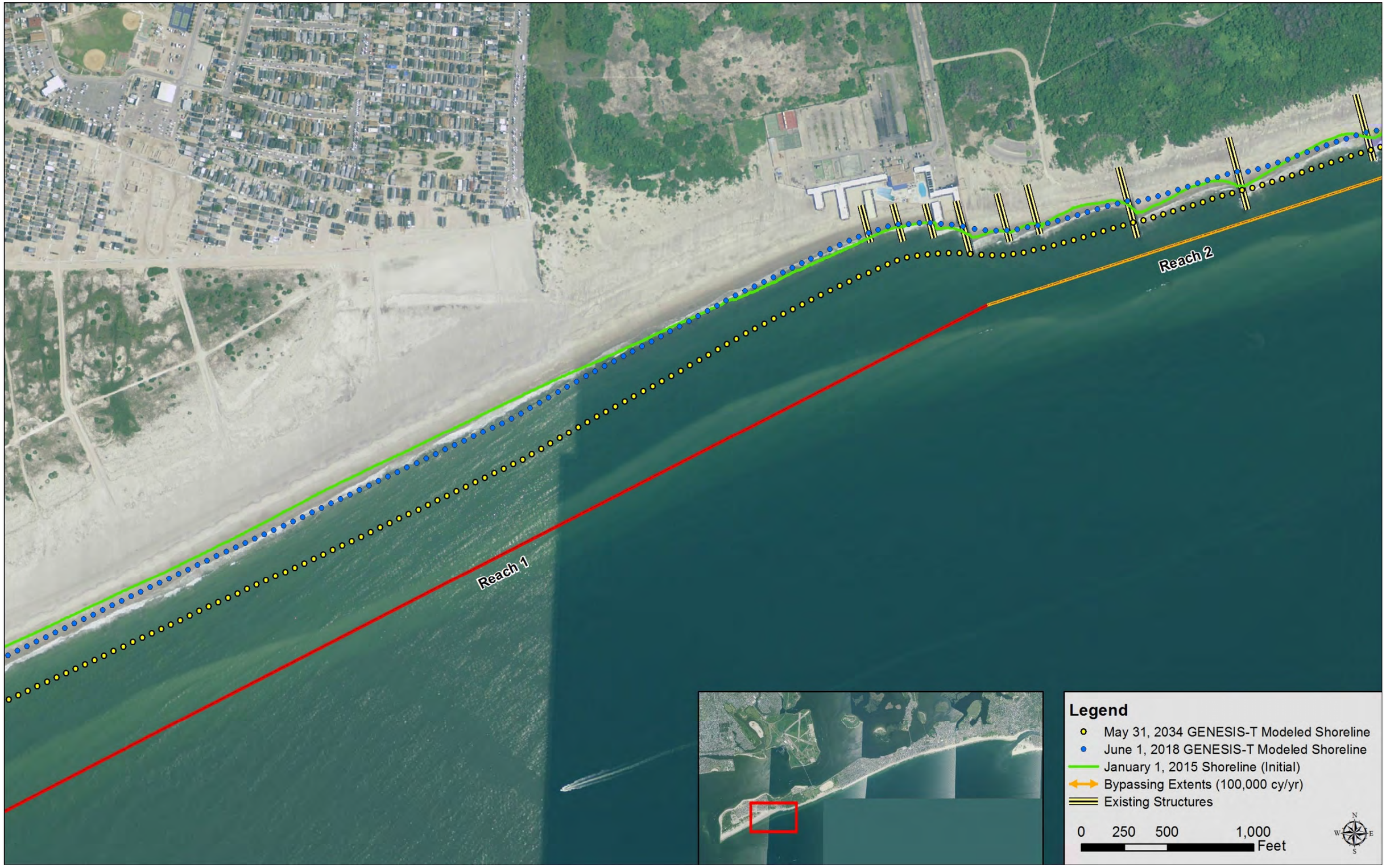

Reach 5
Legend
May 31, 2034 GENESIS-T Modeled Shoreline
June 1, 2018 GENESIS-T Modeled Shoreline
January 1, 2015 Shoreline (Initial)
Bypassing Extents (100,000 cy/yr)
Existing Structures
0 250 500 1,000
Feet
N
W
E
S

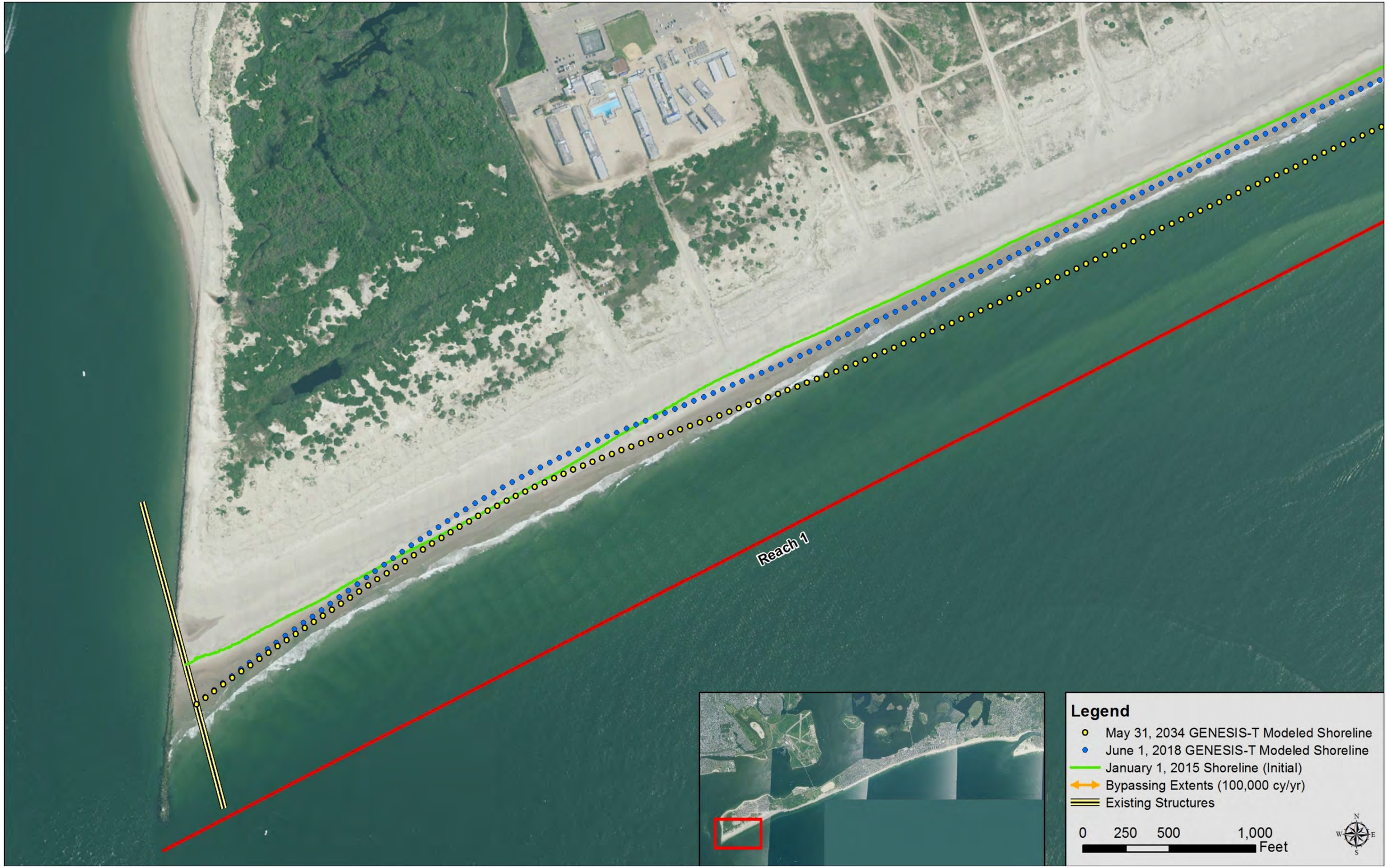

Reach 5
Reach 4
Legend
May 31, 2034 GENESIS-T Modeled Shoreline
June 1, 2018 GENESIS-T Modeled Shoreline
January 1, 2015 Shoreline (Initial)
Bypassing Extents (100,000 cy/yr)
Existing Structures
0 250 500 1,000
Feet
N
W
E
S


Reach 4
Reach 3
Legend
May 31, 2034 GENESIS-T Modeled Shoreline
June 1, 2018 GENESIS-T Modeled Shoreline
January 1, 2015 Shoreline (Initial)
Bypassing Extents (100,000 cy/yr)
Existing Structures
0 250 500 1,000
Feet
N
W
E
S


Reach 3
Reach 2
Legend
May 31, 2034 GENESIS-T Modeled Shoreline
June 1, 2018 GENESIS-T Modeled Shoreline
January 1, 2015 Shoreline (Initial)
Bypassing Extents (100,000 cy/yr)
Existing Structures
0 250 500 1,000
Feet
N
W
E
S


Reach 2
Legend
May 31, 2034 GENESIS-T Modeled Shoreline
June 1, 2018 GENESIS-T Modeled Shoreline
January 1, 2015 Shoreline (Initial)
Bypassing Extents (100,000 cy/yr)
Existing Structures
0 250 500 1,000
Feet
N
W
E
S

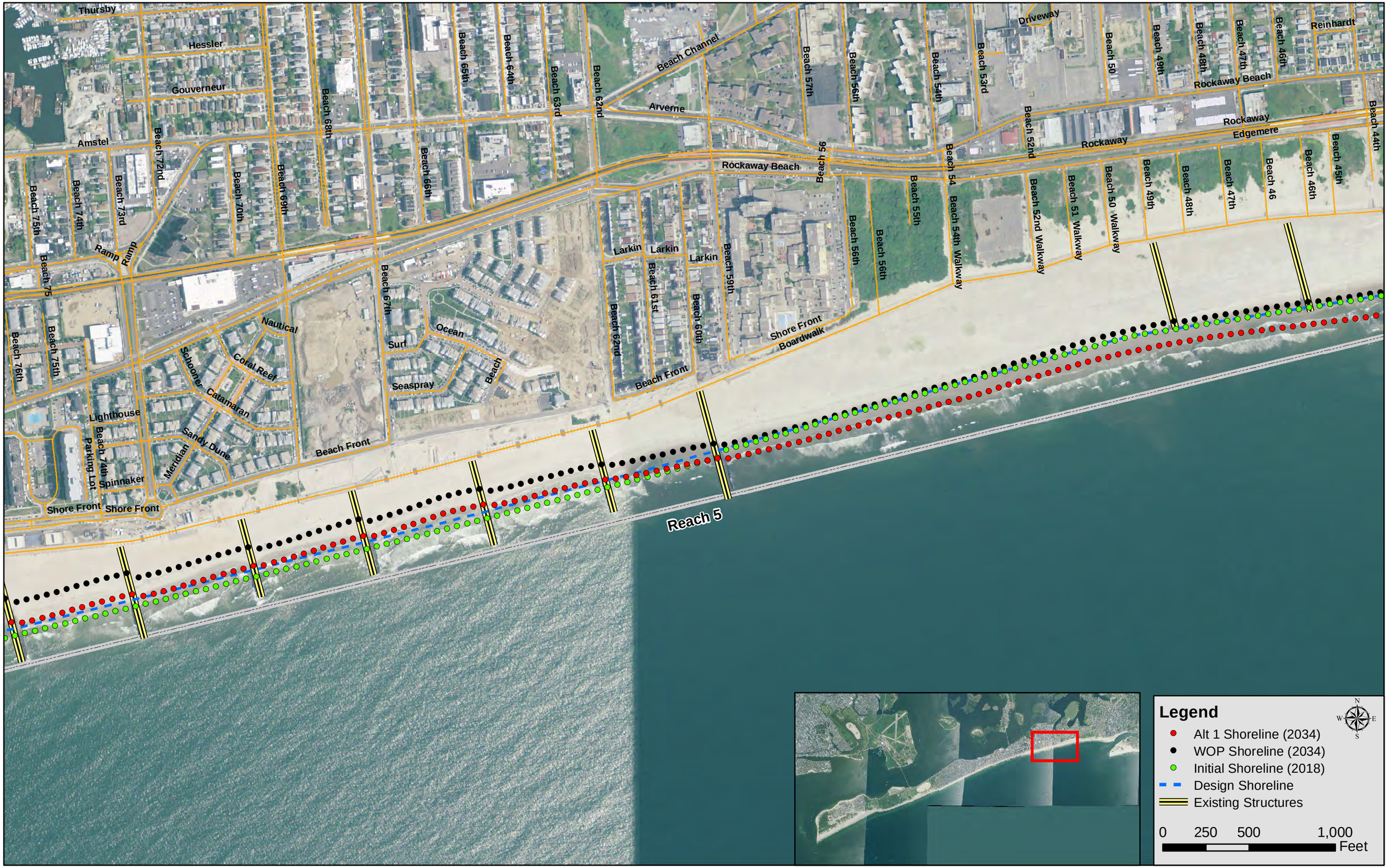

Reach 2
Reach 1
Legend
May 31, 2034 GENESIS-T Modeled Shoreline
June 1, 2018 GENESIS-T Modeled Shoreline
January 1, 2015 Shoreline (Initial)
Bypassing Extents (100,000 cy/yr)
Existing Structures
0
250
500
1,000
Feet
N
W
E
S


Reach 1
Legend
May 31, 2034 GENESIS-T Modeled Shoreline
June 1, 2018 GENESIS-T Modeled Shoreline
January 1, 2015 Shoreline (Initial)
Bypassing Extents (100,000 cy/yr)
Existing Structures
0 250 500 1,000
Feet
N
W
E
S

GENESIS-T ALTERNATIVE 1 – BEACH RESTORATION



Norton
Hantz
Beach Channel
Beach 48th
Beach 47th
Beach 46th
Beach 45th
Beach 44th
Beach 43rd
Beach 42nd
Beach 41st
Beach 40th
Beach 39th
Beach 38th
Beach 37th
Beach 36th
Beach Channel
Ramp
Rockaway
Deerfield
Beach 31st
Beach 30th
Beach 29th
Beach 28th
Beach 27th
Camp
Edgemere
Watjean
Highland
Fernside
Crest
Beach 20th
Beach 19th
Seagirt
Beach 17th
Edgemere
Beach 31st
Beach 32nd
Beach 27th
Beach 26th
Ramp
Beach 49th
Rockaway Beach
Rockaway
Beach 35th
Beach 34
Beach 33
Lewmay
Beach 29th
Beach 28th
Beach 28th
Beach 27th
Beach 26th
Alley
Beach 25th
Beach 24th
Parking Lot
Parking Lot
Parking Lot
Parking Lot
Parking Lot
Driveway
Walkway
Beach 37th
Beach 36
Surf
Beach 31st
Beach 30th
Beach 44th
Beach 43rd
Beach 42nd
Beach 40th
Spray View
Spray View
Spray View
Boardwalk
Beach 48th
Beach 47th
Beach 46
Beach 46th
Beach 45th
Walkway
Reach 6
Reach 5
Legend
Alt 1 Shoreline (2034)
WOP Shoreline (2034)
Initial Shoreline (2018)
Design Shoreline
Existing Structures
0
250
500
1,000
Feet
N
W
E
S


Thursby
Hessler
Gouverneur
Amstel
Beach 72nd
Beach 73rd
Beach 74th
Beach 75th
Beach 75
Beach 76th
Beach 75th
Beach 70th
Beach 69th
Beach 68th
Beach 66th
Beach 65th
Beach 64th
Beach 63rd
Beach 62nd
Beach Channel
Arverne
Rockaway Beach
Beach 57th
Beach 56th
Beach 54th
Beach 53rd
Driveway
Beach 52nd
Beach 50
Beach 49th
Beach 48th
Beach 47th
Beach 46th
Reinhardt
Rockaway Beach
Rockaway
Edgemere
Beach 44th
Beach 45th
Beach 46th
Beach 46
Beach 47th
Beach 48th
Beach 49th
Beach 50 Walkway
Beach 51 Walkway
Beach 52nd Walkway
Beach 54th Walkway
Beach 54
Beach 55th
Beach 56th
Beach 56th
Beach 56th
Beach 59th
Beach 60th
Beach 61st
Beach 62nd
Larkin
Larkin
Larkin
Shore Front
Boardwalk
Beach Front
Ramp
Ramp
Beach 67th
Surf
Ocean
Beach
Seaspray
Nautical
Schooner
Catamaran
Coral Reef
Lighthouse
Beach 74th
Parking Lot
Spinnaker
Sandy Dune
Meridian
Beach Front
Shore Front
Shore Front
Reach 5
Legend
Alt 1 Shoreline (2034)
WOP Shoreline (2034)
Initial Shoreline (2018)
Design Shoreline
Existing Structures
0
250
500
1,000
Feet
N
W
E
S


Ramp
Cross Bay Bridge
Beach Channel
Old Beach 86th
Beach 88th
Beach 87th
Beach 86th
Beach 85th
Java
Ramp
Beach 84th
Beach 83rd
Beach 82nd
Beach Channel
Rockaway
Rockaway
Beach 81st
Beach 80th
Beach 79th
Beach 76th
Beach 75th
Beach 73rd
Schooner
Coral Reef
Catamaran
Meridian
Beach 90th
Beach 89th
Gull
Beach 92nd
Alley
Beach 91st
Unnamed Street
Ruland
Beach 88th
Beach 86th
Rockaway Beach
Parking Lot
Beach 77th
Parking Lot
Beach 74th
Spinnaker
Water
Beach Front
Shore Front
De Sota
Beach 96th
Beach 97th
Beach 98th
Cross Bay
Beach 95th
Beach 94th
Holland
Beach 92nd
Beach 91st
Beach 90th
Holland
Beach 86th
Beach 84th
Unnamed Street
Shore Front
Beach Channel
Seaside
Constance
Ida
Marie
Beach 104th
Rockaway
Steeplechase
Dalys
Alley
Beach 99th
Beach 101st
Louise
Beach 100th
Beach 96th
Beach 97th
Cross Bay
Beach 93rd
Shore Front
Ramp
Rockaway Beach
Beach 102
Beach 105th
Beach 102nd
Boardwalk
Reach 4
Reach 5
Legend
Alt 1 Shoreline (2034)
WOP Shoreline (2034)
Initial Shoreline (2018)
Design Shoreline
Existing Structures
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250
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1,000
Feet
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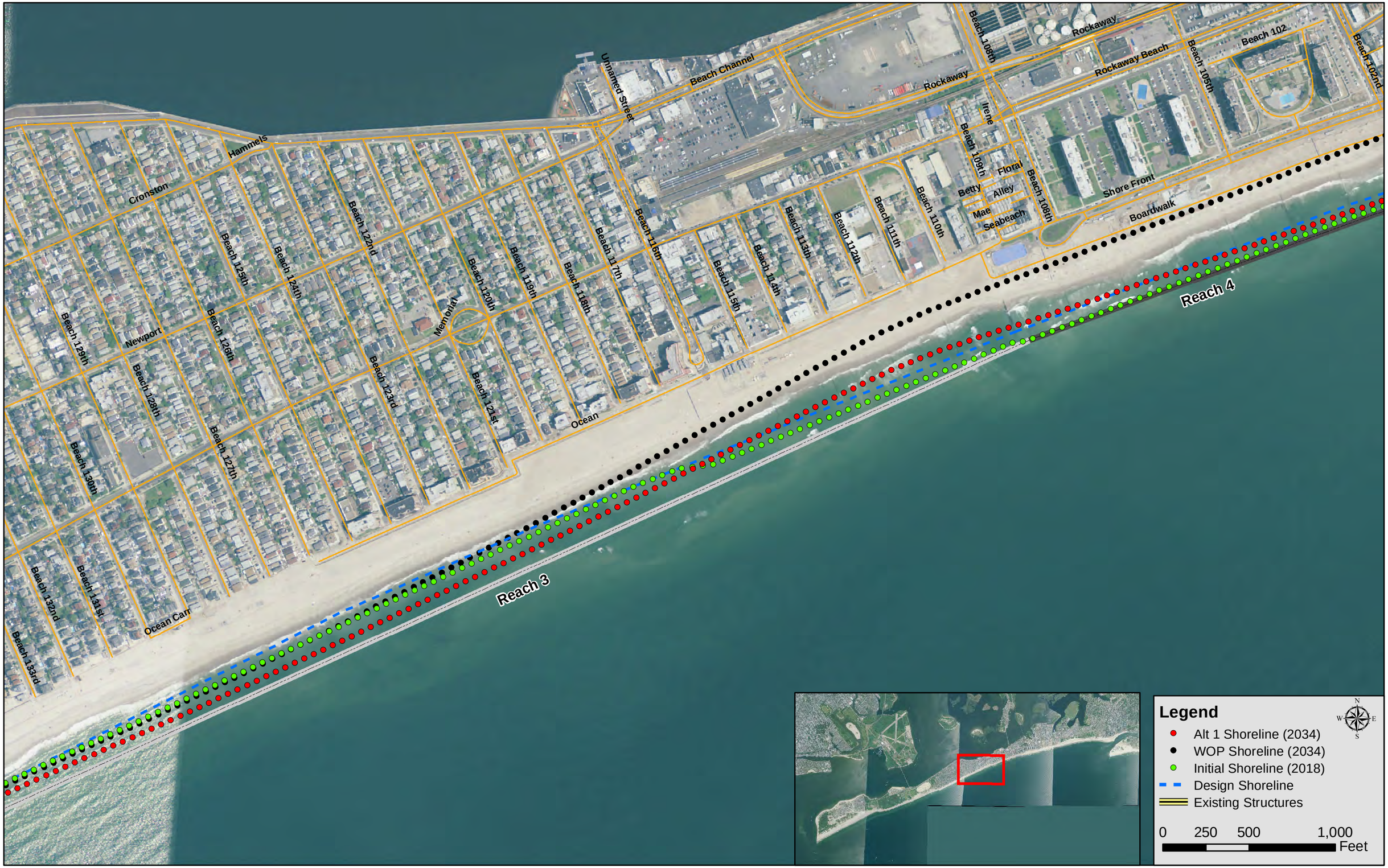
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Beach Channel
Rockaway
Beach 108th
Rockaway
Rockaway Beach
Beach 105th
Beach 102
Beach 102nd
Irene
Beach 109th
Floral
Alley
Betty
Mae
Seabeach
Beach 108th
Shore Front
Boardwalk
Beach 110th
Beach 111th
Beach 112th
Beach 113th
Beach 114th
Beach 115th
Beach 116th
Beach 117th
Beach 118th
Beach 119th
Beach 120th
Memorial
Beach 121st
Beach 123rd
Ocean
Hammels
Cronston
Beach 122nd
Beach 125th
Beach 124th
Beach 129th
Newport
Beach 126th
Beach 128th
Beach 127th
Beach 130th
Beach 132nd
Beach 131st
Beach 133rd
Ocean Cam
Reach 4
Reach 3
Legend
Alt 1 Shoreline (2034)
WOP Shoreline (2034)
Initial Shoreline (2018)
Design Shoreline
Existing Structures
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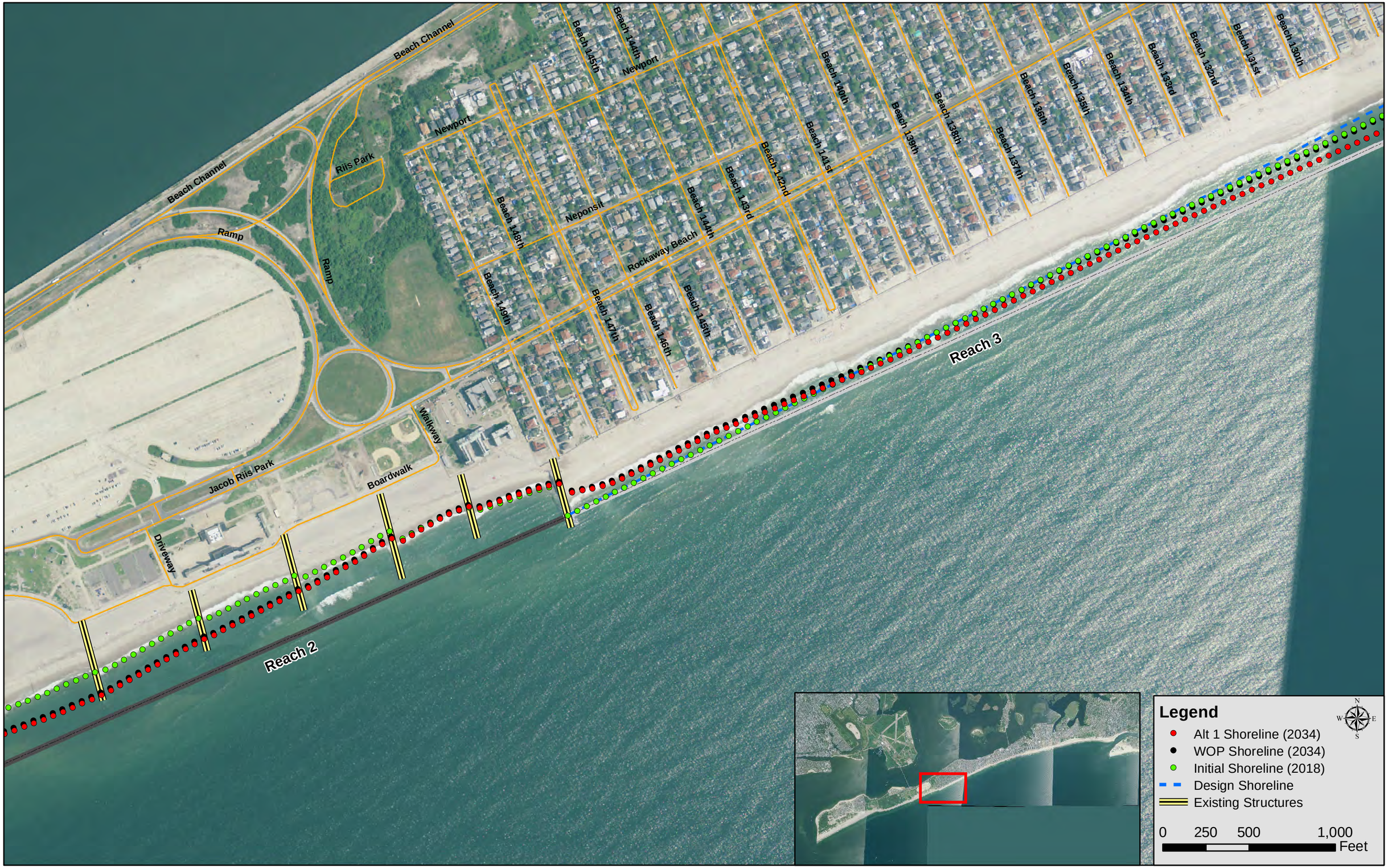
Beach Channel
Beach Channel
Riis Park
Ramp
Ramp
Newport
Newport
Beach 145th
Beach 144th
Beach 148th
Beach 149th
Neponsit
Beach 147th
Beach 146th
Rockaway Beach
Beach 144th
Beach 143rd
Beach 142nd
Beach 141st
Beach 140th
Beach 139th
Beach 138th
Beach 137th
Beach 136th
Beach 135th
Beach 134th
Beach 133rd
Beach 132nd
Beach 131st
Beach 130th
Reach 3
Walkway
Jacob Riis Park
Boardwalk
Driveway
Reach 2
Legend
Alt 1 Shoreline (2034)
WOP Shoreline (2034)
Initial Shoreline (2018)
Design Shoreline
Existing Structures
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Bayside
Seabreeze
Seabreeze
Roxbury
Bayside
Alley
Park End
Alley
Unnamed Street
Ramp
Beach Channel
Walkway
Jacob Riis Park
Driveway
Boardwalk
Browns
Creekside
Alley
Roxbury
Alley
Alley
Alley
Harmony
Oceanview
Alley
Beach 178
Hillside
Alley
Church
Parking Lot
Beach 184th
Rockaway Point
Rockaway Point
Unnamed Street
Hero
Steele
Steele
Unnamed Street
Barret
Ramp
Heinzelman
Murray
Davis
Unnamed Street
Unnamed Street
Beach 169th
Unnamed Street
East
Unnamed Street
Marshall
Center
Range
Hero
Unnamed Street
Unnamed Street
Shore
Reach 2
Legend
Alt 1 Shoreline (2034)
WOP Shoreline (2034)
Initial Shoreline (2018)
Design Shoreline
Existing Structures
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Point Breeze
Hillcrest
8th
Beach 208th
Ocean
Beach 207th
10th
Alley
9th
Arcadia
Bedford
Beach 203
Courtney
Manville
Neptune
Thetford
Doris
Reid
Beach 201st
Janet
Graham
Oceanside
Walkway
Essex
7th
Lincoln
Kildare
Jamaica
Irving
Hudson
Gotham
Fulton
Beach 207
Atlantic
Devon
Brant
Breezy Point
Beach 193rd
Beach 193
West
Range
Shore
Reach 2
Reach 1
Legend
Alt 1 Shoreline (2034)
WOP Shoreline (2034)
Initial Shoreline (2018)
Design Shoreline
Existing Structures
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Feet
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Reach 1

Legend

- Alt 1 Shoreline (2034)
- WOP Shoreline (2034)
- Initial Shoreline (2018)
- Design Shoreline
- Existing Structures

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0 250 500 1,000
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GENESIS-T ALTERNATIVE 2 – BEACH RESTORATION + REDUCED EROSION CONTROL


Norton
Hantz
Beach Channel
Beach 48th
Beach 47th
Beach 46th
Beach 45th
Beach 44th
Beach 43rd
Beach 42nd
Beach 41st
Beach 40th
Beach 39th
Beach 38th
Beach 37th
Beach 36th
Beach Channel
Ramp
Rockaway
Edgemere
Deerfield
Beach 31st
Beach 30th
Beach 29th
Beach 28th
Beach 27th
Beach 26th
Camp
Wavecrest
Edgemere
Highland
Fernside
Ramp
Beach 20th
Beach 19th
Beach 17th
Seagirt
Beach 49th
Rockaway Beach
Rockaway
Beach 48th
Beach 47th
Beach 46
Beach 46th
Beach 45thWalkway
Beach 44th
Beach 43rd
Beach 42nd
Beach 40th
Beach 37th
Beach 36
Beach 35th
Beach 34
Beach 33
Beach 32nd
Beach 31st
Beach 30th
Beach 29th
Beach 28th
Beach 27th
Beach 26th
Beach 25th
Beach 24th
Alley
Lewmay
Surf
Spray View
Spray View
Spray View
Boardwalk
Parking Lot
Parking Lot
Parking Lot
Parking Lot
Parking Lot
Driveway
Walkway
Reach 5
Reach 6
Legend
Alt 2 Shoreline (2034)
Alt 2 Structures
WOP Shoreline (2034)
Initial Shoreline (2018)
Design Shoreline
Existing Structures
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Feet


Thursby
Hessler
Gouverneur
Amstel
Beach 72nd
Beach 73rd
Beach 74th
Beach 75th
Beach 75
Beach 76th
Beach 75th
Ramp
Ramp
Beach 70th
Beach 69th
Beach 68th
Beach 67th
Beach 66th
Beach 65th
Beach 64th
Beach 63rd
Beach 62nd
Beach Channel
Arverne
Rockaway Beach
Beach 57th
Beach 56th
Beach 54th
Beach 53rd
Driveway
Beach 52nd
Beach 50
Beach 49th
Beach 48th
Beach 47th
Beach 46th
Reinhardt
Rockaway Beach
Rockaway
Edgemere
Beach 44th
Beach 54
Beach 55th
Beach 56th
Beach 56th
Beach 56th
Beach 54th Walkway
Beach 52nd Walkway
Beach 51 Walkway
Beach 50 Walkway
Beach 49th
Beach 48th
Beach 47th
Beach 46
Beach 46th
Beach 45th
Larkin
Larkin
Larkin
Beach 62nd
Beach 61st
Beach 60th
Beach 59th
Beach Front
Shore Front
Boardwalk
Nautical
Coral Reef
Schooner
Catamaran
Lighthouse
Beach 74th
Parking Lot
Spinnaker
Sandy Dune
Meridian
Shore Front
Shore Front
Surf
Ocean
Seaspray
Beach
Beach Front
Reach 5
Legend
Alt 2 Shoreline (2034)
Alt 2 Structures
WOP Shoreline (2034)
Initial Shoreline (2018)
Design Shoreline
Existing Structures
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Feet
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Ramp
Cross Bay Bridge
Beach Channel
Old Beach 86th
Beach 88th
Beach 87th
Beach 86th
Beach 85th
Java
Ramp
Beach 84th
Beach 83rd
Beach 82nd
Beach Channel
Rockaway
Rockaway
Beach 81st
Beach 80th
Beach 79th
Beach 76th
Beach 75th
Beach 73rd
Schooner
Coral Reef
Catamaran
Meridian
Beach 90th
Beach 89th
Gull
Alley
Beach 91st
Beach 92nd
Unnamed Street
Ruland
Beach 88th
Beach 86th
Rockaway Beach
Parking Lot
Beach 77th
Parking Lot
Beach 74th
Spinnaker
Water
Beach Front
Shore Front
De Sota
Beach 96th
Beach 97th
Beach 98th
Cross Bay Beach 95th
Holland
Beach 90th
Beach 84th
Unnamed Street
Beach 86th
Beach 94th
Beach 92nd
Beach 91st
Holland
Cross Bay
Beach 93rd
Shore Front
Beach Channel
Seaside
Constance
Ida
Marie
Beach 104th
Rockaway
Dalys
Steeplechase
Alley
Beach 99th
Beach 101st
Louise
Beach 100th
Ramp
Rockaway Beach
Beach 102
Beach 105th
Beach 102nd
Boardwalk
Reach 4
Reach 5
Legend
Alt 2 Shoreline (2034)
Alt 2 Structures
WOP Shoreline (2034)
Initial Shoreline (2018)
Design Shoreline
Existing Structures
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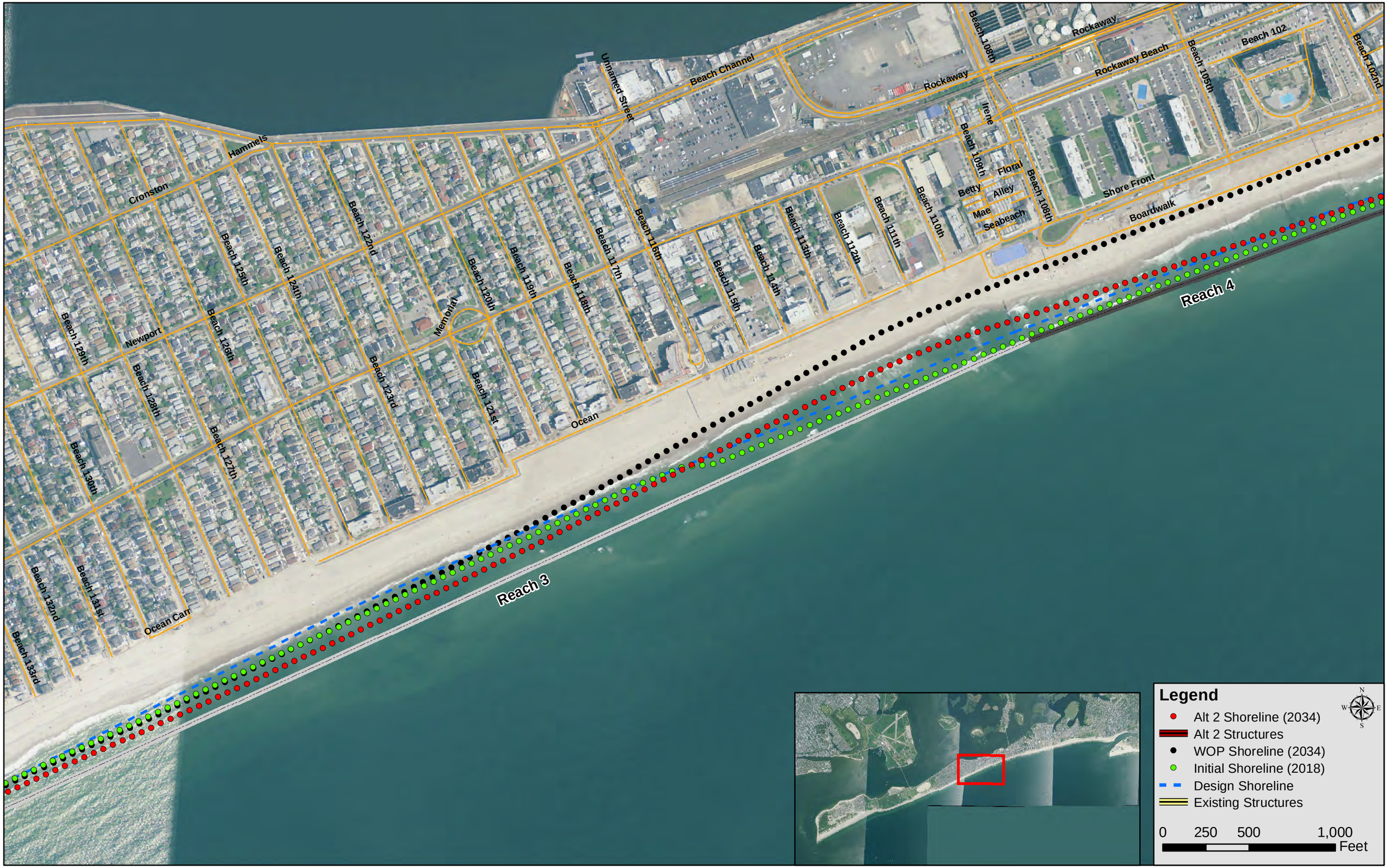
Beach Channel
Unnamed Street
Hammels
Cronston
Newport
Rockaway
Rockaway Beach
Beach 102
Beach 102nd
Beach 105th
Beach 108th
Irene
Beach 109th
Floral
Alley
Betty
Mae
Seabeach
Beach 110th
Beach 111th
Beach 112th
Beach 113th
Beach 114th
Beach 115th
Beach 116th
Beach 117th
Beach 118th
Beach 119th
Beach 120th
Memorial
Beach 121st
Beach 122nd
Beach 123rd
Beach 124th
Beach 125th
Beach 126th
Beach 127th
Beach 128th
Beach 129th
Beach 130th
Beach 131st
Beach 132nd
Beach 133rd
Shore Front
Boardwalk
Ocean
Ocean Cam
Reach 4
Reach 3
Legend
Alt 2 Shoreline (2034)
Alt 2 Structures
WOP Shoreline (2034)
Initial Shoreline (2018)
Design Shoreline
Existing Structures
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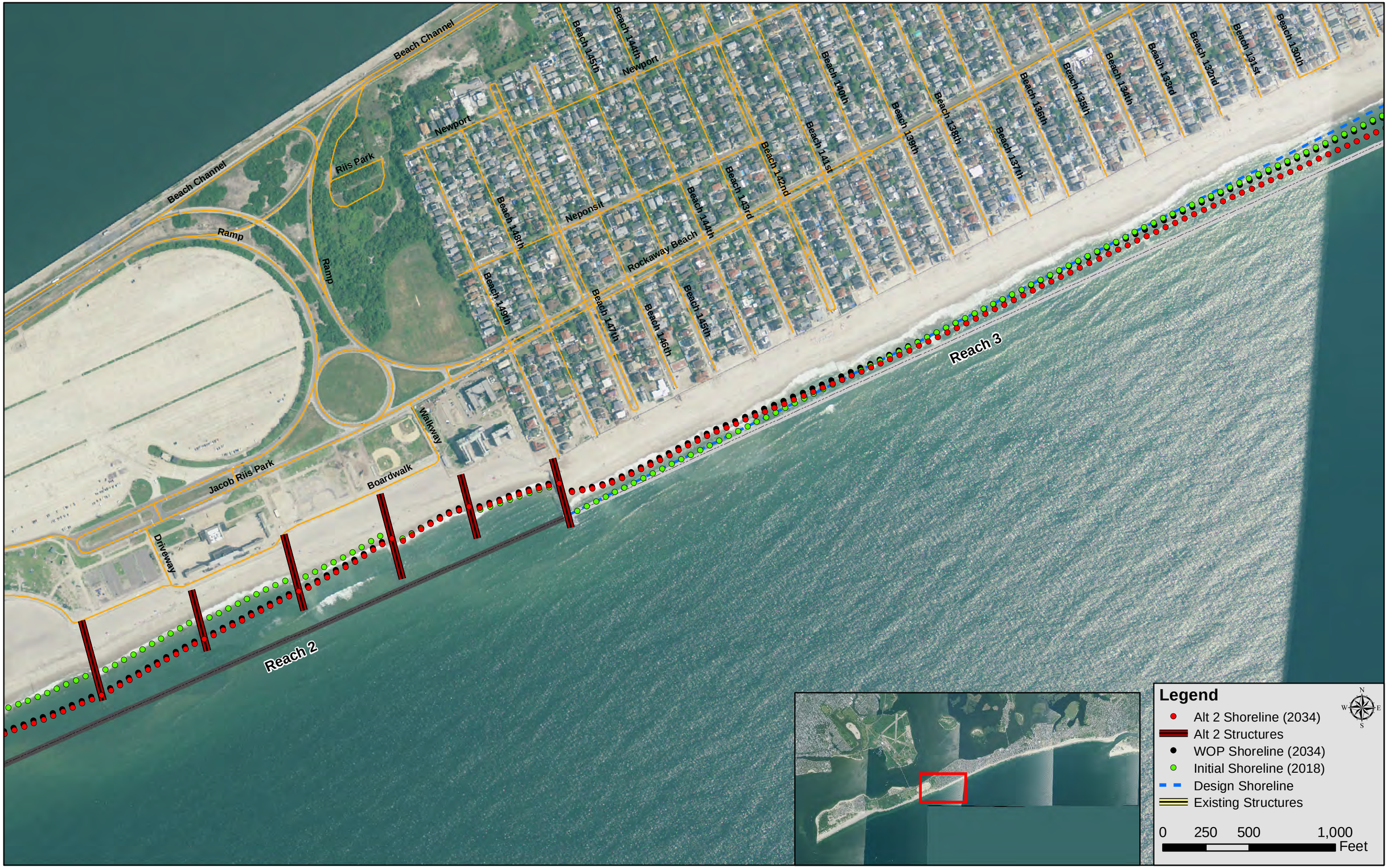
Beach Channel
Beach Channel
Riis Park
Ramp
Ramp
Beach 145th
Beach 144th
Newport
Newport
Beach 140th
Beach 139th
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Beach 137th
Beach 136th
Beach 135th
Beach 134th
Beach 133rd
Beach 132nd
Beach 131st
Beach 130th
Beach 148th
Neponsit
Beach 143rd
Beach 142nd
Beach 141st
Beach 144th
Rockaway Beach
Beach 149th
Beach 147th
Beach 146th
Beach 145th
Walkway
Jacob Riis Park
Boardwalk
Driveway
Reach 3
Reach 2
Legend
Alt 2 Shoreline (2034)
Alt 2 Structures
WOP Shoreline (2034)
Initial Shoreline (2018)
Design Shoreline
Existing Structures
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Feet


Bayside
Seabreeze
Seabreeze
Roxbury
Bayside
Alley
Park End
Alley
Unnamed Street
Ramp
Beach Channel
Walkway
Jacob Riis Park
Driveway
Boardwalk
Browns
Creekside
Alley
Roxbury
Alley
Alley
Alley
Harmony
Oceanview
Alley
Beach 178
Hillside
Alley
Church
Parking Lot
Beach 184th
Rockaway Point
Rockaway Point
Unnamed Street
Hero
Steele
Steele
Unnamed Street
Barret
Ramp
Heinzelman
Murray
Davis
Unnamed Street
Unnamed Street
Beach 169th
Unnamed Street
East
Unnamed Street
Marshall
Center
Hero
Unnamed Street
Unnamed Street
Unnamed Street
Shore
Range
Reach 2
Legend
Alt 2 Shoreline (2034)
Alt 2 Structures
WOP Shoreline (2034)
Initial Shoreline (2018)
Design Shoreline
Existing Structures
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Point Breeze
Hillcrest
8th
Beach 208th
Ocean
Beach 207th
Alley
10th
9th
Arcadia
Bedford
Beach 203
Courtney
Manville
Neptune
Thetford
Doris
Janet
Reid
Beach 201st
Beach 193rd
West
Range
Shore
Oceanside
7th
Essex
Walkway
Graham
Lincoln
Kildare
Jamaica
Irving
Hudson
Gotham
Fulton
Beach 207
Atlantic
Devon
Brant
Breezy Point
Reach 2
Reach 1
Legend
Alt 2 Shoreline (2034)
Alt 2 Structures
WOP Shoreline (2034)
Initial Shoreline (2018)
Design Shoreline
Existing Structures
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Reach 1
Legend
Alt 2 Shoreline (2034)
Alt 2 Structures
WOP Shoreline (2034)
Initial Shoreline (2018)
Design Shoreline
Existing Structures
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Feet

GENESIS-T ALTERNATIVE 3 – BEACH RESTORATION + INCREASED EROSION CONTROL


Norton
Hantz
Beach 48th
Beach 47th
Beach Channel
Beach 46th
Beach 45th
Beach 44th
Beach 43rd
Beach 42nd
Beach 41st
Beach 40th
Beach 39th
Beach 38th
Beach 37th
Beach 36th
Beach Channel
Ramp
Rockaway
Edgemere
Deerfield
Beach 31st
Beach 30th
Beach 29th
Beach 28th
Beach 27th
Beach 26th
Camp
Wavlean
Edgemere
Highland
Fernside
Ramp
Crest
Beach 20th
Beach 19th
Beach 17th
Seagirt
Beach 49th
Rockaway Beach
Rockaway
Beach 48th
Beach 47th
Beach 46
Beach 46th
Beach 45thWalkway
Beach 44th
Beach 43rd
Beach 42nd
Beach 40th
Beach 37th
Beach 36
Beach 35th
Beach 34
Beach 33
Beach 32nd
Beach 31st
Lewmay
Beach 31st
Beach 30th
Beach 29th
Beach 28th
Beach 28th
Beach 27th
Beach 26th
Alley
Beach 25th
Beach 24th
Parking Lot
Parking Lot
Parking Lot
Parking Lot
Parking Lot
Driveway
Walkway
Surf
Spray View
Spray View
Spray View
Boardwalk
Reach 5
Reach 6
Legend
Alt 3 Shoreline (2034)
Alt 3 Structures
WOP Shoreline (2034)
Initial Shoreline (2018)
Design Shoreline
Existing Structures
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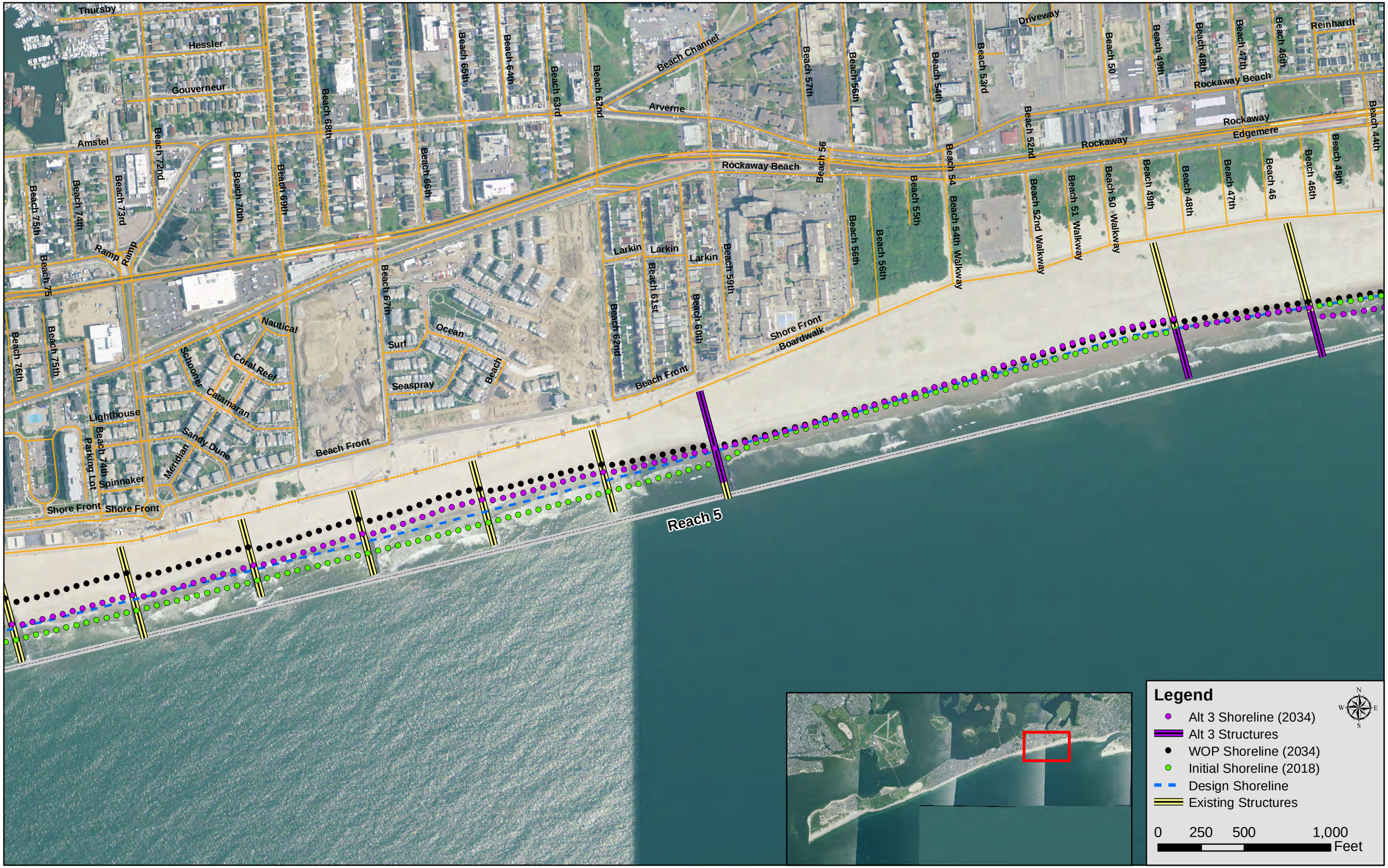
Feet



Thursby
Hessler
Gouverneur
Amstel
Beach 72nd
Beach 68th
Beach 65th
Beach 64th
Beach 63rd
Beach 62nd
Beach Channel
Arverne
Beach 57th
Beach 56th
Beach 54th
Beach 53rd
Driveway
Beach 50
Beach 49th
Beach 48th
Beach 47th
Beach 46th
Reinhardt
Rockaway Beach
Rockaway
Edgemere
Beach 44th
Beach 52nd
Beach 56
Beach 54
Beach 55th
Beach 54th Walkway
Beach 52nd Walkway
Beach 51 Walkway
Beach 50 Walkway
Beach 49th
Beach 48th
Beach 47th
Beach 46
Beach 46th
Beach 45th
Beach 75th
Beach 74th
Beach 73rd
Beach 70th
Beach 69th
Beach 66th
Ramp
Ramp
Beach 75
Beach 76th
Beach 75th
Larkin
Larkin
Larkin
Beach 61st
Beach 60th
Beach 59th
Beach 62nd
Beach 67th
Nautical
Ocean
Surf
Beach
Seaspray
Beach Front
Shore Front
Boardwalk
Schooner
Coral Reef
Catamaran
Lighthouse
Sandy Dune
Beach Front
Beach 74th
Parking Lot
Meridian
Spinnaker
Shore Front
Shore Front
Reach 5
Legend
Alt 3 Shoreline (2034)
Alt 3 Structures
WOP Shoreline (2034)
Initial Shoreline (2018)
Design Shoreline
Existing Structures
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Legend
Alt 3 Shoreline (2034)
Alt 3 Structures
WOP Shoreline (2034)
Initial Shoreline (2018)
Design Shoreline
Existing Structures
0 250 500 1,000
Feet
Reach 4
Reach 5
Beach Channel
Rockaway Beach
Shore Front
Boardwalk
Rockaway
Ramp
Cross Bay Bridge
De Sota
Holland
Ruland
Gull
Java
Beach 104th
Beach 105th
Beach 102
Beach 101st
Beach 102nd
Beach 100th
Beach 99th
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Beach 96th
Beach 95th
Beach 94th
Beach 93rd
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Beach 91st
Beach 90th
Beach 89th
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Beach 87th
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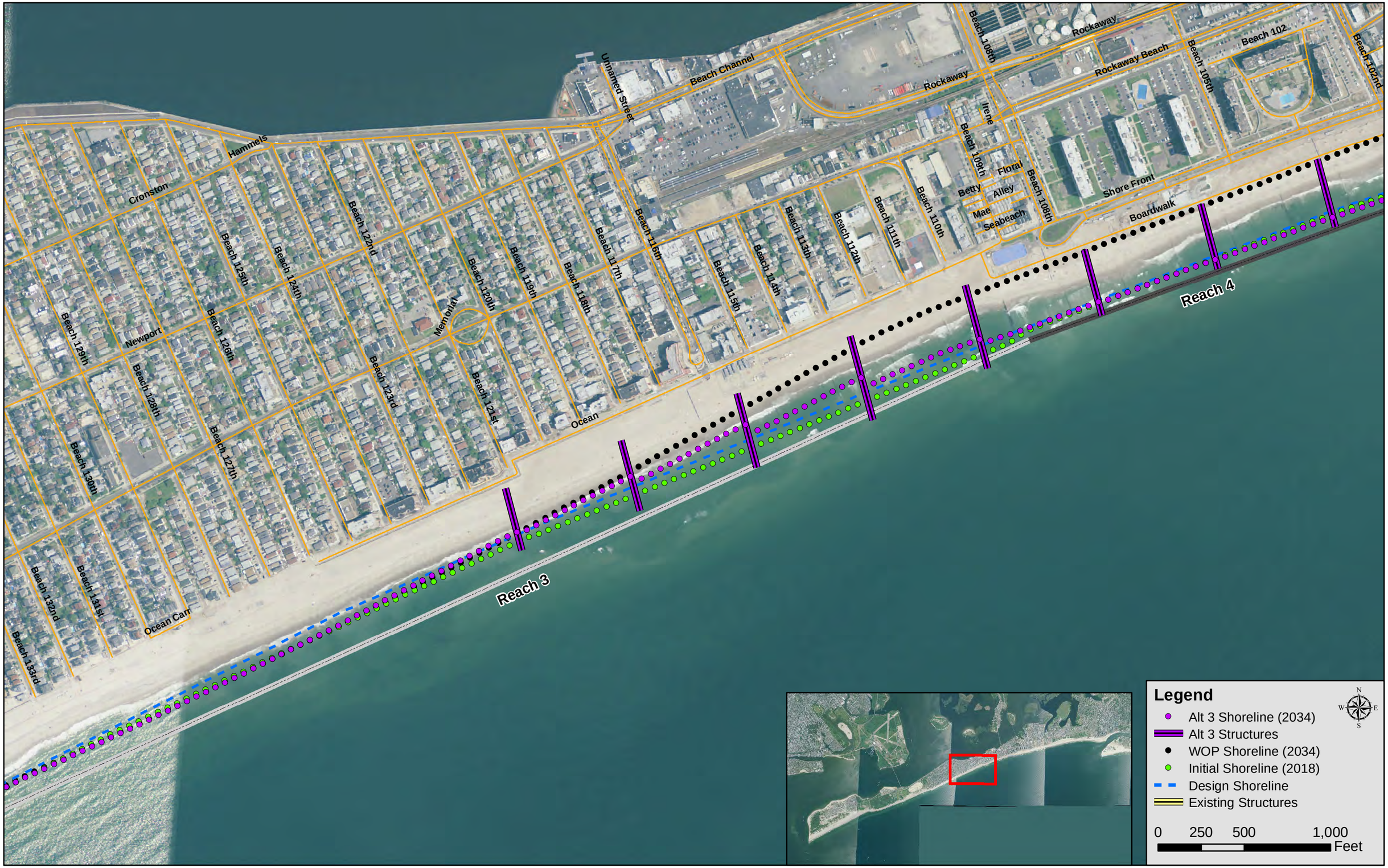
Beach 83rd
Beach 82nd
Beach 81st
Beach 80th
Beach 79th
Beach 77th
Beach 76th
Beach 75th
Beach 74th
Beach 73rd
Parking Lot
Unnamed Street
Alley
Cross Bay
Seaside
Constance
Ida
Marie
Dalys
Steeplechase
Louise
Old Beach 88th
Spinnaker
Water
Beach Front
Meridian
Catamaran
Schooner
Coral Reef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Beach Channel
Rockaway
Beach 108th
Rockaway
Rockaway Beach
Beach 105th
Beach 102
Beach 102nd
Irene
Beach 109th
Floral
Alley
Betty
Mae
Seabeach
Beach 108th
Shore Front
Boardwalk
Beach 110th
Beach 111th
Beach 112th
Beach 113th
Beach 114th
Beach 115th
Beach 116th
Beach 117th
Beach 118th
Beach 119th
Beach 120th
Memorial
Beach 121st
Beach 123rd
Beach 122nd
Beach 124th
Beach 125th
Beach 126th
Beach 127th
Beach 128th
Beach 129th
Beach 130th
Beach 131st
Beach 132nd
Beach 133rd
Hammels
Cronston
Newport
Ocean
Ocean Cam
Reach 4
Reach 3
Legend
Alt 3 Shoreline (2034)
Alt 3 Structures
WOP Shoreline (2034)
Initial Shoreline (2018)
Design Shoreline
Existing Structures
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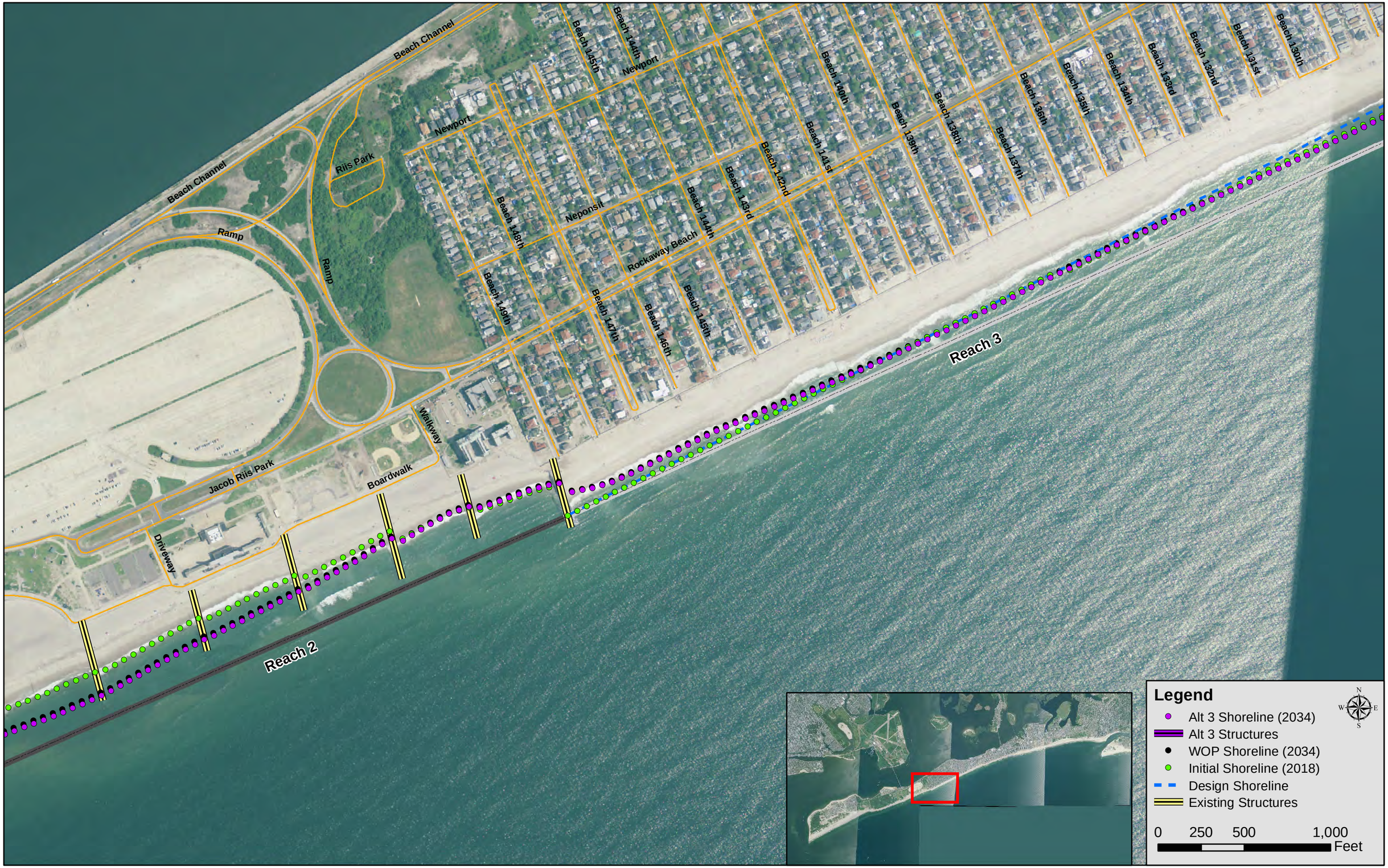
Feet


Beach Channel
Beach Channel
Riis Park
Ramp
Ramp
Beach 145th
Beach 144th
Newport
Newport
Beach 148th
Neponsit
Beach 149th
Beach 147th
Beach 146th
Beach 144th
Beach 143rd
Beach 142nd
Rockaway Beach
Beach 145th
Beach 141st
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Beach 139th
Beach 138th
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Beach 136th
Beach 135th
Beach 134th
Beach 133rd
Beach 132nd
Beach 131st
Beach 130th
Walkway
Jacob Riis Park
Boardwalk
Driveway
Reach 3
Reach 2
Legend
Alt 3 Shoreline (2034)
Alt 3 Structures
WOP Shoreline (2034)
Initial Shoreline (2018)
Design Shoreline
Existing Structures
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Seabreeze
Roxbury
Bayside
Alley
Park End
Hillside
Church
Unnamed Street
Ramp
Beach Channel
Walkway
Jacob Riis Park
Driveway
Boardwalk
Barret
Steele
Heinzelman
Davis
Beach 169th
Murray
Hero
Beach 178
Harmony
Oceanview
Browns
Creekside
Parking Lot
Beach 184th
Rockaway Point
East
Marshall
Center
Range
Shore
Reach 2
Legend
Alt 3 Shoreline (2034)
Alt 3 Structures
WOP Shoreline (2034)
Initial Shoreline (2018)
Design Shoreline
Existing Structures
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Point Breeze
Hillcrest
8th
Beach 208th
Ocean
Beach 207th
10th
Alley
9th
Arcadia
Bedford
Beach 203
Courtney
Alley
Manville
Neptune
Thetford
Doris
Reid
Beach 201st
Janet
Graham
Oceanside
Walkway
Essex
7th
Lincoln
Kildare
Jamaica
Irving
Hudson
Gotham
Fulton
Beach 207
Atlantic
Devon
Brant
Breezy Point
Beach 193rd
Beach 193
West
Range
Shore
Reach 2
Reach 1
Legend
Alt 3 Shoreline (2034)
Alt 3 Structures
WOP Shoreline (2034)
Initial Shoreline (2018)
Design Shoreline
Existing Structures
0
250
500
1,000
Feet
N
W
E
S

Reach 1

Legend

- Alt 3 Shoreline (2034)
- Alt 3 Structures
- WOP Shoreline (2034)
- Initial Shoreline (2018)
- Design Shoreline
- Existing Structures

N
W E
S

0 250 500 1,000
Feet