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Wave Runup and Overtopping Analysis for South San Francisco Bay Shoreline Study

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Draft

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Draft report

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Under South San Francisco Bay Shoreline Study

Monitored by Coastal and Hydraulics Laboratory
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Abstract: This report describes the wave runup and overtopping analysis of shoreline levees in South San Francisco Bay. Overtopping of earthen levees produces fast, turbulent flow velocities that can damage the protective grass covering and expose the underlying soil to erosion. A rapid loss of unprotected soil during an overtopping event could lead to loss of levee crest elevation, and possibly breaching of the protective structure. Assessment of levee reliability requires knowledge of probable storm scenarios and estimates of hydrodynamic parameters, including shear stress and overtopping.

Several Boussinesq-equation models were considered for the assessment of wave runup and overtopping, including COULWAVE and BOUSS-2D, but BOUSS-1D was finally selected due to the large number of simulations required. A total of 312 combinations of predicted tidal surge and wind-generated wave conditions (input parameters) were considered: significant wave height, wave peak period, and water level. The differences between Year-0 and Year-50 simulations are reflected as bathymetry variations. Maximum overtopping values for Year-0 ($WL_{year\text{-}0}$ = 12 ft) ranged from 1.4 to 17.2 cfs/ft, while for Year 50 ($WL_{year\text{-}0}$ = 14 ft), maximum overtopping values ranged from 7.1 to 36.6 cfs/ft.

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Preface

The South San Francisco Bay Shoreline Study wave modeling was performed in support of the San Francisco District, U.S. Army Corps of Engineers. The purpose was to generate wave information to support water level, levee breaching, and sediment transport components of the shoreline study. This report summarizes the wave modeling component of the study.

This report was prepared by Dr. Norberto C. Nadal and Dr. Jeffrey A. Melby, Harbors, Entrances, and Structures Branch of ERDC-CHL, under the supervision of Ms. Jackie Pettway, Chief of Harbors, Entrances, and Structures Branch, Dr. Jack Davis, Chief of Navigation Division, Dr. Rose Kress, Deputy Director of CHL, and Dr. William Martin, Director of CHL. COL Gary E. Johnston was Commander and Executive Director of ERDC, and Dr. James R. Houston was ERDC Director.

Unit Conversion Factors, Non-SI to SI Units of Measurement

Multiply	By	To Obtain
cubic feet	0.02831	cubic meters
cubic yards	0.7645549	cubic meters
feet	0.3048	meters
inches	0.0254	meters
miles (nautical)	1,852	meters
miles (U.S. statute)	1,609.347	meters
square miles (U.S. statute)	2.59	square kilometers
mbar	1.00	hPa

1 Introduction

The South San Francisco Bay Shoreline Study (SSFBSS) is a multipart flood control project that aims to update the level of flood protection along South Bay's southern shoreline, within the San Francisco Bay. This project intends to reduce tidal and fluvial damage for the counties of Alameda, Santa Clara, and San Mateo, in California. The SSFBSS is a joint cooperative effort between the U.S. Army Engineer Research and Development Center (USACE) and the U.S. Army Engineer District, San Francisco (CESPN). The task outlined in this report consisted of determining wave runup and overtopping on both intact and damaged levees in the South Bay area. The forcing was primarily from storm wave and surge conditions.

Flooding processes in the South Bay shoreline area are dominated by tidal levels within the bay. Coastal flooding can be exacerbated by other processes, such as: locally generated wind waves, runoff from nearby creeks, and surge from winter storms. Flooding from these processes can lead to overtopping of the protection levees and structural instability problems. A primary concern is that South Bay often exhibits standing wave characteristics where diurnal tides in South Bay are amplified by more than 50 percent relative to the tides at San Francisco Bay. The study area is also the receiving water body for numerous creeks that drain into tidal sloughs connected to the bay.

Wave Runup and Overtopping Analysis

This task consists in performing wave runup and overtopping analysis. Wave runup, wave overtopping, and steady flow overtopping are the main mechanisms that cause flooding, structural instability and erosion of levees. Time-dependent erosion of coastal structures is a relatively recent modeling capability with research being driven by the need for more rigorous risk evaluations (Melby 2008). Most of the previous work was focused on developing non-time-dependent empirical models. Little is known about the details of wave-induced erosion of levees.

Breaking wave runup and overtopping modeling is a developing field as well. Phase-resolving models, such as those based on Boussinesq and Reynolds-averaged Navier-Stokes equations, are being employed with some success (Melby et al 2009). However, computational demands from these models limit their effectiveness for risk-based studies. Presently, the most reasonable model for predicting runup and overtopping on steeply sloping coastal structures is based on Boussinesq equations.

2 Levee Overtopping and Breaching

Earthen levees are used extensively to protect populations and infrastructure against flooding. Ideally, all levees would have crest elevations with enough freeboard to prevent overtopping for any possible scenario. However, economics and other factors often dictate levees to be designed with lower crest elevations that allow for an acceptable risk of overtopping during extreme events.

As discussed by Hughes and Nadal (2009), damage to earthen levees can occur as a consequence of either:

a. wave-only overtopping when the water level is below the levee crest elevation,

b. storm surge overflow when the water level exceeds the crest elevation, or,

c. combination of both wave overtopping and surge overflow.

Overtopping of earthen levees produces fast, turbulent flow velocities on the landward-side slope that can damage the protective grass covering and expose the underlying soil to erosion. A rapid loss of unprotected soil during an overtopping event could lead to loss of levee crest elevation, and possibly breaching of the protective structure. Therefore, the crown and landward-side slopes of those levees that are at risk of overtopping are usually protected with some type of strengthening method such as turf reinforcement, soil strengthening, or hard armoring. Assessment of levee reliability requires knowledge of probable storm scenarios and estimates of hydrodynamic parameters, including shear stress and overtopping.

3 Numerical Modeling

Comparison of Boussinesq Models

Initially, horizontally two-dimensional (2DH) models were considered because a moderate number of design storms was expected (less than 20 storms). The space domain was discretized into 14 2DH grids and tests were completed using the 2DH Boussinesq models BOUSS-2D and COULWAVE, for the maximum expected water level and wave conditions for several domains. Both models solve the fully nonlinear Boussinesq equations as derived by Wei et al. (1995) using finite-difference method. BOUSS-2D is a phase-resolving wave model that includes the following phenomena: shoaling, refraction, diffraction, full/partial reflection and transmission, bottom friction, nonlinear wave-wave interactions, wave breaking and dissipation, wave runup and overtopping of structures, wave-current interaction, and wave-induced currents (Demirbilek et al., 2005).

One major difference is that BOUSS-2D uses a one-equation turbulence closure model to simulate wave breaking in the surf zone (Nwogu and Demirbilek, 2001) while COULWAVE uses an eddy viscosity approach similar to that of Kennedy et al. (2000). Preliminary tests showed the numerical model BOUSS-2D to be more stable than COULWAVE, but only for spectral unidirectional waves. Conversely, COULWAVE was more stable when running directional spectral waves. Even though COULWAVE was run on a cluster with 16 processors, it required about the same processing time as BOUSS-2D because it requires a higher resolution and much smaller time step for stability than BOUSS-2D. At this preliminary stage, COULWAVE was chosen because of the following advantages: (1) it can be run in parallel on a supercomputer, and (2) it allows for a tidal value to be specified therefore allowing the water level in the ponds to be different from the water level in the bay.

Decision to Switch from 2DH to 1DH

Historical storms were being hindcast in parallel to the development of the levee runup and overtopping models. Therefore it was decided to develop a lookup table approach for predicting wave runup and overtopping. In this process, the ranges of pertinent offshore hydrodynamic parameters are determined and a parameter space of all possible combinations of tide, surge and wave conditions are quantified. The offshore parameter space is then used as input to the Boussineq model. The hydrodynamics on the structure are then determined for the full range of possible offshore wave and water level conditions. Wave height (H_{m0}), peak wave period (T_p), and water level (WL) were identified as the most important variables to compose the parameter space. Because of the shallow depths and relatively uniform isobaths within the South Bay, the incoming wave direction was assumed to be relatively constant and of second order importance, and was therefore not included in the parameter space.

Because the storm wave conditions were primarily long-crested and shore normal, a decision was made to switch from the 2DH Boussinesq model to a vertically one-dimensional (1DH) model and to use a version of BOUSS-2D called BOUSS-1D. This decision was based on the large number of simulations which had to performed, the amount of time available, and the number of successful runs with the 2DH models. The main advantages of 1DH models are that they run much faster, and are much more stable; but the 1DH Boussinesq models do not include wave refraction and therefore must be oriented in the direction of the incoming wave. This requires changing the orientation of each 1D transect for different wave directions.

Input Parameters

The limits of the parameter space were established based on a sensitivity analysis of wave runup and overtopping estimates. This was done using the 1DH model BOUSS-1D on the parameter space to determine the resolution needed for each parameter.

The final parameter space was defined as follows:

a. H_{m0} = 0.4, 0.6, 0.8, 1.0 m (1.3, 2.0, 2.6, 3.3 ft)

b. T_p = 5, 6, and 7 sec

c. WL = 2.0 - 4.5 m (NAVD88) at 0.1 m intervals (6.6 - 14.8 ft)

Here wave height is the incident spectral significant wave height, H_{m0} = $4(m_0)^{1/2}$, where m_0 is the zeroth moment of the incident wave energy density spectrum ($m_n = \int f^n S(f)df$). The wave period is the peak wave period, T_p, corresponding to the inverse of the peak frequency of the incident wave spectrum, and WL is water level measured from NAVD88 datum.

The parameter permutations translate into a total of 312 combinations of wave and water levels conditions. It is clear that this would be too many simulations to perform using a 2DH model. The same parameter space is used for both Year-0 and Year-50 simulations. The differences between Year-0 and Year-50 are reflected in bathymetry changes, as explained in Section 5.

4 Location of Transects

The shoreline levees in the pond areas were divided into reaches based on regional uniformity of orientation, geometry and crest height. A representative transect was used to characterize each reach at the location of the lowest crest height along a given reach. The levee transects were then oriented into the approximate incoming wave direction. The incoming wave direction was calculated by assuming a wave angle perpendicular to offshore isobath (approximately 1 km from the levee) and simulating the wave refraction using the spectral wave model CMS-Wave (Lin et al, 2008).

Table 2
Levee Transects and their Locations

Transect ID	**X** (meters) *UTM NAD83*	**Y** (meters) *UTM NAD83*
1	580006.6	4146013
2	580299.2	4145656
3	580593.3	4145465
4	580856.6	4145446
5	581045.3	4145485
6	581271.9	4145444
7	581502.2	4145172
8	581711.7	4145245
9	581884.4	4145210
10	582026.0	4145208
11	582169.7	4145257
12	582313.3	4145273
13	582473.6	4145264
14	582595.6	4145192
15	582715.2	4145039
16	582794.6	4144917
17	582948.9	4144796
18	583073.9	4144810
19	583223.3	4144867
20	583381.0	4144918
21	583519.2	4144983
22	583686.3	4145029
23	583849.3	4145036
24	583986.5	4145098
25	584158.7	4145088
26	584327.1	4145116
27	584475.2	4145215
28	584649.2	4145320
29	584780.3	4145402
30	584955.4	4145441
31	585082.8	4145461
32	585210.2	4145487

Figure 1. Location of levee transects #1 through #32 (left to right).

5 Transect Profiles: Year-0 vs. Year-50

Plots of all 32 levee transects used for the numerical simulation are presented in *Appendix A*. These plots show how the levees were schematized in the BOUSS-1D model for Year-0, and also illustrate the modifications made to account for Year-50 bathymetry variations.

For Year-50, it was assumed that the effect of sea level rise will be the raising of the offshore transect by 0.72 feet. The fringe marshes would move up, but not the levees themselves. Therefore, the bathymetry was shifted up in the BOUSS-1D model, but levee cross-sections remained unchanged. This is based on the assumption that the existing profile below MHHW (7.5 ft NAVD) is in equilibrium for sediment supply and mobility of the sediments by waves and currents, as amended by the effects of the local vegetation and other biological stability effects.

6 Simulation Results

The results from BOUSS-1D were read and post-processed using MATLAB scripts. The following five hydrodynamic parameters were simulation outputs:

a. Wave transmission coefficient [-]

b. Shear stress duration (shear stress to storm duration ratio) [-]

c. Mean shear stress [psf]

d. Shear stress-1% (average of the highest 1% of the values) [psf]

e. Overtopping [cfs/ft]

Descriptions of these output parameters are presented in the following sections. The look-up tables developed for both Year-0 and Year-50 simulation results are presented in a *separate Excel file*.

Wave Transmission Coefficients

The wave transmission coefficient, K_t, is defined as the ration between transmitted and incident wave heights:

$$K_t = \frac{H_t}{H_{m0}} \tag{1}$$

where H_t = transmitted spectral significant wave height, which is computed on the landward side of the levee. The incident spectral significant wave height, H_{m0}, is introduced to the simulation 100 ft offshore the transect bathymetry.

Shear Stress

The momentum equation of the Boussinesq model includes two terms that dissipate energy: (1) a wave breaking term parameterized by an empirical eddy-viscosity coefficient ν, and (2) a bottom friction parameterized by the quadratic drag law with an empirical friction coefficient, f_w

$$\boldsymbol{u}_{at} + \ldots + \frac{1}{h+\eta}\nabla\left[\nu(h+\eta)\nabla\cdot\boldsymbol{u}_a\right] + \frac{1}{h+\eta} f_w \boldsymbol{u}_b \left|\boldsymbol{u}_b\right| = 0 \tag{2}$$

where

u_a = velocity at an arbitrary elevation

u_b = velocity at the seabed

h = water depth

η = water surface elevation

In post-breaking waves, a fraction of the dissipated wave energy would act to suspend and transport sediments. Hence, we have output an overall shear stress, τ, which does work to dissipate energy and includes contributions from both the bottom friction term and the wave breaking term:

$$\tau = \nabla\left[\nu(h+\eta)\nabla \cdot \boldsymbol{u}_a\right] + f_w \boldsymbol{u}_b \left|\boldsymbol{u}_b\right| \tag{3}$$

Wave-Only Overtopping

The BOUSS-1D model can provide wave overtopping estimates. However, when the water level is higher than the levee crest, overtopping is produced by a combination of surge overflow and waves which cannot be modeled with the BOUSS-1D model. This condition is known as negative freeboard. In order to provide overtopping estimates for this condition, empirical relationships were employed, as discussed in Section 7.

7 Negative Freeboard: Combined Overflow and Wave Overtopping

Combined overflow and wave overtopping has been much less studied than wave-only or surge-only overtopping. During this condition, every wave overtops the levee, and the peak instantaneous discharge can be several times the value of the steady overflow discharge. The overtopping flow is unsteady in time and spatially non-uniform. Examinations of combined overflow and wave overtopping include the European Overtopping Manual (Pullen et al, 2007) and a study by Reeve et al (2008), which employed a numerical wave flume based on the Reynolds-averaged Navier-Stokes equations.

The Overtopping Manual provides slightly revised versions of the equations developed by Schüttrumpf et al. (2001). Schüttrumpf et al developed equations for prediction of dimensionless average wave overtopping discharge based on laboratory experiments with zero freeboard only. However, the Overtopping Manual proposed that a rough approximation for combined overflow and wave overtopping could be determined by superimposing the average wave overtopping discharge for zero freeboard with the steady surge overtopping that would occur in the absence of waves.

One of the most recent and comprehensive laboratory studies was performed by Hughes and Nadal (2009); see also CHETN III-79 (Nadal and Hughes, 2009). In this study measurements acquired during 27 wave flume experiments of a levee with a trapezoidal cross section in a two-dimensional laboratory wave/flow flume at a nominal prototype-to-model length scale of 25-to-1.The overtopping data were used to develop empirical predictive equations for average overtopping discharge, cumulative distribution of instantaneous overtopping discharge, the distribution of individual overtopping wave volumes, and shear stresses on the landward-side slopes of overtopped levees.

Hughes and Nadal (2009) provided the following empirical equation to estimate average combined overflow and wave overtopping:

$$\frac{q_{ws}}{\sqrt{g\ H_{m0}^{3}}} = 0.034 + 0.53\left(\frac{-R_c}{H_{m0}}\right)^{1.58} \quad ; \quad R_c < 0 \tag{4}$$

where

q_{ws} = combined wave and surge overtopping

H_{m0} = significant wave height

R_c = negative freeboard.

The term R_c must be a negative number so the ratio in brackets will be positive. Eq. (4) was developed for an idealized levee with seaward-side levee slopes of 1-on-4.25 and landward-side slopes of 1-on-3. The applicability of this equation for different slopes is uncertain. Different seaward-side levee slopes could influence the wave overtopping, but these effects should decrease as surge level increases. Like with any other empirical equation, caution should be exercised when applying Eq. (4) outside laboratory test conditions.

8 Conclusions

After comparing the results for both the Year-0 and Year-50 results, the main findings are:

Shear Stress

For a given water level ($WL_{year\text{-}0} = WL_{year\text{-}50}$), the differences in shear stress can be significant for many of the levee transects. The raising of the bathymetry by 0.72 ft had a "smoothing" effect on the cross-section geometry, which lead to lesser shear stress magnitudes.

These comparisons are for identical water levels. When the actual sea level rise for Year-50 is considered (+0.72 ft WL), both the shear stress and overtopping magnitudes are considerably higher when compared to Year-0 results, particularly for water levels near levee crest elevation.

Wave Overtopping

For a given water level ($WL_{year\text{-}0} = WL_{year\text{-}50}$), the differences in overtopping are minimal.

a. In the case of wave-only overtopping (positive free board), a slight reduction in overtopping is observed in the Year-50 results.

b. In the case of overflow and wave overtopping (negative freeboard), the overtopping is the same, since only the negative freeboard magnitude in considered in Eq. (4).

Maximum overtopping values for Year-0 ($WL_{year\text{-}0}$ = 12 ft) ranged from 1.4 to 17.2 cfs/ft, corresponding to freeboards from -0.2 to -3.0, respectively. Similarly, for Year-50 ($WL_{year\text{-}0}$ = 14 ft), maximum overtopping values ranged from 7.1 to 36.6 cfs/ft.

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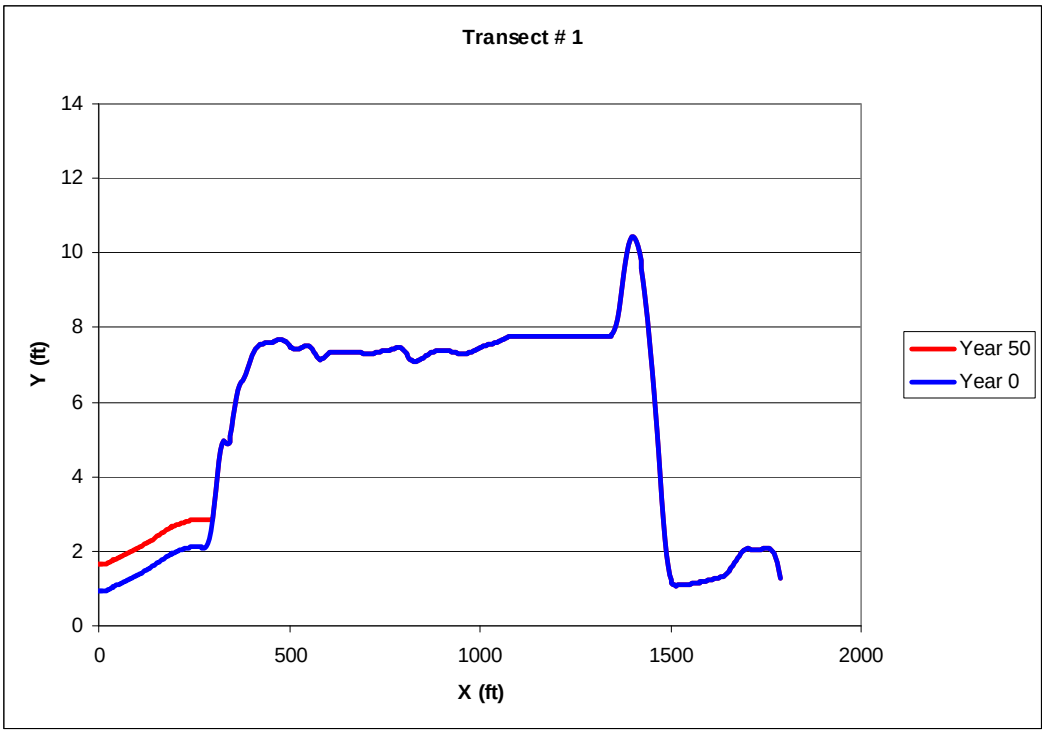
Appendix A: Transect Profiles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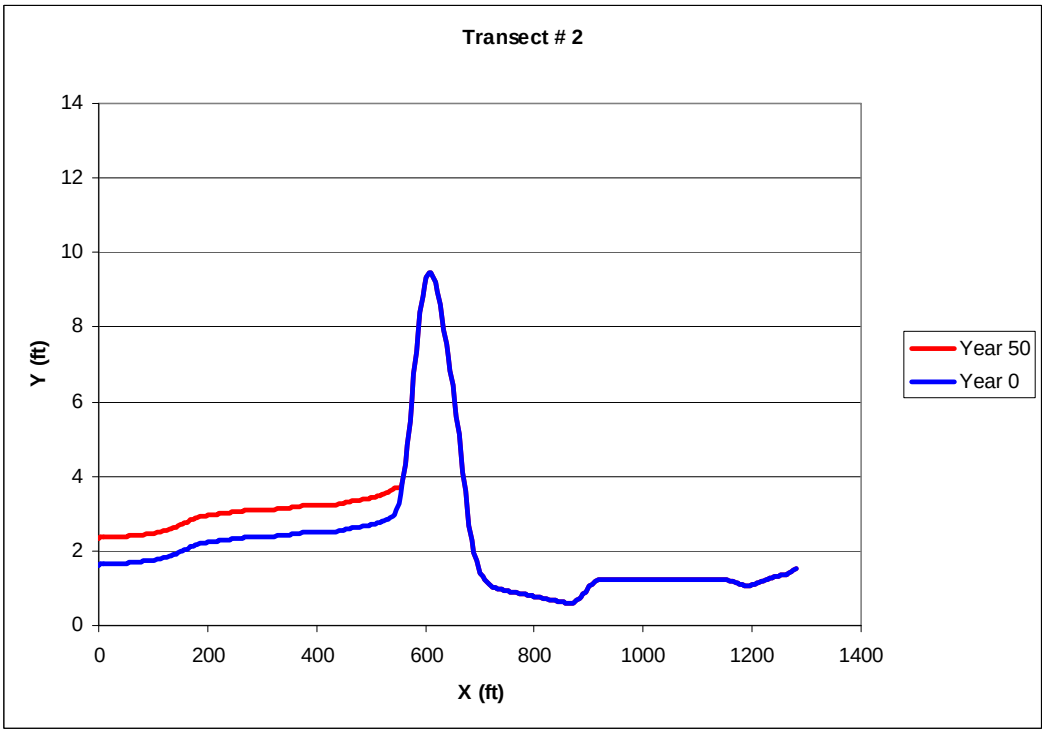
2
4
6
8
10
12
14
0
500
1000
1500
2000
Year 50
Year 0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X (ft)
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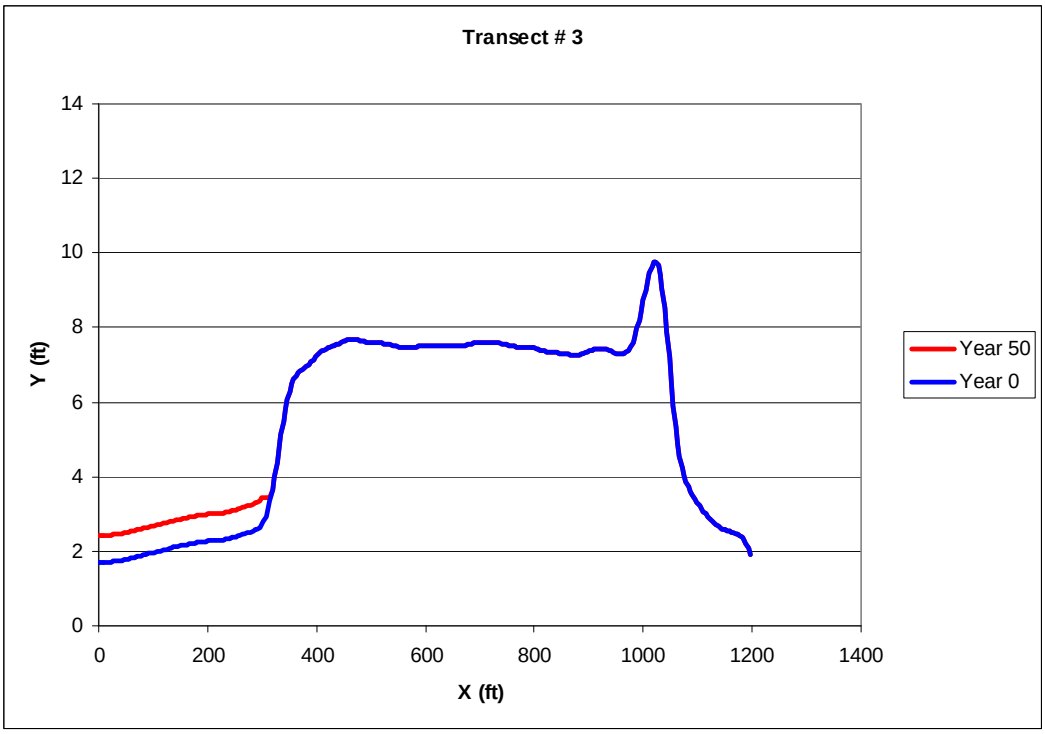
1000
1200
1400
Year 50
Year 0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14
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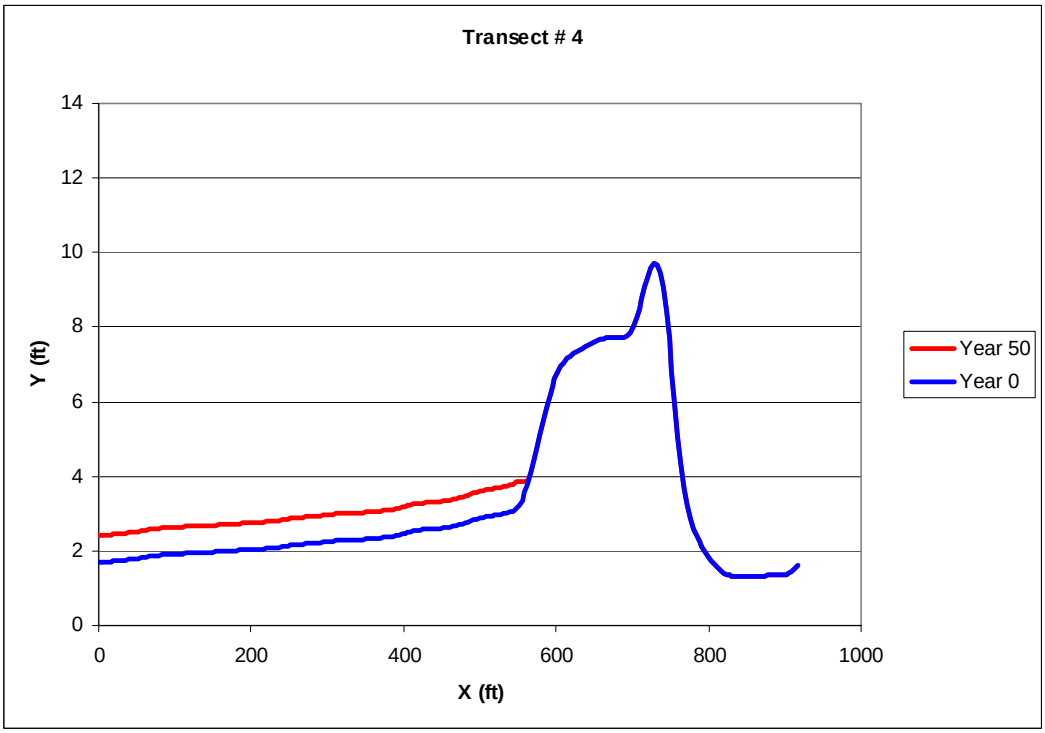
600
800
1000
1200
1400
Year 50
Year 0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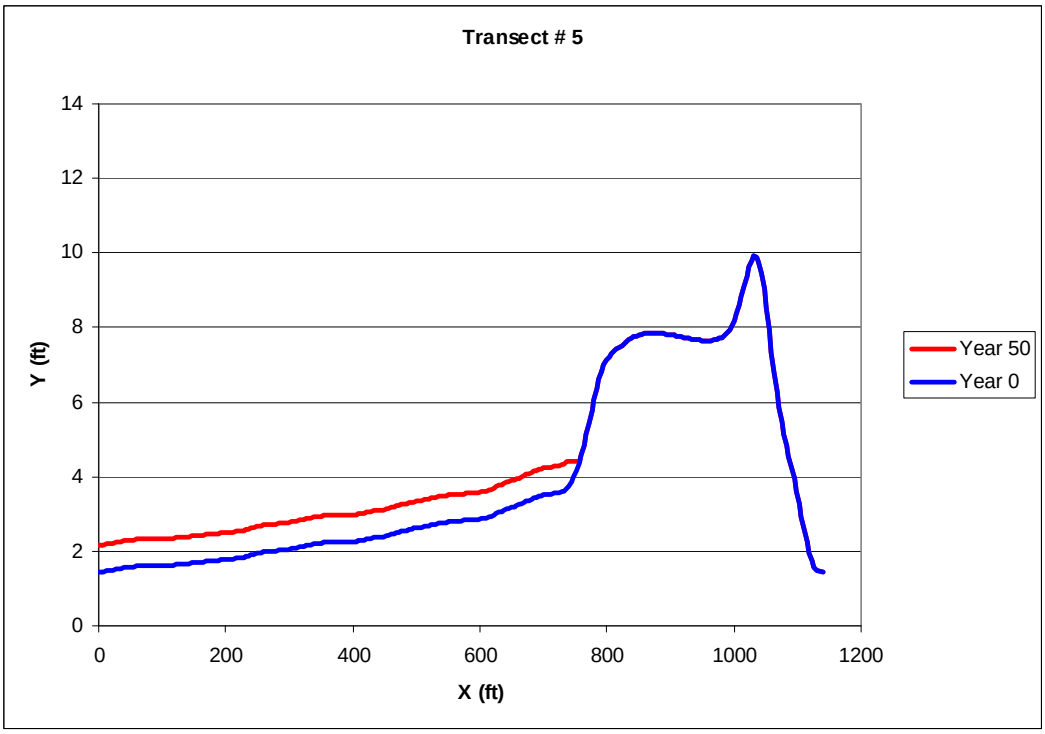
800
1000
Year 50
Year 0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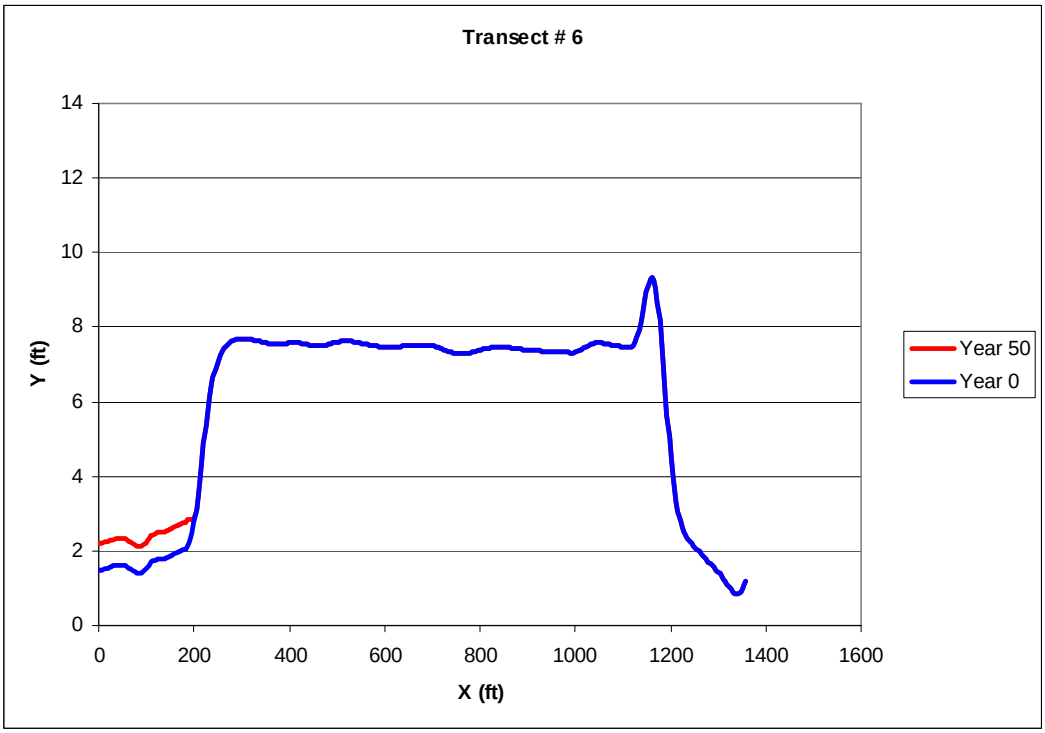
1200
Year 50
Year 0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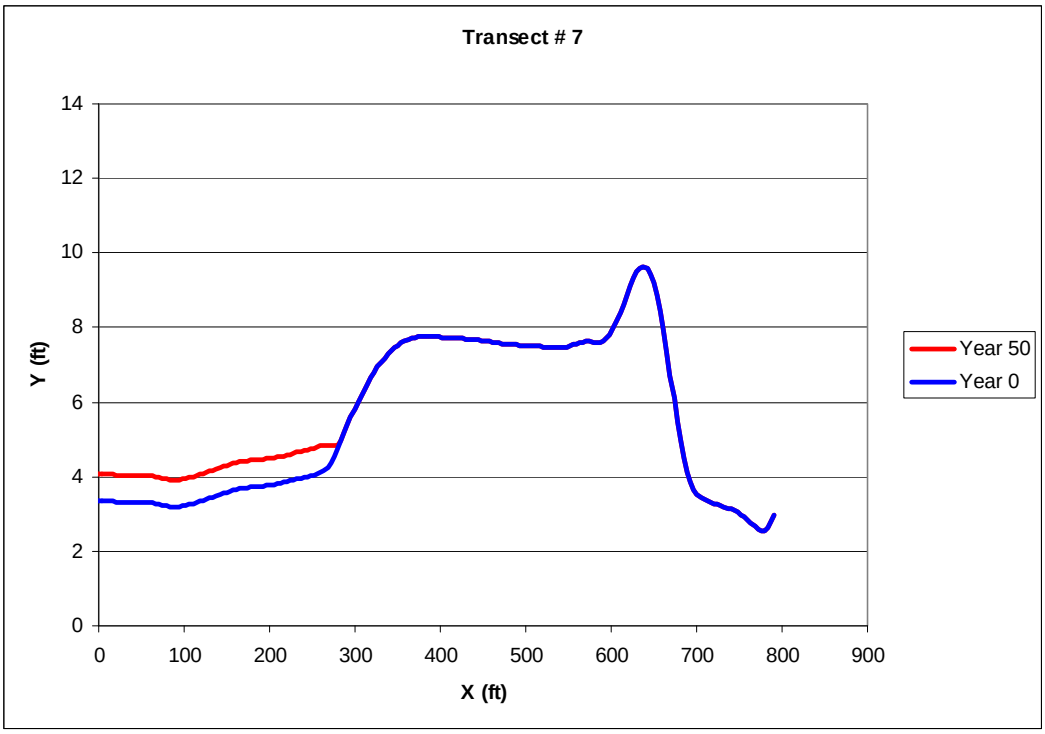
1400
1600
Year 50
Year 0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Y (ft)
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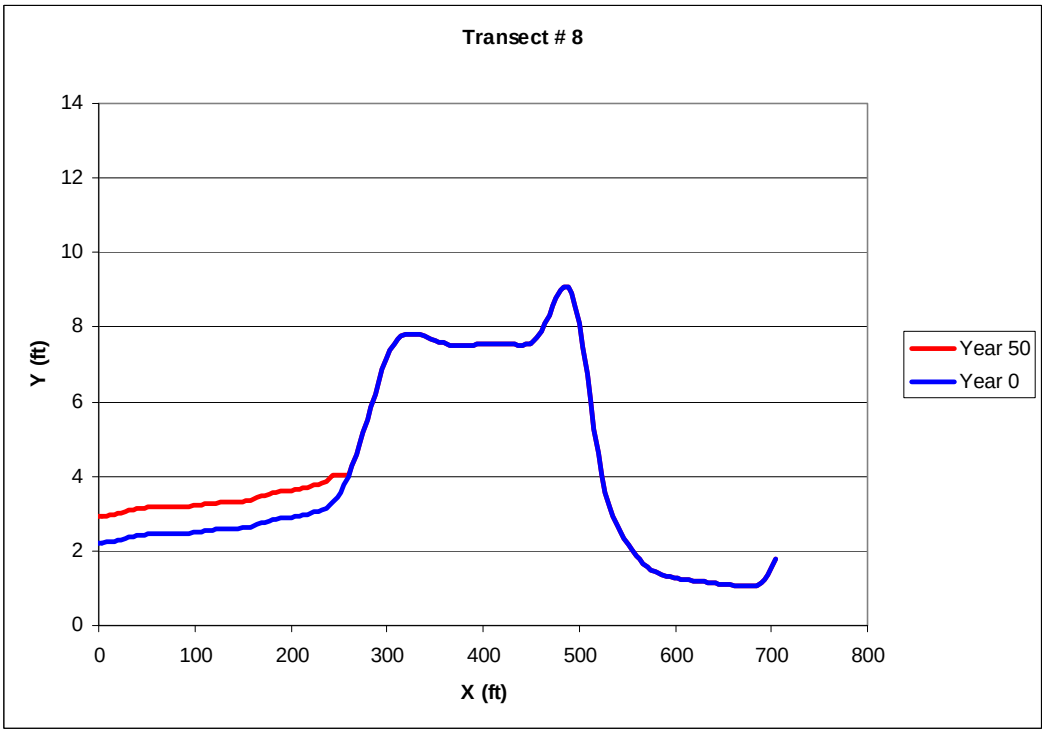
600
700
800
900
Year 50
Year 0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X (ft)
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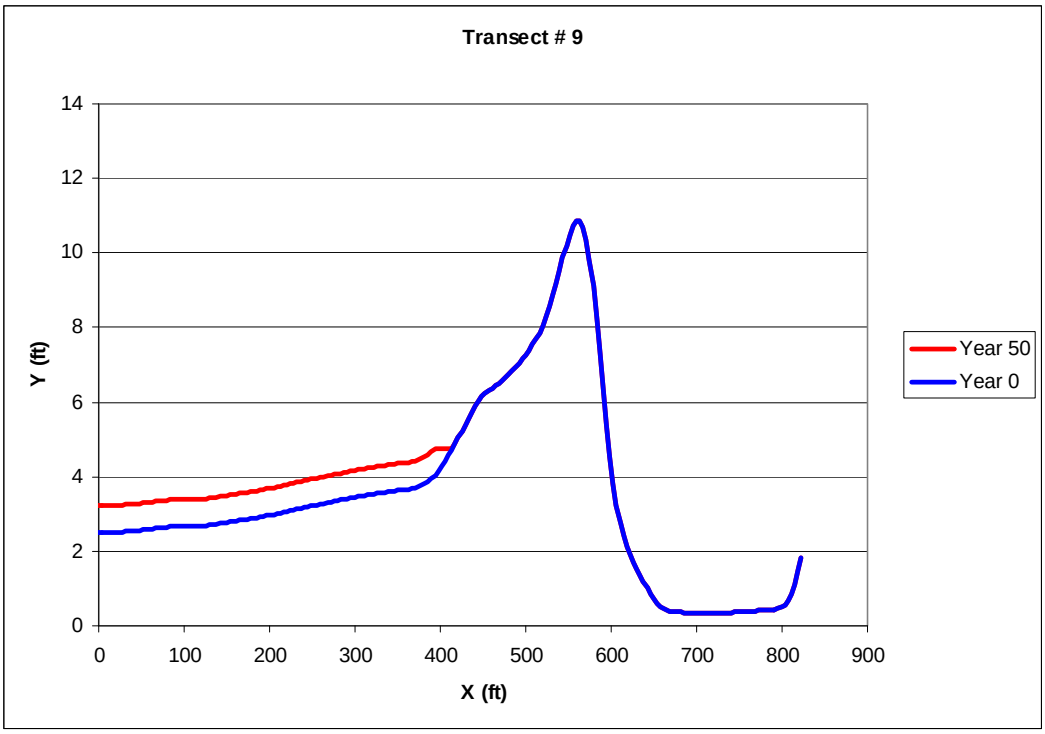
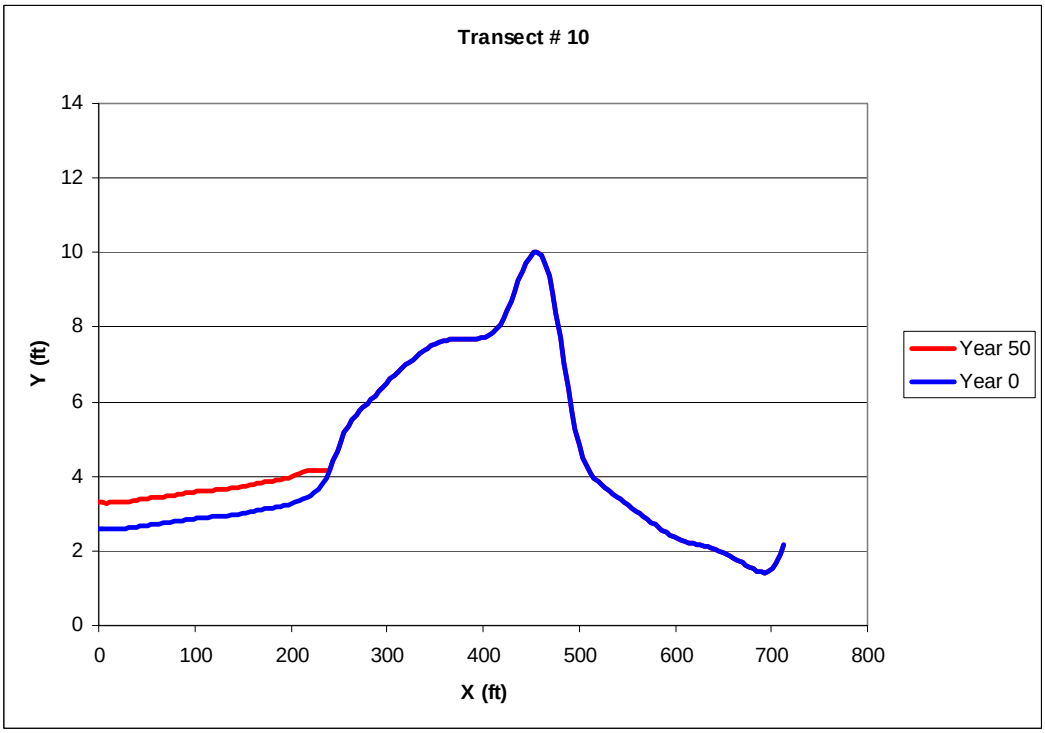
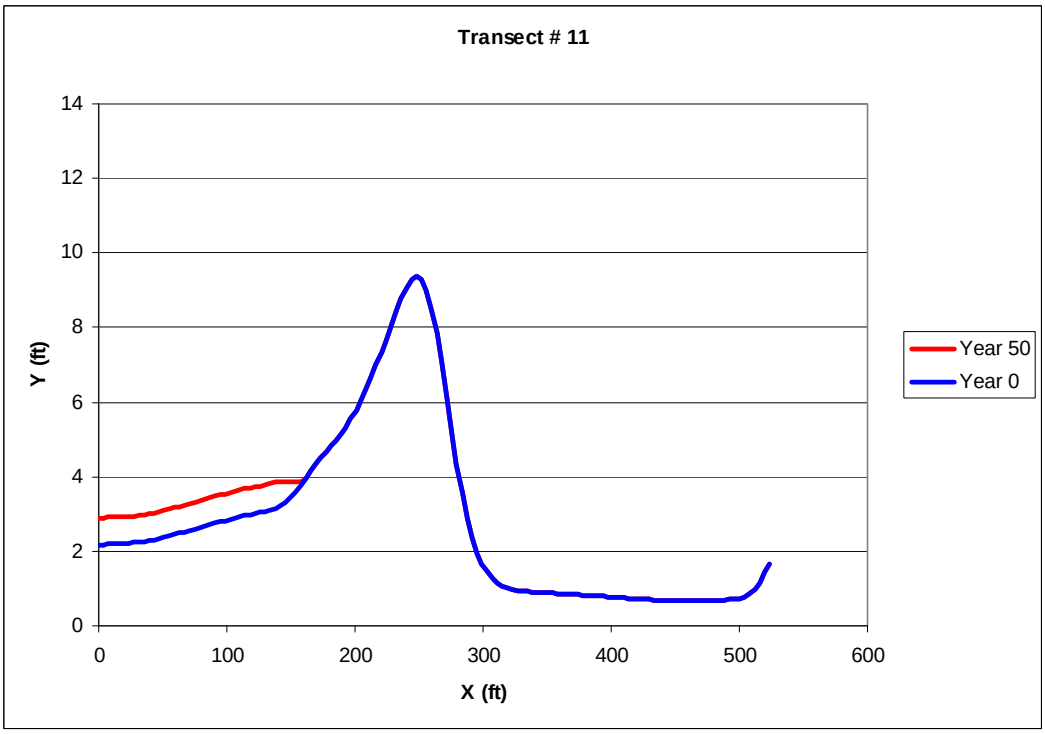
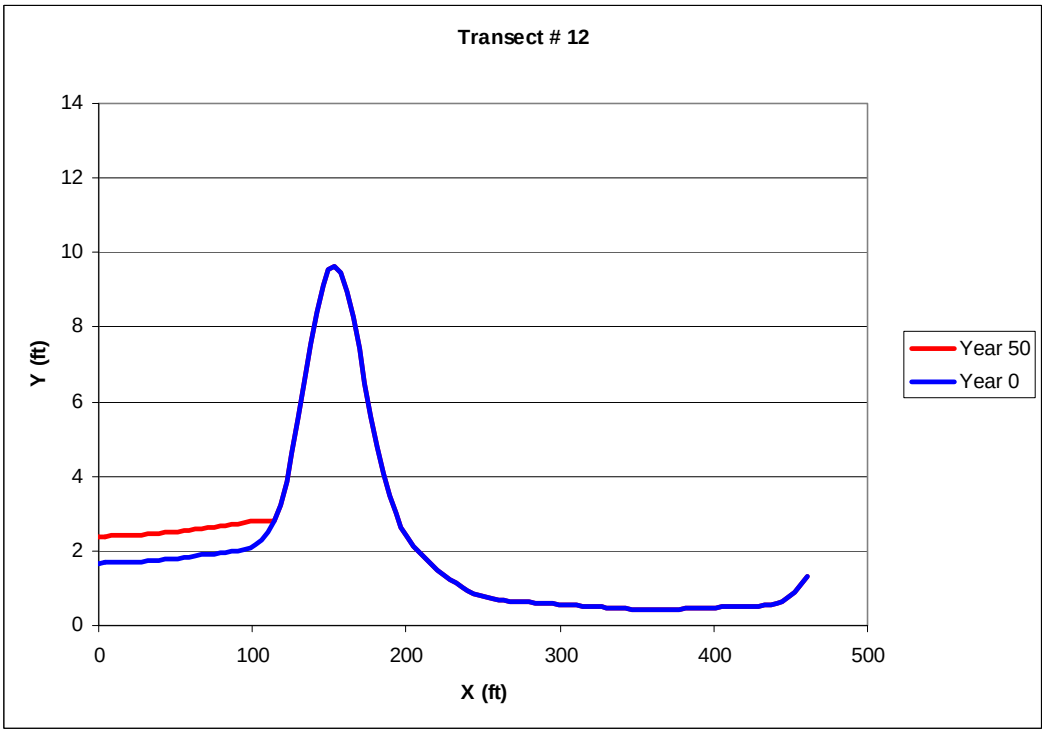
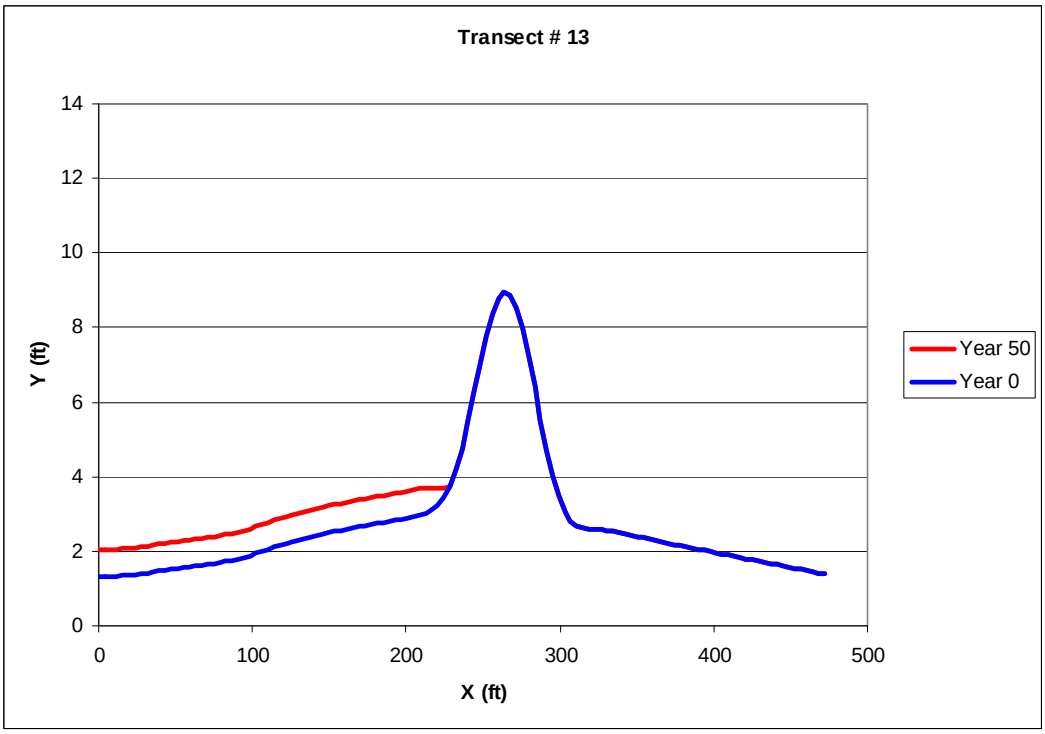
6
8
10
12
14
0
100
200
300
400
500
600
700
800
Year 50
Year 0

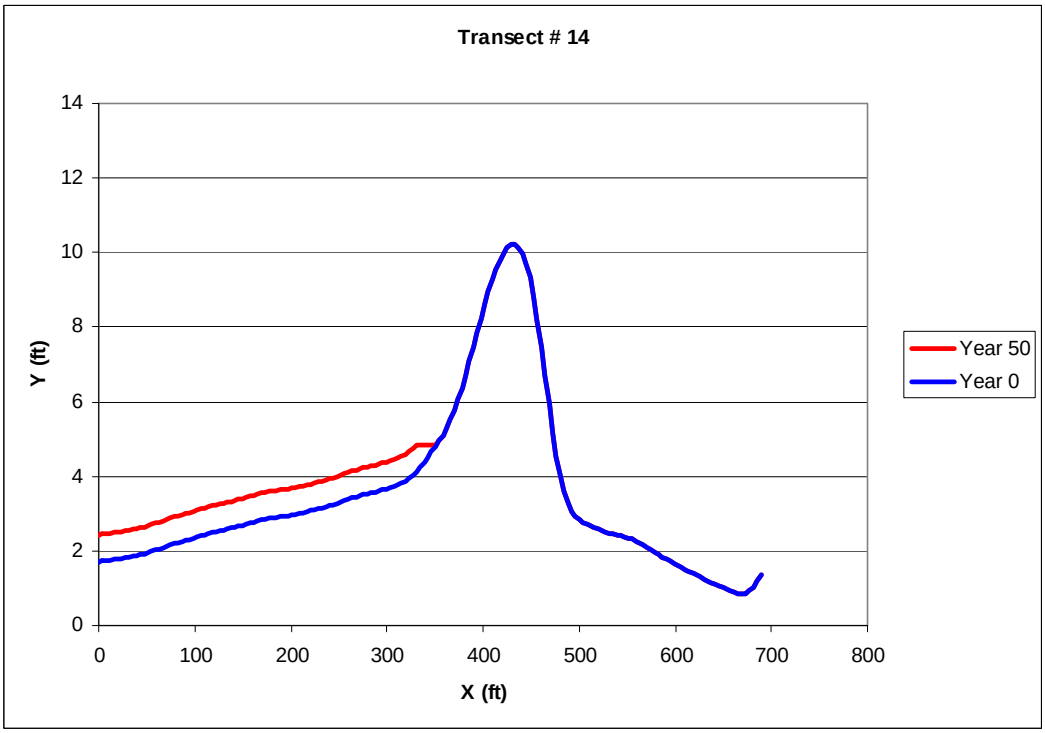

Transect # 9
Y (ft)
X (ft)
0
2
4
6
8
10
12
14
0
100
200
300
400
500
600
700
800
900
Year 50
Year 0

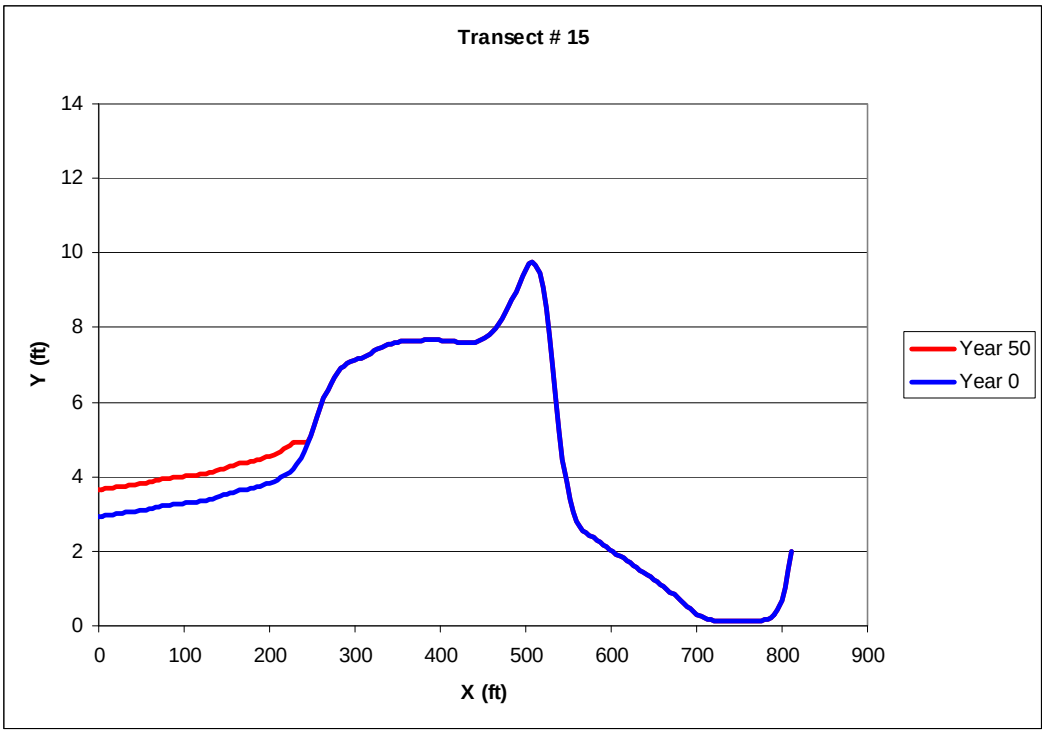

Transect # 10
Y (ft)
X (ft)
0
2
4
6
8
10
12
14
0
100
200
300
400
500
600
700
800
Year 50
Year 0

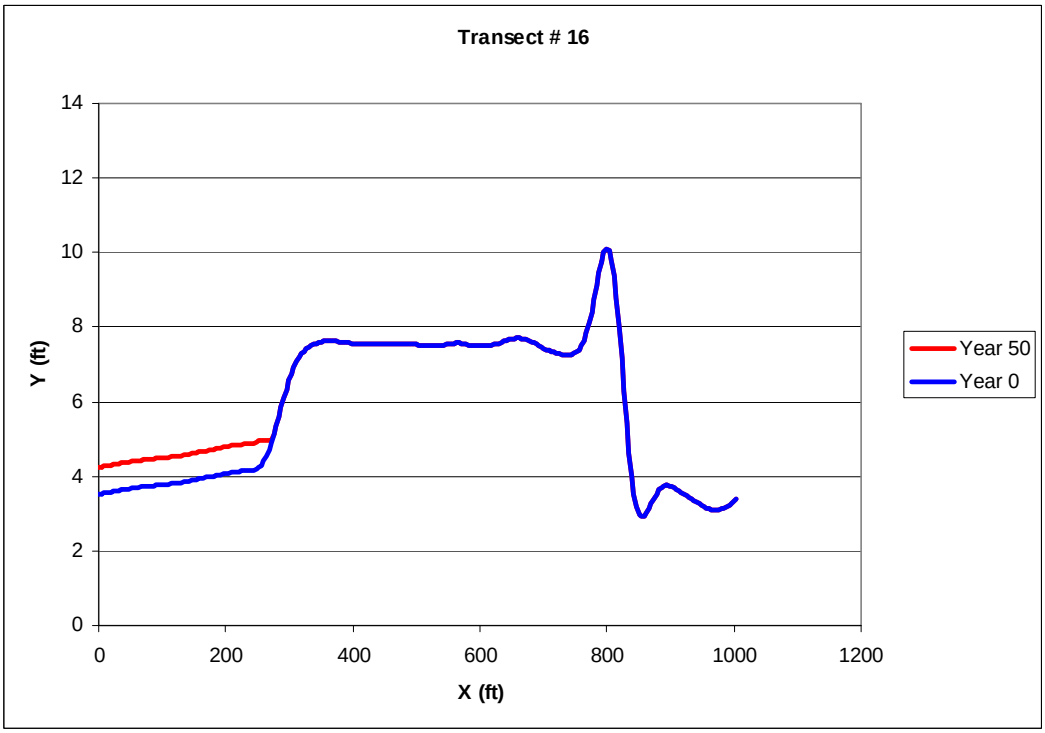

Transect # 11
Y (ft)
X (ft)
0
2
4
6
8
10
12
14
0
100
200
300
400
500
600
Year 50
Year 0

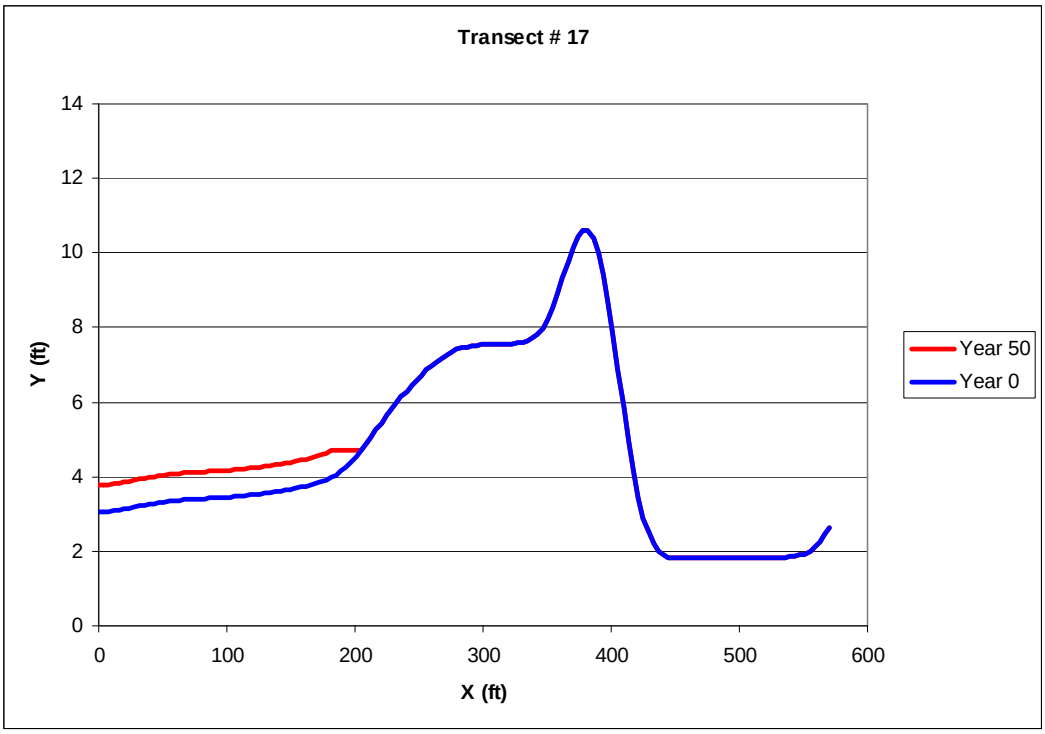

Transect # 12
Y (ft)
X (ft)
0
2
4
6
8
10
12
14
0
100
200
300
400
500
Year 50
Year 0

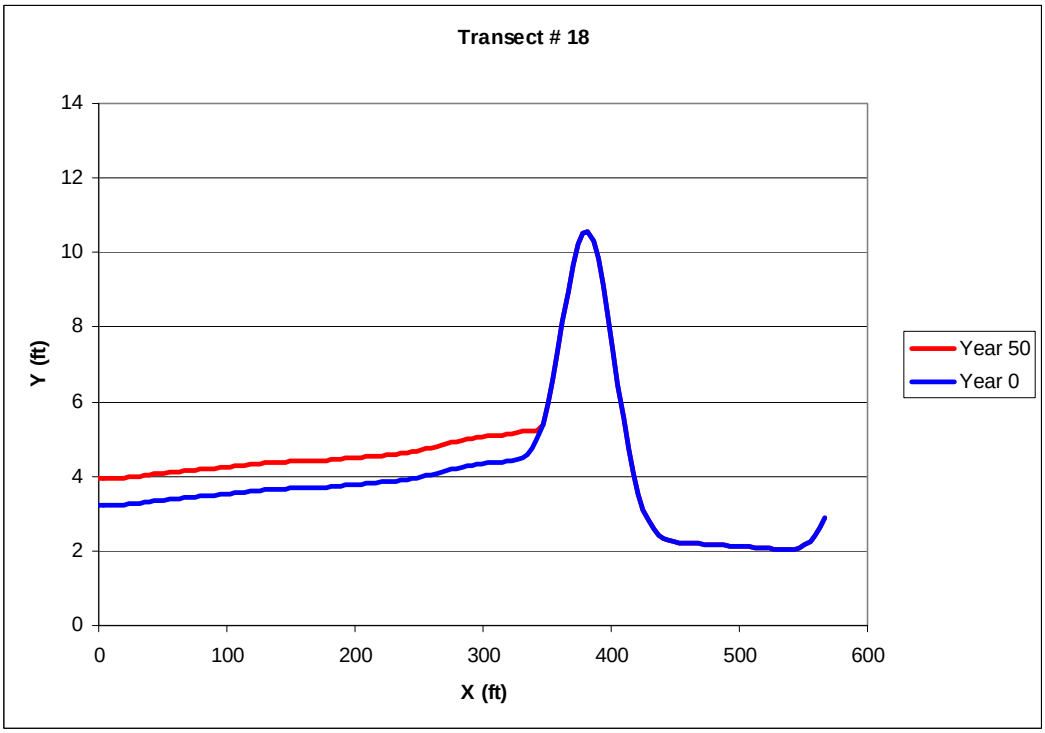

Transect # 13
Y (ft)
X (ft)
0
2
4
6
8
10
12
14
0
100
200
300
400
500
Year 50
Year 0

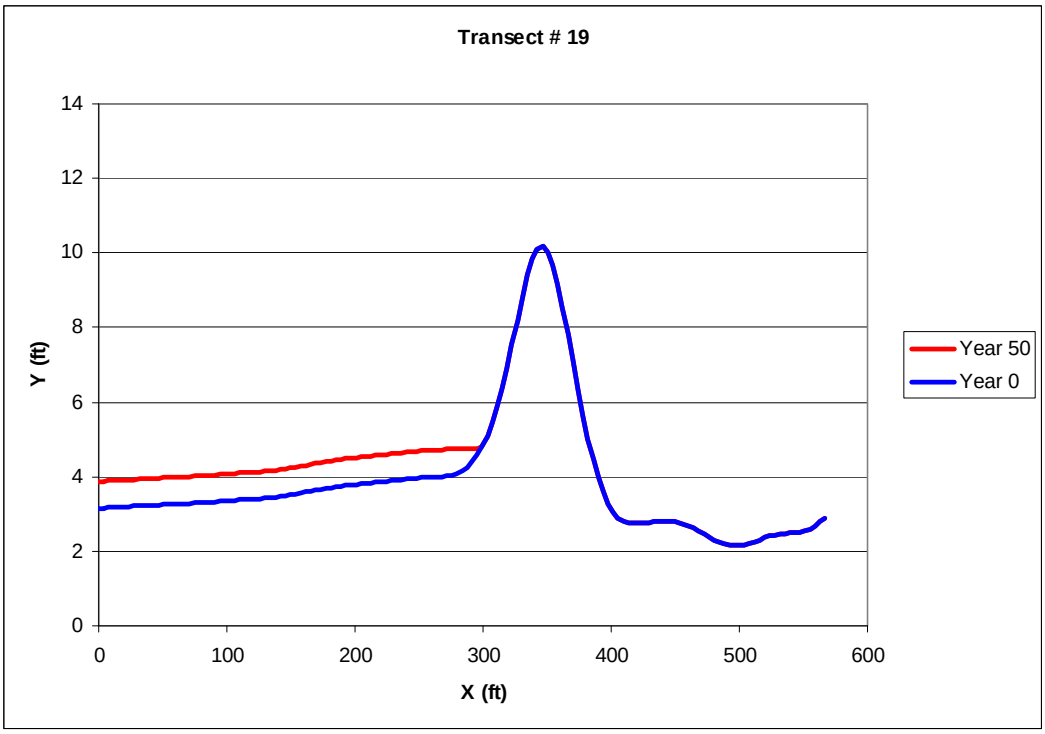

Transect # 14
Y (ft)
X (ft)
0
2
4
6
8
10
12
14
0
100
200
300
400
500
600
700
800
Year 50
Year 0

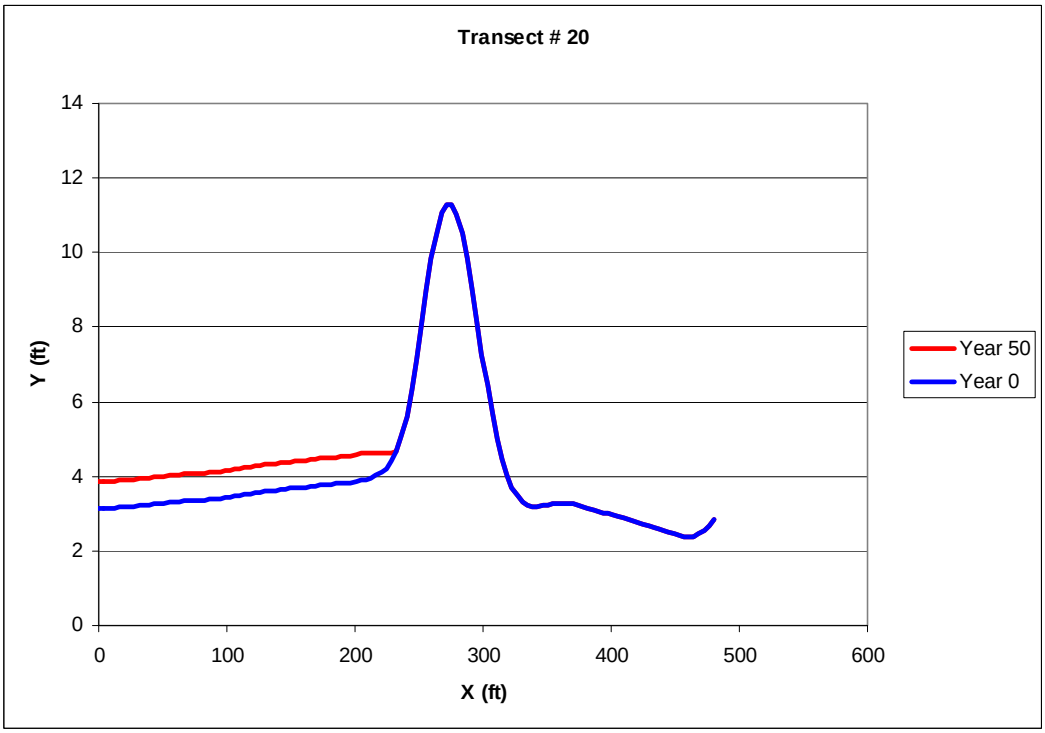

Transect # 15
Y (ft)
X (ft)
0
2
4
6
8
10
12
14
0
100
200
300
400
500
600
700
800
900
Year 50
Year 0

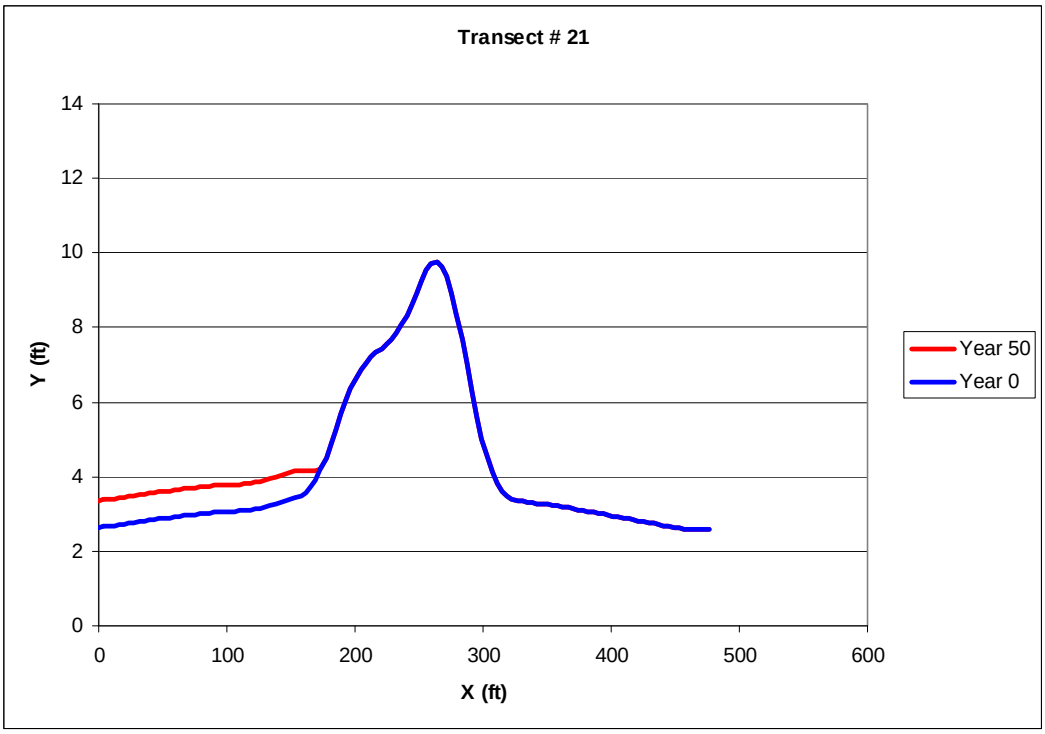

Transect # 16
Y (ft)
X (ft)
0
2
4
6
8
10
12
14
0
200
400
600
800
1000
1200
Year 50
Year 0

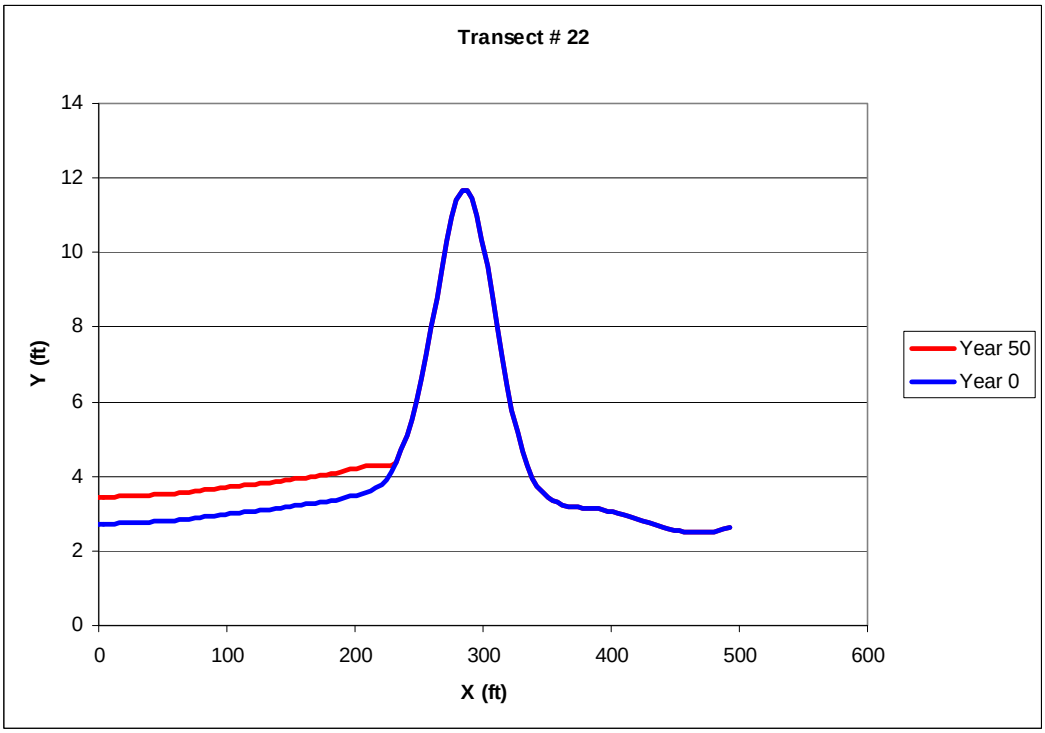

Transect # 17
Y (ft)
X (ft)
0
2
4
6
8
10
12
14
0
100
200
300
400
500
600
Year 50
Year 0

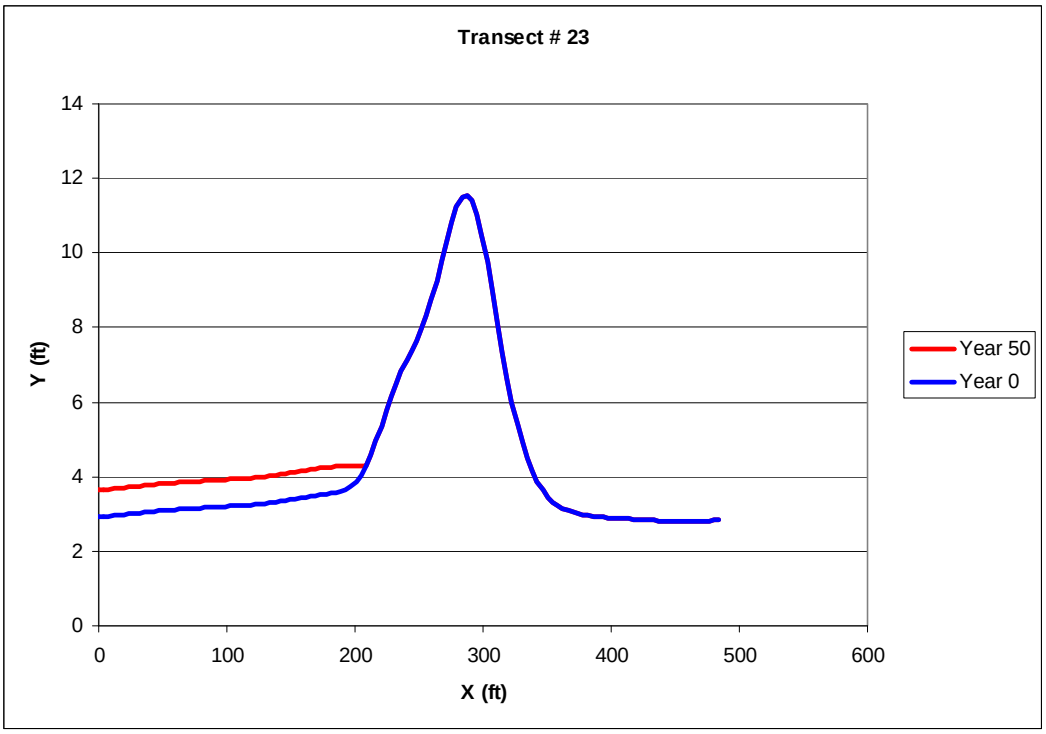

Transect # 18
Y (ft)
X (ft)
0
2
4
6
8
10
12
14
0
100
200
300
400
500
600
Year 50
Year 0

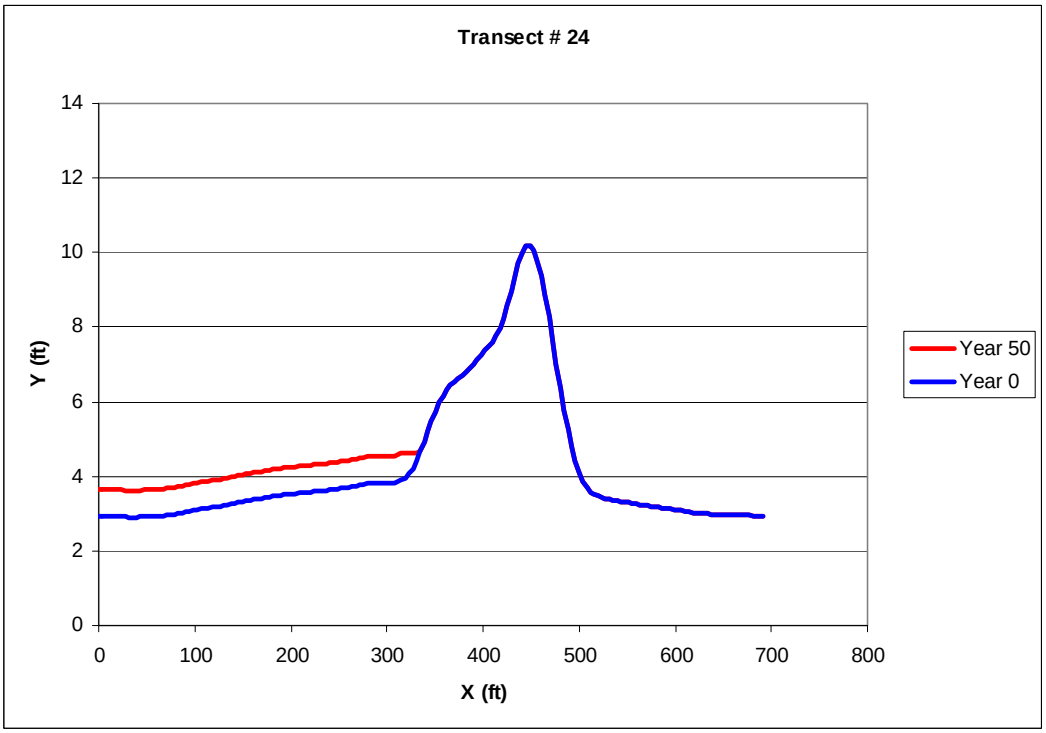

Transect # 19
Y (ft)
X (ft)
0
2
4
6
8
10
12
14
0
100
200
300
400
500
600
Year 50
Year 0

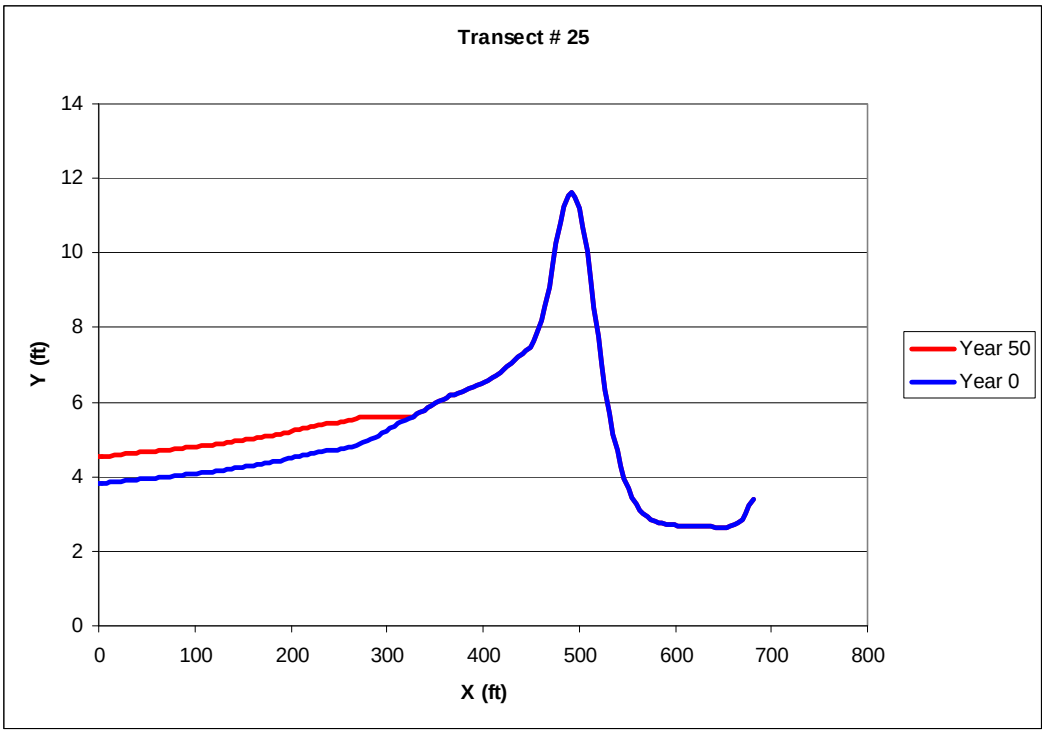

Transect # 20
Y (ft)
X (ft)
0
2
4
6
8
10
12
14
0
100
200
300
400
500
600
Year 50
Year 0

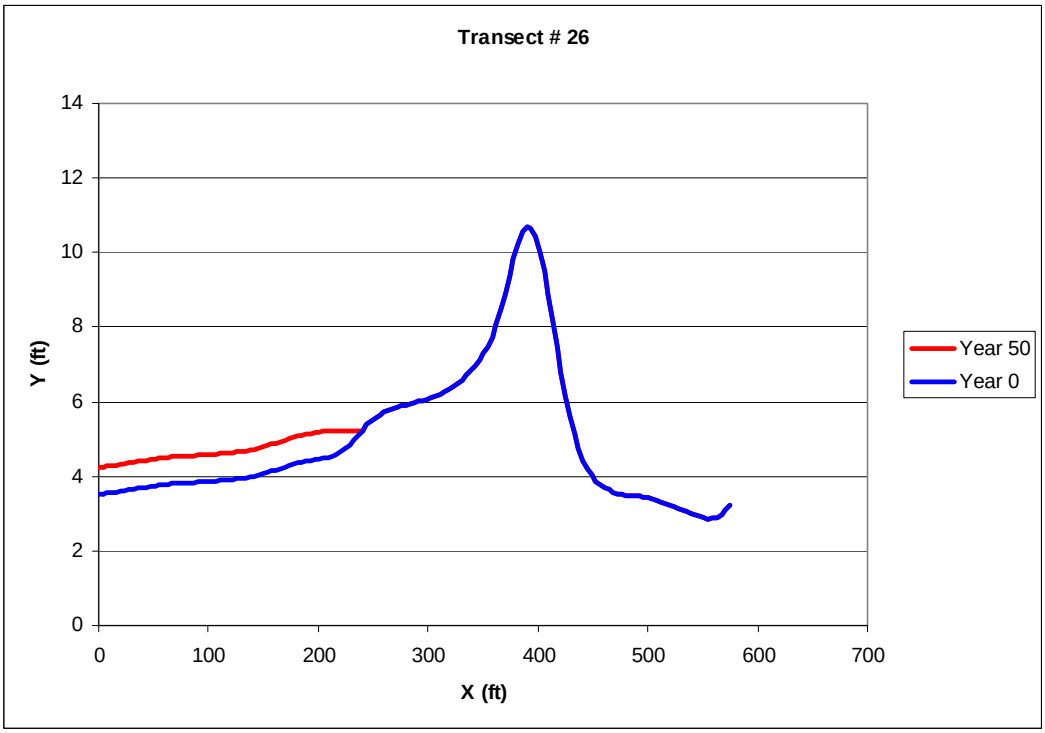

Transect # 21
Y (ft)
X (ft)
0
2
4
6
8
10
12
14
0
100
200
300
400
500
600
Year 50
Year 0

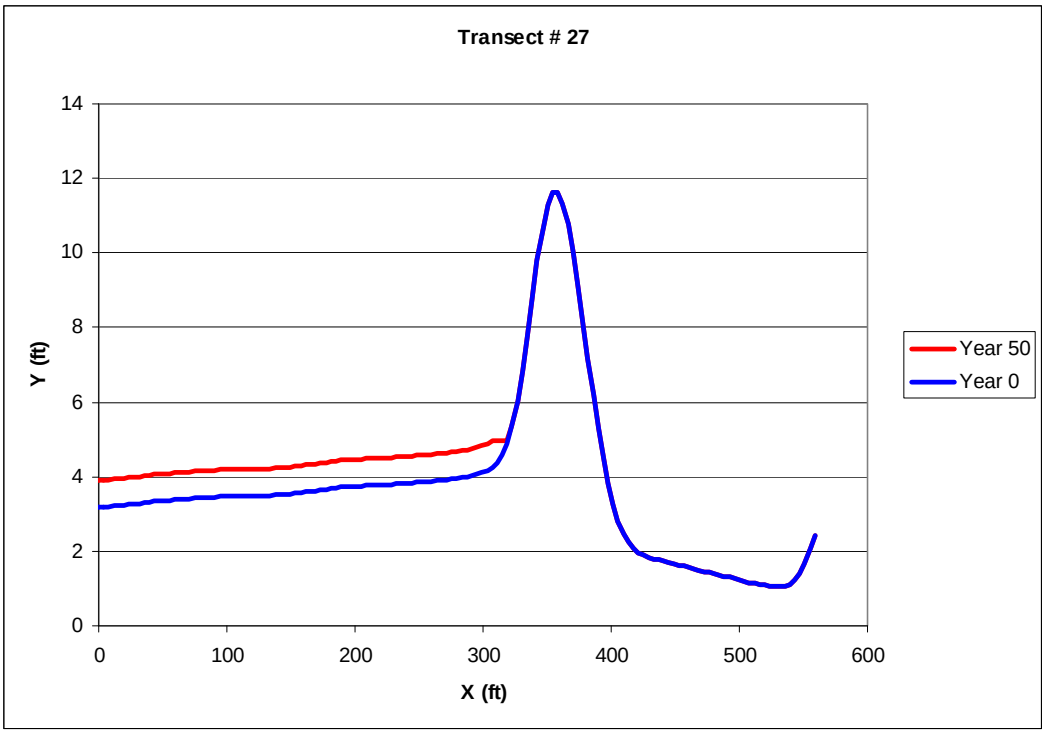

Transect # 22
Y (ft)
X (ft)
0
2
4
6
8
10
12
14
0
100
200
300
400
500
600
Year 50
Year 0

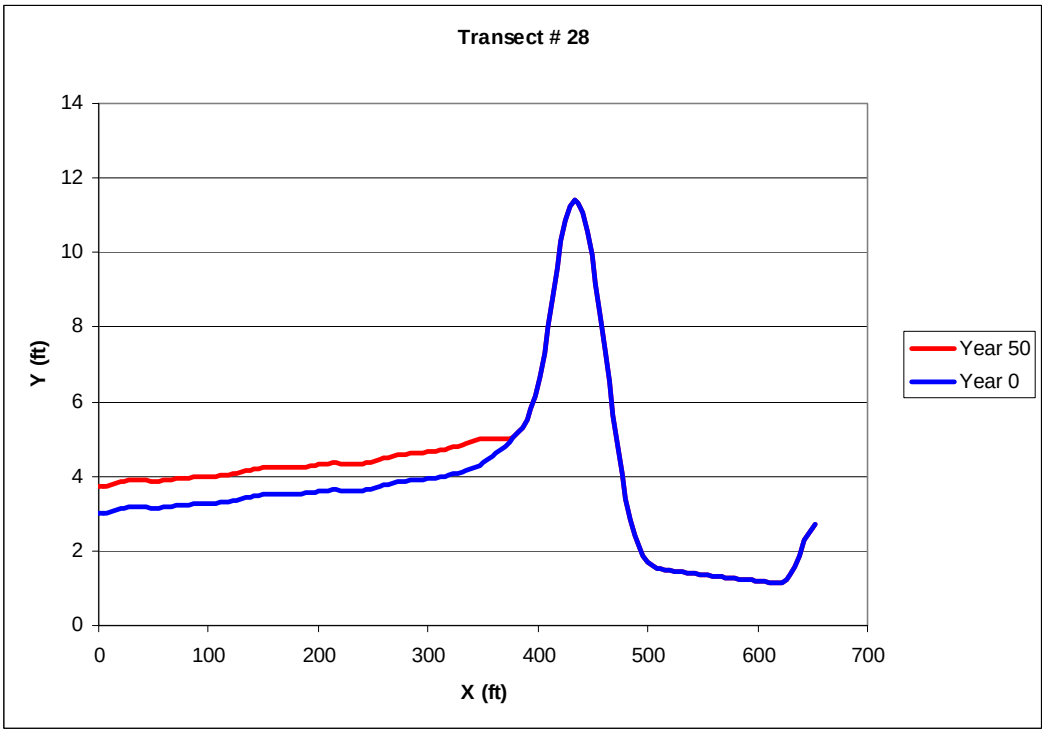

Transect # 23
Y (ft)
X (ft)
0
2
4
6
8
10
12
14
0
100
200
300
400
500
600
Year 50
Year 0

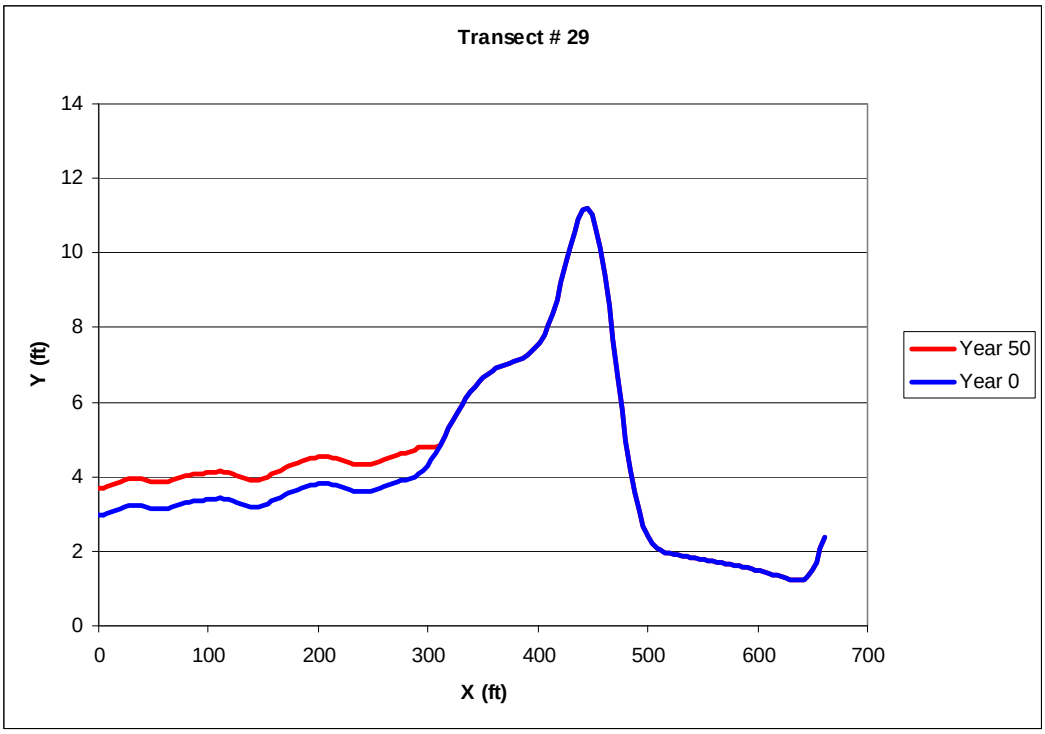

Transect # 24
Y (ft)
X (ft)
0
2
4
6
8
10
12
14
0
100
200
300
400
500
600
700
800
Year 50
Year 0

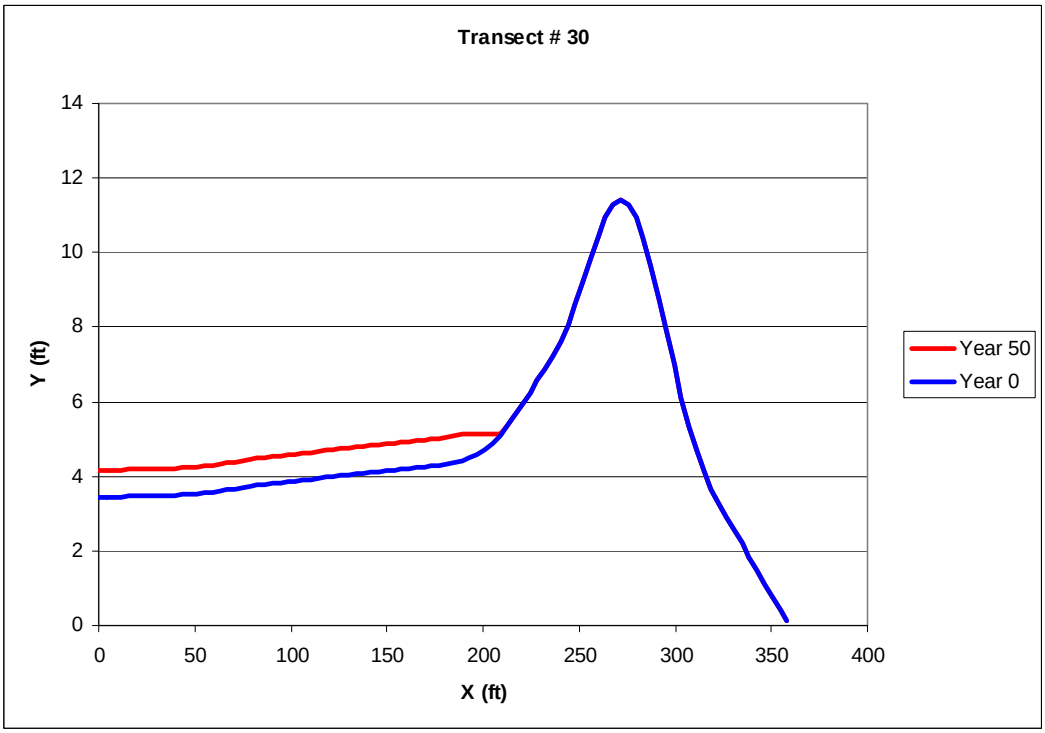

Transect # 25
Y (ft)
X (ft)
0
2
4
6
8
10
12
14
0
100
200
300
400
500
600
700
800
Year 50
Year 0

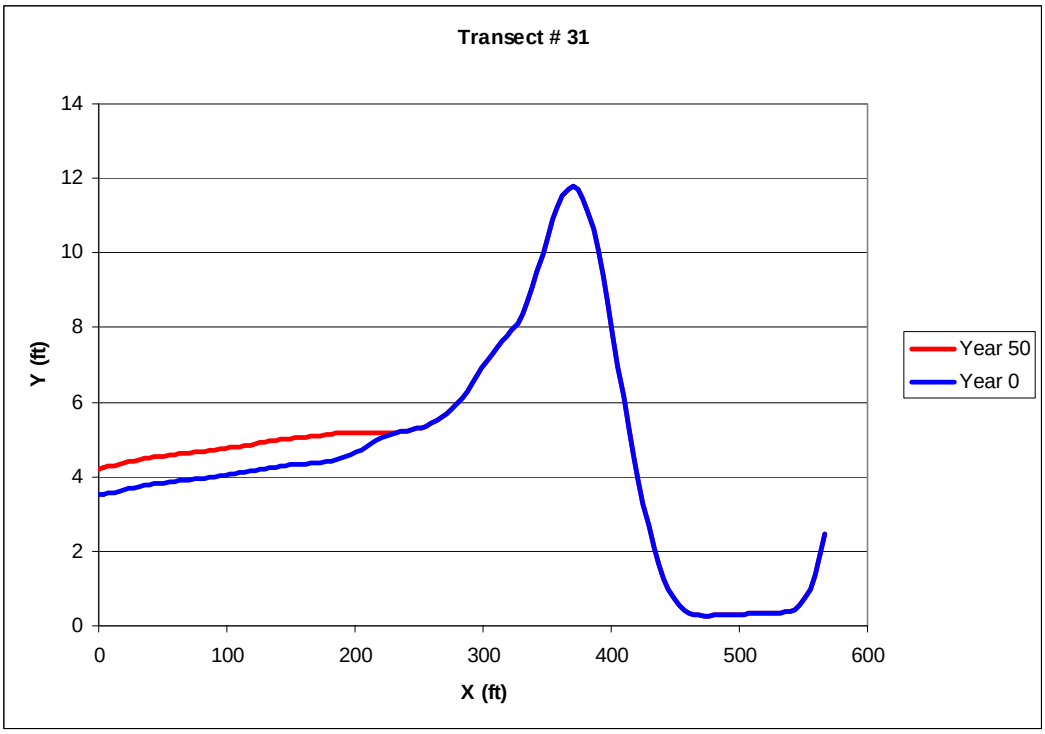

Transect # 26
Y (ft)
X (ft)
0
2
4
6
8
10
12
14
0
100
200
300
400
500
600
700
Year 50
Year 0

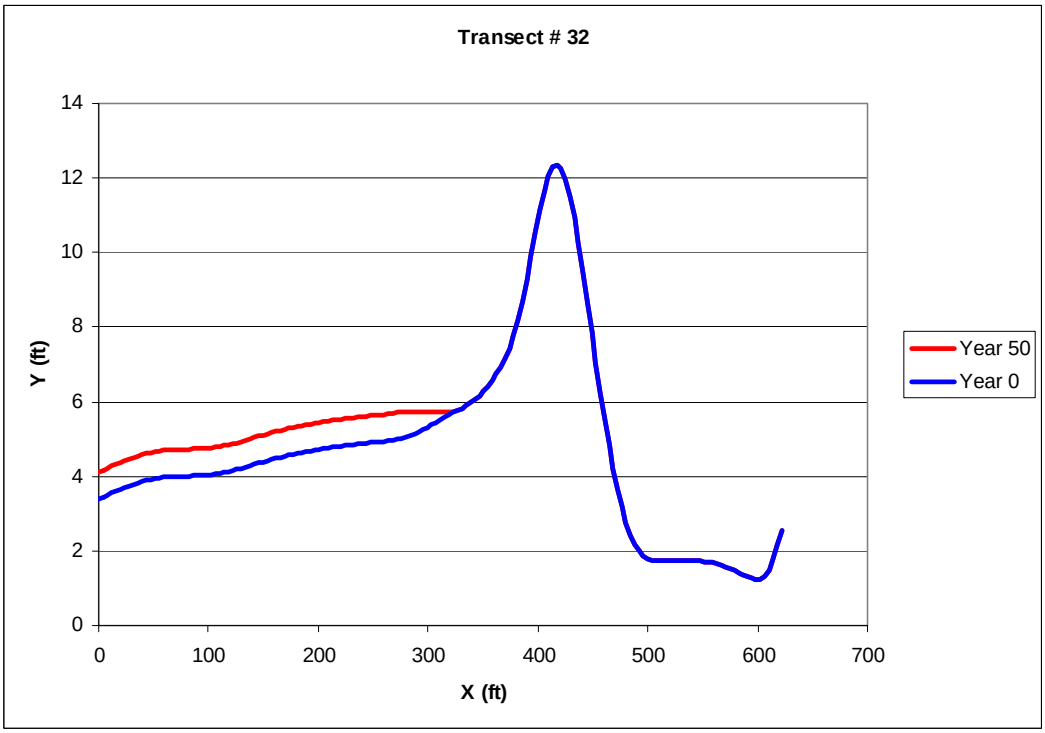

Transect # 27
Y (ft)
X (ft)
0
2
4
6
8
10
12
14
0
100
200
300
400
500
600
Year 50
Year 0


Transect # 28
Y (ft)
X (ft)
0
2
4
6
8
10
12
14
0
100
200
300
400
500
600
700
Year 50
Year 0


Transect # 29
Y (ft)
X (ft)
0
2
4
6
8
10
12
14
0
100
200
300
400
500
600
700
Year 50
Year 0


Transect # 30
Y (ft)
X (ft)
0
2
4
6
8
10
12
14
0
50
100
150
200
250
300
350
400
Year 50
Year 0


Transect # 31
Y (ft)
X (ft)
0
2
4
6
8
10
12
14
0
100
200
300
400
500
600
Year 50
Year 0


Transect # 32
Y (ft)
X (ft)
0
2
4
6
8
10
12
14
0
100
200
300
400
500
600
700
Year 50
Year 0