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1. INTRODUCTION

Saugatuck Harbor is located on the south eastern shore of Lake Michigan at the mouth of the Kalamazoo River (see Figure 1-1). In 1904, the entrance channel was stabilized by the construction of two shore perpendicular jetties that are now maintained by the U.S. Army Corps of Engineers (USACE).

The USACE is also authorized to investigate the mitigation of shore damage attributable to Federal Navigation under the authority of Section 111 of Rivers and Harbors Act of 1968. In November 1992, a preliminary study concluded that a more detailed study was necessary to determine the correlation between the erosion along Lake Michigan and the Federal navigation structure at Saugatuck.

Baird & Associates were retained by the Detroit District to undertake a detailed study of possible past and present impacts of the jetty structures (also known as Piers) on the adjacent shores in order to assess the role of the federal harbor in long-term shoreline change at Saugatuck and the surrounding areas.

The scope of work included a comprehensive analysis program which included the following:

- A review of the regional geomorphology and calculation of historic shoreline change at Saugatuck and adjacent areas.
- Detailed hydrodynamic and sediment transport modeling of a 50-year period to describe the movement of sand around the jetties and quantify the effect of the federal structures on the regional littoral transport.
- Longshore transport modeling and sediment budget analysis to quantify past and present shoreline change and sediment bypassing trends.

Through the application of state-of-the-art numerical modeling and the development of innovative analysis techniques, the present study will determine the likely effect of the harbor structures on local shoreline change and regional littoral transport, in both the past and present. In addition, assuming that future wave, lake level and river flow conditions remain similar to those in the last hundred years, the findings will be extrapolated to assess the possible future influence of the federal structures.

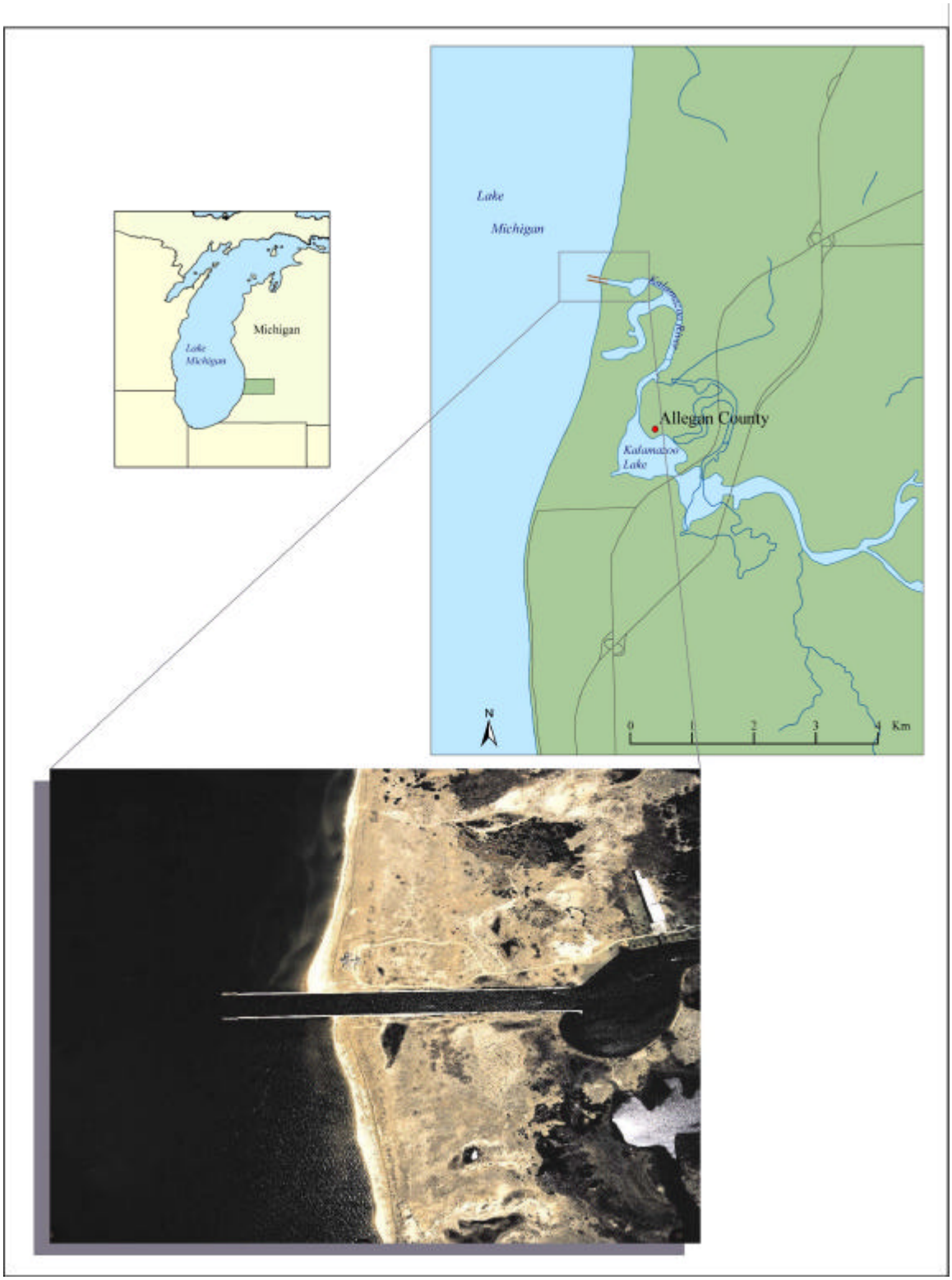


Figure 1-1. Saugatuck Harbor Location Plan.

2. DATA SOURCES

This section describes the spatial and temporal data used in the study and includes a description of the raw data as well as extended time-series data sets developed from various sources to produce environmental conditions for a continuous 50-year period required for the model investigation.

These modeling tasks are described in the sections that follow and include three main types of analysis:

- Detailed hydrodynamic and sediment transport modeling of conditions representative of a 50-year period,
- Detailed hydrodynamic and sediment transport modeling of discrete storm events, and
- LST (Longshore Sand Transport) modeling with COSMOS for a 50-year period.

2.1 Waves

The nearest WIS station to Saugatuck is Station 55. Baird's 50-year hindcast for this station was available through the LMPDS (Lake Michigan Potential Damage Study, Baird & Associates, 2001). The data set was developed using Baird's 1D parametric hindcast model for the period from 1956 to 1998 (43 years). The wave rose presented in Figure 2-1 indicates that waves are mostly from the NW and SW quadrants and the scatter plot in Figure 2-2 shows that offshore significant wave heights (H_s defined as the average height of the highest one third of the waves in a wave train) are less than 6 m and their peak periods (T_p defined as the period of waves with the highest energy) are less than 12 s.

In order to run 50-year simulations, the hindcast period was extended from 43 years to 50 years based on a statistical analysis of wave energy from 1956 to 1998 to define appropriate wave conditions for the data gap from 1949 to 1955 (for a description of the methodology, see Baird & Associates, 2001).

2.2 Water Levels

Long term monthly mean lake levels for Lake Michigan from 1865 to 2001 were available through the LMPDS and were used in the 50-year simulations. Figure 2-3 shows that the range in monthly levels during the last 50 years is approximately 1.9 m, with a low of –0.42 m CD (Chart Datum) in 1964 and a high of +1.5 m (CD) in 1986.

For the detailed storm simulations (see Section 4.2), hourly water levels were used to ensure that local short-term variations and associated surges were included. Data to 1998 (the extent of the wave hindcast period) used the Holland gage measurements (Station

Wave Rose

Station 55 - Offshore Wave Heights - 1956 to 1998

Current Time Interval:

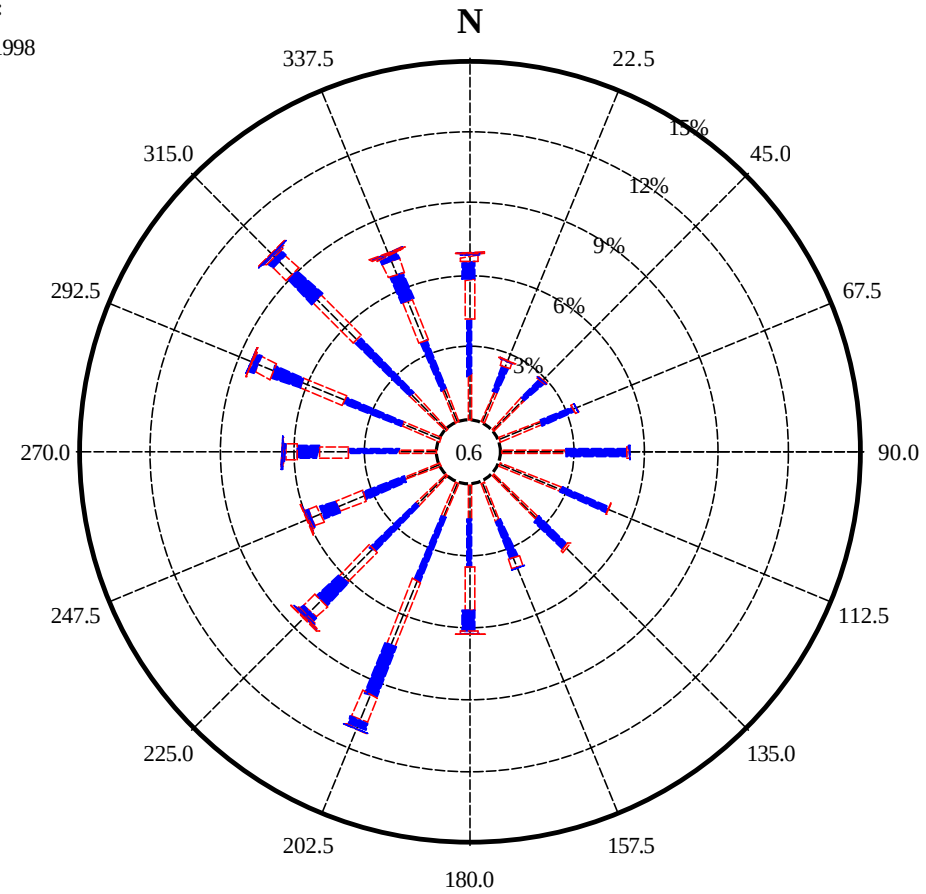
01 Jan, 1956 to 31 Dec, 1998

Season Selection:

All

Wave Height (m)

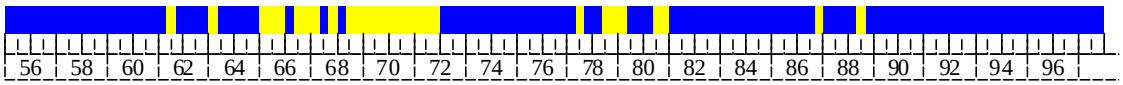
5.50 - 6.00
5.00 - 5.50
4.50 - 5.00
4.00 - 4.50
3.50 - 4.00
3.00 - 3.50
2.50 - 3.00
2.00 - 2.50
1.50 - 2.00
1.00 - 1.50
0.50 - 1.00
0 - 0.50
calm



Time Series Legend:

Equal or < 90 % Between 90 and 100 % 100 %

Minimum Scale Rate: 2 months



Data Range: 01 Jan 1956 05AM to 31 Dec 1998 09PM

Calm Wave Conditions: Wave heights = 0.0 m
All conditions during periods of shore ice

Waves:
Source File: P:\10407.20 Saugatuck Harbor Impact\Data\Wave Hindcast\Baird Hindcast at WIS M55\b55_19
Data Coverage: 01 Jan 1956 05AM to 31 Dec 1998 09PM
Linear Refraction: Not Applied
TMA Transformation: Not Applied
Depth Offshore: N/A
Depth Inshore: N/A
Profile Azimuth: N/A

Water Levels: No water level data.

Shore Ice: No shore ice data.

Figure 2-1. Deepwater Wave Conditions.

Wave Period Frequency Distribution By Height (All)

Station 55 - Offshore Wave Heights - 1956 to 1998

Date Range: 01 Jan 1956 05AM to 31 Dec 1998 09PM

Season: All

Wave Height *Hs* (m)	Wave Period Frequency Distribution (%)						Total	Maximum Period
	0-2.0	2.0-4.0	4.0-6.0	6.0-8.0	8.0-10.0	10.0-12.0		
0-0.5	6.19	22.15	1.82	0.00			30.16	6.90
0.5-1.0		10.57	22.53	0.16			33.26	7.17
1.0-1.5		0.01	15.25	2.51			17.77	7.65
1.5-2.0			2.55	8.15	0.00		10.70	8.03
2.0-2.5			0.07	5.03	0.02		5.12	8.41
2.5-3.0				1.25	0.49		1.74	8.96
3.0-3.5				0.11	0.34		0.45	9.25
3.5-4.0				0.00	0.14		0.15	9.70
4.0-4.5					0.04	0.00	0.04	10.19
4.5-5.0					0.01	0.01	0.01	10.52
5.0-5.5					0.00	0.00	0.01	10.42
5.5-6.0					0.00		0.00	9.88
Totals	6.19	32.73	42.21	17.22	1.04	0.01		
Cum.	6.8	39.5	81.7	98.9	100.0	100.0		

0.6% Calm Conditions (Wave Height<0)

Frequency rounded to two decimal places (ie 0.001 shown as 0.00)

Figure 2-2. Deepwater scatter plot.

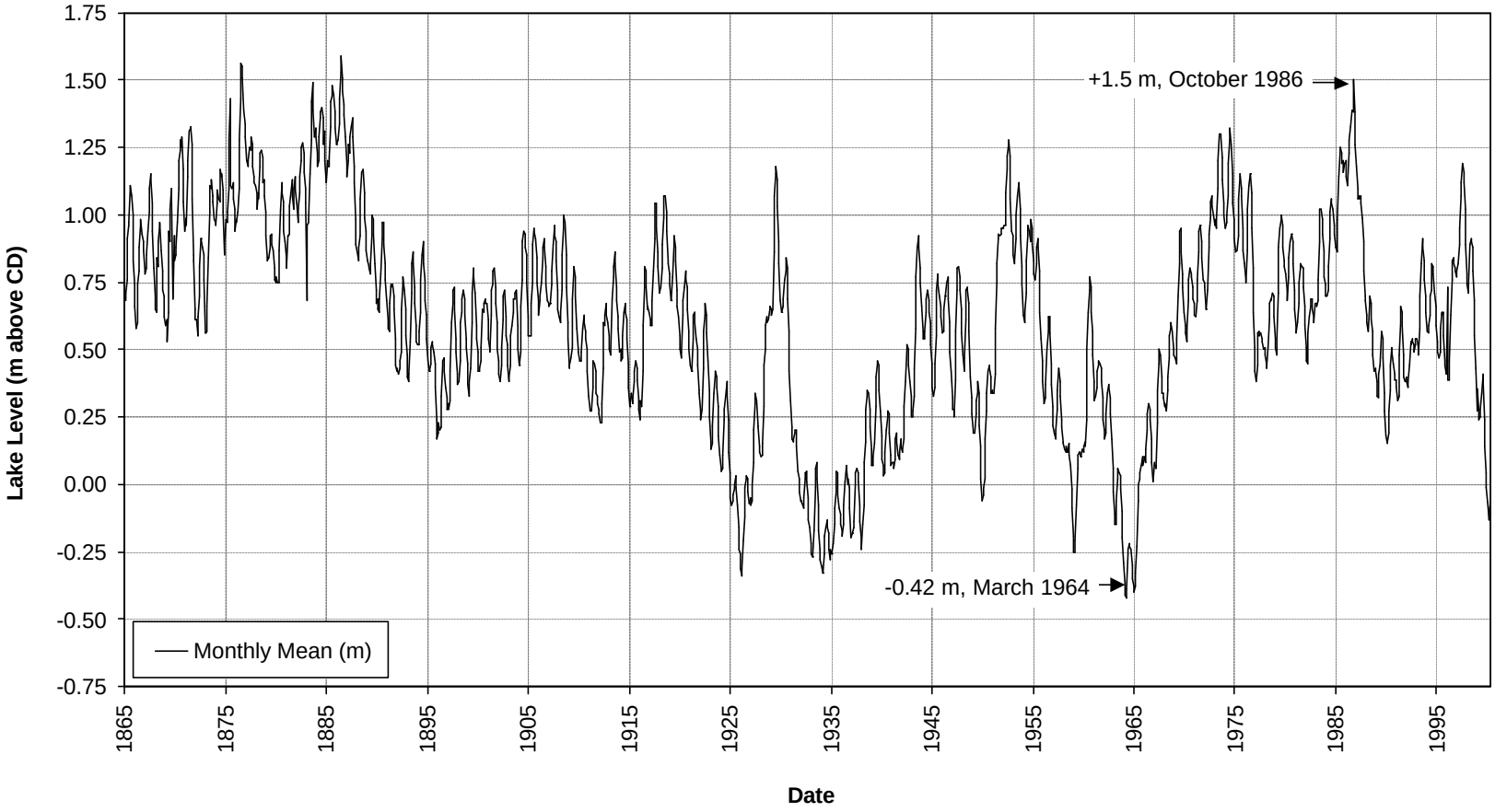


Figure 2-3. Lake Michigan monthly mean water levels (USACE Huron-Michigan gage network).

9087031) for which digital data are available starting in 1970. Figure 2-4 shows one year of the 29-year record (January 1970 to December 1970) that includes a number of surges over monthly mean levels.

2.3 Ice Cover

The present study used ice cover data available from the LMPDS study (USACE, 2001) in which weekly ice charts for Lake Michigan from 1973 to 1998 were discretized for each 1 km reach to account for the protective effect of shore-fast ice in the nearshore wave climate. The 26-year ice time series was extended to a 50-year record assuming that conditions from 1973 to 1998 were representative of the 1949 to 1972 period of missing data.

2.4 River Discharge

Discharge measurements for the Kalamazoo River were obtained from the USGS National Water Information System (http://waterdata.usgs.gov/nwis/discharge). Station locations are shown in Figure 2-5 with additional details summarized in Table 1.

Table 1. River Discharge Data[a].

Station	Site Location	Start	End	Observations
04108660	Kalamazoo River near New Richmond MI	01 Apr 1994	31 Oct 1995	579
04108500	Kalamazoo River near Fennville MI	01 Apr 1924	30 Sep 1993	23194

[a] Digital time-series data of daily mean streamflow.

Although the New Richmond station (04108660) was the closest gage to the Saugatuck Harbor jetties, less than two years of data were available at this location and it was necessary to use data from farther upstream at the Fennville station 04108500. Although farther from the study site, over 63 years of continuous time series measurements were available, dating back to 1924.

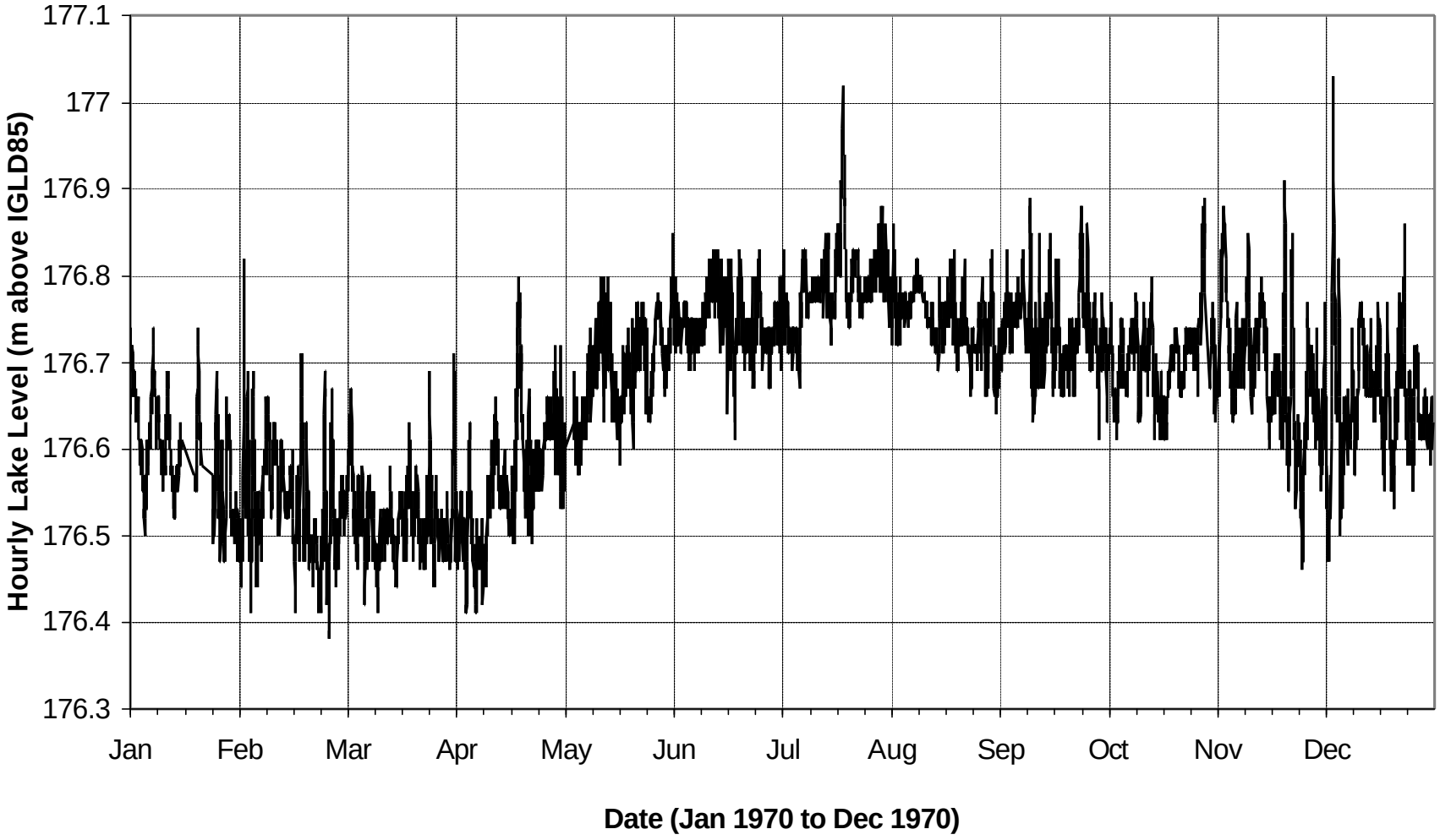


Figure 2-4. Hourly water levels at Holland gage (Station 9087031).

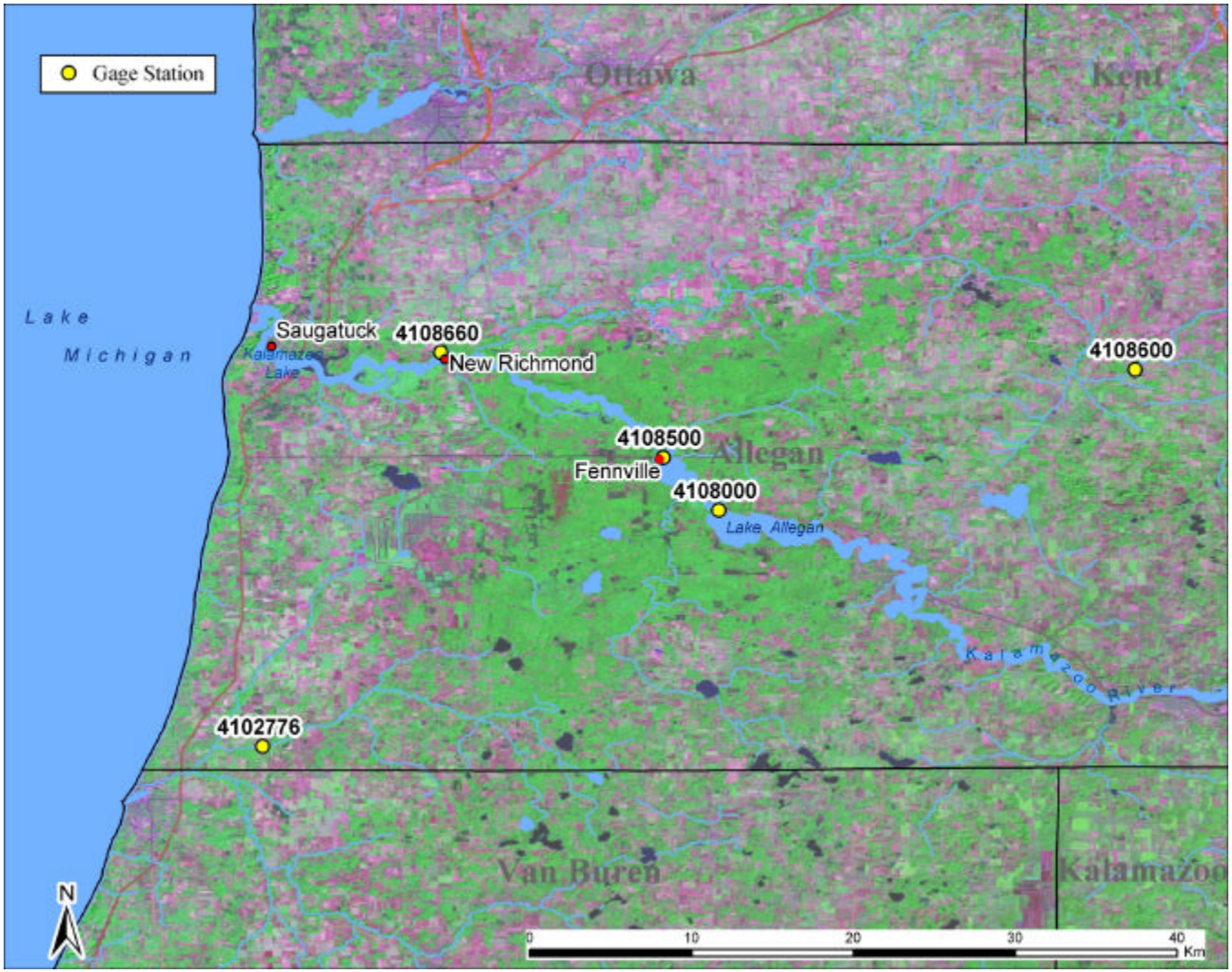


Figure 2-5. Stream flow gage locations.

Table 2 shows that although average daily discharge values are within 25% of each other, the ratio for maximum daily discharge is more than double.

Table 2. Discharge Statistics.

Station	Site Location	Average Daily Discharge (m^3/s) [a]	Maximum Daily Discharge (m^3/s) [b]
04108660	Kalamazoo River near New Richmond MI	56	147
04108500	Kalamazoo River near Fennville MI	42	306
Ratio	04108500 to 04108660	75%	208%

[a] Average for data record.
[b] Maximum for data record.

2.5 Bathymetry

The bathymetry was produced from a new SHOALS survey data conducted in the summer of 2001 and profile survey data obtained in 2001. The depths corresponded well to each other at the interface of the two datasets and were therefore merged together without any problem. SHOALS point data with elevations values that were obviously on the jetty but fell outside the traced jetty lines were deleted. Finally, a regional lakewide NOAA-NOS (National Oceanic and Atmospheric Administration - National Ocean Service) bathymetry data set was used to extend the data to depths of 18 to 25 m. Bringing these data sources together, a 5 m resolution bathymetry grid, covering approximately 5 km of shoreline, out to the 25 m depth contour, was created.

2.6 Topography

A detailed topographic data set was provided by the Detroit District and generated from the 1999 aerial photography at Saugatuck for the LMPDS, including: toe and top of bluff mapping, buildings, roads and coastal structures. Topographic LIDAR was also collected at Saugatuck to provide detailed elevation data covering several kilometres inshore of the waterline.

2.7 Profile Surveys and Lakebed Changes

Ocean Surveys Inc. performed a series of profile surveys on May 4 to 9, 2001 at the harbor for the Detroit District of the United States Army Corps of Engineers. The offshore extent of the profiles ranged from 8 to 15 m in depth. Figure 2-6 shows a plan

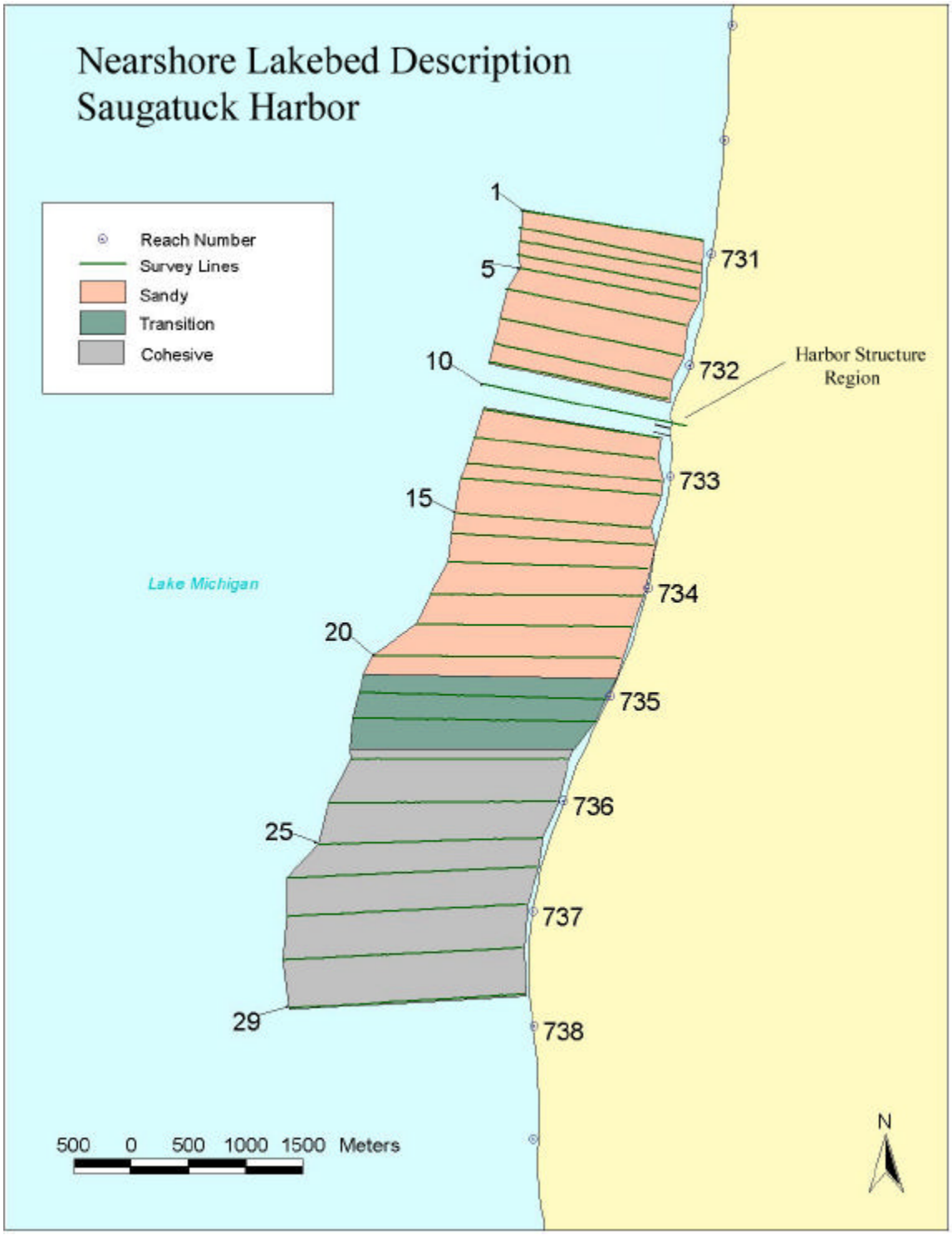


Figure 2-6. Nearshore profile locations.

view of the survey lines and includes a general description of the lakebed profile features. Profiles are included in Appendix A together with the 1999 SHOALS data and the 1948 NOAA countywide survey. The profiles between Reaches 731 to 734 (lines 1 to 15) are either barred or smooth, typical features of sandy profiles, and indicate that the lakebed has been stable at 6 m and deeper below chart datum for the past 50 years. A lowering of lake bottom is observed in profile lines 16 to 19, which are smooth profiles with steep bars, and extends to depths of 12 to 14 m below CD. It is not clear if this is because of inaccuracies in the profile surveys or from local erosion along these lines. Profile 20 has been stable for the past 50 years and a transition from sandy profile to cohesive profile is observed in lines 21 (Reach 735) to 24 (Reach 736). As will be discussed in the coming chapters, most of the development and evolution of fillet beaches (accumulated triangular sandy beaches against the piers) is expected to have occurred between 1904 and 1948. Pre-1948 profile surveys would therefore be among the most valuable information and are, unfortunately, not available.

2.8 Sand Thickness

Sand thickness data and soil data were available from approximately 30 jet probings performed around Saugatuck Harbor by Detroit District. Sampling locations are shown in Figure 2-7 and the results are summarized in Figure 2-8. Looking at these results, it was concluded that there is sufficient sand cover in the immediate study area for it to be classified as a sandy shoreline.

2.9 Grain Size

Grain size analysis data of approximately 15 surface grab samples obtained by Peterson Dredge were available through the Detroit District. The data are GIS referenced and summarized in Figure 2-9. Looking at this figure, it was concluded that the median grain size of sand in the area is D_{50}=0.3 mm.

2.10 River Sediment Sources

A preliminary assessment of sediment inputs from the Kalamazoo River was undertaken using available data from the USGS National Water Information System and standard sediment transport calculation procedures.

Suspended Sediment

Measured suspended sediment concentrations upstream of Kalamazoo Lake were analyzed to provide a preliminary estimate of the potential suspended sediment load. Figure 2-10 shows the sampling location (Station 4108690) from the USGS Water

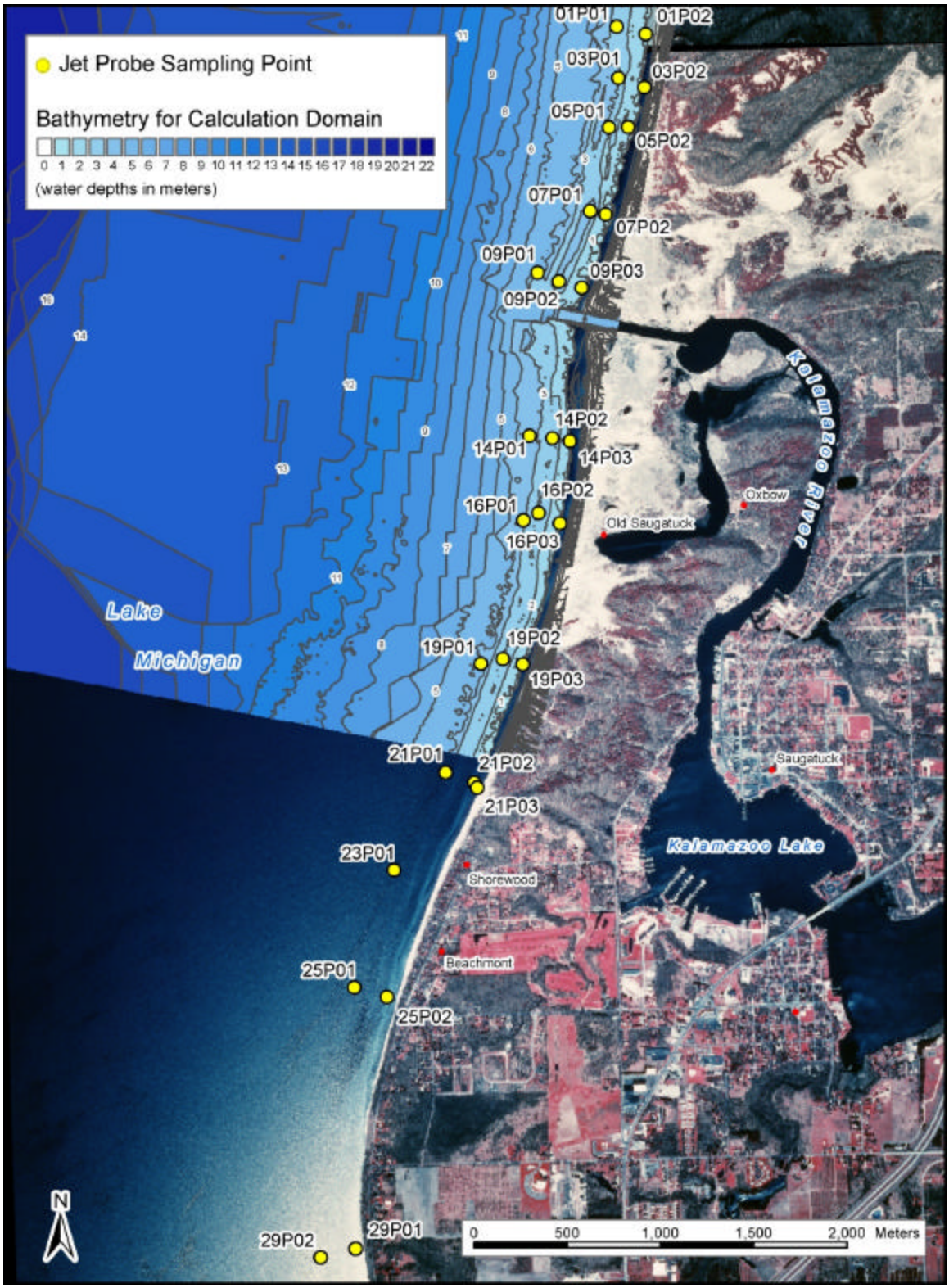


Figure 2-7. Jet probe locations for sand thickness measurements.

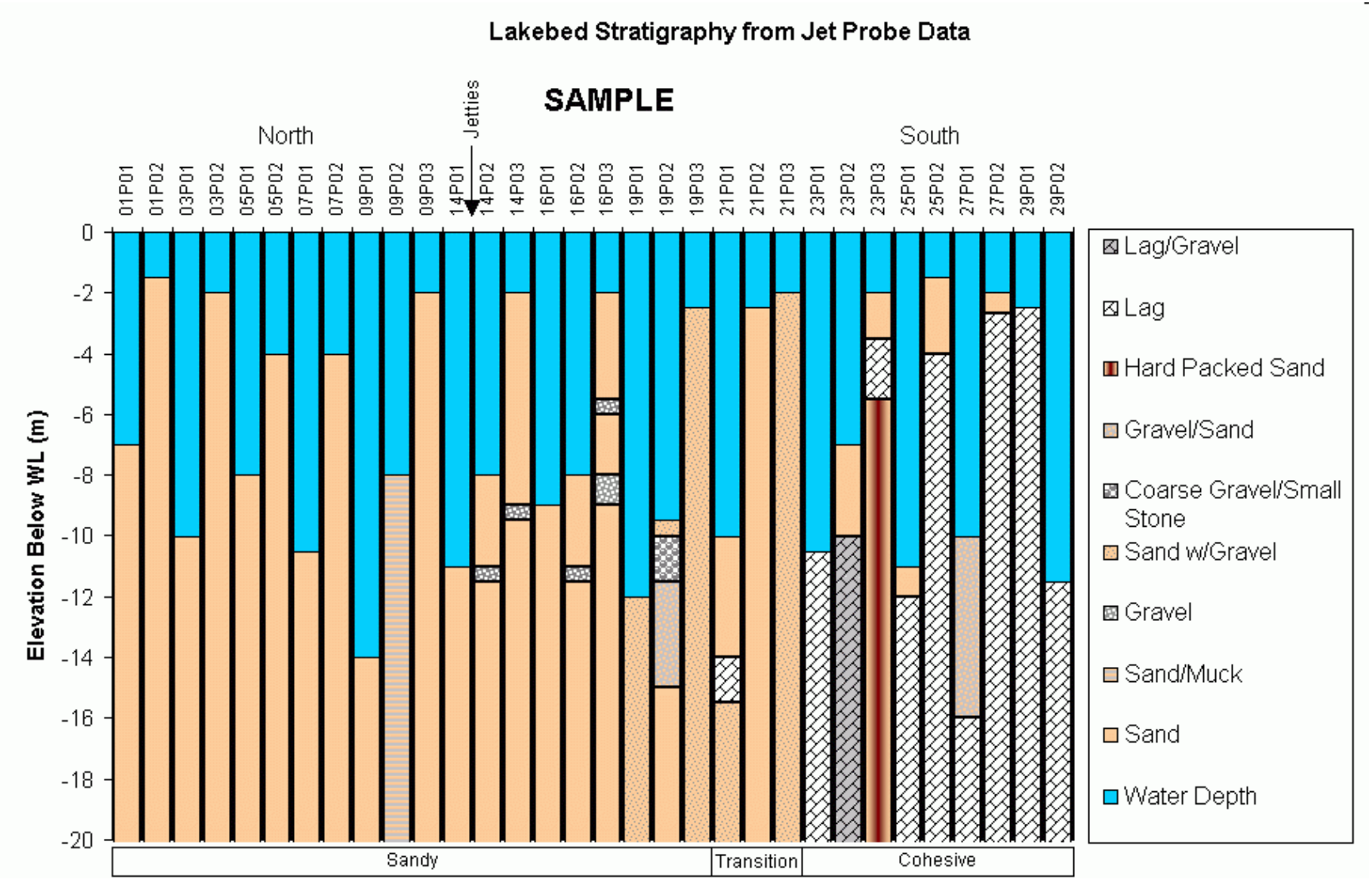


Figure 2-8. Sand thickness from jet probe data.

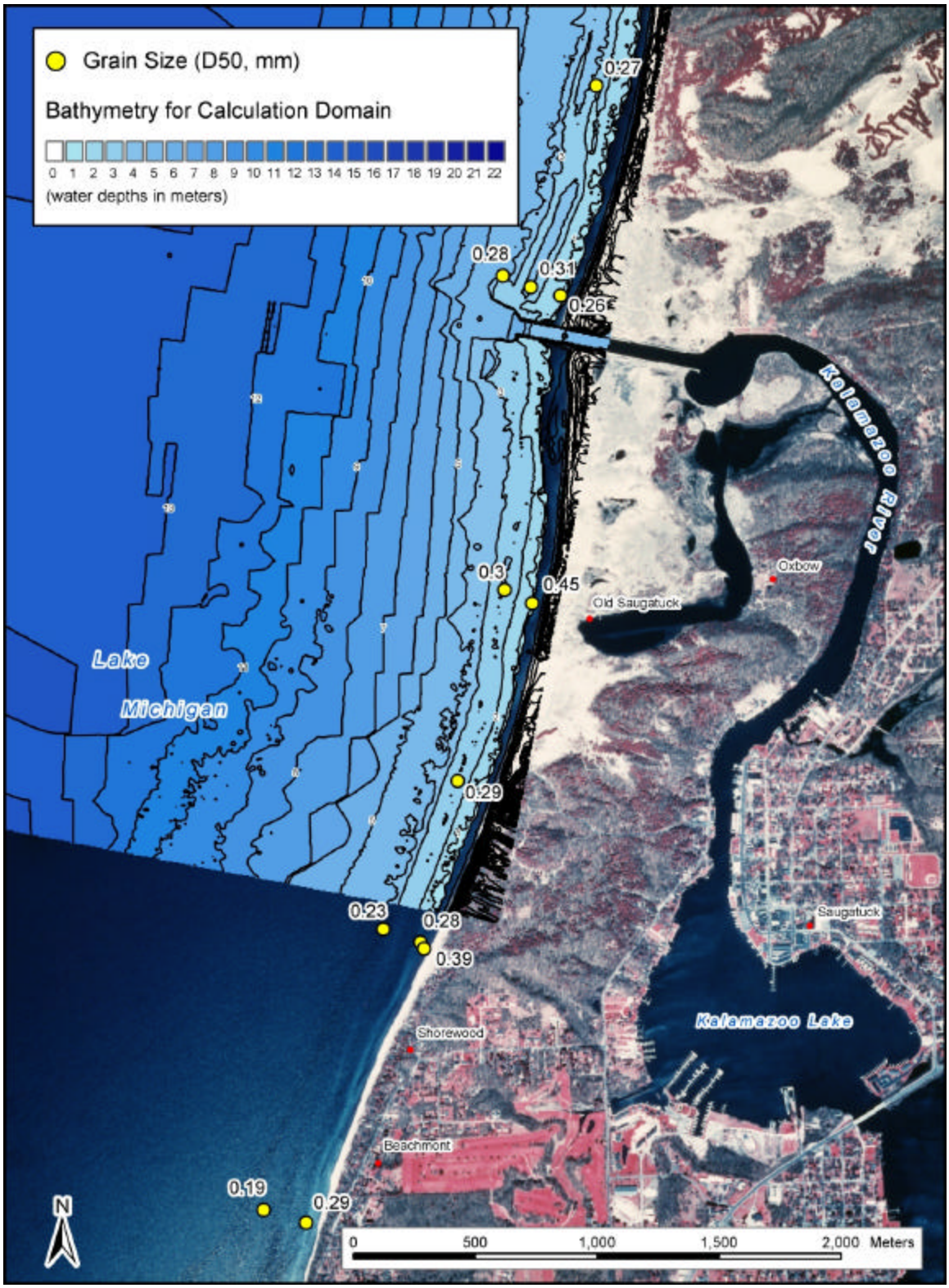


Figure 2-9. Sediment grain size.

Figure 2-10. Location of suspended sediment measurements.

Quality database for which 122 samples were available for the period from October 1973 to September 1986.

Measured suspended sediment concentrations were correlated with concurrent river discharge measurements (see Figure 2-11) and applied to the long-term daily discharge records to estimate the suspended sediment load at Kalamazoo Lake.

Average annual values from a time series analysis are shown in Table 3 and correlate well with estimates based on suspended sediment concentrations associated with average discharge rates at both discharge stations.

Table 3. Estimated Potential Suspended Load at Kalamazoo Lake.

Reference Station	Site Location	Average Daily Discharge (m^3/s) [a]	Average Annual Suspended Sediment Load (m^3/yr)	
			Based on Average Discharge	Based on Time Series
04108660	Kalamazoo River near New Richmond MI	56	16,400	16,500
04108500	Kalamazoo River near Fennville MI	42	12,300	12,100

[a] Assumes instantaneous discharge at the time of suspended sediment measurement equivalent to average daily discharge.

The suspended sediment loads in Table 3 are based on measurements upstream of Kalamazoo Lake and are therefore considered to be upper limit values for potential supply to downstream portions of the Kalamazoo River. In fact, it is expected that most of the suspended sediment load is actually deposited in Kalamazoo Lake due to the large cross-sectional area and significantly lower current speeds.

A review of the lakebed contours in Kalamazoo River and Douglas Bayou (NOAA Chart 14906) suggest that suspended sediment deposition probably does occur in these upstream areas (see Figure 2-12).

There is, however, insufficient information on the size distribution of suspended sediment to undertake a settling velocity analysis and estimate sediment deposition rates. It is likely that sand fraction, if any, would settle out in Kalamazoo Lake and only the silt and clay fractions would reach Lake Michigan.

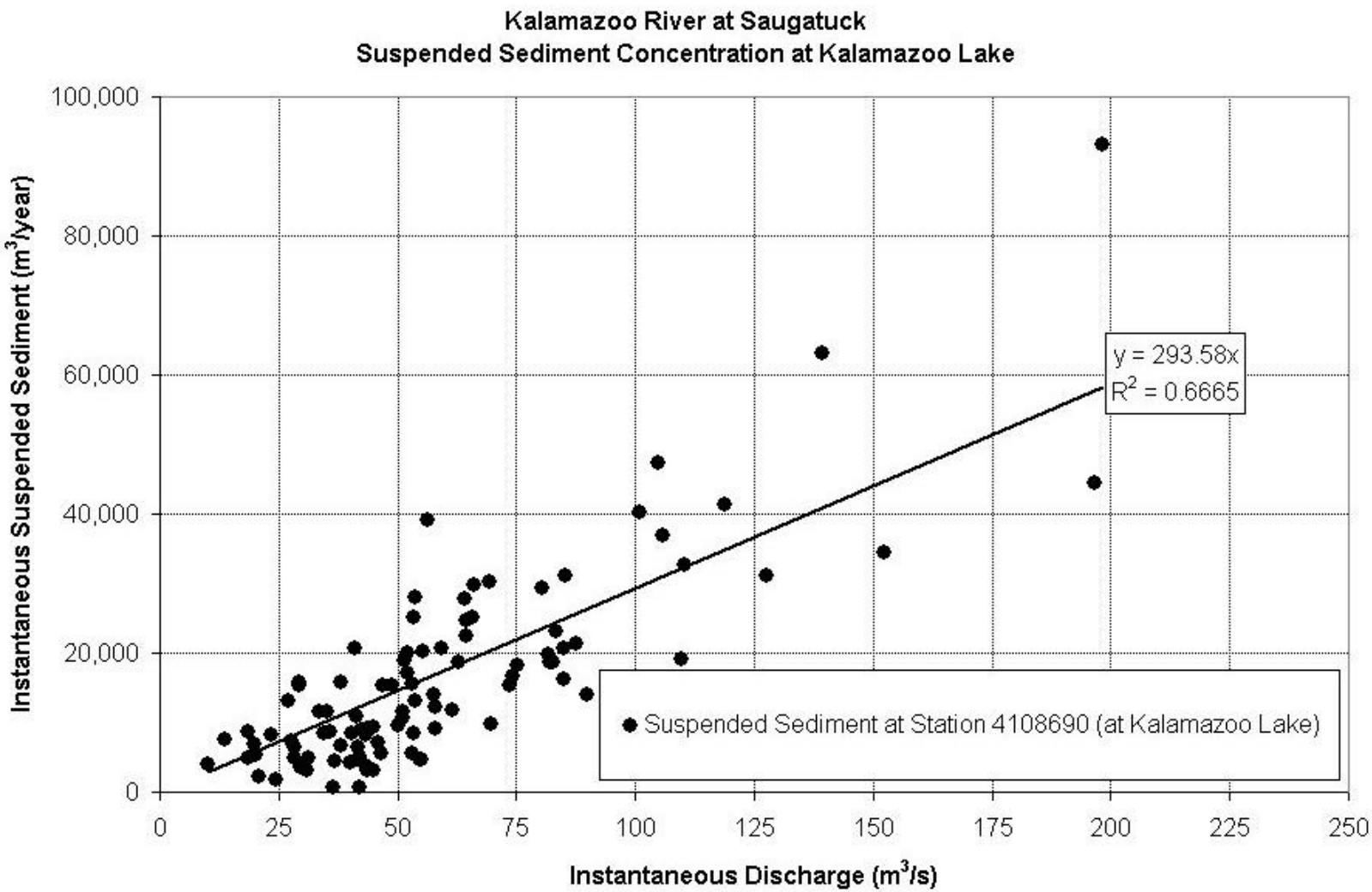


Figure 2-11. Suspended Sediment Correlation.

Figure 2-12. Saugatuck Harbor (NOAA Chart 14906 - 23rd Edition, 1997).

Bed-Load

The threshold velocity for 0.3 mm sand is approximately 0.4 m/s. Calculations based on the measured daily mean discharge rates and river cross-sectional geometry at the jetties suggest that velocities are less than the threshold value except in above average flow conditions.

Excluding high discharge scenarios, it is expected that most of the bed-load transport would be deposited in the lower reach of the Kalamazoo River or navigation channel and would probably be dredged before reaching Lake Michigan.

Based on the above limited information, sediment inputs from the river to the sand transport processes around the harbor were assumed to be negligible and were not included in the present study.

3. SHORELINE MORPHOLOGY

To some extent, a general understanding of local sediment transport processes and the influence of the federal harbor structures can be developed from indirect (regional) indicators and historical observations of past shoreline changes at Saugatuck and adjacent areas.

In this section, the results of three such analyses are presented to provide an initial impression of the past and present impact of the jetties. These analyses include the following:

- A review of historic recession rates for shoreline reaches to the North and South of the federal harbor,
- Calculation of sand volume accumulation in the adjacent fillet beaches, and
- A review of the local shoreline classification and regional geomorphology.

3.1 Recession Rates

In this section, dune and bluff recession rates are discussed. A dune is a ridge or hill, which forms when nearshore deposits of sand are blown to the back of the beach and colonized by vegetation. A bluff, on the other hand, is a steep cliff of consolidated glacial sediments that develops as a result of shoreline erosion by wave attack. The bluff face is generally void of vegetation due to active erosion processes.

Sandy reaches erode due to gradients in longshore sediment transport (LST) and cross-shore transport related to, among other things, cycles in water levels (rising levels). In a recent study by the Detroit District (USACE, 2001) it was determined that the published historical bluff and dune recession rates may not be sufficient for detailed sediment budget calculations. Baird therefore initiated a detailed GIS analysis of the bluff and dune lines for the LMPDS project on a reach-by-reach basis, and new (bluff top to top) recession rates for the period of 1938 to 1999 were calculated. Values of Annual Recession Rate and Annual Standard Deviation are summarized in Table 3-1 for the study area (from Holland, Reach 722, to Saugatuck, Reach 736). A negative value for ARR represents accretion. The dunes in the reaches next to the harbor jetties (reaches 732 and 733) advanced considerably during the historic low lake levels between 1955 and 1968. In other reaches the bluffs or dunes have been eroding since 1938 by as much as 0.42 m/yr (average for the period). Dunes, however, are very sensitive to the changes in water levels. They should be considered as part of the fillet beaches and their growth or disappearance is a result of sand volume exchange or redistribution inside the fillet system. The above erosion rates, therefore, cannot reflect the actual volume changes of the fillets.

Table 3-1 Annual Recession Rates in the Study Area (Reaches 722 to 736)

Reach	1938 to 1955		1955 to 1968		1967/68 to 1978		1978 to 1999		1938 to 1999	
(1 km)	Top to Top (m/yr)		Top to Top (m/yr)		Top to Top (m/yr)		Top to Top (m/yr)		Top to Top (m/yr)	
	ARR	ASD	ARR	ASD	ARR	ASD	ARR	ASD	ARR	ASD
HOLLAND JETTIES										
722	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
723	0.13	0.07	0.36	0.15	0.65	0.44	0.15	0.08	0.37	0.12
724	0.26	0.18	0.14	0.07	0.36	0.22	0.13	0.07	0.21	0.10
725	0.56	0.51	0.27	0.29	0.51	0.29	0.14	0.06	0.35	0.20
726	0.44	0.32	0.12	0.06	0.32	0.28	0.13	0.05	0.29	0.23
727	0.27	0.29	0.44	1.15	0.38	0.28	0.14	0.07	0.28	0.33
728	0.66	0.90	0.55	0.80	0.22	0.13	0.13	0.06	0.38	0.32
729	0.68	0.71	0.42	0.73	0.39	0.28	0.17	0.12	0.40	0.29
730	0.51	0.30	0.67	0.62	0.39	0.22	0.13	0.05	0.39	0.17
731	0.46	0.30	0.28	0.32	0.35	0.24	0.14	0.10	0.27	0.15
732 (0-250m)	0.52	0.12	0.33	0.30	0.30	0.14	0.13	0.04	0.55	0.06
732 (>250m)	-0.41	0.59	-8.94	3.55	0.44	0.17	0.14	0.17	-0.37	0.56
SAUGATUCK JETTIES										
733	0.40	0.11	-5.45	3.22	0.32	0.13	0.13	0.07	-0.86	0.63
734	0.61	0.29	0.39	0.14	0.22	0.12	0.14	0.04	0.33	0.10
735	1.15	0.57	1.55	1.68	0.48	0.39	0.13	0.04	0.76	0.41
736	0.57	0.23	2.43	1.42	0.75	0.35	0.12	0.03	0.82	0.32
Average	**0.52**	**0.37**	**0.61**	**0.59**	**0.41**	**0.26**	**0.14**	**0.06**	**0.42**	**0.22**

Reaches with insufficient data to calculate shoreline change rates

Top of Dune Crest shoreline change rates influenced by fillet beach

Negative values are accretion areas

3.2 Fillet Beach Volumes

The mouth of the Kalamazoo River was stabilized at Saugatuck in 1904 with the construction of two shore perpendicular jetties. Since 1904 sediment accumulation has occurred in the fillet beaches north and south of the jetties. The accumulation in these beaches represents a sink for the local sediment budget calculations at Saugatuck. The methodology for the detailed volume calculations is discussed in the following steps:

1. The 2001 acoustic lake bed profiles and topographic LIDAR data were used to assemble detailed 2D profiles from the dunes at the back of the beach to a depth of 15 m below Chart Datum;

2. The historic 1904 map of the harbor, the 1948 NOAA bathymetric soundings, the results from the numerical modeling of LST and Baird's experience from similar harbor investigations on Lake Michigan (i.e. St. Joseph jetties) were used to define a depth of closure for sediment accumulation from 1904 to present (i.e. maximum lakeward extent of sediment accumulation);

3. The historic 1904 shoreline computed from the 1904 deed by the USACE, Detroit District (9-10/179 drawing, December 20, 1999) was used to define pre-jetty 2D profiles at locations corresponding to the 2001 lines (see #1); and

4. 2D profile volumes from 1904 to 2001 were calculated with BMAP (Beach Morphology Analysis Package) in the CEDAS Package (Coastal Engineering and Design Analysis System, USACE). These 2D profile volumes were converted to 3D calculations based on the spatial extent of the existing fillet beaches (comparison of 1904 to 2001 shorelines). Table 3-2 below summarizes the volume calculations for the fillet beaches.

Table 3-2. Fillet Beach Calculations.

1904 to 2001 Volume Calculations		Total Volume of Sediment Accumulation Since 1904
North Fillet Beach (m^3)	South Fillet Beach (m^3)	
1,370,000	970,000	2,340,000

In summary, 2.34 million m^3 of sand have accumulated in the fillets at Saugatuck, of which 1.37 million m^3 has been impounded in the north fillet and the remaining 0.97 million m^3 in the south fillet beach.

3.3 Regional Geomorphology

Shorelines in the vicinity of Saugatuck Harbor are classified as sandy from Reach 722 (Holland) to Reach 736 (about 4 km south of the jetties) where it changes to cohesive (see Figure 2-6). Formation of fillet beaches may have caused these areas to become sandy over time. The present work focuses on the role of the harbor and its possible impact on the rate of sediment supply to the adjacent shorelines North and South of the harbor.

4. HYDRODYNAMIC AND SEDIMENT TRANSPORT MODELING

In order to better understand the behavior of the beach at Saugatuck and to clarify the patterns of sediment transport around the jetties and their role in the regional sediment budget, a hydrodynamic and sediment tranport analysis was completed. The analysis is conducted based on calculated wave field and depth averaged flows in two horizontal dimensions. The driving forces considered behind the movement of sediments are the wave-induced orbital motion, the nearshore current system including longshore and circulation currents, and the undertow.

The 2D model consists of three components, a spectral wave model (where the wave field is calculated by the spectral energy conservation equation), a hydrodynamic model to describe wave generated nearshore currents and circulations (driven by radiation stresses predicted with the spectral wave transformation model) and a sediment transport model based on the sheet flow transport formula of Dibajnia et al (2001). The sediment transport model considers the influence of the non-linear orbital velocities and the undertow. The advantage of this approach is that it provides a full spatial description of nearshore currents and sand transport in and around a full range of beach retaining structures including groins/headlands, breakwaters and submerged breakwaters.

Numerical calculations were performed to understand the nearshore current patterns around the jetties under different wave conditions. A 242×241 mesh with grid size of 10 m (resulting in a calculation area of 2410×2400 m) was selected for the hydrodynamic model. Figure 4-1 shows the calculation domain and its bathymetry.

4.1 Example Calculations

An incident wave height of 2 m, wave period of 9 s and wave direction of 302.5 degrees corresponding to NW waves was selected as the first example. Figure 4-2 shows the calculated wave height and directions for Mitsuyasu directional spreading $\boldsymbol{S}_{max}$ of 35 (i.e. swell with short decay distance and relatively large wave steepness, see Goda 1985). It may be seen that significant breaking occurs at the entrance of the harbor. Figure 4-3 shows the calculated nearshore current velocity vectors. A strong longshore current bypassing the jetties is observed. This current is believed to cause significant bypassing during storms. A circulation current is also observed downdrift of the south jetty. This circulation is responsible for the shape and location of the beaches next to the jetties.

Figure 4-4 shows the resulting sediment transport vectors. Convergence of transport rate in the mid-surf zone indicating the formation of a bar is observed. Transport rates around the head of the jetties are directed towards the southwest and therefore generate a southward bypassing component. The longshore component of transport was integrated over the cross-shore at each point along the shore and the results are summarized in

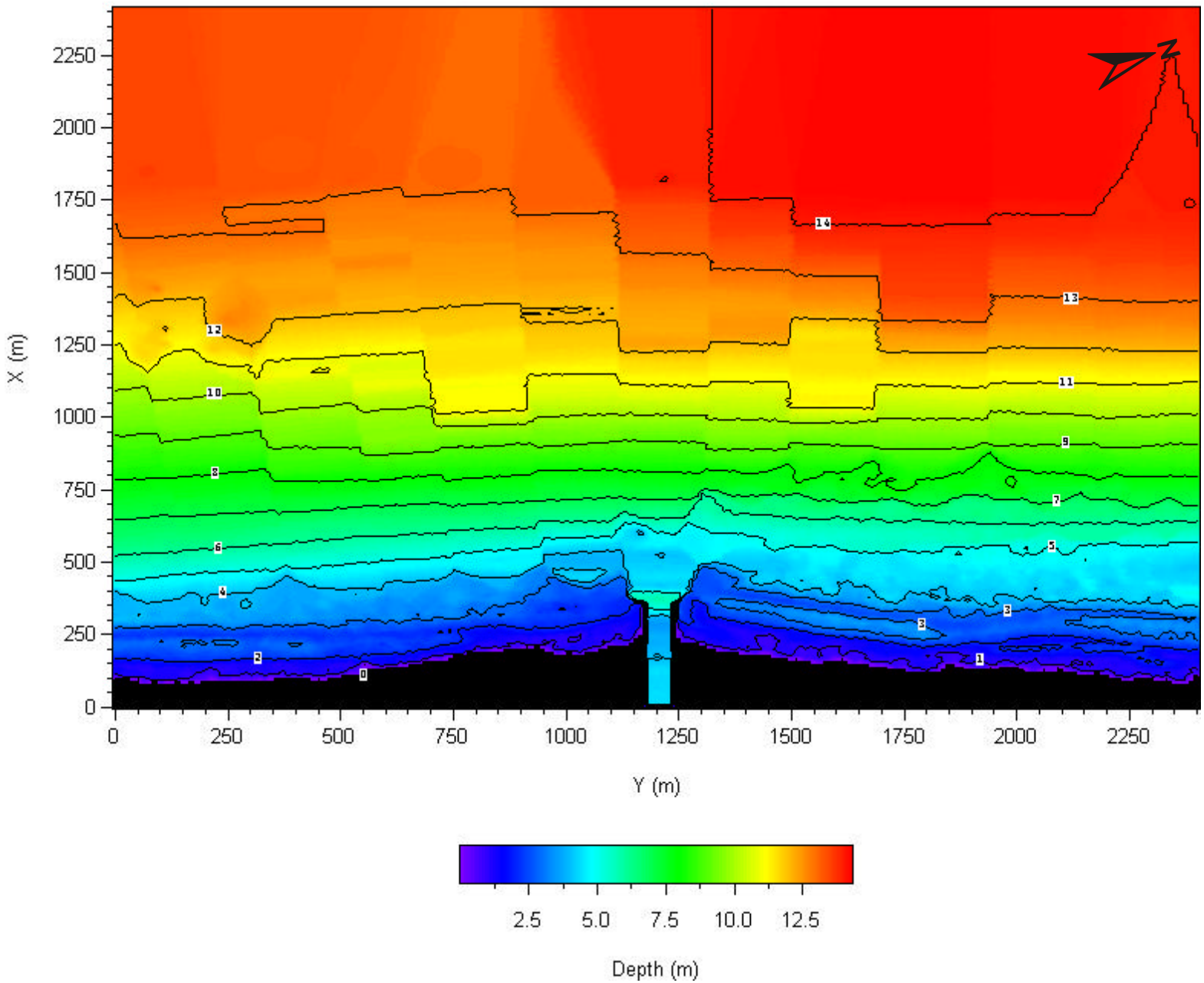


Figure 4-1. Calculation domain and bathymetry

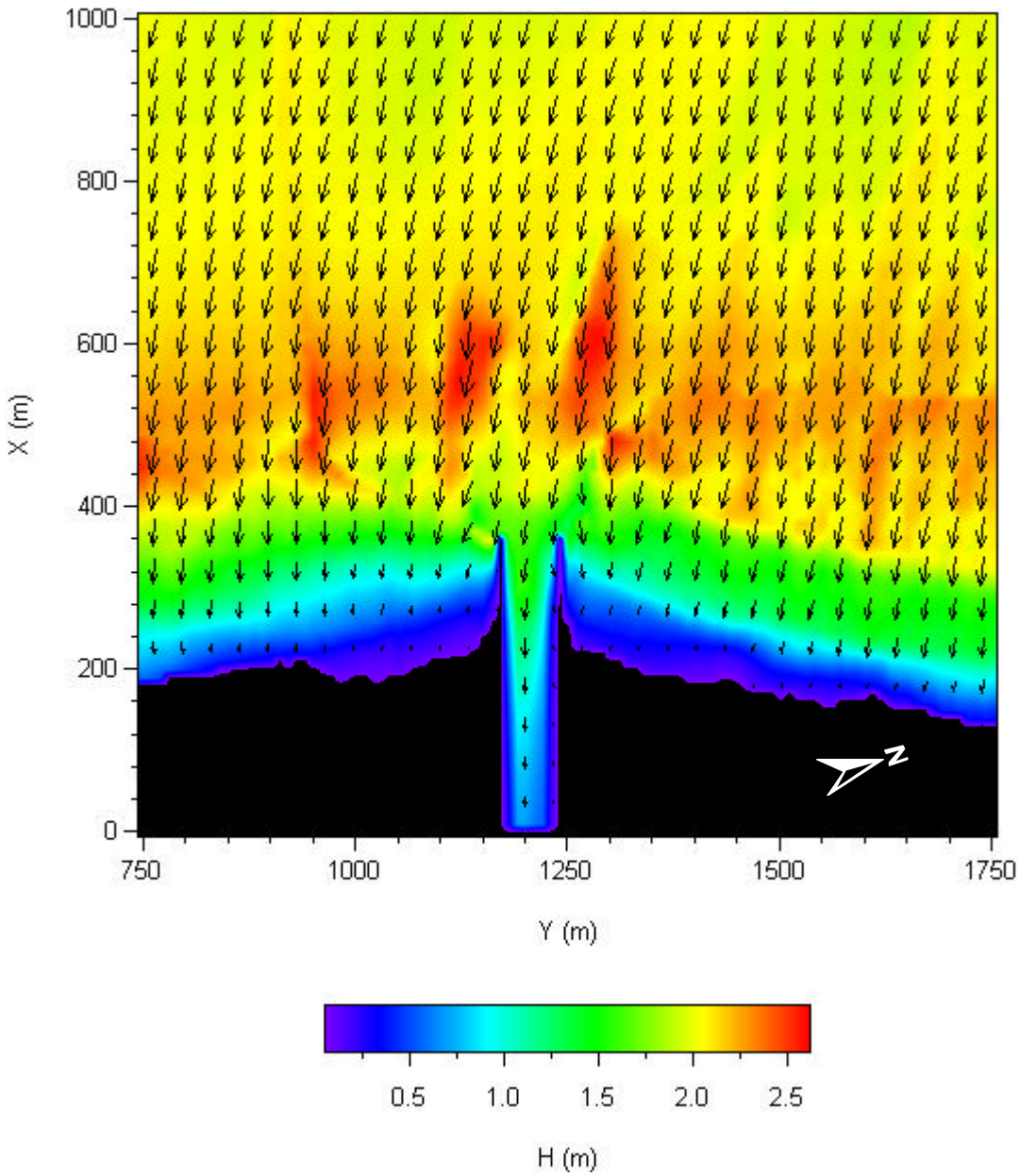


Figure 4-2. Wave height and direction around the jetties (Example case: H=2 m, T=9 s)

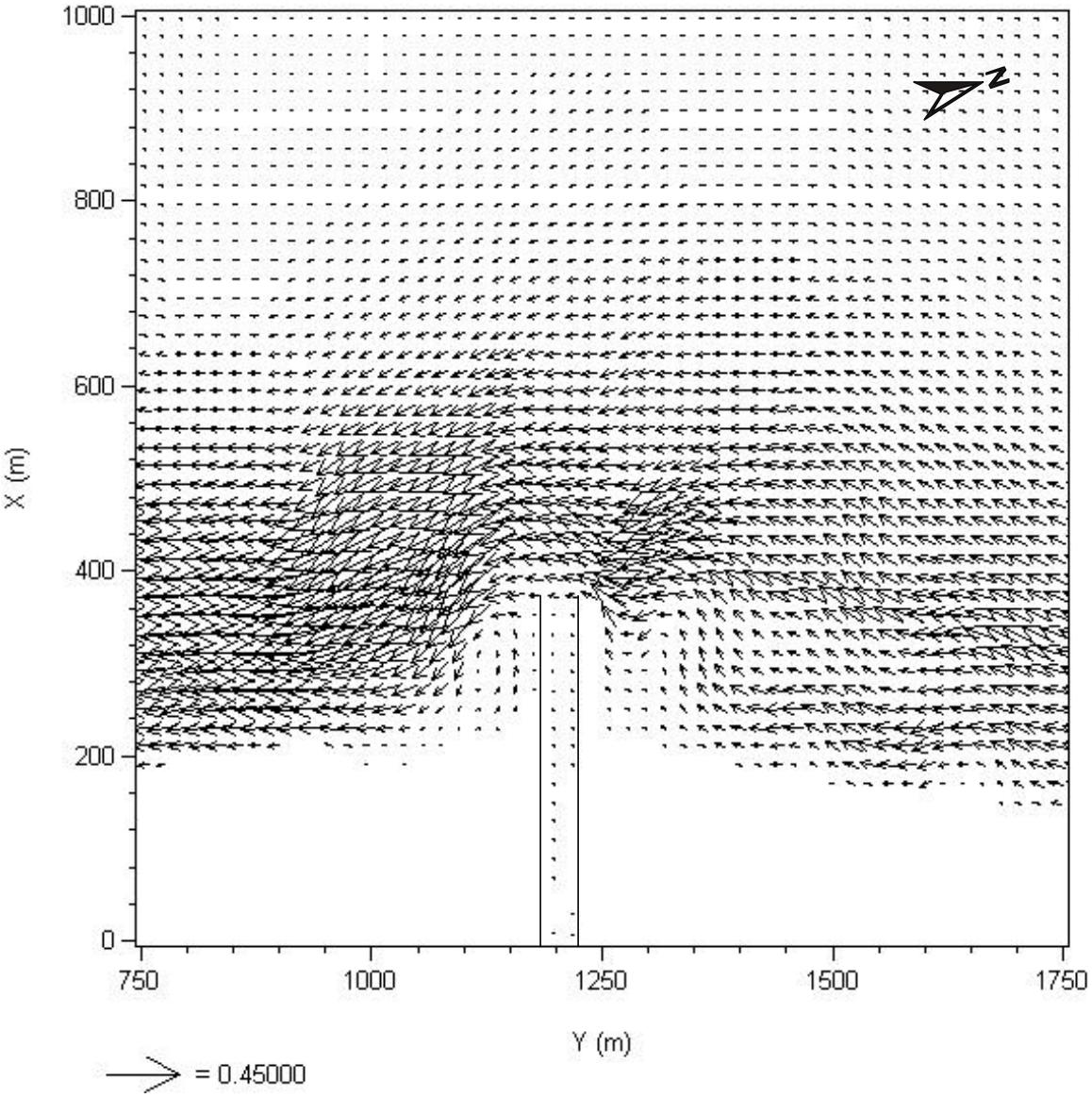


Figure 4-3. Nearshore current vectors (m/s) around the jetties (Example case: H=2 m, T=9 s).

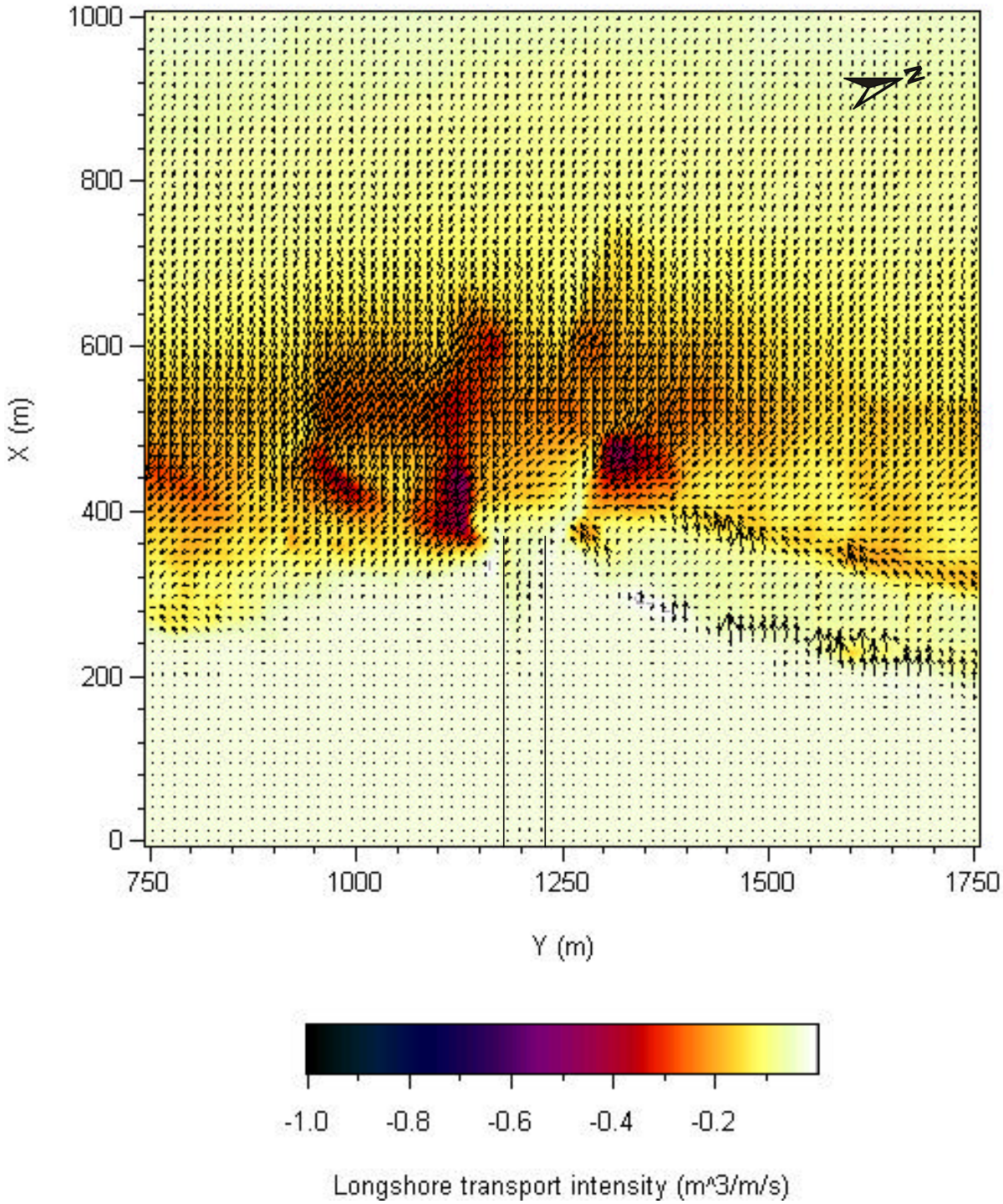


Figure 4-4. Sediment transport vectors around the jetties (Example case: H=2 m, T=9 s)

Figure 4-5. Although there is local variability in LST rates due to local bathymetry, in general there appears to be complete local bypassing of LST under this wave condition. The local bypassing ratio was defined as the average LST between the two jetties divided by the average incoming LST across the first 500 m from the northern (southern) side boundary of the calculation domain, where LST is almost uniform, for NW (SW) waves. In other words, the local bypassing ratio represents the amount of sand that locally passes from one side of the navigation channel to the other. It does not represent the overall bypassing rate for the harbor and neighbouring fillet beaches.

Next, a hypothetical uniform bathymetry with straight shoreline and the 360 m jetties in place is examined to reproduce the conditions just after the construction of jetties. The results are shown in Figures 4-6 to 4-8. The distribution of LST for this case is shown in Figure 4-5 for comparison with the previous case. It leads to deposition in the updrift fillet and erosion at the downdrift fillet, and indicates that in this case the local bypassing ratio is about 65% for this particular wave condition.

The third example is for similar conditions to the first but with a non-zero river discharge. The river discharge is 240 m^3/s (i.e. 40 m^3/s/calculation grid), which represents an extreme event in the record. Figures 4-9 and 4-10 show the nearshore currents and sediment transport vectors, respectively. It may be seen that the river flow causes a seaward sediment transport at the harbor entrance, which can potentially lead to formation of a bypassing bar. Alongshore distribution of LST for this case is shown in Figure 4-5. Compared to the zero discharge case, the LST decreases near the updrift jetty, indicating that the river flow acts as a barrier, reducing the local bypassing ratio to 65%.

It may be seen that the second and third examples both represent a local bypassing ratio of 65%. In the former example there is no fillet beach and the jetties work in full length to stop the longshore transport of sand. In the latter case, however, the river flow works as an additional barrier (extended jetty) that blocks the alongshore flow of sediment.

4.2 NW and WSW Storm Simulations

An analysis of storm waves (Hs greater than 2 m) showed that peak storm conditions are more often from the NW and WSW directions than from the West. As such, a typical NW event and WSW event were identified from the hindcast to simulate the detailed hydrodynamic and sediment transport conditions around the harbor jetties during storm events. The March 23rd 1972 and November 27th 1988 events were selected and details are summarized in Table 4-1.

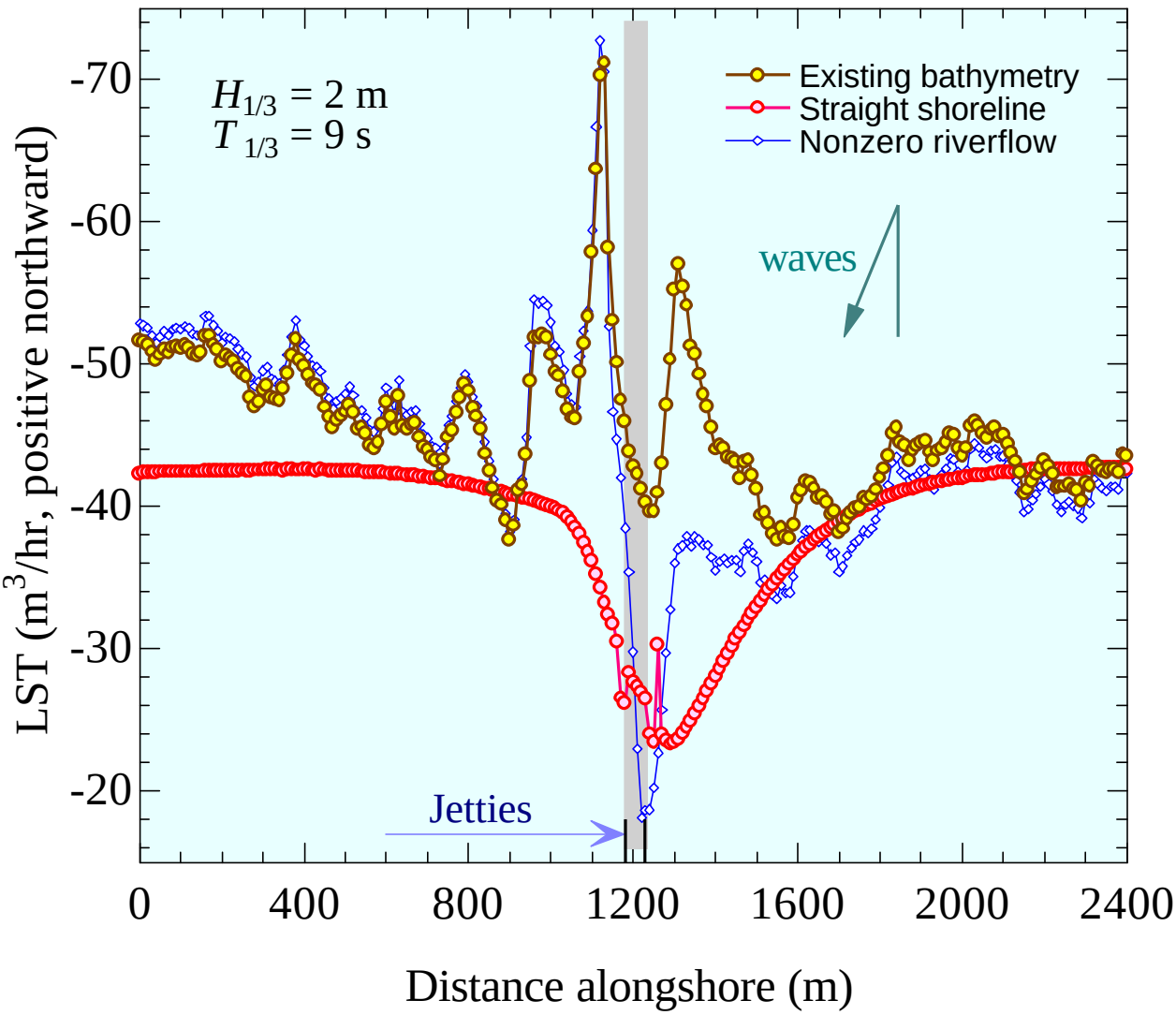


Figure 4-5. Alongshore variation of LST for existing and initial bathymetries.

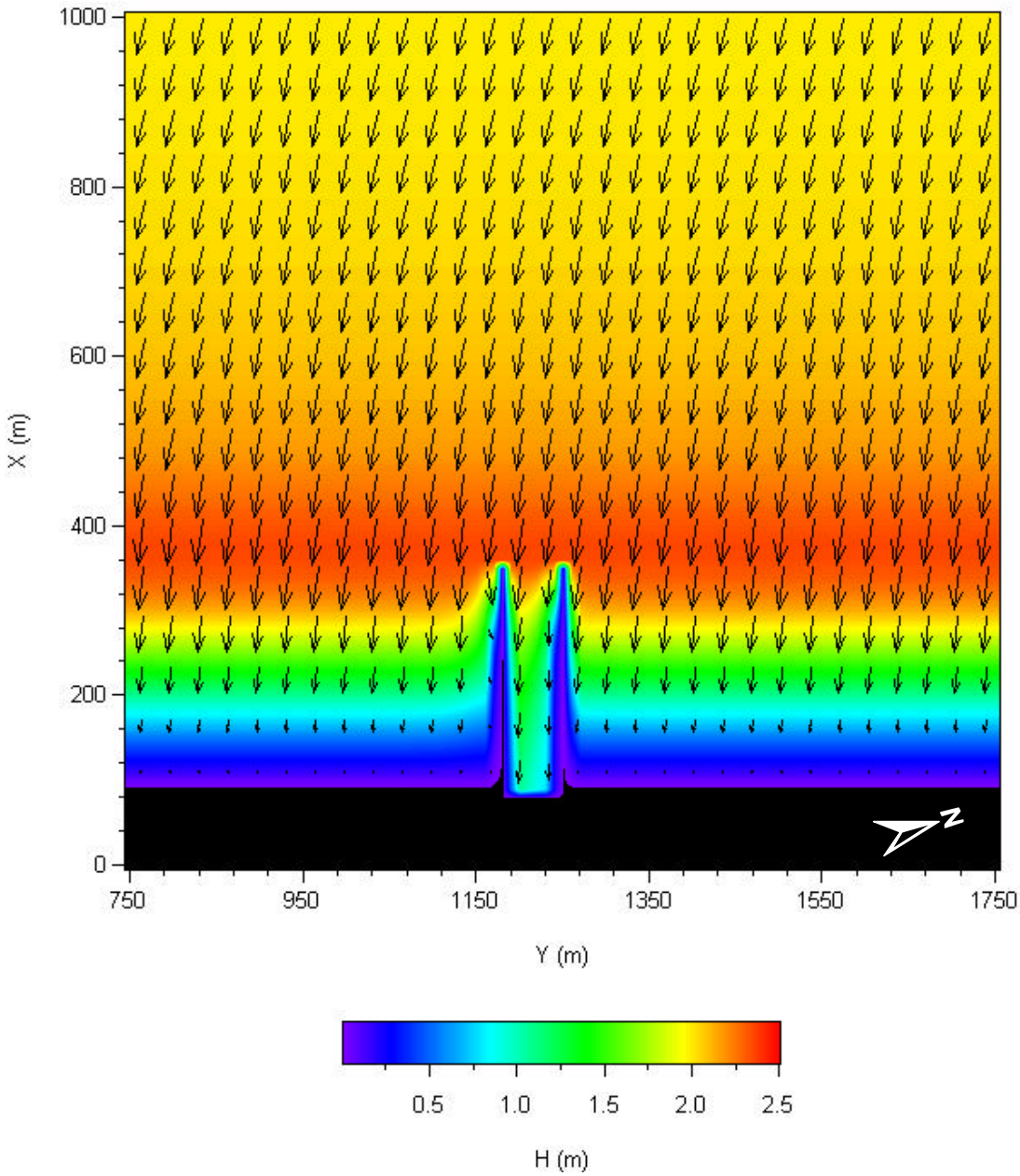


Figure 4-6. Wave height and direction for the straight shoreline bathymetry (Example case: H=2 m, T=9 s).

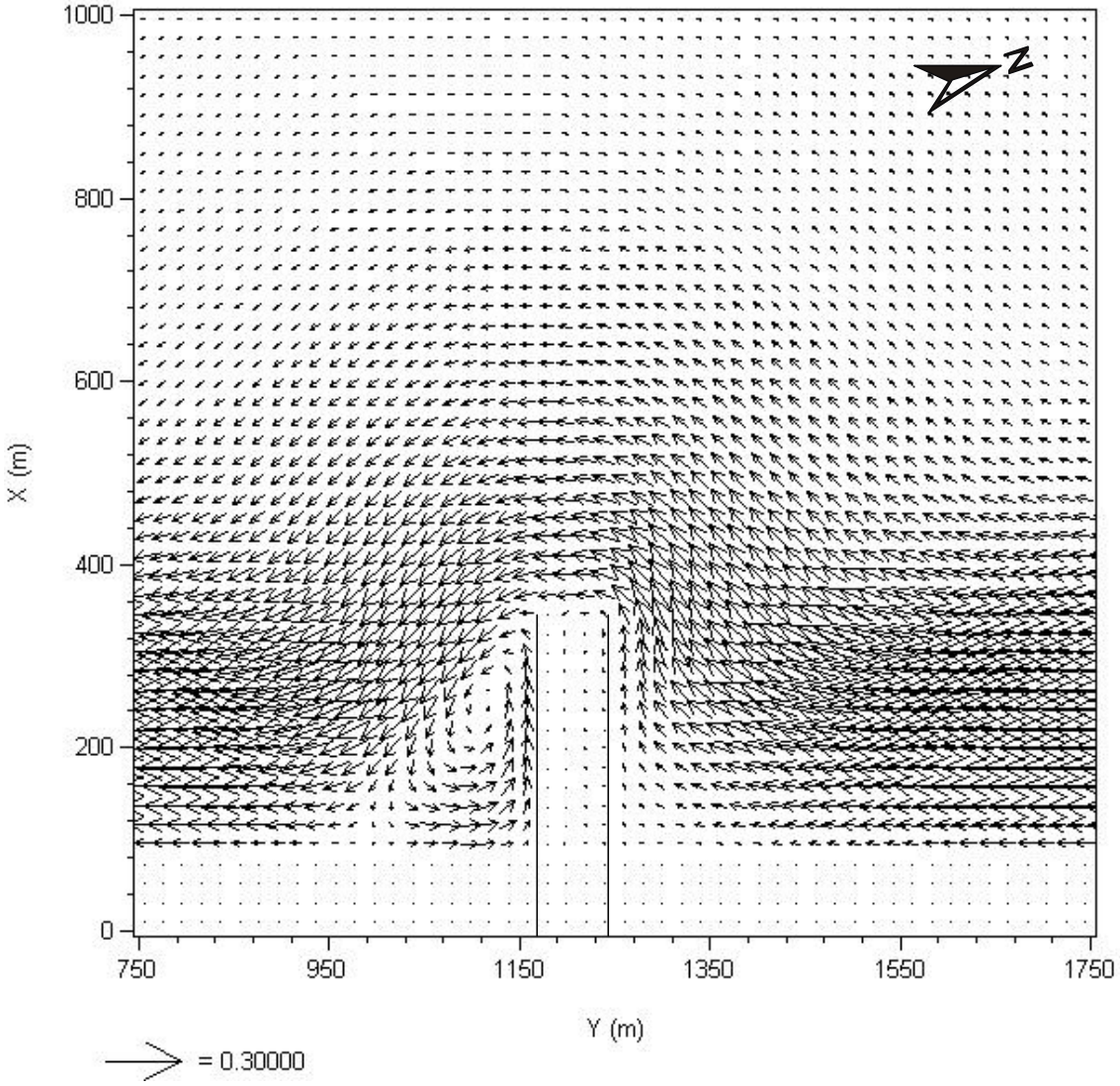


Figure 4-7. Nearshore current vectors (m/s) around the jetties for the straight shoreline bathymetry (H=2 m, T=9 s).

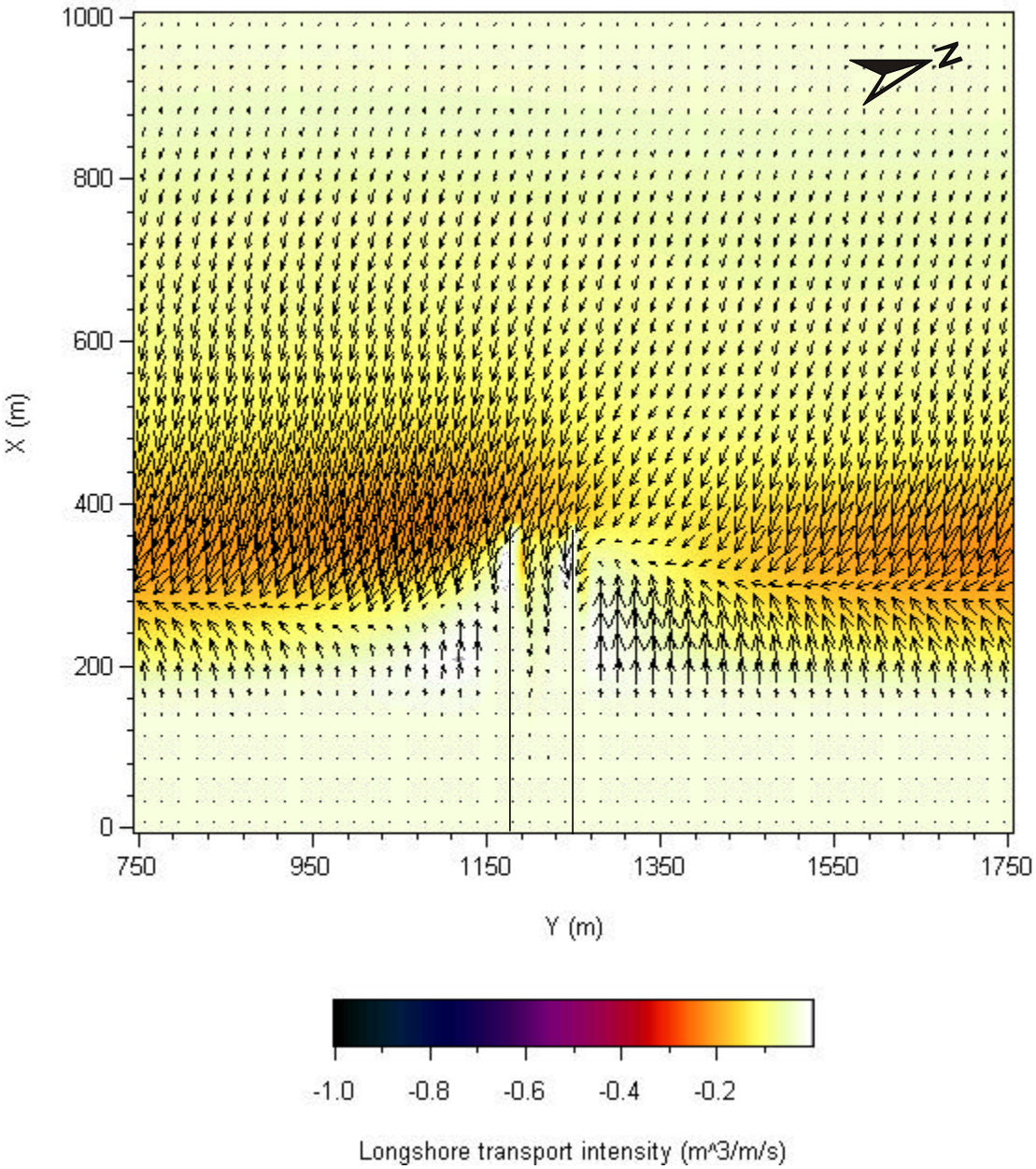


Figure 4-8. Transport vectors around the jetties – straight shoreline bathymetry (H=2 m, T=9 s).

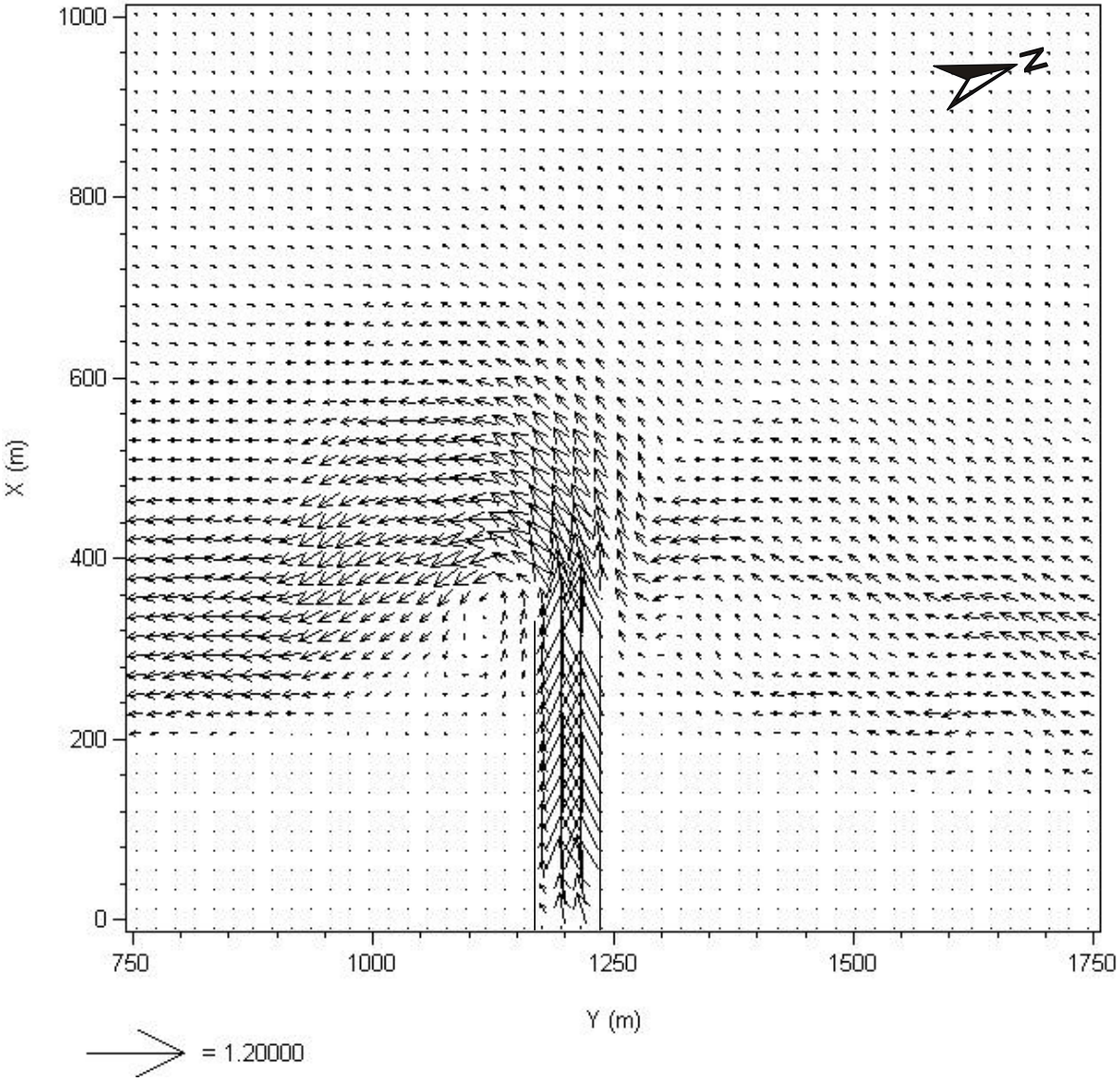


Figure 4-9. Nearshore current vectors (m/s) around the jetties with nonzero riverflow (existing bathymetry).

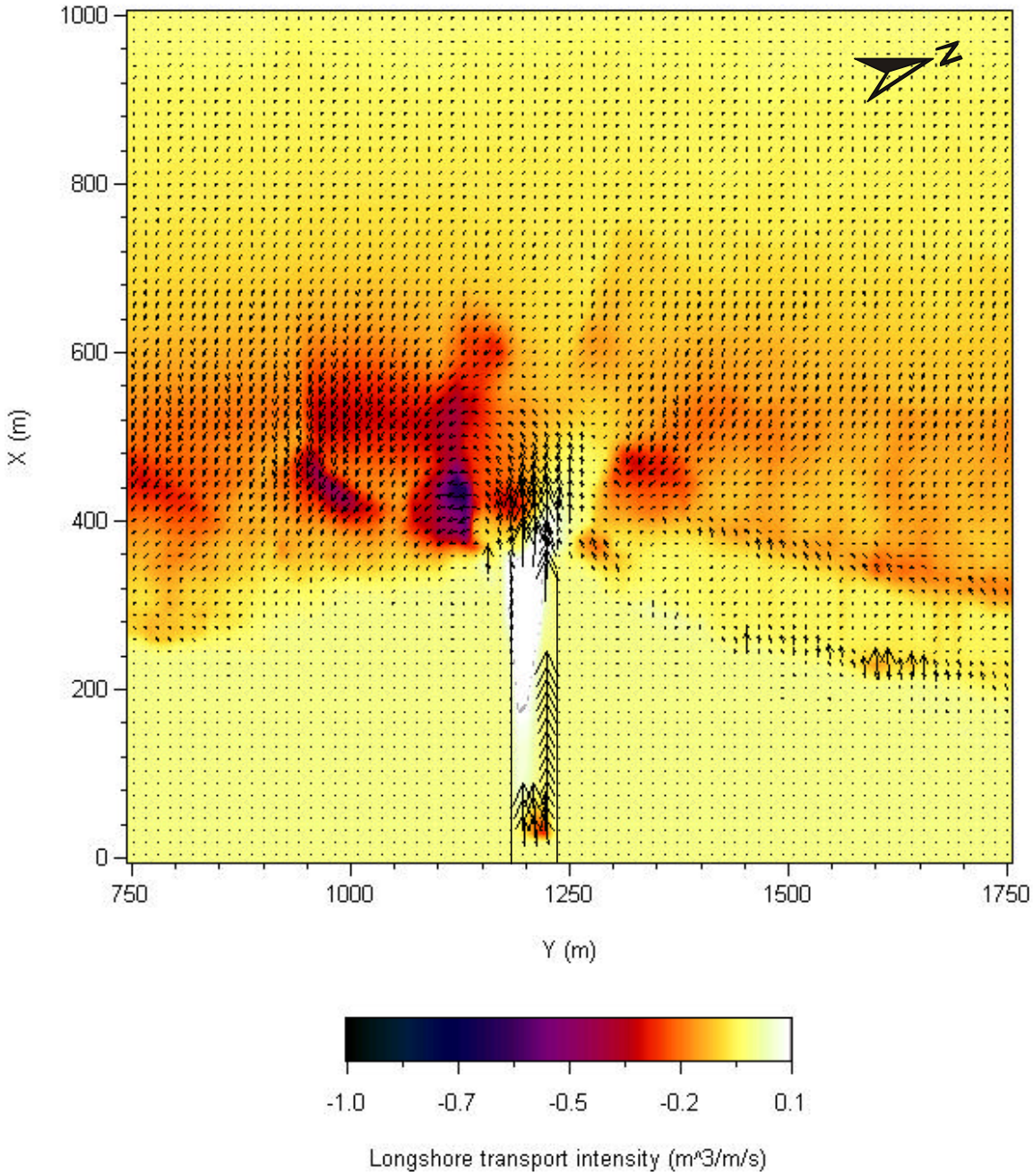


Figure 4-10. Sediment transport vectors around the jetties with nonzero riverflow (existing bathymetry).

Table 4-1. Storm Parameters for Selected NW and WSW Events.

Direction (from) [a b]	Date [a]	Hs (m) [a b]	Tp (s) [a]	Duration (h) [c]	Lake Level (m) [d]	River Flow (m^3/s) [e]
NW (307º)	72/Mar/23-04h	2.35	8.2	19	0.73	79.5
WSW (255º)	88/Nov/27-19h	2.33	7.9	14	0.47	74.0

[a] At storm peak.
[b] Transformed to 14 m water depth.
[c] Duration over threshold Hs = 2 m.
[d] Two-day average lake level (above IGLD'85 datum).
[e] Two-day average daily streamflow based on USGS Station 4108500.

Although associated average lake levels and river discharge were included from historic measurements, storm surge variations during the storm (computed from hourly water level data) were found to be of minor importance (less than 0.15 m) and were therefore ignored.

Statistically, the March 1972 storm is a typical annual event from the NW while the return period for the November 1988 event is approximately 5 years (WSW events tend to be milder). Referring to Table 1, river flows in the storm events are above average but less than the maximum on record.

Storm time-series were discretized into eight 6-hour steps for detailed hydrodynamic modeling over a 48-hour period (see Figure 4-11 and Figure 4-12). The resolution for wave heights and periods was 0.25 m and 0.25 s respectively, and directions were resolved to the nearest 15 degrees.

The main difference between the storms are their directional characteristics, which are described as follows:

1. The March 1972 event starts from the WNW on the rising limb, peaks at the NW, and clocks to the NNW on the falling limb.

2. The November 1988 event starts from the SW on the rising limb, peaks at the WSW, and clocks to the NW on the falling limb.

As a result, longshore sediment transport in the NW March 1972 event is north to south, while in the WSW November 1988 event it is from south to north (except in the NW tail). The longshore component of transport was again integrated in the cross-shore direction and Figure 4-13 and Figure 4-14 show the results at each 6-hour step for the NW and SW events respectively.

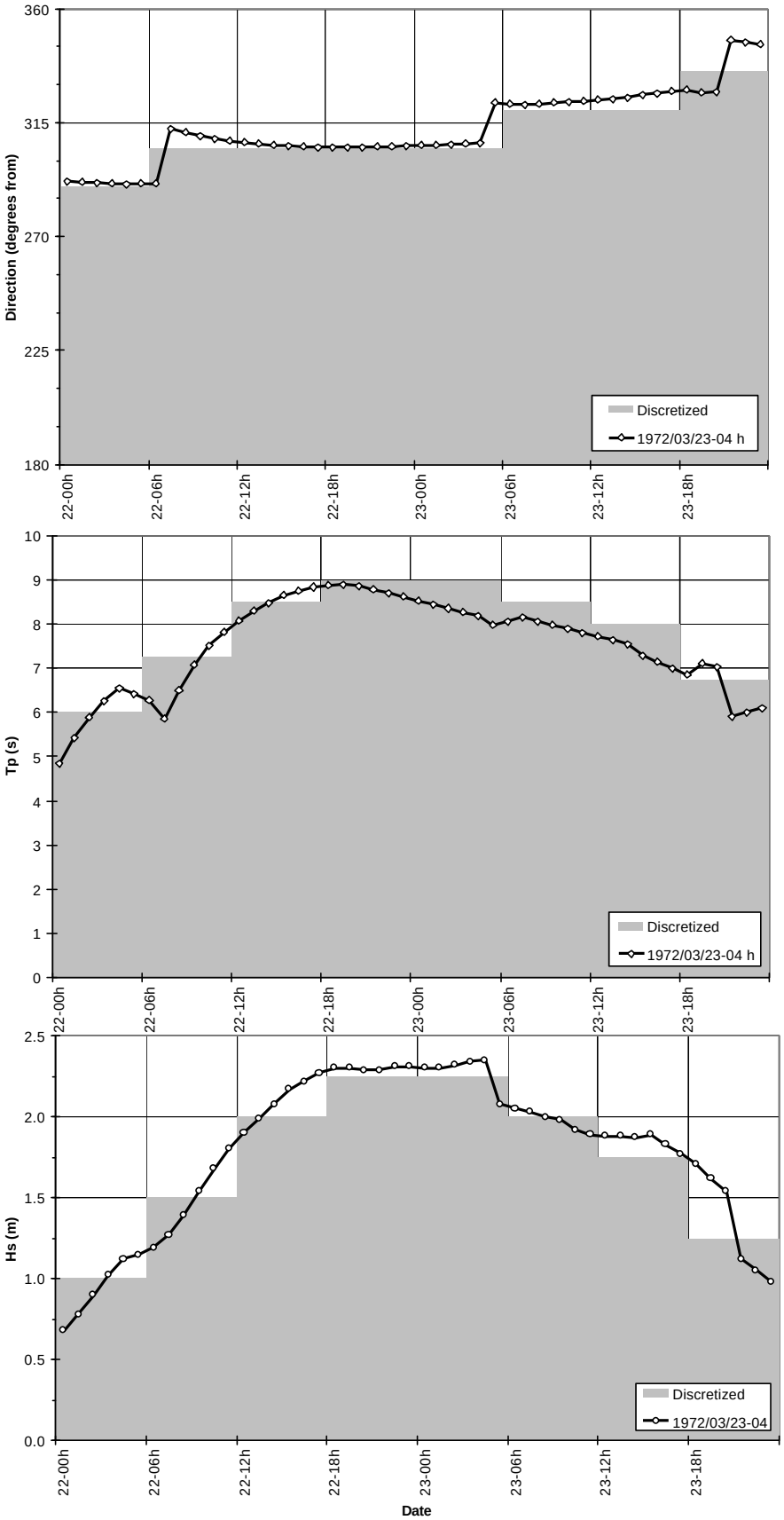


Figure 4-11. March 1972 NW Storm.

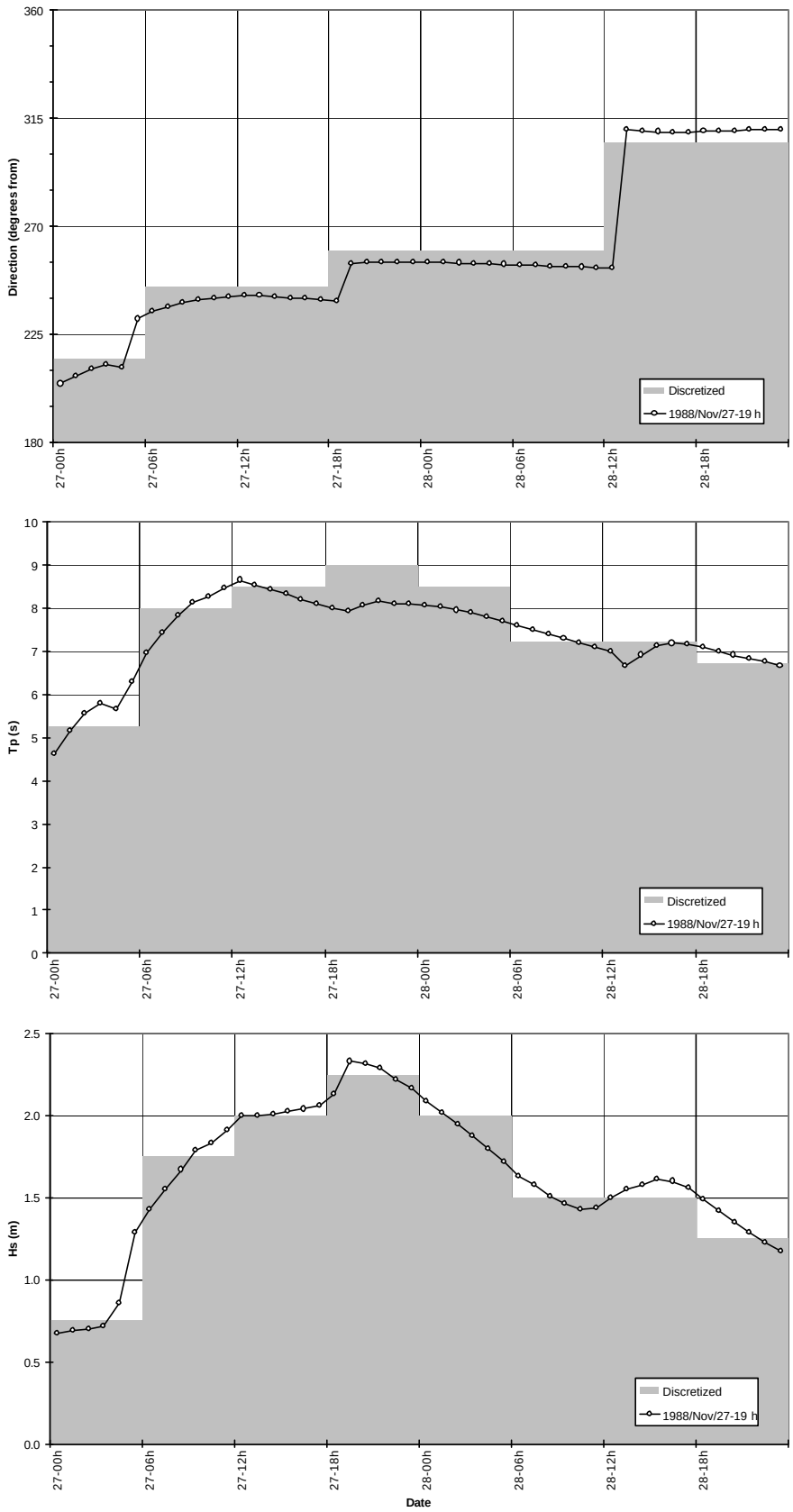


Figure 4-12. November 1988 WSW Storm.

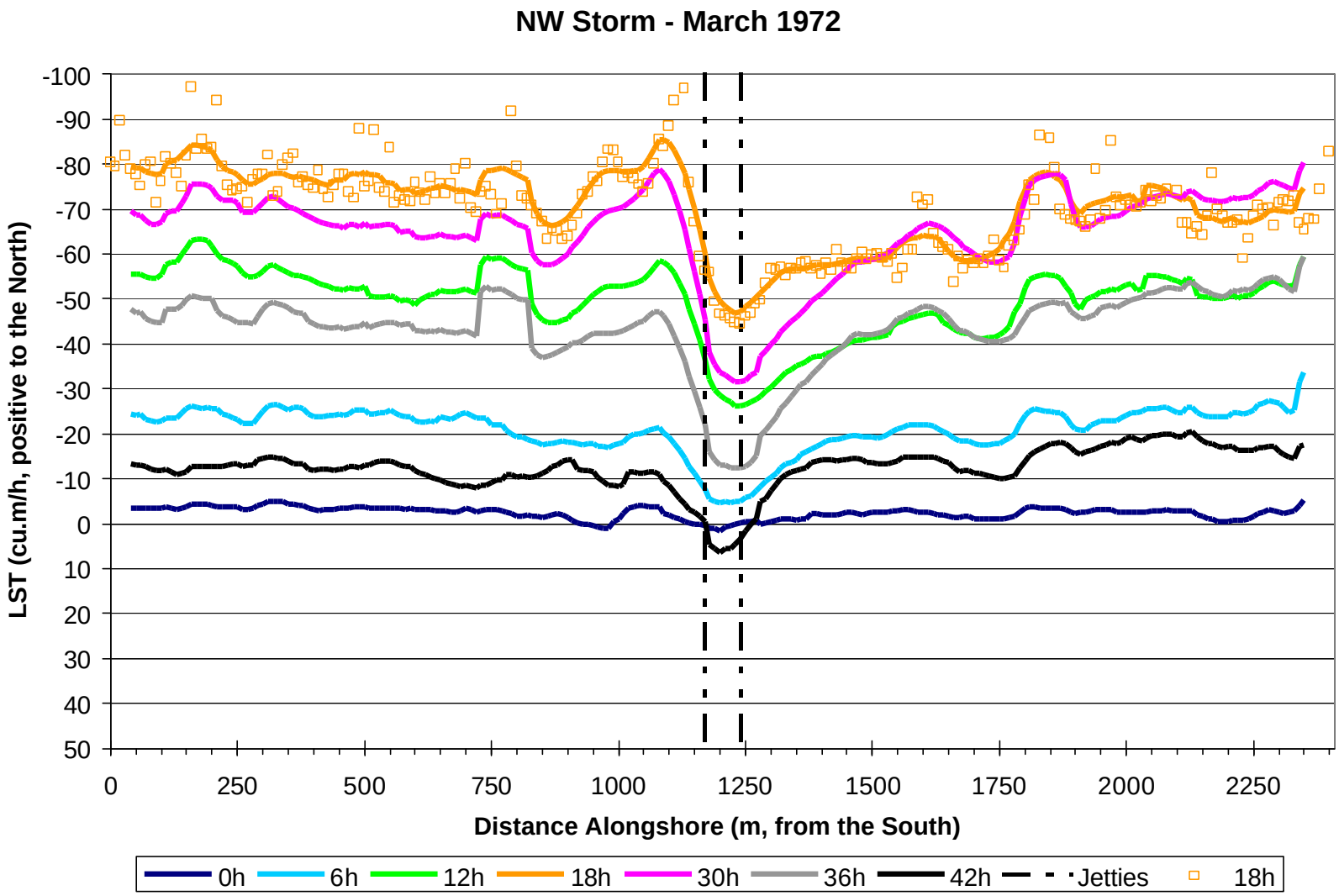


Figure 4-13. LST Trends (March 1972 NW Storm).

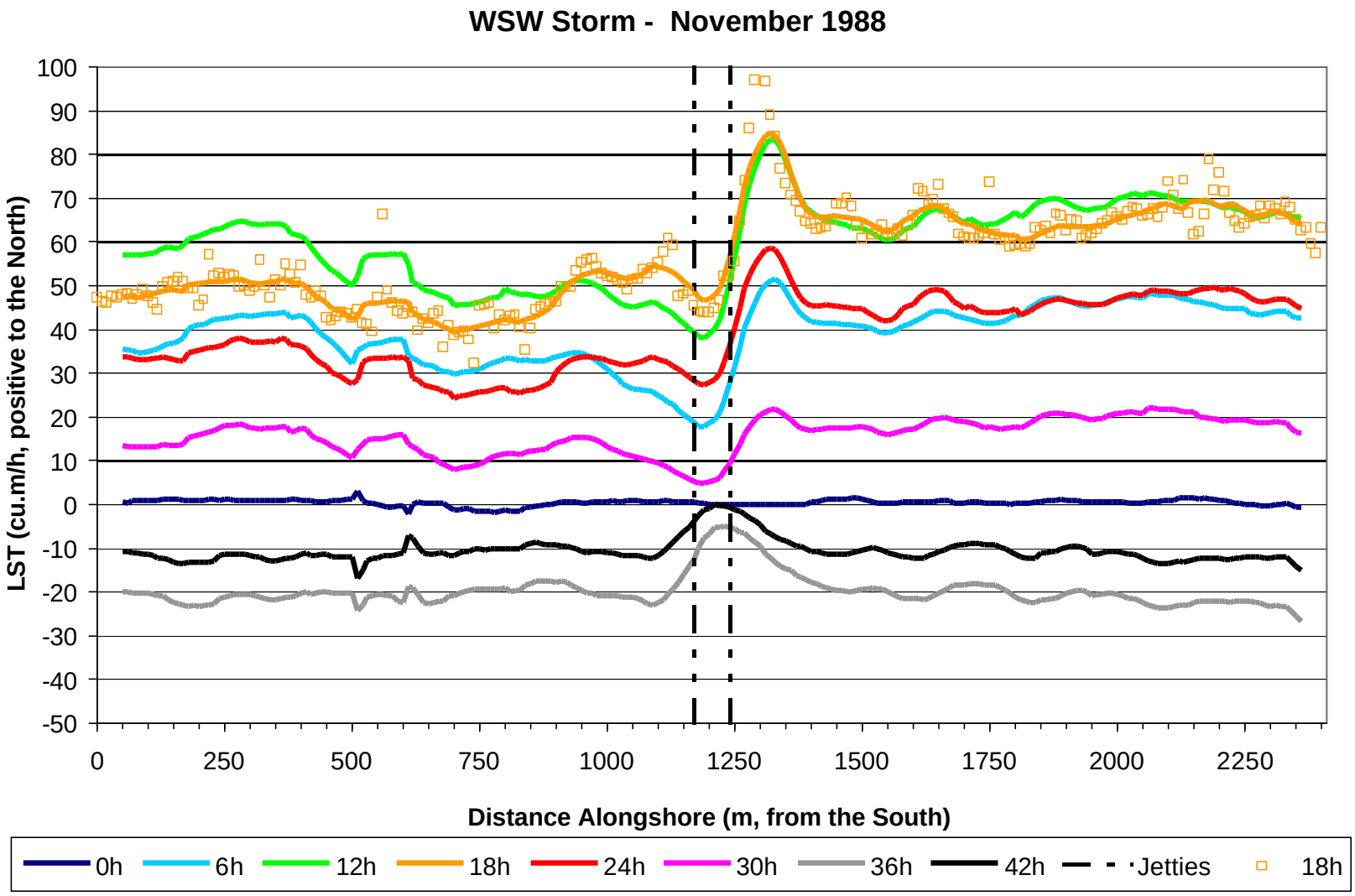


Figure 4-14. LST Trends (WSW November 1988 Storm).

The curves are a moving average of more variable individual values and help to clarify the spatial and temporal trends in LST (both variations in the alongshore direction and from one step to another in the storm sequence). In Figure 4-13 the point values and the moving average are compared for the 18 hour result.

The following general observations are made:

- LST is generally largest at the peak of the storm as would be expected.
- Strong gradients in LST occur at the jetties as a result of changes in longshore current speed and direction due to the influence of the river and local depth variations at the harbor mouth. Integrated LST on the downdrift side is generally slightly larger than on the updrift side due to the effect of the river flow - which appears to be redirected and assimilated into the prevailing longshore current.

Local bypassing ratios and LST rates for the NW and WSW storms are summarized in Table 4-2, which shows that local bypassing ratios are greater than 50% during the 18-hour period when the storm intensity is greatest. At the actual peak of the storm, when longshore sediment transport rates are generally greatest, local bypassing ratios are 68% and 95% for the NW and WSW events respectively. These results are consistent with those in Section 4.1 which showed high local bypassing ratios for similar wave conditions. Recall that the bypassing rate is locally defined (across the mouth of the harbor) and is not the same as the bypassing rate for the harbor and fillet together. This concept is explained in more detail in Section 5.

Table 4-2. Bypassing Ratio in Storm Simulations[a].

Storm		Storm Segment (h)							
		0-6	6-12	12-18	18-24	24-30	30-36	36-42	42-48
NW Mar 1972	Bypassing Ratio	-4%	16%	52%	68%	68%	45%	24%	-41%
	LST (m^3/h)[b] Direction	3 S	26 S	54 S	71 S	71 S	73 S	53 S	18 S
WSW Nov 1988	Bypassing Ratio	-3%	47%	66%	95%	80%	32%	27%	5%
	LST (m^3/h)[b] Direction	1 N	39 N	60 N	49 N	34 N	15 N	21 S	12 S

[a] Bypassing (%) = Average LST between jetties / Average LST at the first 500 m of the updrift boundary.
[b] Average LST at the first 500 m of the updrift boundary.

Table 4-2 also shows lower bypassing ratios for the rising and falling limbs of the storm sequence. Waves conditions are less intensive for these segments and it is expected that the river flow dominates over wave driven longshore currents to disrupt sediment transport across the mouth of the harbor.

4.3 Long-term Modeling

Although the above 2D model used in previous sections can provide valuable insight into the problem, it is computationally time consuming and, at present, it cannot be applied to an hourly time series of wave conditions spanning over more than several days. A method was therefore required to apply the results of the 2D modeling to a long-term analysis. A Determination was made to run the 2D model for a collection of representative wave, water level and river flow conditions and formulate the results as functions of the relevant hydraulic parameters. The idea was to use the resulting formulations in the long-term analysis rather than the 2D model itself, which would have been impractical. The formulations, however, are site-specific as they are obtained for the special arrangement of structures and bathymetry at Saugatuck.

A series of calculations were conducted to determine the overall trends of bypassing and its variation with changing incident wave conditions, water levels and river flow discharge. The applied wave conditions were selected based on a statistical analysis of the 50-year wave hindcast and cover five different wave heights, four wave periods, and seven wave directions (see Table 4-3). Bin settings were 0.5 m for wave heights, 2 s for wave periods, and 22.5 degrees for wave directions (16 point compass). Calm conditions less than *H*s = 0.5 m or *T*p = 4 s were not included in the analysis.

Table 4-3: Matrix of Wave Periods (*T*p) by Height and Direction [a]

Wave Direction [b]		Wave Height (*H*s) [c]				
Direction	Central Azimuth	1	1.5	2	2.5	3
SSW	202.5º	5,7				
SW	225º	5,7,9	5,7,9	7,9	7,9	
WSW	247.5º	5,7	5,7	7,9	7,9	
W	270º	5,7	5,7,9	7,9	7,9	9
WNW	292.5º	5,7	5,7,9	7,9	7,9	9,11
NW	315º	5,7,9	5,7,9	7,9	9	11
NNW	337.5º	5,7				

[a] Wave Period in seconds.
[b] Direction from.
[c] Wave height in metres.

The profile azimuth at the jetties is 280 degrees. The present bathymetry and the straight beach bathymetry (representing the shoreline right after installation of the jetties) were used. For the present bathymetry two water levels of 0 m and 1.4 m above Chart Datum were examined as representatives of historic low and high water levels, respectively. Calculations were conducted for three river flow discharges of 0, 20 and 40 m^3/s/calculation grid. The piers are 60 m apart and therefore there are 6 grid cells across the mouth. The selected river flow discharges thus correspond to total river flows of 0, 120 and 240 m^3/s, respectively. The sand grain size was 0.3 mm. In total, 471 cases were calculated requiring about 900 hours of model simulation.

In order to classify the results, a parameter is required to represent the above environmental conditions appropriately. Many authors have formulated the longshore sand transport (LST) rate as a function of the longshore component of the energy flux entering the surf zone. We employed the Kamphuis (1991) approach because of its simplicity and used the following parameter:

$$\Theta = H^2 T^{1.5} \mathrm{Sin}^{0.6}\ 2a$$

where H, T and a are the incident wave height, wave period and wave angle, respectively. It should be mentioned that the original formulation includes the bottom slope and the sediment grain size, which are omitted here because they are the same for all the cases. Also in the original formulation, the above values are to be specified at the breaking point. However, the values at the offshore boundary of the calculation domain were applied because the parameter was used only for classification of the results.

Figure 4-15 shows the relation between calculated LST rates (average incoming LST across the first 500 m from the side boundary of the calculation domain) and the parameter Θ. Negative values for Θ correspond to NW waves. A well-defined trend is observed that can be formulated as indicated in the figure. It should be mentioned that the LST rates are the output from the 2D model without any calibration and therefore do not necessarily match actual values. This is, however, of minor concern because the main interest is in ratios of bypassing and accumulation rather than their actual quantities. For the actual LST values, the COSMOS simulations will be used.

Figure 4-16 is a plot of the calculated bypassing ratios for three different river flows versus the parameter Θ, when the water level is at Chart Datum (i.e. =0). Each point represents one of the 471 model simulations. The figure thus shows how the bypassing ratio changes with the river flow discharge. The solid lines in this figure indicate the formulations established for this study. The variation is particularly significant for smaller values of Θ (lower LST rates) so that there could even be considerable inverse bypassing (negative bypassing ratio) under the higher flow rate of Q=40 m^3/s/calculation grid. In such a case, the diffusion of the outgoing river flow dominates the weak longshore current so that the LST at the harbor mouth is in the opposite direction of the LST coming into the calculation domain. This is, however, an extreme river flow rate that rarely

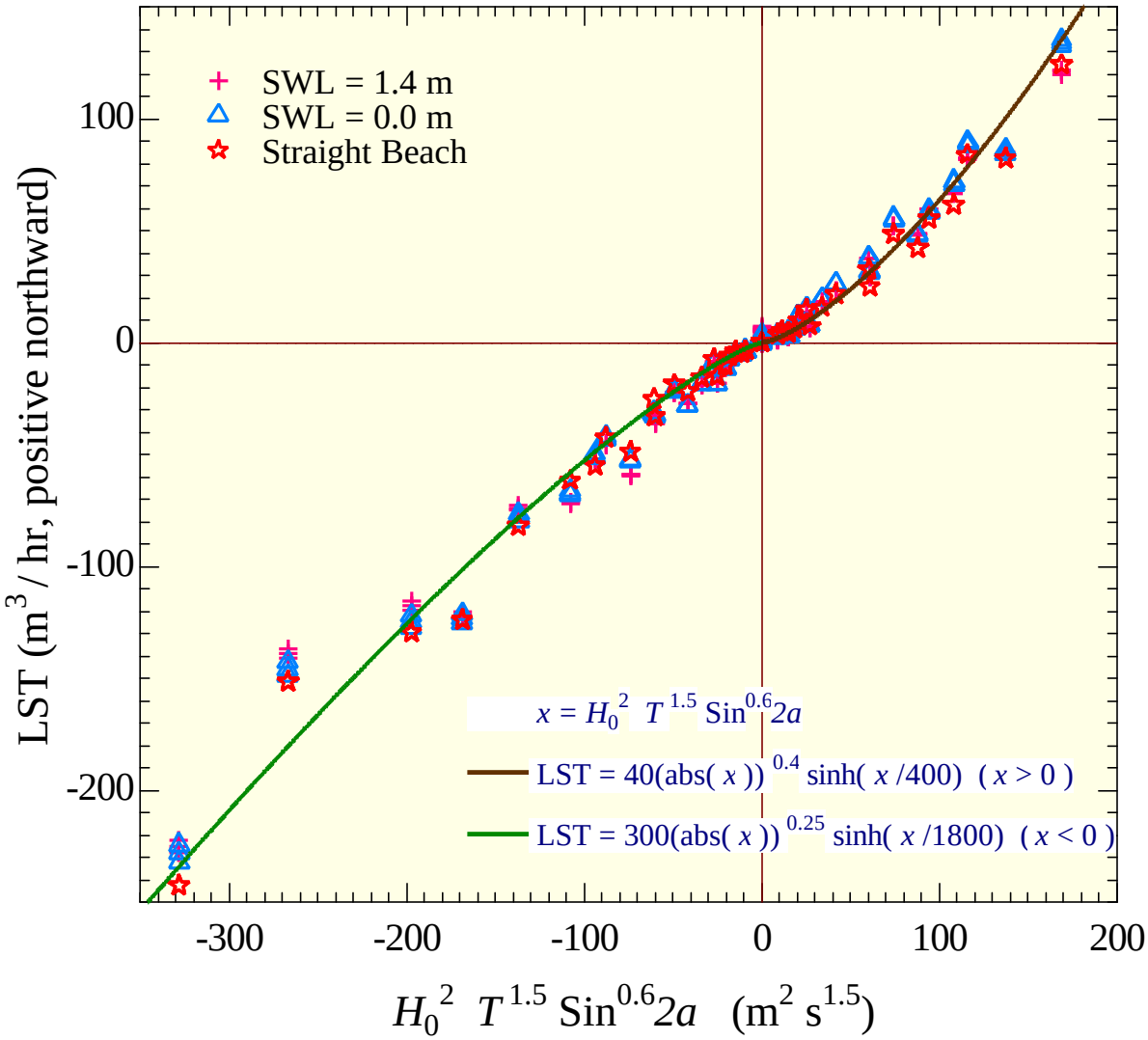


Figure 4-15. The selected parameter vs calculated longshore sand transport rates. Positive and negative values of the horizontal axis correspond to SW and NW waves, respectively.

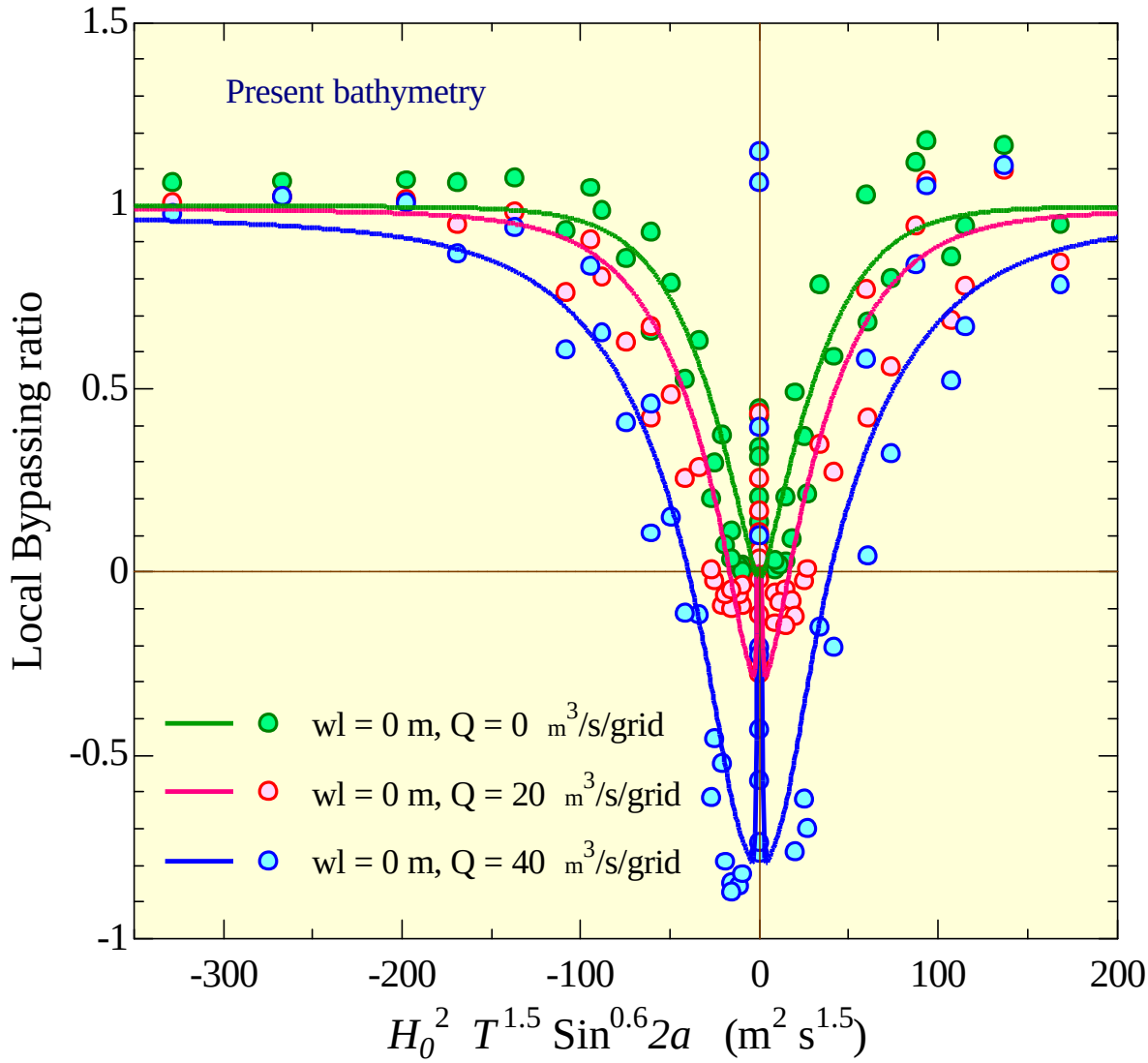


Figure 4-16. Effect of river flow on the local bypassing ratio (water level=0.0 m). The points are from 2D model results and the solid lines are the fitted curves to them. Positive and negative values of the horizontal axis correspond to SW and NW waves, respectively.

happens. Also the effect is for smaller values of Θ, or in other words for low LST rates that are of minor concern anyway. For larger values of Θ the river discharge has only a minor effect on the bypassing ratios.

Figure 4-17 shows the same results when the water level is 1.4 m above Chart Datum. It may be seen that bypassing ratio is less sensitive to the river discharge under this higher water level. This is simply because under a higher water level, the depth is greater, and the depth-averaged velocity due to the river flow becomes smaller. The solid lines in this figure again represent the formulations developed for this study. Figure 4-18 shows the bypassing ratios for a discharge of Q=20 m^3/s/calculation grid over the present bathymetry with the two different water levels as well as over the straight shoreline bathymetry. It is observed that bypassing ratios are generally higher for the lower water level. The results for the straight beach did not change with the river flow discharge because the tip of the jetties was located outside of the surf zone due to its distance from the shoreline (360 m). Bypassing ratios were formulated as functions of Θ for different water levels and river flow discharges and are given as an Appendix B at the end of this report.

4.4 Analysis of Bypassing Rates for NW and SW Waves

The bypassing and LST rate formulations obtained in Section 4.3 were applied to the 50-year wave hindcast data in a time series analysis to obtain the overall local bypassing ratios at Saugatuck. It will be discussed in Chapter 5 of this report that the LST at Saugatuck has northward and southward components of the same order of magnitude. It is very important to understand that the harbor in the past has behaved as a barrier (i.e. the jetties) and a sink (i.e. the fillet beaches) against both of the LST components. Bypassing of sediment also occurs in both directions but to different extents because the waves and consequently the bathymetry on both sides of the harbor are not identical. It is therefore necessary to define bypassing and deposition parameters in two directions separately, and then integrate the results to evaluate the overall influence of the harbor. The 50-year wave data was thus split in NW and SW wave files to estimate the long-term bypassing ratios corresponding to waves from the two directions. The river flow discharge data and the monthly mean water level data were also used. The wave files consisted of successive monthly scatter data because the water level data was monthly. The river flow data were also converted to monthly means. Although this eliminated the peak flow events, its influence was expected to be minor because: 1) duration of peak flow events is short, 2) high river flow is not usually coincident with intense wave activity, and 3) it was found in previous sections that the river flow has minor impact on high LST rates. The 50-year average local bypassing ratios were obtained as shown in the following table.

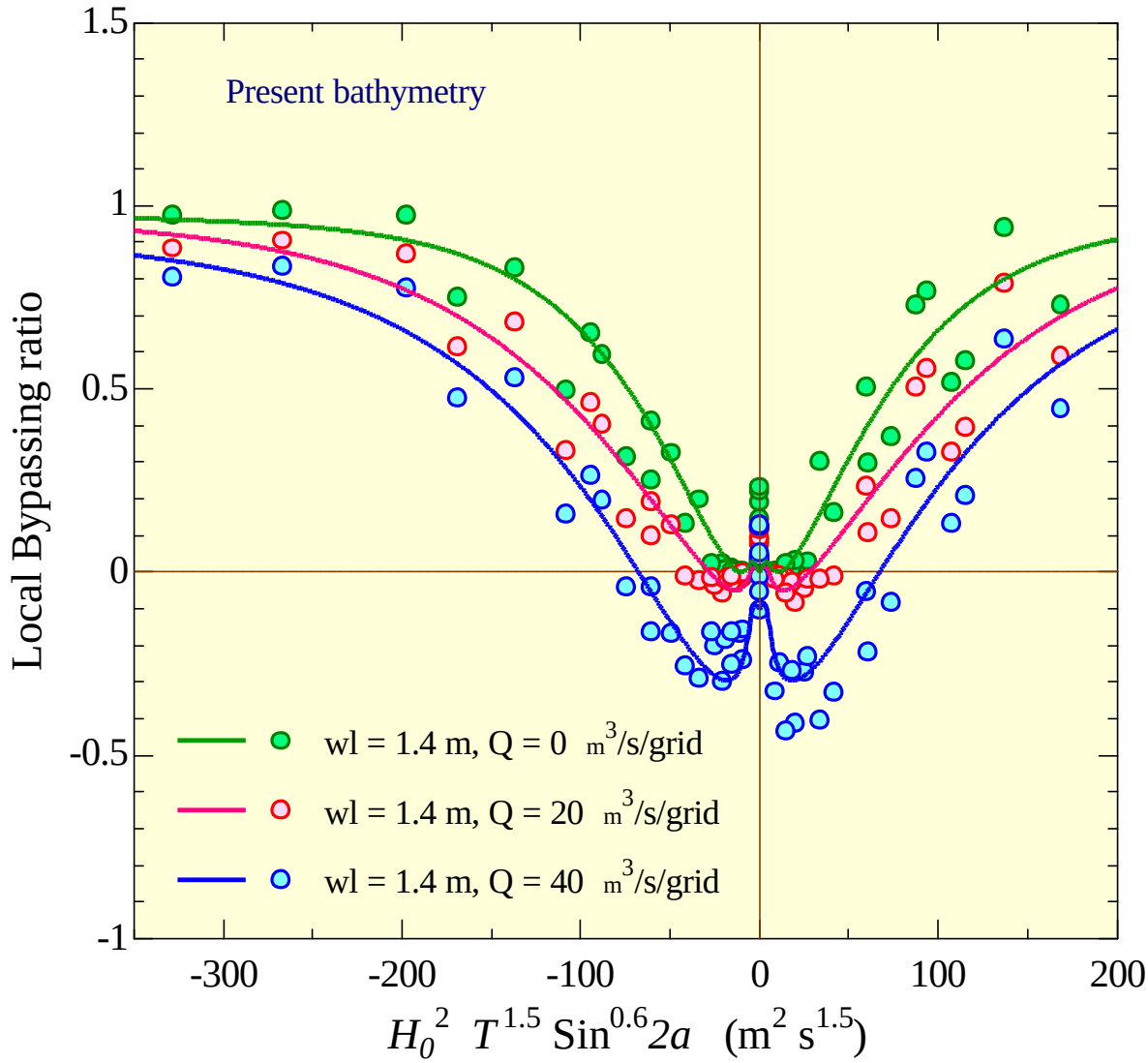


Figure 4-17. Effect of river flow on the local bypassing ratio (water level=1.4 m). The points are from 2D model results and the solid lines are the fitted curves to them. Positive and negative values of the horizontal axis correspond to SW and NW waves, respectively.

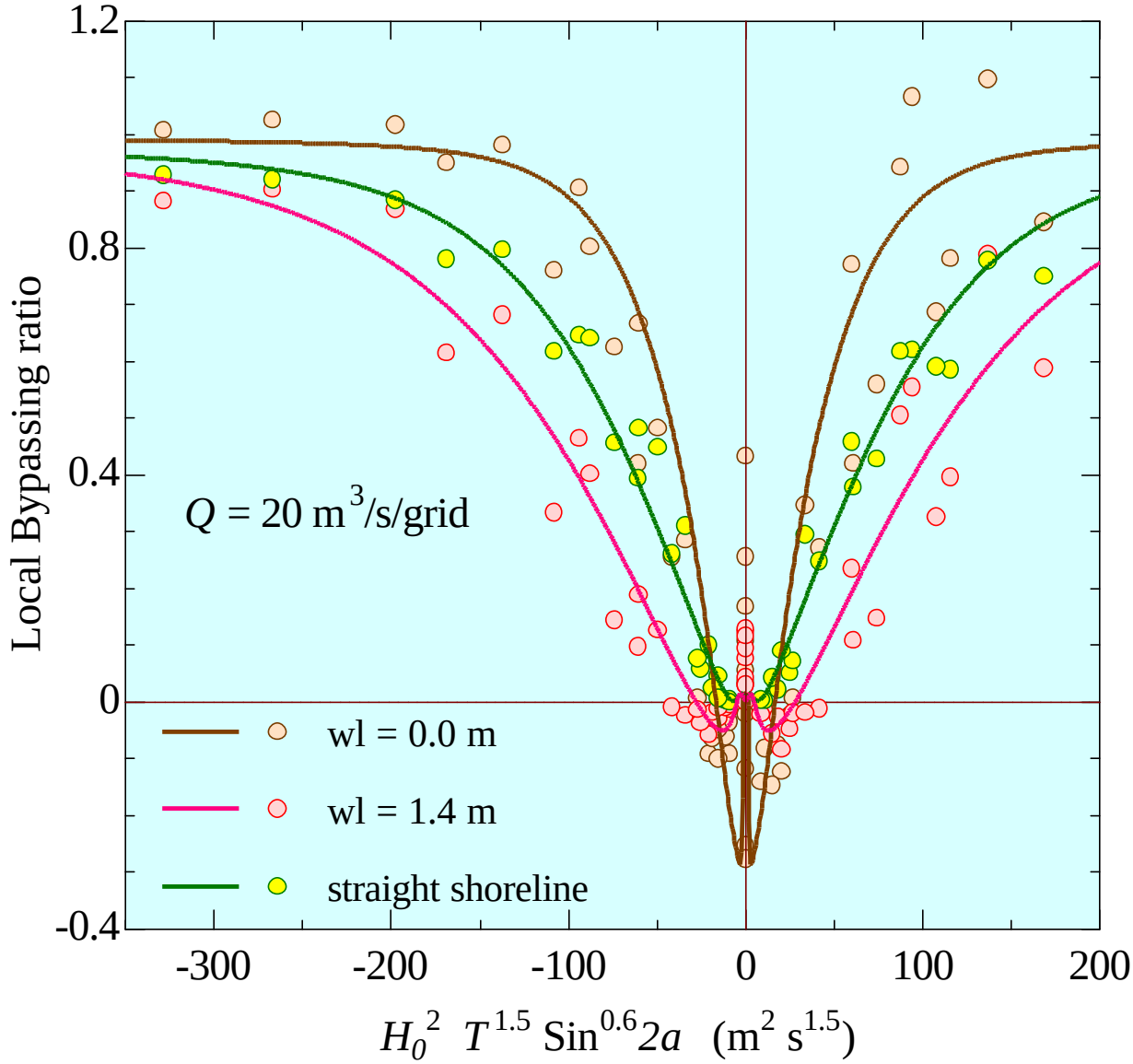


Figure 4-18. Effect of water level and bathymetry on the local bypassing ratio. The points are from 2D model results and the solid lines are the fitted curves to them. Positive and negative values of the horizontal axis correspond to SW and NW waves, respectively.

Table 4-4. 50-year Average Local Bypassing Ratios.

Waves	NW	SW
Straight Beach	46%	42%
Present Bathymetry	57%	55%

Although the bypassing ratio is close to 100% for large waves with the present bathymetry as shown in Section 4.1, the 50-year average local bypassing ratios for NW and SW waves are 57% and 55%, respectively. While the bypassing rate across the channel mouth is not 100%, the defect (difference between the bypass rate and 100%) is made up by erosion from the downdrift fillet for smaller waves. The primary reason for this is the bimodal nature of wave direction as sketched in Figure 4-19. The consequence is that there is always a circulation current next to the downdrift jetty that reshapes the beach. As the wave direction occasionally changes between NW and SW, this circulation appears on both sides of the harbor alternatively and therefore the shoreline can never reach the tip of the jetties. Namely, there will always be a setback distance (i.e. between tip of the jetty to the place where beach intersects the jetty), while full local bypassing requires nearly zero setback. Full local bypassing is attained only in strong unidirectional sediment environments where sediment continuously accumulates in updrift side of the groin/jetty/breakwater and the setback distance is almost zero. An example is shown in Figure 4-20 from a project Baird has undertaken in Ghana, West Africa. The above results will be implemented in the next chapter to determine the overall impact of the harbor structures on its neighboring updrift and downdrift shorelines, in other words, the “harbor” bypassing rate.

It is important to note that in the above analysis it is assumed that an evenly distributed sand veneer is permanently available on the lakebed throughout the calculation domain. The bypassing and erosion ratios could significantly change for non-uniform sand veneer distributions. Data on spatial and temporal distribution of sand thickness are therefore crucial for more accurate assessments. While sufficient information is available to support the assessment of the current conditions, our assessment of historic conditions will require assumptions that are not fully substantiated.

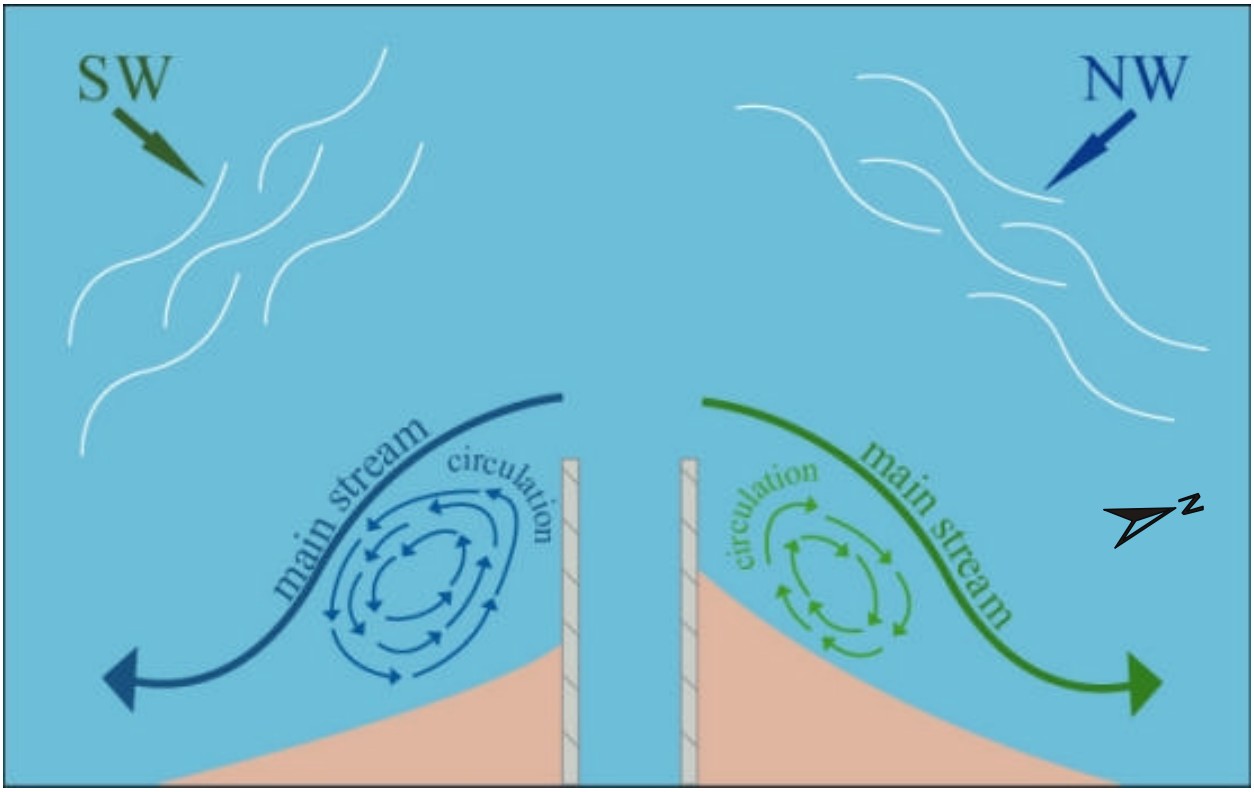


Figure 4-19: Schematics of downdrift circulations due to NW and SW waves.

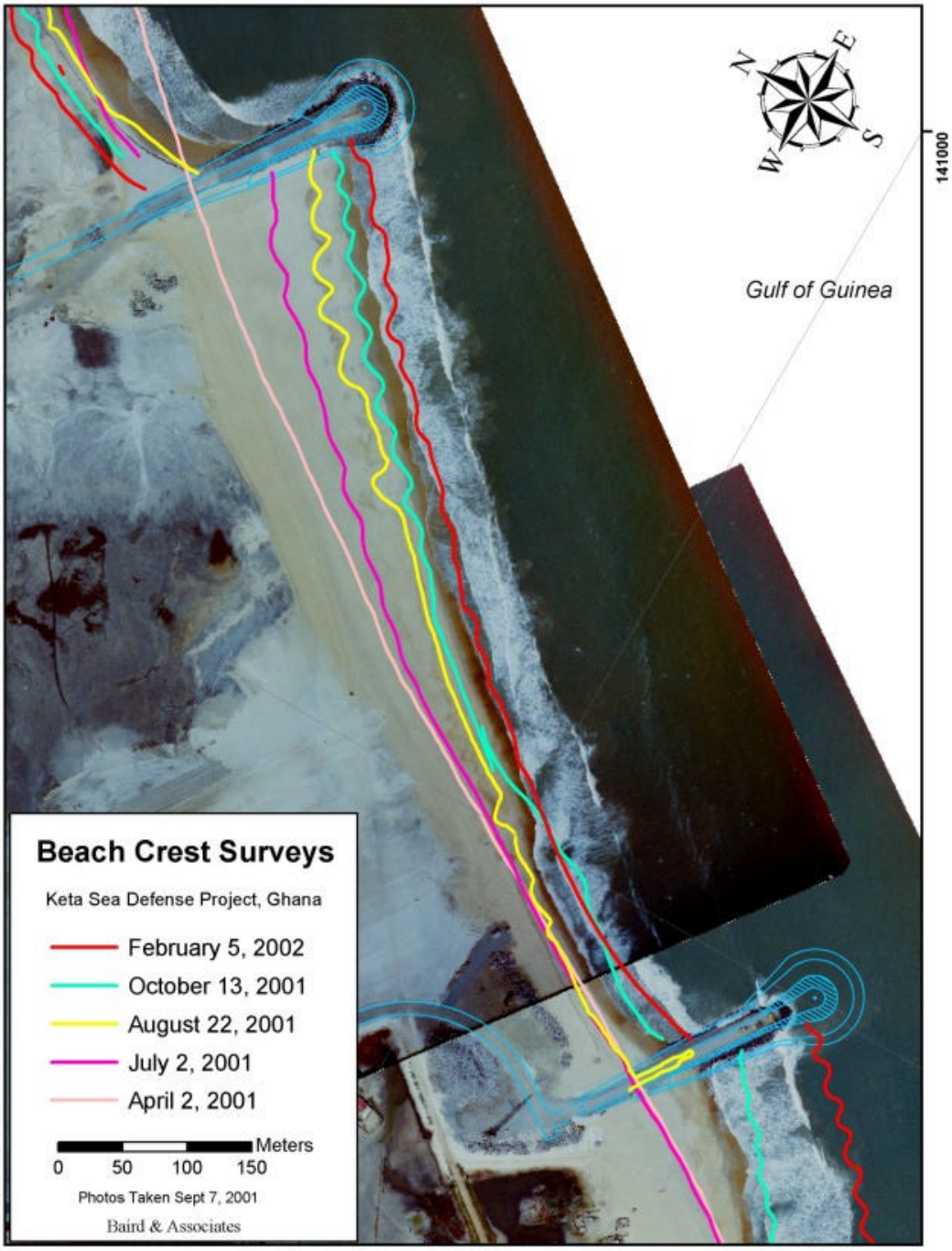


Figure 4-20. Development of a fillet beach in a unidirectional transport environment.

5. SEDIMENT BUDGET

A sediment budget assessment attempts to reconcile all sinks, sources, inputs, and outputs of sediment within a central volume. This approach provides the framework to describe and understand morphology change. This section describes the development of a sediment budget for several historic periods and for the current situation. It is this analysis that ultimately allows for an assessment of the current influence of the harbor on adjacent shoreline erosion processes.

The central volume consists of the harbor and its fillet beaches. The inputs include the northward and southward components of sand transport. Internally, the sink, and source, consist of dredge volume, bathymetry and fillet beach changes. Essentially, if this central volume of the harbor and its fillet beaches is determined to be a sink for sediment moving along the coast, the harbor will have an influence on the erosion processes of the adjacent shoreline.

5.1 LST Rates

The COSMOS model was applied to calculate the LST rates for Reaches 728 to 736 on a yearly basis over the 50-year period of 1949 to 1998. It was assumed that all the profiles are sandy and therefore the results should be considered as potential LST rates for any reach with insufficient sand cover. The wave data was again split into NW and SW wave files to estimate contributions from each direction in addition to the net LST. Figure 5-1 shows variations in net LST rates over the 50-year period at each reach. In terms of magnitude of net transport, it varies from zero to 500,000 m^3/year with southward and northward maximums that occurred in 1992 and 1958, respectively. Figure 5-2 shows the variation in net LST as well as its northward and southward components at Reach 728 over 50 years. The northward annual net transport during the period from1956 to 1961 is the result of fewer waves than usual coming from the north, and not stronger south waves. In other words, more variations are observed in the magnitude of the southward component as compared to the northward LST that varies within a limited range.

Although a net southward transport is predominant, a considerable variation in net LST is observed every few (3 to 5) years. Northward LST was predominant from 1956 to 1961 as well as during the recent years indicating a multi-decadal periodicity as well. All the above features can be accounted for by considering the periodicities of the synoptic scale and meso scale weather systems that affect the Great Lakes in general, and Lake Michigan in particular. The synoptic scale weather systems that affect the Great Lakes are essentially extra-tropical cyclones (ETC's). Weather forecasters usually categorize these into Gulf (of Mexico) lows, Texas lows, Colorado lows, Alberta lows etc. They either move from the south or west towards the Great Lakes. Only on rare occasions, hurricanes or tropical cyclones (TC's) from the Atlantic and the Caribbean Sea traverse the Great Lakes.

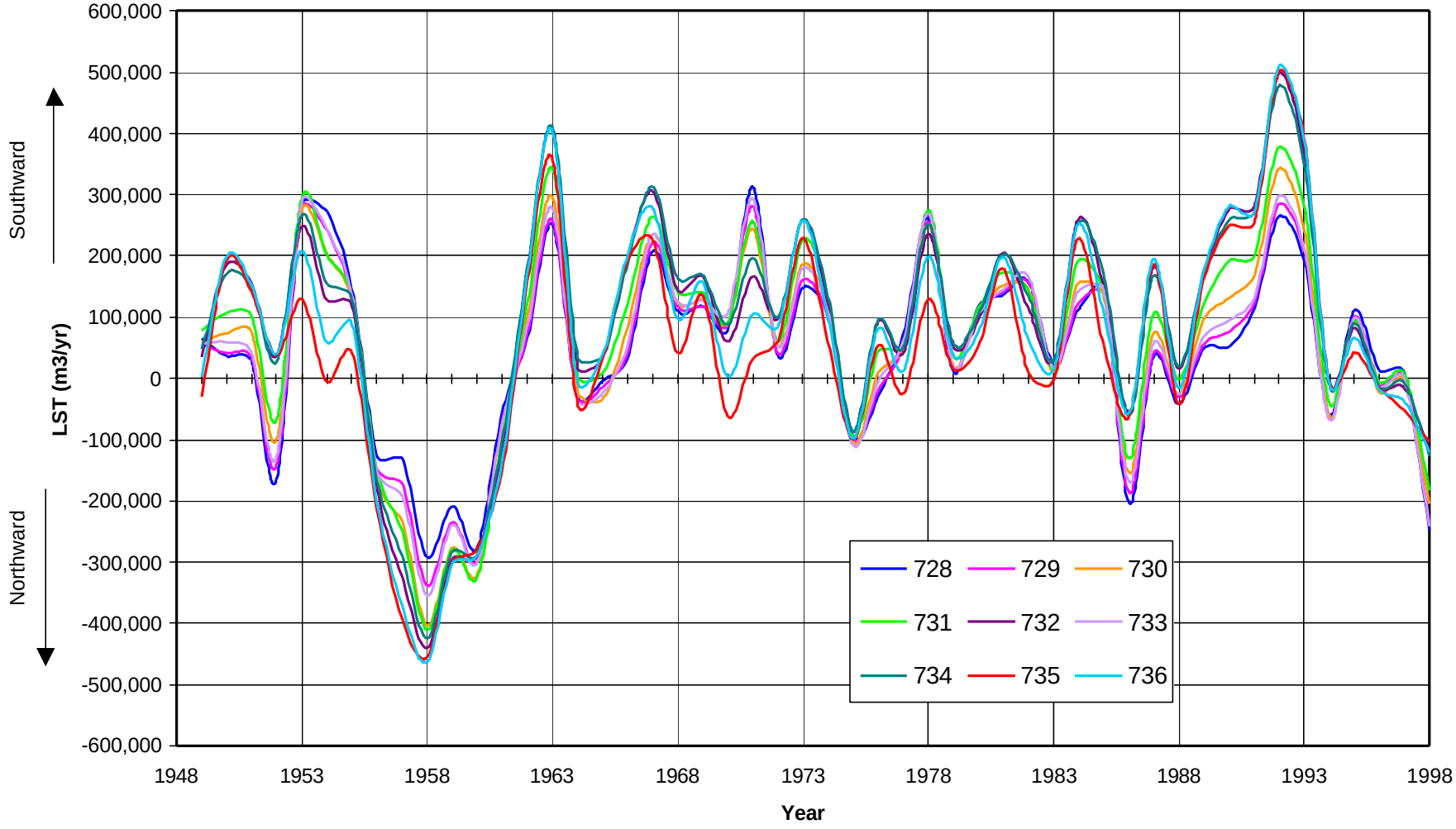


Figure 5-1. Variation in net LST from 1949 to 1998 at Saugatuck for each reach.

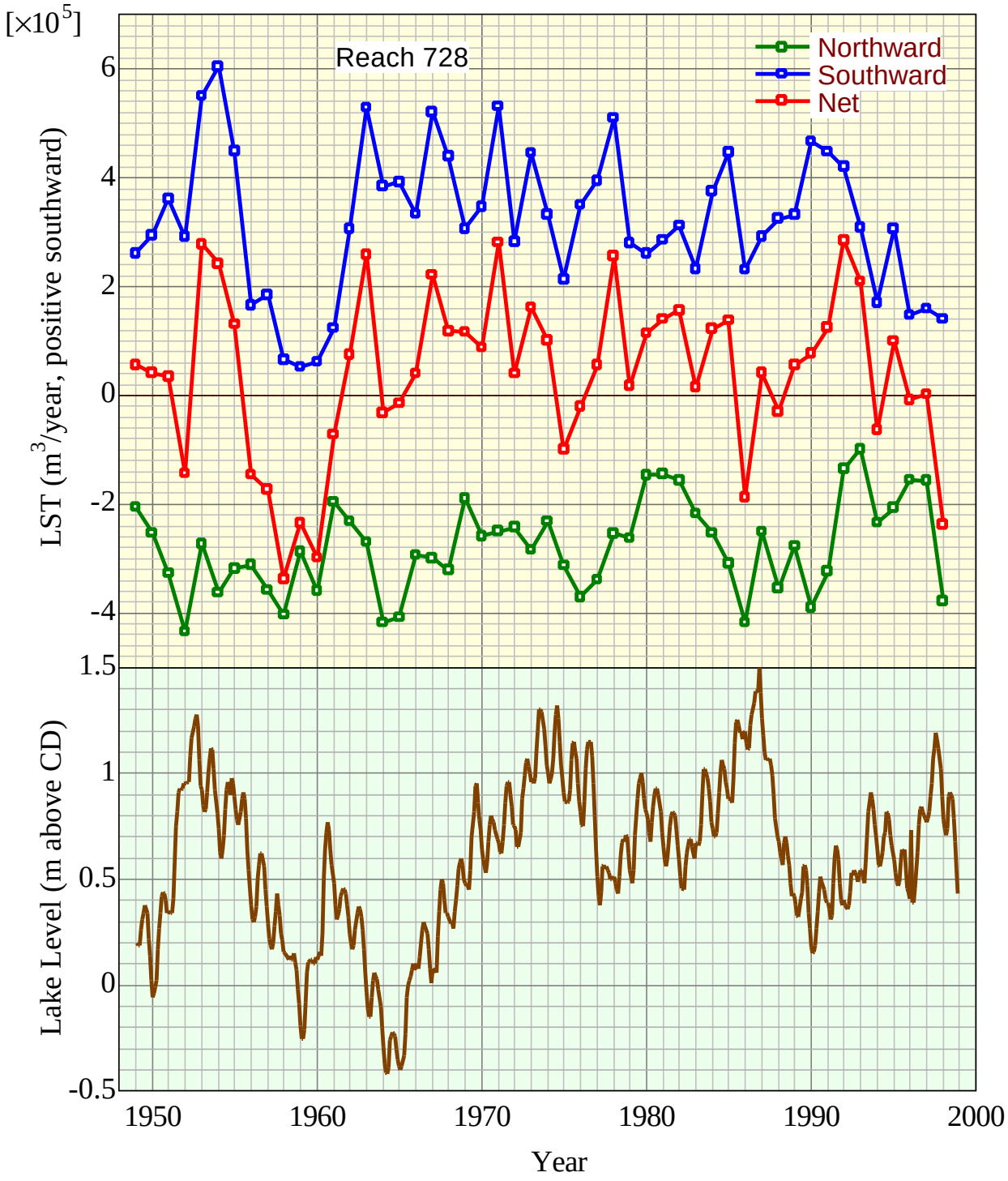


Figure 5-2. Annual variation of net LST and its northward and southward components at Reach 728 and the corresponding historic Lake Levels.

The ETC's that affect the Great Lakes in general, and Lake Michigan in particular, show significant trends in the shift of their tracks. The east-west shift shows an approximate periodicity of about ten years, whereas the north-south shift has a rough periodicity of about thirty years. It may be noted that this 30-year periodicity agrees well with the half period of the North Atlantic Oscillation (NAO) in which the positive phase and negative phase of NAO exhibit an approximate periodicity of about 30 years. The 3 to 5 year periodicity in the computed LST can be attributed to the presence of meso scale phenomena, which are embedded within the synoptic scale weather systems.

In terms of 50-year average values, Figure 5-3 shows NW and SW as well as net LST rates at each reach averaged over the calculation period. It may be seen that average southward LST ranges from 285,000 to 325,000 m^3/year, while average northward transport is between 240,000 and 280,000 m^3/year. The net transport also varies from reach to reach. In developing a sediment budget, however, the interest is in an area that covers the harbor jetties to beyond the end of their corresponding fillet beaches. The averaged net annual LST, therefore, is estimated from the results at reaches 728, 729 and 735 to be in the range of 40,000 to 44,000 m^3/year for the selected 50-year time period. All of the above rates should be considered as potential values subject to availability of enough sand on the beach. On a cohesive shoreline, when sand volumes in the nearshore exceed the thickness of active sediment motion during storm events, the sand cover may be considered as thick enough to provide the potential LST rates and to protect the underlying cohesive substratum from downcutting. Nairn (1992) determined the boundary between sufficient (sandy shores) and insufficient (cohesive shores) sand volumes to be roughly 250 m^3/m. As will be shown in the sediment budget analysis (Section 5.3), it was likely that there was far less sand available than this threshold value immediately after the construction of jetties when the lakebed was not yet completely covered by sufficiently thick layers of sand.

Figure 5-4 shows the cross-shore distribution of total LST after 50 years at Reach 728. The existence of two bars on the profile results in two peaks in the LST curves. This is because over a bar the depth is shallow resulting in larger depth average current and near-bottom orbital velocities. A larger near-bottom orbital velocity results in more intensive stirring of sediment and a larger current velocity can simply transport all that sediment. There is also a third peak near the shoreline in the swash zone followed by a change in transport direction from south to north. The northward transport is mainly in the area above the Chart Datum and indicates that the transport is mostly northward during high water levels. This can be confirmed in Figure 5-2 for high lake level events of 1952, 1960, 1975, 1986 etc.

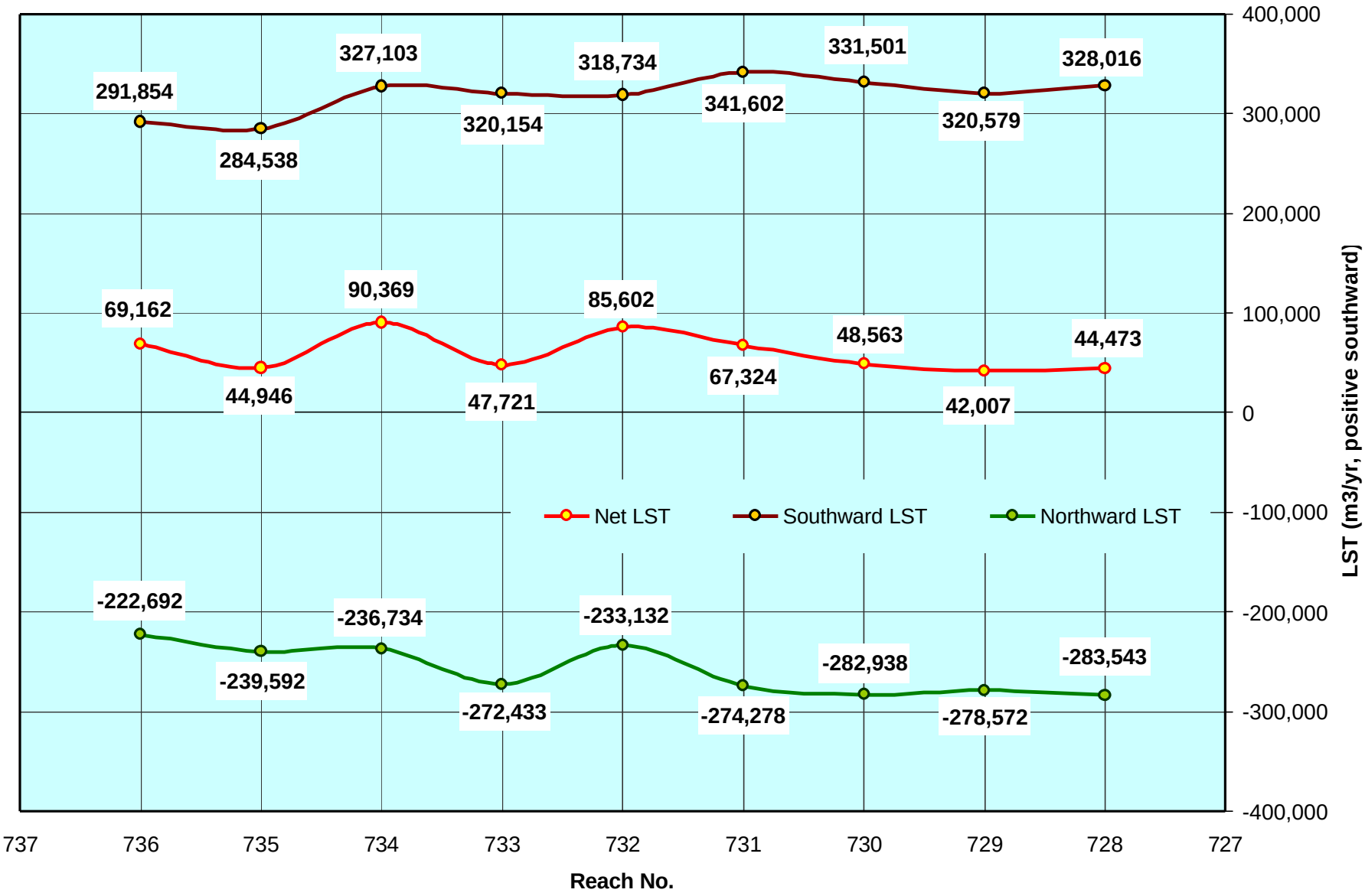


Figure 5-3. Averaged southward, northward and net LST rates at Saugatuck for each reach.

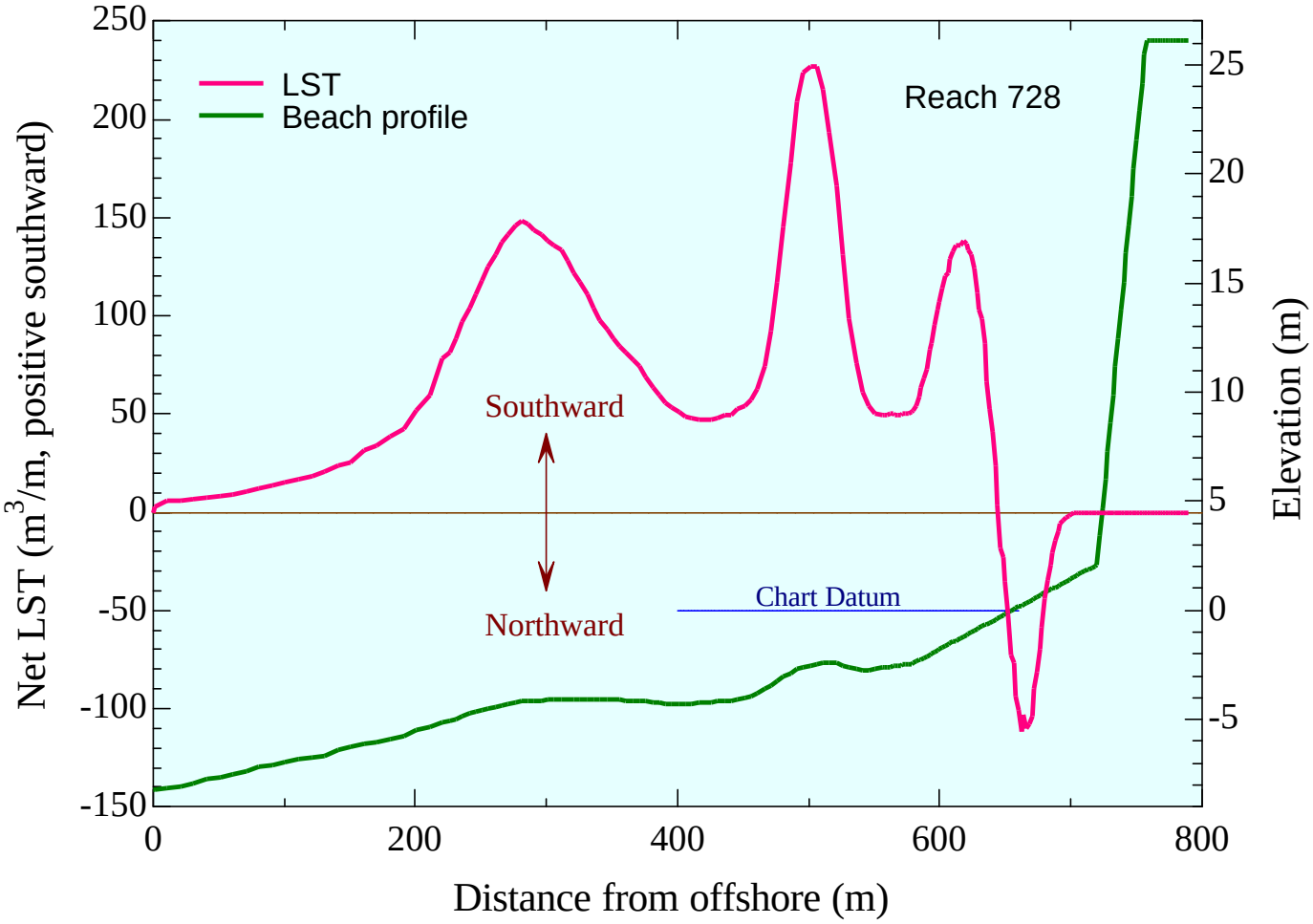


Figure 5-4. An example of cross-shore distribution of LST.

5.2 Dredge Quantities

Dredge quantities are an important element in constructing the sediment budget. Table 5-1 shows the dredging records since the construction of the jetties started in 1904. The dredge quantities represent dredging from just the outer harbor and not the inner channels. According to this table, in average 10,550 m^3/year has been dredged to maintain the navigation channel over 98 years. It is, however, natural to expect that less dredging was required during the initial stages of harbor operation compared to later stages when the fillet beaches were full allowing more sand to be deposited in the harbor entrance.

Table 5-1 suggests that there was no dredging between 1916 and 1959. This may seem questionable particularly when the very low water levels of both 1929 and 1934 are considered (see Figure 5.5), but could also be true for the following reasons. First, this period corresponds to early stages of the harbor's life when transport rates had been significantly less than their potential values (due to lack of enough sand on the lakebed) and the sediment had been mostly trapped by the newly constructed jetties. Another reason is that large quantities of material had been removed prior to this period (in 1909 and 1912), which altogether could have kept the channel open until 1959. In 1957 to 1959 waves coming from north were small and LST was unidirectional towards the north. Also, lake levels went down in 1959 with historic lows recorded between1962 and 1967. Higher dredging volumes are observed in these years, whereas not much dredging was required in periods of high lake levels in the mid 1980's. We may therefore assume an average dredging volume of 6,000 m^3/year from 1904 to 1959. An average of 17,500 m^3 has then been dredged annually between 1962 and 2001. The increase in dredging may be explained by the low lake levels in the early 1960's and also by increased sediment bypassing as a result of the fillet beaches reaching their capacities.

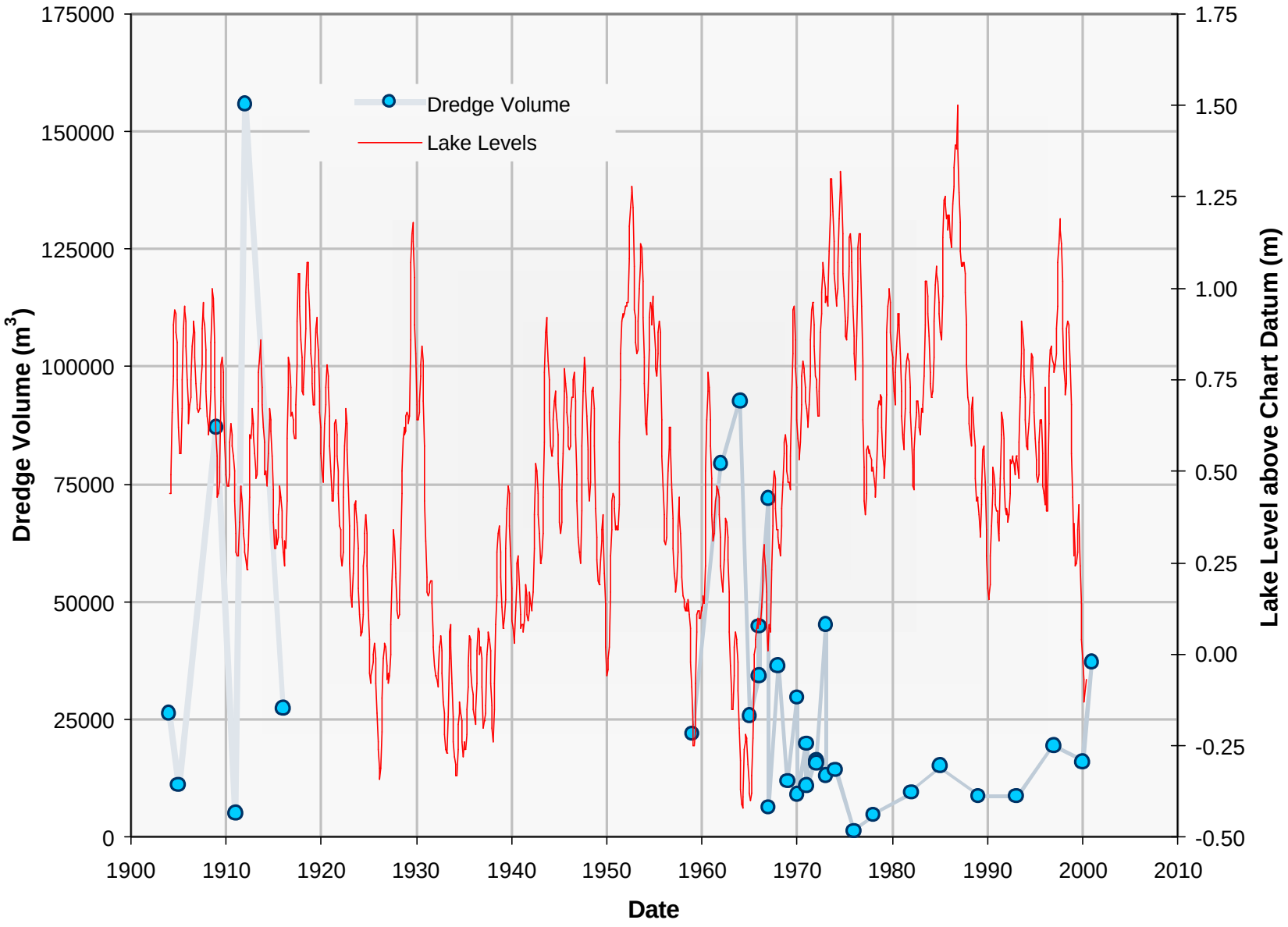


Figure 5.5. Dredge Volumes and Lake Levels.

Table 5.1 SAUGATUCK DREDGING AND PLACEMENT VOLUMES

Fiscal Year	Finish Date	QTY Dredged (CY)	Dredge Area	Placement Area
1904	N/A	34441	N/A	N/A
1905	N/A	14529	N/A	N/A
1909	N/A	113885	N/A	N/A
1911	N/A	6766	N/A	N/A
1912	N/A	203854	N/A	N/A
1916	N/A	35868	N/A	N/A
1959	N/A	28850	N/A	N/A
1962	N/A	103876	N/A	N/A
1964	N/A	121155	N/A	Open Water
1965	5-Jun-65	33834	N/A	Open Water
1966	31-May-66	44873	N/A	Open Water
1966	N/A	58650	N/A	Open Water
1967	3-Jun-67	94254	N/A	Open Water
1967	N/A	8250	N/A	Open Water
1968	1-Jun-68	47750	N/A	Open Water
1969	1-Jun-69	15600	N/A	Open Water
1970	13-Jul-69	38802	N/A	Open Water
1970	19-May-70	11850	N/A	Open Water
1971	8-Jun-71	25998	N/A	Open Water
1971	3-Apr-71	14350	N/A	Open Water
1972	3-Jun-72	21380	N/A	Open Water
1972	14-May-72	20600	N/A	Open Water
1973	25-May-73	59132	N/A	Open Water
1973	9-May-73	17100	N/A	Open Water
1974	N/A	18729	N/A	Open Water
1976	5-Sep-75	1631	N/A	Open Water
1978	N/A	6225	N/A	Open Water
1982	N/A	12436	N/A	BEACH 200'-2700' S OF S JETTY
1985	16-Aug-85	19880	Outer Contour - 40+36	BEACH 200'-2700' S OF S BREAKWATER
1989	16-Feb-90	11427	Critical Shoals 25+00E-7+00W	BEACH 200'-2700' S OF S JETTY
1993	7-Aug-93	11461	Outer Contour - 25+00	BEACH 200' - 2500' S OF S JETTY
1997	17-Jul-97	25612	6+00W - 37+50E	BEACH 200'-2700' S OF S JETTY
2000	23-May-00	21101	Critical Shoals	BEACH
2001	26-Jun-01	48698	Outer Harbor	BEACH 500'-3300'S OF THE S OF JETTY

5.3 Sediment Budget

Based on the results obtained so far, a sediment budget for the Saugatuck Harbor jetties will be developed in this section. The initial and existing conditions will be considered separately because of the differences in bathymetry and sand availability. Before that, however, there is another important piece of information required. It may be seen in Figure 4-5 (Section 4.1) for the straight shoreline bathymetry that the LST rate at the downdrift jetty starts at about the bypassing rate (the average LST at the river mouth) and gradually increases until it reaches its maximum at the downdrift boundary of the calculation domain. It is this gradient in LST that causes the downdrift erosion. An “erosion ratio” is defined as the spatially averaged value of LST from the downdrift jetty to the point that it reaches its maximum (and the gradient becomes almost zero) divided by the average incoming LST rate across the first 500 m from the updrift boundary of the calculation domain. Figure 5-6 shows the relation between calculated erosion ratios and the parameter Θ for the straight shoreline bathymetry. The 50-year average erosion ratios are calculated as before and summarized in the following table.

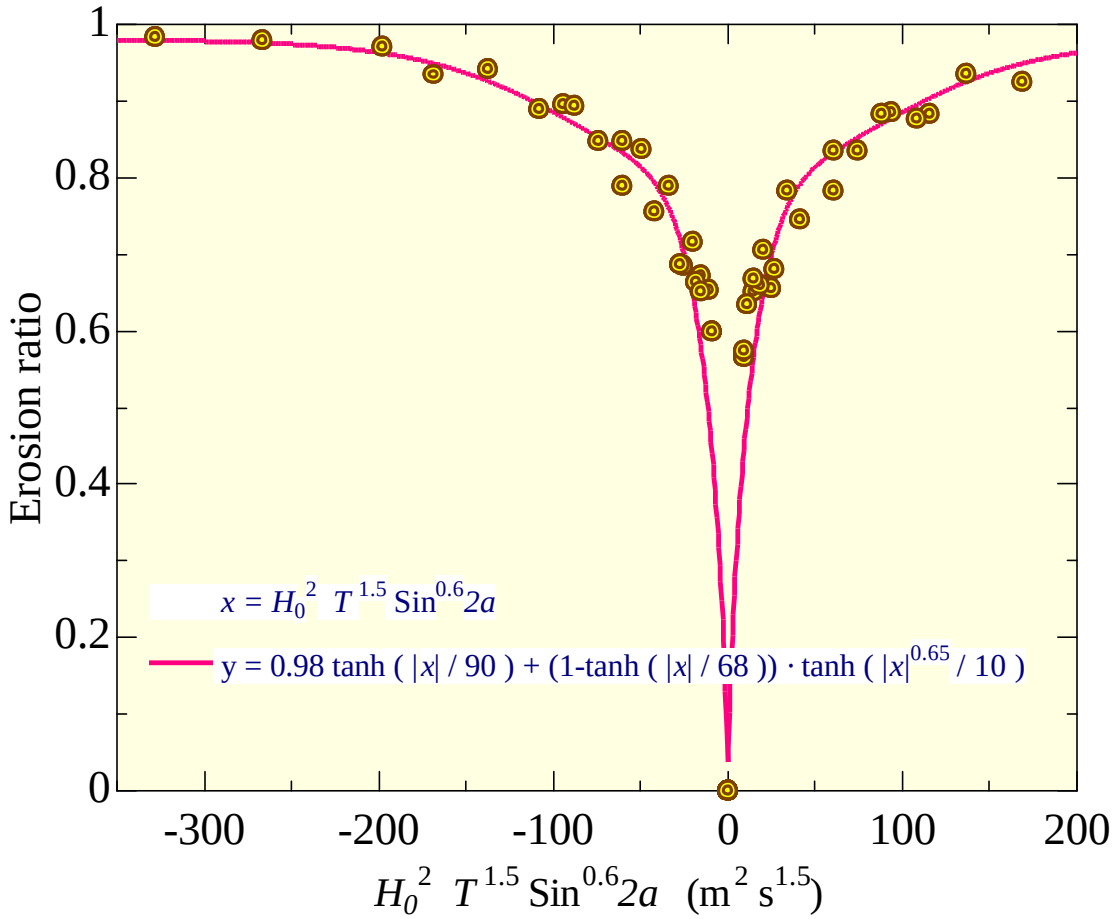


Figure 5-6. Erosion ratio vs the longshore transport parameter. The points are from 2D model results and the solid line is the fitted curve to them. Positive and negative values of the horizontal axis correspond to SW and NW waves, respectively.

Table 5-2. 50-year Average Downdrift Erosion Ratios.

Waves	NW	SW
Straight Beach	80%	80%
Present Bathymetry	94%	100%

The present erosion ratio for SW waves is 100% because the north fillet beach is well developed, allowing complete re-establishment of LST on the downdrift beach. Therefore, an erosion ratio of 100% means that the natural flow of sand to the downdrift shorelines is not disturbed. The south fillet beach, however, is smaller than the north fillet (as explained in Figure 4.19) and therefore the erosion ratio at this fillet is 94% for NW waves.

Figure 5-7 shows the schematics of sediment budget analysis. It was discussed before that waves in Saugatuck are bi-directional, coming from both north and south of the harbor. Superscripts "N" and "S" indicate quantities corresponding to north and south waves, respectively. Other symbols used in this figure are defined in the following.

Q_{total} : LST from a certain direction (i.e. either north or south) (m^3/year)

Q_{bypass} : bypassing volume (m^3/year)

Q_{in} : the portion of Q_{total} : that is trapped (not bypassed) (m^3/year)

Q_{infill} : volume deposited in the fillet every year (m^3/year)

Q_{dredge} : dredge volume (m^3/year)

Q_{erosion} : volume eroded from the fillet by the waves coming from the opposite side of the harbor (m^3/year)

r_{bypass} : local bypassing ratio

r_{erosion} : erosion ratio

ΔV : change in the fillet volume in n years (m^3)

n : number of years considered for the sediment budget analysis

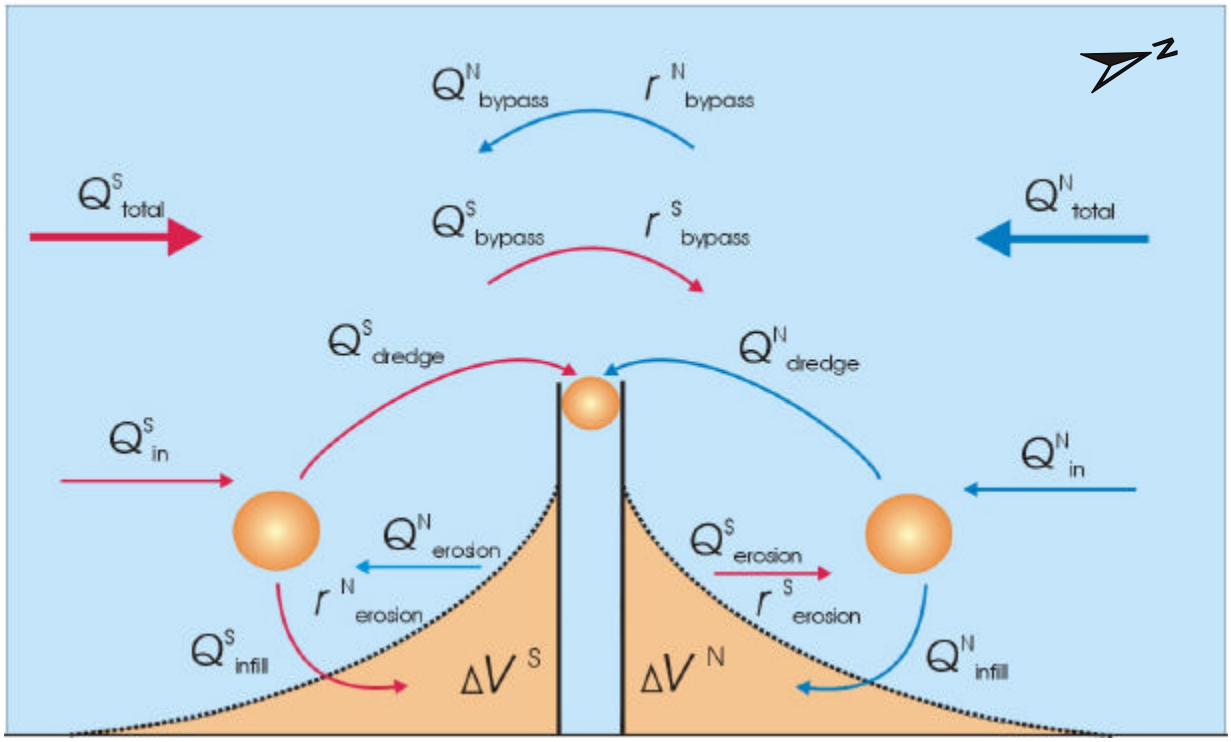


Figure 5-7. Schematics of sediment budget at Saugatuck.

The following relations hold among the above parameters:

$$Q_{\text{in}} = Q_{\text{total}} - Q_{\text{bypass}} = Q_{\text{infill}} + Q_{\text{dredge}} \tag{1}$$

$$Q_{\text{bypass}} = Q_{\text{total}} \times r_{\text{bypass}} \tag{2}$$

$$Q_{\text{erosion}} = Q_{\text{total}} \times r_{\text{erosion}} - Q_{\text{bypass}} \tag{3}$$

Conservation of sediment volume leads to the following system of equations:

$$\begin{cases} Q_{\text{in}}^{\text{S}} - Q_{\text{erosion}}^{\text{N}} - Q_{\text{dredge}}^{\text{S}} = \Delta V^{\text{S}} / n \\ Q_{\text{in}}^{\text{N}} - Q_{\text{erosion}}^{\text{S}} - Q_{\text{dredge}}^{\text{N}} = \Delta V^{\text{N}} / n \end{cases} \tag{4}$$

Which upon substitution from (1) to (3) gives:

$$\begin{cases} (1 - r_{\text{bypass}}^{S})Q_{\text{total}}^{\text{S}} - (r_{\text{erosion}}^{\text{N}} - r_{\text{bypass}}^{\text{N}})Q_{\text{total}}^{\text{N}} = \Delta V^{\text{S}} / n + Q_{\text{dredge}}^{\text{S}} \\ (1 - r_{\text{bypass}}^{\text{N}})Q_{\text{total}}^{\text{N}} - (r_{\text{erosion}}^{\text{S}} - r_{\text{bypass}}^{\text{S}})Q_{\text{total}}^{\text{S}} = \Delta V^{\text{N}} / n + Q_{\text{dredge}}^{\text{N}} \end{cases} \tag{5}$$

Where the bypassing and erosion ratios can be estimated from the results of 2D numerical modeling. Sediment from river is discarded in the above analysis, but can be simply included by considering corresponding dredge and fillet volume contributions. The above equations may be solved for Q_{total} if the fillet volume and dredging information is available. This is the case at Saugatuck and, therefore, Equation (5) can provide estimates of initial LST rates, which must have been far less than their potential values, calculated in Section 4.1. The available dredging data, however, represents total dredge volumes and it is necessary to assume a relationship between the dredging related to the north and south directed components of transport to solve the equations. We may reasonably assume that they are proportional to their corresponding fillet volumes:

$$Q_{\text{dredge}}^{\text{S}} / Q_{\text{dredge}}^{\text{N}} = \Delta V^{\text{S}} / \Delta V^{\text{N}} \tag{6}$$

On the other hand, if information on Q_{total} is provided, Equation (5) can be solved to predict the sum of infilling and dredging volumes in the future. Further assumptions will then be required to decompose the results into dredging and infilling components. In case that there is a sediment discharge from the river, assumptions will again be required to determine its contributions to dredging and infilling volumes.

First, the 1904 to 1938 period, which is from the start of construction of jetties to the date of first available aerials (n=34), is considered. Figure 5-8 has been made by tracing the waterline from available historic air photos and shows the evolution of the fillet beaches

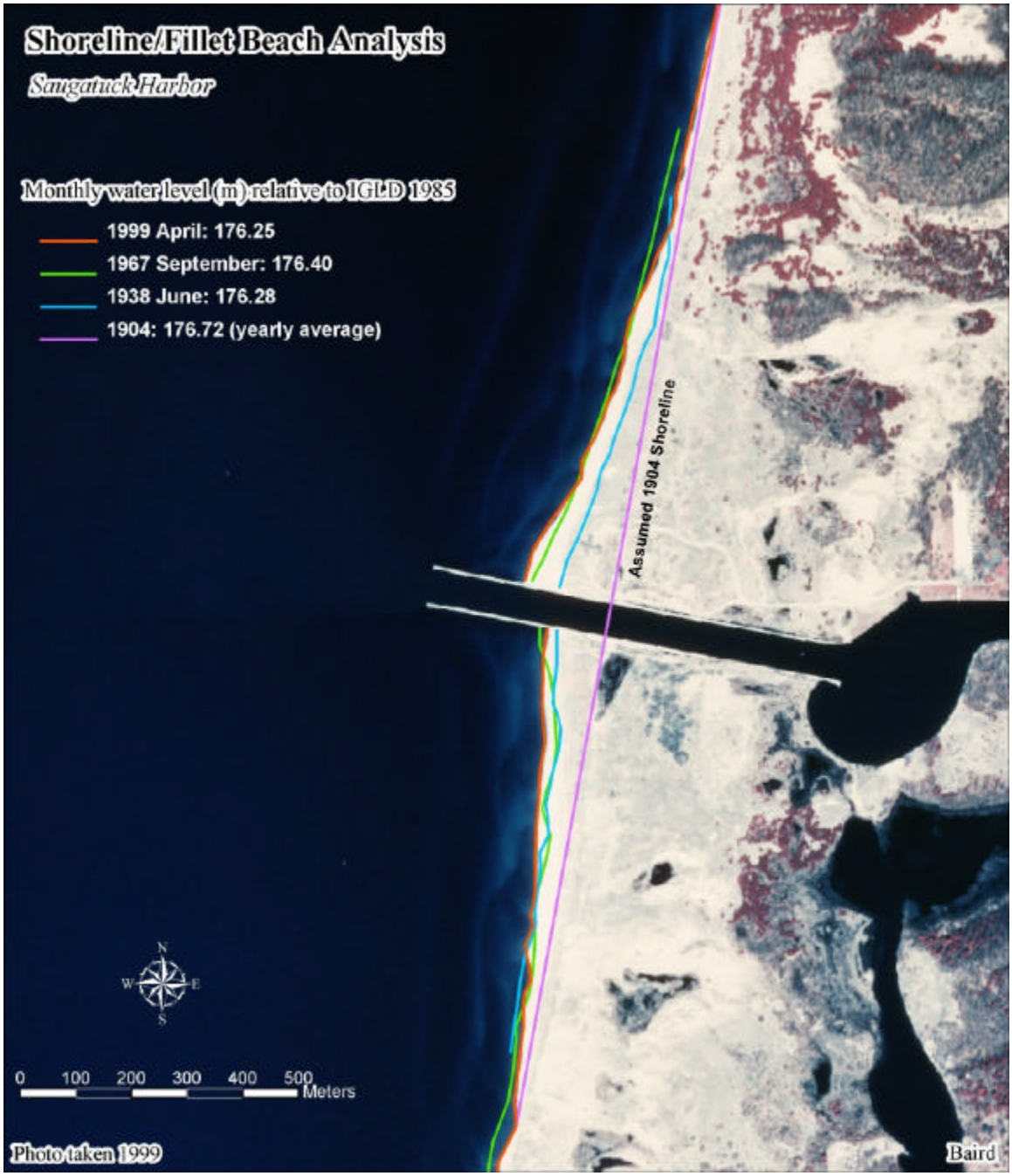


Figure 5-8. Evolution of the fillet beaches at Saugatuck.

at Saugatuck over the past century. Assuming a straight pre-construction shoreline as shown in the figure, the shoreline from the aerials taken in 1938 lays halfway to the existing shoreline. Assuming that the fillet volume is linearly proportional to shoreline position (this is the key assumption in one line models), leads to the conclusion that about half of the present beach fillets had been developed by 1938. We may roughly assume that this corresponds to 685,000 m^3 ($= \Delta V^N$) in the north fillet and 485,000 m^3 ($= \Delta V^S$) in the south fillet. It should be mentioned that the sandy fillet might have a different slope from the original cohesive profile resulting in a nonlinear relation between fillet volumes and shoreline position (i.e. greater incremental fillet volumes as the shoreline advances into deeper waters). Unfortunately, historic profile surveys are not available to establish an accurate relationship.

The average dredge volume (deposition in the channel) of 6,000 m^3/year was split into north and south components according to Equation (6). The bypassing and erosion ratios were obtained by interpolating between their initial and existing values. Equations (5) were then solved for Q_{total} and the results are shown in Figure 5-9. The average incoming LST from the north for the first 34-year period was found to be 134,000 m^3/year. The incoming LST from the south was 115,000 m^3/year. These values are about one third to one half of the potential values predicted by COSMOS in Section 5-1. It should be mentioned once again that these results are based on the assumption of uniform sand veneer over the entire area of interest. In some cases, for example, only a narrow band of thick sand may be available in the vicinity of the shoreline. Figure 5-10 shows the percentage of LST that occurs from the shoreline out to a certain depth as a function of that depth at Reach 728. It may be seen that one third of the total LST occurs between the shoreline and 2.5 to 3 m water depth. The above transport rates, therefore, roughly indicate the existence of sand cover out to 2.5 m below CD, a condition that is compatible with an historically sand-poor system. An iteration procedure is then required to subsequently re-evaluate the bypassing and erosion ratios (i.e. repeat the 2D calculations with the new sand thickness information) and solve Equations (5) until the solution converges. The above transport rates, therefore, indicate the upper limits of the solution. The bottom limits, corresponding to a narrow band of sand available, are the 16,500 and 23,500 m^3/year values.

In summary, during the 1904 to 1938 period, of the incoming 134,000 m^3/year southward LST, 64,500 m^3/year was bypassed to the downdrift side. The remaining 69,500 m^3/year was stopped by the jetties, of which 20,000 m^3/year was deposited into the north fillet, 3,500 m^3/year was dredged and 46,000 m^3/year was washed back to the updrift shores by the incoming south waves. Similarly, of the incoming 115,000 m^3/year northward LST, 52,000 m^3/year was bypassed to the northern beaches. The remaining 63,000 m^3/year was stopped by the jetties, of which 14,000 m^3/year went to the south fillet, 2,500 m^3/year was dredged and 46,500 m^3/year was washed back to the southern shores by the incoming waves from the north. As a result both the shorelines north and south of the harbor were deprived of 16,500 and 23,500 m^3/year of sand, respectively, during this period because of the harbor.

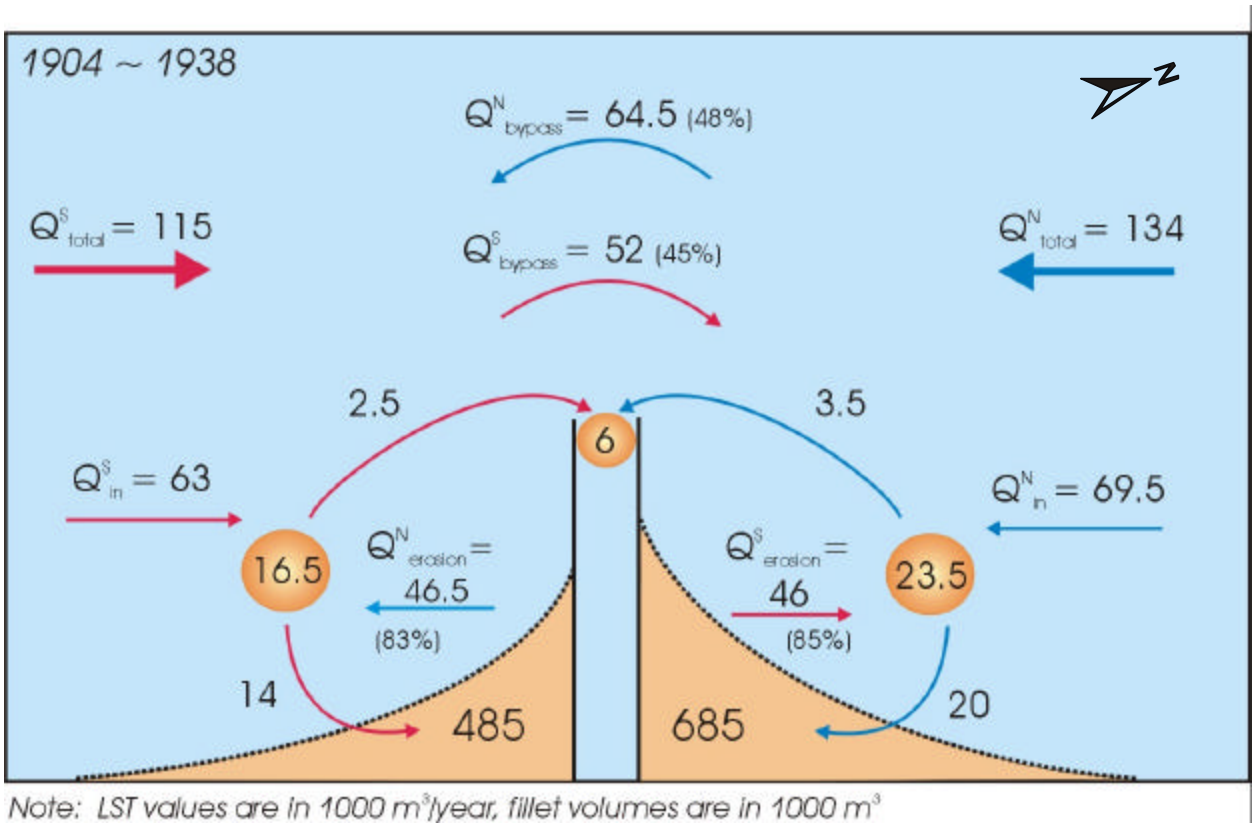


Figure 5-9. Sediment budget at Saugatuck for 1904 ~ 1938.

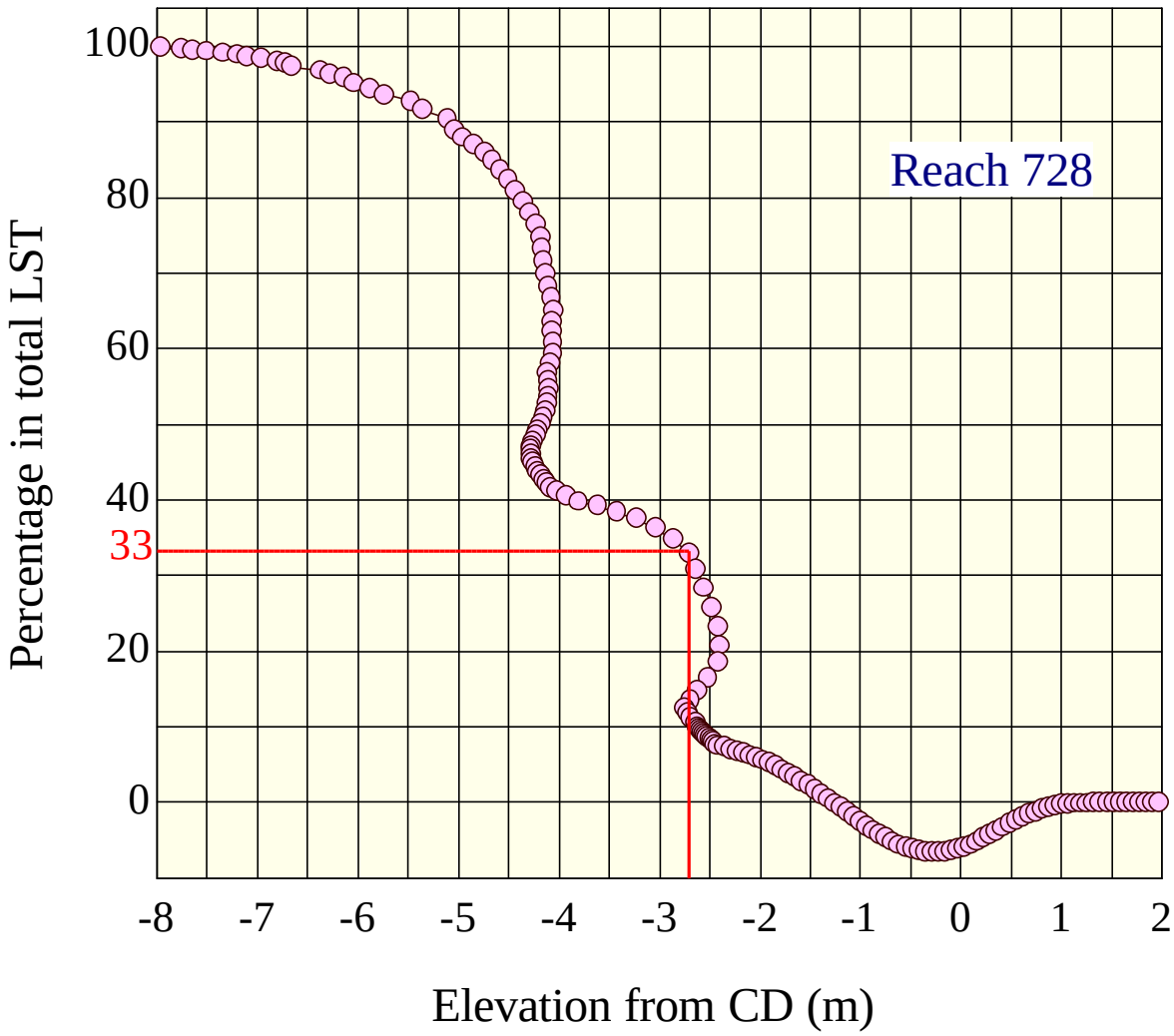


Figure 5-10. Percentage of total LST as a function of water depth.

It is a common practice among coastal engineers to discuss the bypassing and infilling rates with respect to the net LST rate. The updrift and downdrift shores are evaluated with respect to the direction of net transport, and it is always the downdrift shoreline that is considered as being suffered from erosion. This type of argument, however, is mostly appropriate for unidirectional transport environments and not for the bi-directional case of this study where the structures work as a sink of sediment for both northward and southward LST components. For example, the net transport in the present case is only 19,000 m^3/year, which is far less than the total infilling and dredge volumes (16,500+23,500=40,000 m^3/year) and therefore is nothing more than an imaginary number. Under a bi-directional transport environment a net transport can only be defined on a natural beach without structures. At Saugatuck it may be defined for beaches far enough from the harbor. During the above period, the net transport on the shorelines north of the harbor would be the sum of 134,000 m^3/year southward and 52,000 m^3/year (bypassed) and 46,000 m^3/year (erosion) northward transports. This results in a net southward LST rate of 36,000 m^3/year. Similarly, on the southern shorelines the net transport is 4,000 m^3/year northward. The north and south net transports are directed towards the harbor and their summation amounts to 40,000 m^3/year.

Figure 5-11 summarizes the sediment budget for the period of 1939 to 1947 (9 years). An historic shoreline map was available for 1947. It was estimated that the fillets on average had reached 63% of their exiting capacity by 1947 (Baird & Associates, 2001). The resulting transport rates are about 27% larger than those obtained for period of 1904 to 1938, which is an indication of existence of more sand on the lakebed in the area around the harbor. Again, both the north and south shorelines were deprived of 16,500 and 23,500 m^3/year of sand, respectively, during this period because of the harbor. Another shoreline map was available for 1999. Figure 5-12 shows the sediment budget for 1948 to 1999 (52 years). Average dredge volume for this period was 14,500 m^3/year and the fillets had reached their capacities sometime towards the end of this period. The calculated average transport rates are about 60% higher than those in Figure 5-10. It is likely that the transport rates have reached their potential values by the end of this period. Shorelines to the north and south were deprived of 13,000 and 18,000 m^3/year of sand, respectively on average through this period.

The present and future conditions are considered next in Figure 5-13. It may be assumed that there is sufficient sand on the nearby beaches such that the LST rates are close to their potential values. The local bypassing ratios are 57% and 55% and the erosion ratios are 94% and 100% for the southward and northward transports, respectively. Of the 300,000 m^3/year incoming from north, 171,000 m^3/year gets bypassed and 129,000 m^3/year is stopped by the jetties and the north fillet. But because of an annual erosion of 117,000 m^3 from the north fillet beach due to SW waves, it results in an infilling rate of only 12,000 m^3/year. Similarly, the present infilling rate of the channel from the south fillet is about 6,000 m^3/year. These together are almost equal to the current annual dredging volume of 17,500 m^3 and therefore there will be no more fillet development.

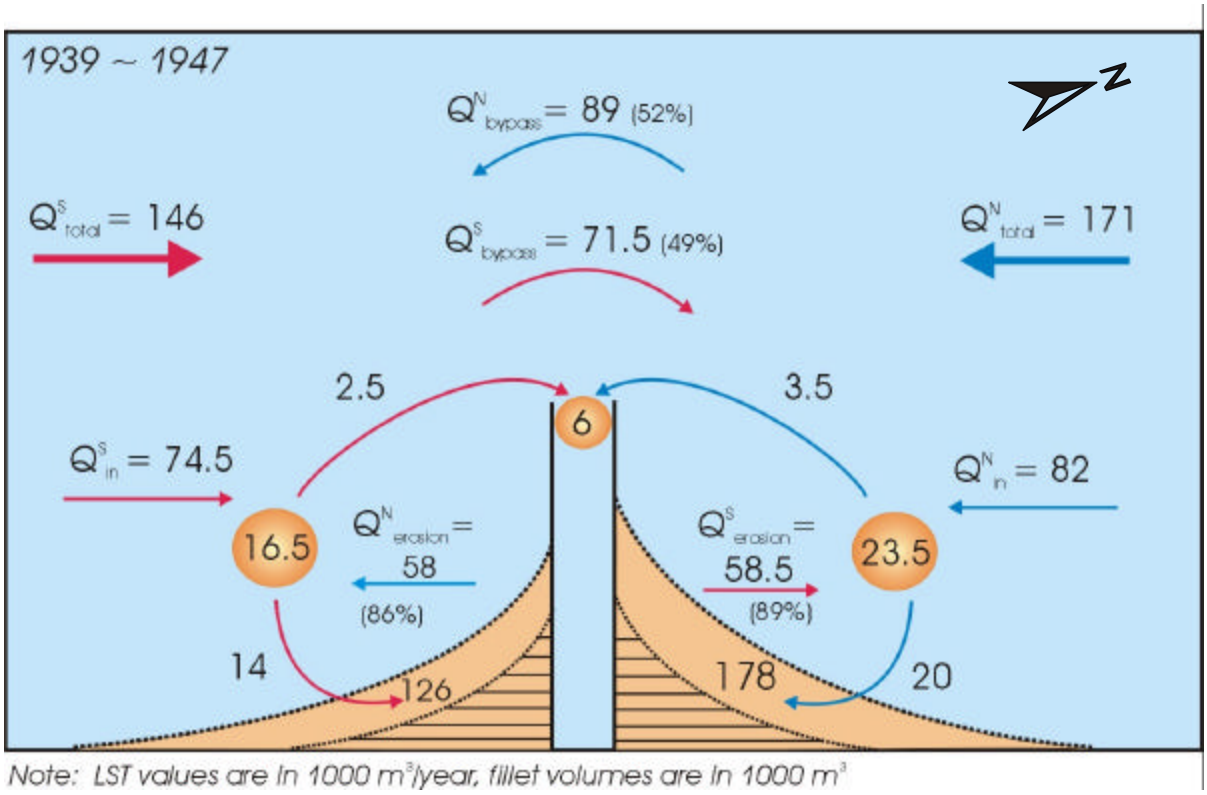


Figure 5-11. Sediment budget at Saugatuck for 1939 ~ 1947.

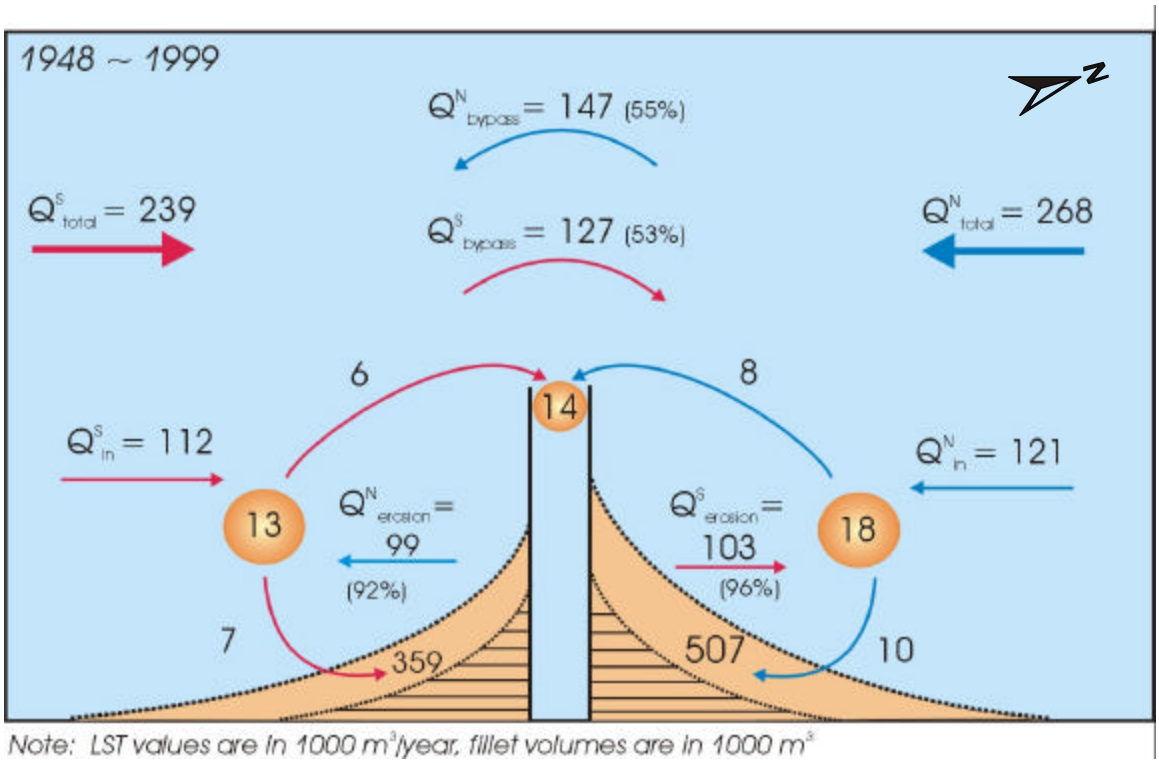


Figure 5-12. Sediment budget at Saugatuck for 1948 ~ 1999.

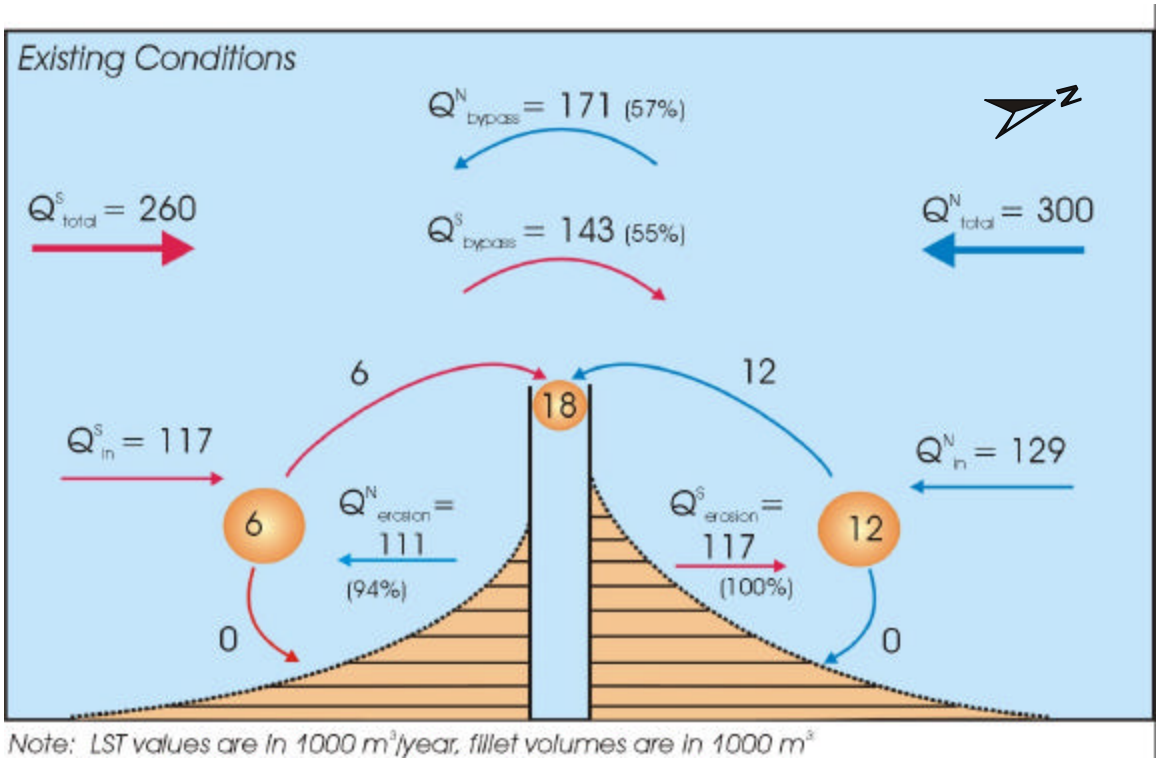


Figure 5-13. Sediment budget at Saugatuck for existing conditions.

The net LST is 40,000 m^3/year towards the south on the north shorelines indicating that the northward transport is completely re-established beyond the harbor and therefore, the shorelines to the north are no longer deprived of sand coming from the south. On the south shorelines, the net LST is 22,000 m^3/year and directed southward. The bypassing rate in terms of the net transport is thus 55%. The difference is the 18,000 m^3/year that should be dredged and put on the south shorelines (well beyond the limit of the south fillet beach). If this is undertaken there will be no impact on the shorelines to the south. It may be concluded that the fillets have reached their capacities and they are in a dynamic equilibrium. Therefore, under the present configurations and as long as the wave climate remains similar to the past 100 years, the harbor will not trap any more sand from the system if the dredged sediment is located properly (i.e. 18,000 m^3/year on average placed well to the south of the south fillet).

It is important to note that this conclusion is based on analysis of long-term average values and therefore, may not apply to individual years when the wave climate may significantly deviate from its average behavior. In the event that the harbor is dredged ever year, relocation of the dredge material should be guided by ongoing estimates of longshore sand transport rates (based on ongoing updates of the wave climate) for the period since the previous dredging contract. In this way, future possible changes in the wave environment can also be accommodated in a best management approach to handling dredged sediment to minimize or eliminate possible erosion impacts.

6. CONCLUSIONS

Through the application of state-of-the-art numerical modeling and the development of innovative analysis techniques, the present study was successful in determining the role of the harbor structures on local shoreline change and regional sediment budget in both past and present. The important conclusions of the present study are summarized below:

- A 2D hydrodynamic and sediment transport numerical model was developed to investigate the details of waves, current and sediment transport around the federal structures at Saugatuck.
- An innovative approach was employed to apply the results of 2D modeling to long-term analysis of sediment transport around the harbor jetties.
- The COSMOS model was applied to estimate the longshore sediment transport rates over the study area for the period of 1949 to 1998 and the observed temporal trends in LST were discussed.
- It was shown, quantitatively, how sand has accumulated in form of fillet beaches due to construction of the harbor jetties.
- Assuming that future environmental conditions remain similar to those in the last hundred years, it was found that the fillet beaches on both sides of the federal structures at Saugatuck have reached a state of dynamic equilibrium that will not accommodate more sand. The only sink of sediment in the future will be the navigation channel. On a long-term basis, it is possible to restore the full natural flow of sand past the harbor by locating the amount of material trapped in the navigation channel onto the southern beaches beyond the south limit of the south fillet beach (i.e. 1.5 to 2 km to the south). It is, however, recommended that the sediment placement (how much and on which side of the harbor) should be defined by monitoring the sand transport on a yearly basis. This is possible by linking the COSMOS model into a real-time wave prediction system (for example the WAVAD model) for Lake Michigan. Incorporating the 2D model results, the system can provide information on how much sand to place and where to mitigate impacts at any harbor on Lake Michigan.
- Whether or not the trapping of sand that occurred in the past influenced the rate of erosion of the adjacent shorelines is difficult if not impossible to assess without knowledge of the thickness of sand cover over the underlying cohesive sediment in the past.
- At present, since the harbor is no longer trapping sand (provided the dredged sediment is properly managed), there will be little or no influence of the harbor on the erosion of adjacent shorelines.
- The 50-year future bluff lines for the downdrift cohesive shores calculated during the LMPDS study should therefore be considered as valid, until (and if) sufficient information on historic sand thickness on those beaches becomes available.

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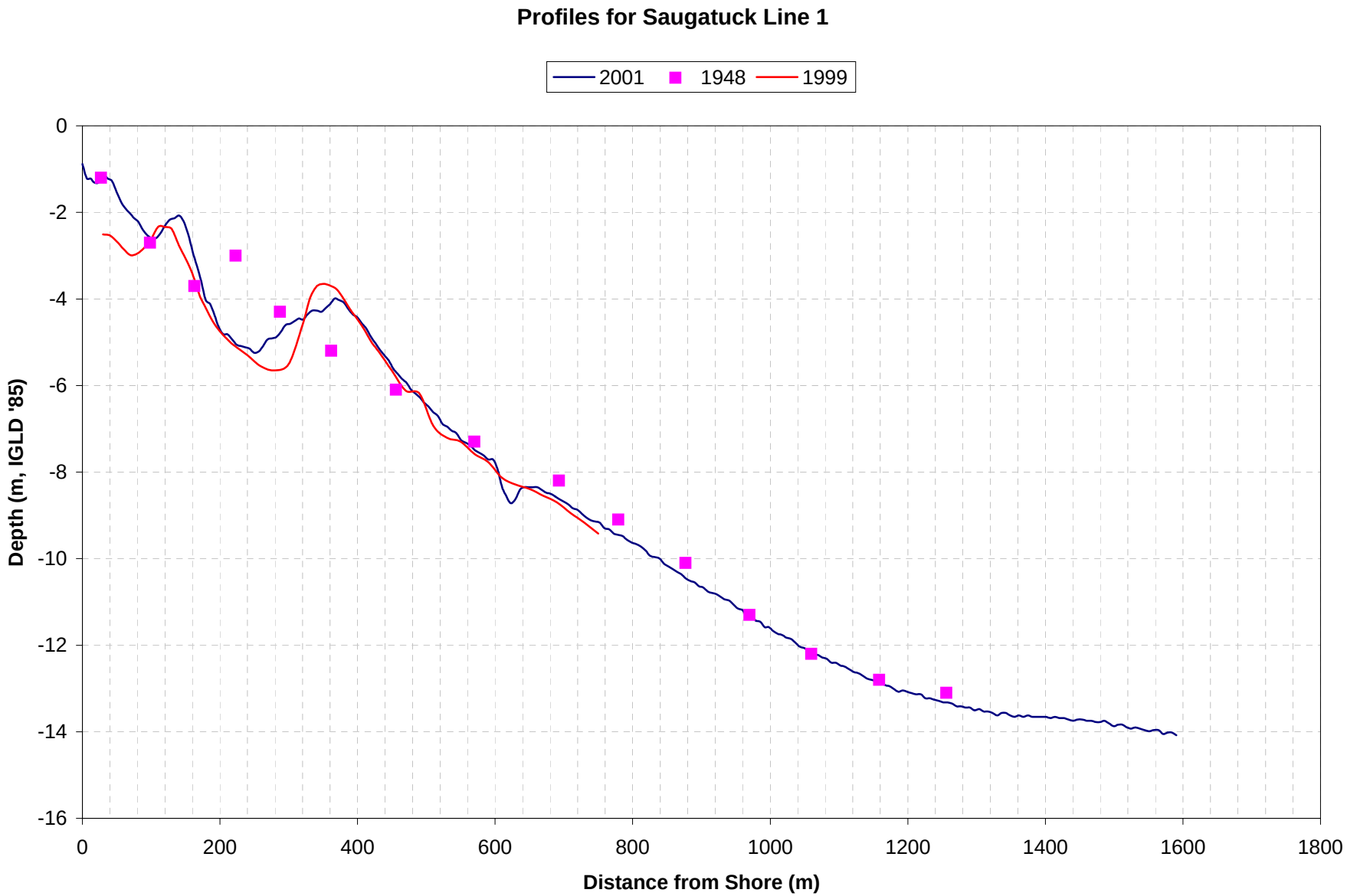
APPENDIX A

BEACH PROFILES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1948
1999
0
-2
-4
-6
-8
-10
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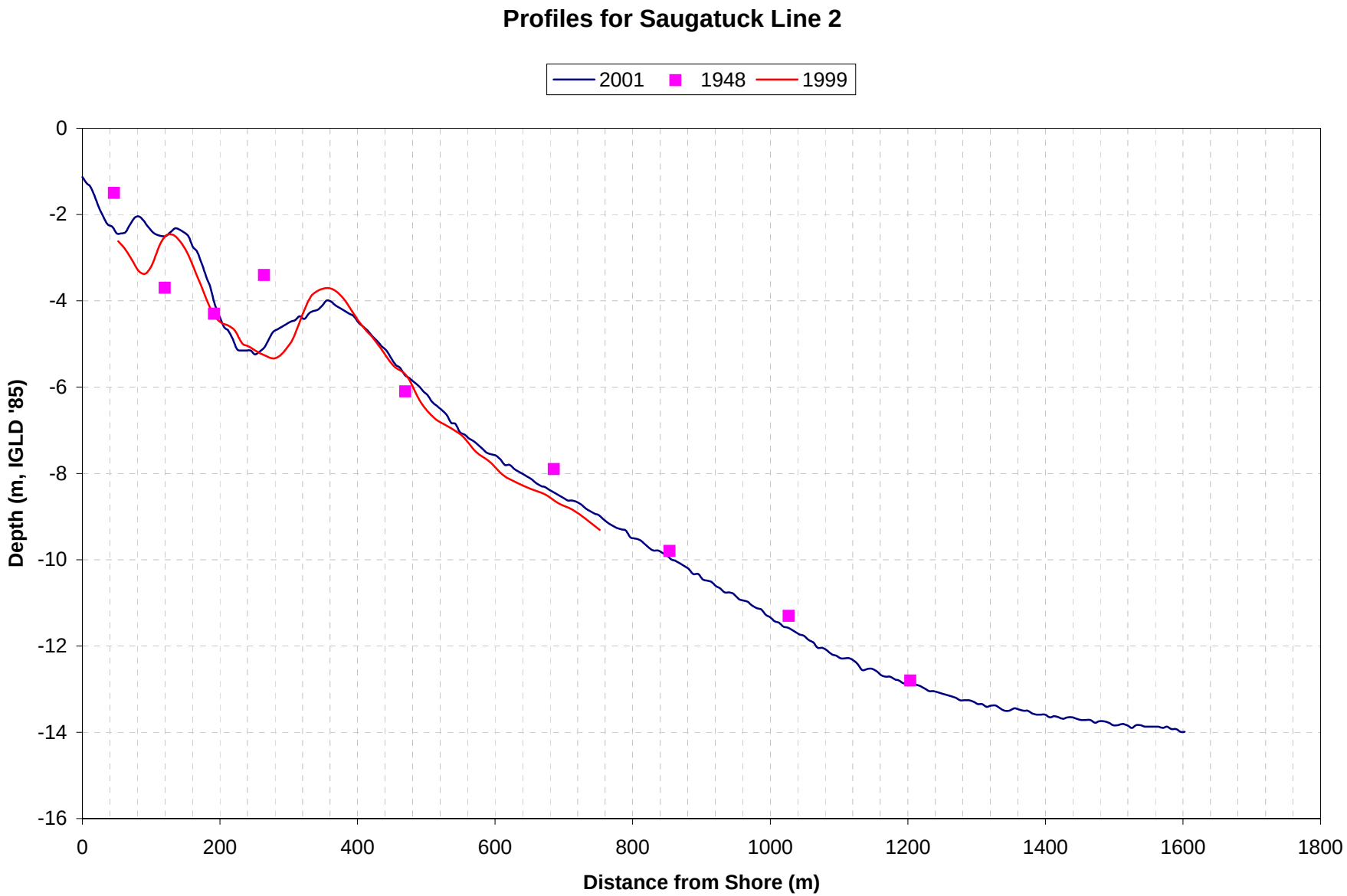
-14
-16
Depth (m, IGLD '85)
0
200
400
600
800
1000
1200
1400
1600
1800
Distance from Shore (m)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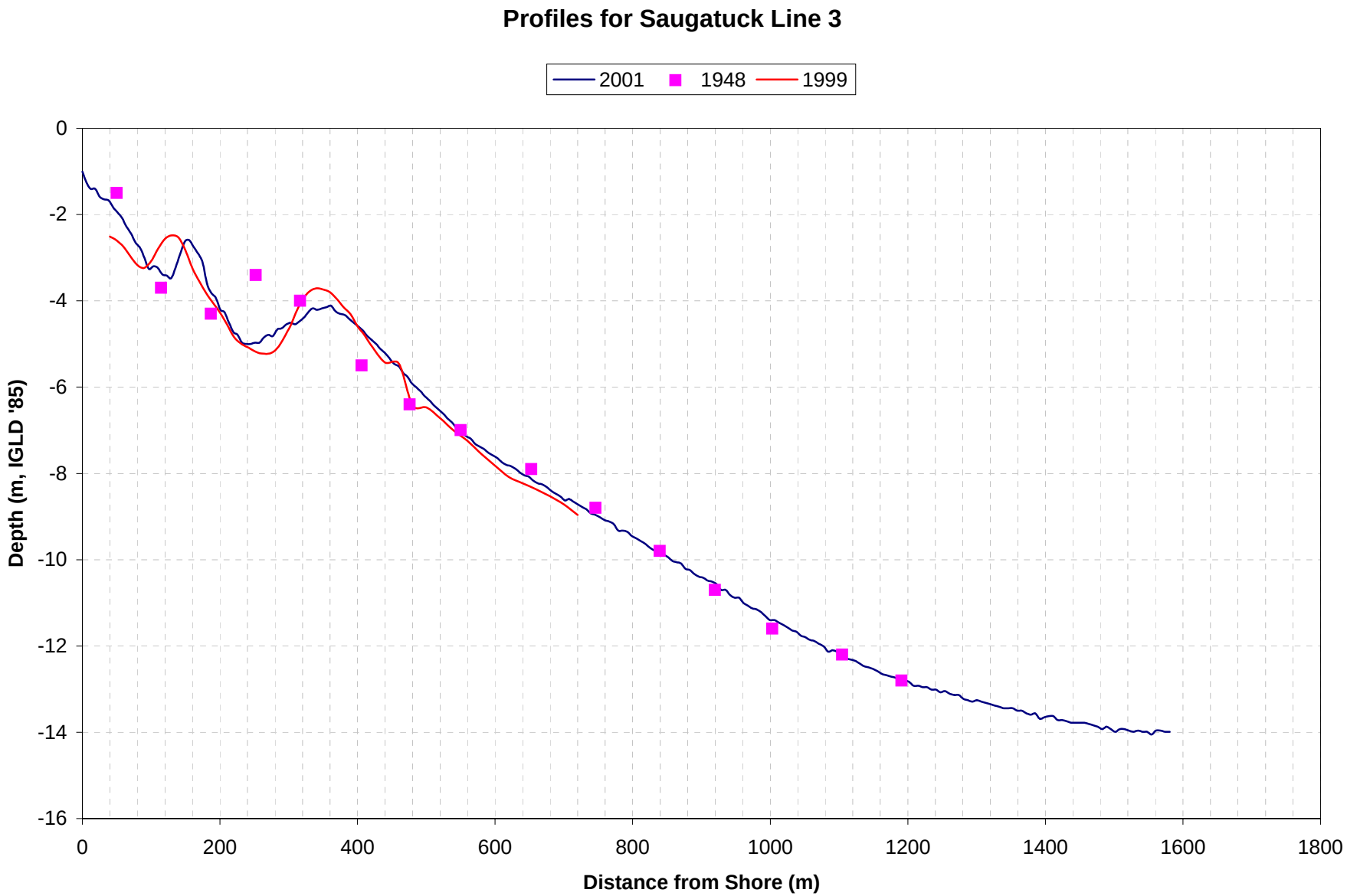
1600
1800
Distance from Shore (m)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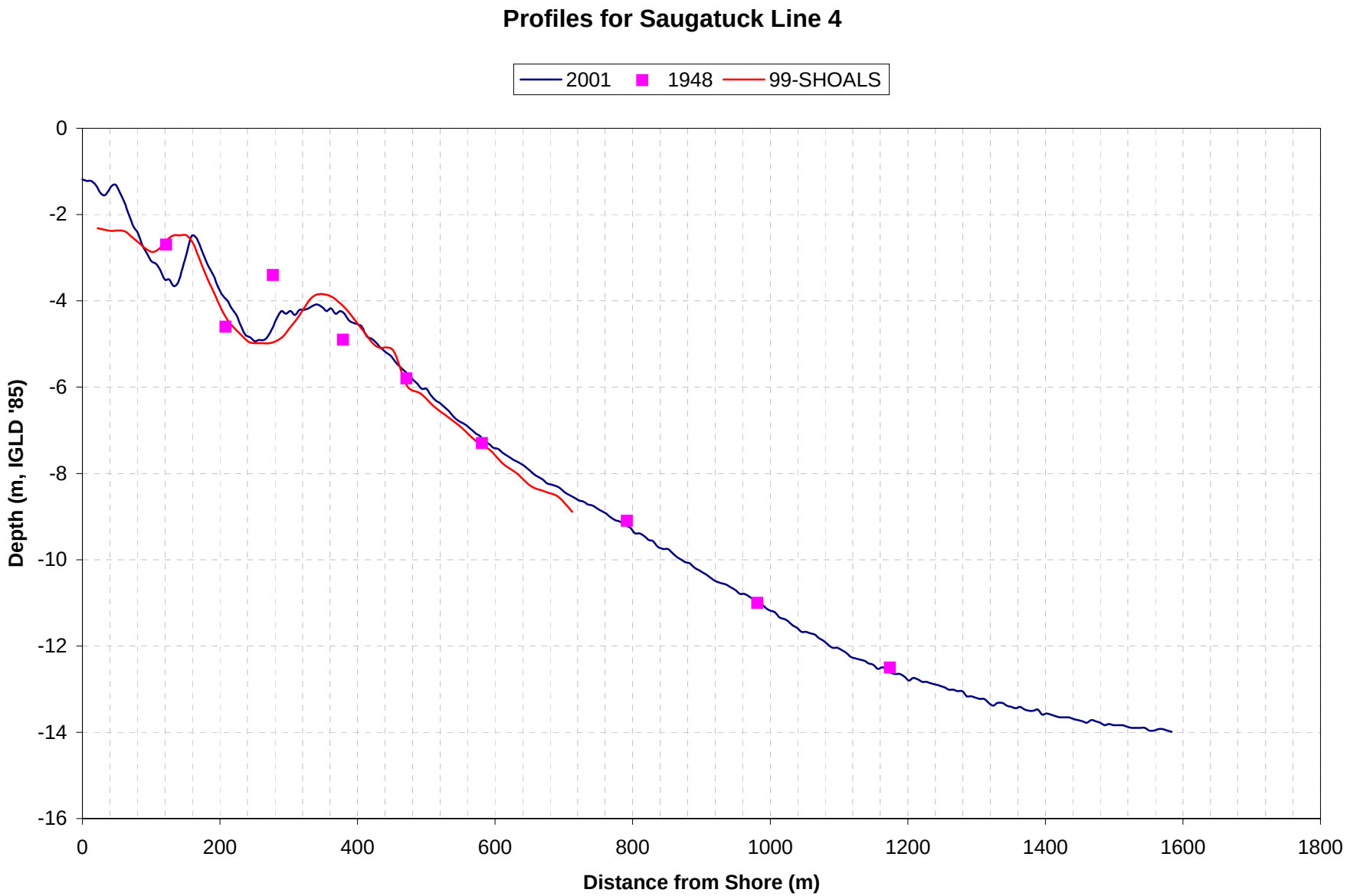
1600
1800
Distance from Shore (m)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1200
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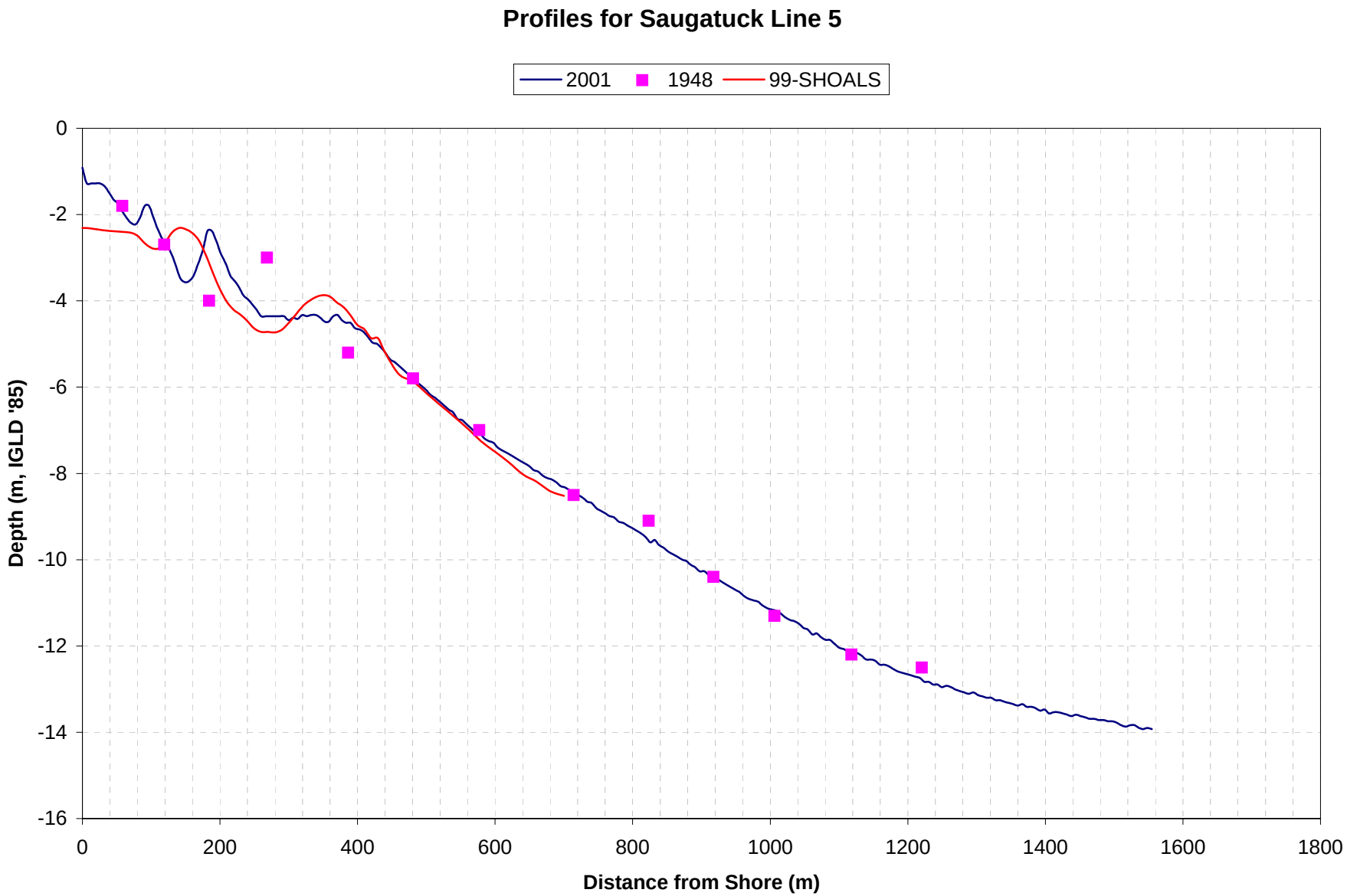
1600
1800
Distance from Shore (m)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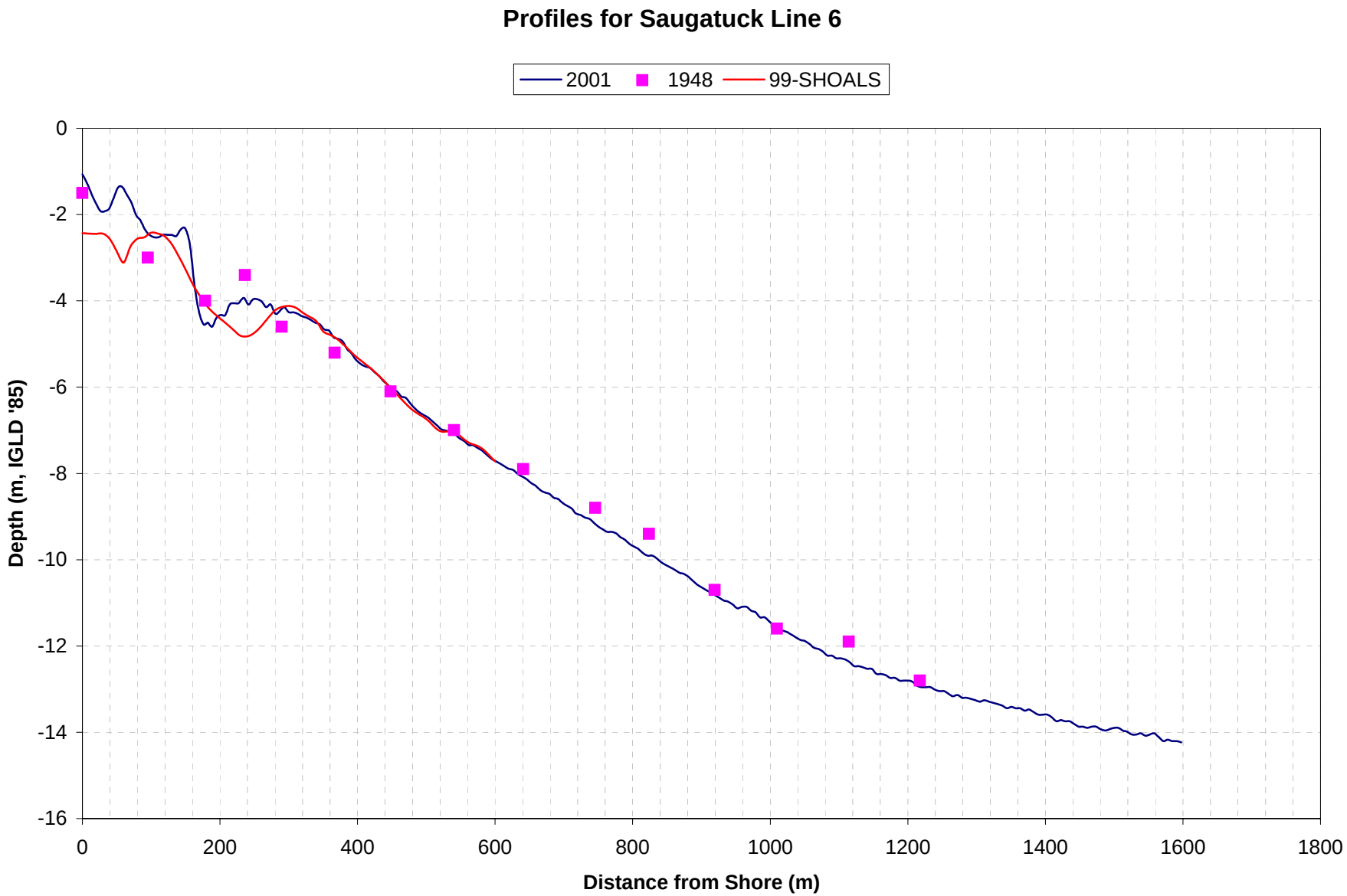
400
600
800
1000
1200
1400
1600
1800
Distance from Shore (m)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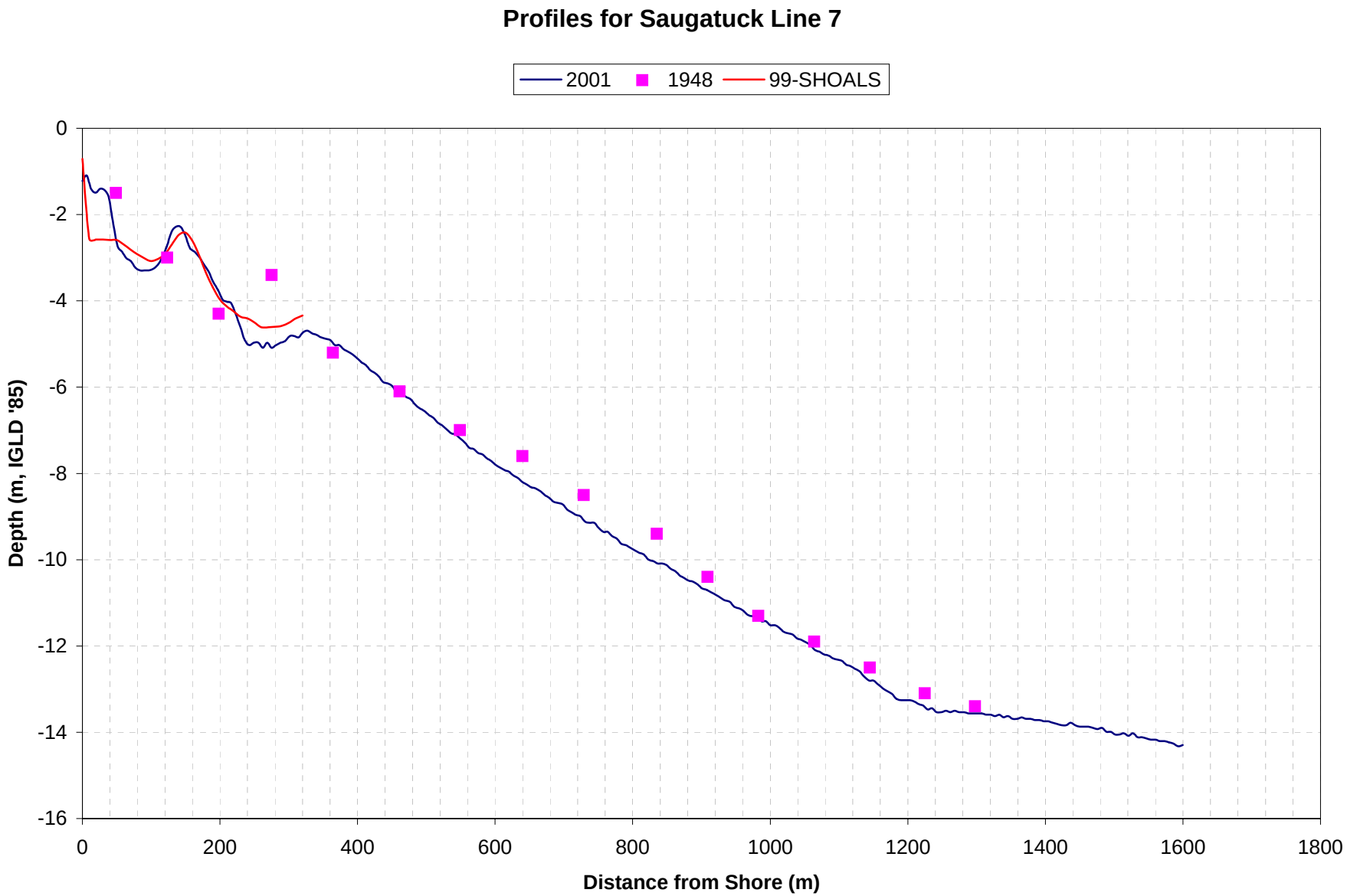
1800
Distance from Shore (m)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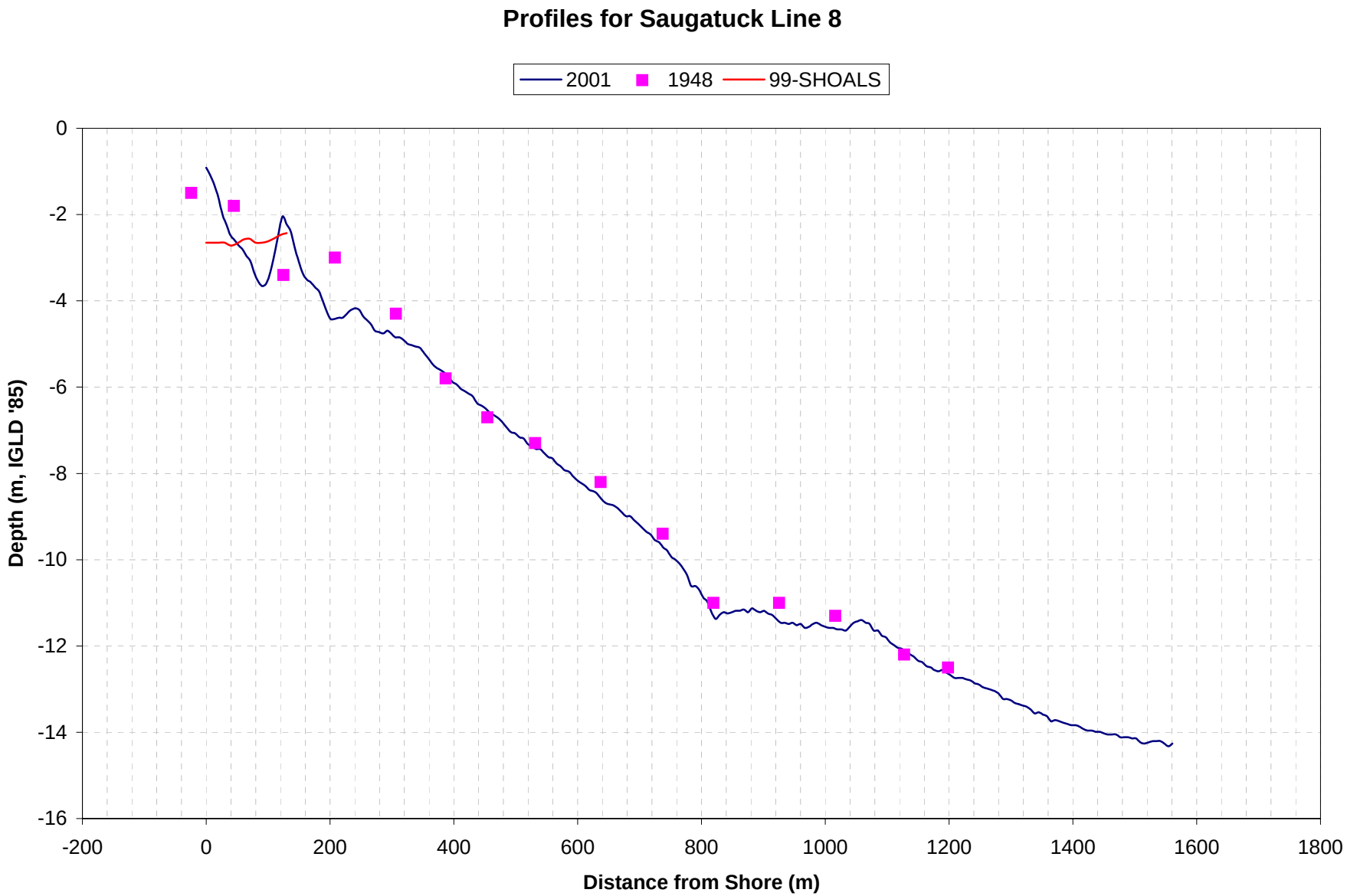
1800
Distance from Shore (m)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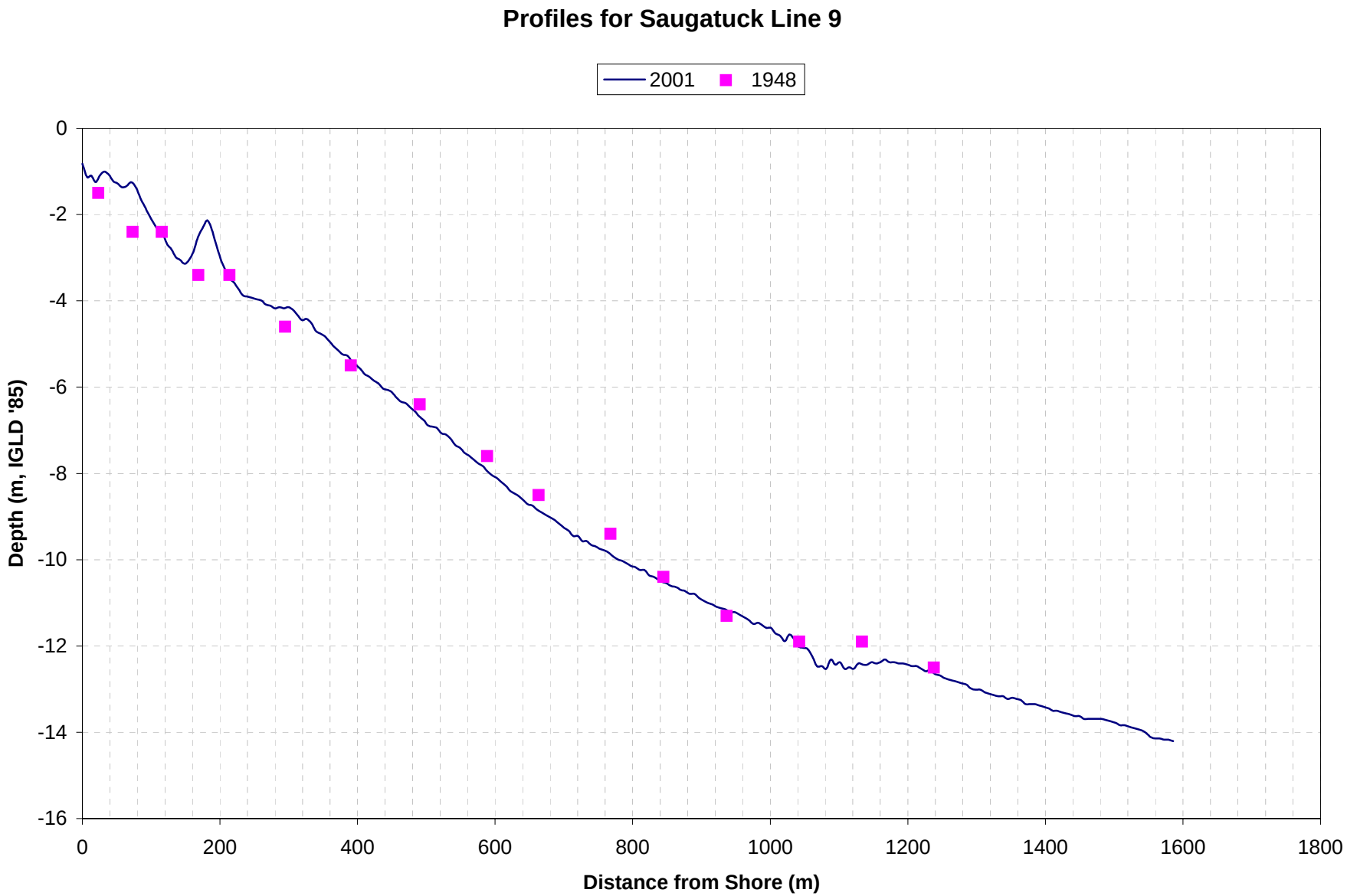
Distance from Shore (m)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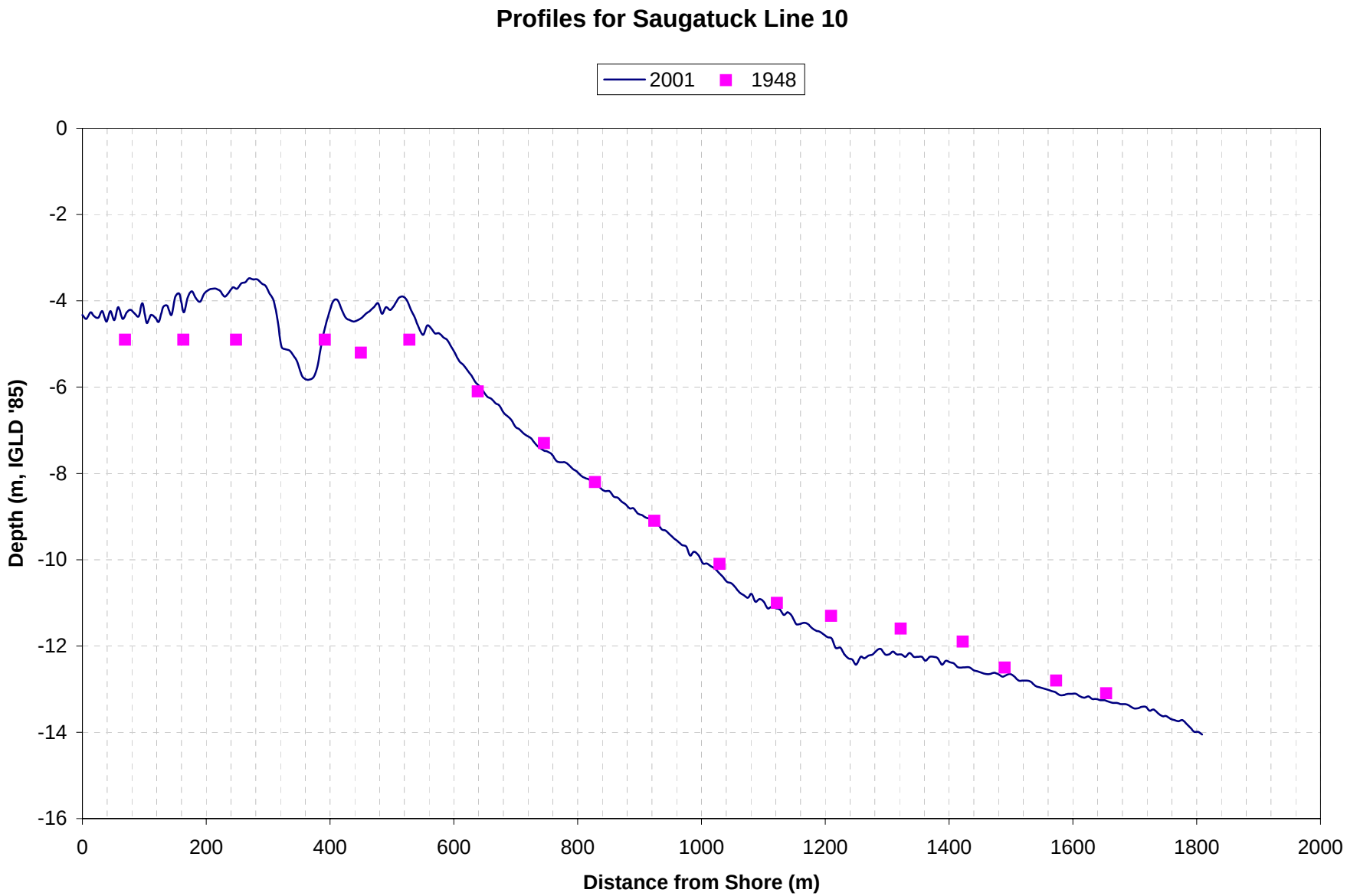
1600
1800
Distance from Shore (m)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2000
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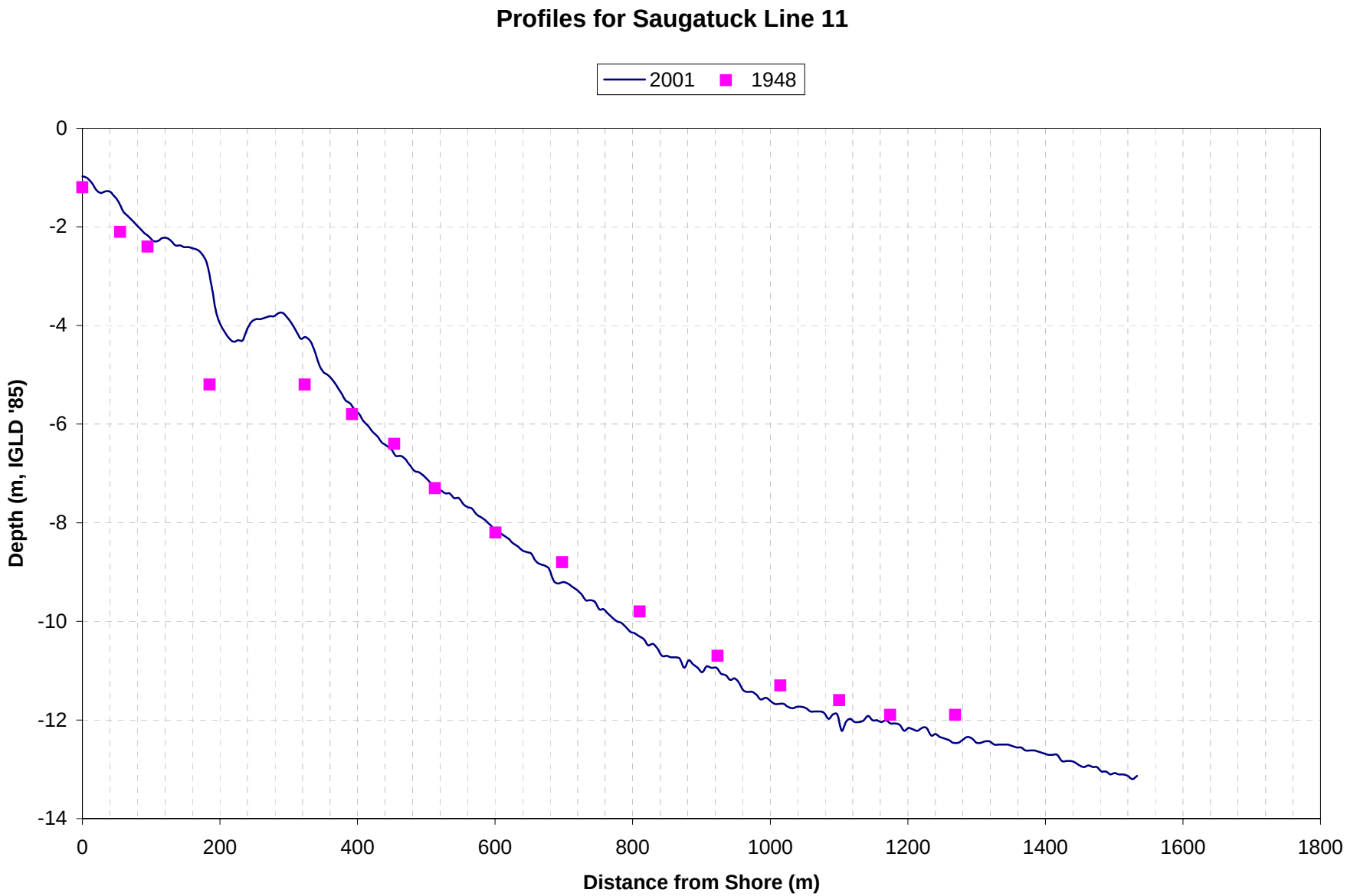
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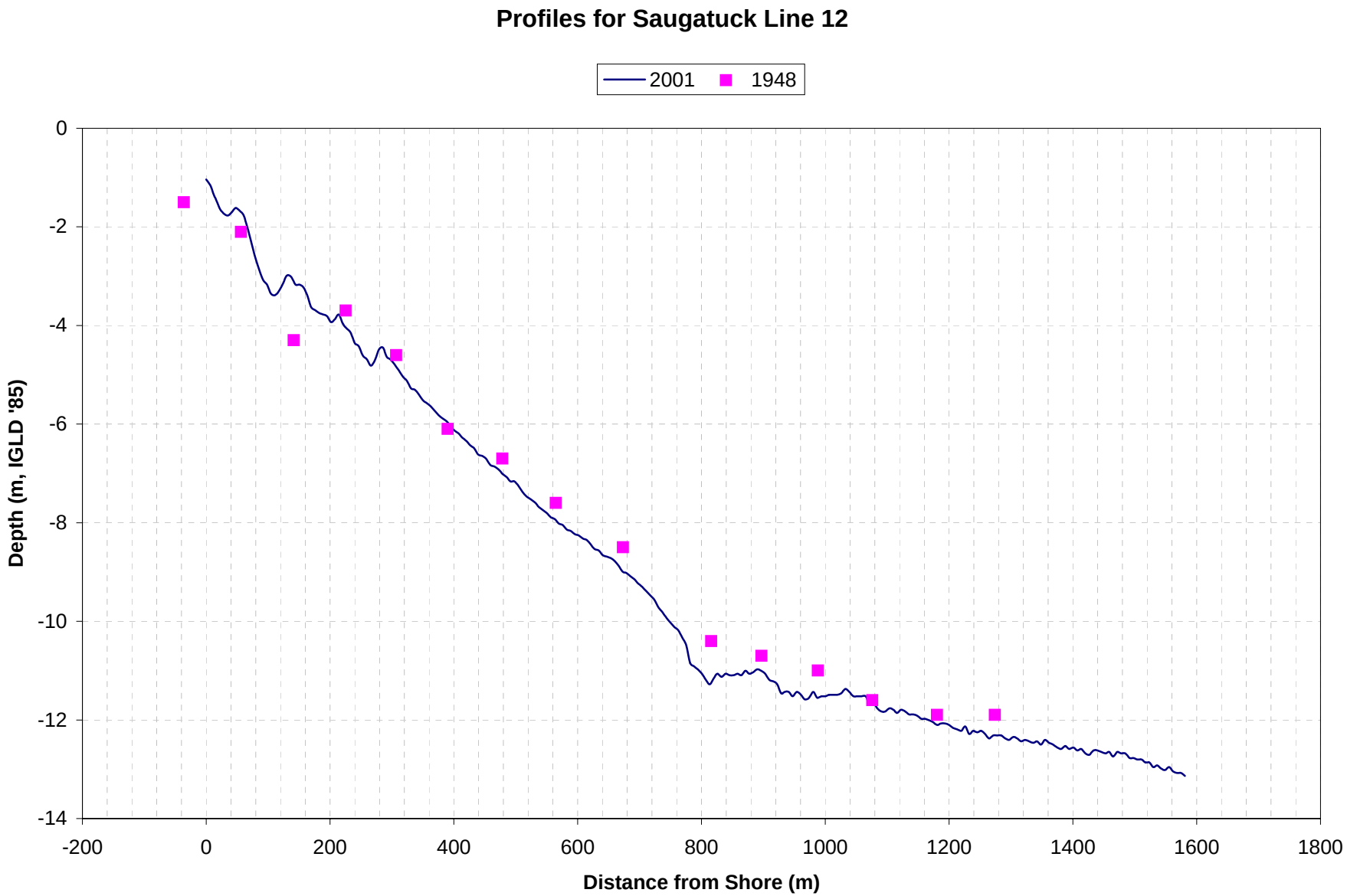
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Distance from Shore (m)

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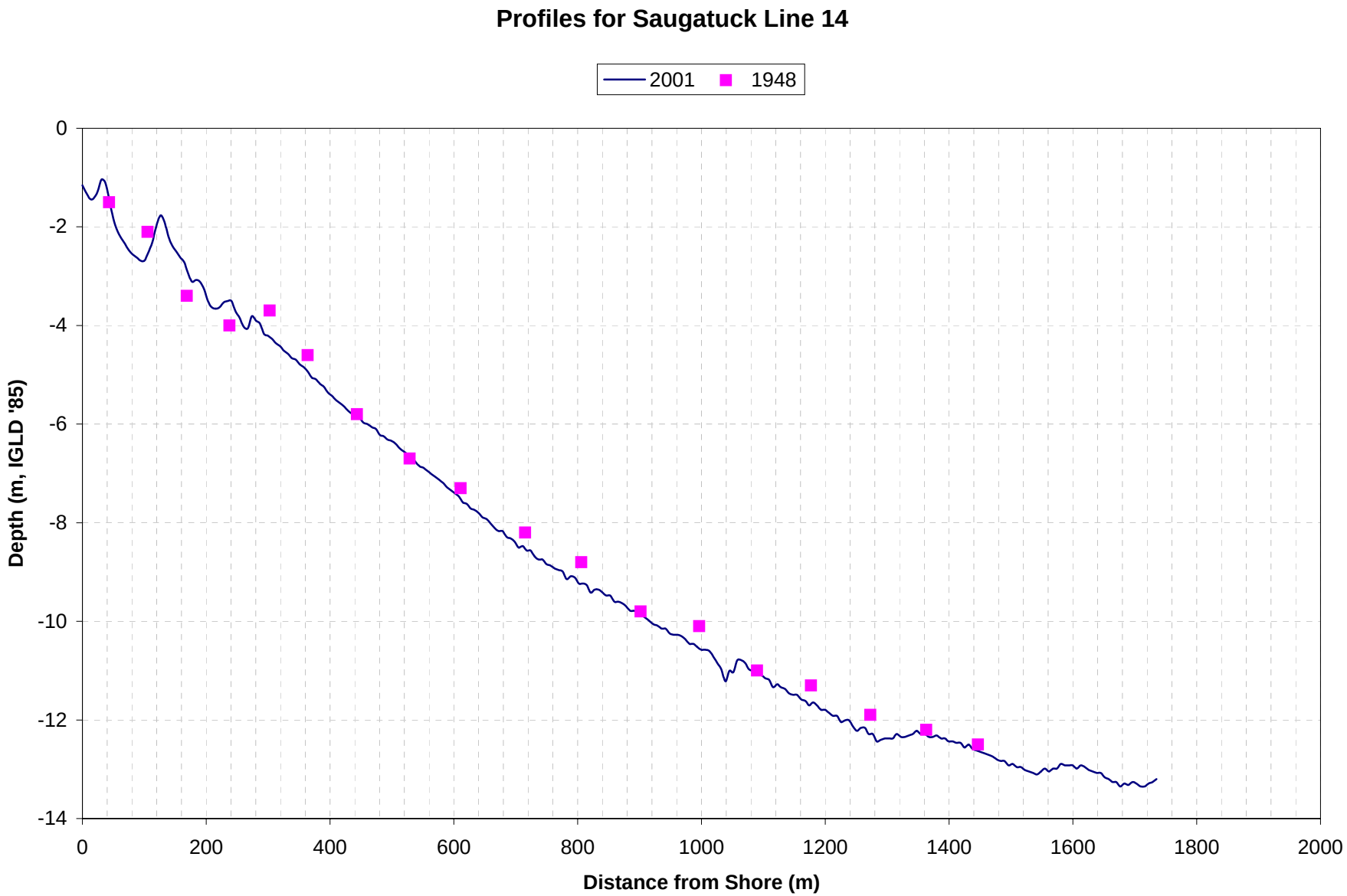
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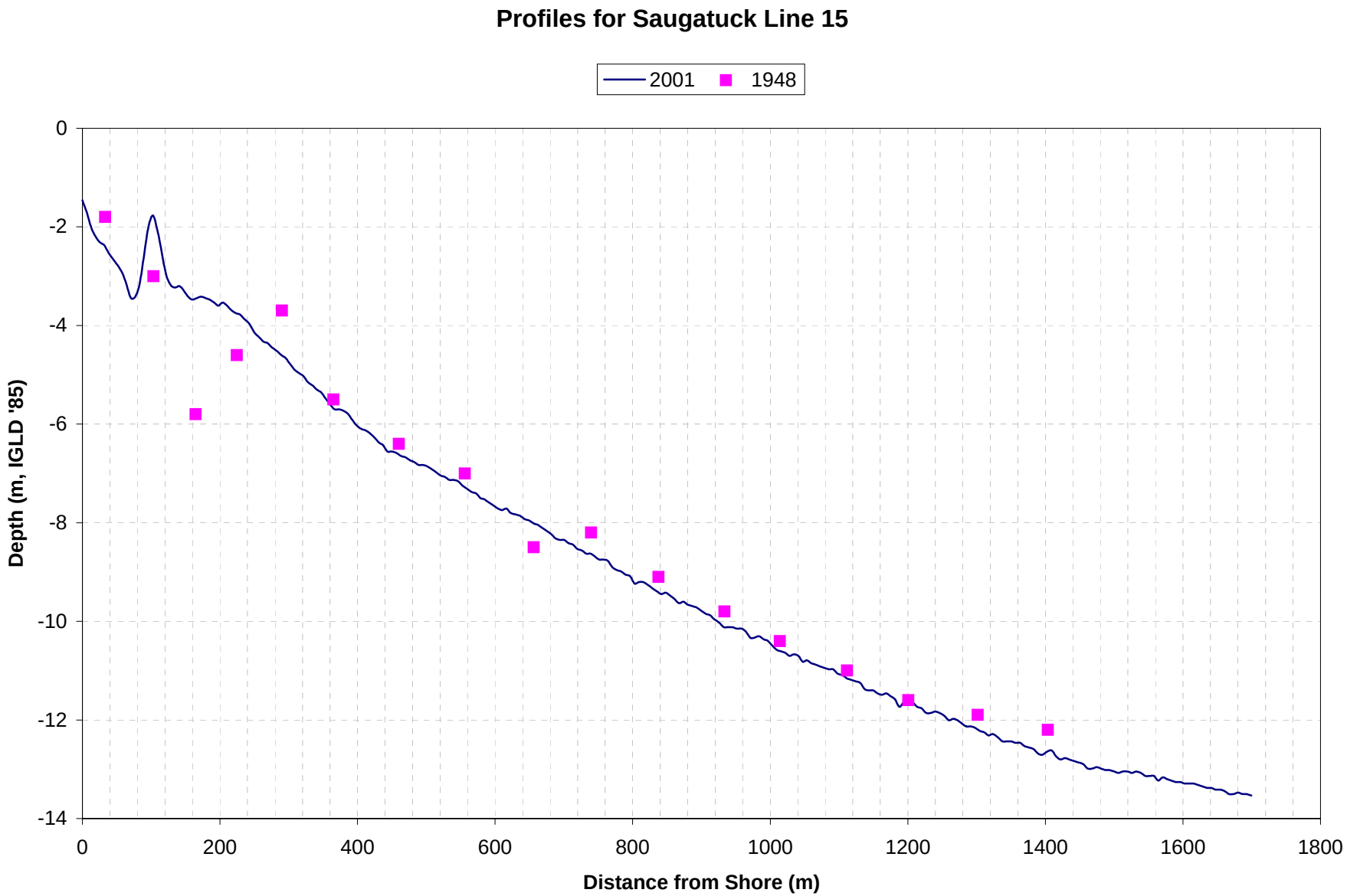
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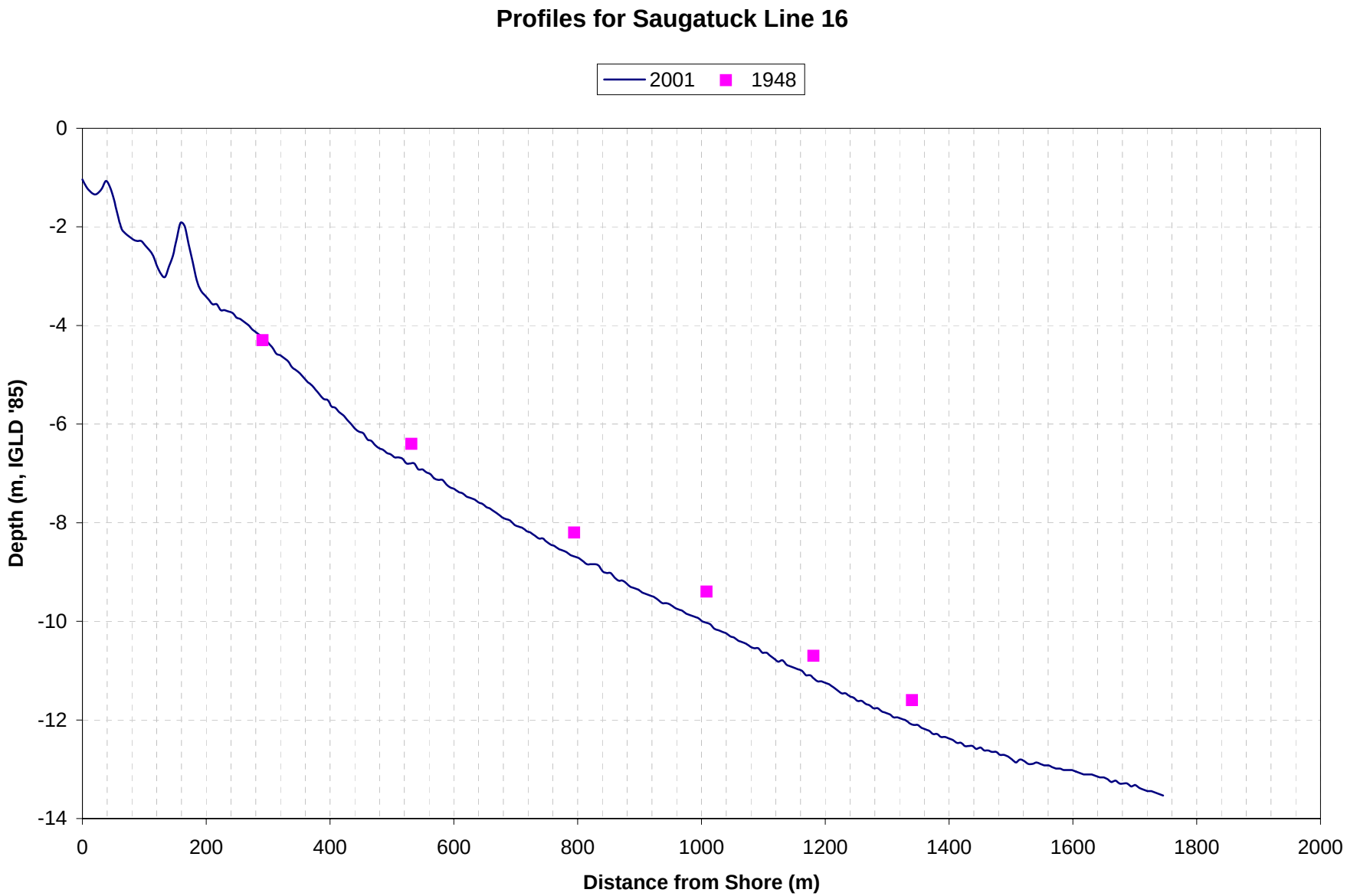
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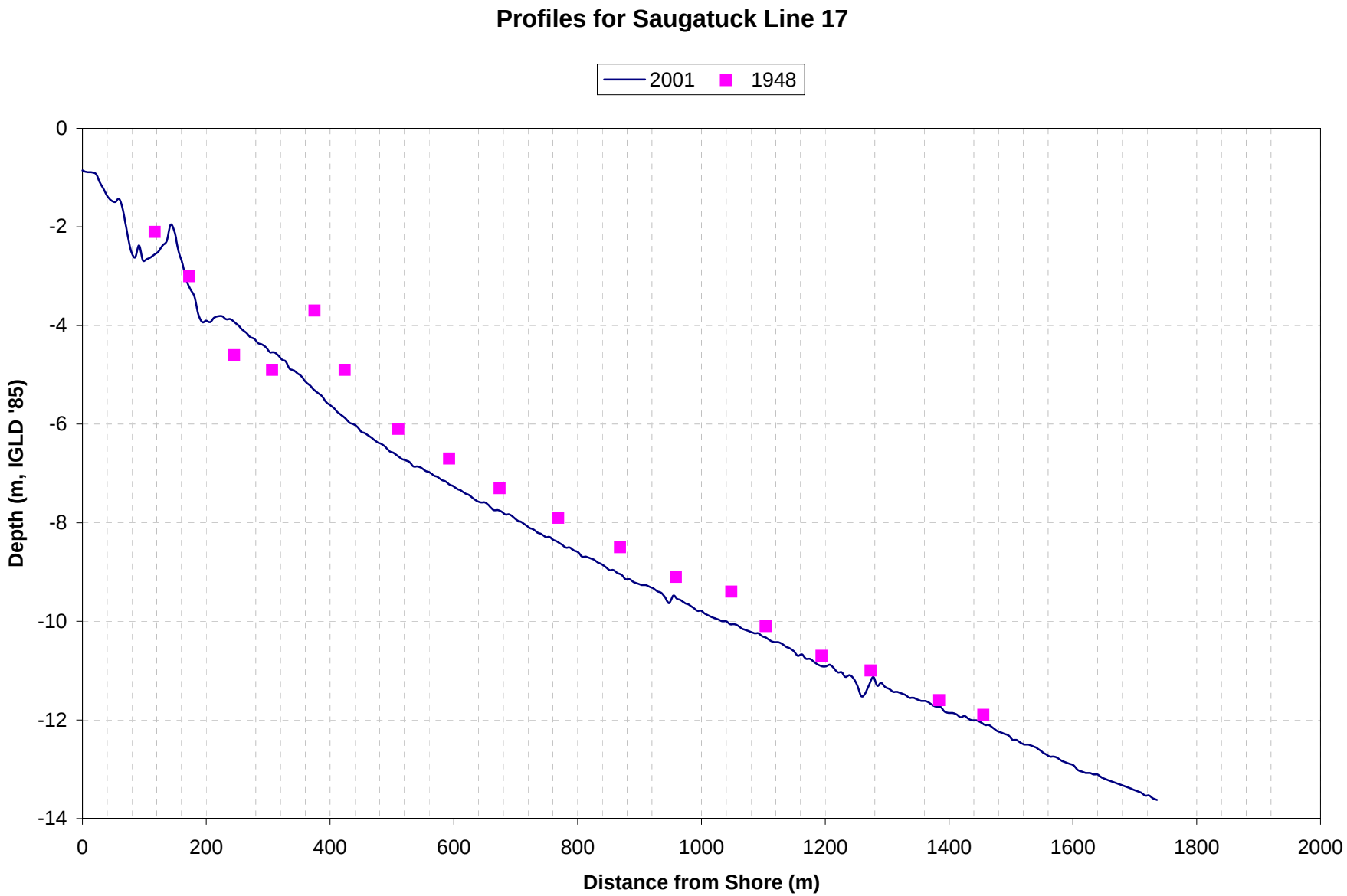
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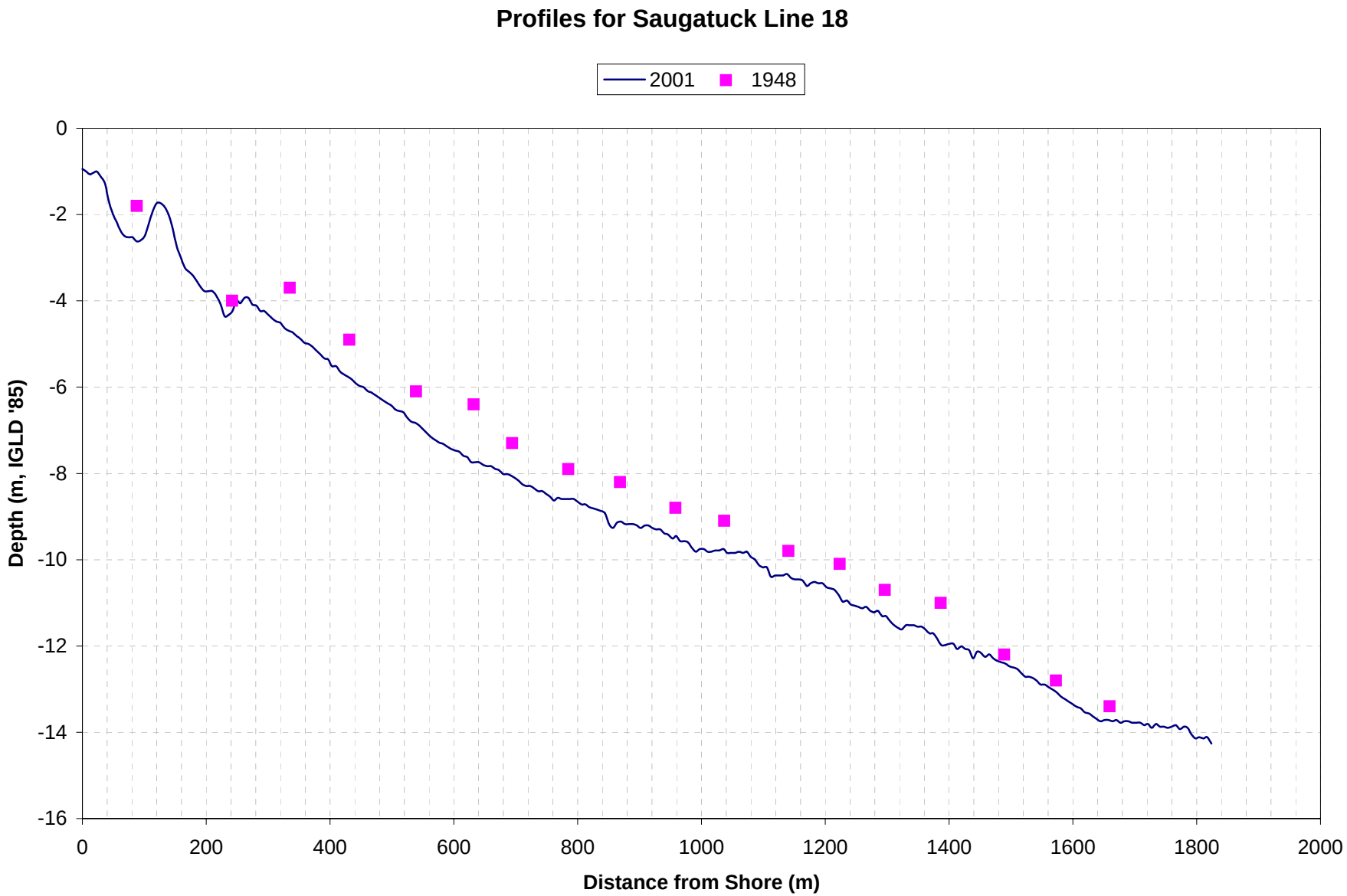
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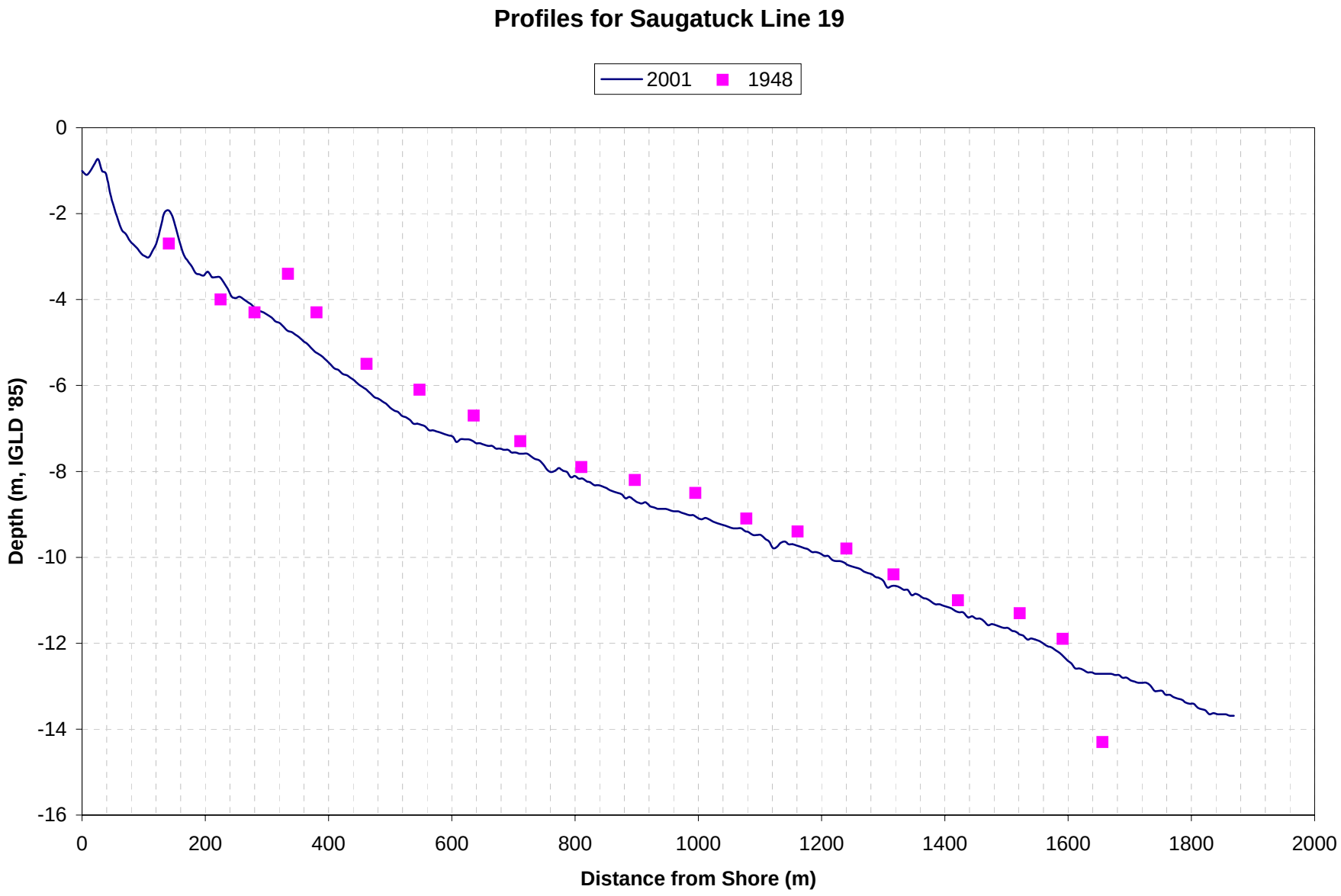
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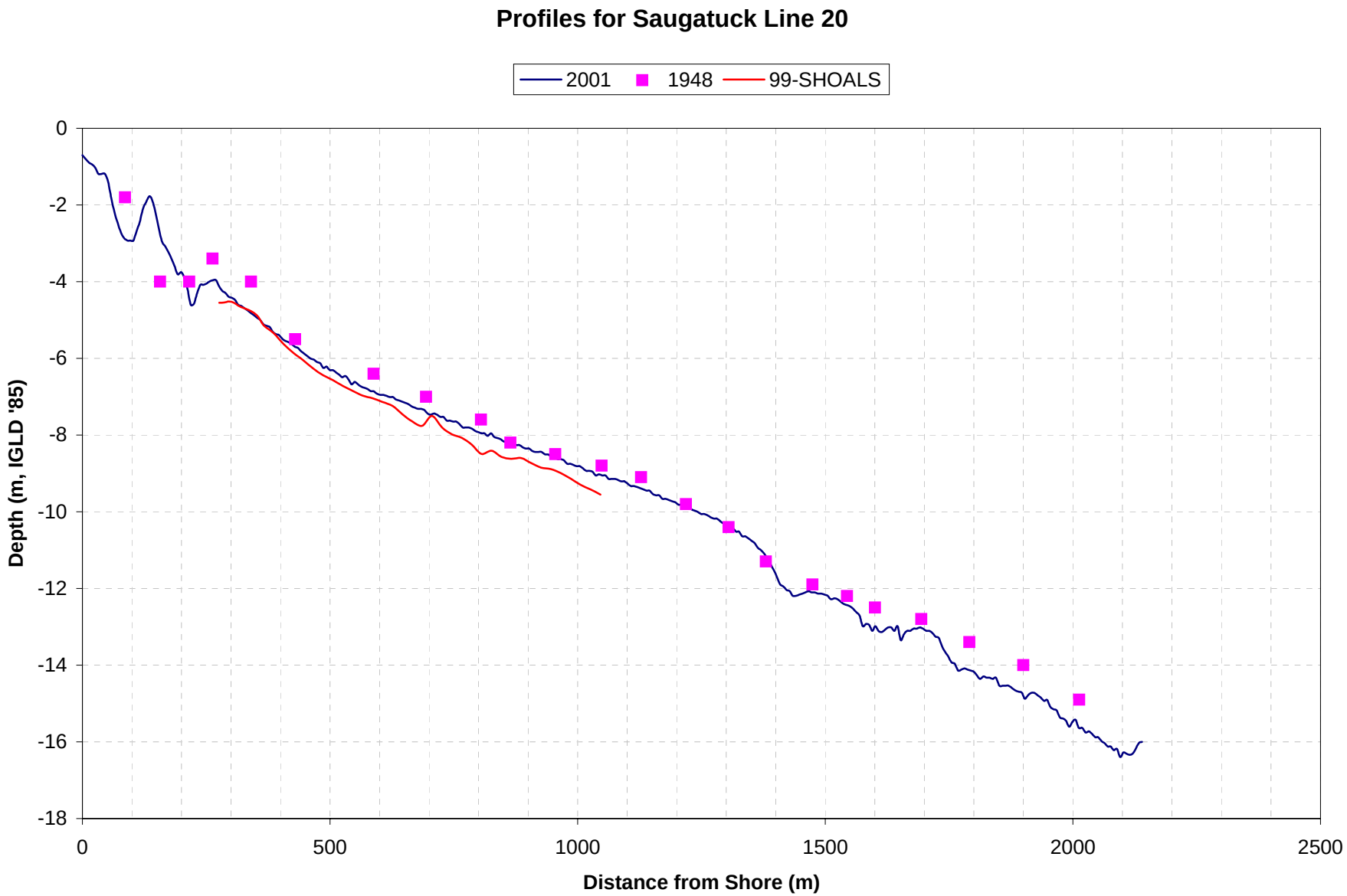
2000
Distance from Shore (m)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Depth (m, IGLD '85)
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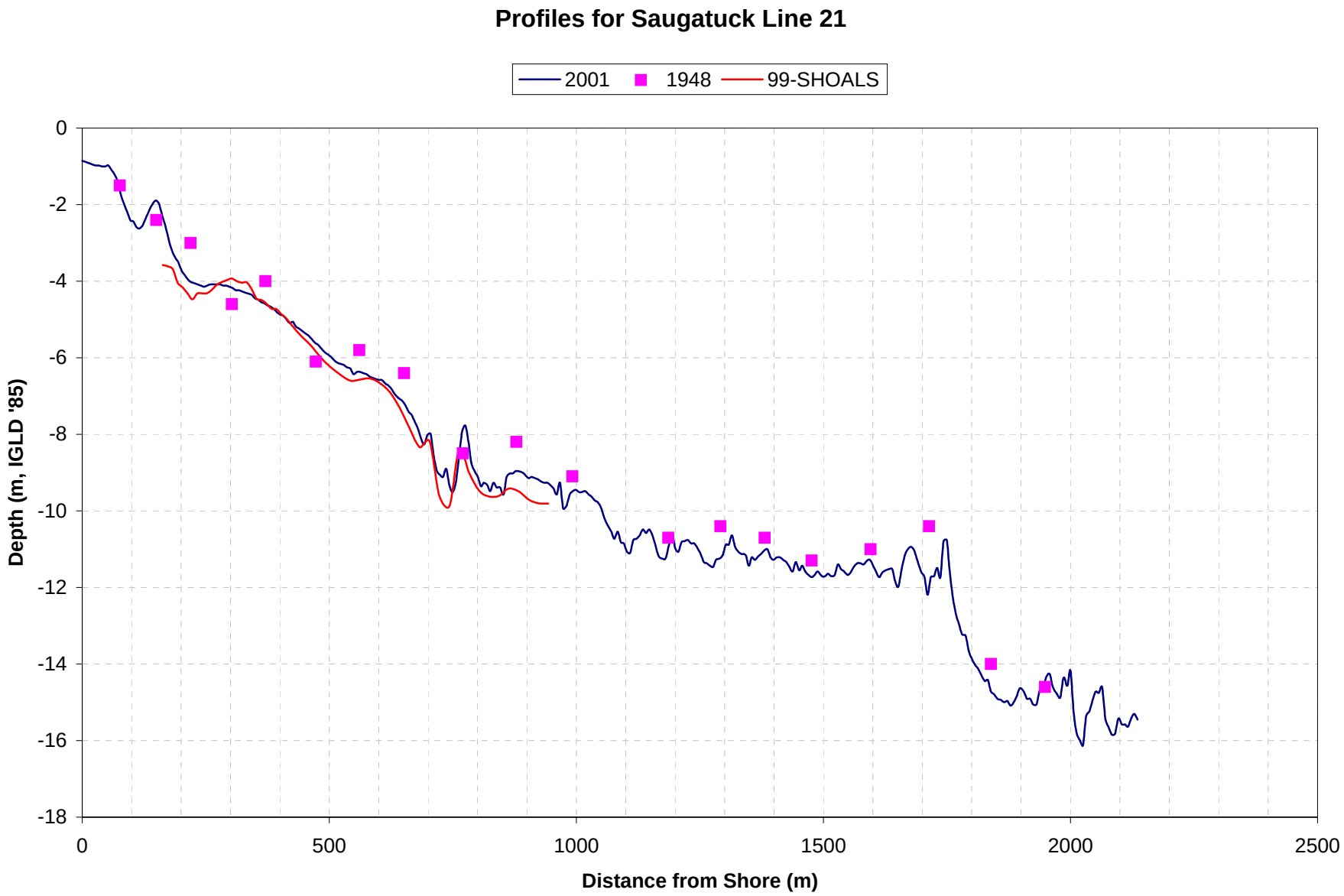
1000
1500
2000
2500
Distance from Shore (m)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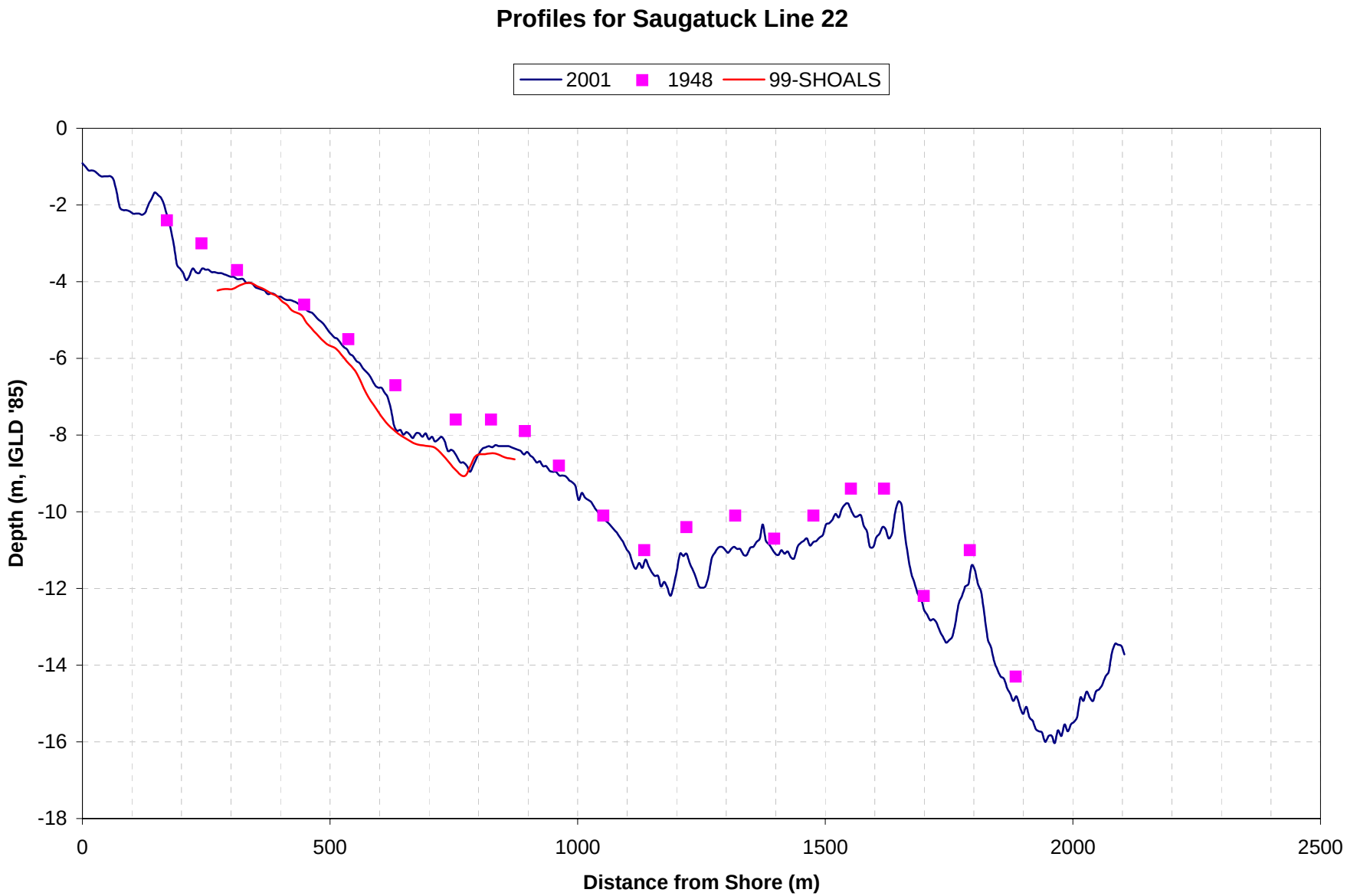
Distance from Shore (m)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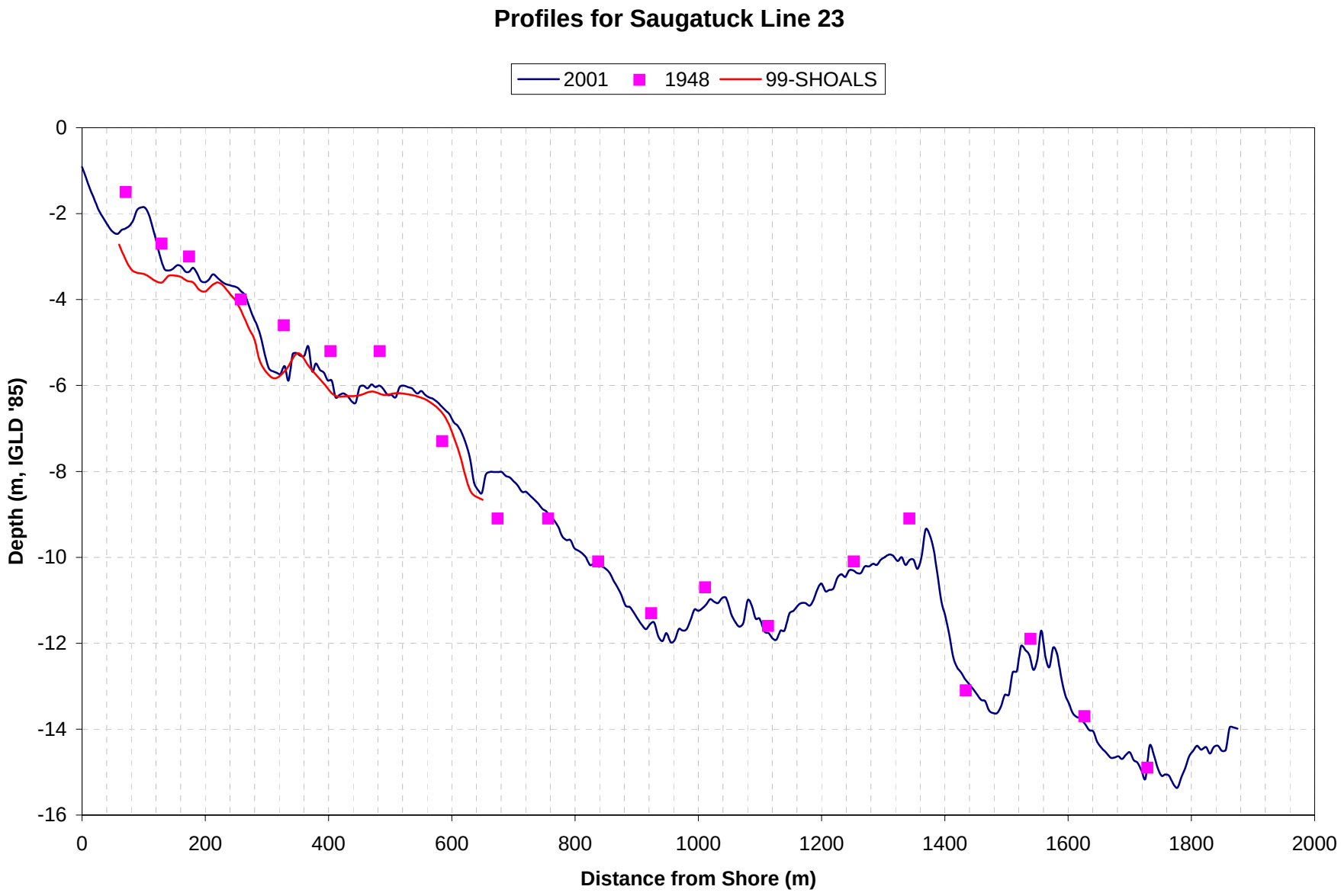
2500
Distance from Shore (m)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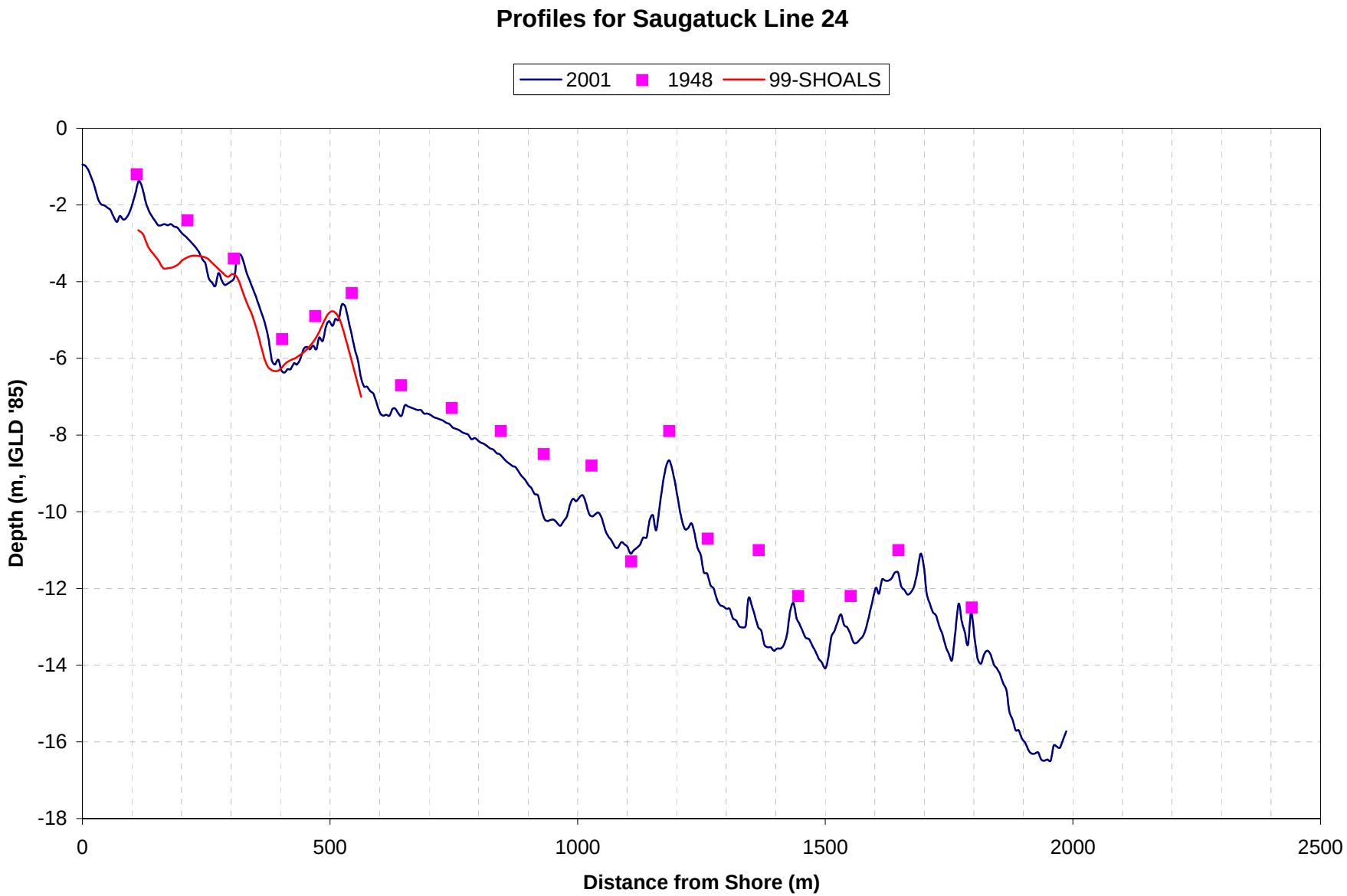
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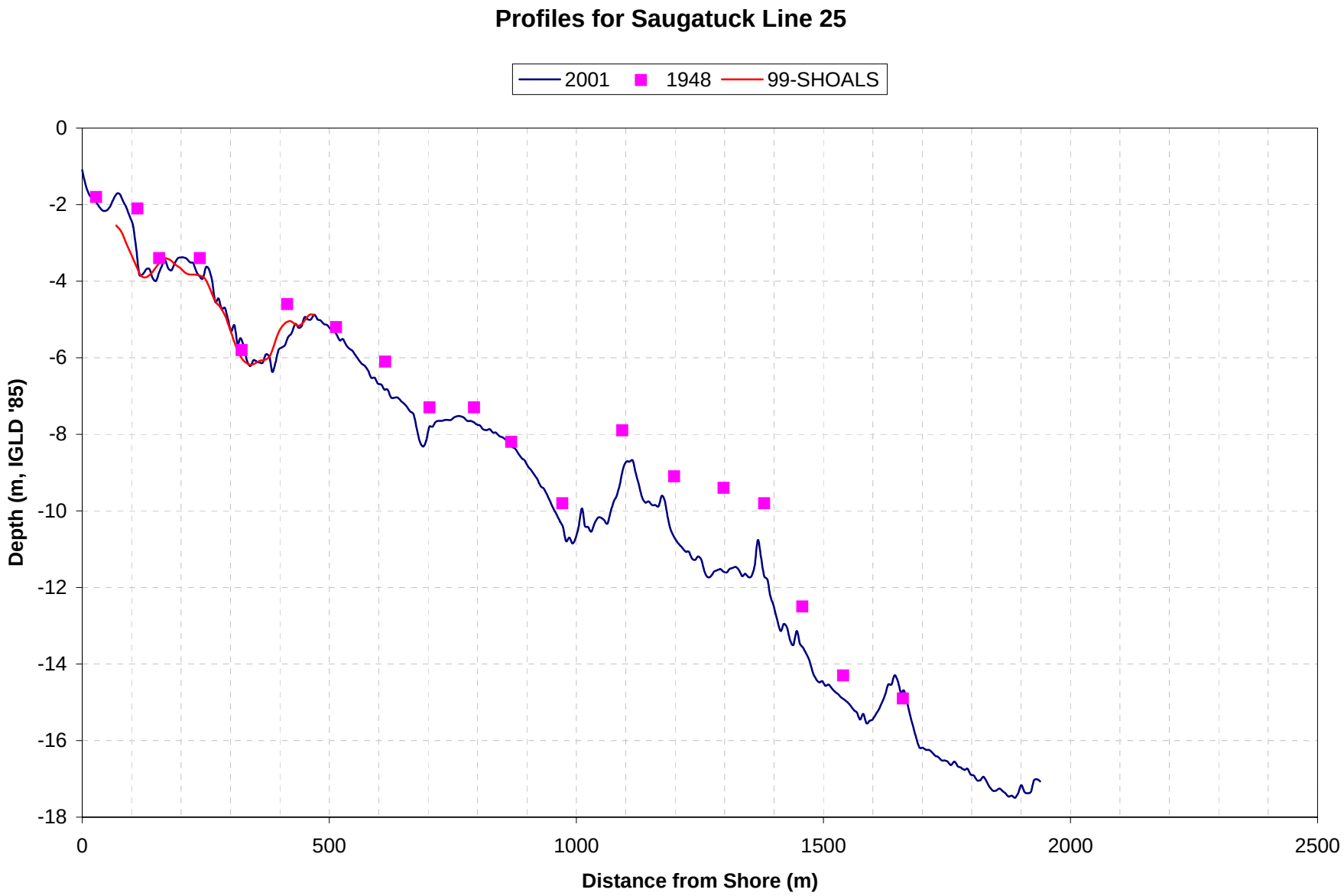
2000
2500
Distance from Shore (m)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Depth (m, IGLD '85)
0
500
1000
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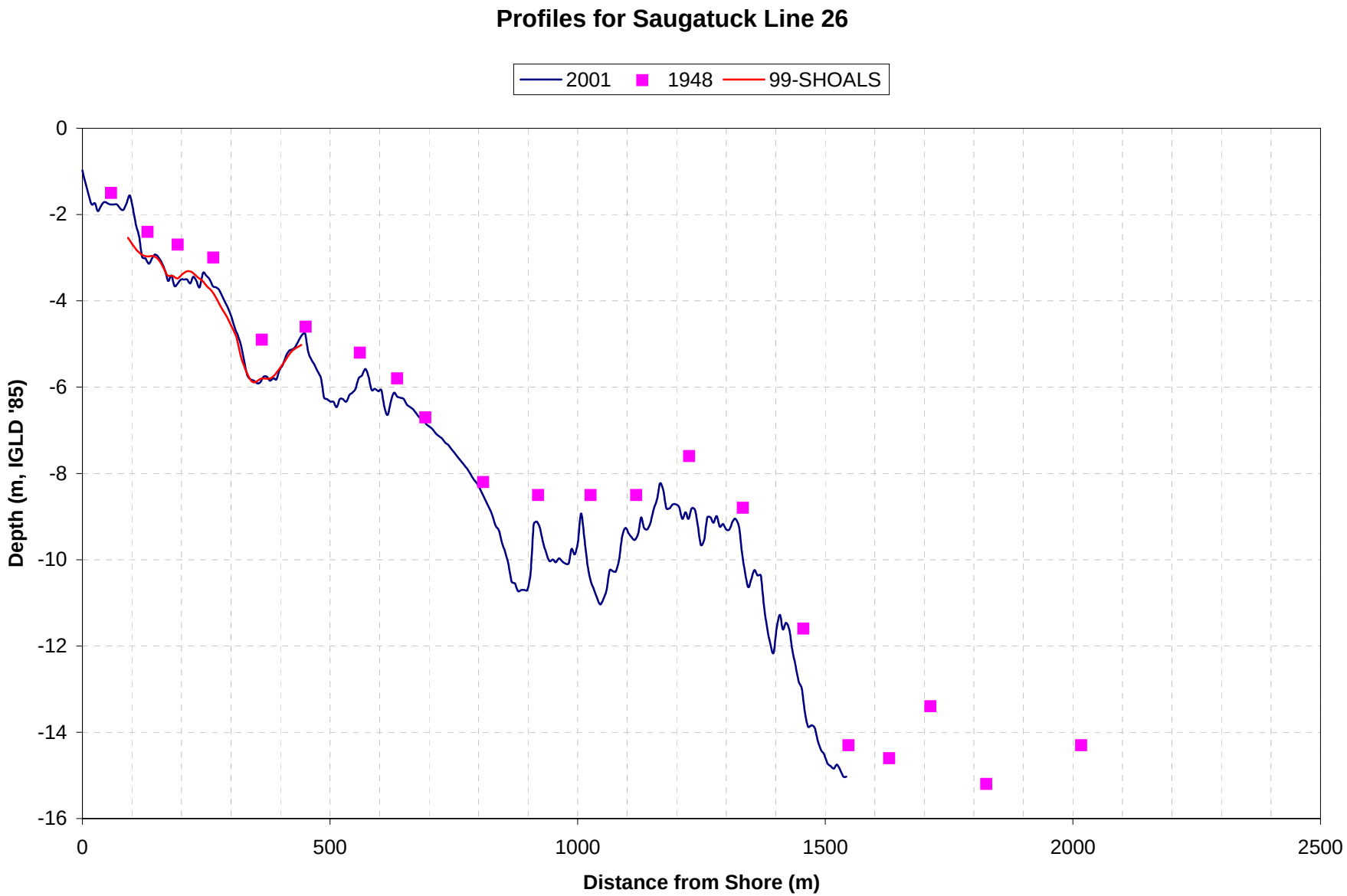
2000
2500
Distance from Shore (m)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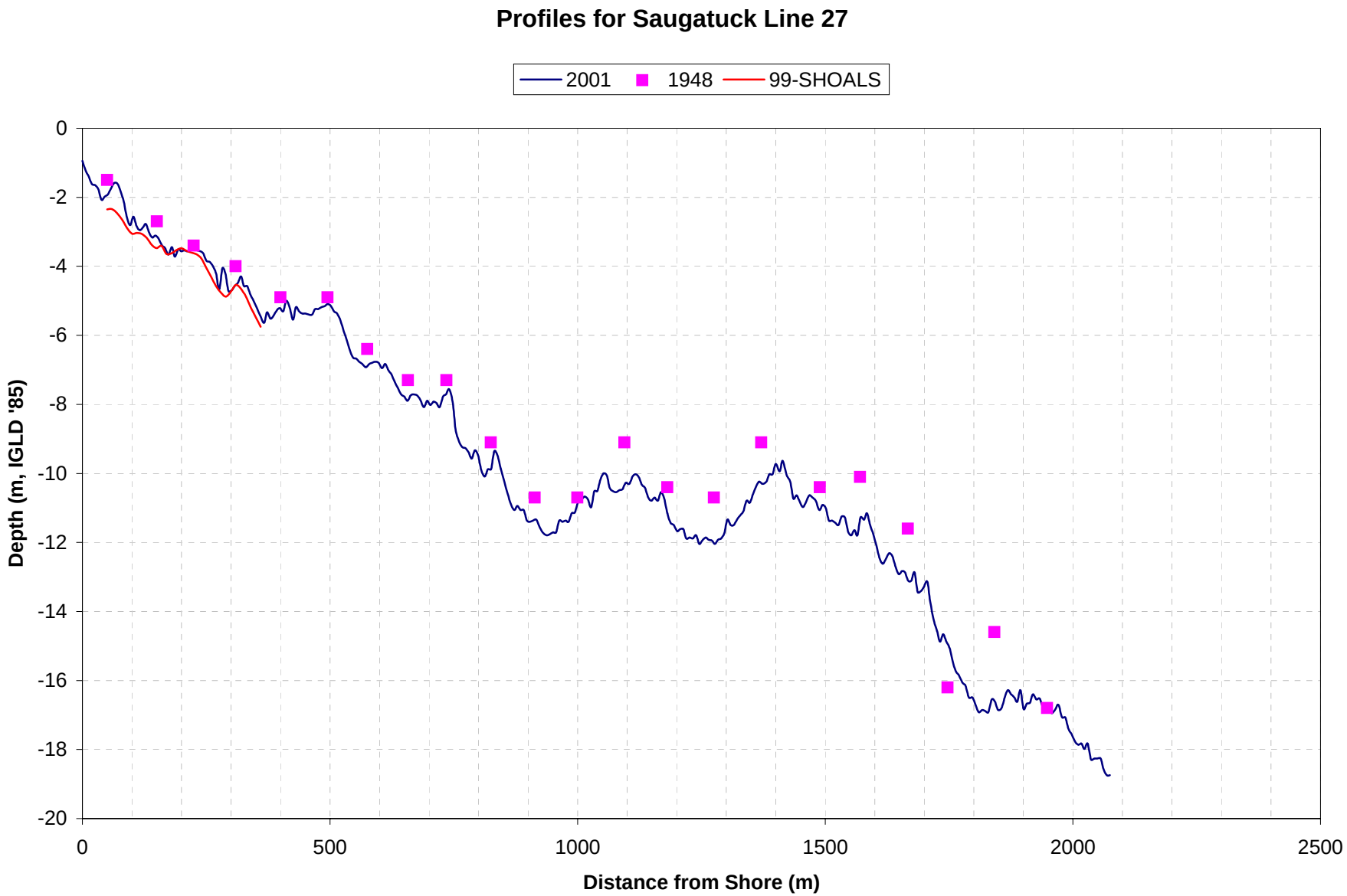
2000
2500
Distance from Shore (m)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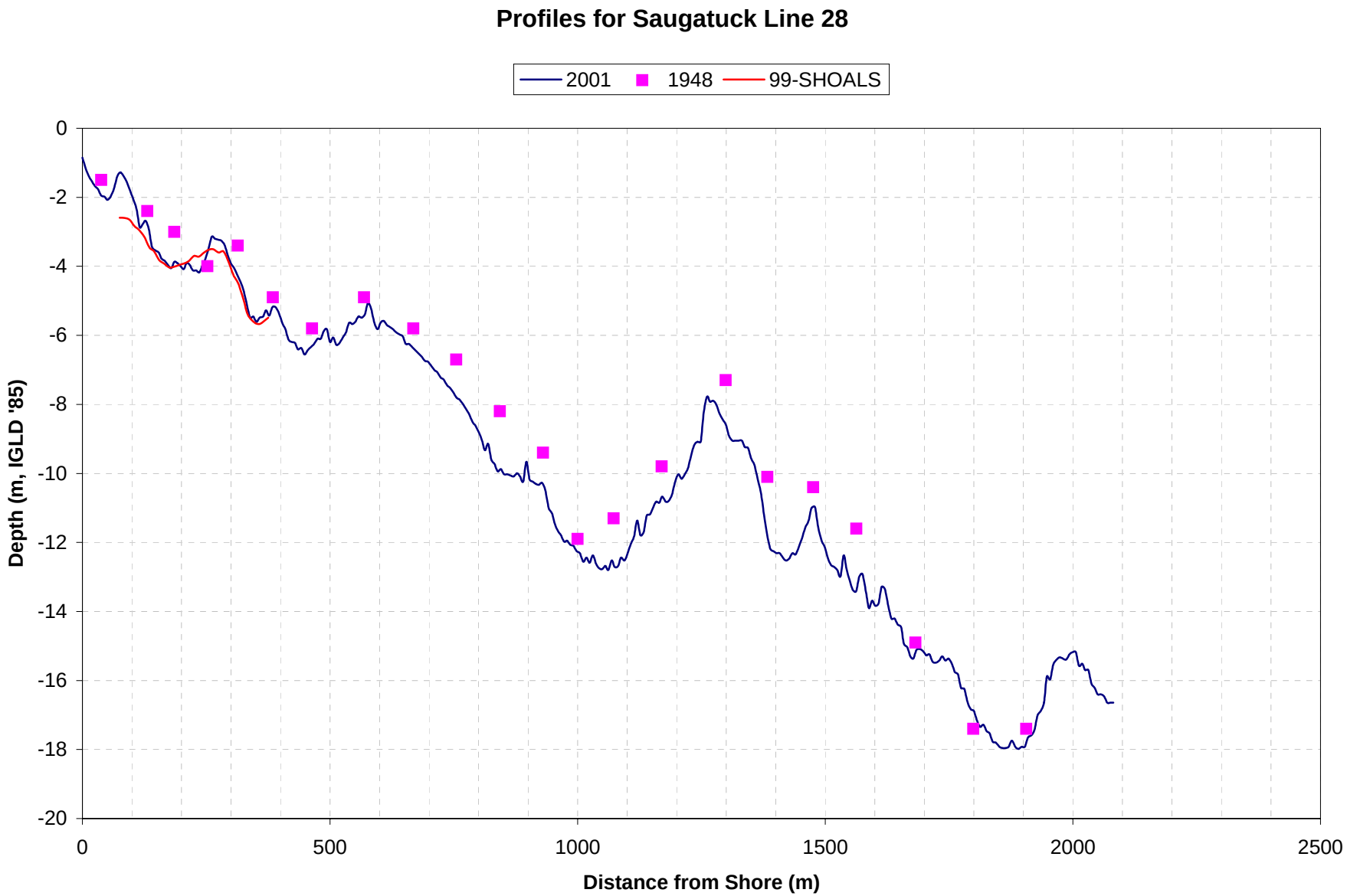
2000
2500
Distance from Shore (m)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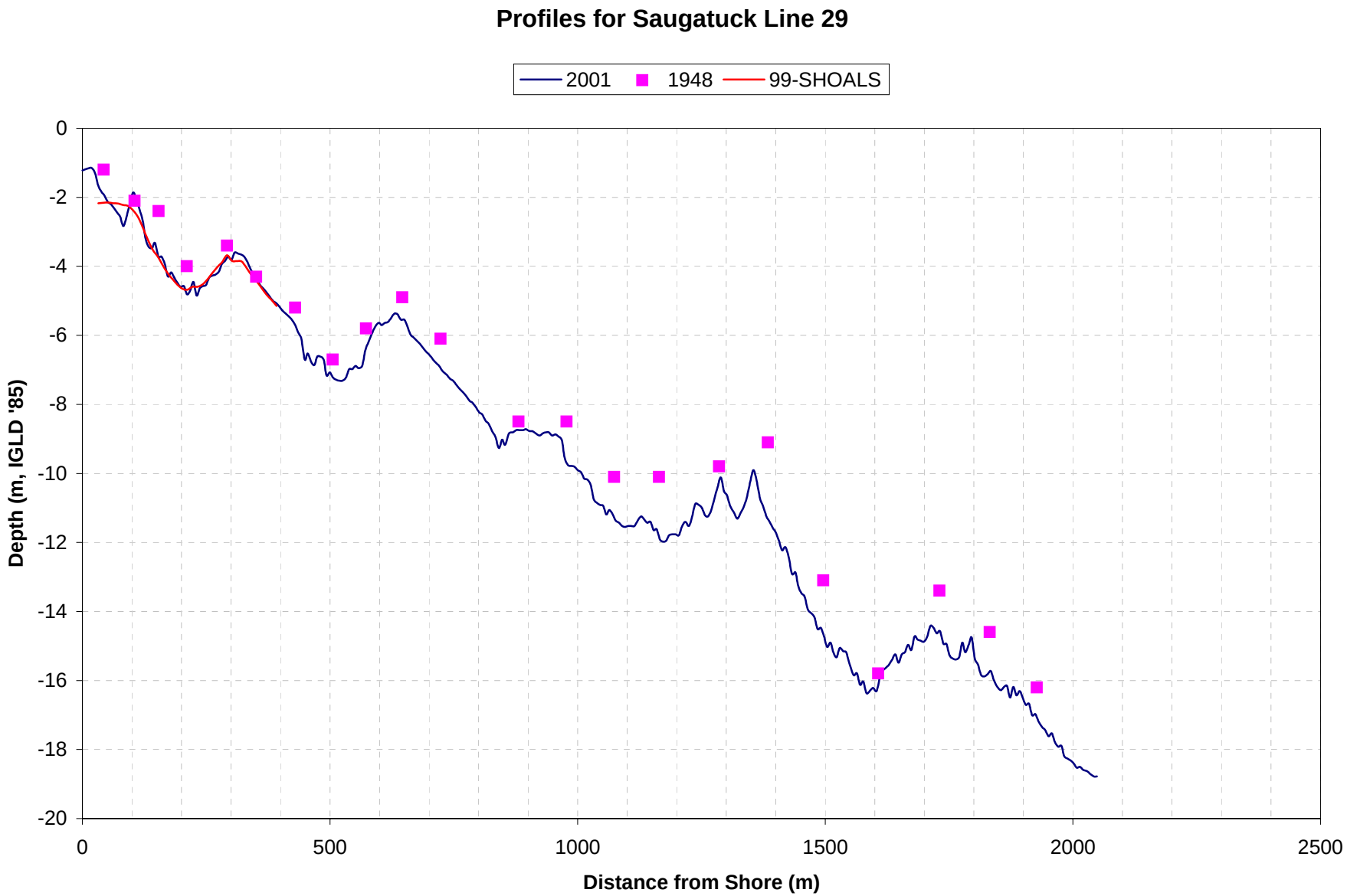
1500
2000
2500
Distance from Shore (m)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-10
-12
-14
-16
-18
-20
Depth (m, IGLD '85)
0
500
1000
1500
2000
2500
Distance from Shore (m)

APPENDIX B

FORMULAS FOR LOCAL BYPASSING RATIO

Appendix B

Formulas for local bypassing ratio:

mwl = 0.0 m, rivel flow = 0.0 :

$$r_{\text{bypass}} = \tanh\left(\frac{\Theta}{50}\right) - 0.05\ \tanh\left(\frac{30}{\Theta^{1.2}}\right) + 0.05\tanh\left(\frac{1}{\Theta^{5}}\right)$$

mwl = 0.0 m, rivel flow = 20.0 m^3/s/calculation grid:

$$r_{\text{bypass}} = \tanh\left(\frac{\Theta}{60}\right) - 0.35\ \tanh\left(\frac{30}{\Theta^{1.2}}\right) + 0.35\tanh\left(\frac{10}{\Theta^{5}}\right)$$

mwl = 0.0 m, rivel flow = 40.0 m^3/s/calculation grid:

$$r_{\text{bypass}} = \tanh\left(\frac{\Theta}{80}\right) - 0.85\ \tanh\left(\frac{50}{\Theta^{1.2}}\right) + 0.85\tanh\left(\frac{1}{\Theta^{5}}\right)$$

mwl = 1.4 m, rivel flow = 0.0 :

$$r_{\text{bypass}} = \tanh\left(\frac{\Theta}{100}\right) - 0.2\ \tanh\left(\frac{60}{\Theta}\right) + 0.2\tanh\left(\frac{11}{\Theta^{1.3}}\right)$$

mwl = 1.4 m, rivel flow = 20.0 m^3/s/calculation grid:

$$r_{\text{bypass}} = \tanh\left(\frac{\Theta}{160}\right) - 0.2\ \tanh\left(\frac{80}{\Theta}\right) + 0.2\tanh\left(\frac{10}{\Theta^{1.3}}\right)$$

mwl = 1.4 m, rivel flow = 40.0 m^3/s/calculation grid:

$$r_{\text{bypass}} = \tanh\left(\frac{\Theta}{160}\right) - 0.5\ \tanh\left(\frac{80}{\Theta}\right) + 0.4\tanh\left(\frac{10}{\Theta^{1.3}}\right)$$

Straight beach:

$$r_{\text{bypass}} = \tanh\left(\frac{\Theta}{110}\right) - 0.13\ \tanh\left(\frac{100}{\Theta}\right) + 0.13\tanh\left(\frac{10}{\Theta^{1.1}}\right)$$