MARINA DEL REY AND BALLONA CREEK FEASIBILITY STUDY

SEDIMENT TRANSPORT ANALYSIS AND REPORT

FINAL SUBMITTAL

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July, 1999

M&N File: 4033-05

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1.0 INTRODUCTION

1.1 Background

In 1996, the Los Angeles District of the U.S. Army Corps of Engineers completed a reconnaissance study to examine problems and potential solutions associated with shoaling and contaminated sediments within the Marina del Rey harbor entrance channels. The preliminary appraisal of costs, benefits, and environmental impacts associated with the potential solutions provided the basis to initiate a feasibility-level study. The purpose of the overall *Marina del Rey and Ballona Creek Feasibility Study* is to develop solutions to both short-term and long-term shoaling problems within the federal channel limits.

The feasibility study is ongoing and is being conducted in phases which include:

Shoaling and Disposal Feasibility Study (draft final report dated February 1998)

This report assessed coastal processes, modeled nearshore circulation and contaminant transport, and investigated subaqueous capping disposal alternatives.

Boat Traffic Impact Assessment (draft final report dated May 1998)

The purpose of this report was to evaluate impacts to boat traffic and navigation safety due to shoaling within the harbor entrance channels. Both recreational boat traffic and emergency response vessel operations were included in the investigation.

Upland Disposal Alternative for Contaminated Sediments (final report dated November 1997)

This report evaluated upland disposal options and opportunities for the contaminated dredge material from the Marina del Rey entrance channel.

Sediment Transport Analysis and Report (this study phase)

This phase of the feasibility study addresses the specific sediment transport aspects associated with the marina entrance channel shoals and evaluates alternative dredge material treatment alternatives.

1.2 Purpose

The purpose of this phase of the *Marina del Rey and Ballona Creek Feasibility Study* was to establish sediment transport patterns within the Marina del Rey harbor area. Primary sediment transport mechanisms addressed in the study include littoral processes and Ballona Creek discharge of storm flows. Alternatives for dredged material treatment were included in the evaluation.

1.3 Study Scope

The study scope included the following:

- *Harbor Shoaling*: Analyze compiled bathymetric and geotechnical data to prepare historic sedimentation rates and shoaling patterns within the harbor navigation channels. Volumes of each generalized sediment size classification were included in the investigation based on existing data.

- *Sedimentation Yield from Ballona Creek*: Utilize existing data to analyze the Ballona Creek watershed potential sediment production and develop a Ballona Creek total annual suspended and bedload sediment yield. Total sediment yield is partitioned into sediment deposited within the harbor navigation channels and sediment discharged to the ocean.

- *Sedimentation Yield from Littoral Transport*: Analyze potential sediment production from littoral transport. The analysis included computation of the sediment budget for the harbor area and development of a total annual sediment yield resulting from littoral transport.

- *Numerical Transport Model*: Employ existing data and a two-dimensional flow model with appropriate sediment transport modules to establish and predict sediment transport rates and sediment depositional patterns within the Marina's navigation channels. Sedimentation rates were predicted based on littoral transport and discharges from Ballona Creek for storm return periods of five (5), twenty-five (25), and one hundred (100) years. Uncertainties of the sediment transport analysis were assessed by risk-based analysis.

- *Dredged Material Treatment Analysis*: Analyze treatment technologies including mechanical separation of fine grained contaminated sediments from coarse grain clean sediments; solidification and stabilization of contaminated sediments with cement and other compounds to produce a structural grade mixture; and dilution of the fine grain contaminated material with clean coarse grain material to produce a material suitable for structural fill. Procedures and cost estimates are included for each treatment alternative. Cost analyses include both first costs and maintenance costs, and are presented in an annualized cost format.

1.4 Report Organization

Section 2.0 quantifies historic entrance channel shoaling patterns and rates. Section 3.0 evaluates the various sources of sediment contribution to formulate the basis of the sediment budget discussed in Section 4.0. The historic sediment budget combined with sediment transport modeling described in Section 5.0 is used to evaluate the efficacy of various measures to reduce the frequency and volume of future maintenance dredging. Section 6.0 presents the results of the dredged material treatment analysis.

2.0 HARBOR ENTRANCE SHOALING

This section quantifies the entrance channel shoaling volumes, patterns and rates based on historic data. Shoal volumes are further quantified by sediment size class within the federal channel limits based on available geotechnical data.

2.1 Historic Shoaling Volumes and Patterns

The objectives of the shoaling volume analysis were to determine the quantities, rates and deposition patterns to formulate the basis for the sediment budget analysis and assist in calibration of the numerical sediment transport model. Eighteen AutoCAD files of hydrographic condition, pre-dredge, and post-dredge bathymetric surveys of Marina del Rey were obtained from the U.S. Army Corps of Engineers for the period between July 1991 and April 1998. Additional surveys that did not cover the entire outer entrance channel area or were not available in digital format were not included in the analysis. The date and type of each survey are list in Table 2.1.

Table 2.1 Marina Del Rey Bathymetric Survey Date And Type

Survey Date	Survey Type
July 1991	Condition
May 1992	Condition
October 1992	Pre-dredge
December 1992	Post-dredge
December 1993	Condition
June 1994	Condition
October 1994	Pre-Dredge
December 1994	Post-dredge
January 1995	Condition
June 1995	Condition
December 1995	Condition
March 1996	Pre-dredge
April 1996	Post-dredge
September 1996	Condition
August 1997	Condition
February 1998	Condition
March 1998	Pre-dredge
April 1998	Post-dredge

Individual pre-dredge surveys were first reviewed to identify shoaling patterns within the entrance channels. Figures 2.1 through 2.4 show bathymetric contour plots for the four pre-dredge surveys identified in Table 2.1. The general trend represented in each of the surveys is a relatively narrow but high tip shoal at the head of the south entrance channel jetty at the mouth of Ballona Creek, and a broader tip shoal at the end of the north jetty. The north jetty tip shoal typically encroaches less distance into the entrance channel. This is also demonstrated by the fact that the available navigable width of the south entrance channel is reduced at a more rapid rate than the north channel entrance.

Sequential combinations of surveys were then examined to determine shoal volumes and patterns within the entrance channel. The entrance channel limits are shown in Figure 2.5. The entrance channel was separated into sub-areas to help quantify the spatial distribution of shoaling rates and patterns. Area A and Area B cover the south and north entrance channel, respectively. Area G represents the advanced maintenance dredging area at the mouth of Ballona Creek. Area H is the north jetty fillet which is used as a sand trap for advanced maintenance dredging at the north entrance.

Shoal volumes were calculated using AutoCAD by overlaying the successive pairs of surveys and calculating relative changes in bottom elevation. Table 2.2 summarizes the shoal volume change by sub-area. Contour plots showing accretion (shoaling) and erosion are shown in Figures 2.6 through 2.22.

Note that the sand trap at the north jetty fillet (Area G) has only been surveyed (and dredged) during the more recent years (since October 1994) of the study period. The table also indicates that not all condition surveys included this area.

Some illustrative examples of the shoal change analyses include the following:

Table 2.2 **Shoal Volume Change By Sub-Area (1991-1998)**

Time Period & Survey Types	Shoal Volume Change by Sub-Area (Cubic Meters)				Total
	Area A	Area B	Area G	Area H	
Jul 1991 – May 1992 Cond. – Cond.	20,483	11,031	50,548		82,062
May 1992 – Oct 1992 Cond. – Pre.	-3,391	-1,967	-1,751		-7,110
Oct 1992 – Dec 1992 Pre. – Post	-20,027	3,724	26,955		10,652
Dec 1992 – Dec 1993 Post – Cond.	26,297	18,785	-13,748		31,334
Dec 1993 – Jun 1994 Cond. – Cond.	1,005	1,504	4,353		6,862
Jun 1994 – Oct 1994 Cond. – Pre.	-9,943	-3,629	-12,132		-25,704
Oct 1994 – Dec 1994 Pre. – Post	-8,806	-14,037	9,041	1,162	-12,640
Dec 1994 – Jan 1995 Post – Cond.	23,569	11,987	1,260	25,153	61,969
Jan 1995 – Jun 1995 Cond. – Cond.	19,291	17,490	7,102	3,820	47,703
Jun 1995 – Dec 1995 Cond. – Cond.	-8,103	-4,983	-6,640		-19,526
Dec 1995 – Mar 1996 Cond. – Pre.	14,071	18,405	1,862		34,338
Mar 1996 – Apr 1996 Pre. – Post	-37,132	-48,910	-3,703	-120,628	-210,373
Apr 1996 – Sep 1996 Post – Cond.	4,909	4,851	-1,208	5,580	14,131
Sep 1996 – Aug 1997 Cond. – Cond.	8,065	9,626	2,515	38,853	59,058
Aug 1997 – Feb 1998 Cond. – Cond.	18,554	25,750	1,591	51,732	97,627
Feb 1998 – Mar 1998 Cond. – Pre.	6,219	1,952	2,469		10,640
Mar 1998 – Apr 1998 Pre. – Post	2,359	-65,108	-1,307	-27,459	-87,915

1. Figure 2.6 shows shoal change contours between the July 1991 and May 1992 condition surveys, and illustrates the typical shoaling pattern within the south entrance channel and

vicinity. Figure 2.20 also illustrates typical shoaling of both the south and north entrance channels.

2. Figure 2.8 provides a clear illustration of the local shoal knockdown that was done between the period October 1992 and December 1992.

3. Figures 2.12, 2.17 and 2.22 show the spatial shoal reduction from dredging.

4. Figure 2.13 illustrates the rapid return of the shoal during the month following the December 1994 post-dredge survey.

5. Figure 2.18 shows a relatively stable entrance depth during the summer months following the March-April 1996 dredging episode. Figure 2.19 illustrates the significant shoaling of the north entrance and sand trap during the ensuing winter.

2.2 Historical Shoaling Rates

Calculation of historical shoaling rates for future dredge planning purposes should cover as many years as possible to account for both annual and seasonal variability. For example, shoaling due to littoral transport will exhibit a moderate range of seasonal and annual variability relative to Ballona Creek sedimentation which is highly dependent on season and demonstrates wide annual variations. Longer data records will therefore tend to average these variations and provide more useful planning estimates.

Historical shoaling rates were analyzed by two different methods. The first method consisted of calculating the volume change within the entrance channel between the first and last available bathymetric surveys (July 1991 to April 1998), and adding the cumulative amount of dredge volume removed during the period since dredging represents additional shoaling that occurred over that estimated by the survey data. Historical dredging volumes for the period of analysis are shown in Table 2.3. Since the 1992 dredging event consisted of material spreading only, no material was removed from the entrance channel and the shoal volume calculated for this period was not corrected for dredging.

Table 2.3 Dredging/Disposal History At Marina Del Rey (1991-1998)

Start Date	End Date	Location	Quantity (M^3)	Disposal Location
10/16/92	11/15/92	Ballona Creek mouth	16,440	Local (knockdown)
11/8/94	12/18/94	Entrance Channel	42,060	POLA shallow water habitat CAD site
3/13/96	4/14/96	Entrance Channel	202,970	Dockweiler Beach
3/98	4/98	Entrance Channel	87,100	Dockweiler Beach, LA-2
		Total	**332,130**	

Table 2.4 provides a comparison of the calculated shoal volume changes over a dredge episode, i.e. pre- to post-dredge surveys in Table 2.2, with the dredged pay quantities (Table 2.3). The 1992 dredge episode is not included since this was a local knockdown with no material removed.

Table 2.4 Comparison Of Calculated Shoal Volume Reduction With Dredged Pay Quantities

Dredge Period	Pay Quantity Cubic Meters	Calculated Volume Change Cubic Meters	Percent Difference
Oct '92 – Dec '92	16,440	Area A reduced 20,027 Area G increased 26,955	N.A.
Oct '94 – Dec ' 94	42,060	12,640	70
Mar '96 – Apr ' 96	202,970	210,373	4
Mar '98 – Apr ' 98	87,100	87,915	1

The results indicate excellent comparison for the latter two dredge episodes. One likely reason is that the dredging occurred during the spring months when little precipitation occurs. Regarding the 1994 dredge event, anecdotal evidence indicates that significant shoaling in the south entrance occurred during the dredging operation which was during winter. The 1992 knockdown also occurred during a relatively wet winter season that would account for a relatively significant amount of shoaling to occur during the dredging operations.

Coming back to the shoaling rate analysis, the net volume change for shoal material in the entrance channel between July 1991 and April 1998 is tabulated in Table 2.5 and illustrated graphically in Figure 2.23.

Table 2.5 Shoal Volume Change (July 1991 – April 1998)

Location	Shoal volume change (cubic meters)
AREA A	+57,419
AREA B	-9,929
AREA G	+67,406
AREA H	-21,787
TOTAL	**+93,109**

The total amount of volume change within the entrance channel (Area A and Area B) and advanced maintenance dredging areas (Area G and Area H) based on hydrographic surveys during July 1991 to April 1998 was calculated to be an increase of 93,109 cubic meters. Adding the cumulative dredge volume of 332,130 cubic meters for the same period results in a total shoal accumulation of 425,239 cubic meters. The resulting average annual shoaling rate for the 6.75-year period is approximately 63,000 cubic meters per year.

A second method was used to estimate the average annual shoaling rate. Volume changes were calculated between sequential bathymetric surveys for periods during which natural processes of shoaling and erosion occurred, i.e. not for periods which included dredging. The shoal volume was then divided by the time period in years (or fraction thereof) between surveys to give an annual shoaling rate for that period. Annual shoaling rates were then averaged to provide an average annual shoaling rate. Note that a sufficient number of surveys is desirable to smooth seasonal and annual variations. The advantage of this calculation method is the spatial variation of shoaling within the entrance can be estimated. This is possible because the calculation does not rely on the use of past dredge volumes which are not sub-area specific. It can also provide information on seasonal variability of shoaling rates. The results of this analysis are summarized in Table 2.6

Table 2.6 Calculation Of Annual Shoaling Rate By Sub-Area (July 1991 – April 1998)

TIME PERIOD	DURA-TION (yrs)	AREA A		AREA B		AREA G		AREA H	
		Vol. (m^3)	Rate (m^3/yr)	Vol. (m^3)	Rate (m^3/yr)	Vol. (m^3)	Rate (m^3/yr)	Vol. (m^3)	Rate (m^3/yr)
Jul 91 – May 92	0.8	20,483	24,580	11,031	13,237	50,548	60,658		
May 92 – Oct 92	0.4	-3,391	-8,138	-1,967	-4,721	-1,751	-4,202		
Dec 92 – Dec 93	1.0	26,297	26,297	18,785	18,785	-13,748	-13,748		
Dec 93 – Jun 94	0.5	1,005	2,010	1,504	3,008	4,353	8,706		
Jun 94 – Oct 94	0.3	-9,943	-29,829	-3,629	-10,887	-12,132	-36,396		
Dec 94 – Jan 95	0.1	23,569	282,828	11,987	143,844	1,260	15,120	25,153	301,836
Jan 95 – Jun 95	0.4	19,291	46,298	17,490	41,976	7,102	17,045	3,820	3,274
Jun 95 – Dec 95	0.5	-8,103	-16,206	-4,983	-9,966	-6,440	-12,880		
Dec 95 – Mar 96	0.3	14,071	56,284	18,405	73,620	1,862	7,448		
Apr 96 – Sep 96	0.4	4,909	11,782	4,851	11,642	-1,208	-2,899	5,580	13,392
Sep 96 – Aug 97	0.9	8,065	8,798	9,626	10,501	2,515	2,744	38,853	42,385
Aug 97 – Feb 98	0.5	18,554	37,108	25,750	51,500	1,591	3,182	51,732	88,683
Feb 98 – Mar 98	0.1	6,219	74,628	1,952	23,424	2,469	29,628		
TOTALS:	6.3	121,026		110,802		36,421		125,138	
AVERAGE:			**+19,364**		**+17,728**		**+5,827**		**+39,517**

The combined annual shoaling rate for the entrance channel and advanced maintenance dredging areas is approximately 82,400 cubic meters per year based on the results presented in Table 2.6. This exceeds the preceding estimate of 63,000 cubic meters per year by about 30 percent. It is anticipated that the 63,000 cubic yards per year is more reliable for the overall shoaling rate since it tends to smooth the more episodic events. However, the method described in Table 2.6 provides information of the spatial distribution of shoaling which is critical, since the south entrance shoals (Area A and Area G) tend to include contaminants and pose significantly greater disposal problems.

Table 2.6 shows the combined south entrance shoaling rate (Area A and Area G) is approximately 25,000 cubic meters per year. This estimate is considered reliable since it is based on a relatively long term set of bathymetric survey data. The Area B estimate of approximately 18,000 cubic meters per year is also considered reliable for the same reason. The Area H shoal rate was based on a more limited set of bathymetric data for recent years only which, based on recent dredging frequency in the north entrance area, has been higher than average. Assuming that the more accurate estimate of the average annual shoaling rate for the total entrance area is 63,000 cubic meters per year, the contribution of Area H can be estimated by subtracting the combined Area A, Area B and Area G rates from the total rate as follows:

Total Shoaling Rate – (Area A + Area G) Rate – Area B Rate = Area H Rate

63,000 – 25,000 – 18,000 = 20,000 cubic meters per year

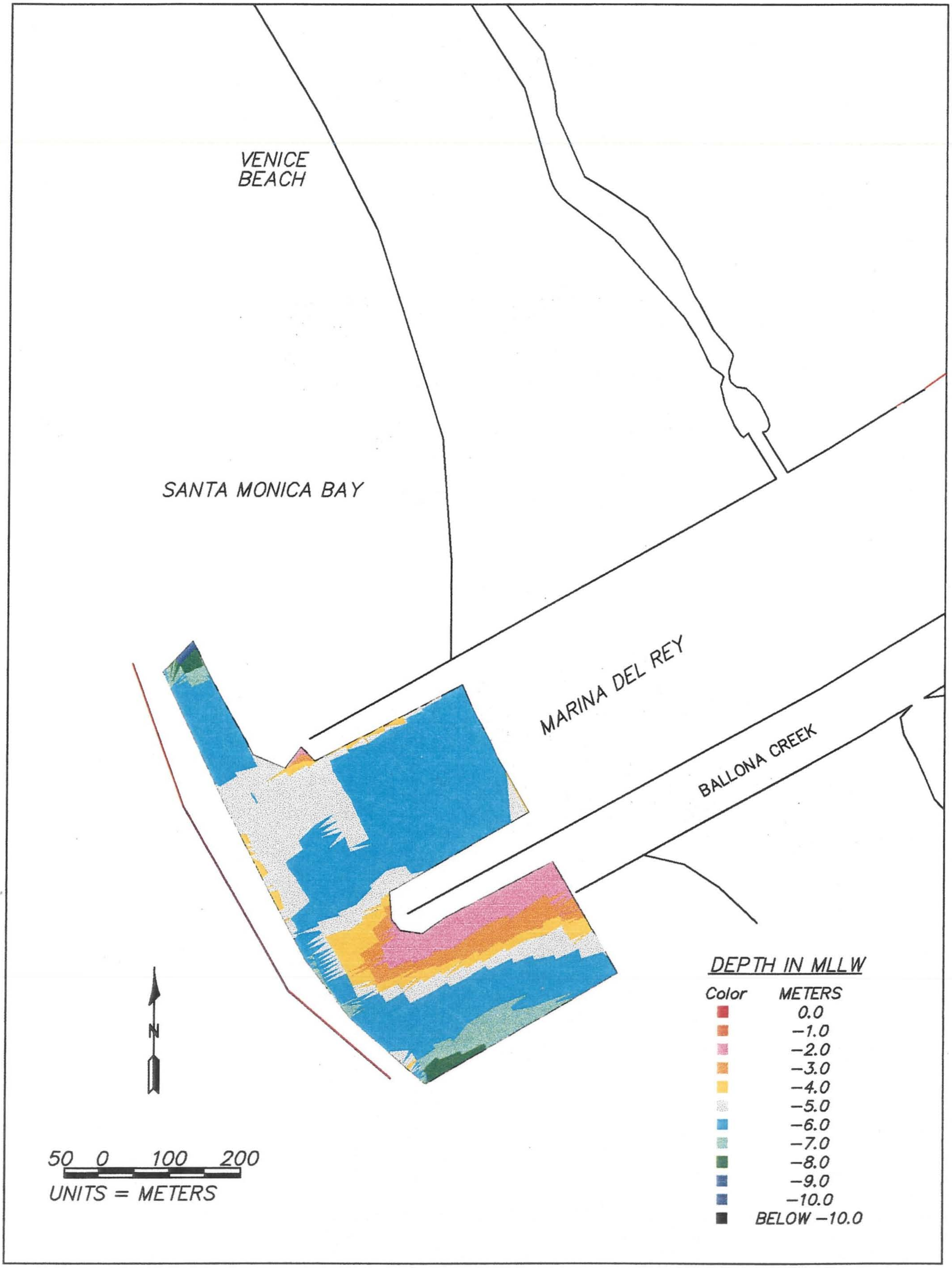
This information is used as the basis for development of a sediment budget for the entrance channel area as described in Section 4.0.

2.3 Sediment Size Distribution

Recent sediment sampling data were reviewed to determine the sediment size distribution within the harbor entrance shoals (Advanced Biological Testing; 1995, 1996). Sampling locations are shown in Figure 2.24. Table 2.7 summarizes the percent distribution by weight for the four general grain size classifications. The results indicate the vast majority of shoal sediments are composed of sand (86%).

Table 2.7 Marina Del Rey Shoal – Grain Size Distribution (Based On Testing Conducted By Advanced Biological Testing (1995 And 1996)

			Percent Composition (%)				Weighted Percent Composition (%)			
Station	Sample	Length (ft)	Gravel	Sand	Silt	Clay	Gravel	Sand	Silt	Clay
VCH95-1	950921-14A	3.0	0.2	97.6	0.7	1.5	0.0	2.9	0.0	0.0
VCH95-1	950921-15A	3.0	1.1	95.9	1.2	1.8	0.0	2.9	0.0	0.1
VCH95-2	950921-10A	3.0	0.5	86.5	10.4	2.6	0.0	2.6	0.3	0.1
VCH95-2	950921-10B	3.0	0.2	88.6	7.7	3.4	0.0	2.7	0.2	0.1
VCH95-2	950921-11A	1.2	1.4	91.3	4.3	3.0	0.0	1.1	0.1	0.0
VCH95-3	950921-9A	6.8	0.9	95.5	1.5	2.1	0.1	6.5	0.1	0.1
VCH95-4	950921-12A	3.0	1.0	95.1	1.4	2.5	0.0	2.9	0.0	0.1
VCH95-4	950921-13A	3.0	1.7	88.6	6.2	3.5	0.1	2.7	0.2	0.1
VCH95-5	950920-9A	3.0	1.3	53.7	34.0	11.0	0.0	1.6	1.0	0.3
VCH95-5	950920-9B	3.0	1.2	59.4	27.2	12.2	0.0	1.8	0.8	0.4
VCH95-5	950920-10A	0.5	0.2	66.0	26.3	7.5	0.0	0.3	0.1	0.0
VCH95-6	950921-4A	2.0	2.1	95.3	0.9	1.7	0.0	1.9	0.0	0.0
VCH95-6	950921-5A	2.0	5.7	91.2	1.2	1.9	0.1	1.8	0.0	0.0
VCH95-6	950921-6A	2.0	0.8	85.6	8.7	4.9	0.0	1.7	0.2	0.1
VCH95-6	950921-7A	2.0	1.4	79.5	12.9	6.2	0.0	1.6	0.3	0.1
VCH95-7	950920-7A	3.0	0.2	96.6	1.7	1.5	0.0	2.9	0.1	0.0
VCH95-7	950920-7B	3.0	0.0	97.5	1.0	1.5	0.0	2.9	0.0	0.0
VCH95-7	950920-8A	1.0	0.2	85.2	10.1	4.5	0.0	0.9	0.1	0.0
VCH95-8	950920-5A	3.0	0.2	96.9	1.3	1.6	0.0	2.9	0.0	0.0
VCH95-8	950920-5B	3.0	0.5	88.7	7.3	3.5	0.0	2.7	0.2	0.1
VCH95-8	950920-6A	1.0	2.7	94.6	1.2	1.5	0.0	0.9	0.0	0.0
VCH95-9	950921-16A	5.0	4.0	83.7	6.9	5.4	0.2	4.2	0.3	0.3
VCH95-10	950920-4A	3.0	1.0	94.0	3.1	1.9	0.0	2.8	0.1	0.1
VCH95-10	950920-4B	2.0	2.1	90.7	4.0	3.2	0.0	1.8	0.1	0.1
VCH95-11	950920-3A	3.0	0.3	68.3	23.4	8.0	0.0	2.0	0.7	0.2
VCH95-11	950920-3B	2.9	0.2	68.5	23.1	8.2	0.0	2.0	0.7	0.2
VCH95-12	950921-1A	2.0	0.2	93.4	3.2	3.2	0.0	1.9	0.1	0.1
VCH95-12	950921-2A	3.0	0.8	79.1	13.8	6.3	0.0	2.4	0.4	0.2
VCH95-12	950921-3A	2.0	1.5	81.2	10.0	7.3	0.0	1.6	0.2	0.1
VCH95-13	950920-1A	3.0	0.1	94.5	2.8	2.6	0.0	2.8	0.1	0.1
VCH95-13	950920-1B	3.0	0.2	86.3	9.6	3.9	0.0	2.6	0.3	0.1
VCH95-13	950920-2A	3.1	0.3	67.4	22.9	9.4	0.0	2.1	0.7	0.3
VCH95-14		8.0	5.5	86.6	3.8	4.1	0.4	6.9	0.3	0.3
VCH95-16		5.6	0.1	97.0	1.5	1.4	0.0	5.4	0.1	0.1
	Totals	100.1					1.4	86.6	7.9	4.1



MARINA DEL REY SEDIMENT STUDY	BATHYMETRY OCTOBER 1992	Figure 2.1

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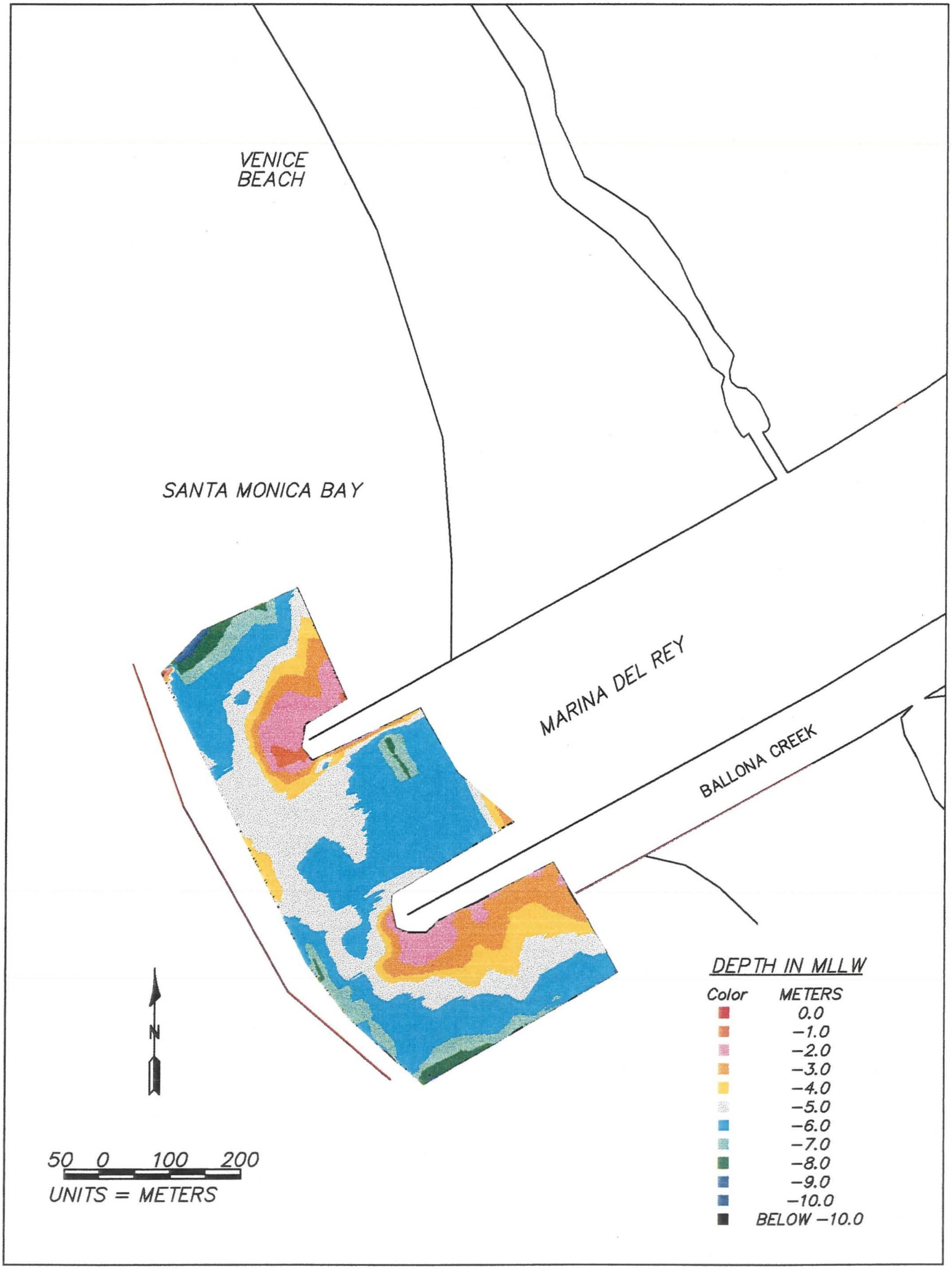

MARINA DEL REY SEDIMENT STUDY

BATHYMETRY OCTOBER 1994

Figure 2.2

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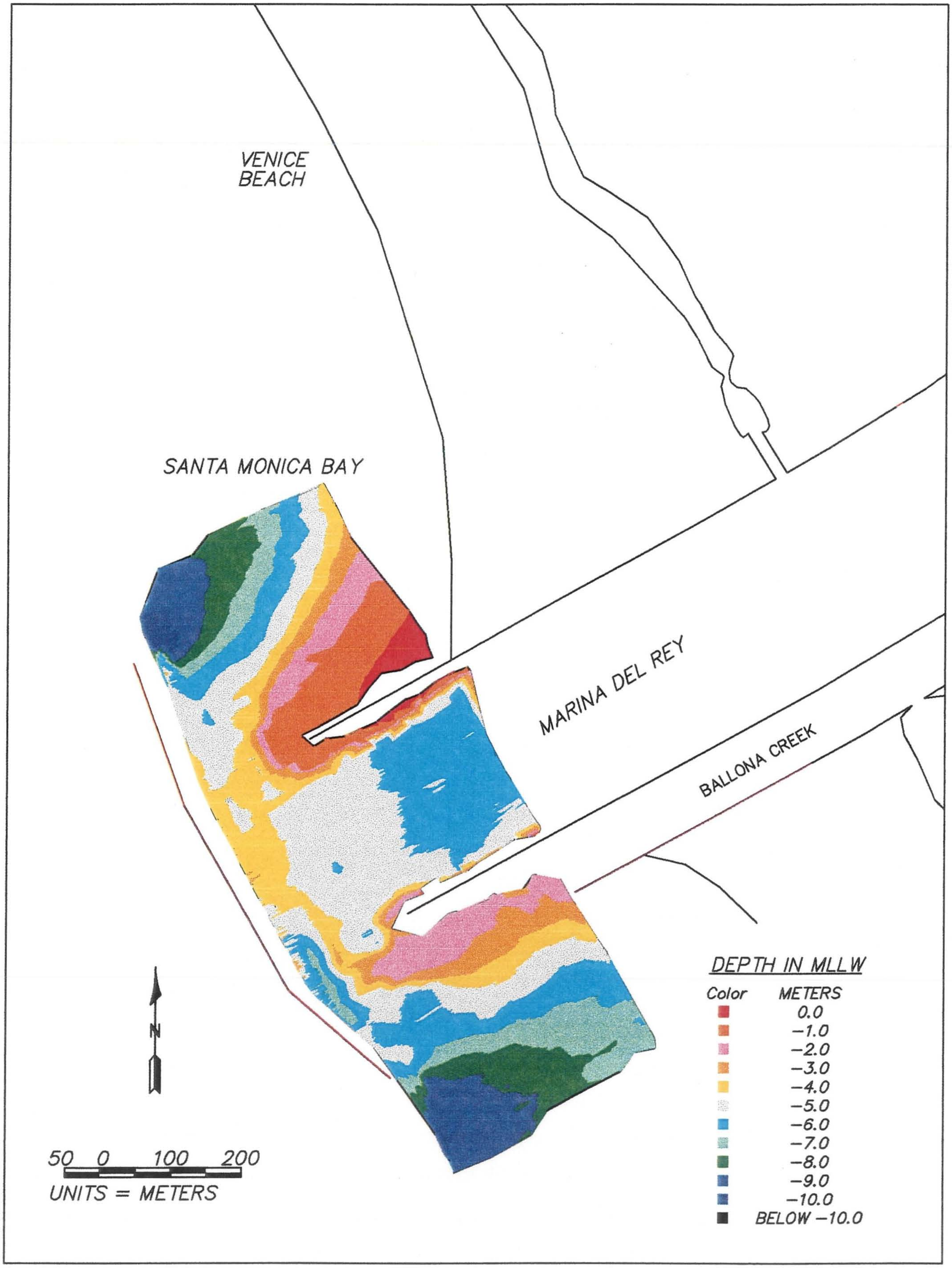

MARINA DEL REY SEDIMENT STUDY	BATHYMETRY MARCH 1996	Figure 2.3

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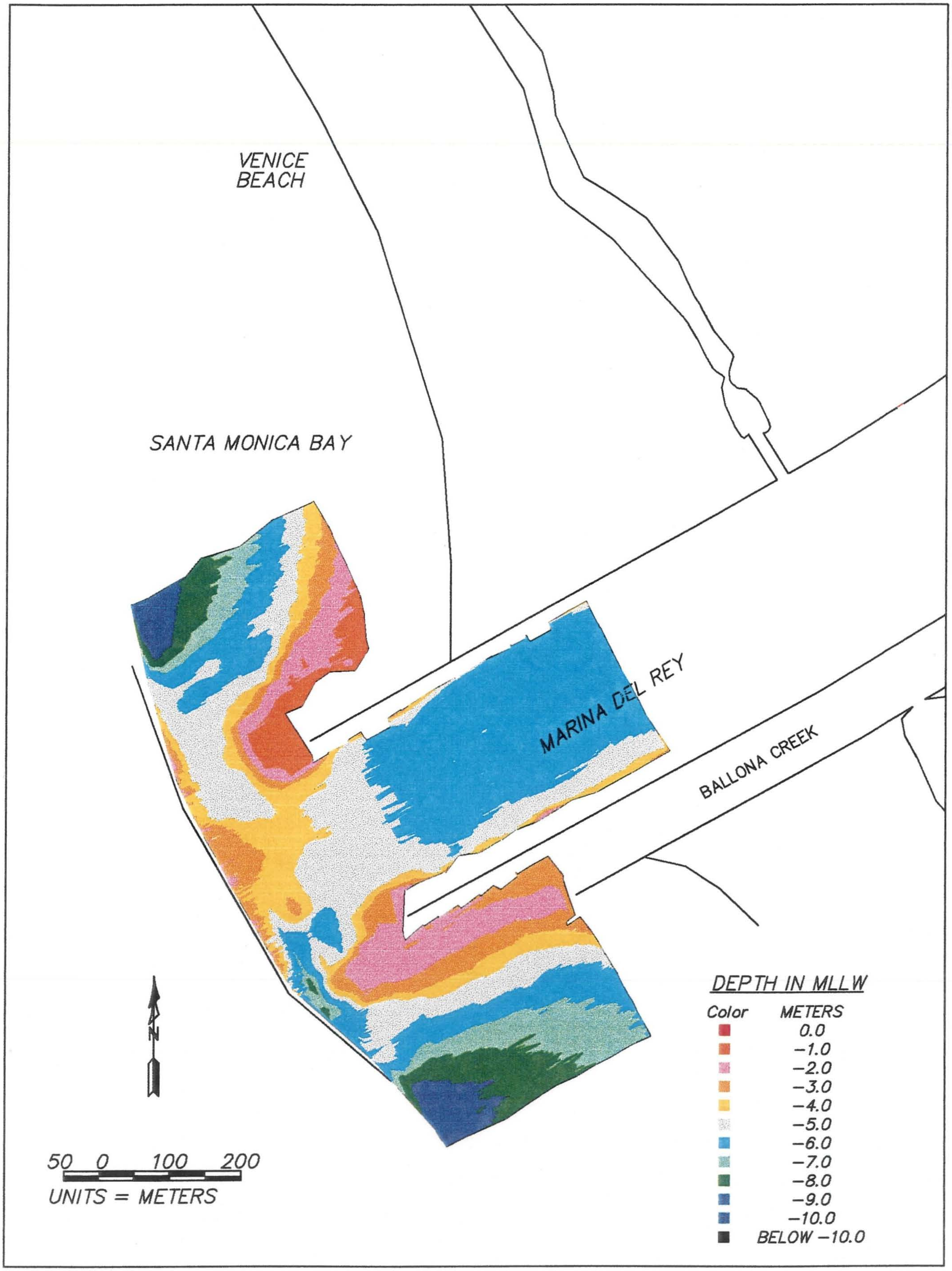

MARINA DEL REY SEDIMENT STUDY	BATHYMETRY MARCH 1998	Figure 2.4

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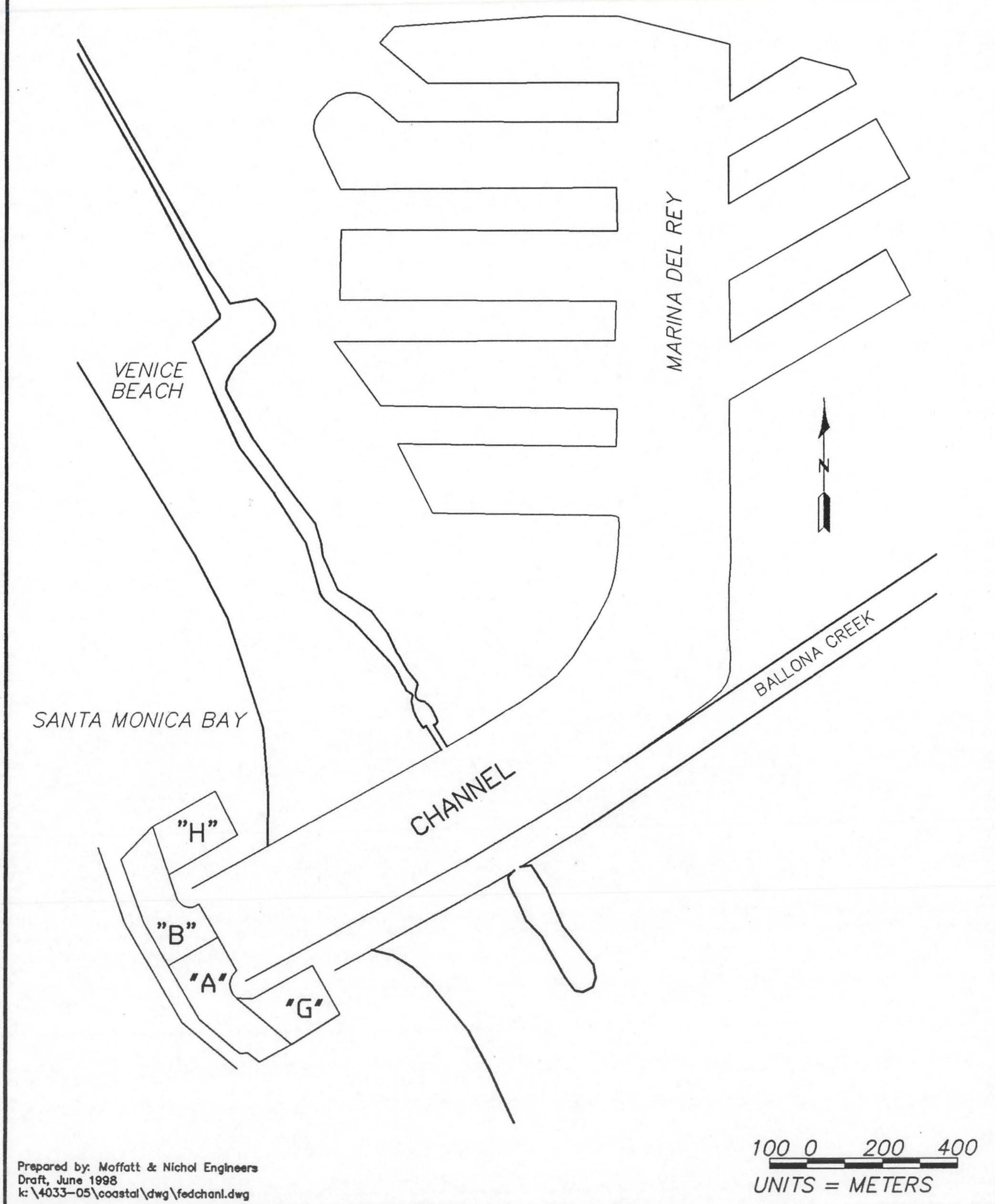

Prepared by: Moffatt & Nichol Engineers
Draft, June 1998
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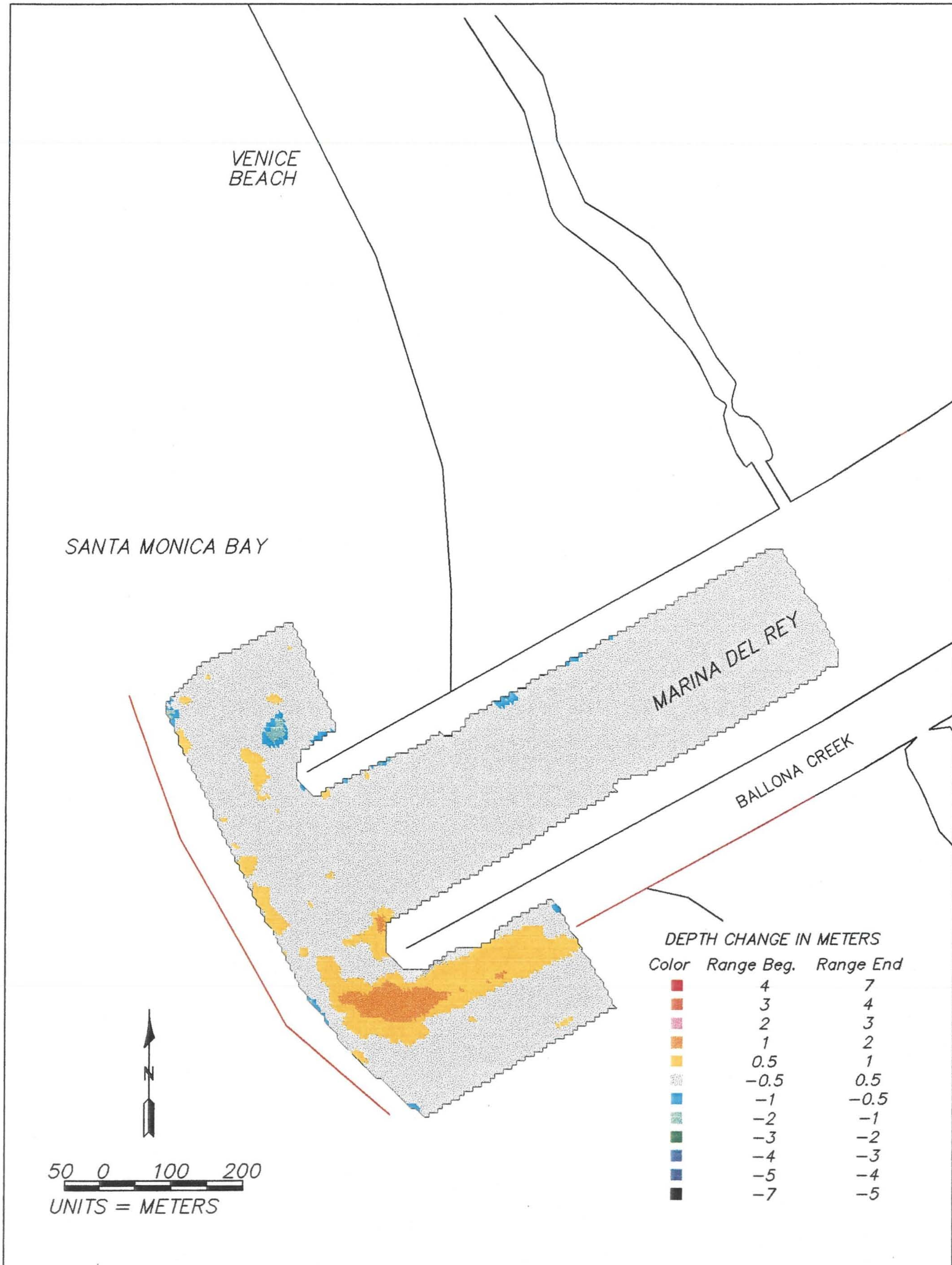
MARINA DEL REY SEDIMENT STUDY	FEDERAL CHANNEL BOUNDARIES	Figure 2.5



MARINA DEL REY SEDIMENT STUDY	BATHYMETRY CHANGE JULY'91 COND TO MAY'92 COND	Figure 2.6

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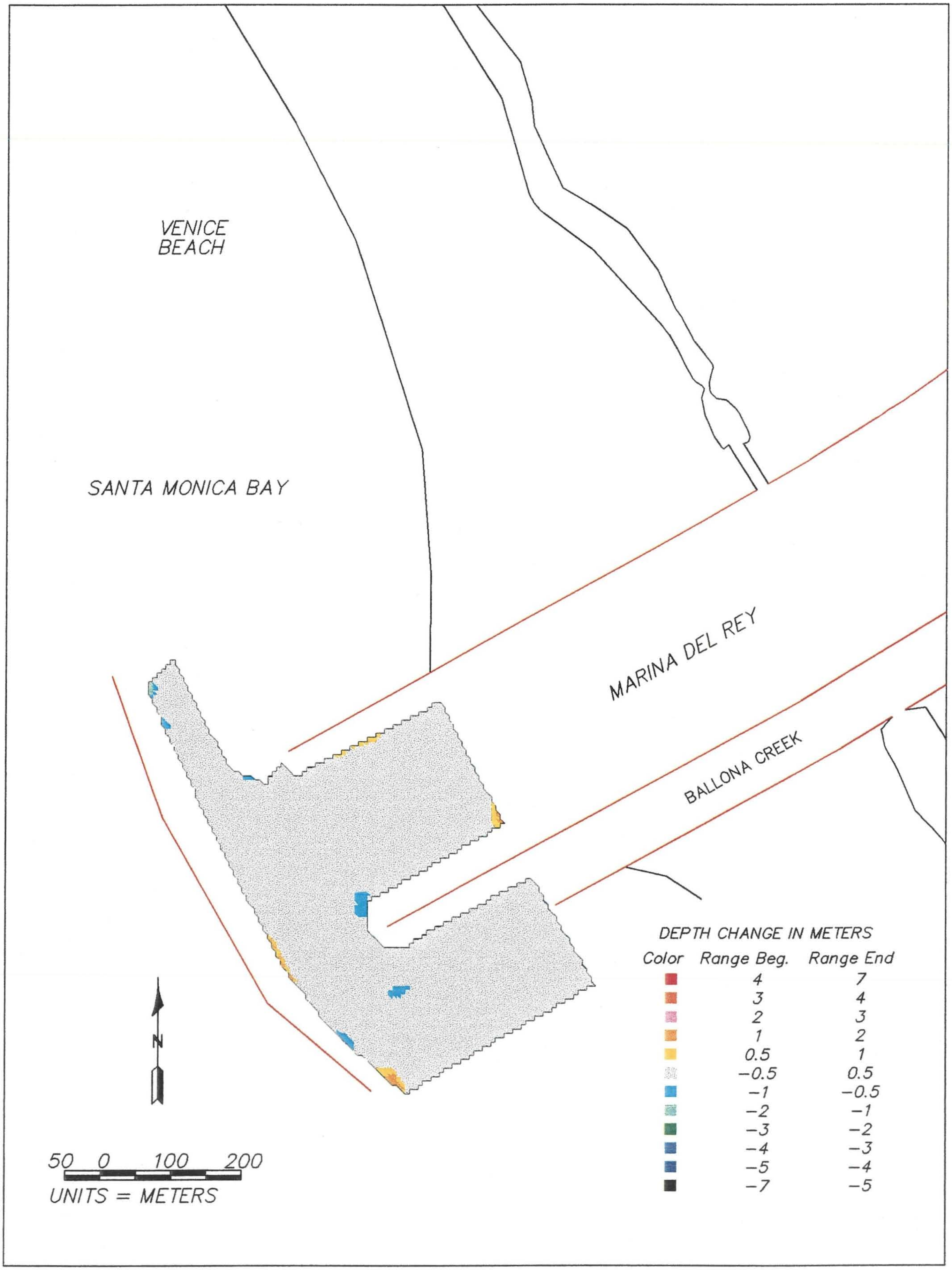

MARINA DEL REY FEASIBILITY STUDY	BATHYMETRY CHANGE MAY'92 COND TO OCTOBER'92 PRE	Figure 2.7

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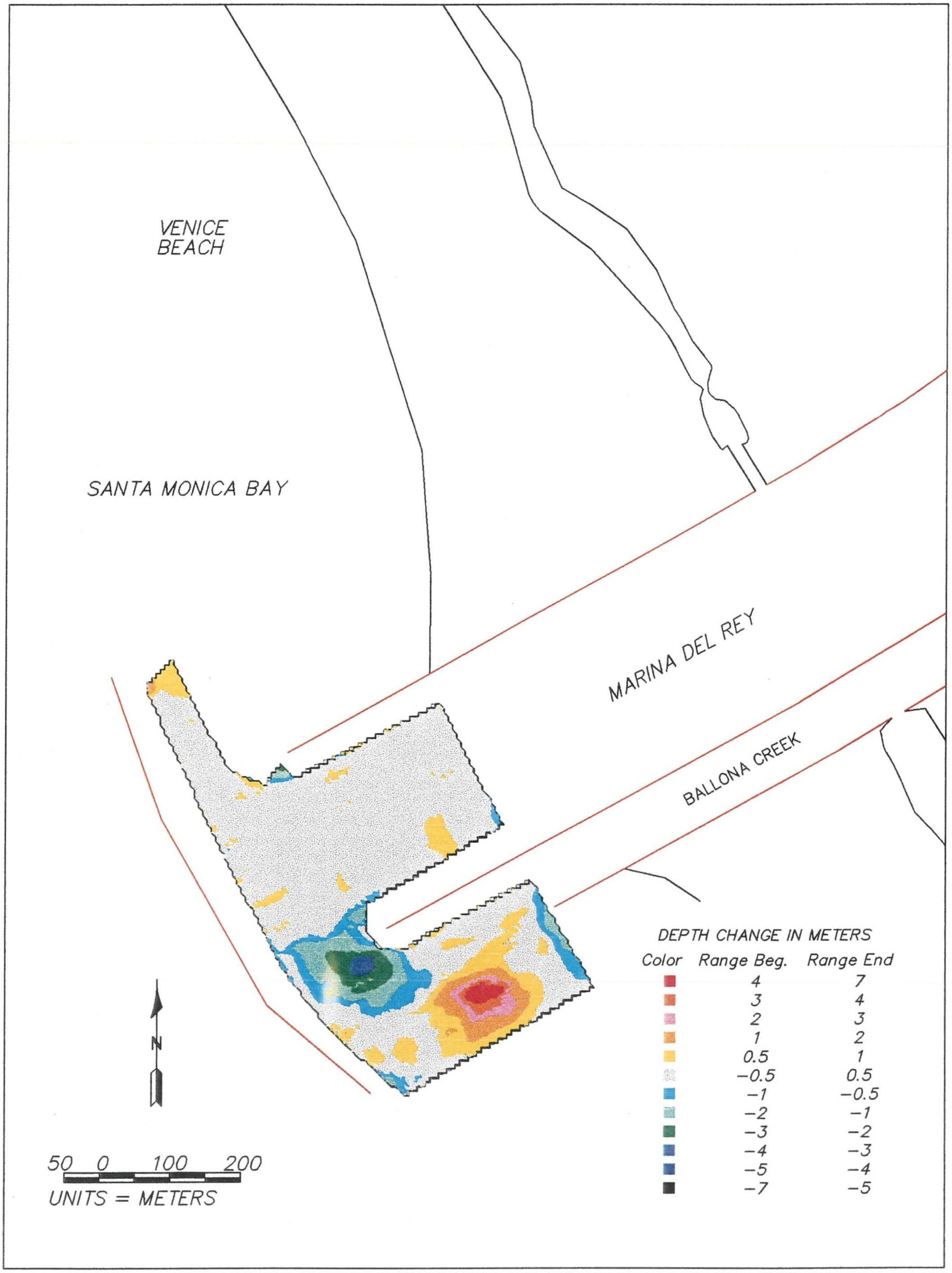

MARINA DEL REY FEASIBILITY STUDY	BATHYMETRY CHANGE OCTOBER'92 PRE TO DECEMBER'92 POST	Figure 2.8

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MARINA DEL REY FEASIBILITY STUDY	BATHYMETRY CHANGE DECEMBER'92 POST TO DECEMBER'93 COND	Figure 2.9

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MARINA DEL REY SEDIMENT STUDY	BATHYMETRY CHANGE DECEMBER'93 COND TO JUNE'94 COND	Figure 2.10

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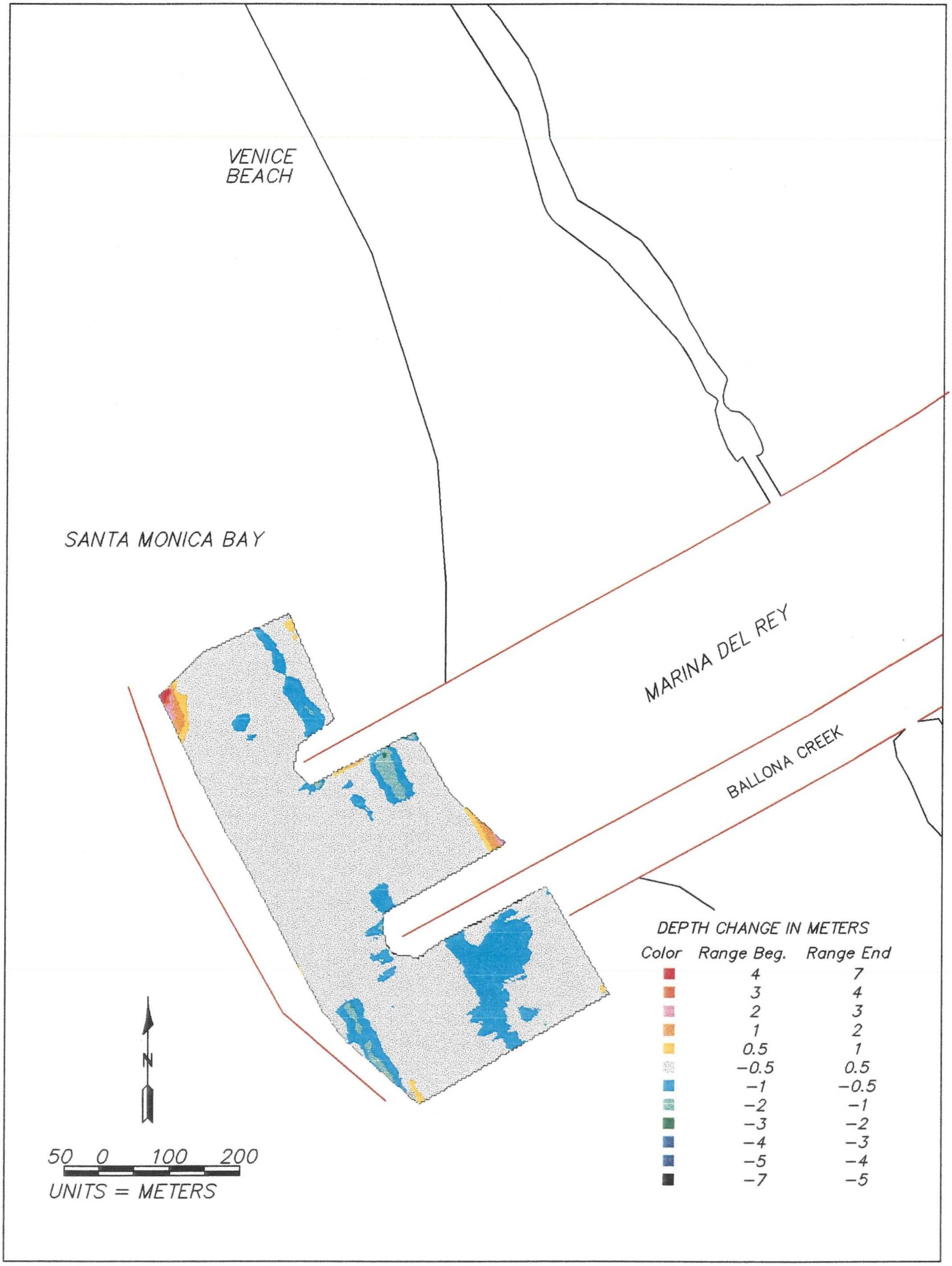

MARINA DEL REY FEASIBILITY STUDY

BATHYMETRY CHANGE
JUNE'94 COND TO OCTOBER'94 PRE

Figure 2.11

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MARINA DEL REY FEASIBILITY STUDY

BATHYMETRY CHANGE
OCTOBER'94 PRE TO DECEMBER'94 POST

Figure 2.12

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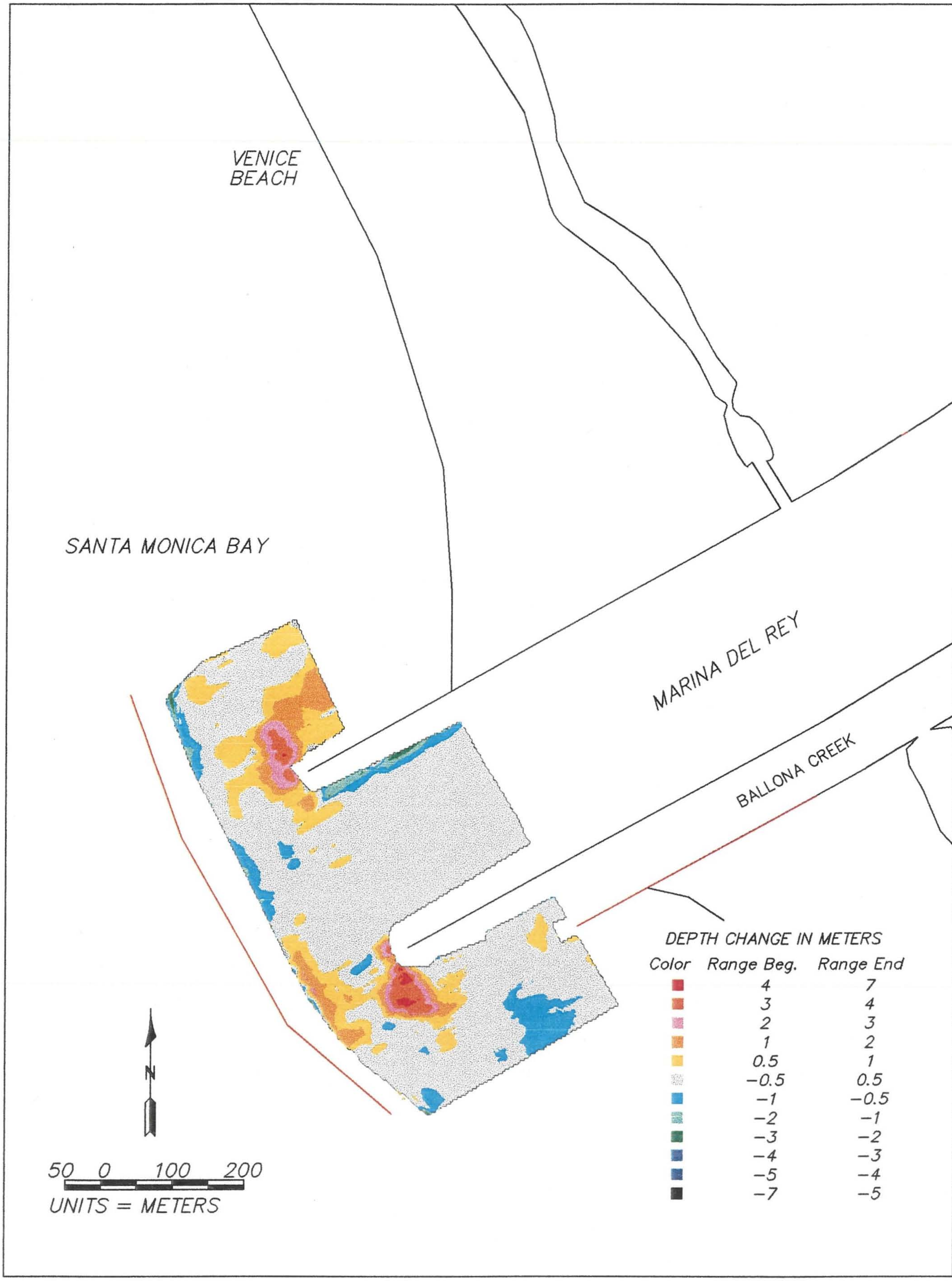

MARINA DEL REY SEDIMENT STUDY

BATHYMETRY CHANGE

DECEMBER'94 POST TO JANUARY'95 COND

Figure 2.13

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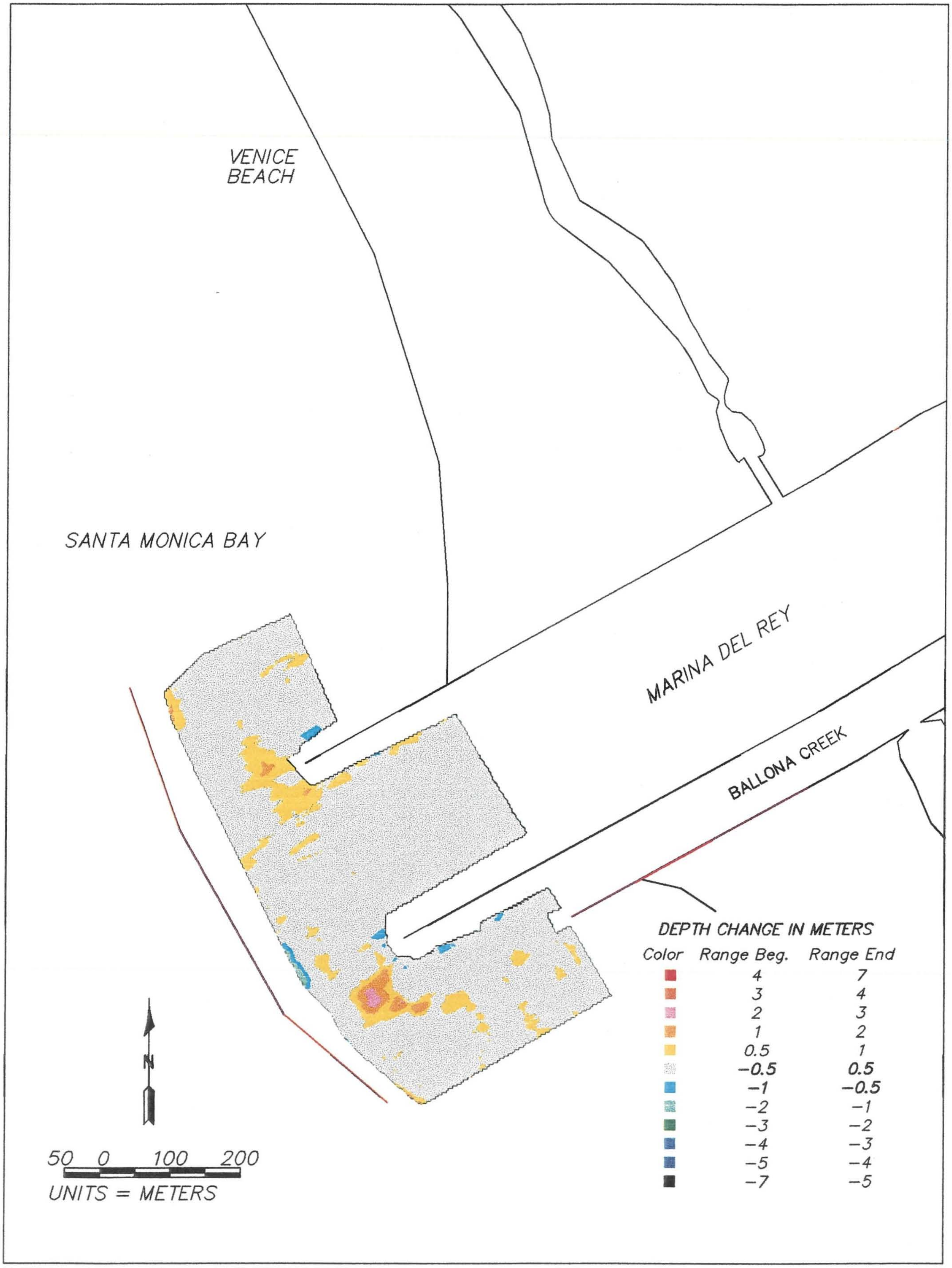

MARINA DEL REY SEDIMENT STUDY	BATHYMETRY CHANGE JANUARY'95 COND TO JUNE'95 COND	Figure 2.14

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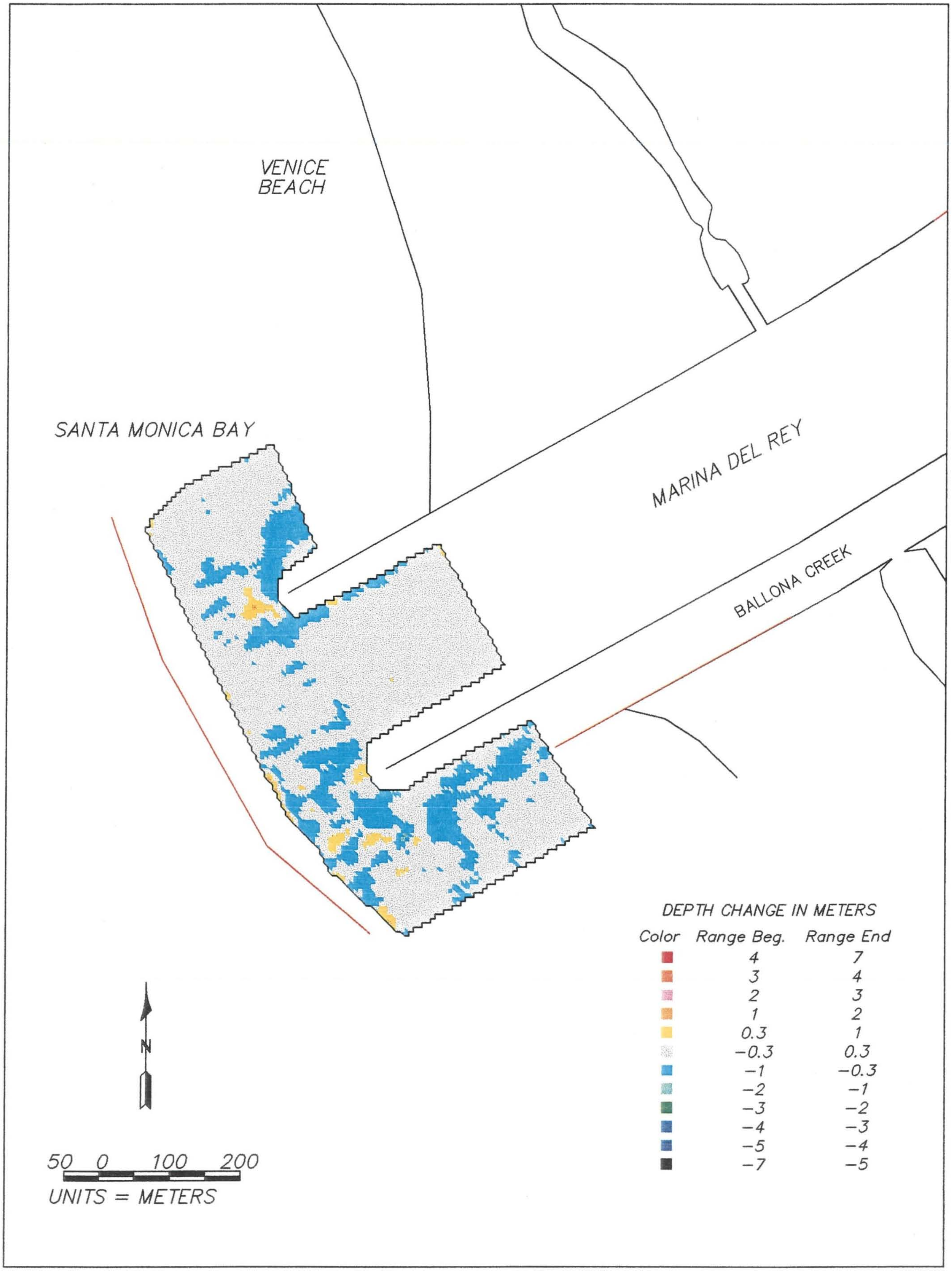

MARINA DEL REY SEDIMENT STUDY	BATHYMETRY CHANGE JUNE'95 COND TO DECEMBER'95 COND	Figure 2.15

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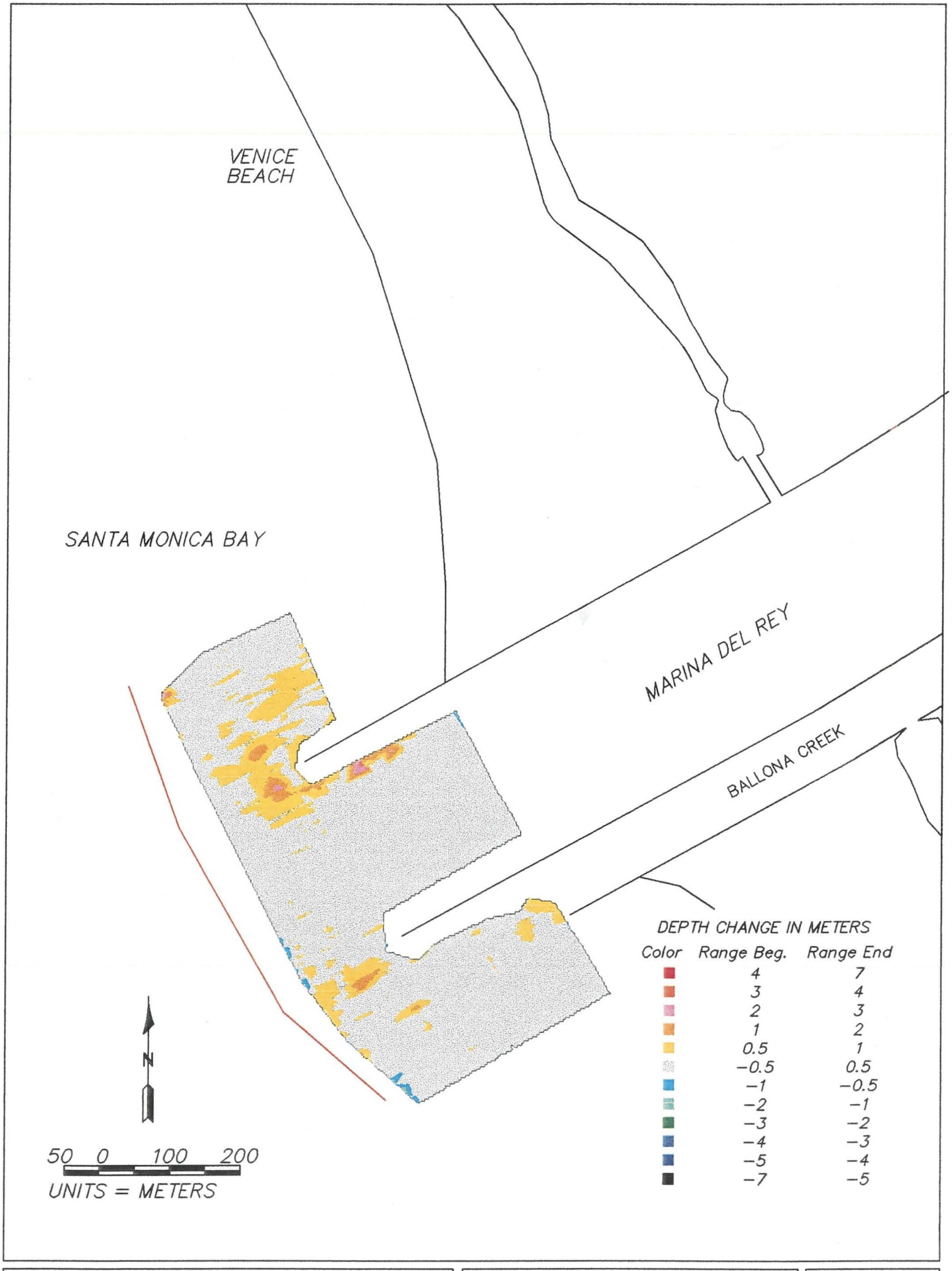

MARINA DEL REY SEDIMENT STUDY	BATHYMETRY CHANGE DECEMBER'95 COND TO MARCH'96 PRE	Figure 2.16

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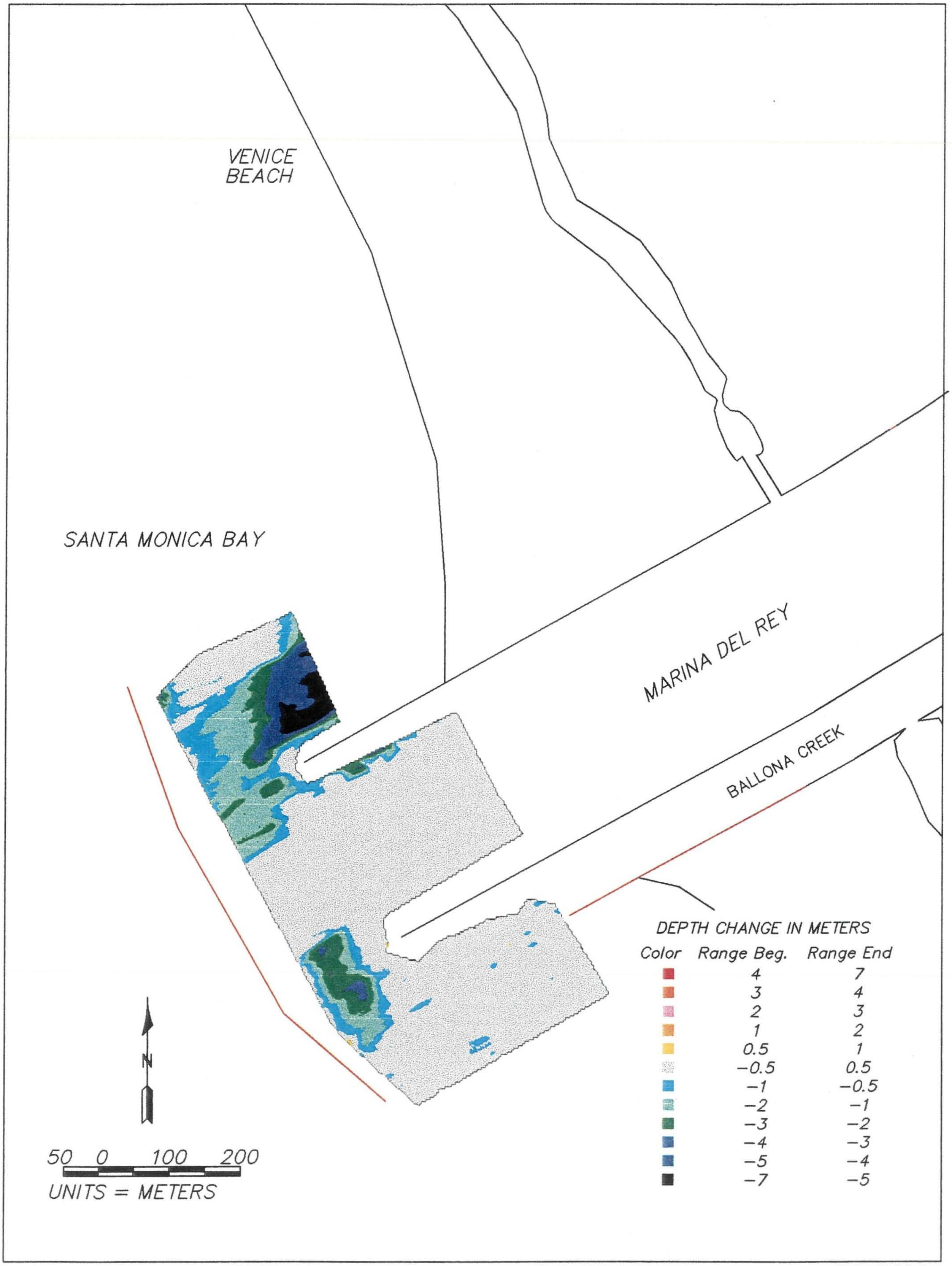

MARINA DEL REY SEDIMENT STUDY	BATHYMETRY CHANGE MARCH'96 PRE TO APRIL'96 POST	Figure 2.17

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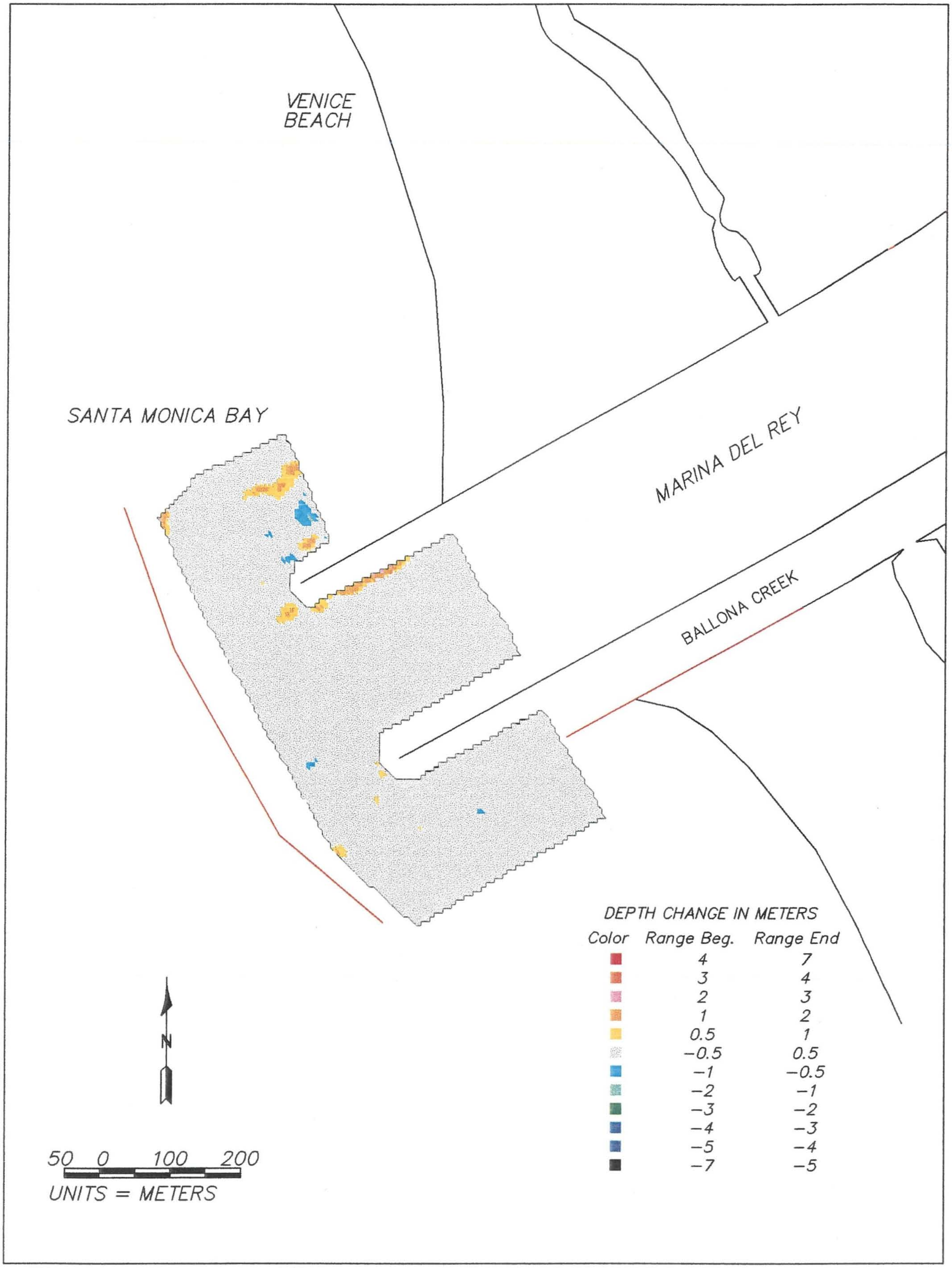

MARINA DEL REY SEDIMENT STUDY	BATHYMETRY CHANGE APRIL'96 POST TO SEPTEMBER'96 COND	Figure 2.18

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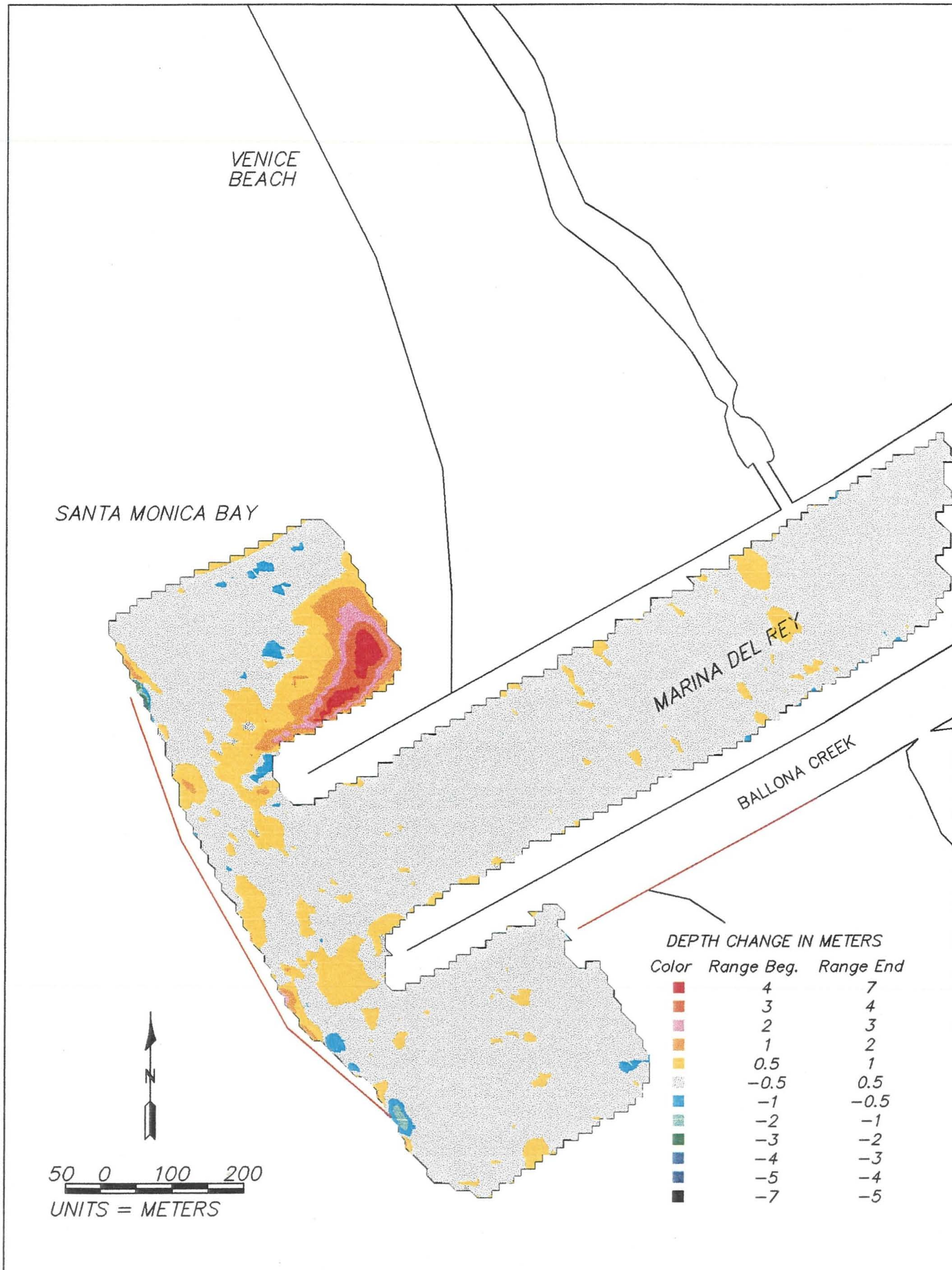

MARINA DEL REY SEDIMENT STUDY	BATHYMETRY CHANGE SEPTEMBER'96 COND TO AUGUST'97 COND	Figure 2.19

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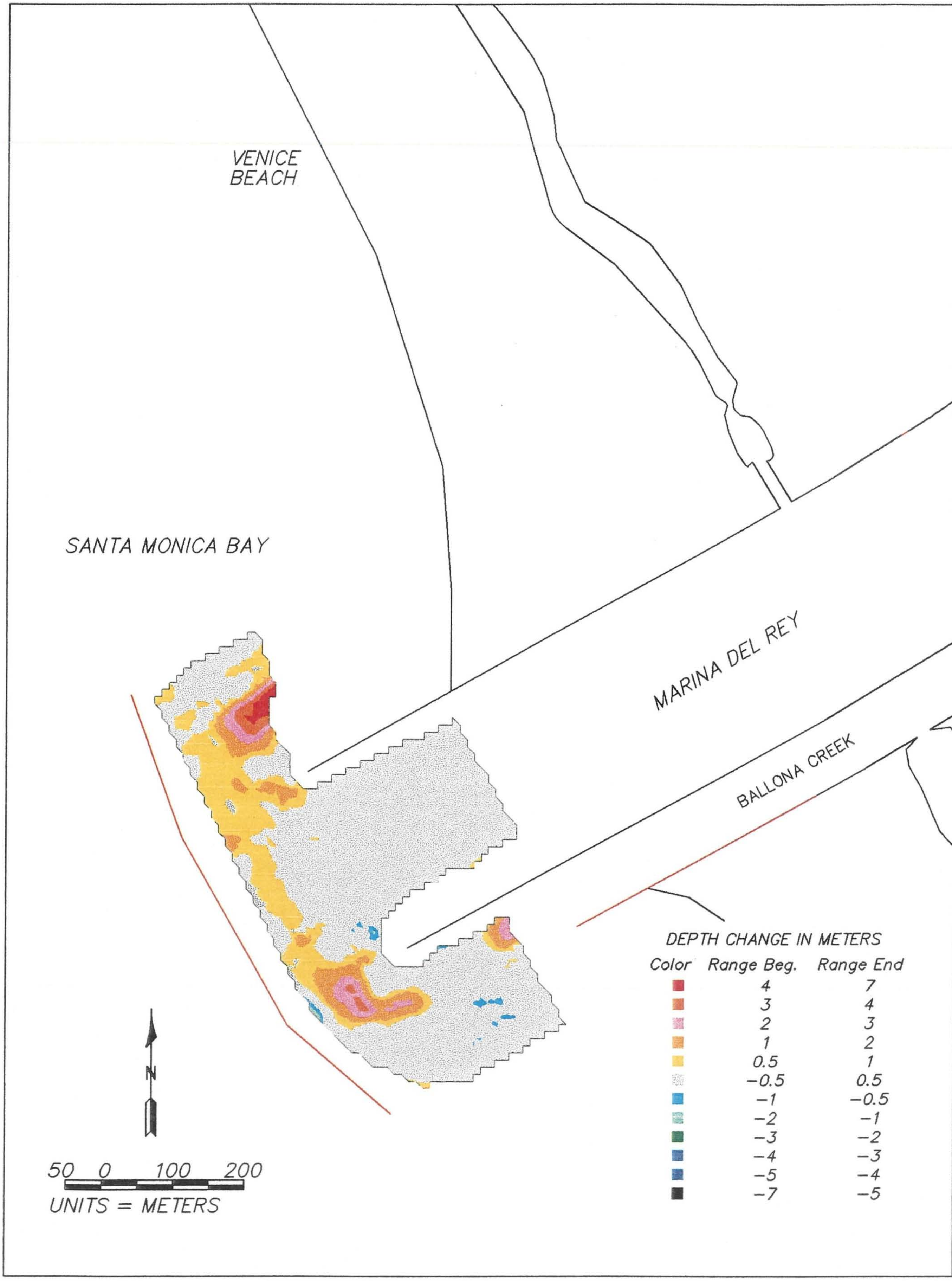

MARINA DEL REY SEDIMENT STUDY	BATHYMETRY CHANGE AUGUST'97 COND TO FEBRURAY'98 COND	Figure 2.20

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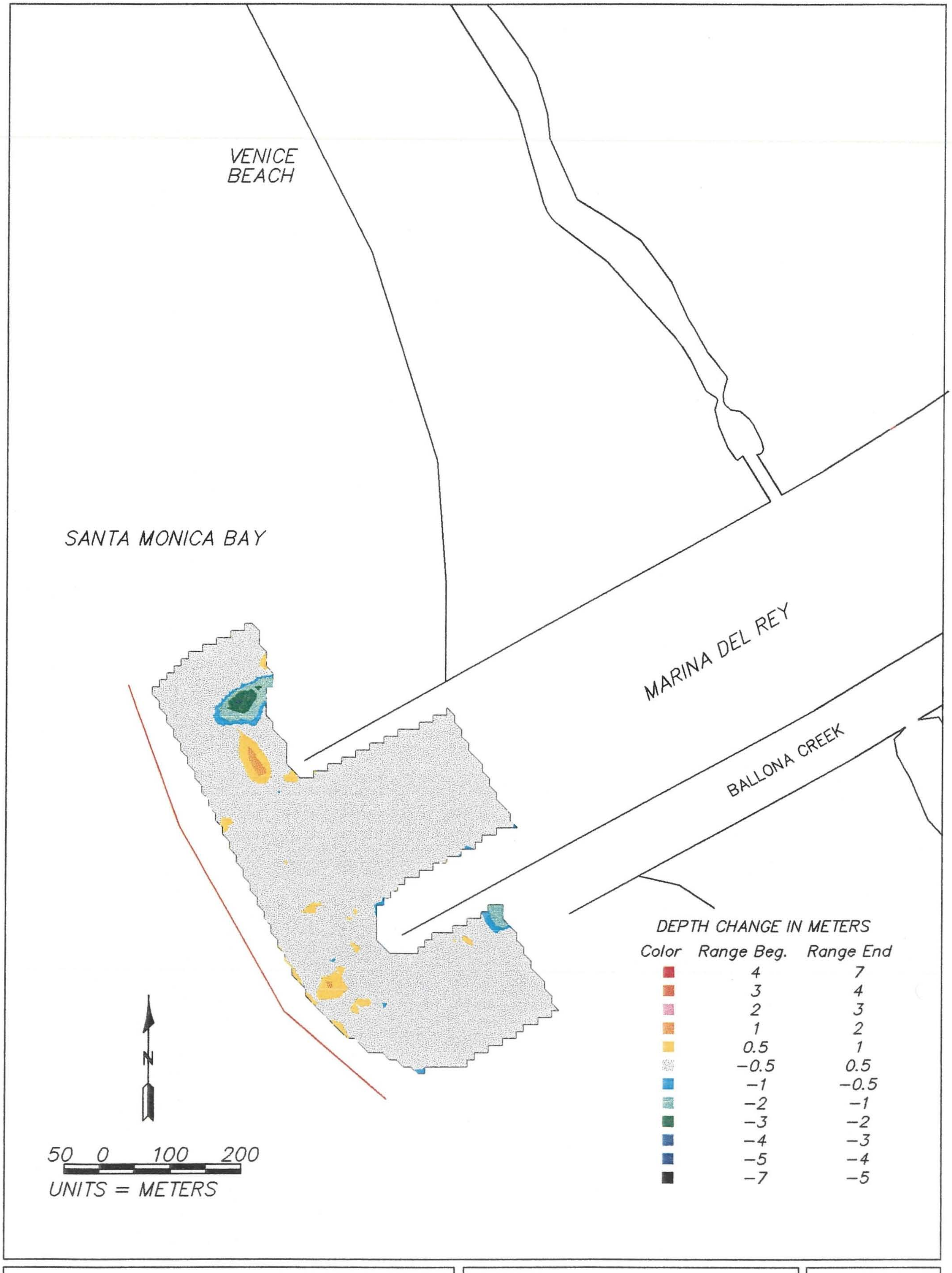

MARINA DEL REY SEDIMENT STUDY	BATHYMETRY CHANGE FEBRUARY'98 COND TO MARCH'98 PRE	Figure 2.21

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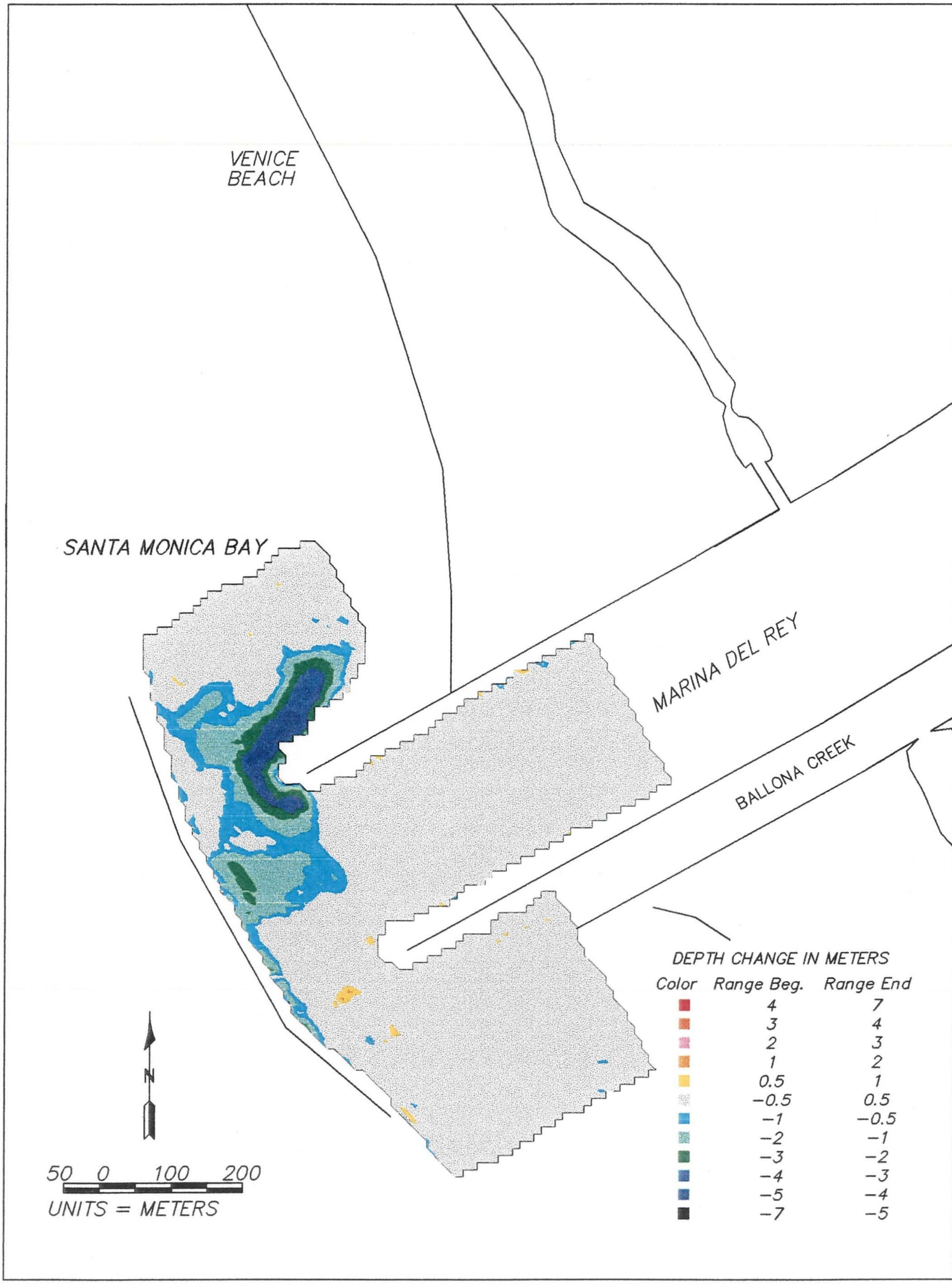

MARINA DEL REY SEDIMENT STUDY	BATHYMETRY CHANGE MARCH'98 PRE TO APRIL'98 POST	Figure 2.22

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MARINA DEL REY SEDIMENT STUDY

BATHYMETRY CHANGE
JULY'91 COND TO APRIL'98 POST

Figure 2.23

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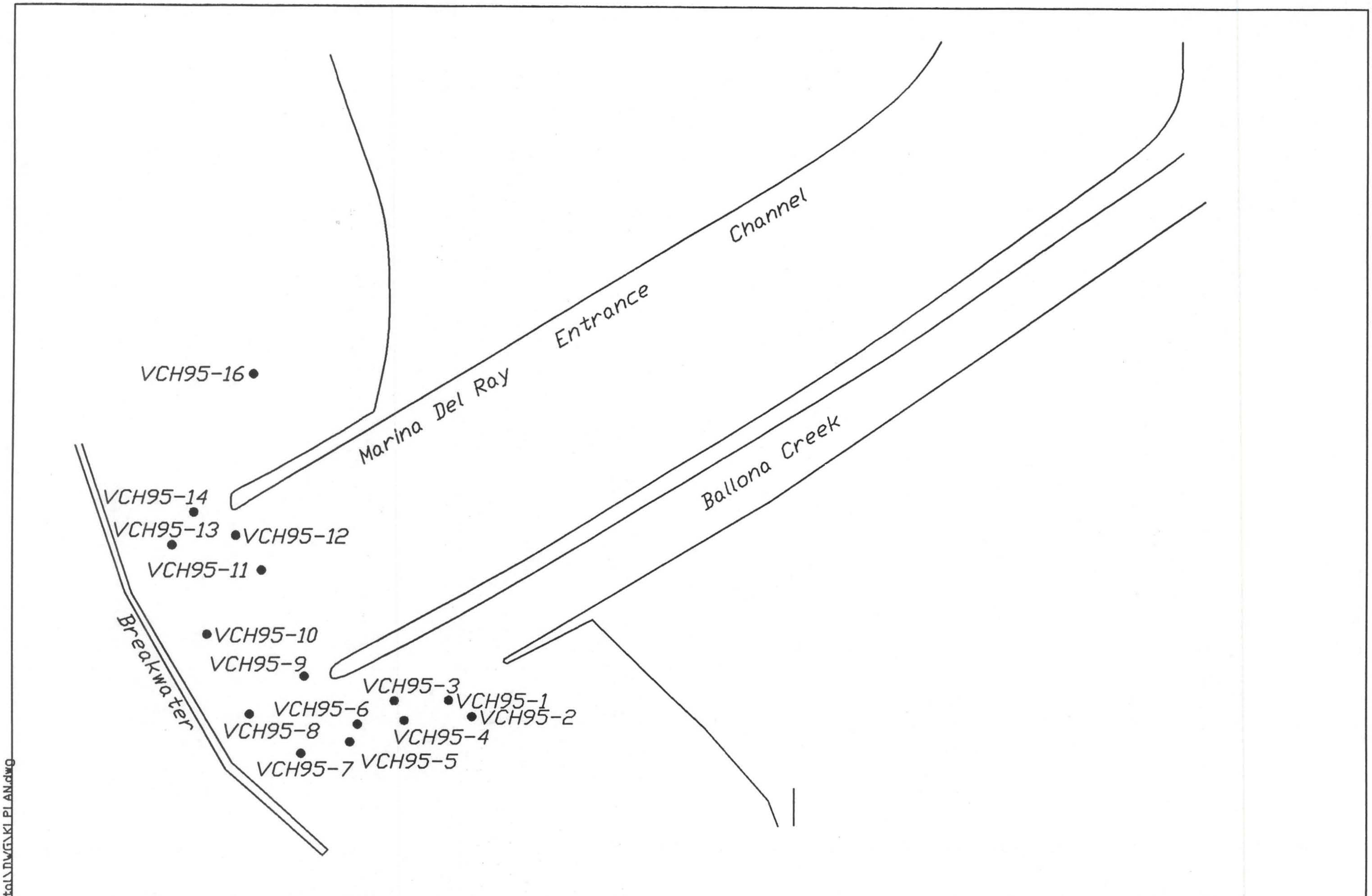

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MARINA DEL REY FEASIBILITY STUDY

1995 SAMPLING SITE PLAN

Figure 2.24

3.0 SEDIMENT SOURCES

The sources of shoaling sediment in the vicinity of Marina del Rey include sediment yield from Ballona Creek, littoral transport, aeolian (wind) transport, and sediment yield from storm drain culverts. The contribution of sediment via aeolian transport and storm drain flow was previously estimated to be very small (USACE 1995). Therefore, the primary sources of shoaling for Marina del Rey are sediment yield from Ballona Creek and littoral transport.

3.1 Sediment Yield from Ballona Creek

This section quantifies the amount of sediment that is deposited within the Marina del Rey entrance channel from Ballona Creek. Correlation between shoaling rate and creek flows is provided. Ballona Creek sediment transport is further characterized by bedload and suspended load contributions, and the sediment composition by grain size is also evaluated.

Ballona Creek drains a watershed of about 329 square kilometers. The watershed includes the Santa Monica Mountains on the north, the cities of Baldwin Hills and Inglewood on the south. The western boundary is approximately one mile inland from the Pacific Ocean and extends from the Santa Monica Mountains southward to Venice and eastward to Baldwin Hills. The eastern boundary extends from the crest of the Santa Monica Mountains southward and westward to the vicinity of central Los Angeles. Tributaries of Ballona Creek include Centinela Creek, Sepulveda Canyon Channel, Benedict Canyon Channel, and numerous storm drains.

Ballona Creek is concrete-lined upstream of Centinela Boulevard. All of its tributaries are either concrete channels or covered culverts. The channel downstream of Centinela Boulevard is trapezoidal in cross-section with grouted riprap side slopes and an earthen bottom. The collected water is discharged into Santa Monica Bay at the mouth of Ballona Creek immediately south of Marina del Rey. More detailed descriptions on Ballona Creek can be found in COELAD, 1998.

3.1.1 Correlation of Sedimentation with Storm Flows

The collected water from the watershed discharges into the lee of the Marina del Rey breakwater in the vicinity of the south entrance channel. Ballona Creek flows are dominated by rainfall runoff during the wet season and nuisance water (domestic, commercial, and industrial uses) during the dry season. Peak flows occur during the wet season, which generally is from November through May; nuisance flows are very small. Since sediment transport is proportional to the square or cube of the river velocity, sediment yield from Ballona Creek is highly dependent on storm flows. The dependency results in highly variable and episodic sediment yields. For example, Table 2.2 shows that storms between December 1994 and January 1995 discharged nearly 25,000 m^3 of sediment into the south entrance channel (Area A and Area G). This matches the average annual shoaling rate in the south entrance area as discussed at the end of Section 2.2 of this report.

Sediment yield from Ballona Creek has been shown to be the main contributor to south entrance channel shoaling (Section 3.2.4). Entrance channel shoal rates were calculated between successive hydrographic surveys (Section 2.0) and statistically correlated with averaged Ballona Creek flow rates during the same periods. Flow data from the Sawtelle Boulevard gage were provided by the District. Figure 3.1 shows a plot of both variables with time and illustrates a fairly strong correlation. A least squares fit is plotted in Figure 3.2 relating flow rate to shoal volume. The data were then analyzed using a 90% confidence Log-Pierson Type III fit to relate monthly shoaling rate as a function of return period in years. The results are shown in Figure 3.3.

3.1.2 Average Annual Sediment Contribution

Southern California Coastal Water Research Project (SCCWRP, 1973) estimated the sand yield from Ballona Creek to be 35,195 m^3/year. Approximately 4,050 m^3/year was estimated to be silt for a total of 39,245m^3/year (State of California, 1977). As described in Section 2.2, the average annual shoaling rate in the south entrance area is approximately 25,000 cubic meters per year. Based upon these estimates, if the total annual Ballona Creek sediment yield is approximately 39,000 cubic meters, and 25,000 is deposited within the south entrance, then it can be concluded that approximately 14,000 cubic meters per year is discharged beyond the harbor entrance. The finer material is deposited offshore while the coarser sand material nourishes downcoast beaches.

3.2 Sediment Yield From Littoral Processes

The Marina del Rey Harbor entrance structures are a barrier to longshore transport of sediments. Sediments accumulate on both sides of the entrance and contribute to the harbor shoaling. The following evaluates the sediment yield in the harbor entrance due to littoral processes. This information is used to formulate the harbor entrance sediment budget which is discussed in Section 4.0.

3.2.1 Geographic Setting

Marina del Rey is located in the Santa Monica littoral cell, which extends from Point Dume to Palos Verdes Point. The geographic setting of the Santa Monica cell is shown in Figure 3.4. The cell is defined by submarine canyons (Dume Canyon and Redondo Canyon) which act as sediment sinks. Santa Monica Bay is bounded on the north by the Santa Monica Mountains, on the east by the sprawling urbanized areas of Los Angeles County, and on the southeast by the hilly areas of Palos Verdes. This setting results in Santa Monica Bay receiving sediments eroded from the southern slopes of the Santa Monica Mountains on the northern beaches, and predominantly sediments from urban runoff on its eastern beaches.

The reach of shoreline within the cell has essentially stabilized due to coastal development. Groins and breakwaters trap sediments while beachfills and bypassing efforts are performed periodically to prevent significant erosion from the beaches. Shoreline erosion from Santa Monica to Redondo Beach has been found to be non-critical (COELAD, 1995).

3.2.2 Study Area Definition

This study required examination of the sediment budget in the immediate vicinity of Marina del Rey for patterns of littoral sediment transport and upland sediment yield to the beaches. In order to effectively study the potential sediment transport adjacent to the harbor it was necessary to set the boundaries sufficiently far away from the project to reflect overall changes in the shoreline, yet close enough to maintain focus and resolution. The shoreline from Pacific Palisades to Marina del Rey north jetty was selected as the boundary for the littoral transport analysis north of Marina del Rey. These boundaries were selected since there have been many estimates made for longshore transport at Santa Monica, and the north jetty acts as a sediment trap for materials transported from upcoast.

3.2.3 Potential Longshore Transport into Marina del Rey from the North

Beachfills and Dredging

Significant quantities of material have been added to the beach from as early as the 1900's and continuing until mid 1980's. These materials were the result of building Pacific Coast Highway, construction of various beach parking lots and related facilities, construction of a sewage treatment plant, and maintenance of Marina del Rey harbor.

The beachfill associated with the disposal of excavated sediments from the construction of the Hyperion Sewage Treatment Plant has, by far, been the most significant artificial source. The single project placed about 10.7 million m^3 of material over 10 km of shoreline. Approximately 3.8 million m^3 of the 10.7 million m^3 were placed north of the Ballona Creek (Marina del Rey) jetty. Pacific Coast Highway was complete as of 1938, therefore no contribution from this source was assumed after this date. Parking lot construction may have contributed a minor amount of material over the years, while dredging to bypass materials trapped behind Santa Monica Breakwater did not add new material to the shoreline. Past beachfill and dredging episodes are listed in Tables 3.1 and 3.2.

Table 3.1 Beachfill History – Pacific Palisades To Marina Del Rey

Date	Placement Location	Source	Purpose	Quantity (M^3)
1945	Venice Beach	Hyperion	Disposal	15,000
1947	Venice Beach	Hyperion	Disposal	3,800,000
			TOTAL	3,915,000

Table 3.2 **Sediment Bypassing And Backpassing History – Pacific Palisades To Marina Del Rey**

Date	Placement Location	Source	Quantity (M^3)
1939	Santa Monica Beach	Santa Monica Breakwater	46,000
1949-50	Santa Monica Beach	Santa Monica Breakwater	734,500
1957	Santa Monica Beach	Santa Monica Breakwater	597,000
1965	Dockweiler Beach	Marina del Rey north fillet	30,000
1973	Venice Beach	Marina del Rey	13,000

Stream and Storm Drain Yield

A source of material to the littoral zone includes erosion from the adjacent watershed with sediments transported via natural streams, creeks and storm drains. Previous studies have estimated sediment yield in southern California with values that vary widely (USACE, 1950, Johnson, 1950, Handin, 1951). For example, a single major storm had erosion estimates ranging from 4,000 to 36,000 m^3 per square kilometer of watershed.

A more recent study (M&N, 1996) applies a method of silt-sand ratio (e.g. Orawide, 1987) to the watershed in the project shoreline where sand yields are unknown. The land use of all watersheds that drain onto the project shoreline beaches (LACDPW, 1994; Stenstrom and Strecker, 1993) was analyzed to estimate potential sand yield. A silt-sand ratio based on data from Ballona Creek (SMBRP, 1994; COELAD, 1994; SCCWRP, 1973) was used for estimating total yields from the watershed in view of land use similarity. The analysis indicates that the watershed that drains onto the Santa Monica/Venice beaches yield 450 m^3/year of silt and 3,900 m^3/year of sand for a total sediment yield of 4,350 m^3/year. Actual quantities will vary widely from year to year depending on the rainfall intensity and watershed condition.

Longshore Transport

The net longshore transport in the area has been observed to be in a southward direction, which is consistent with the prevailing wave direction. Transport direction is subject to seasonal reversals with northerly transport more likely occurring in summer months during periods of large southern Pacific swell. Southerly transport is approximately seven times greater than northerly transport within the cell (Marine Advisors, 1958).

There have been many estimates made for longshore transport along Santa Monica Beach as listed in Table 3.3.

Table 3.3 Previous Estimates of Longshore Transport Rates

Source	Method	Estimate (m^3/year)
Ingle, "The Movement of Beach Sand," 1966	Fluorescent sand tracer study	188,000
DMJM/Tekmarine, Inc. "Santa Monica Pier Reconstruction Project," May 1984	Surveys, empirical equations	147,000
COELAD, "Cooperative Research and Data Collection Program, Coast of Southern California," 1970		191,000
COELAD, "Santa Monica Breakwater Reconnaissance Report," April 1989	Estimate	153,000
COELAD, "Santa Monica Breakwater Feasibility Report," 1995	Surveys, empirical equations	168,000 to 268,000 Ave. = 218,000
	AVERAGE	179,400

Beach Volume Change

Long-term beach volume change over the period 1953-1990 was calculated based on beach profiles presented in the study report prepared for the County of Los Angeles (Coastal Frontiers, 1992). The 1953 profiles were selected as the baseline condition since most of the major construction projects along the coast had been complete. Although the accuracy of the early County survey data (before 1989) has been considered questionable and the data need substantial correction to be useful, the data contained in the County report were confirmed to be correct (Leidersdorf, 1996). The volumetric calculations were performed based on the entire shore-normal surveyed ranges of the profiles to the extent the existing data can support.

It was found that there is an overall gain of beach volumes over the 37-year period. Stations along Santa Monica Beach showed a substantial increase in beach width (approximately 100 meters) between surveys, however the profiles downdrift of the Venice Breakwater show a shoreline retreat of approximately 30 meters from 1953 to 1990. It is noteworthy that there was a shoreline advance of approximately 160 meters prior to 1953 in response to the 1946-8 Hyperion fill and that no additional nourishment has since been provided, except the 13,000 m^3 backpassed from Marina del Rey.

Volumes were calculated using the average-end-area method, considering the volume change above approximately –10 m. The volume of accumulation from Santa Monica Pier and Marina del Rey was approximately 5,000,000 m^3. This volume translates to 135,600 m^3/year.

Exchange with Inner Shelf

The volumetric offshore gain/loss across the offshore boundary between the littoral zone and the inner-continental shelf has not been explicitly quantified for the project shoreline. Existing historical survey transects do not extend far enough offshore to provide cases for an accurate estimate of long-term exchange of sediments between the littoral zone and the innershelf. Historical beach profiling data (County of Los Angeles, 1992), however, do indicate a sustained significant deepening of the seabed at the location of transition between innershelf and lower foreshore at depths between –6 m and –12 m MLLW during the period 1953 and 1989. The profiles taken before 1953 and after 1989 form two distinct groups with profiles converging to two very different depths offshore. The post-1989 profiles are typically 0.5 to 1.5 m deeper than the pre-1953 profiles offshore of approximately –6m MLLW (transition from littoral zone to inner shelf). The volume of material lost along Santa Monica and Venice Beaches between –6 meters and the seaward limit of the surveys between 1953 and 1990 was estimated to be approximately 2,000,000 m^3.

The offshore deepening observed from beach profiling might be related to the erosion of the innershelf seabed caused by episodic rare extreme storm wave events. Since it takes a severe event comparable to the rare extreme storm of 1988 to cause any wave induced bathymetric changes to the relatively deep innershelf; the deepened offshore feature may remain for an extended period of time before being altered to another depth by a sufficiently large storm. Since this study used the apparent volumetric change between 1953 and 1990 in the beach volume change calculation, the gain or loss of sediment from the innershelf was implicitly included.

Budget of Littoral Transport from the North

The net transport rate north of Marina del Rey was calculated based on volume balance against other sediment budget components in the reach. The approximate mean value of 179,400 m^3 was used as the sediment rate along Santa Monica Beach.

The results of the sediment budget analysis were compiled for the entire project shoreline and summarized in Table 3.4. The budget presents an overall, quantitative, representation of sediment sources, sinks and routing within the project shoreline.

Table 3.4 Sediment Budget: Santa Monica To Marina Del Rey (1953-1990)

Component	Qty (m^3)	Ave Rate (m^3/year)	Ref
Beachfill	13,000	350	(1,2)
Dredging	30,000	810	(3)
Stream/Storm Drain Yield		4,350	(4)
Beach Volume Change	5,000,000	135,600	(1)
Net Longshore Transport In		179,400	(5)
Net Longshore Transport Out		47,690	(6)

Ref:

(1) County of Los Angeles (1992)
(2) COELAD (1995)
(3) COELAD record
(4) Stenstrom & Strecker (1993)
(5) Based on average upcoast rate of 179,400 m^3/year
(6) Based on volume balance

It was determined that the rate of sediments supplied to the beach immediately north of the Marina del Rey north breakwater and north entrance is 47,690 m^3/year.

3.2.4 Potential Longshore Transport into Marina del Rey from the South

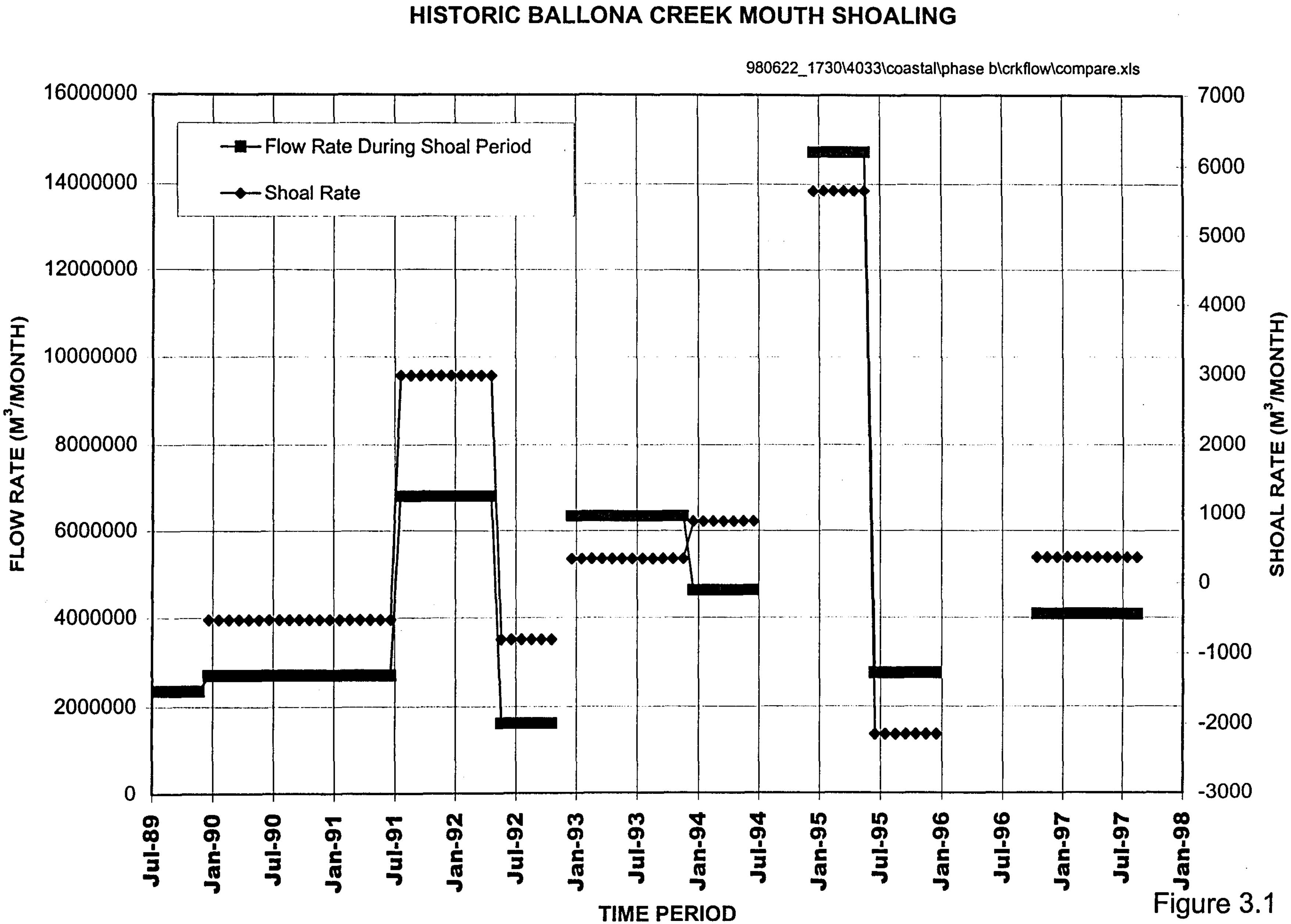
The direction of littoral transport is influenced by the direction from wave energy. Southern Hemisphere swell in the summer and pre-frontal seas in the winter have the potential to drive sediment upcoast and into Marina del Rey. As a result, shoaling in Marina del Rey would be from occasional northward reversals of longshore drift rather than sediment discharge from Ballona Creek.

In order to verify the source of shoaling, physical testing was performed on the sediments from the shoal near the Ballona Creek mouth (Soule et al, 1993). Samples were taken from the shoal in the month of October 1990, 1991, and 1992 and were analyzed against the runoff data for the three water years (October – September). The sediment composition of the shoal and the strength of flow discharge showed a strong correlation.

During the dry year of 1990, discharge through Ballona Creek was weak and was only able to carry fine sediments through the channel and deposit them at the mouth. The result was a relatively silty texture in the surface layer with a low fraction of sand. In the water year of 1991, there were a few more storm events and rainfall runoff was increased slightly. The increased strength of discharge resulted in the deposition of more fine sediments since flow was not strong enough to carry a significant amount of bedload consisting of the coarser sand. Testing showed a decrease in percent sand fraction due to an increase in silt fraction. In 1992, the strength of discharge nearly doubled the previous years due to significant rainfall-runoff events. A large increase in the amount of coarse sediments transported through the channel and deposited at the mouth was discovered.

The pattern of grain size distribution at the Ballona Creek mouth shows a coarse-to-fine gradation of sediments. This is consistent with the mechanism of successive deposition of sediments along the flow direction of a developed sediment-laden plume ejected from a channel. The methods of identifying the transport direction from sedimentary signatures can be found in McLaren (1981) and McLaren and Bowles (1985).

These observations on sediment composition and depositional patterns suggest that the sediments which form the shoals in the area fronting Ballona Creek mouth have been produced by Ballona Creek through sediment related discharges rather than upcoast reversals of longshore transport.



HISTORIC BALLONA CREEK MOUTH SHOALING
980622_1730\4033\coastal\phase b\crkflow\compare.xls
Flow Rate During Shoal Period
Shoal Rate
FLOW RATE (M³/MONTH)
SHOAL RATE (M³/MONTH)
16000000
14000000
12000000
10000000
8000000
6000000
4000000
2000000
0
7000
6000
5000
4000
3000
2000
1000
0
-1000
-2000
-3000
Jul-89
Jan-90
Jul-90
Jan-91
Jul-91
Jan-92
Jul-92
Jan-93
Jul-93
Jan-94
Jul-94
Jan-95
Jul-95
Jan-96
Jul-96
Jan-97
Jul-97
Jan-98
TIME PERIOD

Figure 3.1

FLOW RATE & SHOALING LINE FIT

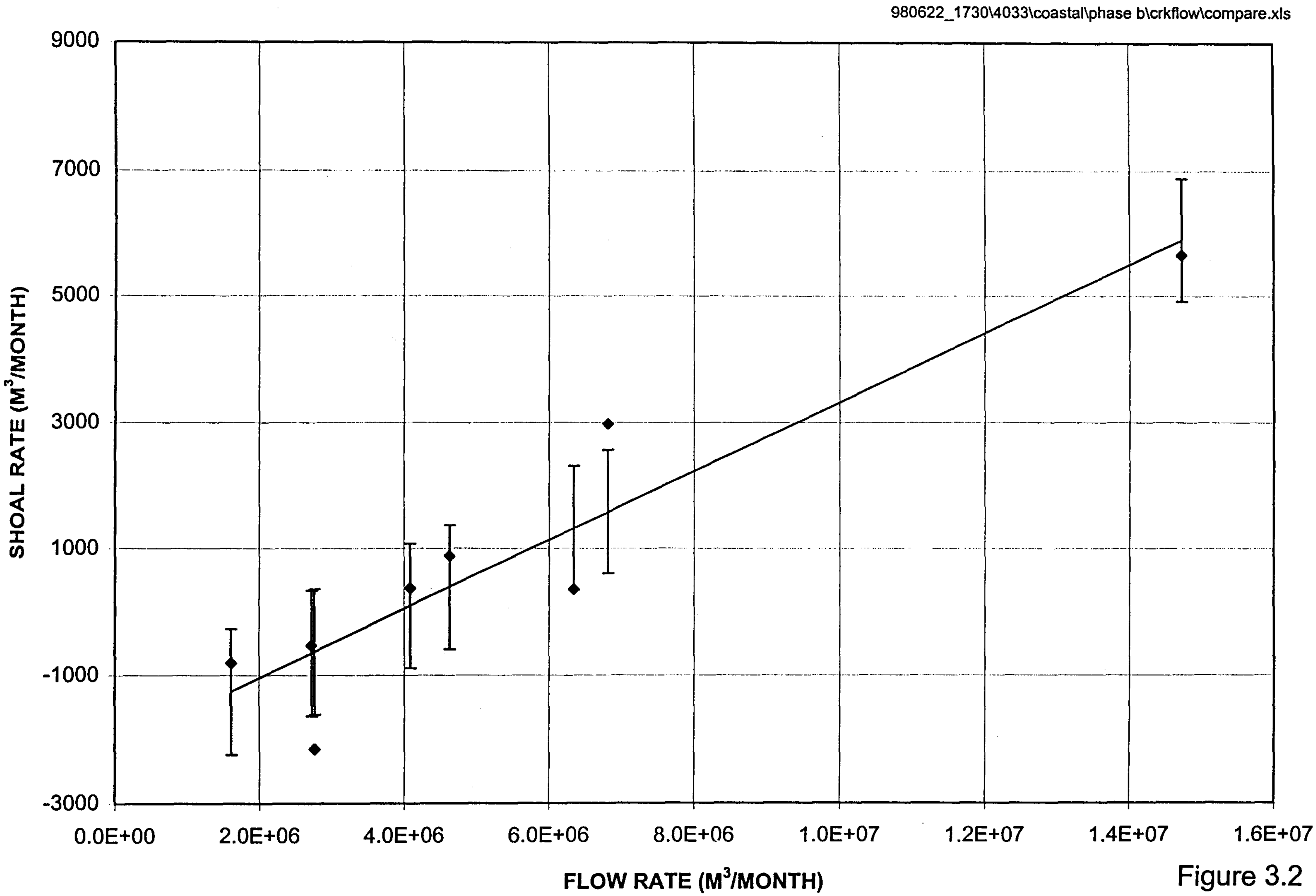


Figure 3.2

BALLONA CREEK SHOALING RISK ANALYSIS

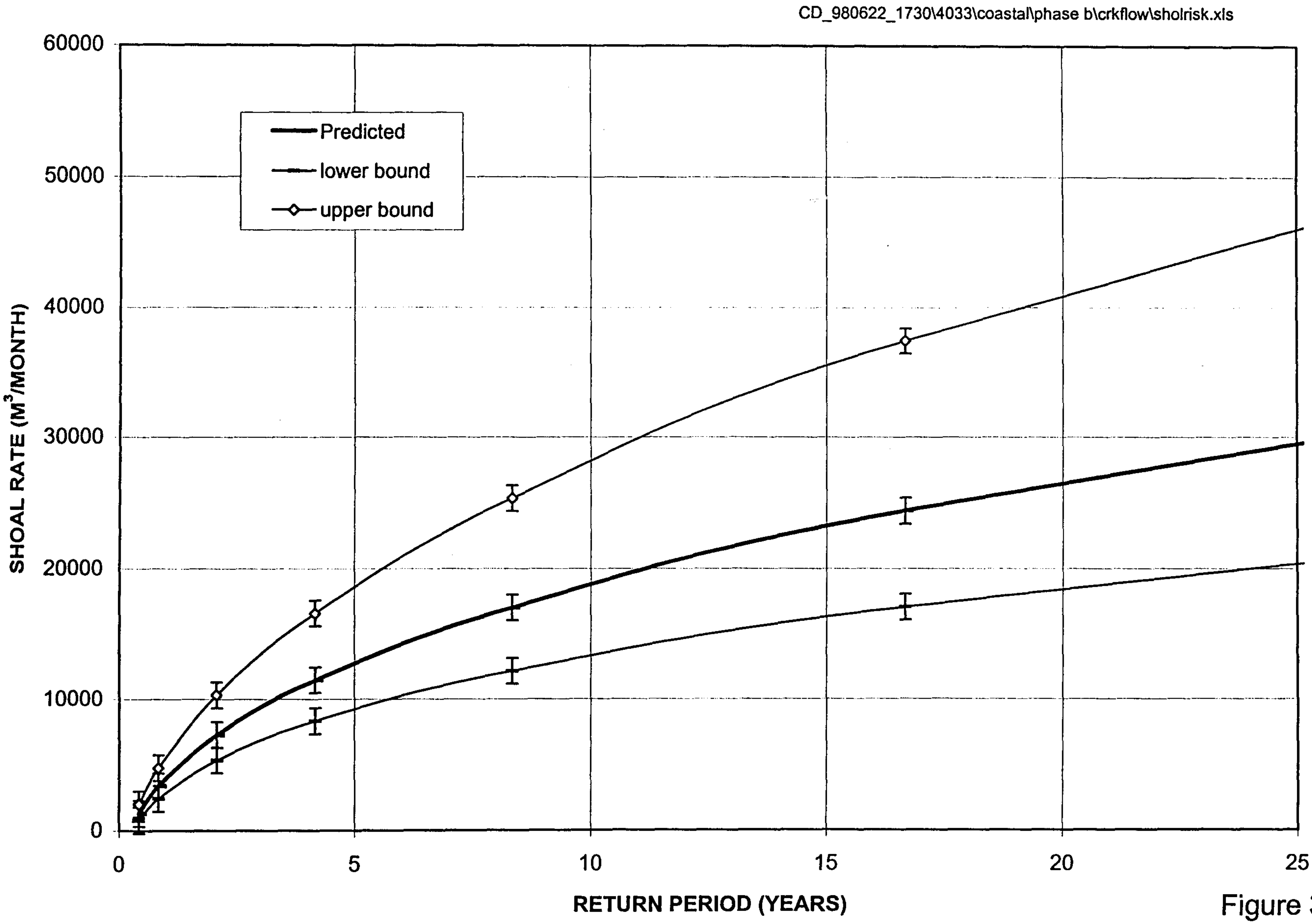
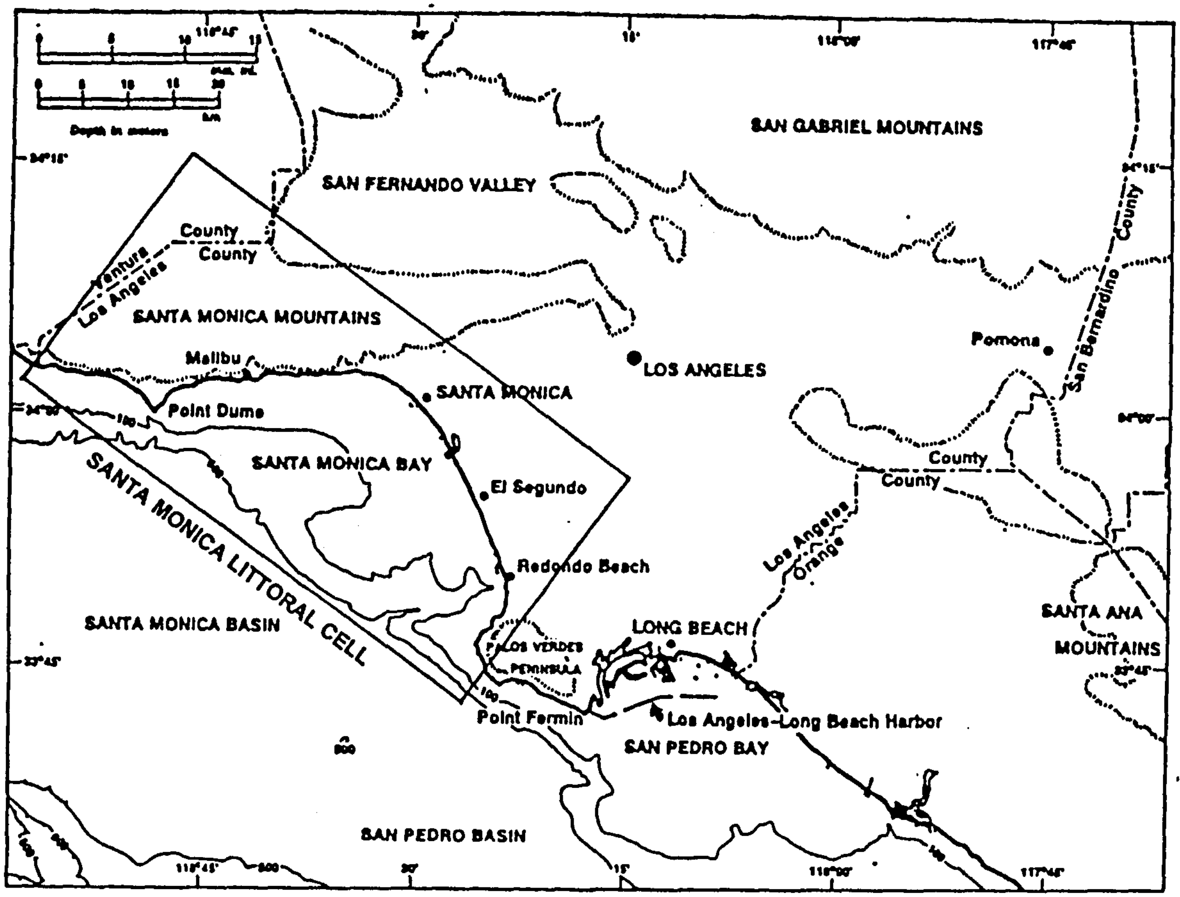


Figure 3.3

SOURCE: SMBRP, 1994
STATE OF CALIFORNIA, 1977



Prepared by: Moffatt & Nichol Engineers
June, 1998

Marina del Rey Sediment Study	Geographic Setting of Santa Monica Littoral Cell	Figure 3.4

xerox pasting

4.0 HARBOR ENTRANCE SEDIMENT BUDGET

The rate, Q, at which sediment is moved into the harbor entrance channel is the transport rate given in units of cubic meters per year. The difference between the amounts of material entering and leaving the harbor in a given time period should equal the shoaling rate. The equation for the sediment budget at Marina del Rey is:

$$\sum Q\text{in} - \sum Q\text{out} = \frac{dV}{dT}$$

Where

$Q\text{in}$ = quantity of material entering the harbor (m^3/year)

$Q\text{out}$ = quantity of material exiting the harbor (m^3/year)

dV/dT = shoaling rate in the harbor (m^3/year)

The sediment budget analysis involved calculating the shoaling rate at the harbor by summing all the gains and losses of the transport components described in the preceding sections. The shoaling rate was also calculated by analyzing hydrographic survey data and past dredging records, as described in Section 2.0.

The following summarizes the various elements of the sediment budget:

1. The net longshore transport rate from the north is about 48,000 cubic meters per year (Section 3.2.3).

2. The net shoaling rates in the north entrance channel (Area B) and sand trap (Area H) are 17,000 cubic meters per year and 20,000 cubic meters per year, respectively, for a total shoaling rate in the north entrance area of 37,000 cubic meters per year (Section 2.2).

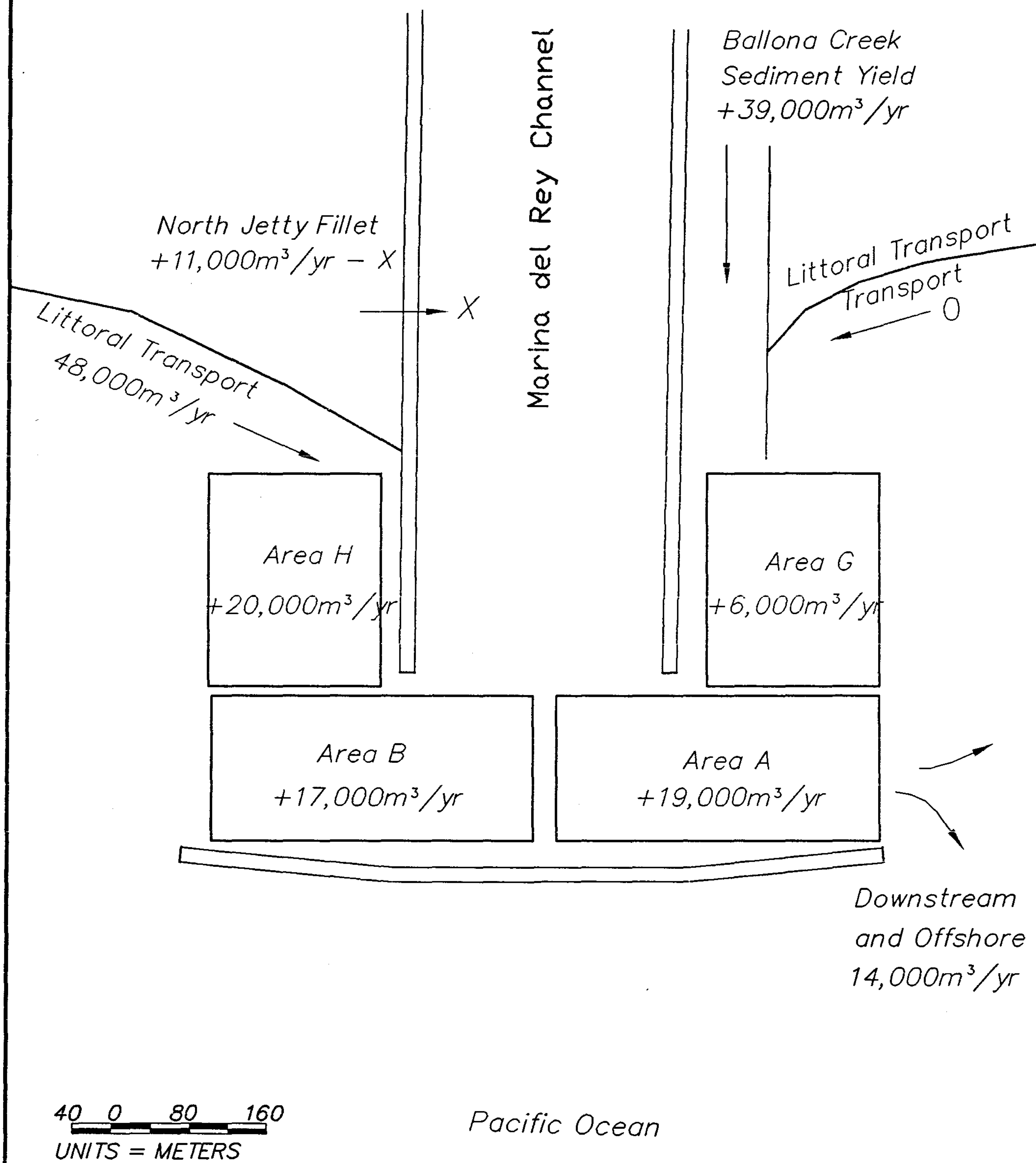
3. It is assumed that there is no net transport of sediment between the north entrance and south entrance channel.

4. The north entrance therefore has a sediment imbalance of 11,000 cubic meters per year, i.e. the shoaling rate calculated in the entrance channel area is 11,000 cubic meters per year less than the net longshore transport coming into the marina vicinity from the north. This imbalance is assumed to be accounted for in a combination of sediment accumulating in the north jetty fillet shoreward of the sand trap area and migrating through the north jetty. Insufficient hydrographic survey data was available to estimate these quantities. However, the relative amounts appear reasonable.

5. Ballona Creek has been estimated to yield a total of 39,000 cubic meters of sand and silt per year (Section 3.1.2). Of this amount, 25,000 cubic meters is estimated to settle in the entrance channel and advanced maintenance dredging area (Section 2.2). The remaining 14,000 cubic meters per year is assumed to be deposited either offshore or transported to downcoast beaches.

6. No littoral transport was assumed to enter the entrance from the south.

The sediment budget is illustrated schematically in Figure 4.1.



Prepared by: Moffatt & Nichol Engineers
Draft, June 1998
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MARINA DEL REY SEDIMENT STUDY	SEDIMENT BUDGET	Figure 4–1

5.0 NUMERICAL MODEL ANALYSIS OF SEDIMENT TRANSPORT

This section presents the numerical modeling results of flow and sediment transport patterns in the Marina del Rey Harbor and Ballona Creek mouth area. The objective of the modeling is to predict sediment transport rate and sediment depositional patterns within the Marina's navigation channels. The hydrodynamic model RMA2 was used to simulate the flow, and the sediment transport model SED2D was used to simulate shoaling patterns. The models were calibrated with available hydrological and field survey data for the period from December 1994 to June 1995. The calibrated models were then used to predict the shoaling rates and patterns within the existing Marina's navigational channels and the efficacy of advanced maintenance dredging alternatives developed to reduce harbor shoaling.

5.1 Description of Model

5.1.1 Model Selection

RMA2 is a generalized finite element model for horizontal, two-dimensional surface flow. The model calculates the velocity field and water elevation at discrete nodal points with given boundary conditions in the format of flow, elevation or velocity either at selected nodes or along a series of nodes. Detailed model descriptions are given in King (1973).

SED2D is a generalized finite element model for horizontal, two-dimensional sediment transport. SED2D can model both cohesive and non-cohesive sediment transport. SED2D utilizes the flow field computed from RMA2 as input and calculates the sediment concentration in the water column from a convection-diffusion equation. The deposition and erosion patterns are determined from the sediment concentration in the water column and interactions between the water column and sediment bed through source/sink terms in the transport equation (BYU, 1994). A detailed model description can be found in Ariathurai (1974).

The RMA2 and SED2D models were chosen because they can simulate physical processes in the river estuary system and general harbor area. They have been applied successfully to numerous project sites. The RMA2/SED2D models also have a number of distinct features. First, the models use a finite element computational grid, which can best resolve meandering land boundaries such as riverbanks, harbors and marinas. Second, both RMA2 and SED2D are part of the surface modeling system FAST-TAB maintained by the U.S. Army Corps of Engineers, and therefore have standardized input and output formats. SED2D can use RMA2 output without any modification. Third, RMA2 and SED2D have an enhanced graphic interface known as SMS (Surface Water Modeling System), which makes grid generation and model results visualization efficient and user-friendly. A detailed description of SMS can be found in the "Surface Water Modeling System Reference Manual" (BYU, 1997).

Model Modifications

The existing version of the SED2D model cannot simulate sediment transport due to waves and current. Since the waves play an important role in sediment erosion, deposition and transportation at the coastal zone area, the original source code was modified to incorporate the wave effects.

Theoretical Basis for Model Modifications

The wave effects were incorporated into the SED2D model in two categories of calculations: the bottom stress calculation and the sediment transport rate calculation.

The bottom shear stress caused only by current, τ_c, is expressed as

$$\tau_c = \frac{1}{2} \rho f_c \bar{u}^{2} \tag{1}$$

where ρ is the water density, f_c is the shear stress coefficient for current, and u is the depth-averaged current velocity.

Under the combined action of wave and current, the bottom shear stress can be significantly increased due to the wave effects superimposed on current. Based on the results of Bijker (1971), the User's Manual for SED2D_WES, Version 1.2 Beta (Roig, et al, 1996) recommended that the bottom shear stress under wave and current, τ_{wc}, can be calculated by a Bijker-type equation as:

$$\tau_{wc} = \frac{1}{2}\rho(f_c \bar{u}^2 + 0.5 f_w u_w^2) \tag{2}$$

where f_w is the shear stress coefficient for wave, and u_w is the maximum wave orbital velocity at bottom calculated with the linear wave theory (Dean and Dalrymple, 1991). The coefficient, f_w, is calculated based on the results of Jonsson (1966,1980), Kamphuis (1975), Kajiura (1968) and Justesen (1988). These results were summarized by Nielsen (1992). By using wave and current induced stress, τ_{wc}, instead of the current induced stress, τ_c, alone, the wave effect is incorporated for sediment resuspension.

Since both waves and current are present, the mean velocity for sediment transportation increases. The mean transportation velocity, u_{wc}, due to the combined action of waves and currents can be calculated as (Vemulakonda et al, 1988):

$$u_{wc} = \bar{u} \left\{ 1 + \frac{1}{2}\left[18.0 \log\left(\frac{10h}{D}\right)\left(\frac{f_w}{2g}\right)^{1/2} \frac{u_w}{\bar{u}} \right]^2 \right\}^{1/2} \tag{3}$$

where h is the water depth and D is the bed roughness, the latter is determined based on the grain sizes at the bottom. Equation (3) is used to obtain the sediment transport rate and not used in the calculation of the bed shear stress.

Implementation

The following steps were taken to incorporate the wave effects into SED2D model:

1. The original source code of the SED2D model was modified to include new statements to calculate the wave and current induced stress, τ_{wc} and mean transportation velocity, u_{wc}.

2. A wave model, REF/DIF, which is described in Section 5.2.4, was used to obtain the wave field prior to the SED2D modeling. The calculated wave field covered an area larger than the RMA2 and SED2D modeling area.

3. With the wave and current information on every node from REF/DIF and RMA2 modeling results, respectively, the modified SED2D model was used to calculate the bottom shear stress, τ_{wc}, under the combined action of wave and current. The τ_{wc}

replaces τ_c in the modified program for sediment erosion, deposition and transport calculations, when both waves and currents exist.

5.1.2 Model Validation Strategy and Considerations

Model validation generally consists of calibration and verification phases. In the case of RMA2/SED2D models, usually two historical time periods delimited by three historical field surveys are needed to accomplish the objective. In the calibration phase, the model is applied to one time period to determine the model calibration parameters. Then, in the verification phase, it is applied to the other time period to reproduce the shoaling volume and patterns without changing the parameters determined in the calibration phase.

The selection of the calibration and verification periods was based on both availability and appropriateness of historical data. Two primary criteria were considered to select the periods. First, there should be field surveys conducted at the beginning and the end of the calibration or verification period, but without dredging activities within the period, so that the shoaling volume and patterns can be compared between the model results and the field data. Second, the shoaling volume in the period should be relatively large so that the typical shoaling related parameters can be determined and, also, potential numerical error can be minimized. The period from December 1, 1994 to June 30, 1995 was chosen to be the calibration period since it represents the largest shoaling volume in recent years and covers a complete wet season. The period from April 1, 1996 to September 30, 1996 was selected to be the verification period, which has the largest shoaling volume for a dry season in recent years.

5.2 Environmental Conditions

5.2.1 Modeling Area

The modeling area, shown in Figure 5.1, included Marina del Rey Harbor, 5.5 km of Ballona Creek from the mouth, Ballona Lagoon, del Rey Lagoon, and the nearshore ocean area around Marina del Rey. The ocean boundary was about 3.1 km from the shoreline located at the 30.0 m (MLLW) contour. The side boundaries were 4.3 km northwest and southeast from Marina del Rey. Placing the open boundaries far away from the area of interest minimized boundary effects. Some features near the side boundaries, such as the groins and offshore breakwater, were not included in the modeling area to keep the boundary areas smooth, thereby enhancing model stability. Omitting these features would not affect the modeling results because they are far away from the study area.

5.2.2 Bathymetry

The ocean bathymetry of the modeling area was based on the 1994 National Oceanic and Atmospheric Administration map of Santa Monica Bay (Chart Number 18744). Data from the Los Angeles County Public Works Department were used for the Ballona Creek bathymetry. Two recent entrance channel survey data sets, the September 1996 survey and the August 1997 survey, were available for the beginning of the sediment modeling study. The September 1996 survey was selected to represent the existing conditions in Marina del Rey Harbor because the same survey data were used in the hydrodynamic and contaminant transport modeling for the previous *Shoaling and Disposal* phase of the *Marina del Rey and Ballona Creek Feasibility Study* (USACE-LAD, February 1998). The comparisons between the August 1997 and September 1996 survey data indicated no significant change in the modeling results between the two surveys.

Modeling simulations were performed for three bathymetric configurations. The configurations included existing conditions and two alternatives. Alternative 1 involved dredging the entrance channel at areas A and B in Figure 5.2 to –6.1 m (MLLW). Alternative 2 involved dredging part of the entrance channel at areas A to –6.1 m (MLLW) and area B to –9.1 m (MLLW). The bathymetry outside the dredging areas for both Alternative 1 and Alternative 2 was the same as the existing condition described above. The bathymetric configurations for existing conditions, Alternative 1, and Alternative 2 are shown in Figures 5.3, 5.4, and 5.5, respectively.

For model calibration and verification, the December 1994 survey data and the April 1996 survey data were used for the entrance channel areas, respectively. The bathymetry for other areas in the modeling domain was the same as the existing condition.

5.2.3 Numerical Model Grid Setup

The finite element grids corresponding to the existing condition, Alternative 1, and Alternative 2 described above are shown in Figure 5.3, 5.4, and 5.5, respectively. A relatively fine element size (2,500 m^2 on average) was used for Marina del Rey Harbor and the nearshore area to appropriately incorporate waves and longshore current. A relatively coarse element size was used for the offshore area (90,000 m^2 maximum) to minimize computation time. The entire modeling area of approximately 40 km^2 was represented by a finite element mesh consisting of 785 elements and 2,519 nodes.

5.2.4 Wave Climate

Ocean waves have significant impact on the sediment transport in the harbor and nearshore area, and therefore must be incorporated into the modeling process. The model requires one set of wave data for calibration, and one set of wave data for prediction of various expected environmental conditions.

<u>Available Wave Data</u>

The SED2D model was modified by Moffatt & Nichol Engineers to include wave height, period, and direction input for a duration coinciding with historical bathymetric surveys and flow records at the project site. Available wave data sources were reviewed to identify the most appropriate sources to develop the wave climate for the model.

The most promising wave data sources included:

- Wave Hindcasts;
- Coastal Data Information Program (CDIP) gages; and
- National Oceanic and Atmospheric Administration (NOAA) buoys, operated by the National Data Buoy Center (NDBC).

Figure 5.6 shows the locations of the available wave hindcasts; Figure 5.7 shows locations of wave gages. NDBC buoy 46025 was selected as the source of wave data due to advantages it provided over other sources. The data sources and their constraints relative to use in this project are described in the following.

Wave hindcasts are computer-simulated models of wave conditions based on local and distant weather conditions. There are many sources and locations of hindcast information as shown in Figure 5.6. Measured wave gage records are preferred over hindcasts when available. Also, hindcast data was not available for time periods coinciding with the survey data used for model calibration and verification, and therefore not considered appropriate for this project.

The remaining buoy data sources were evaluated for their proximity to the project site, similar sheltering, recording of energy and directional data, and period of record coinciding with survey data. The nearest buoy to the project site is the NOAA Redondo Beach buoy, shown in Figure 5.7. However, this was not used because of sheltering effects from the Palos Verdes Peninsula being different than that found at Marina del Rey.

NDBC buoy 46025 is located in deepwater seaward of the project site and has sporadically recorded wave energy since 1982 (CDIP, 1998), and energy and direction since 1991 (NODC, 1997). The data record was semi-continuous during the 7-year period from 1991 to 1997 with less than 3 months of missing data, resulting in a greater than 96 percent complete record. The buoy is sheltered from northern swells in the same manner as the project site and is exposed to the south. Because of similar sheltering and the deepwater location, this wave record can be transformed to the project site with the least loss of accuracy. On this basis, the NDBC buoy 46025 wave record was selected as the database for this project.

The buoy 46025 data record was purchased from the National Oceanographic Data Center in compact disc form. It consists of data from 1991 through 1997. These data were filtered and modified to provide hourly significant wave height, peak spectral period, and the direction corresponding to the peak frequency of the spectral density plots.

Wave Transformation

To develop the wave record in the nearshore areas including the Marina del Rey entrance channels, the waves recorded by the NDBC buoy were transformed closer to the model area using numerical wave transformation methods. The propagation of deep-water waves to the shore over an irregular bottom bathymetry and around islands is mainly dependent on five phenomena including refraction, shoaling, diffraction, reflection and energy dissipation. Three numerical models were employed to transform the recorded buoy waves to the vicinity of the project site. The approach is shown graphically in Figure 5.8. The first two models were used to develop wave amplification factors and new directions. These factors and directions were used to modify the wave record from the buoy location to a further offshore, more uniform deep water depth. This new location allows a point wave record to be applied along a uniform offshore boundary. This uniform deep water wave record was then transformed shoreward, using the same two computer models, to the offshore boundary of the REF/DIF model. REF/DIF was then used to provide detailed wave amplification factors and directions for the model area. These final wave amplification factors were multiplied by the previous wave height record yielding a new wave height record. The new directions were also substituted.

The following describes the various wave transformation methods used:

RD Model

Wave amplification factors were extracted from a refraction/diffraction (RD) model performed by O'Reilly and Guza (1993). This model incorporates the effects of refraction, diffraction, shoaling, and boundary friction to predict changes in wave conditions over bathymetry. The model includes the entire Southern California Bight, thus requiring large grid spacing for calculation efficiency. This large grid spacing limits the amount of detail that can be extracted at specific locations, such as the model region. In addition, it provides accurate transformation of wave heights across varying bathymetry, but does not provide accurate information on the changes in wave direction.

The RD model provides wave amplification factors for thousands of nodes across the Southern California Bight. These factors are all referenced to deep water. Factors were extracted for the NDBC buoy location, and also for the offshore boundary of the REF/DIF grid shown on Figure 5.9. The amplification factors along the REF/DIF offshore grid boundary were then spatially averaged to create one amplification factor applied along the entire boundary. Transformation of one wave height record was accomplished with the following equation:

$$H_n = H_b \times \frac{K_n}{K_b}$$

where:

H_n = near shore wave height;
H_b = wave height at buoy;
K_b = wave amplification factor from deep water to buoy location; and
K_n = wave amplification factor from deep water to near shore location.

REFRAC Model

To provide changes in wave direction, a refraction/shoaling model called REFRAC was used. Developed by Moffatt & Nichol Engineers (Headland, 1984), this model traces wave rays over the specified bathymetry, and provides both wave amplification factors and directions. Wave height and direction calculations are independent in the REFRAC model. Only the direction changes were taken from this model, since the RD predicted amplification factors are more accurate.

Wave directions were extracted from the REFRAC output at the buoy location and at the offshore boundary of the REF/DIF grid boundary. Directions were averaged across this boundary creating one direction value. For each wave record direction at the buoy location, the associated wave direction for the offshore REF/DIF grid boundary was substituted, thus creating a new wave record for input to REF/DIF.

REF/DIF Model

To create wave amplification factors and new directions on a more detailed scale than was provided by the RD and REFRAC models, another numerical model called REF/DIF (Kirby and Dalrymple, 1993) was used. The grid cells chosen for REF/DIF modeling are of high enough resolution (30.5 m x 30.2 m) to include detailed nearshore features. These small grid cells were used to model wave conditions at site specific locations including the Marina entrance channel area. Figure 5.10 shows the REF/DIF grid and bathymetry in the vicinity of the Marina del Rey harbor.

The input wave record to REF/DIF was modified using amplification factors and substitute directions in the same manner as in the RD and REFRAC models. Output values consisted of new wave records located at the nodes of the SED2D model grid. Interpolation between the REF/DIF grid nodes was required to extract factors and directions at the SED2D nodes.

Wave Statistical Analysis

Daily wave averages and averaged wave climate were determined at the project site for input to the SED2D model and input to longshore current calculations. The daily average wave condition was used for model calibration and the averaged wave climate was used combined with varying flows from Ballona Creek to make predictions of the variability of the local hydrodynamics and sediment transport

Daily Wave Averages

Daily wave averages were obtained by energy weighted averaging of the wave height, direction and period over the specific date in the calibration and verification periods. The averaging process was done on the REF/DIF offshore grid boundary, which is about 9,100 meters west of the Marina del Rey shoreline at a water depth of approximately 65 meters. After the averaging, REF/DIF was run to bring the waves to the different nearshore and harbor locations. Then the REF/DIF generated wave field was interpolated to the RMA2/SED2D grid nodes. In the three periods with large Ballona Creek discharges, e.g., January 1, 1995 to January 15, 1995 and March 9, 1995 to March 13, 1995 in the calibration period and April 15, 1996 to April 20, 1996 in the verification period, the average wave condition for each day was used as wave input condition for the SED2D model. The calculated daily wave average data are shown in Table 5.1. In the days between January 1 and January 7 of 1995, and between January 13 and January 15, the wave data were not available. The daily wave average results for January 8, 1995 through January 12, 1995 were then extended to those days to replace the missing data in the calculation.

Table 5.1 Daily Wave Averages in the Calibration and Verification Periods

Storms in Calibration Period January 1, 1995 to January 15, 1995 and March 9, 1995 to March 13, 1995				Storms in Verification Period April 15, 1996 to April 20, 1996			
Date	Wave Height (Meters)	Wave Period (Seconds)	Wave Direction (Deg. TN)	Date	Wave Height (Meters)	Wave Period (Seconds)	Wave Direction (Deg. TN)
1/8/95	0.8	13.0	175	4/15/96	0.5	13.3	244
1/9/95	0.9	13.4	171	4/16/96	1.0	13.4	255
1/10/95	1.8	10.3	171	4/17/96	1.4	12.5	228
1/11/95	1.6	13.7	208	4/18/96	1.1	10.2	233
1/12/95	1.4	16.7	190	4/19/96	1.1	9.0	245
3/9/95	1.0	14.1	238	4/20/96	1.1	8.4	242
3/10/95	0.9	9.1	170				
3/11/95	3.1	12.1	223				
3/12/95	2.5	14.1	223				
3/13/95	1.5	12.7	251				

For the calibration and verification purposes, average wave conditions over the above three periods of storm events, over the complete calibration period (December 1,1994 to June 30,1995) and complete verification period (April 1,1996 to September 30,1996) were also calculated.

Average Wave Climate

The average wave climate is the numerically averaged wave conditions expected in the project area. The averaging process was the same as for the daily averaged waves except over a longer time period (the seven-year period from 1991 to 1997 was used in the present study). After the REF/DIF modeling, a wave field was generated in the Marina del Rey area. This wave condition was used with varied Ballona Creek flow rates to explore the relationship between ocean waves and creek flows on the sediment transport in the project area. Since the wave conditions were different depending on the locations, a typical averaged wave climate was determined for a location at the 300 meter contour offshore from the Marina del Rey harbor by averaging the wave height and direction along the contour line. Based on the seven-years (1991 to 1997) of available transformed buoy data, the average prevailing wave height and period are about 1.1 meters and 11.6 seconds, respectively. The principal direction of the prevailing waves is about

244 degrees azimuth. In the SED2D model simulations, the REF/DIF generated wave field was used as input conditions.

5.2.5 Longshore Current

Longshore current velocity and the area of surfzone in which the longshore current occurs depend on wave characteristics (i.e., wave height, period, and direction) and beach profile conditions. After the waves were transformed to the vicinity of the Marina del Rey harbor and nearshore area, the averaged wave height and direction were calculated along the outermost surf zone boundary for various calibration and design wave conditions. This boundary is located at approximately 300 meters offshore. The averaged wave conditions then were used in the longshore current calculations. The reason to select the surf zone boundary was to minimize the effect of local shoreline variation on the wave direction.

Using the Longuet-Higgin's longshore transport formula (Shore Protection Manual, 1984) the longshore current velocities were obtained from the previously mentioned breakerline wave conditions. These longshore current velocities were incorporated into RMA2 and SED2D models. The calculated longshore current velocity values and associated average nearshore wave conditions are listed in Table 5.2.

Table 5.2 Calculated Longshore Current Values and Associated Average Nearshore Wave Conditions

Time Period	Longshore Current (M/S)	Average Wave Height (M)	Average Wave Direction (Degrees, TN)	Direction along Shoreline*
January 1995 Storm (1/8/95 to 1/12/95)	0.47	1.2	212	To North
March 1995 Storm (3/9/95 to 3/13/95)	0.15	1.1	232	To North
April 1996 Storm (4/15/96 to 4/20/96)	0.01	1.1	239	To North
Calibration Period (12/1/94 to 6/30/95)	0.18	1.2	249	To South
Verification Period (4/1/96 to 9/30/96)	0.14	1.1	248	To South
Seven-year (3/1991 to 7/1997)	0.13	1.1	247	To South

* Wave direction 240 degrees TN is approximately normal to the shoreline adjacent to the Marina del Rey Harbor.

5.2.6 Flood Discharge From Ballona Creek

The Los Angeles County Department of Public Works (LACDPW) measured river elevations in Ballona Creek continuously since 1928. The elevations were converted to flowrate using a rating table established for the measuring station. The station is located at the Ballona Creek and Sawtelle Boulevard intersection. A statistical analysis of the discharge data was conducted to determine river discharges and corresponding return periods based on the time period between 1928 and 1992. The results of the statistical analysis are presented in Table 5.3.

Table 5.3 Extreme Discharges For Ballona Creek (Sawtelle Boulevard Station)

Return Period (year)	**Discharge (m^3/sec)**
1	220
5	500
10	620
25	780
50	910
100	1,020

As discussed in Section 3.1.2, Ballona Creek yields about 35,170 m^3 of sand and 4,050 m^3 of silt to the Marina del Rey south entrance channel area (State of California, 1977; SCCWRP, 1973). Based on the observations (USACE-LAD, 1998), sediment production is highly variable and depends on the land use, water management, and hydrological conditions within the Ballona Creek Watershed. The sediment deposited at the mouth of Ballona Creek immediately after the 1994-1995 storm season provides evidence that severe storm events produce significant shoaling in the south entrance channel.

Since the 1989 water year, LACDPW elevation data for Ballona Creek Sawtelle Station are available in digital form for small time intervals (5 or 15 minutes) in addition to the daily averaged data. These time series data were processed using the rating table provided by LACDPW to obtain the instantaneous flowrate information, which was used in the calibration process. The 5-, 25-, and 100-year return period storm flood hydrographs were also derived and are shown in Figure 5.11.

5.2.7 Tides

The effect of tidal elevation variations was included in the sediment transport analysis. Tides in the vicinity of Marina del Rey consist of diurnal and semidiurnal constituents typical of the Southern California Coast. Statistical tidal benchmarks calculated from data collected in the Los Angeles Outer Harbor are summarized in Table 5.4. These benchmarks are representative of the tidal conditions at Marina del Rey during the latest tidal epoch from 1960 through 1978.

Table 5.4 Statistical Tide Information at Los Angeles Outer Harbor

Tide	Water Surface Elevation (m, MLLW)
Highest Tide (Jan. 27, 1983)	2.4
Mean Higher High Water (MHHW)	1.7
Mean High Water (MHW)	1.5
Mean Tide Level (MTL)	0.9
Mean Low Water (MLW)	0.3
Mean Lower Low Water (MLLW)	0.0
Lowest Tide (Dec. 17, 1933)	-0.8

In the calibration process, the hourly tidal data for the Los Angeles Outer Harbor (Station 9410660) in the corresponding time periods were downloaded from NOAA's website as ocean input boundary conditions for the hydrodynamic model. Figure 5.12 shows the tide elevation from January 1, 1995 to January 15, 1995 in the calibration period. For the design simulation runs, the synthetic tide elevation in September 1996 was selected to be consistent with the bathymetry used in the modeling. One spring and one neap tide were selected as boundary conditions. The selected spring tide starts from 19:00, September 9, 1996 until 2:00, September 12, 1996. The selected neap tide starts from 13:00, September 3, 1996 until 20:00, September 5, 1996. The total simulation time is 55 hours for both the spring tide and the neap tide. The tidal elevations varied with time as shown in Figures 5.13 and 5.14 for spring and neap tides, respectively. Both tides included more than two diurnal cycles and one diurnal cycle included two semi-diurnal cycles. The same tide elevation for the ocean boundary was applied to the side boundaries.

5.2.8 Grain Size Distribution

The sedimentary condition in Marina del Rey Harbor was analyzed for two relatively distinct parts of the harbor: the entrance channel areas and the basin areas. The sedimentary condition of the surrounding shoreline and upstream in Ballona Creek were also evaluated. The following analysis provides the range of grain size and materials for the sediment model input conditions.

Entrance Channel Areas

The USACE South Pacific Division Laboratory conducted physical tests on 29 boring samples taken in the vicinity of Marina del Rey harbor entrance areas in December 1993. The results were originally analyzed and presented in the "Feasibility Study for a Capped Dredged Material Disposal Site in Santa Monica Bay" (Moffatt & Nichol, 1993) and the "Final Environmental Assessment, Marina del Rey Maintenance Dredging and Contained Aquatic Disposal Demonstration Project" (USACE-LAD, 1994).

The following was observed:

- Sediments in the shoals adjacent to the inner sides of the jetties along the entrance channel are substantially similar to those in the sand spits near the tips of the middle and north jetties. These sediments are sand with a typical size of about 0.2 mm.
- Sediments progressively get finer toward the centerline of the entrance channel. The typical median grain size in the center of the channel is .02 mm. The difference in gradation of sediments from fine to coarse might be explained by the fact that the material near the sides is more shallow than the center of the channel and is exposed to persistent wave action that would tend to extract the finer material, leaving the coarser.
- Sediments at the Ballona Creek mouth are slightly coarser near the middle jetty with a typical grain size of about 0.3 mm, and are similar to those from the middle jetty spit where the median grain size is about 0.2 mm.
- Sediments near Ballona Lagoon on the north side of the entrance channel exhibit a clear coarse-to-fine gradation away from the lagoon outlet, which suggests the presence of a sediment-laden discharge from the lagoon into the entrance channel.
- Sediments near the bend of the navigation channel are predominantly silt/clay.

The sedimentary composition and depositional pattern in the area fronting Ballona Creek mouth suggests that Ballona Creek is a primary contributor of sediments.

Basin Areas

Based on data presented in Soule *et al.* (1993), the surface sediments in the basin areas are predominantly silt/clay.

Comparison with Nearby Beaches and Upstream Ballona Creek

Sediments from Dockweiler Beach to the south of Marina del Rey Harbor have a median grain size of about 0.14 mm (Toxcan, 1991), and consist of about 98% sand and 2% fines with about 69% between 0.062 and 0.125 mm. It appears that the median size is appreciably smaller than that at the Ballona Creek mouth which is typically about 0.3 mm.

Sediments trapped on the north fillet bear the signature of those on the upcoast beaches. These sediments have a median size of about 0.2 mm, which is finer than those at the Ballona Creek mouth (0.3mm), but about the same as those found along the inner sides of the jetties in the outer part of the entrance channel (0.2 mm). Detailed distributions are presented in USACE-LAD, 1995.

Long-term data on sediments in the upstream reaches of Ballona Creek are unavailable. Based on the observations, characteristics of sediments in the upstream reaches are in general transient. Therefore, utilizing short-term data is difficult to characterize sediments in the creek. On the other hand, the deposits at the Ballona Creek mouth consist of sediments from repeated deposition from Ballona Creek discharges and thus provide a valid indication of the characteristics of upstream sediments, at least for the sandy loads, over numerous runoff discharge events. More detailed analysis can be found in USACE-LAD, 1998.

5.2.9 Sediment Concentration

Sediment concentration is one of the key elements in the sediment modeling effort. Very limited field data were found for Ballona Creek stream flow and Marina del Rey harbor (UCLA, 1997), and none has been found for the nearshore areas. The related findings from the UCLA report can be summarized as follows.

- Suspended sediment measurements were conducted at the Sawtelle Boulevard Bridge during storms on January 31, 1996 and March 4-5, 1996. Three measurements were made during the first event and four were made during the second event.

- Sediment concentrations for the January 31, 1996 storm were approximately 0.08 g/L, 0.17 g/L, and 0.04 g/L. The corresponding storm flows were 11 m^3/s, 62 m^3/s, and 142 m^3/s, respectively.

- Sediment concentrations for the March 4-5, 1996 storm were approximately 0.17 g/L, 0.09 g/L, 0.06 g/L, and 0.31 g/L. The corresponding storm flows were 17 m^3/s, 147 m^3/s, 51 m^3/s, and 6 m^3/s, respectively.

The above field data were measured at flowrates significantly less than the flowrates for the model calibration and the statistical extreme events. No trend from the measured data can be observed to form a relationship between the sediment concentration and the flowrate in the Ballona Creek. Based on the observations from other rivers, as the flowrate becomes larger, sediment concentration would get higher in the river. However, this relationship is highly dependent on the watershed, river bottom materials and the duration of the storm events. In order to perform numerical model simulations, a linear concentration and flowrate relation was adopted to provide a range of concentration data to be used in the model simulations. This relation between sediment concentration and flowrate was calibrated during the calibration process. The model then predicted sediment volume and pattern consistent with the survey record in the Ballona Creek mouth and Marina del Rey entrance channel areas in the calibration period. The calibrated model was then applied to evaluate the future shoaling with different dredging alternatives. The relationship used in the model between the suspended sediment concentration and the corresponding flowrate in the Ballona Creek is shown in Figure 5.15.

In the numerical model, the total sediment concentration is used as the input condition, while the field concentration data were obtained for suspended load only. Thus the ratio of bed load to suspended load had to be determined. In the report titled "Coastal Sediment Delivery by Major Rivers in Southern California" by Brownlie and Taylor in February 1981, the results of the bedload to suspended load ratios are presented in Table 5.5.

Table 5.5 Ratio of Bedload to Suspended Load for Some Southern California Rivers

River	Ratio
Ventura	0.136
Santa Clara	0.0526
Los Angeles	0.100 (estimate)
Santa Margarita	0.100 (estimate)
San Luis Rey	0.100 (estimate)
San Dieguito	0.100 (estimate)
San Diego	0.100 (estimate)
Tijuana	0.100 (estimate)

Since this information is not available for Ballona Creek, based on Table 5.3, a ratio of 0.1, which was estimated for several other similar Southern California rivers, was selected for the sediment modeling.

5.3 Model Validation

The RMA2/SED2D models were validated for Marina del Rey/Ballona Creek project site through calibration and verification based on the historical field survey data. The model validation strategy and considerations were discussed in Section 5.1.3. Calibration and verification procedures and results are presented in the following sections.

5.3.1 Model Calibration

The RMA2/SED2D model was calibrated against the survey shoaling records in the Marina del Rey entrance channel area for the period from December 1, 1994 to June 30, 1995. The calculated shoaling volume was compared with the survey data to determine the modeling parameters and to justify the input conditions. The primary parameters that need to be determined through the calibration process were the sediment grain size and sediment concentrations. The entrance channel area was divided into sub-areas as shown in Figure 5.16 to better quantify spatial shoaling variations. Area A and Area B cover the south and north entrance channel, respectively. Area G represents the advanced maintenance dredging area at the mouth of Ballona Creek. Area H is the north jetty fillet, which is used as a sand trap for advanced maintenance dredging at the north entrance.

Based on observations, the shoaling in the south entrance channel area was due to the sediment carried downstream by the larger flood flows of the Ballona Creek. However, the shoaling in the north entrance channel area was caused primarily by southward longshore sediment transport, which is a wave dominant and long-term process. Therefore, due to the distinctive characteristics of these two physical phenomena, the calibration process was also divided into two steps. The first was the simulation for the flood dominant, short-term large river discharge events. The second was the simulation for the wave dominant, long-term longshore transport process. The results from the two steps then were combined to obtain the total shoaling volume changes in the harbor and entrance channel areas in the calibration period.

Since large discharge flows bring majority of the sediment downstream to the river mouth, the calibration for the flood dominant events included two time periods, in which four large flow events occurred. The first time period was from January 1, 1995 to January 15, 1995, which included three large flows from the Ballona Creek with the averaged peak flowrate of 430 m^3/s. The second time period was from March 9, 1995 to March 13, 1995, which included the largest peak discharge (675 m^3/s) in the calibration period. The input wave conditions were changed daily. The hydrograph for the entire calibration period is shown in Figure 5.17.

The second step of the calibration involved only base flows from Ballona Creek and averaged wave conditions over the calibration period. Since the input conditions, tide from the ocean boundary, flow from the Ballona Creek and the daily averaged wave condition, did not vary much over the calibration period, the simulation time was set to be five days. The shoaling volume computed for the five days was then linearly extrapolated to obtain the total shoaling volume for the entire calibration period.

The parameters used in the calibration computations are as follows. The sediment grain size was 0.1 mm, which is approximately the logarithmic average of the sampled sediment size in the Marina del Rey Harbor area, including the south and north entrance channels (USACE-LAD, 1998). The estimation of sediment concentration in the Ballona Creek was based on a linear relationship between the sediment concentration and the flowrate, which was discussed in Section 5.2.9. The relation was calibrated in the calibration process, and the result is shown in Figure 5.15. Since the data from Figure 5.15 represent suspended sediment only, the chosen values were then multiplied by 1.1 to obtain the total sediment concentration. Inman and Bagnold's method (Dean, G. Robert and Dalrymple, A. Robert, 1996) was used to obtain the concentration field in the surf zone. A detailed description of this method is presented in Appendix 1. In the calibration simulation, the dimensionless sediment concentration was chosen to be 0.006 for the surf zone. The diffusion coefficient was selected as 30 m^2/s for the calibration simulations.

The shoaling patterns from the first step of the calibration are shown in Figures 5.18 to 5.20. Figure 5.18 shows the bottom elevation change after the first time period (January 1, 1995 to January 15, 1995). Figure 5.19 shows detailed bathymetric changes near the entrance to Ballona

Creek during the same period. It can be seen from these two figures that the large river discharges bring the sediment to the south and main entrance channel areas. Figure 5.20 shows the bed change during the second time period (March 9, 1995 to March 13, 1995) of the first step of the calibration, where again the river flood dominated sediment pattern can be seen.

Figure 5.21 shows the bed change from the calibration for sediment transport due to longshore currents. Figure 5.22 shows detailed bathymetric changes near the entrance to the Ballona Creek during the same period. The results illustrate accumulation of longshore sediment in the north entrance channel area.

The modeled shoaling patterns are compared with the survey records in Figure 5.23 and 5.24. In these two figures, computed bed changes are plotted on the left-hand side and the measured bed changes are on the right-hand side. The model results shown in these figures did not include the sediment deposition due to longshore transport. Hence, comparison between model prediction and survey records should only be made for the south entrance channel area. The predicted sediment depositional patterns are similar to those survey observations. In addition, maximum depositions were observed at the vicinity of the south-side of the tip of the middle jetty for both computed and measured results.

The results of the shoaling volume computation from the two steps of the calibration were combined to present the sediment volume changes during the calibration period. Table 5.6 shows the shoaling volume comparisons between the measured and computed results.

Table 5.6 Shoal Volume Comparison Between Field Data and Model Results for Calibration Period (December 1994 to June 1995) (m^3)

	Area A		**Area B**		**Area G**		**Area H**	
Survey Data	42,860		29,477		8,362		26,653	
Numerical Model Results	Step 1*	Step 2*	Step 1	Step 2	Step 1	Step 2	Step 1	Step 2
	43,659	1,615	3,915	21,426	19,764	1,698	675	26,076
	Total		Total		Total		Total	
	45,274		25,341		21,462		26,751	
Model Results Relative to Survey Data	6%		-14%		156%		1%	

* Step 1--- river discharge, Step 2--- longshore transport.

From Table 5.6, it can be seen that the model simulation achieved good results in predicting the total shoaling volumes in the federal channel (Area A and B) with differences of 6%, -14%. Area G and H differences were 156% and 1%, respectively. Thus, based on the sediment pattern and volume comparisons between the model and the survey results, the calibration parameters discussed above were considered appropriate for the Marina del Rey entrance channel sediment modeling situations, and were used for the simulations of the alternatives.

5.3.2 Model Verification

For verification of the calibrated RMA2/SED2D model, the same parameters used for calibration were used to predict the shoaling pattern and volume for the period from April 1996 to September 1996. This verification period was during a dry season. Similar to the calibration process, the verification process was also divided into two steps, 1) the verification for the Ballona Creek flood events, and 2) the verification for the longshore transport processes.

The first step of the verification was conducted for the single storm event (between April 9, 1996 to April 13,1996) during the verification period. The shoaling patterns are shown in Figures 5.25 and 5.26. Figure 5.25 shows the bottom elevation change after the storm event. Figure 5.26 shows detailed bathymetric changes near the entrance to the Ballona Creek during the same period. The hydrograph of this period is shown in Figure 5.27, which indicates the discharge for the storm event had a relatively small flowrate of 92 m^3/s compared with the one-year return period flood of 220 m^3/s. From Figure 5.25 and 5.26 it is evident that the flow with this magnitude does not carry a lot of sediment to the Ballona Creek mouth area. Some of the sediment settled in the upstream channel. The sediment in the entrance channels were primarily through local sediment redistribution.

The second step of the verification was the simulation with base flow from Ballona Creek and averaged wave condition over the entire verification period. Figure 5.28 shows the bed change from the second step of the verification and Figure 5.29 shows detailed bathymetric changes near the entrance to the Ballona Creek during the same period. Sediment accumulation occurred mainly near the Marina's north entrance channel.

Results for the shoaling volume are shown in Table 5.7. The comparison in Area B and H between the model result and the field data shows very good agreement. In Area G, the model did not predict the negative volume. However, the volume in the area was relatively small compared to the other three areas. In Area A, the model predicted less shoaling volume compared with the survey record. This indicates that the shoaling volume in the area may not have been solely due to the April flood. Another possible source of shoaling in that region could be the local sediment redistribution as discussed previously. In view of the overall comparisons, the calibrated model performed well in reproducing the shoaling pattern and volume over a different time period.

Table 5.7 Shoal Volume Comparison Between Field Data and Model Results for the Verification Period (April 1996 to September 1996) (m^3)

	Area A		Area B		Area G		Area H	
Survey Data	4,909		4,851		-1,208		5,580	
Numerical Model Results	Step 1	Step 2	Step 1	Step 2	Step 1	Step 2	Step 1	Step 2
	1,188	~0	54	4,912	1,323	~0	-27	6,086
	Total		Total		Total		Total	
	1,188		5,007		1,323		6,086	
Model Results Relative to Survey Data	-76%		2%		---		9%	

The results from the model validation show that the RMA2/SED2D model is capable of satisfactorily predicting the shoaling pattern and volume changes in Marina del Rey Harbor and entrance channel areas.

5.3.3 Model Sensitivity

The parameters discussed in Section 5.3.1, such as sediment concentration, grain size and diffusion coefficients, were determined through the calibration process. In this section, simulations with or without wave conditions are compared to see the wave effects on the model simulation. Also, the effects from different tidal conditions are discussed.

Effects of the Waves

Simulations were conducted in the calibration process for both with and without wave conditions to compare the wave effects on the sediment transport in the Marina del Rey entrance channel areas. The bed changes in the second time period of the calibration are shown in Figure 5.20. The bed changes for the same simulation with no-wave condition are shown in Figure 5.30. The comparison between Figure 5.20 and Figure 5.30 shows that with the waves (Figure 5.20), the shoaling area is larger. It shows waves can keep more sediment suspended, and with the wave action, the sediment can spread out in a larger area. Simulations for other cases showed the similar results.

Spring Tide vs. Neap Tide

All simulations were conducted with the input tidal condition as either spring tide or neap tide except the calibration and verification simulations, in which the tidal conditions in the respective periods were used. For a 5-year flood, shoaling area is larger with spring tide than neap tide, which can be seen by comparing Figure 5.31 (spring tide) with Figure 5.32 (neap tide). The phase used in the computation between the tides and flood flows was that the flood peak reaches upstream Ballona Creek grid boundary when the tide is at the second low tide level along the ocean-side grid boundary. However, for larger floods, e.g. 25-year flood, different tides would not make much difference as shown in Figure 5.33 (spring tide) and 5.34 (neap tide). This shows that with large discharges from Ballona Creek, the sediment transport in the entrance channels is dominated by the river flow.

5.4 Analysis of Sediment Depositional Patterns and Rates

The purpose of this numerical modeling effort was to establish and predict sediment transport rates and sediment depositional patterns within the Marina del Rey entrance channels. After the model was calibrated and verified, it was used to simulate the sediment transport in the entrance channel area for different alternatives with specified environmental conditions. The results were analyzed to compare the shoaling rates among different alternatives.

5.4.1 Description of Model Simulation Cases

The simulation conditions for Marina del Rey Harbor sediment transport study are listed in Table 5.8. The simulations were conducted for both spring and neap tides, with discharges from Ballona Creek for storm return periods of 5-, 25- and 100 years. Three bathymetric configurations -Existing, Alternative 1 and Alternative 2 were modeled. Averaged wave conditions between 1991 and 1997 were used. The selection of the simulation conditions was based on the scope of this study.

Table 5.8 Marina del Rey Sediment Study RMA2 and SED2D Simulation Conditions

Simulation No.	Tidal Condition	Ballona Discharge		Bathymetric condition	Wave Condition	Figure No. For bed change results
		Return Period	Peak Flow Rate (m^3/s)			
1	Spring	5-yr	500	Existing	7-yr average wave	5.31, 5.35
2	Spring	25-yr	780	Existing	"	5.33, 5.36
3	Spring	100-yr	1020	Existing	"	5.37
4	Spring	5-yr	500	Alt. 1	"	5.41, 5.52
5	Spring	25-yr	780	Alt. 1	"	5.39, 5.53
6	Spring	100-yr	1020	Alt. 1	"	5.42
7	Spring	5-yr	500	Alt. 2	"	5.43, 5.54
8	Spring	25-yr	780	Alt. 2	"	5.40, 5.55
9	Spring	100-yr	1020	Alt. 2	"	5.44
10	Neap	5-yr	500	Existing	"	5.32, 5.56
11	Neap	25-yr	780	Existing	"	5.34, 5.57
12	Neap	100-yr	1020	Existing	"	5.46
13	Neap	5-yr	500	Alt. 1	"	5.47, 5.58
14	Neap	25-yr	780	Alt. 1	"	5.45, 5.59
15	Neap	100-yr	1020	Alt. 1	"	5.48
16	Neap	5-yr	500	Alt. 2	"	5.49, 5.60
17	Neap	25-yr	780	Alt. 2	"	5.50, 5.61
18	Neap	100-yr	1020	Alt. 2	"	5.51

For the above simulations, the selected phase between the tides and flood flows was that the flood peak reaches upstream Ballona Creek grid boundary when the tide is at the higher low tide level (at the time of 18.75 hour in the simulation time period) along the ocean side grid boundary. This phase difference between flood and tide was selected because it represents the worst condition in terms of the Marina del Rey Harbor contamination concentration (USACE-

LAD, 1998). The simulation time was 55 hours. The figure numbers for each simulation are also listed in Table 5.8. These figures are contour plots showing the bottom elevation changes in the Marina del Rey Harbor entrance channel areas. The second figure numbers in the right column of Table 5.8 represent the figures which show the bed changes for 5- and 25-year flood with the same plotting scale as for the 100-year flood to compare the effects due to different floods. The other figures have different plotting scales for the respective flood discharges.

5.4.2 Results

The sediment transport rates and patterns in the Marina del Rey entrance channels are affected by waves, tides, discharge flows from Ballona Creek, and different dredging alternatives. The effects from waves and tidal conditions are discussed in Section 5.3. In this section, the model sediment transport results for flood discharge from Ballona Creek for return periods of five (5), twenty-five (25) and one hundred (100) years and for existing, Alternative 1, and Alternative 2 bathymetric conditions will be analyzed. The predictions on the entrance channel shoaling will also be presented.

Results from Different Flood Discharges

The simulations with flood discharges of 5-, 25-, and 100-year return period indicate that larger floods from Ballona Creek bring down significantly more sediment to the Ballona Creek mouth and Marina del Rey south entrance channel area as shown in Figures 5.35, 5.36 and 5.37 for the existing condition. Similar results can be found for Alternative 1 and Alternative 2. The shoaling can occur almost at the same time the peak flow occurs. Figure 5.38 shows the time history of the bed change at a location near the tip of the middle jetty in the south entrance channel (Gage 1593 in Figure 5.2) with existing bathymetry, for 5-, 25- and 100-year return period discharges. The flood peak for each simulation was at 18.75 hour at the Ballona Creek upstream boundary (shown in Figure 5.11). It can be seen from Figure 5.38 that the majority of the sediment was deposited shortly after the flood flow started.

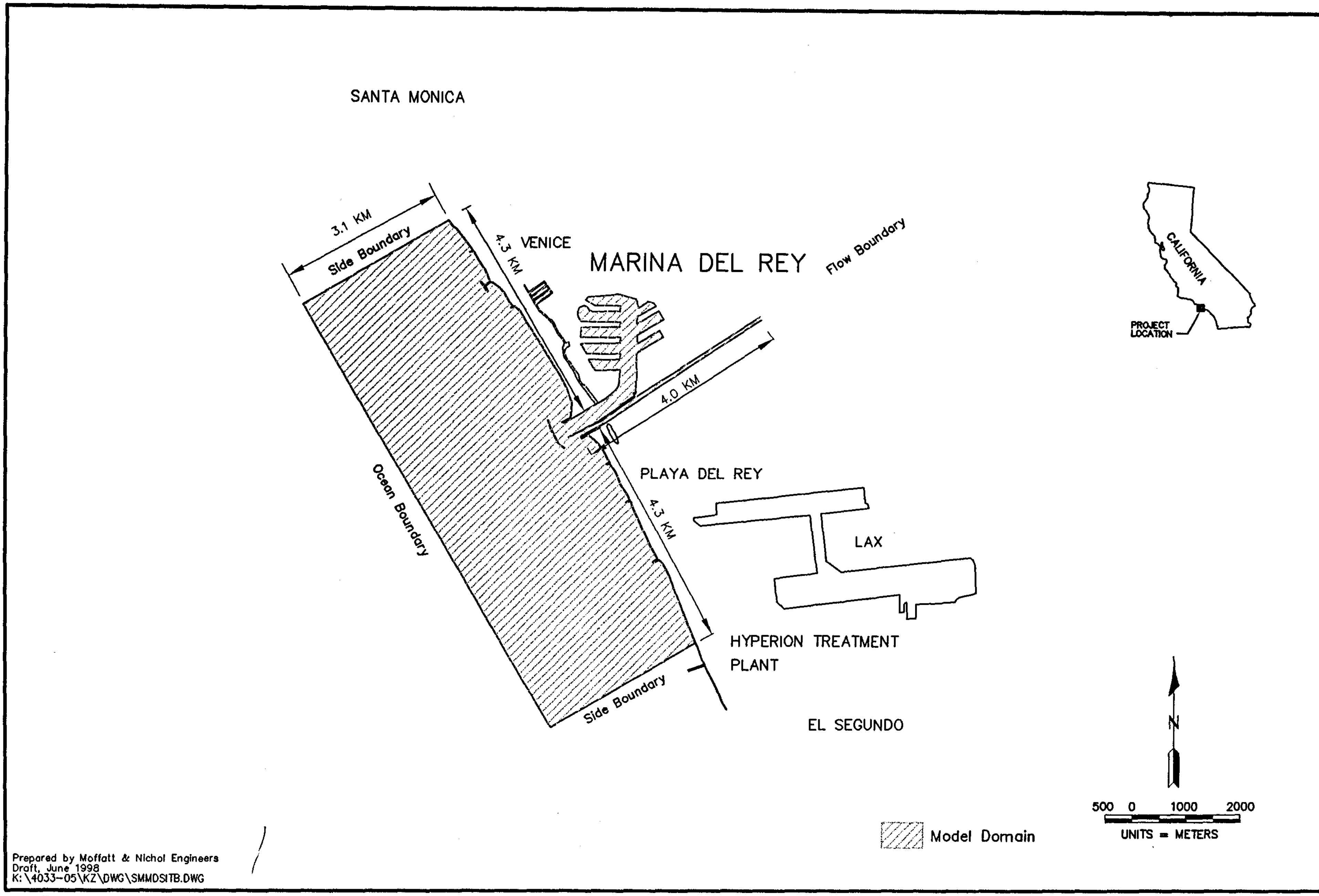
Comparisons Between the Alternatives

The modeling results show that the dredging alternatives can significantly modify the shoaling patterns and the volume inside the Marina del Rey entrance channel area. For example, the result for existing condition with spring tide and 25-year flood (Figure 5.33) shows there is a significant sediment volume that is pushed into south and main entrance channel area, while at the Ballona Creek mouth area, there is relatively less shoaling. For Alternative 1 and Alternative 2 with spring tide and 25-year flood (Figure 5.39 and 5.40), the sediment accretion started at the Ballona Creek mouth because of the deeper depth in the river mouth area so that less sediment is transported to the south and main entrance channel areas.

To better quantify the comparisons between existing condition, Alternative 1 and Alternative 2, the bottom elevations and their changes for a typical cross-section from the mouth of the Ballona Creek to the Federal breakwater are plotted in Figures 5.62 to 5.64. The cross-section location (cross-section A-A) is shown in Figure 5.2. This cross-section cuts through major bed changes due to flood flows for all three bathymetric conditions. In Figures 5.62 to 5.64, the top portion of the graph shows the original channel bottom elevation and the elevation after the storm, and the bottom portion shows the elevation changes relative to the respective original elevations. The far right points in the plots represent the location near the Ballona Creek mouth but still in the river channel. The far left points are at the foot of the breakwater. The second points from right are at the boundary where the proposed dredging starts near the Ballona Creek mouth. The Federal South Entrance Channel limits (east and west boundaries of the navigation channel) are also shown in Figures 5.62 to 5.64. Each figure shows the comparisons among the alternatives with the same flood discharge. All the results shown were obtained with the spring tide ocean boundary condition.

The following observations are consistent with all three flood conditions: First, inside the Federal South Entrance Channel, the existing condition has the largest sediment deposition. The bottom change is nearly two times larger than Alternatives 1 and 2, which means the implementation of Alternative 1 and 2 would reduce the shoaling rate by about fifty percent. Second, Alternative 1 and 2 have comparable sediment deposition inside the Federal Channel. The shoaling rate difference between these two alternatives in the Federal Channel is very small. Alternative 2 has larger shoaling volume near the Ballona Creek mouth due to its deep bottom (dredged to 9 meters below MLLW). The bed change in the mouth area is approximately 1 meter, 3 meters and 5 meters for 5-, 25- and 100-year flood, respectively.

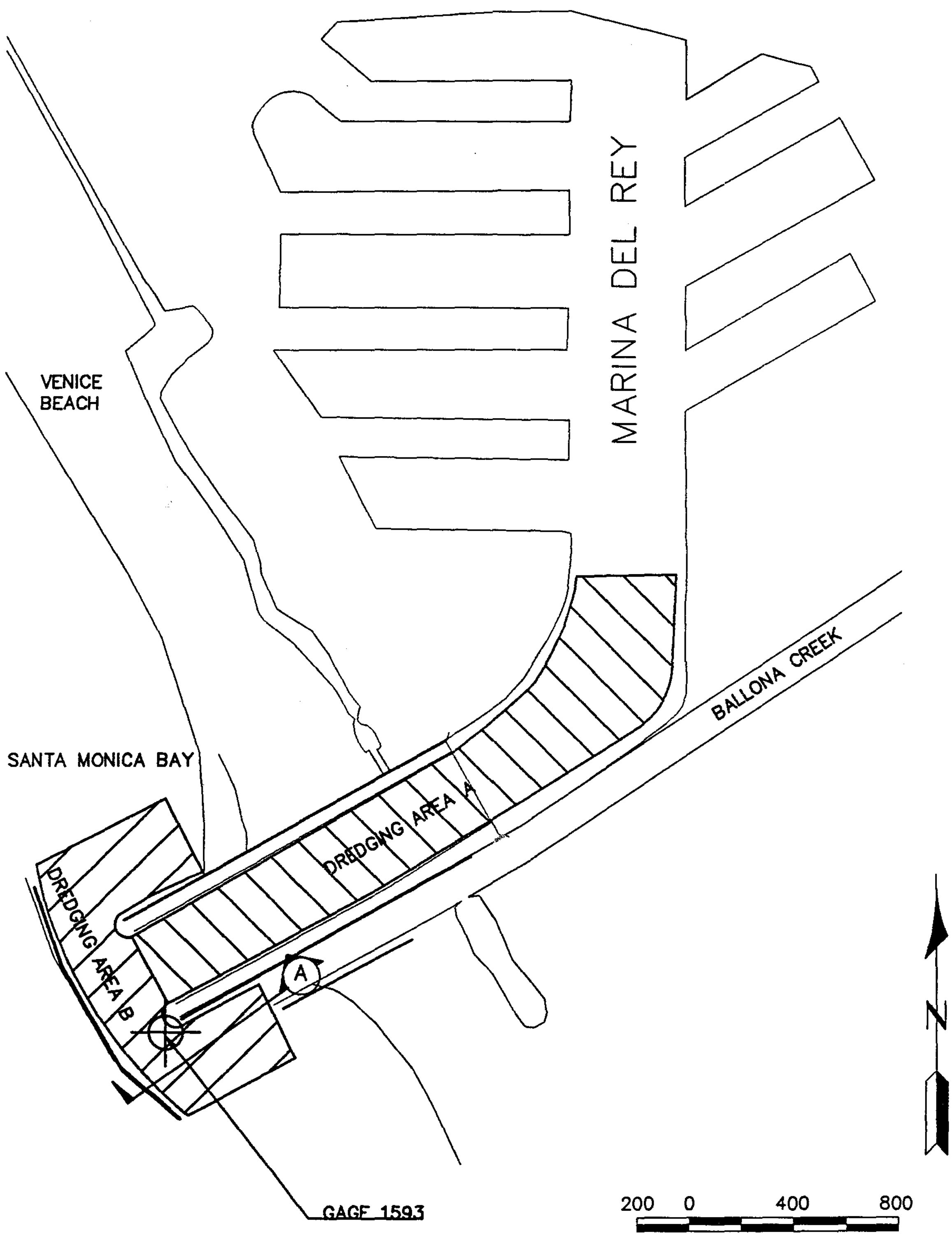
In summary, existing condition has the largest shoaling rate in the south entrance channel. Both dredging alternatives would significantly reduce the shoaling in the south and main entrance channels. In terms of shoaling rate inside the Federal Channels, Alternative 1 and 2 would have no significant differences.



Marina del Rey and Ballona Creek Sediment Transport Studies

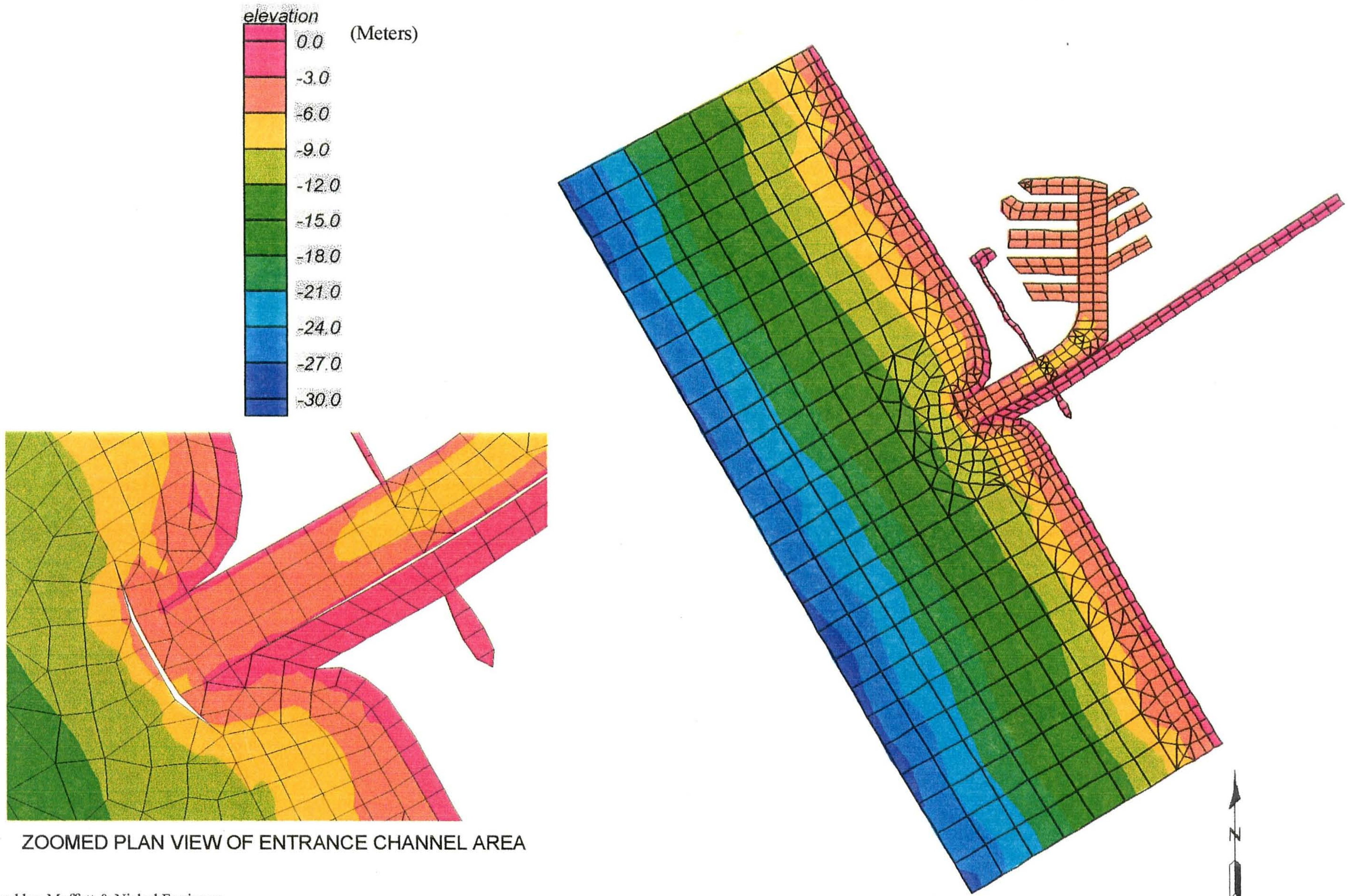
Modeling Domain

Figure 5.1



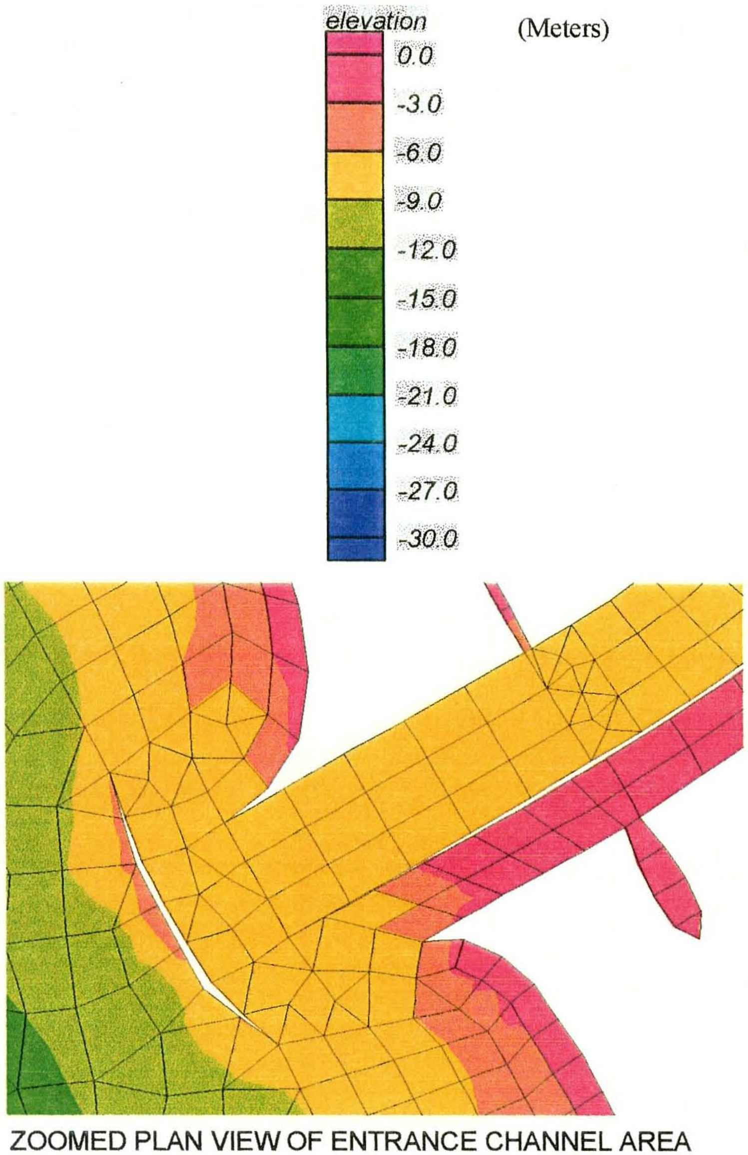
Prepared by Moffatt & Nichol Engineers
Draft, June 1998
k: \4033-05\coastal\dwg\dredge.dwg

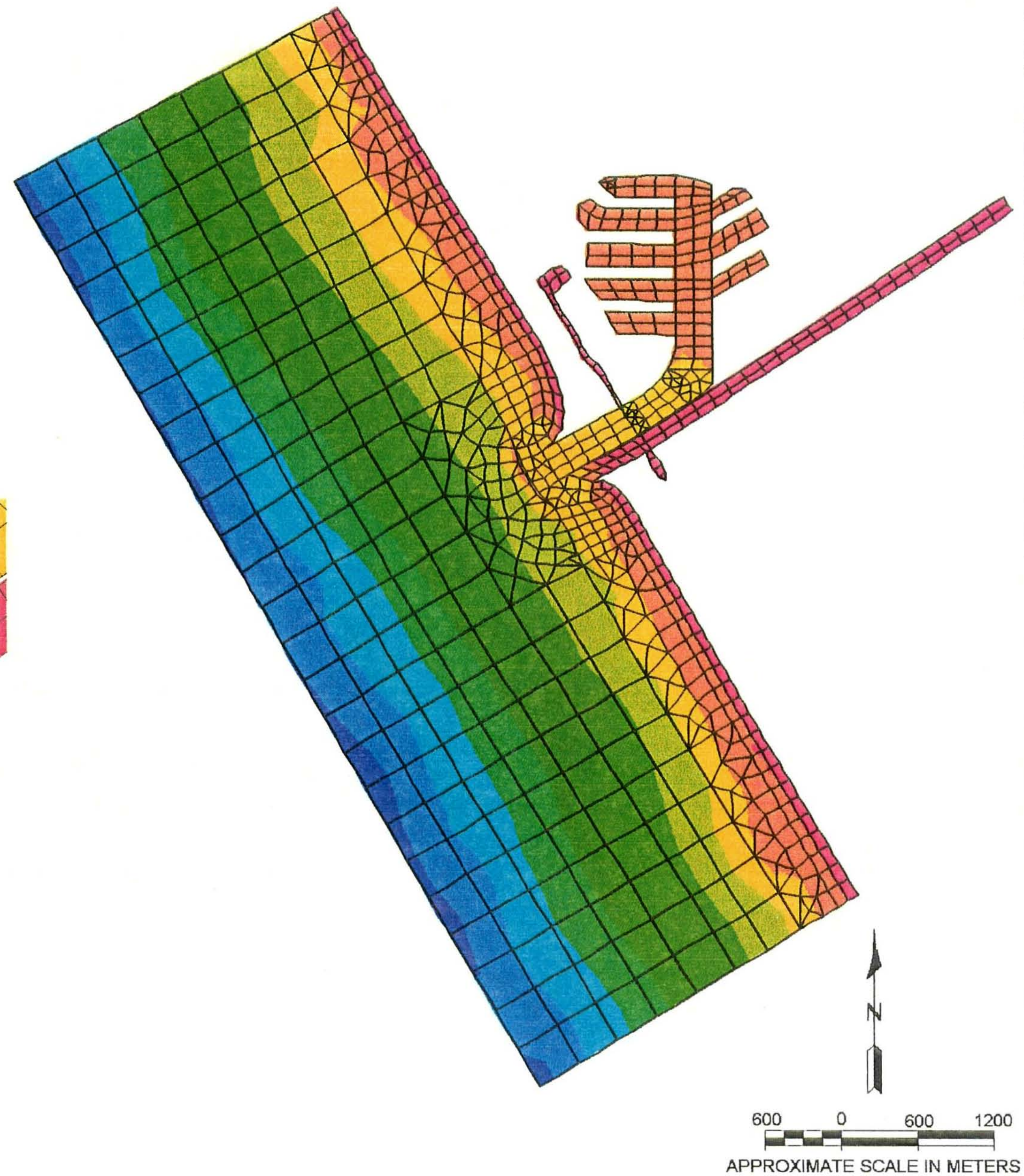
Marina del Rey and Ballona Creek Sediment Transport Studies	Dredging Area	Figure 5.2



Prepared by: Moffatt & Nichol Engineers
Draft, May 1998
k:\4033-05\graphs\Bathmtry.ppt
from Si.geo

Marina del Rey and Ballona Creek Sediment Transport Studies	Existing Bathymetry and Grid	Figure 5.3



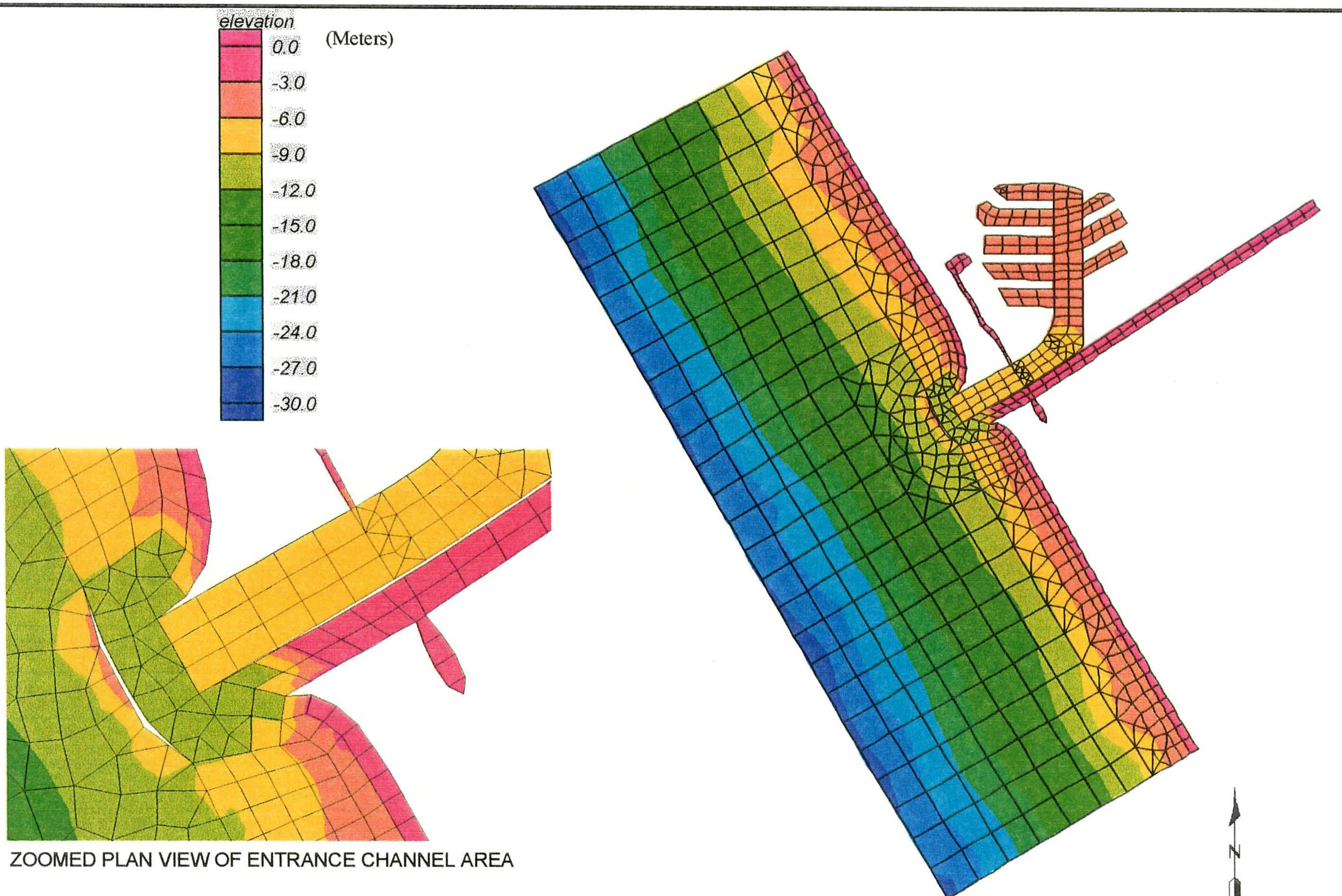


Prepared by: Moffatt & Nichol Engineers
Draft, May 1998
k:\4033-05\graphs\Bathmtry.ppt
from Si_1.geo

Marina del Rey and Ballona Creek Sediment Transport Studies

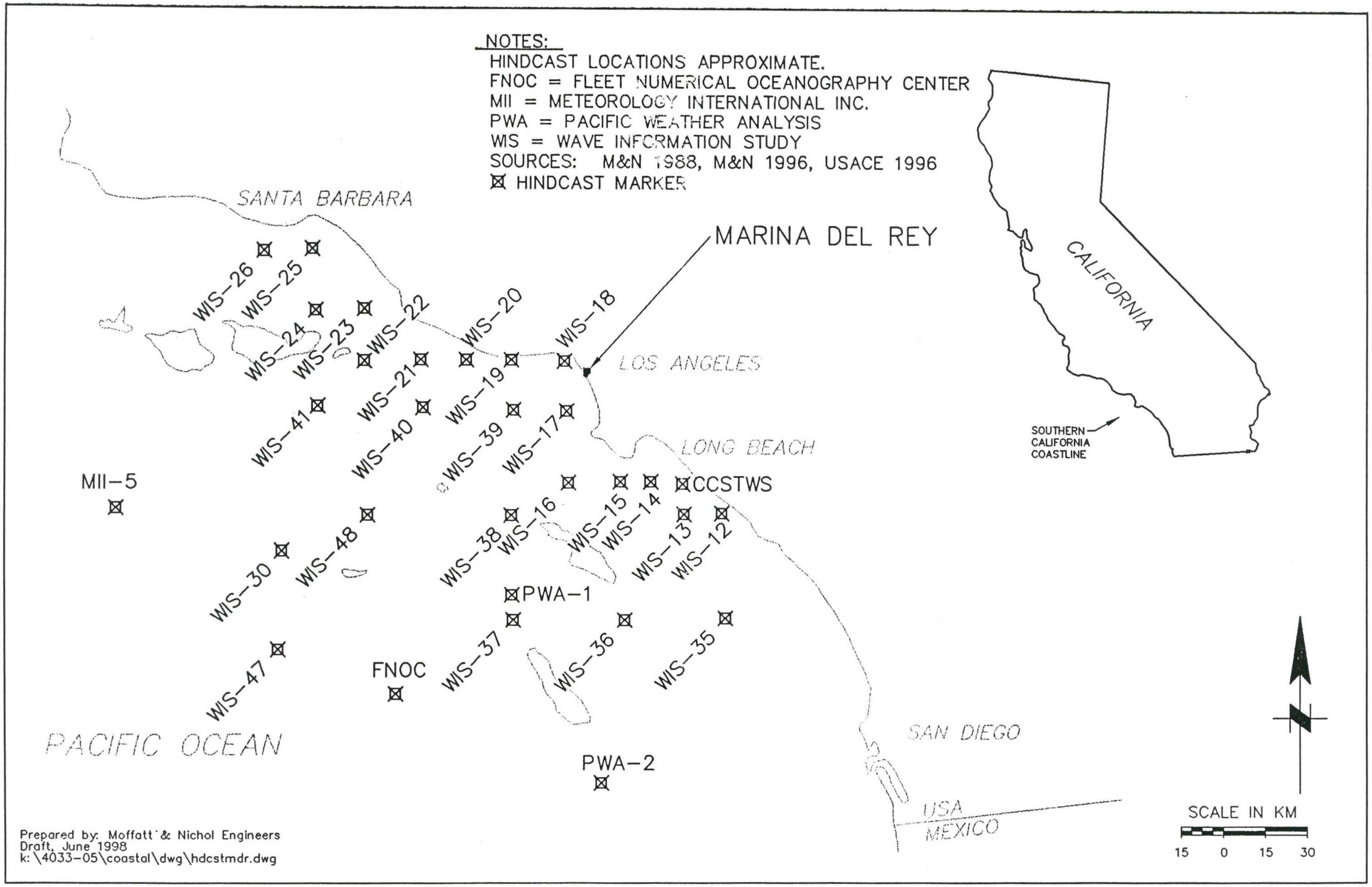
Alternative 1
Bathymetry and Grid

Figure
5.4



Prepared by: Moffatt & Nichol Engineers
Draft, May 1998
k:\4033-05\graphs\Bathmtry.ppt
from Si_deep2.geo

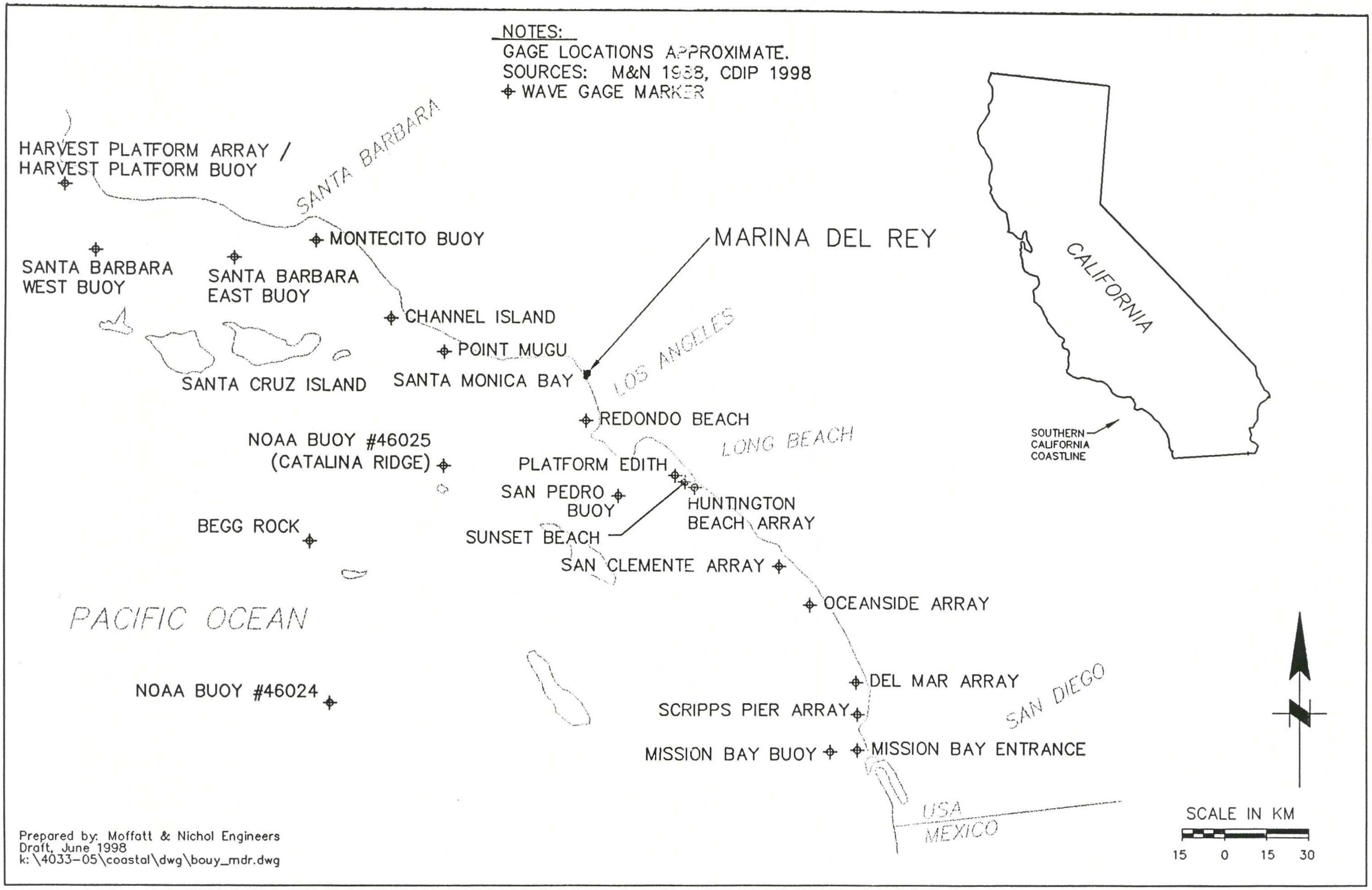
Marina del Rey and Ballona Creek Sediment Transport Studies	Alternative 2 Bathymetry and Grid	Figure 5.5



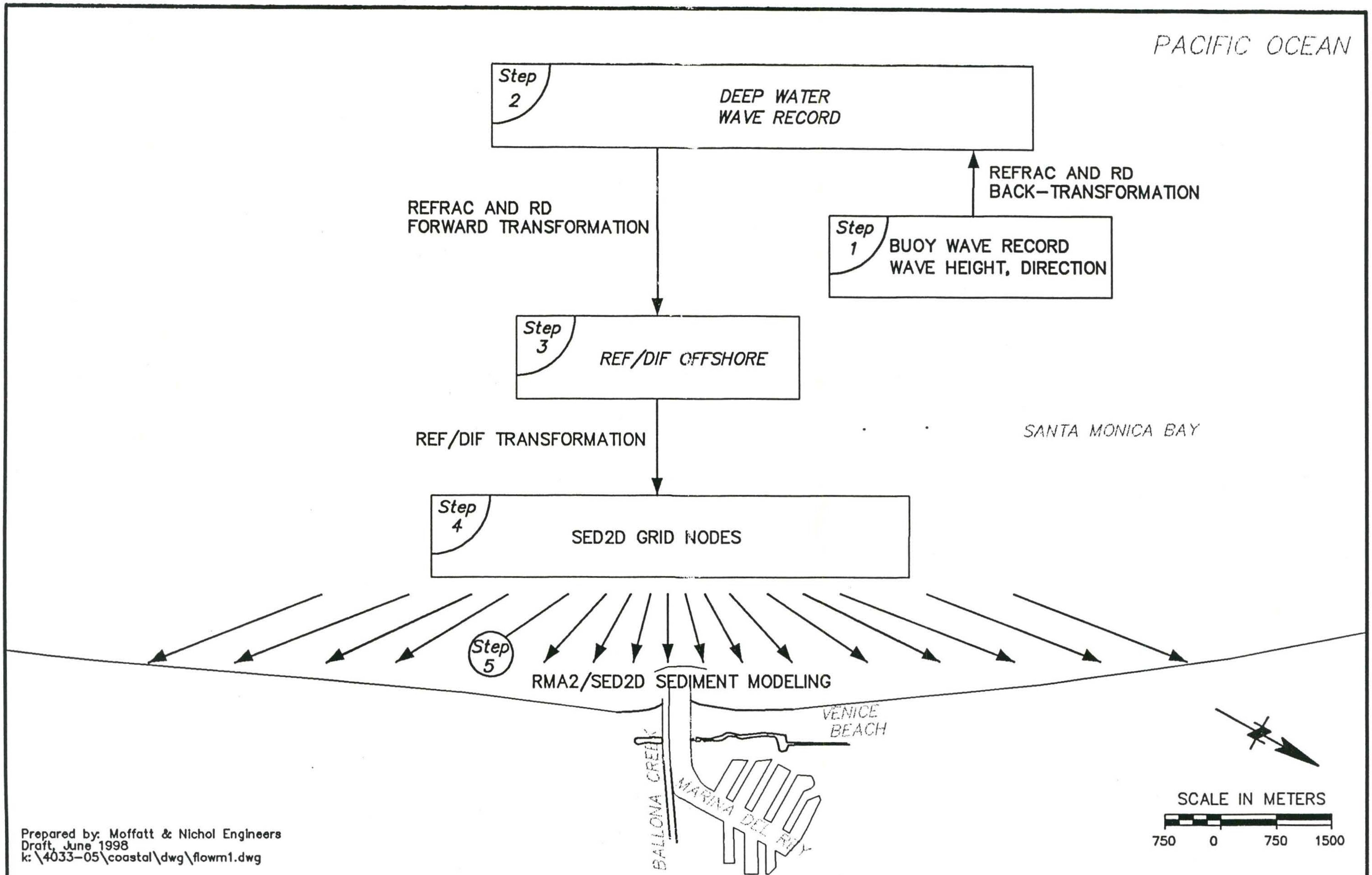
Marina del Rey and Ballona Creek Sediment Transport Studies

Hindcast Location Map

Figure 5.6



Marina del Rey and Ballona Creek Sediment Transport Studies	Wave Gage Location Map	Figure 5.7



Prepared by: Moffatt & Nichol Engineers
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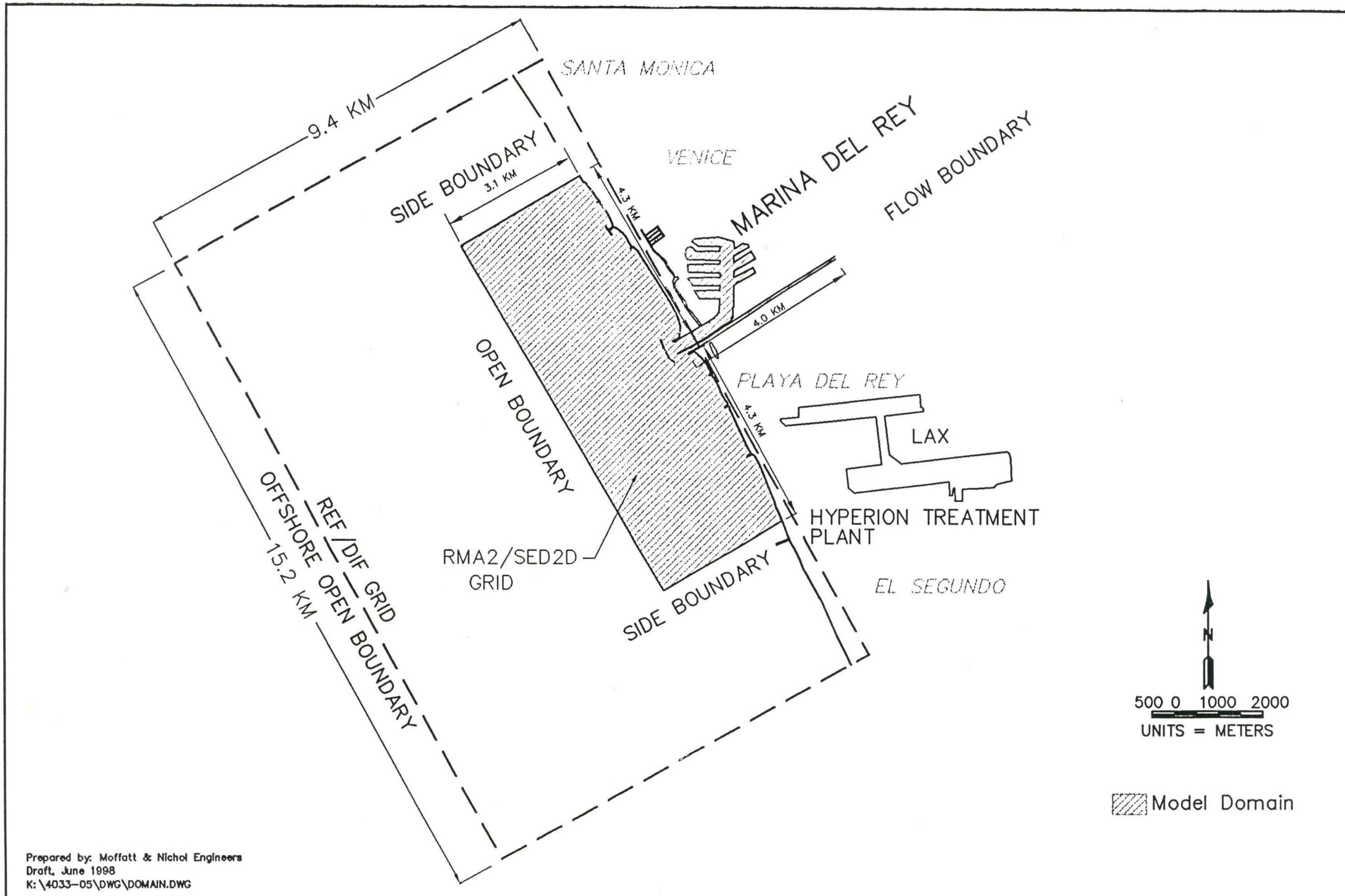
k:\4033-05\coastal\dwg\flowm1.dwg

Marina del Rey and Ballona Creek Sediment Transport Studies	Wave Data Processing Flow Chart	Figure 5.8



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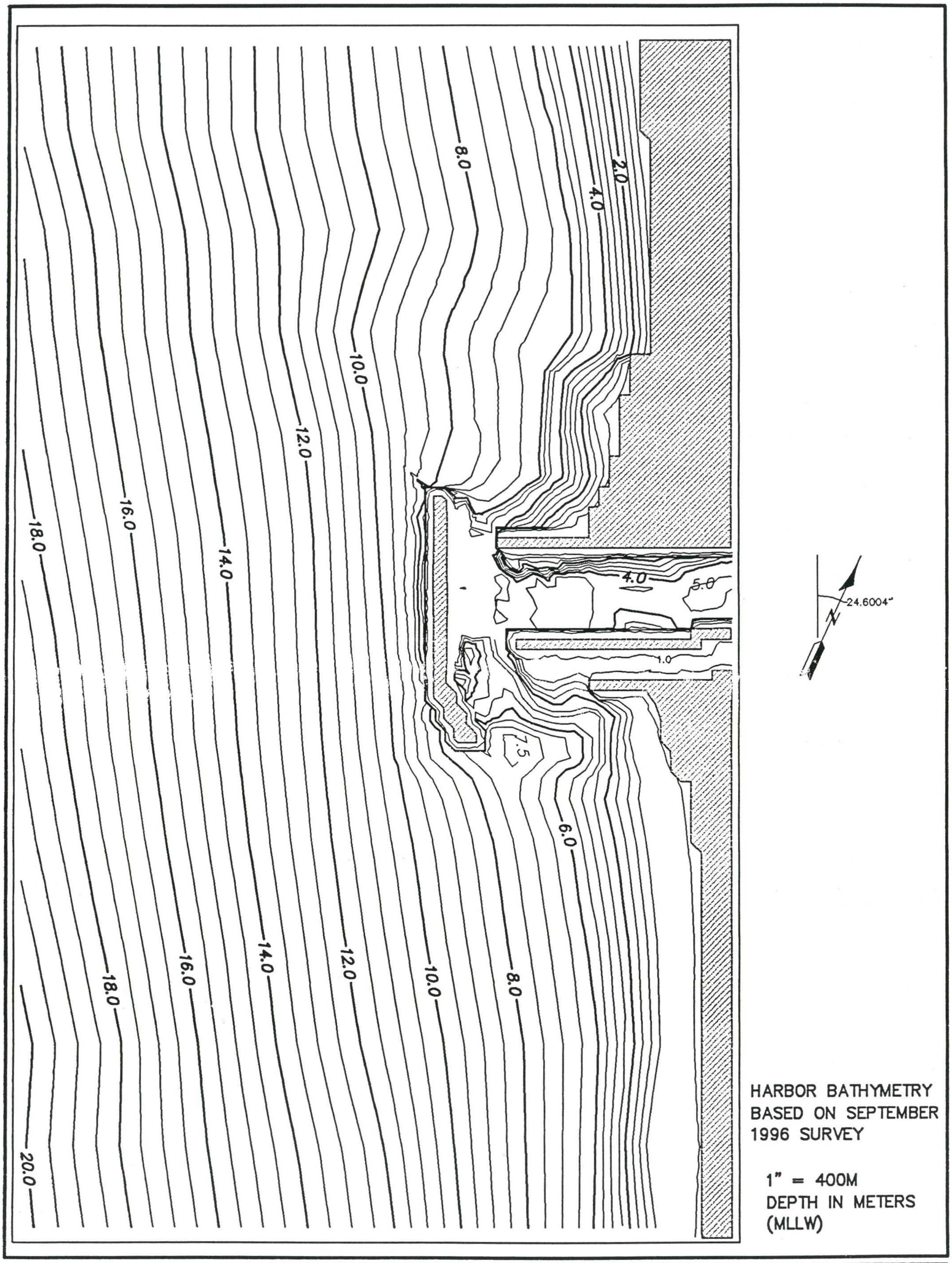
K:\4033-05\DWG\DOMAIN.DWG

Marina del Rey and Ballona Creek Sediment Transport Studies	Modeling Domain	Figure 5.9



Marina Del Rey Sedimentation Study	Bathymetry of REF/DIF Grid	Figure 5.10

K:\4033-05\COASTAL\KZ\DWG\GRIDBATH.DWG

Ballona Creek Hydrographs for 5, 25 and 100 Year Flood

Time at Peak Flow: T_{peak}=18.75 hour

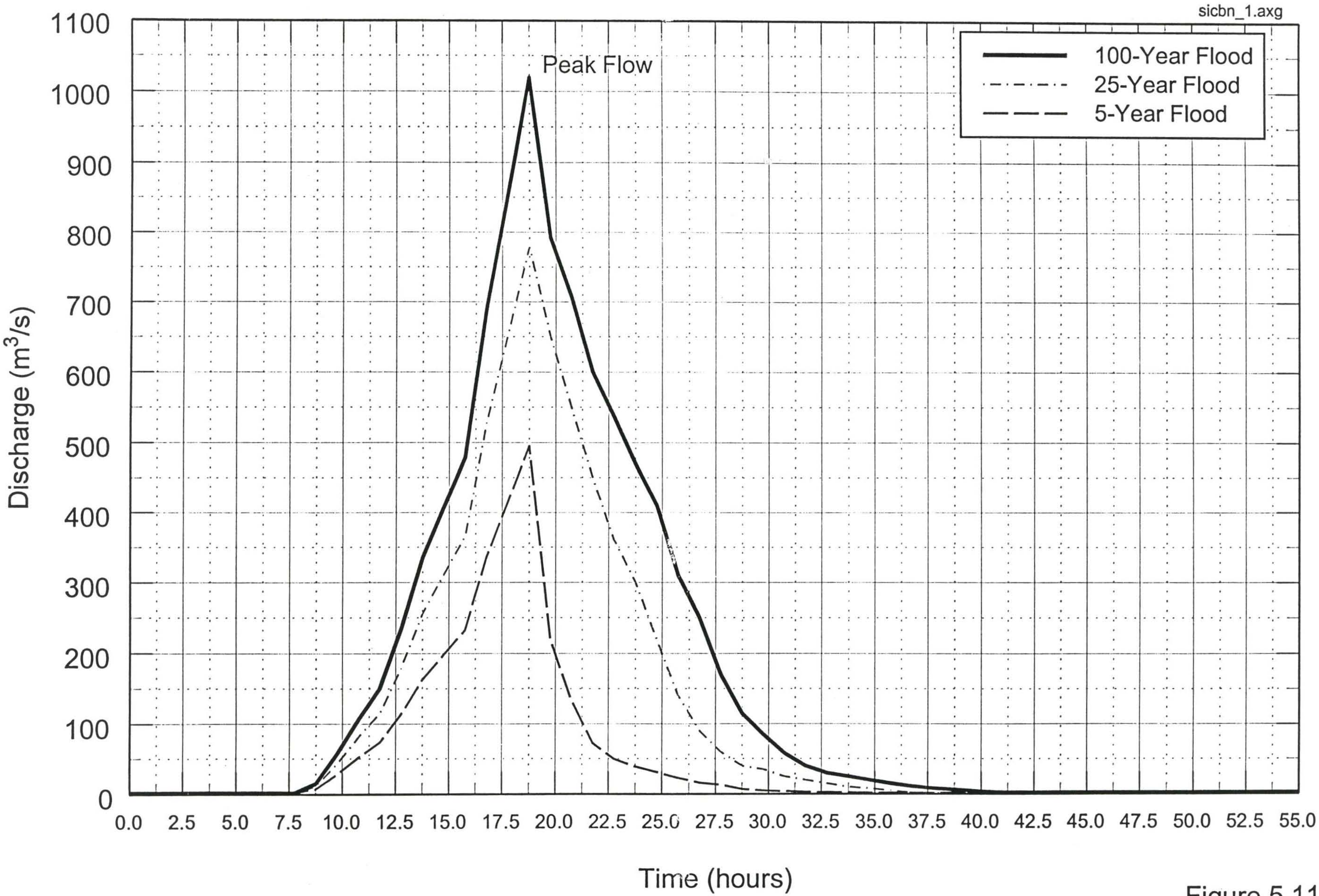
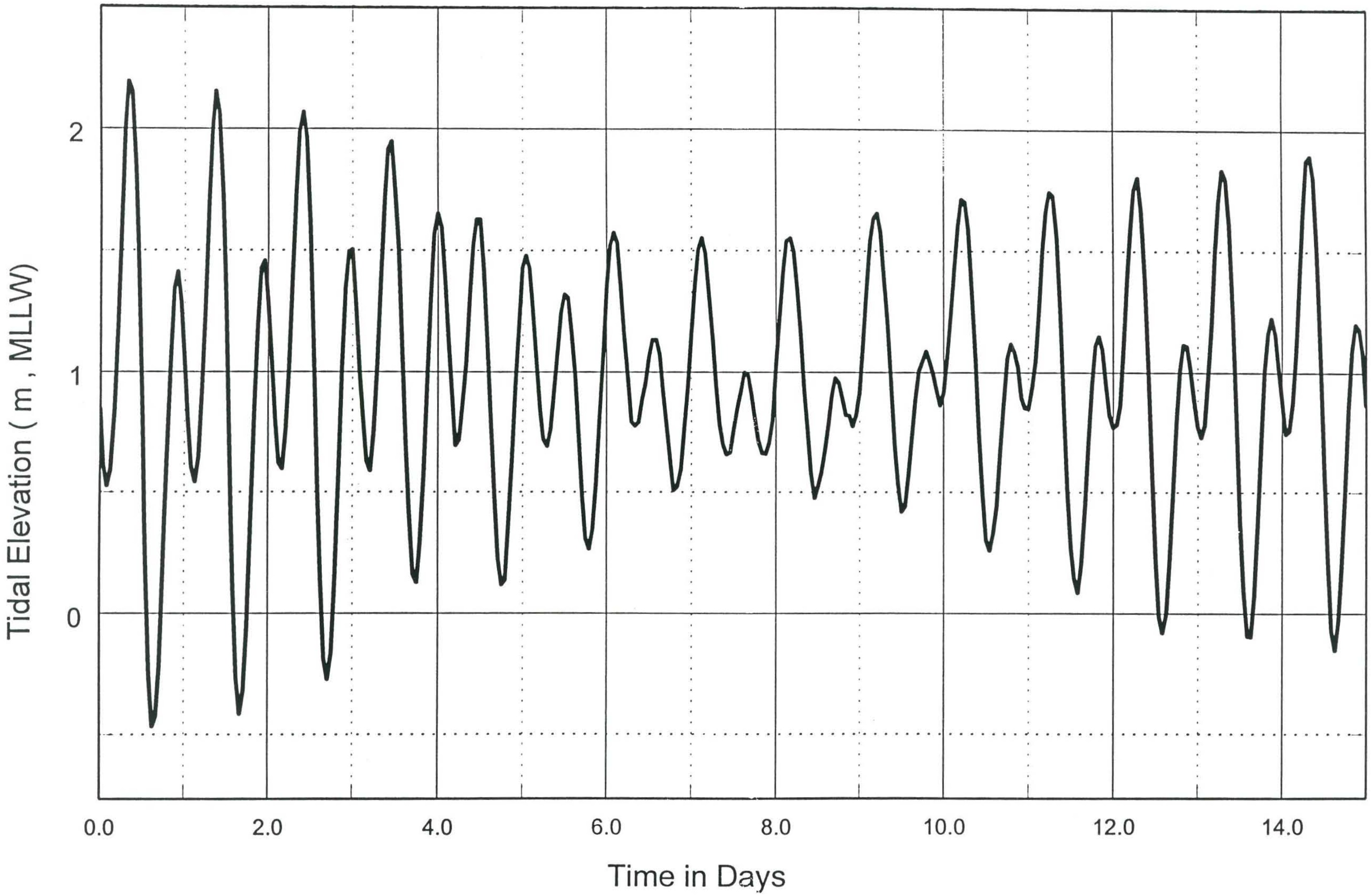


Figure 5.11



Prepared by: Moffatt & Nichol Engineers

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f:/4033_05/coastal/elev.axg

Marina del Rey Sediment Study	NOAA Measured Tidal Levels at LA Outer Harbor From 01/01/95 to 01/15/95	Figure 5.12

Selected Spring Tide For Modeling

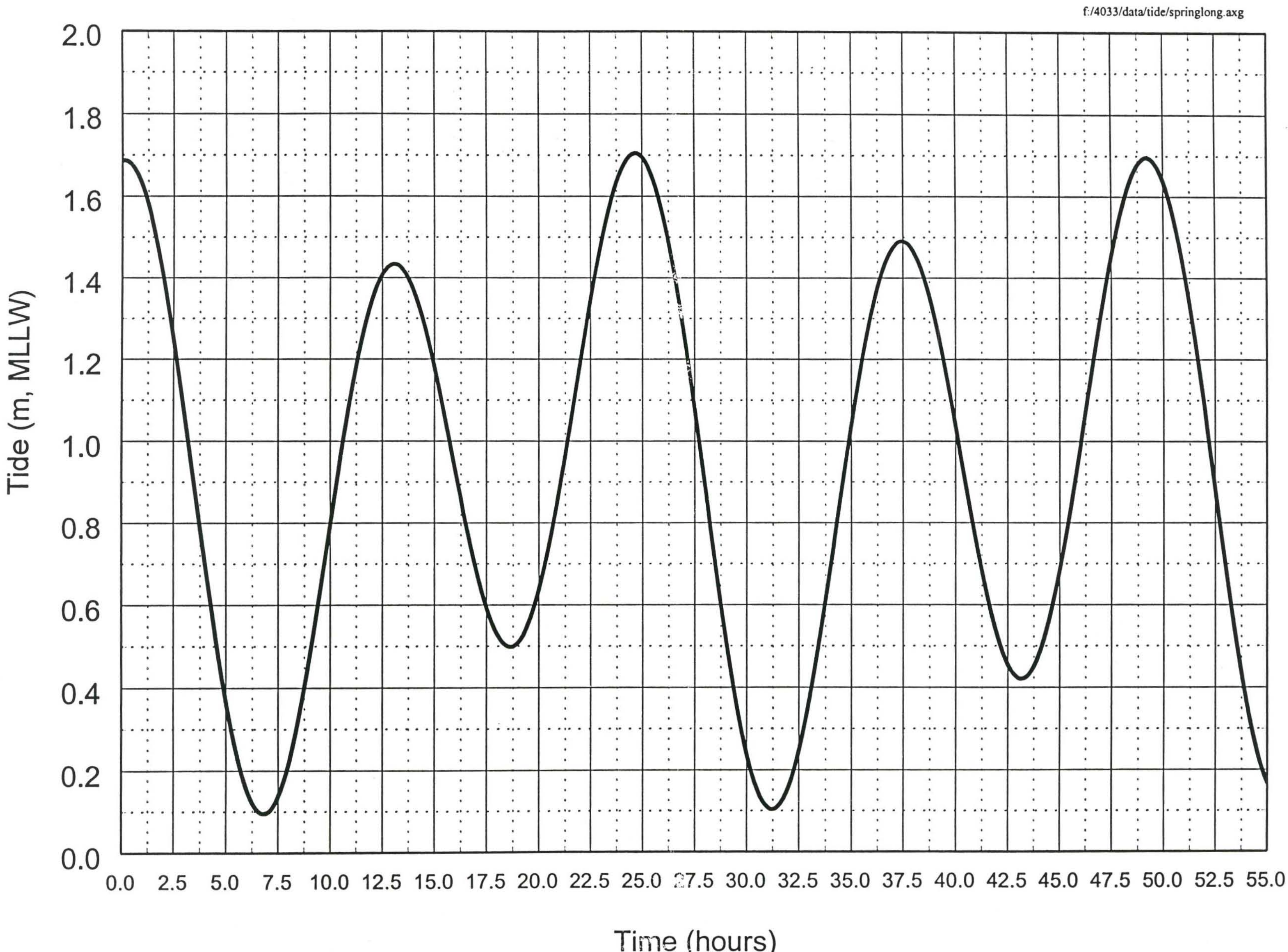


Figure 5.13

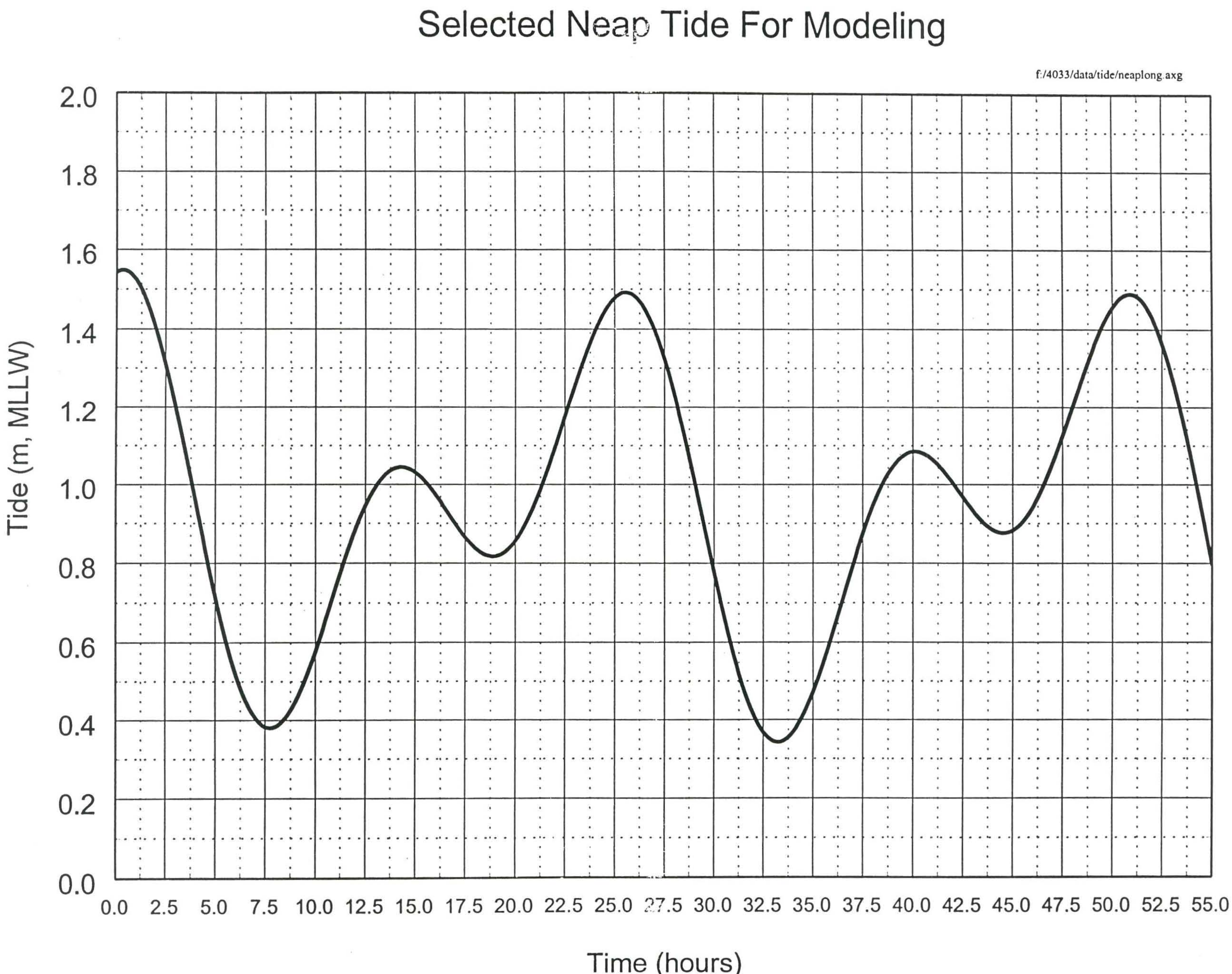

Selected Neap Tide For Modeling
f:/4033/data/tide/neaplong.axg
Tide (m, MLLW)
2.0
1.8
1.6
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0.0
0.0 2.5 5.0 7.5 10.0 12.5 15.0 17.5 20.0 22.5 25.0 27.5 30.0 32.5 35.0 37.5 40.0 42.5 45.0 47.5 50.0 52.5 55.0
Time (hours)

Figure 5.14

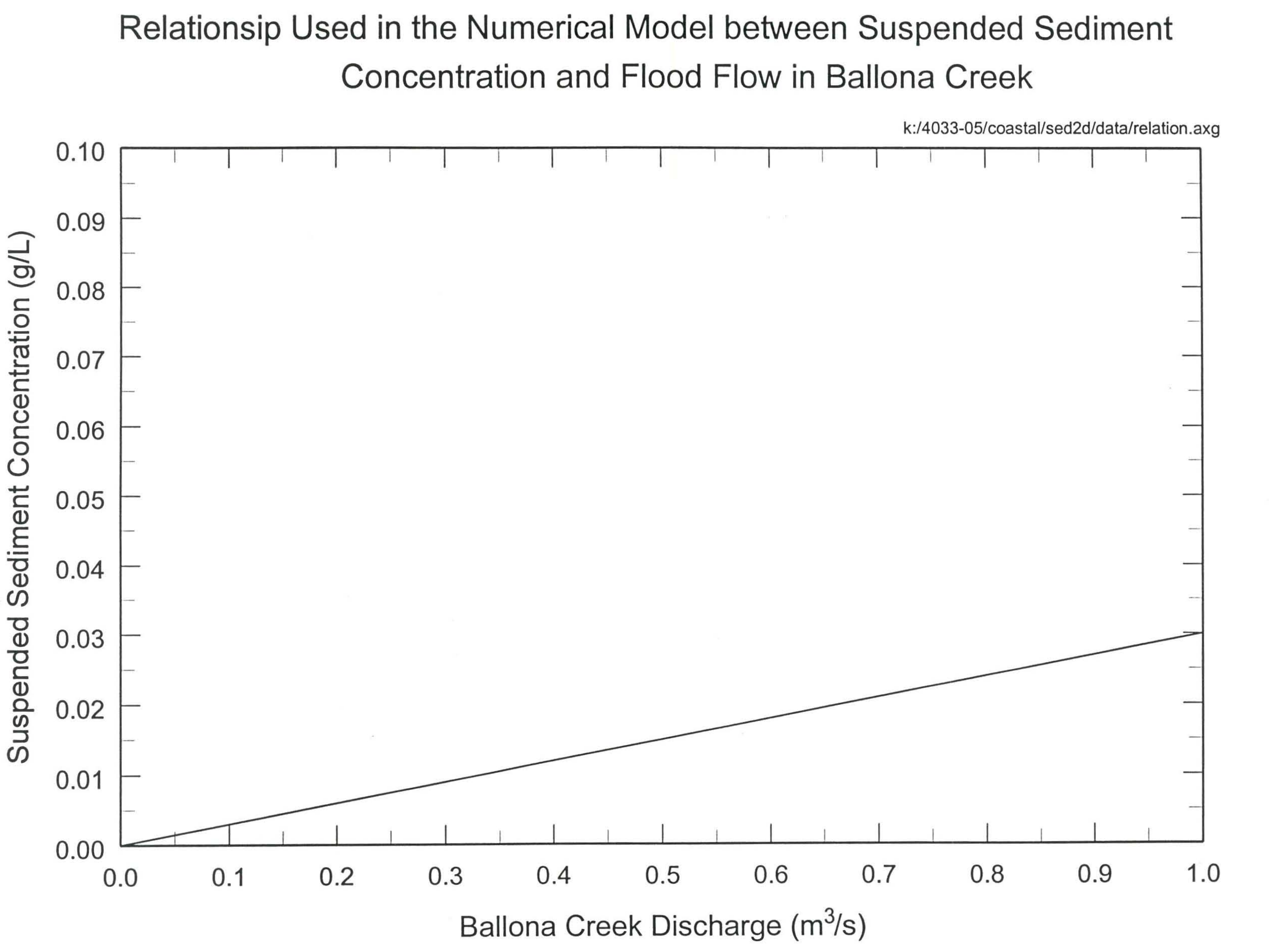
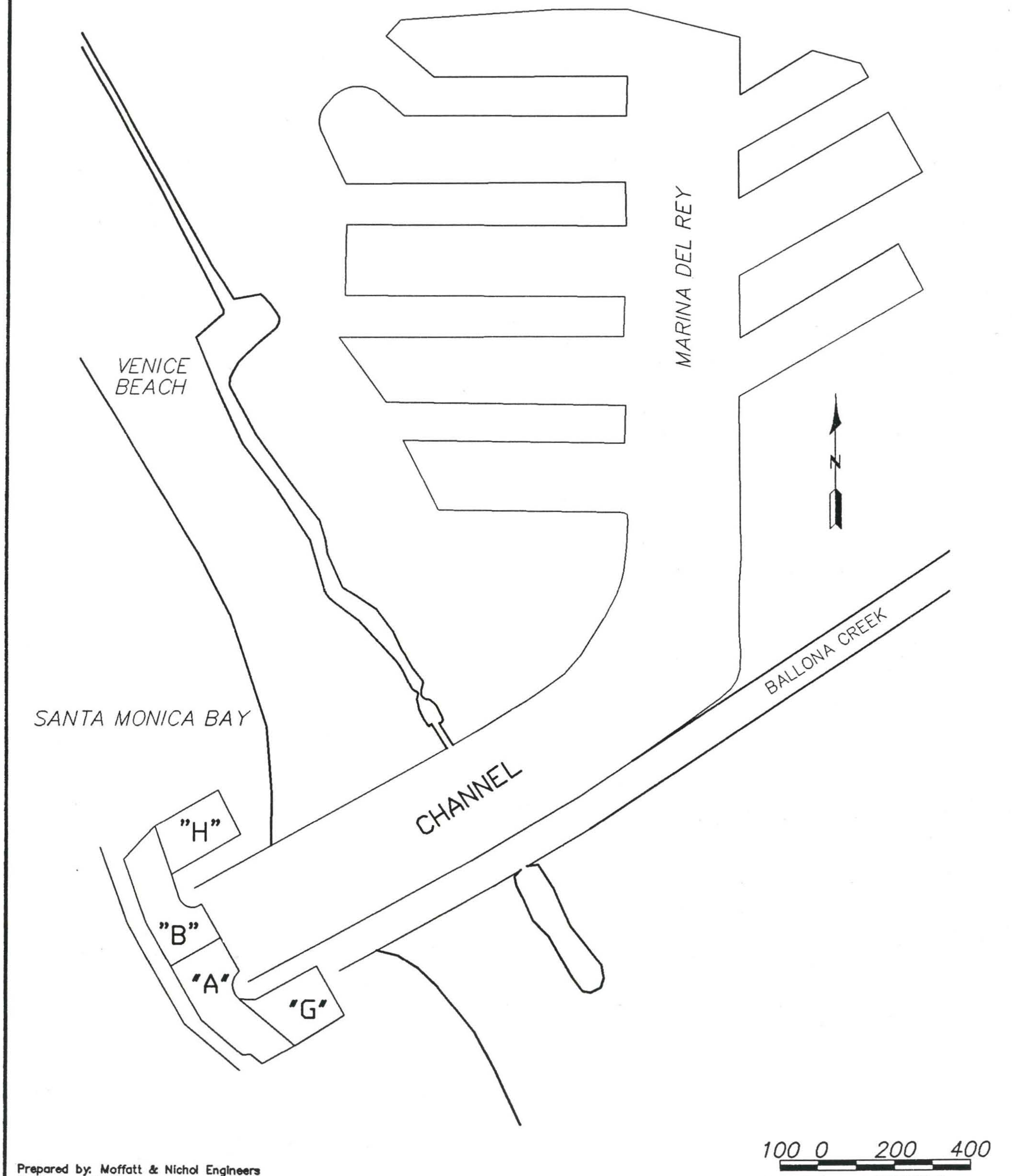

Relationsip Used in the Numerical Model between Suspended Sediment Concentration and Flood Flow in Ballona Creek
k:/4033-05/coastal/sed2d/data/relation.axg
Suspended Sediment Concentration (g/L)
0.10
0.09
0.08
0.07
0.06
0.05
0.04
0.03
0.02
0.01
0.00
0.0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
Ballona Creek Discharge (m3/s)

Figure 5. 15



Prepared by: Moffatt & Nichol Engineers
Draft, June 1998
k:\4033-05\coastal\dwg\fedchanl.dwg

MARINA DEL REY SEDIMENT STUDY	FEDERAL CHANNEL BOUNDARIES	Figure 5-16

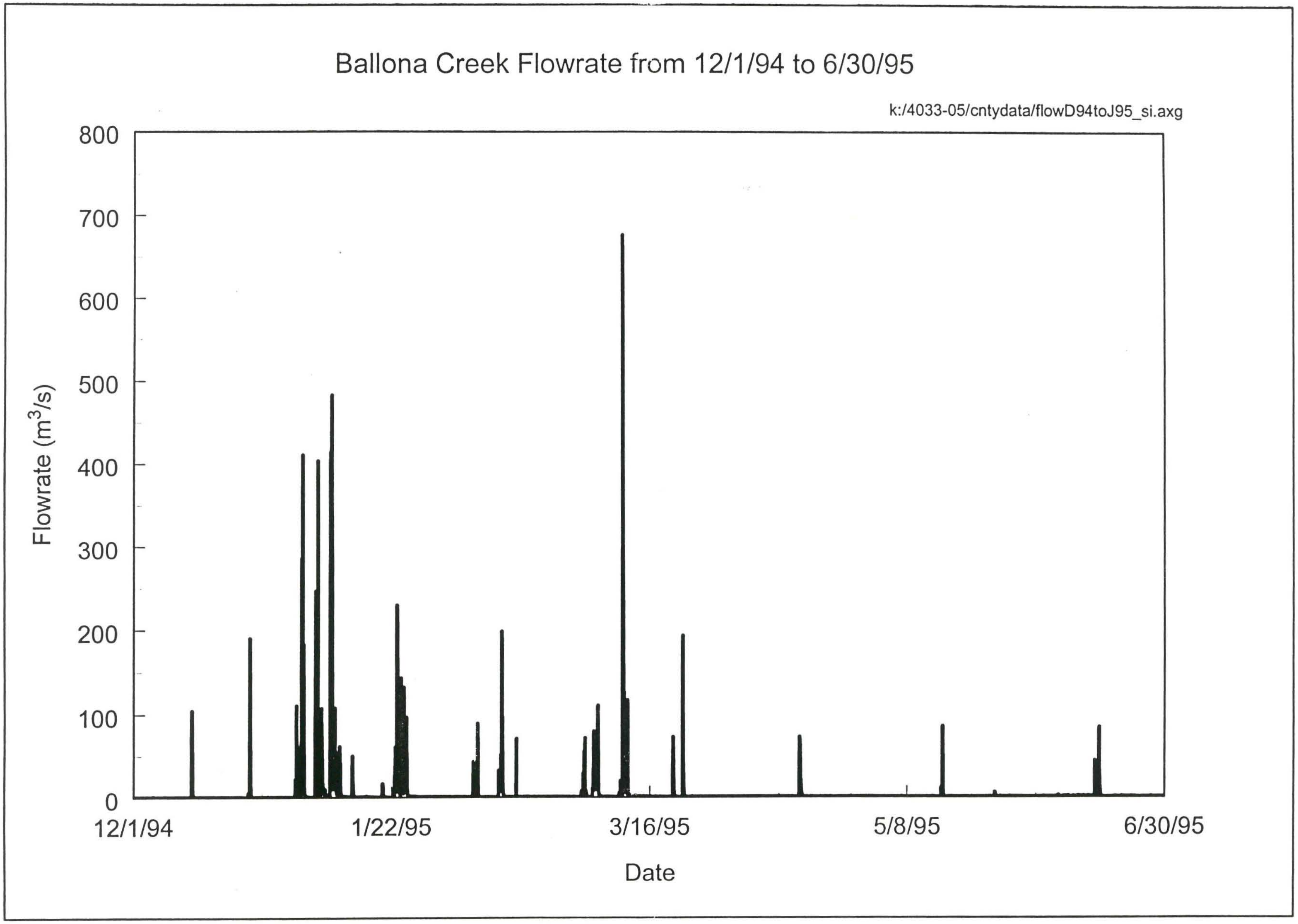
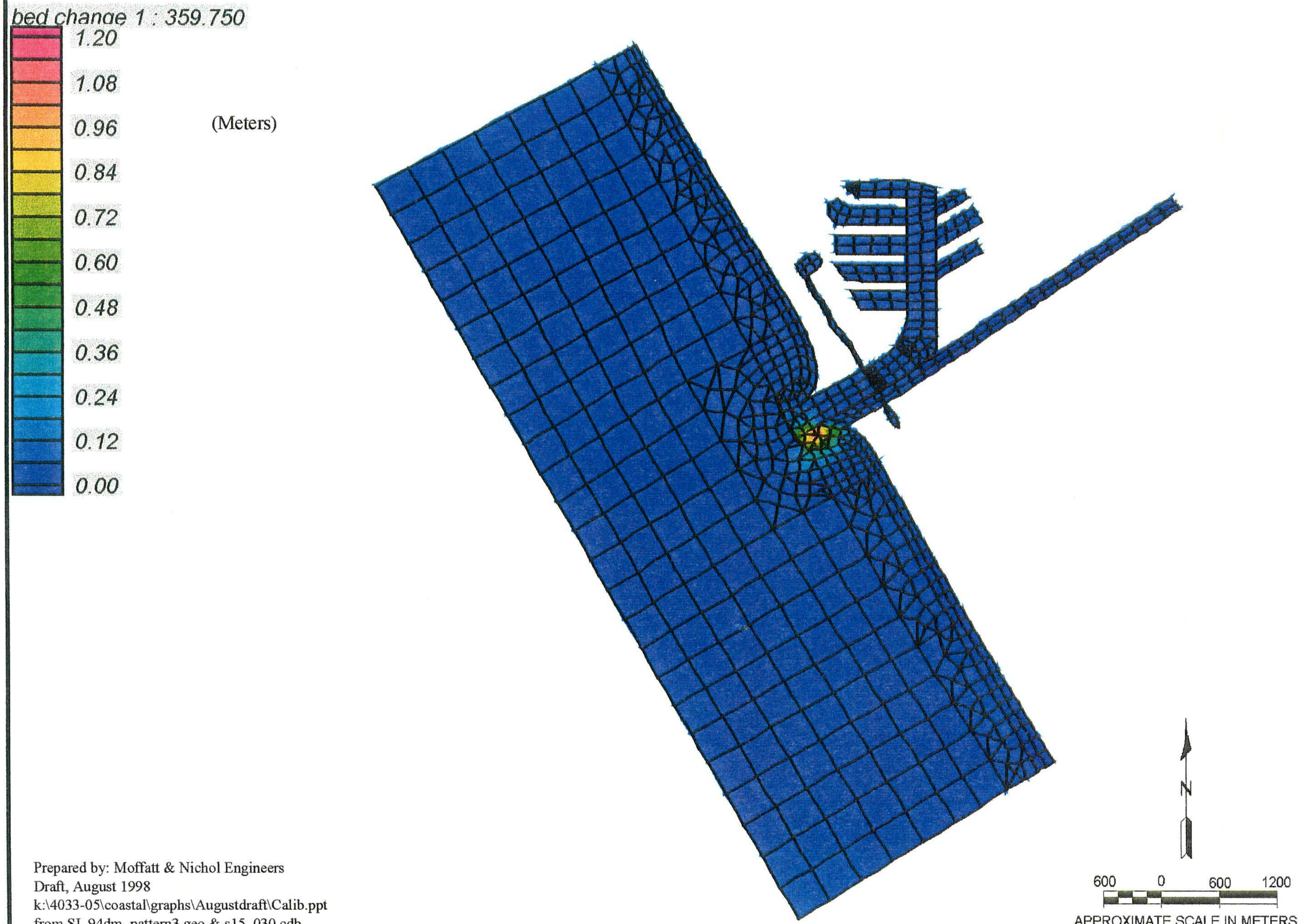
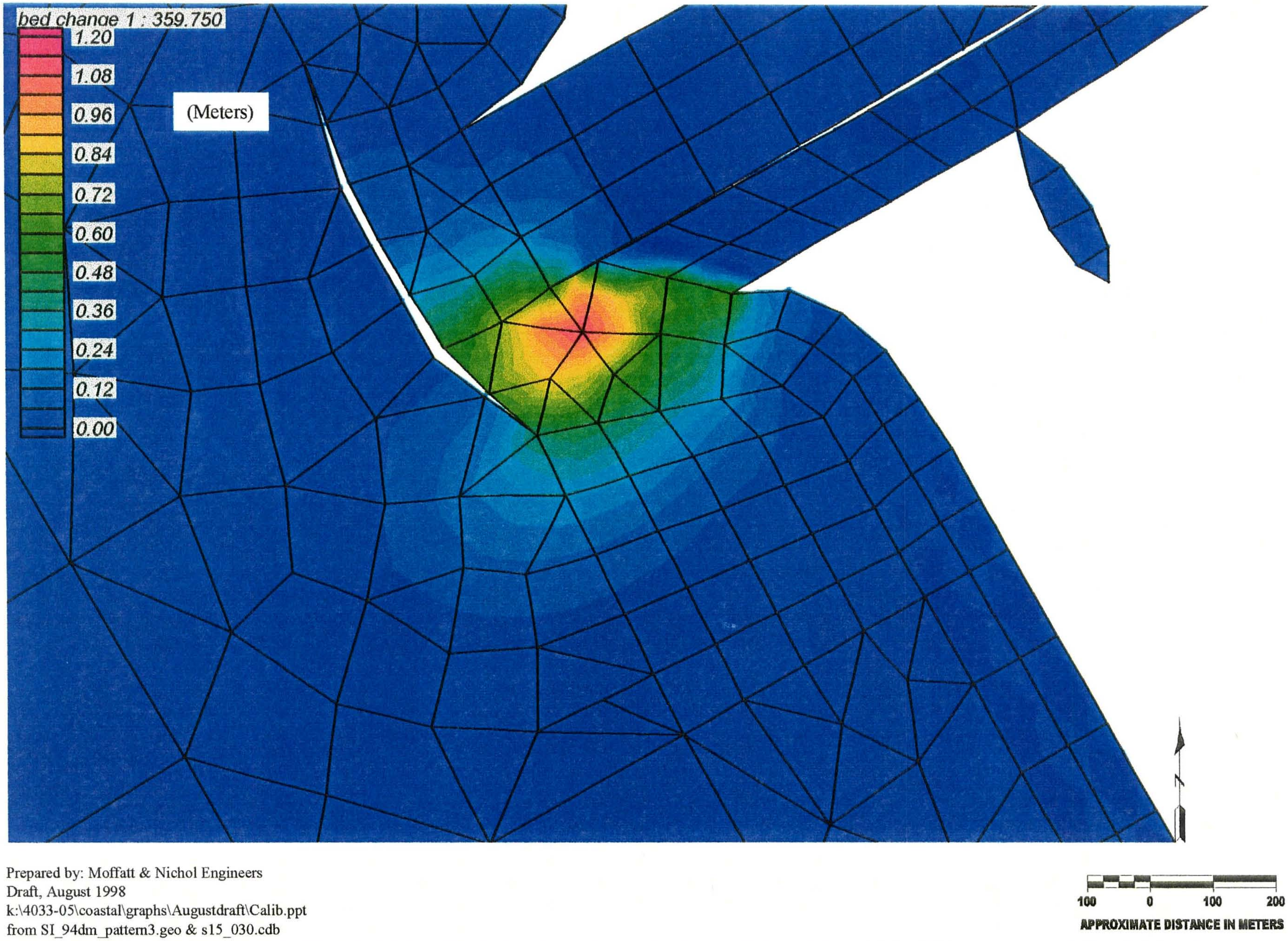

Ballona Creek Flowrate from 12/1/94 to 6/30/95
k:/4033-05/cntydata/flowD94toJ95_si.axg
800
700
600
500
400
300
200
100
0
Flowrate (m3/s)
12/1/94
1/22/95
3/16/95
5/8/95
6/30/95
Date

Figure 5.17



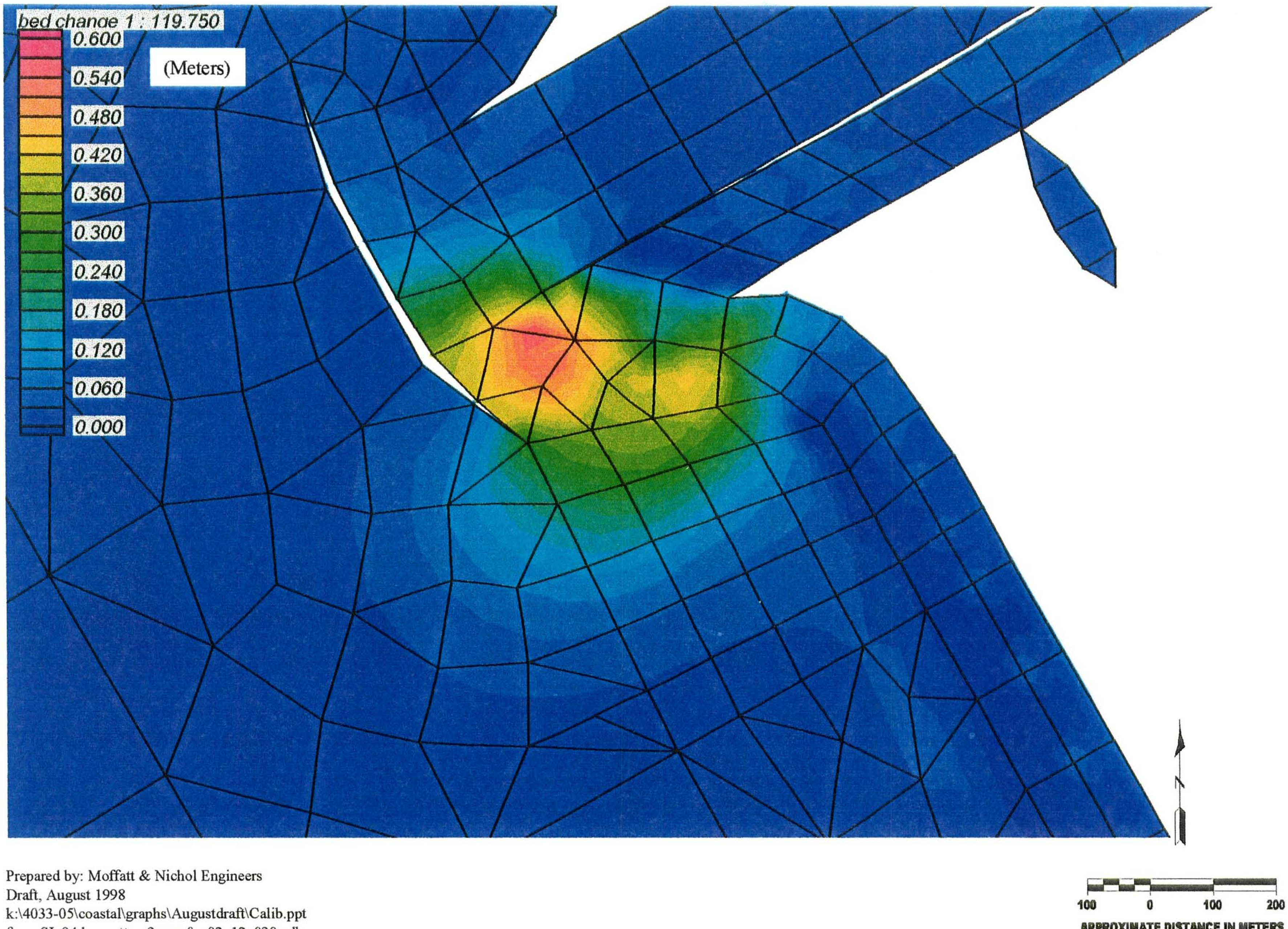
Prepared by: Moffatt & Nichol Engineers
Draft, August 1998
k:\4033-05\coastal\graphs\Augustdraft\Calib.ppt
from SI_94dm_pattern3.geo & s15_030.cdb

Marina del Rey and Ballona Creek Sediment Transport Studies	Bed elevation change after the first group of storms (1/1/95 to 1/15/95) in the calibration period (12/94 to 6/95), Grain size D_{50}=0.1 mm	**Figure** 5.18



Prepared by: Moffatt & Nichol Engineers
Draft, August 1998
k:\4033-05\coastal\graphs\Augustdraft\Calib.ppt
from SI_94dm_pattern3.geo & s15_030.cdb

Marina del Rey and Ballona Creek Sediment Transport Studies	Zoomed bed elevation change after the first group of storms (1/1/95 to 1/15/95) in the calibration period (12/94 to 6/95), Grain size D_{50}=0.1 mm	Figure 5.19



Prepared by: Moffatt & Nichol Engineers
Draft, August 1998
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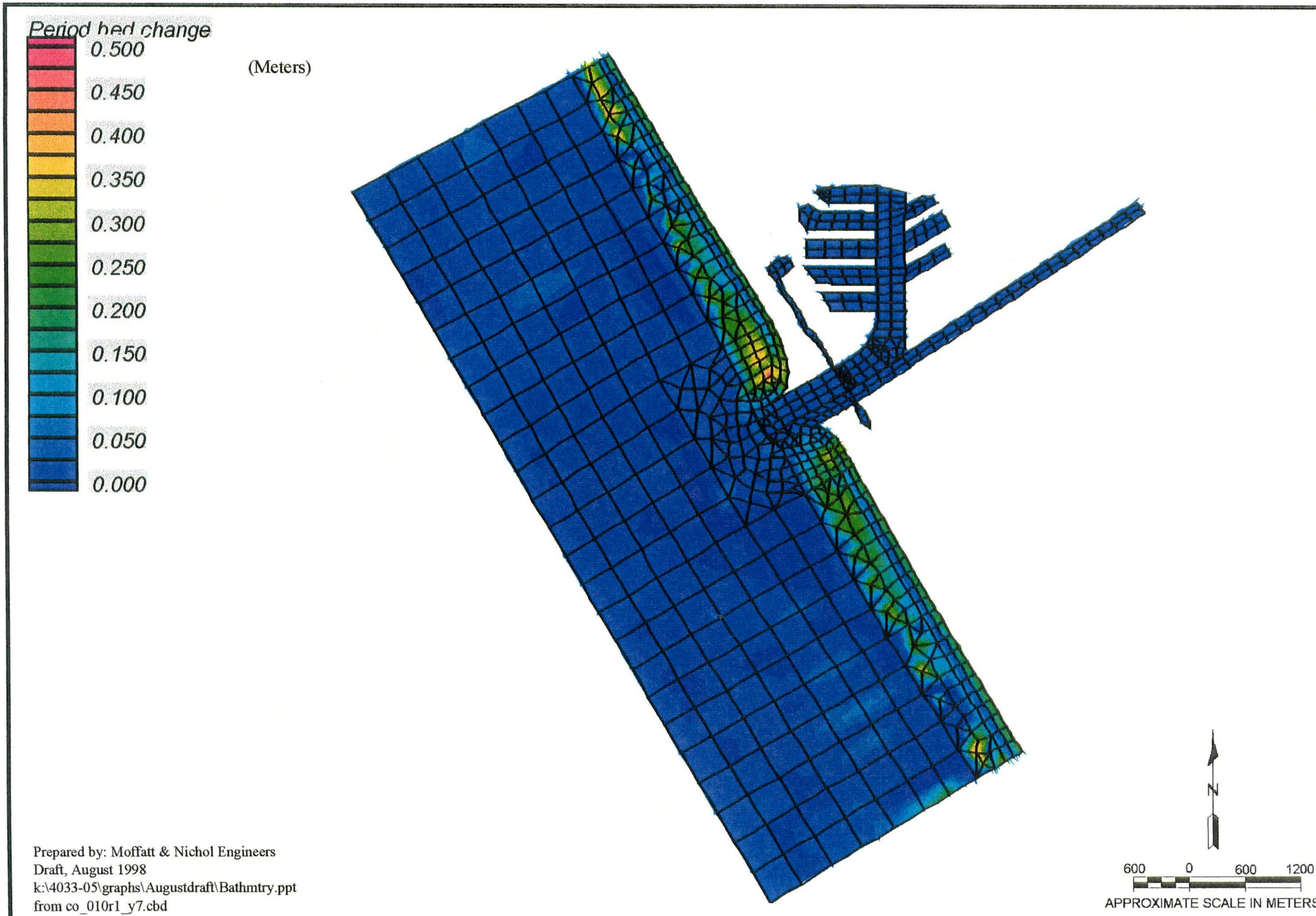
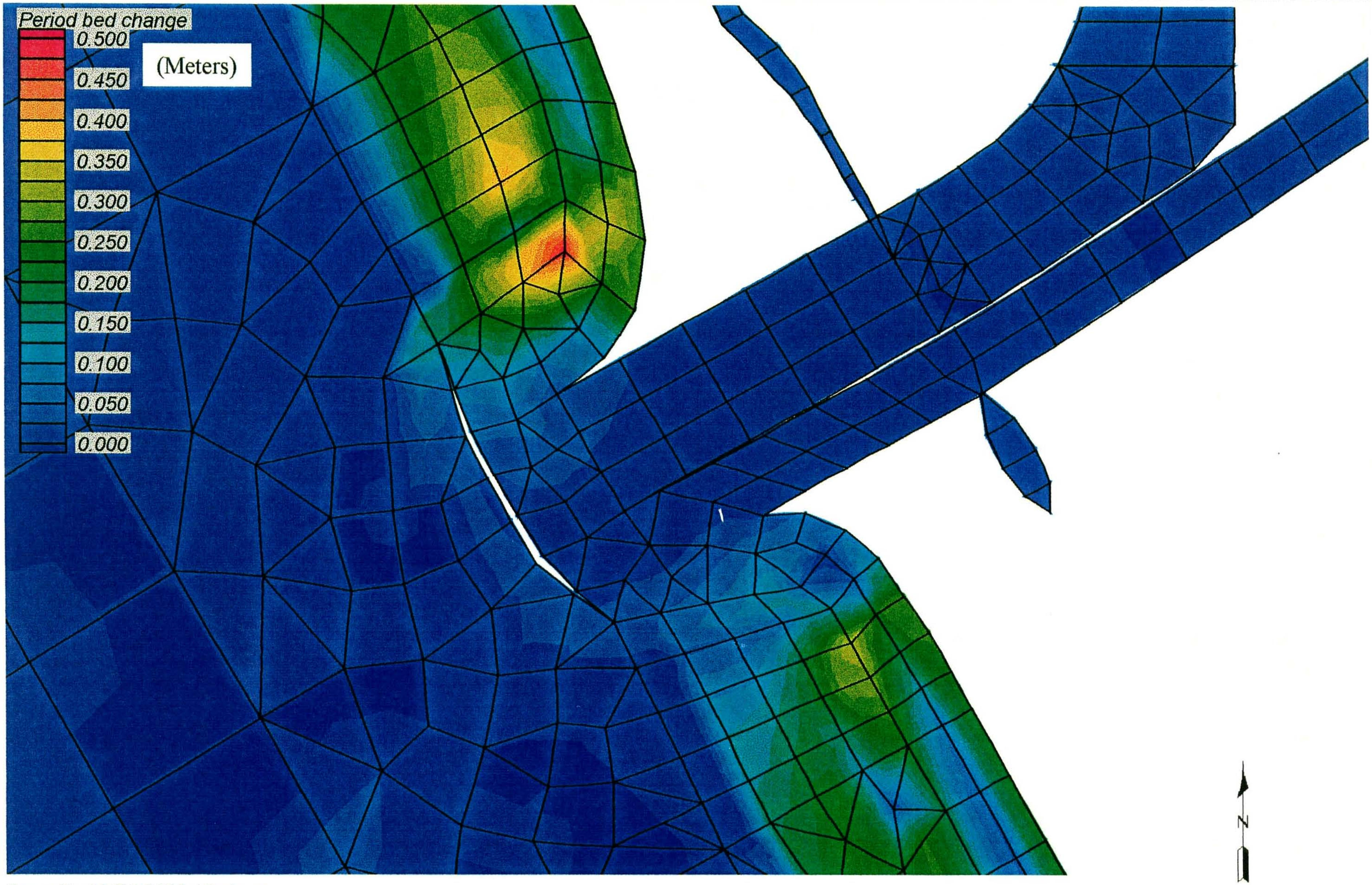
from SI_94dm_pattern3.geo & s02_12_030.cdb

Marina del Rey and Ballona Creek Sediment Transport Studies	Zoomed bed elevation change after the first group of storms (3/9/95 to 3/13/95) in the calibration period (12/94 to 6/95), Grain size D_{50}=0.1 mm	Figure 5.20



Prepared by: Moffatt & Nichol Engineers
Draft, August 1998
k:\4033-05\graphs\Augustdraft\Bathmtry.ppt
from co_010r1_y7.cbd

Marina del Rey and Ballona Creek Sediment Transport Studies	Bed Change, Longshore Sediment Transport Calibration Calibration Period 12/94 to 6/95	**Figure** 5.21



Prepared by: Moffatt & Nichol Engineers
Draft, August 1998
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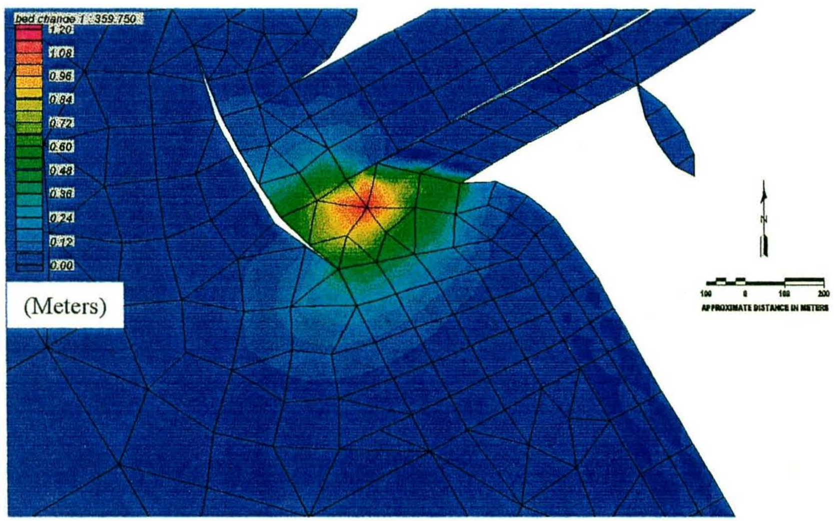
from co_010r1_y7.cbd

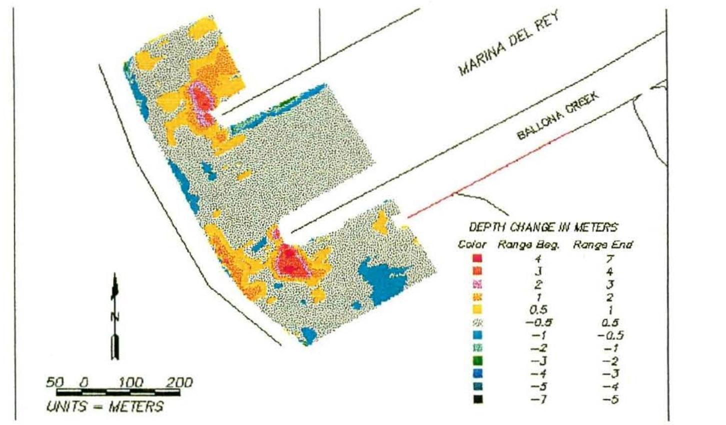
Marina del Rey and Ballona Creek Sediment Transport Studies	Zoomed Bed Change Longshore Sediment Transport Calibration Calibration Period 12/94 to 6/95	Figure 5.22

MODEL

(Not Including Longshore Transport)



SURVEY

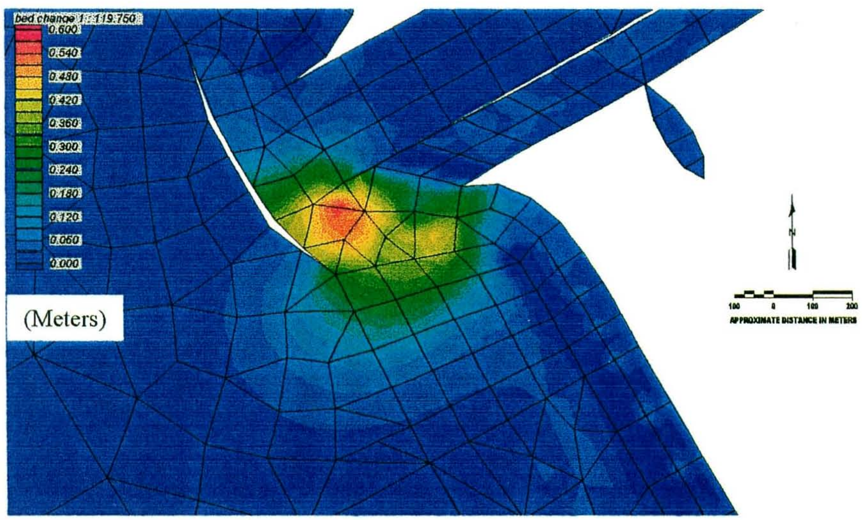


Prepared by: Moffatt & Nichol Engineers
Draft, August 1998
k:\4033-05\coastal\graphs\Augustdraft\~~Calib~~ compar.ppt
from SI_94dm_pattern3.geo & s15_030.cdb

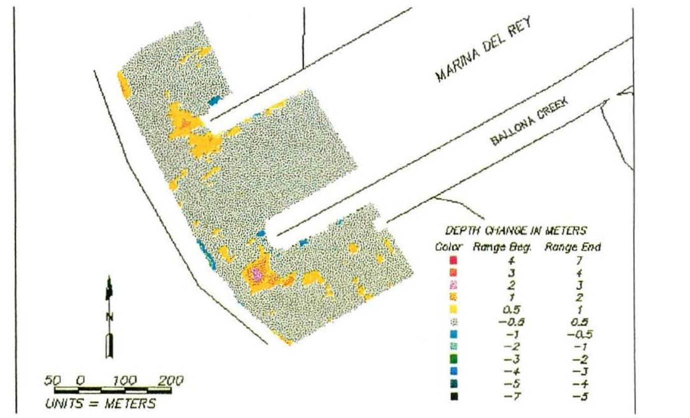
Marina del Rey and Ballona Creek Sediment Transport Studies	Comparison of Bed Change in the Period 12/94 to 1/95	Figure 5.23

MODEL

(Not Including Longshore Transport)

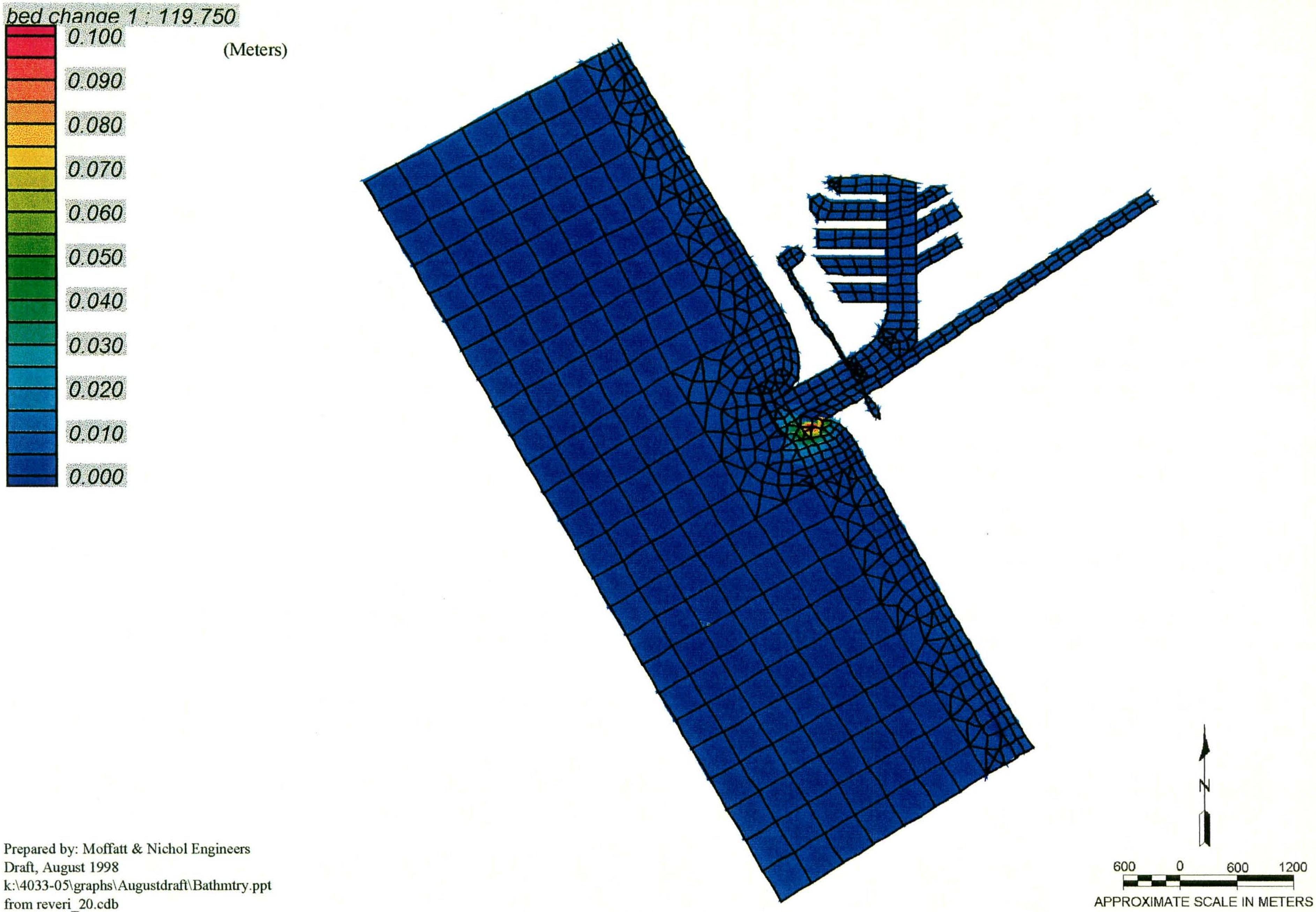


SURVEY



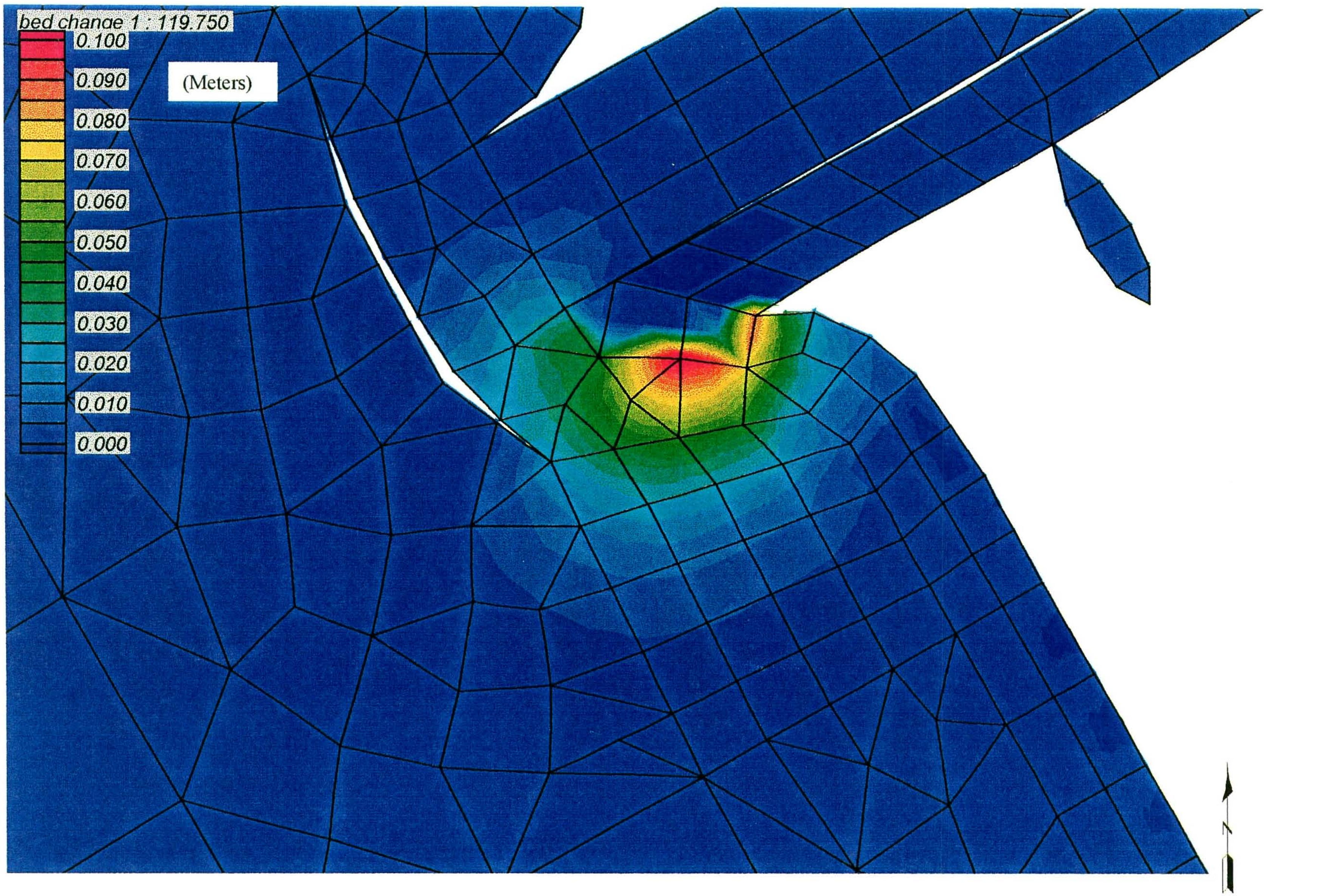
Prepared by: Moffatt & Nichol Engineers
Draft, August 1998
k:\4033-05\coastal\graphs\Augustdraft\Compar.ppt
from SI_94dm_pattern3.geo & s02_12_030.cdb

Marina del Rey and Ballona Creek Sediment Transport Studies	Comparison of Bed Change in the Period 1/95 to 6/95	Figure 5.24



Prepared by: Moffatt & Nichol Engineers
Draft, August 1998
k:\4033-05\graphs\Augustdraft\Bathmtry.ppt
from reveri_20.cdb

Marina del Rey and Ballona Creek Sediment Transport Studies	Bed Change After Storm Event 4/15/96 to 4/19/96 in the Verification Period 4/96 to 9/96	**Figure** 5.25



Prepared by: Moffatt & Nichol Engineers
Draft, August 1998
k:\4033-05\graphs\Augustdraft\Bathmtry.ppt
from reveri_20.cdb

Marina del Rey and Ballona Creek Sediment Transport Studies

Zoomed Bed Change of Storm Event
4/15/96 to 4/19/96

Figure
5.26

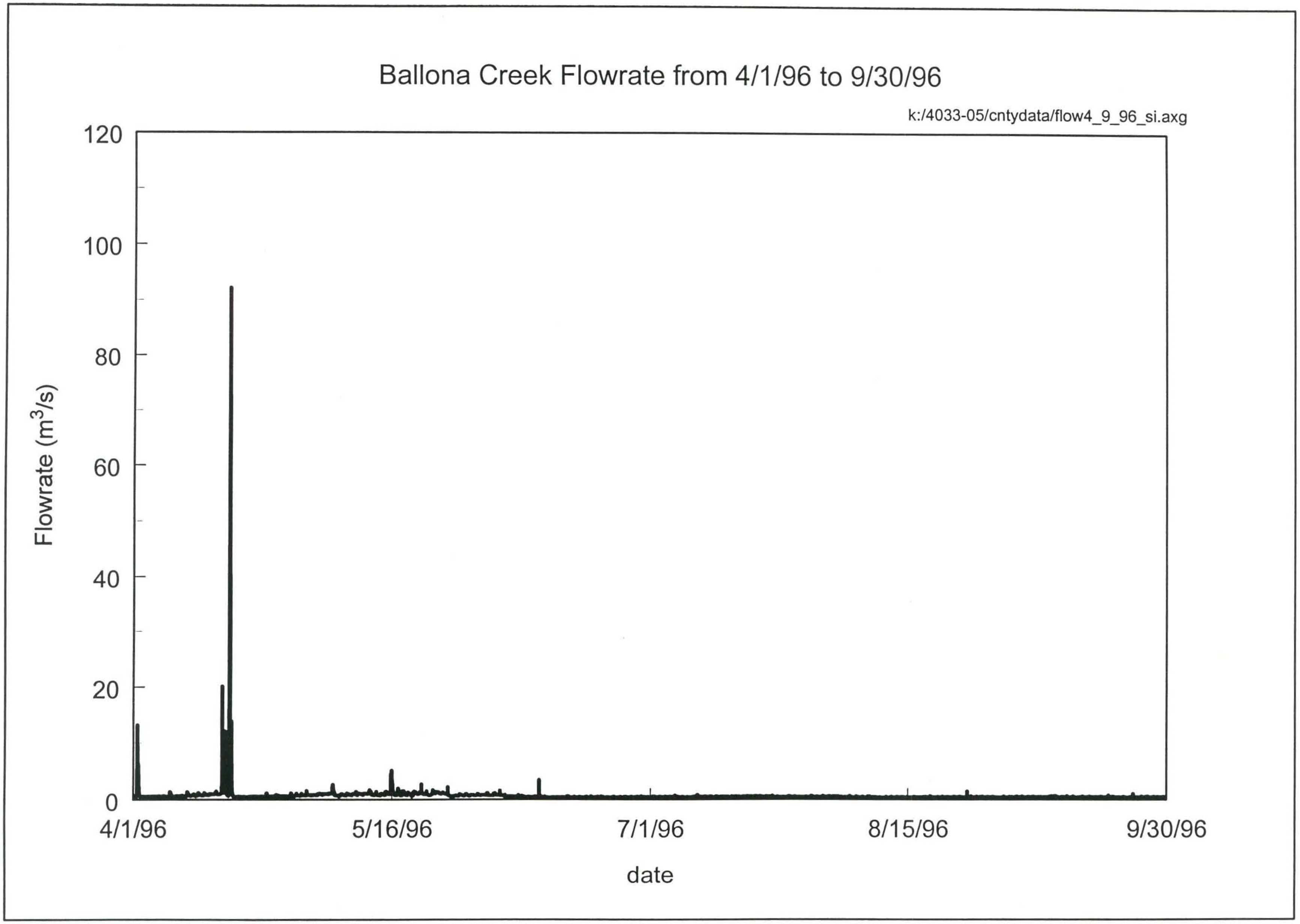
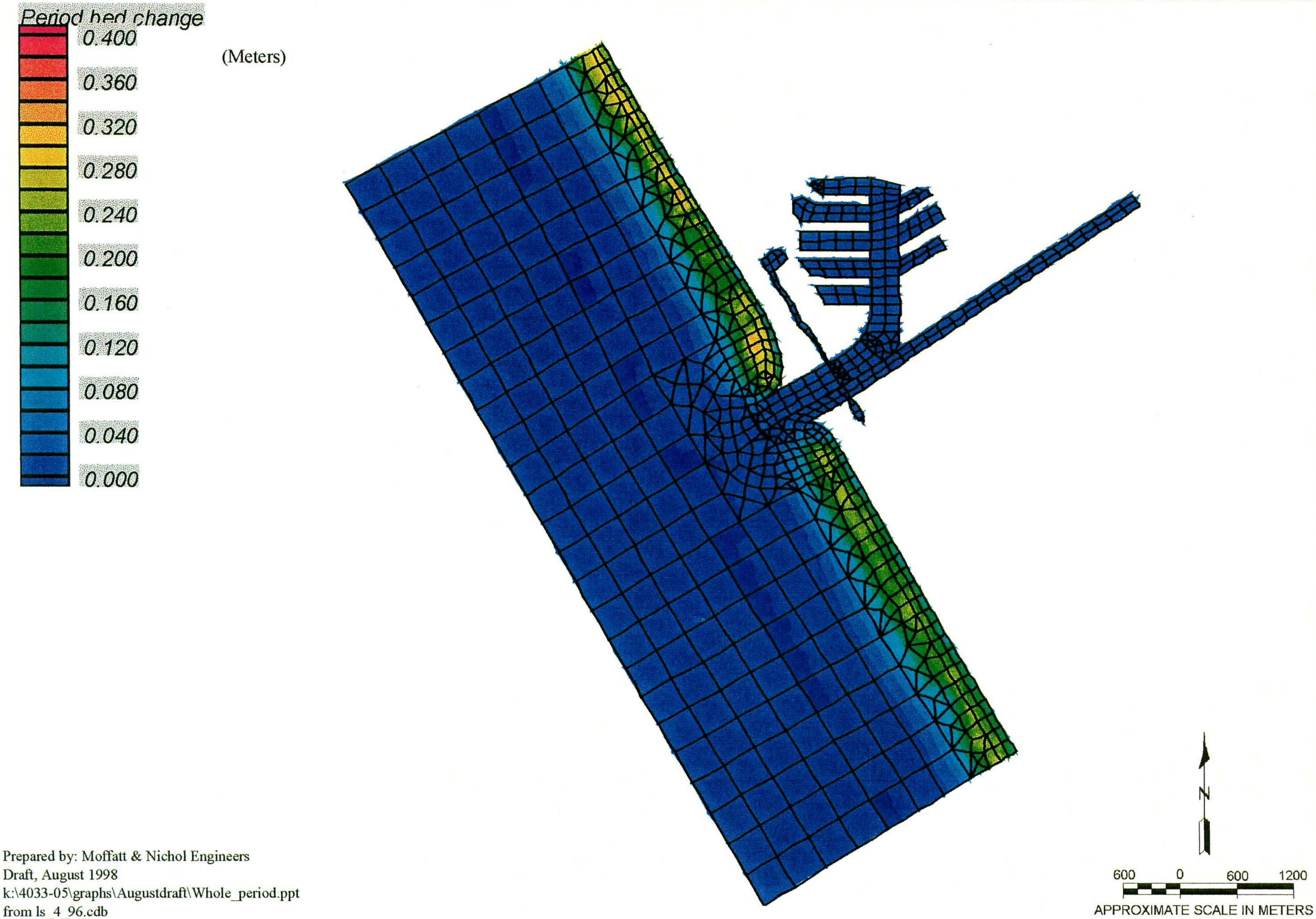

Ballona Creek Flowrate from 4/1/96 to 9/30/96
k:/4033-05/cntydata/flow4_9_96_si.axg
120
100
80
60
40
20
0
Flowrate (m3/s)
4/1/96
5/16/96
7/1/96
8/15/96
9/30/96
date

Figure 5.27



Prepared by: Moffatt & Nichol Engineers
Draft, August 1998
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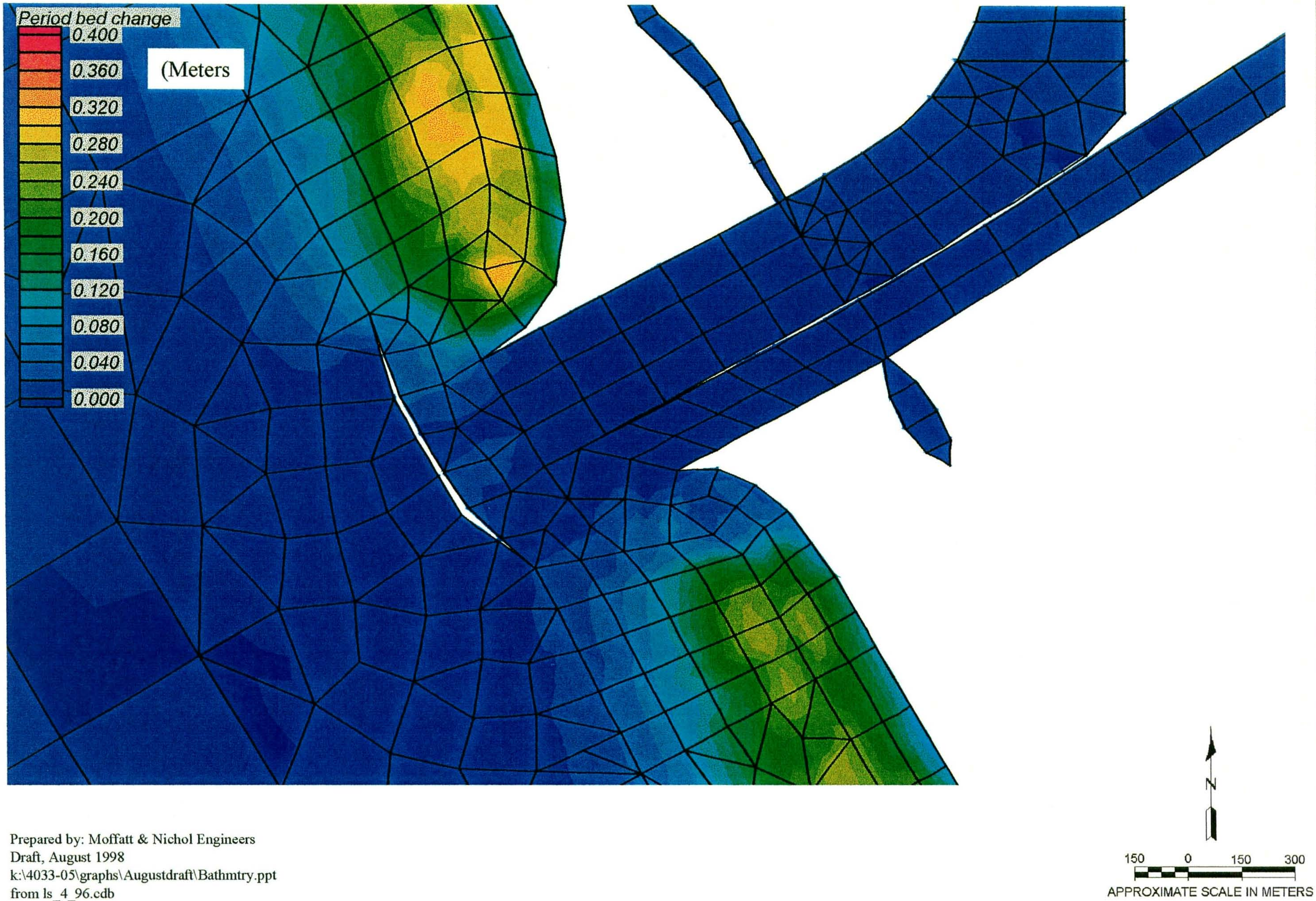
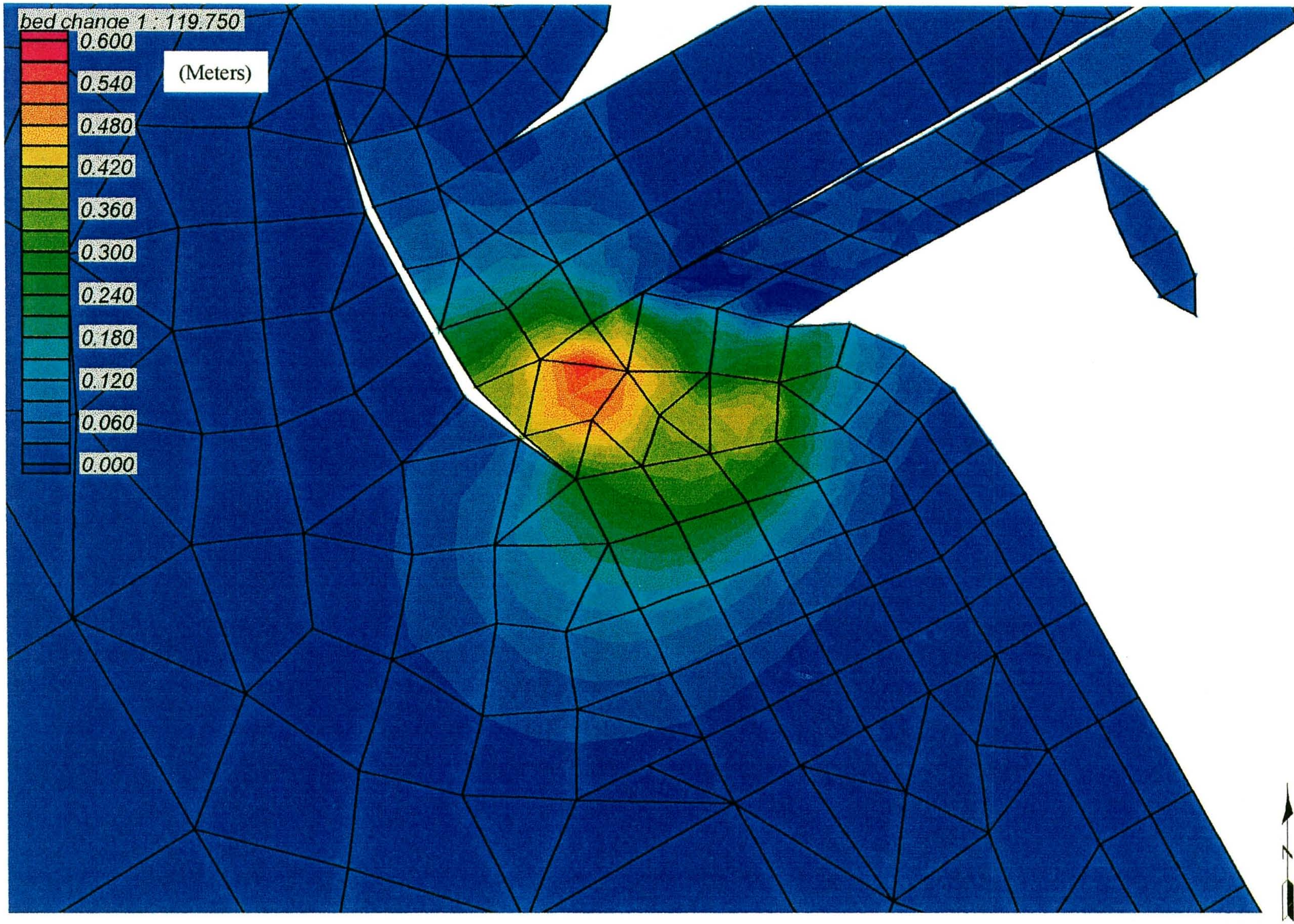
from ls_4_96.cdb

Marina del Rey and Ballona Creek Sediment Transport Studies	Bed Change, Long Shore Sediment Transport Verification Verification Period 4/96 to 9/96	Figure 5.28



Prepared by: Moffatt & Nichol Engineers
Draft, August 1998
k:\4033-05\graphs\Augustdraft\Bathmtry.ppt
from ls_4_96.cdb

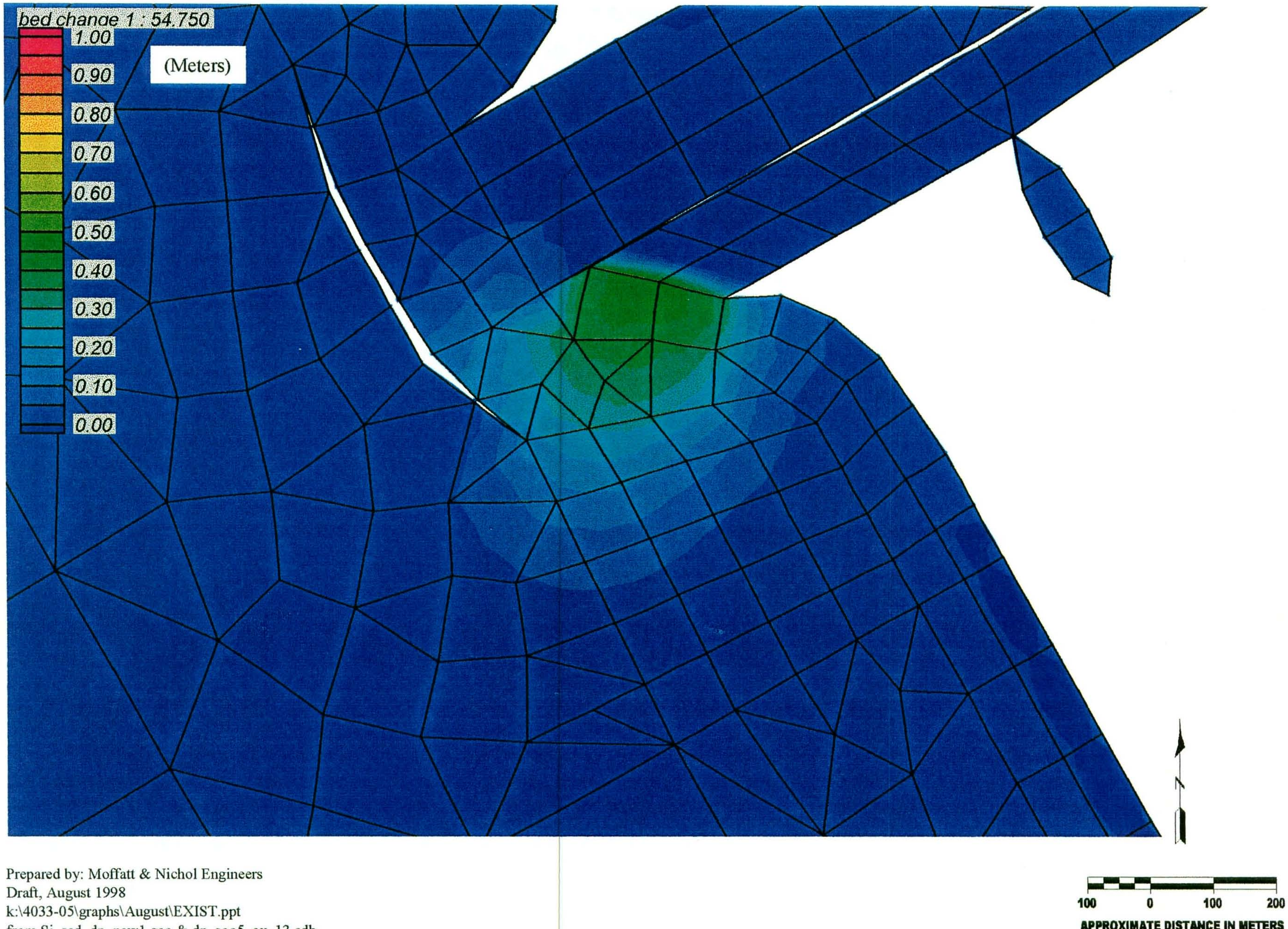
Marina del Rey and Ballona Creek Sediment Transport Studies	Zoomed Bed Change Long Shore Sediment Transport Verification Verification Period 4/96 to 9/96	Figure 5.29



Prepared by: Moffatt & Nichol Engineers
Draft, August 1998
k:\4033-05\coastal\graphs\Augustdraft\Calib.ppt
from SI_94dm_pattern3.geo & s02_12_030_NW.cdb

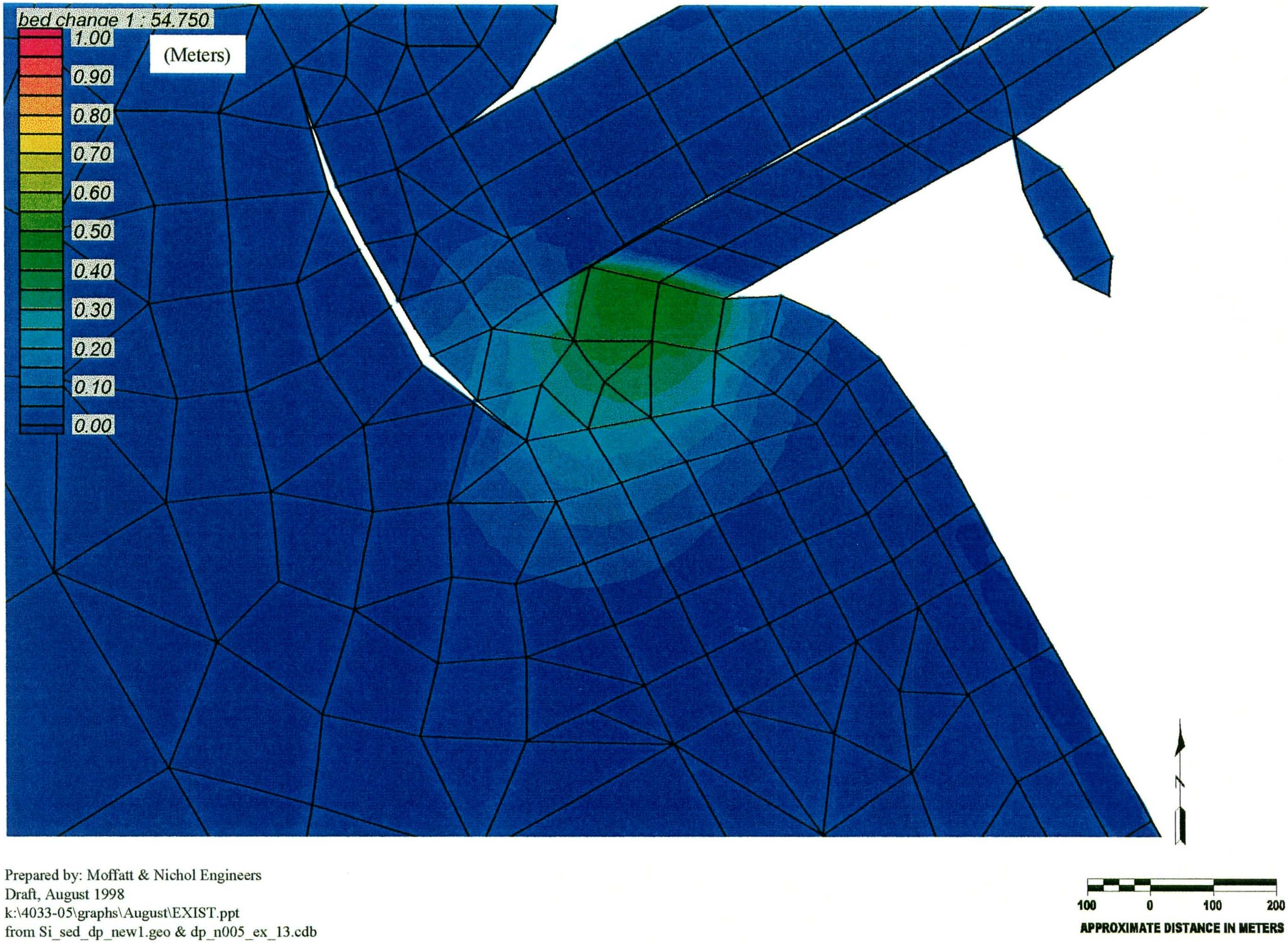
100 0 100 200
APPROXIMATE DISTANCE IN METERS

Marina del Rey and Ballona Creek Sediment Transport Studies	Zoomed bed elevation change after the first group of storms (3/9/95 to 3/13/95) in the calibration period (12/94 to 6/95), Grain size D_{50}=0.1 mm, No Wave	Figure 5.30



Prepared by: Moffatt & Nichol Engineers
Draft, August 1998
k:\4033-05\graphs\August\EXIST.ppt
from Si_sed_dp_new1.geo & dp_soo5_ex_13.cdb

Marina del Rey and Ballona Creek Sediment Transport Studies	Bed Change, Existing Bathymetry, Spring Tide, 5-Year Flood, Average Wave	**Figure 5.31**



Prepared by: Moffatt & Nichol Engineers
Draft, August 1998
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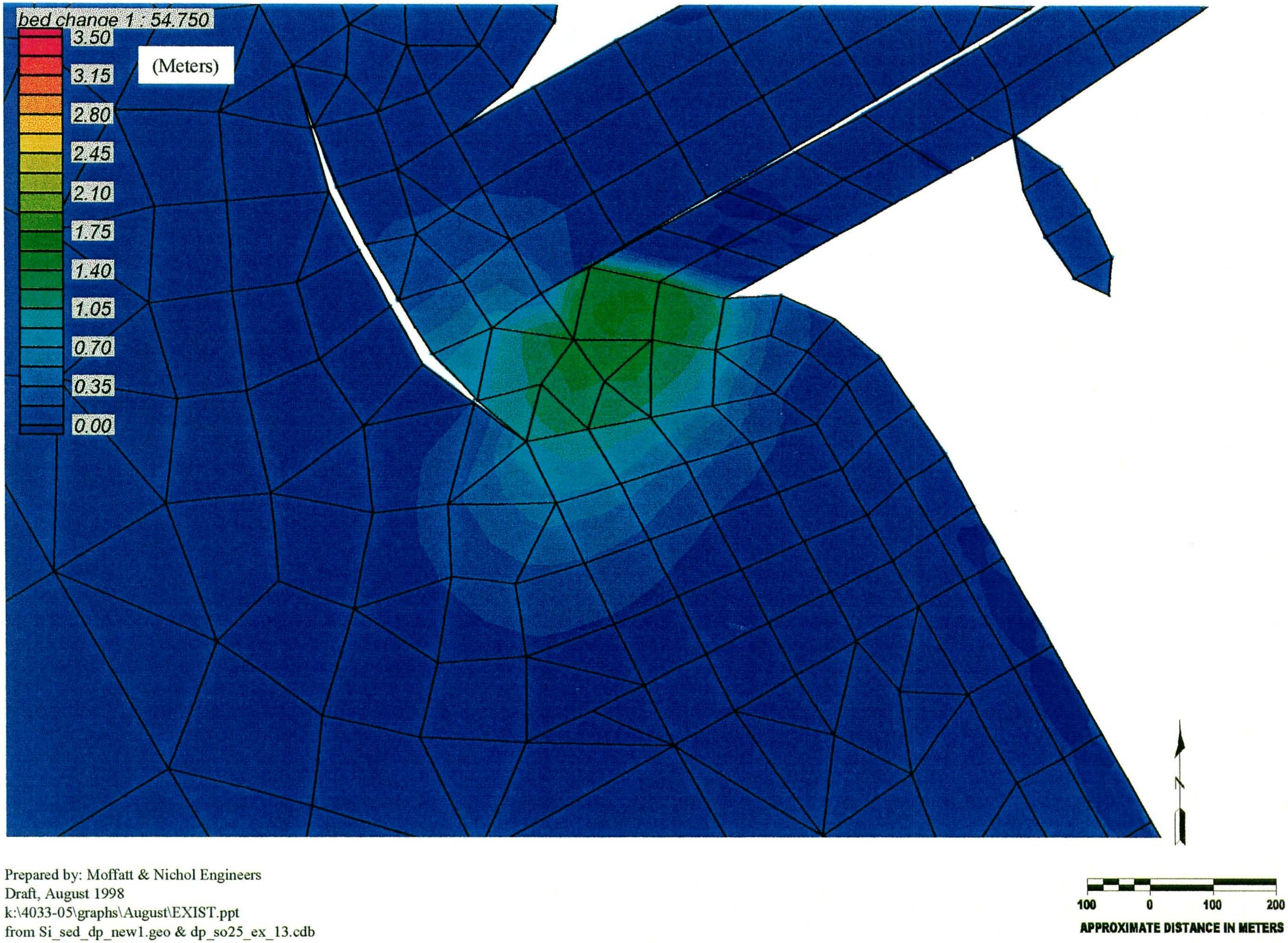
from Si_sed_dp_new1.geo & dp_n005_ex_13.cdb

Marina del Rey and Ballona Creek Sediment Transport Studies	Bed Change, Existing Bathymetry, Neap Tide, 5-Year Flood, Average Wave	**Figure** 5.32



Prepared by: Moffatt & Nichol Engineers
Draft, August 1998
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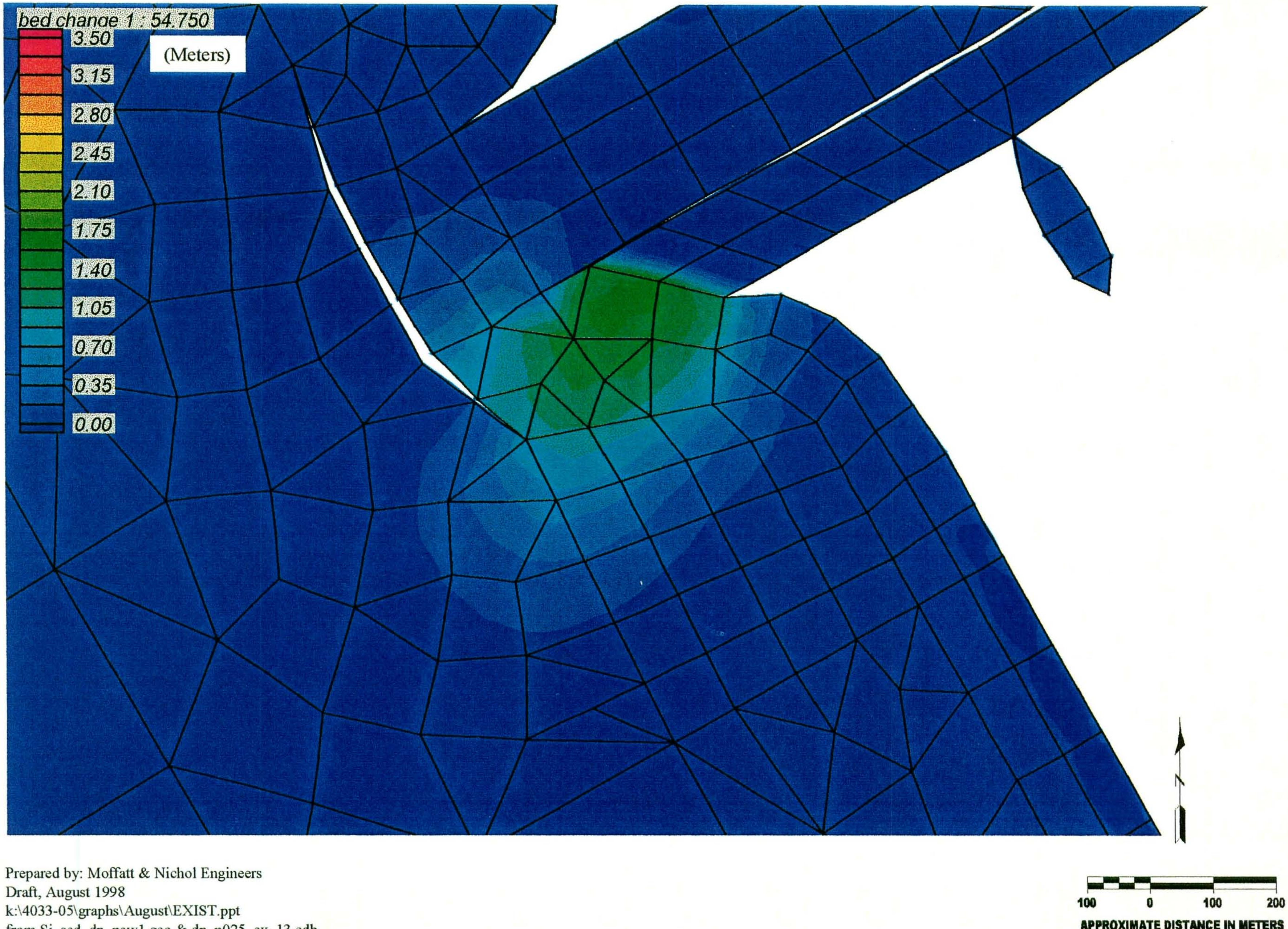
from Si_sed_dp_new1.geo & dp_so25_ex_13.cdb

Marina del Rey and Ballona Creek Sediment Transport Studies

Bed Change, Existing Bathymetry, Spring Tide, 25-Year Flood, Average Wave

Figure 5.33



Prepared by: Moffatt & Nichol Engineers
Draft, August 1998
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from Si_sed_dp_new1.geo & dp_n025_ex_13.cdb

Marina del Rey and Ballona Creek Sediment Transport Studies

Bed Change, Existing Bathymetry, Neap Tide, 25-Year Flood, Average Wave

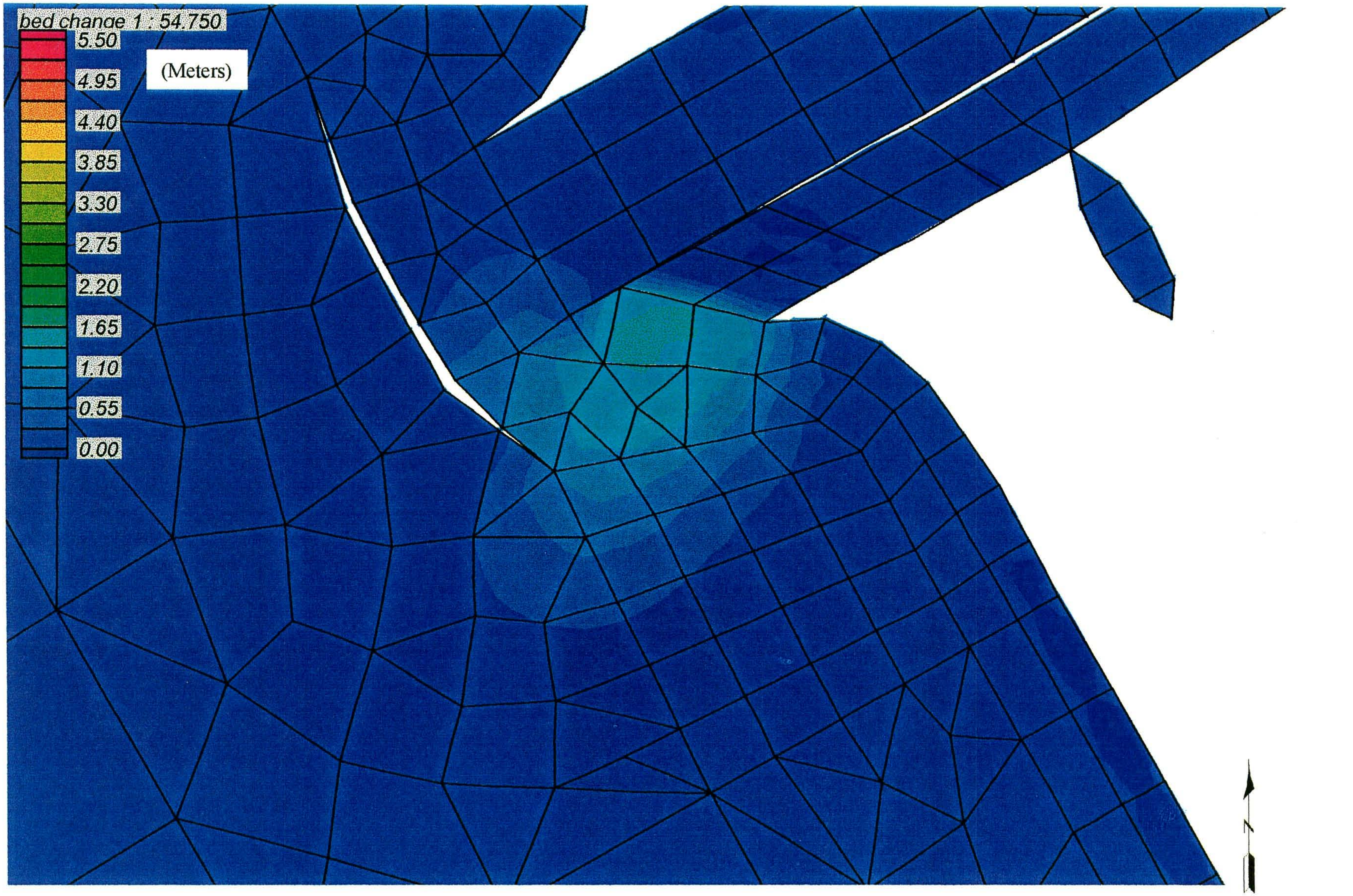
Figure 5.34

Prepared by: Moffatt & Nichol Engineers
Draft, August 1998
k:\4033-05\graphs\August\EXISTsamescale.ppt
from Si_sed_dp_new1.geo & dp_soo5_ex_13.cdb

Marina del Rey and Ballona Creek Sediment Transport Studies

Bed Change, Existing Bathymetry, Spring Tide, 5-Year Flood, Average Wave

Figure 5.35



Prepared by: Moffatt & Nichol Engineers
Draft, August 1998
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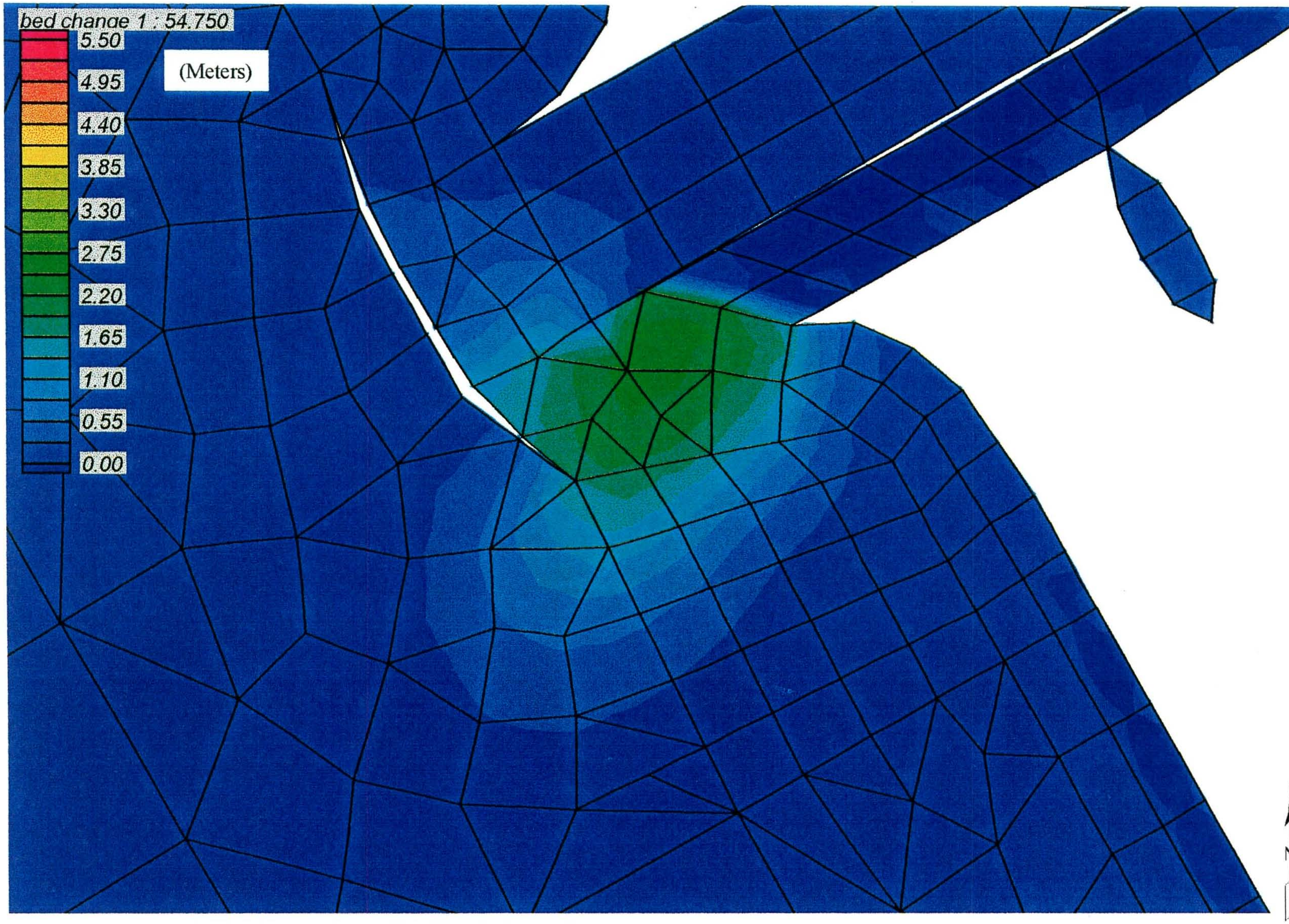
from Si_sed_dp_new1.geo & dp_so25_ex_13.cdb

Marina del Rey and Ballona Creek Sediment Transport Studies

Bed Change, Existing Bathymetry, Spring Tide, 25-Year Flood, Average Wave

Figure 5.36



Prepared by: Moffatt & Nichol Engineers
Draft, August 1998
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from Si_sed_dp_new1.geo & dp_s100_ex_13.cdb

100 0 100 200
APPROXIMATE DISTANCE IN METERS

Marina del Rey and Ballona Creek Sediment Transport Studies	Bed Change, Existing Bathymetry, Spring Tide, 100-Year Flood, Average Wave	Figure 5.37

Bed elevation change at gage 1593 for existing bathymetry, Spring Tide

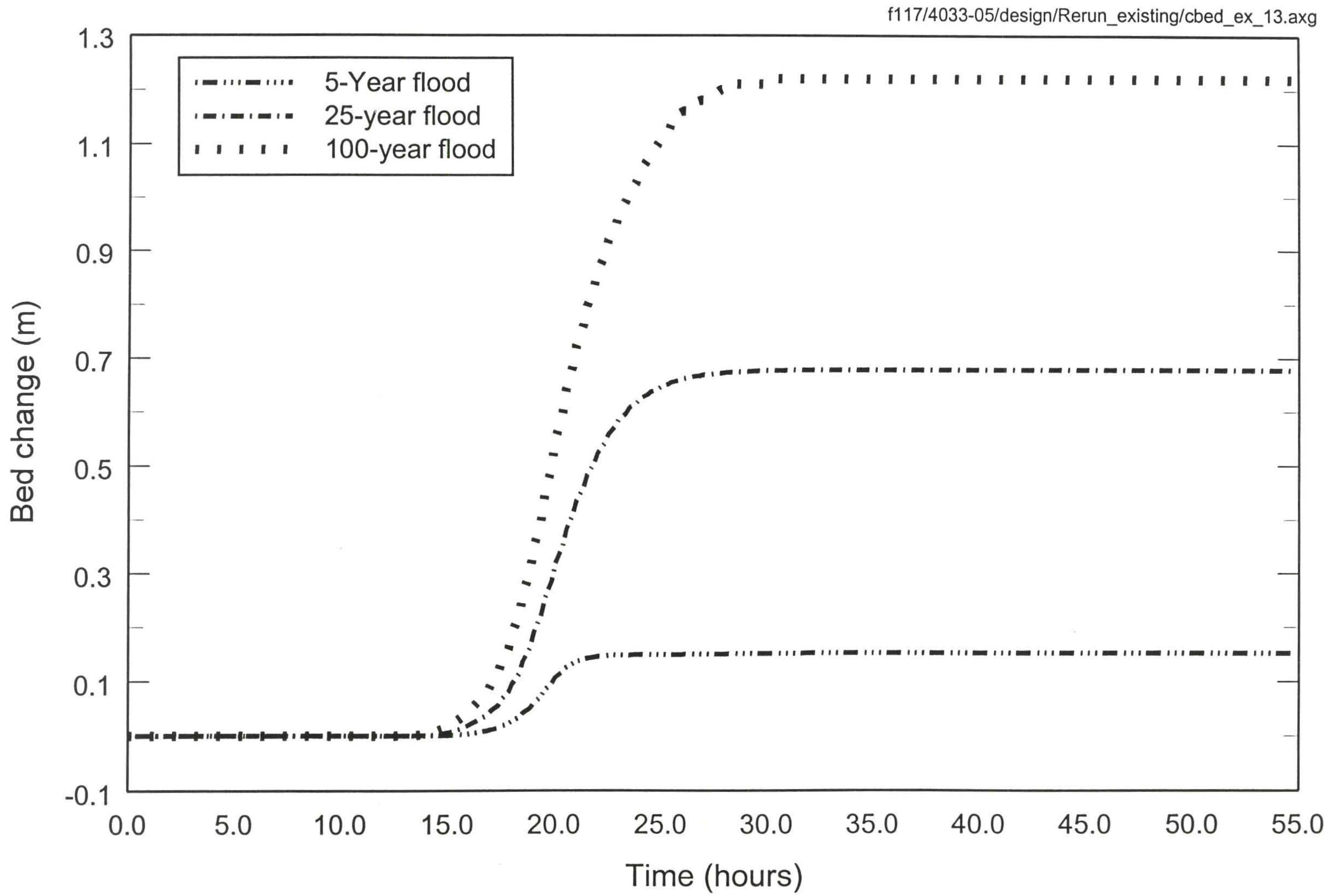
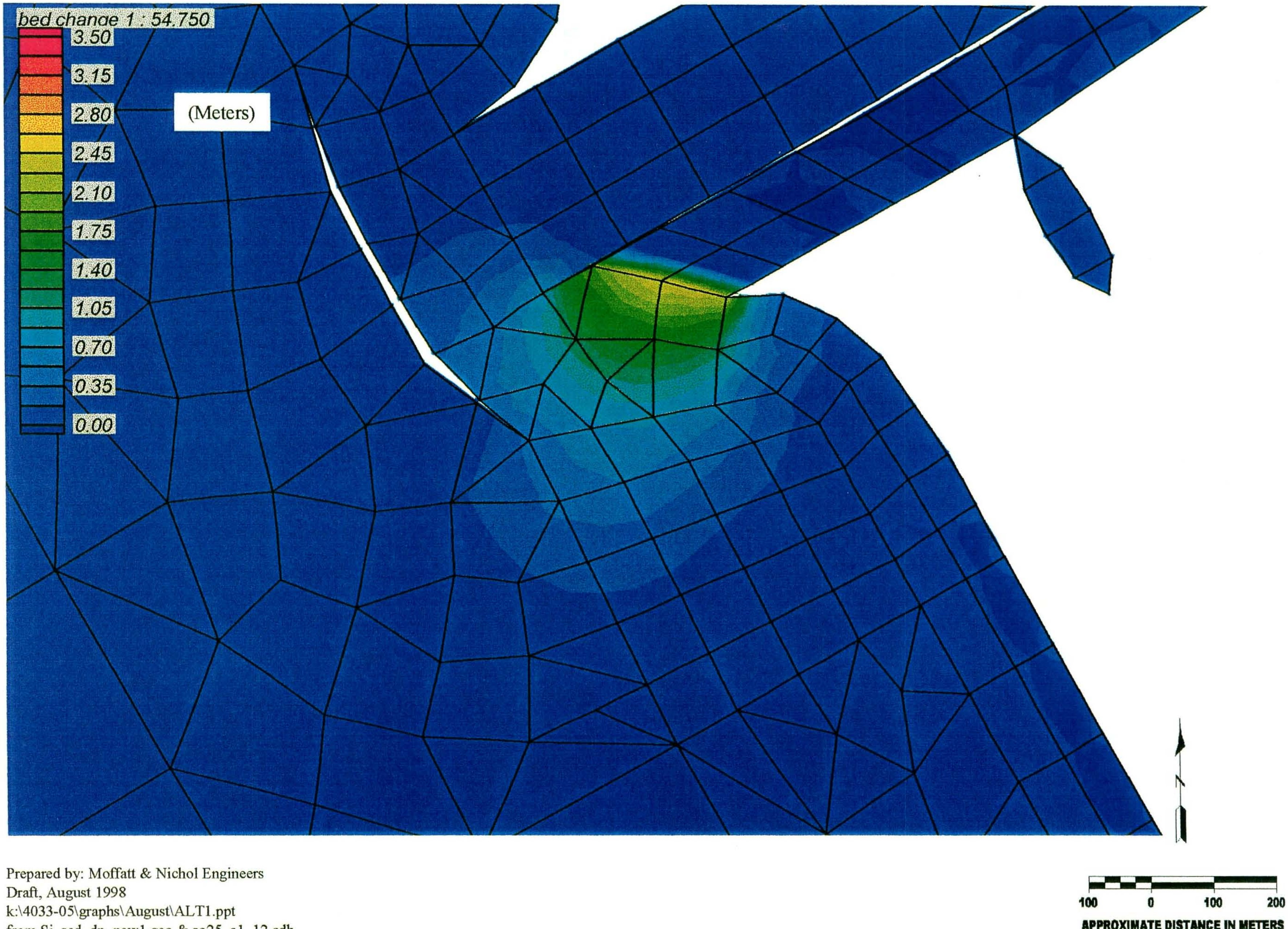


Figure 5.38

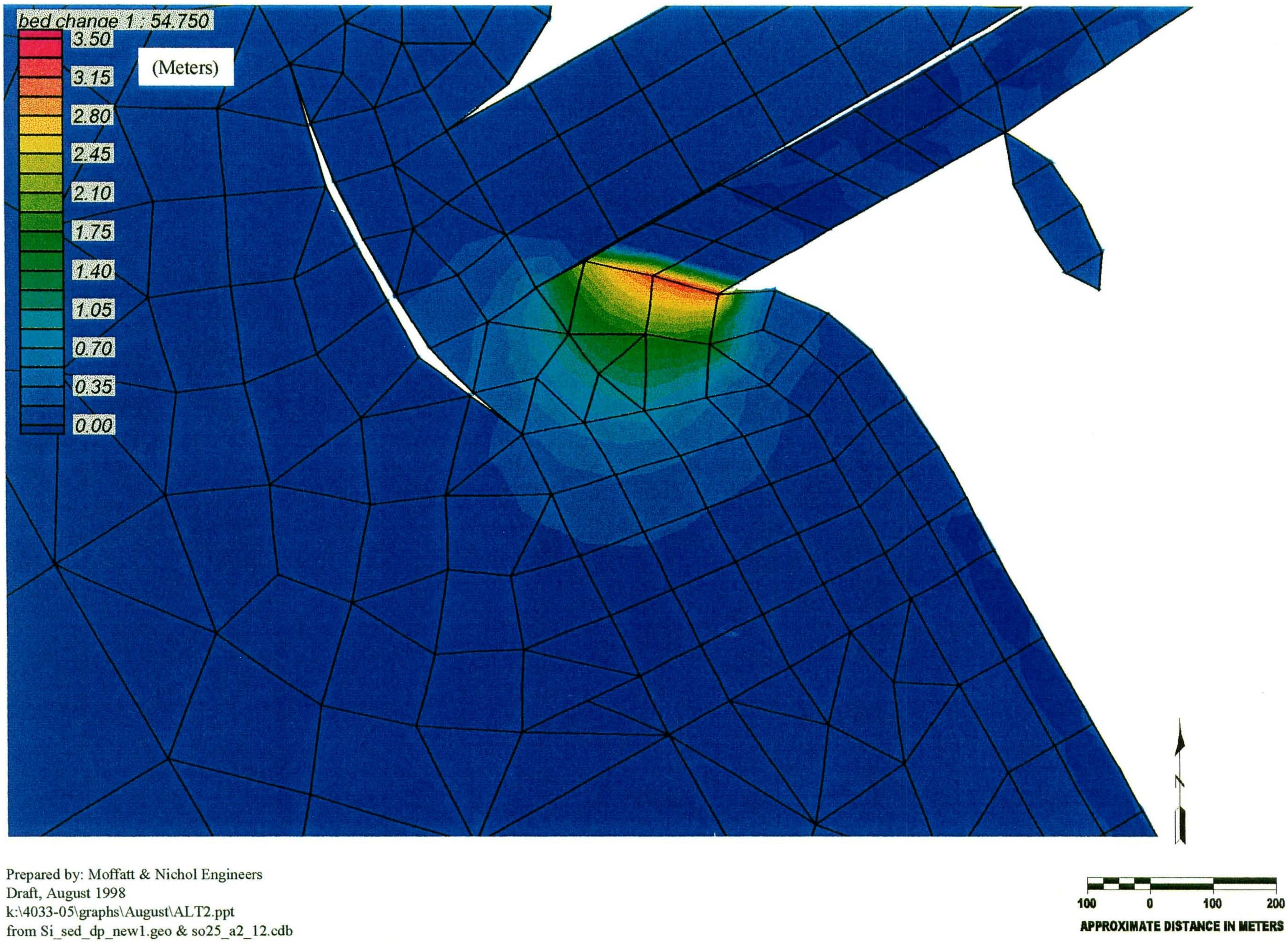


Prepared by: Moffatt & Nichol Engineers
Draft, August 1998
k:\4033-05\graphs\August\ALT1.ppt
from Si_sed_dp_new1.geo & so25_a1_12.cdb

Marina del Rey and Ballona Creek Sediment Transport Studies

Bed Change, Alternative 1 Bathymetry, Spring Tide, 25-Year Flood, Average Wave

Figure 5.39



Prepared by: Moffatt & Nichol Engineers
Draft, August 1998
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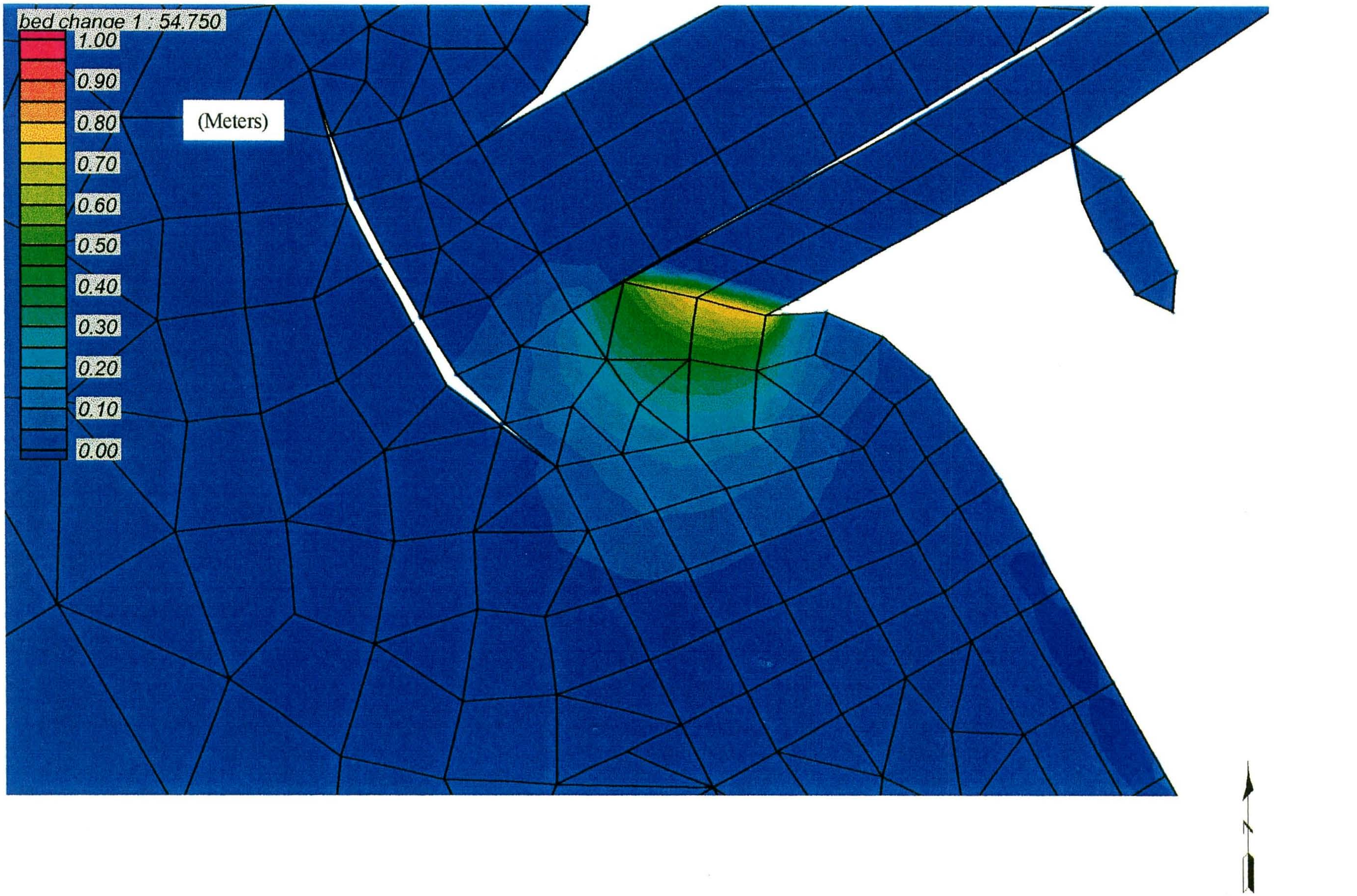
from Si_sed_dp_new1.geo & so25_a2_12.cdb

Marina del Rey and Ballona Creek Sediment Transport Studies

Bed Change, Alternative 2 Bathymetry, Spring Tide, 25-Year Flood, Average Wave

Figure 5.40

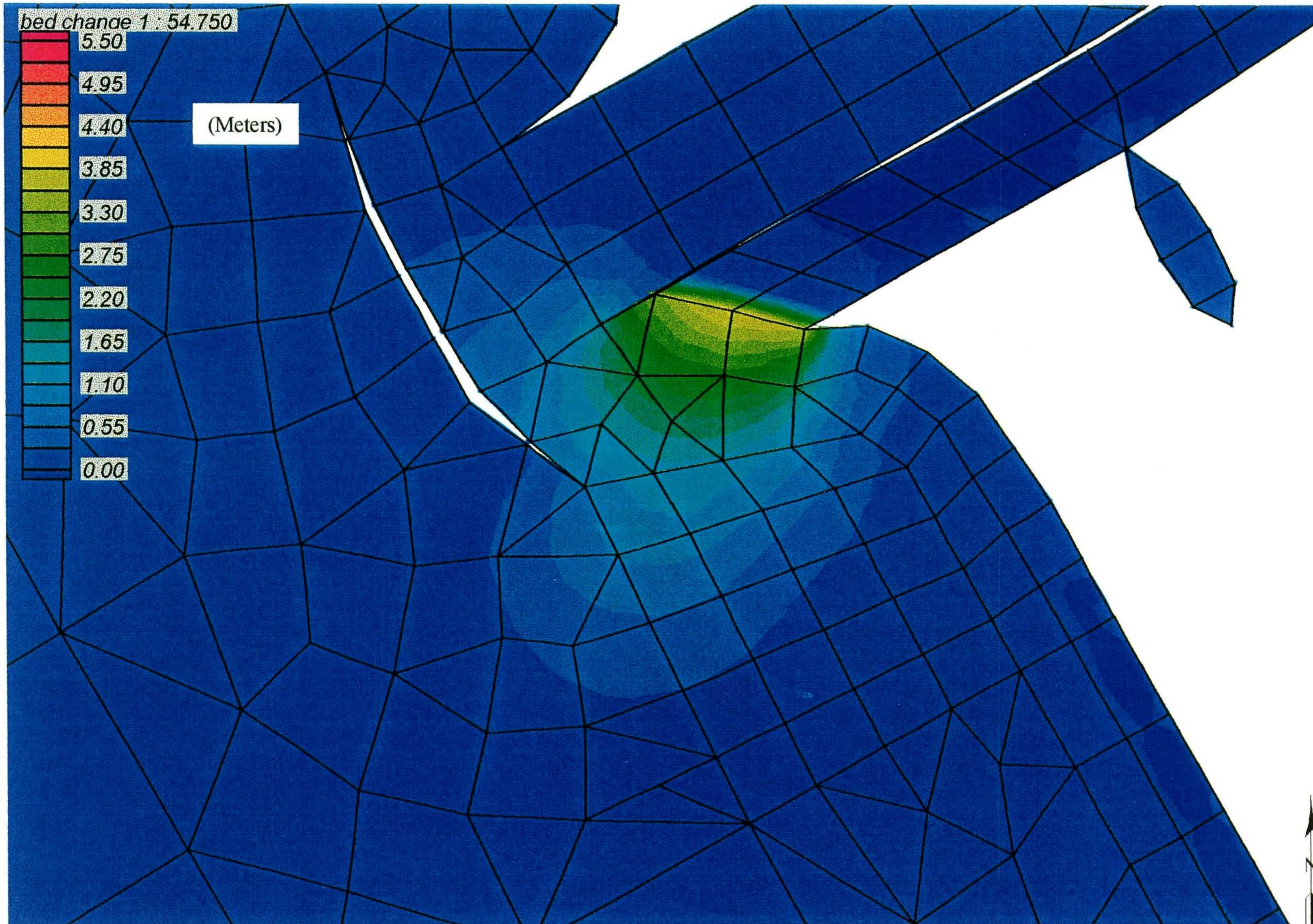


Prepared by: Moffatt & Nichol Engineers
Draft, August 1998
k:\4033-05\graphs\August\ALT1.ppt
from Si_sed_dp_new1.geo & soo5_a1_12.cdb

Marina del Rey and Ballona Creek Sediment Transport Studies

Bed Change, Alternative 1 Bathymetry, Spring Tide, 5-Year Flood, Average Wave

Figure 5.41



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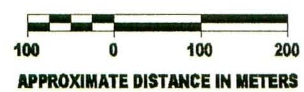
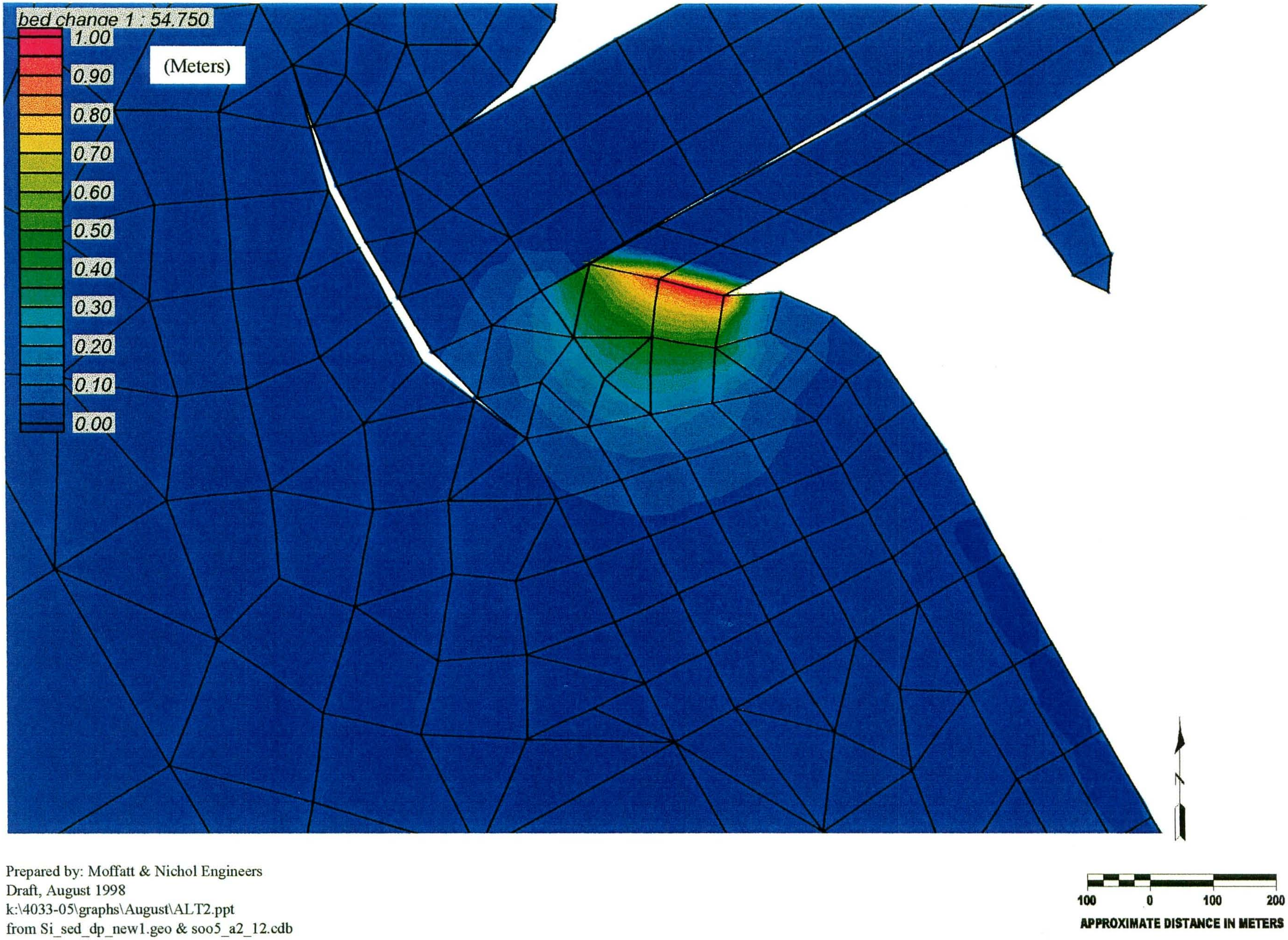


Marina del Rey and Ballona Creek Sediment Transport Studies

Bed Change, Alternative 1 Bathymetry, Spring Tide, 100-Year Flood, Average Wave

Figure 5.42



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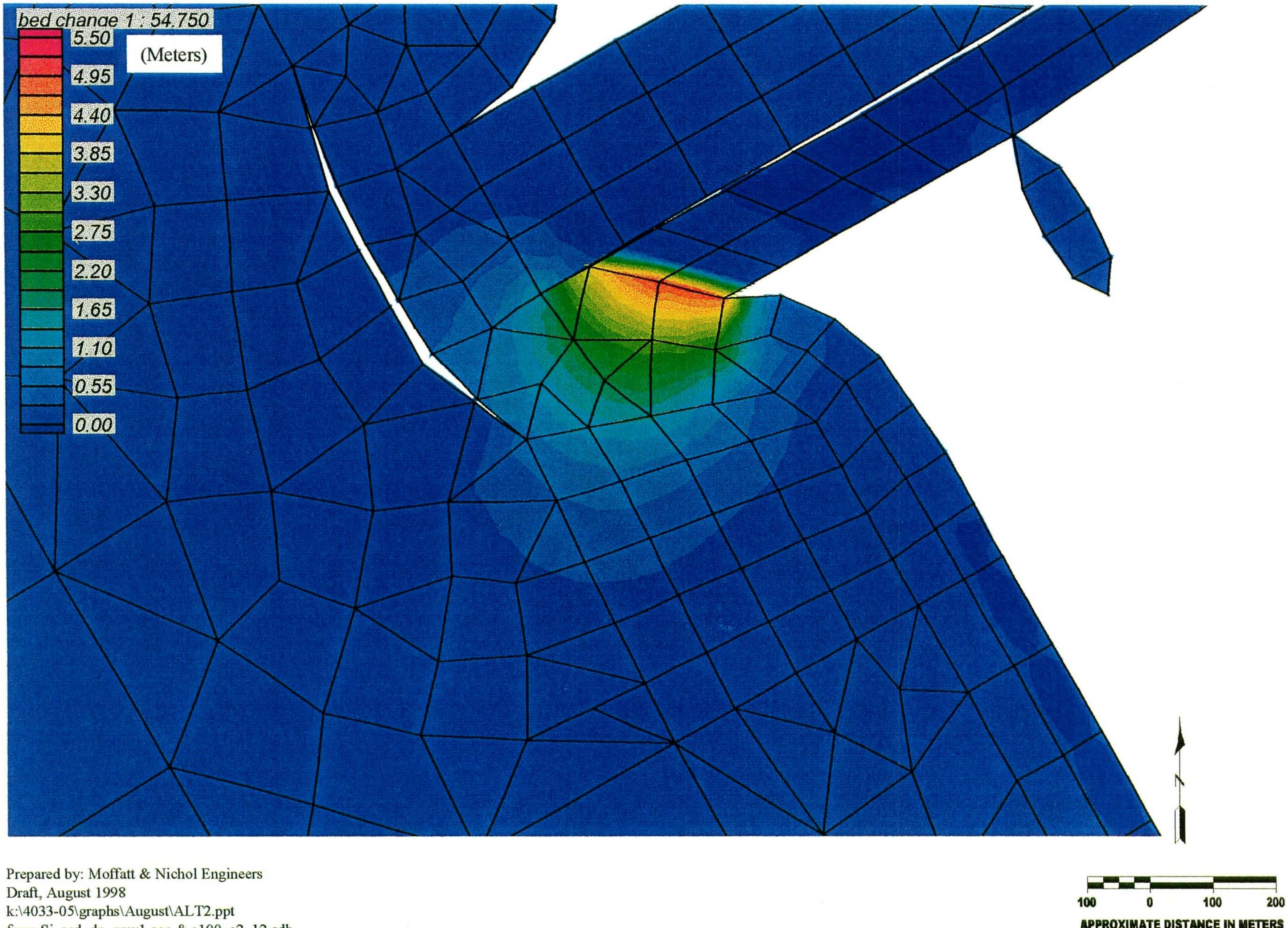
Marina del Rey and Ballona Creek Sediment Transport Studies

Bed Change, Alternative 2 Bathymetry, Spring Tide, 5-Year Flood, Average Wave

Figure 5.43



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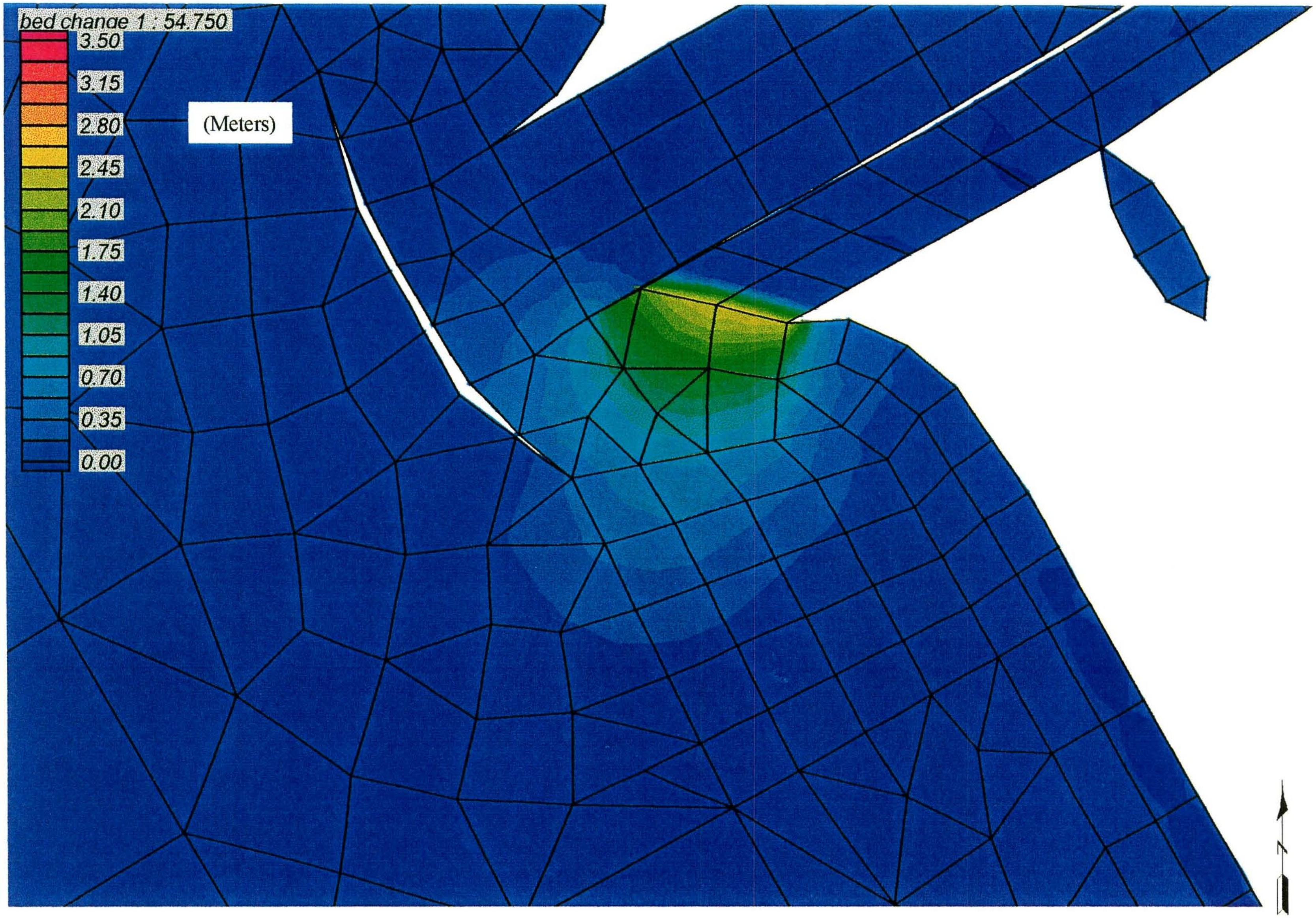
Marina del Rey and Ballona Creek Sediment Transport Studies	Bed Change, Alternative 2 Bathymetry, Spring Tide, 100-Year Flood, Average Wave	Figure 5.44



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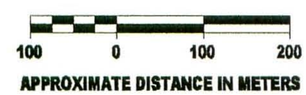
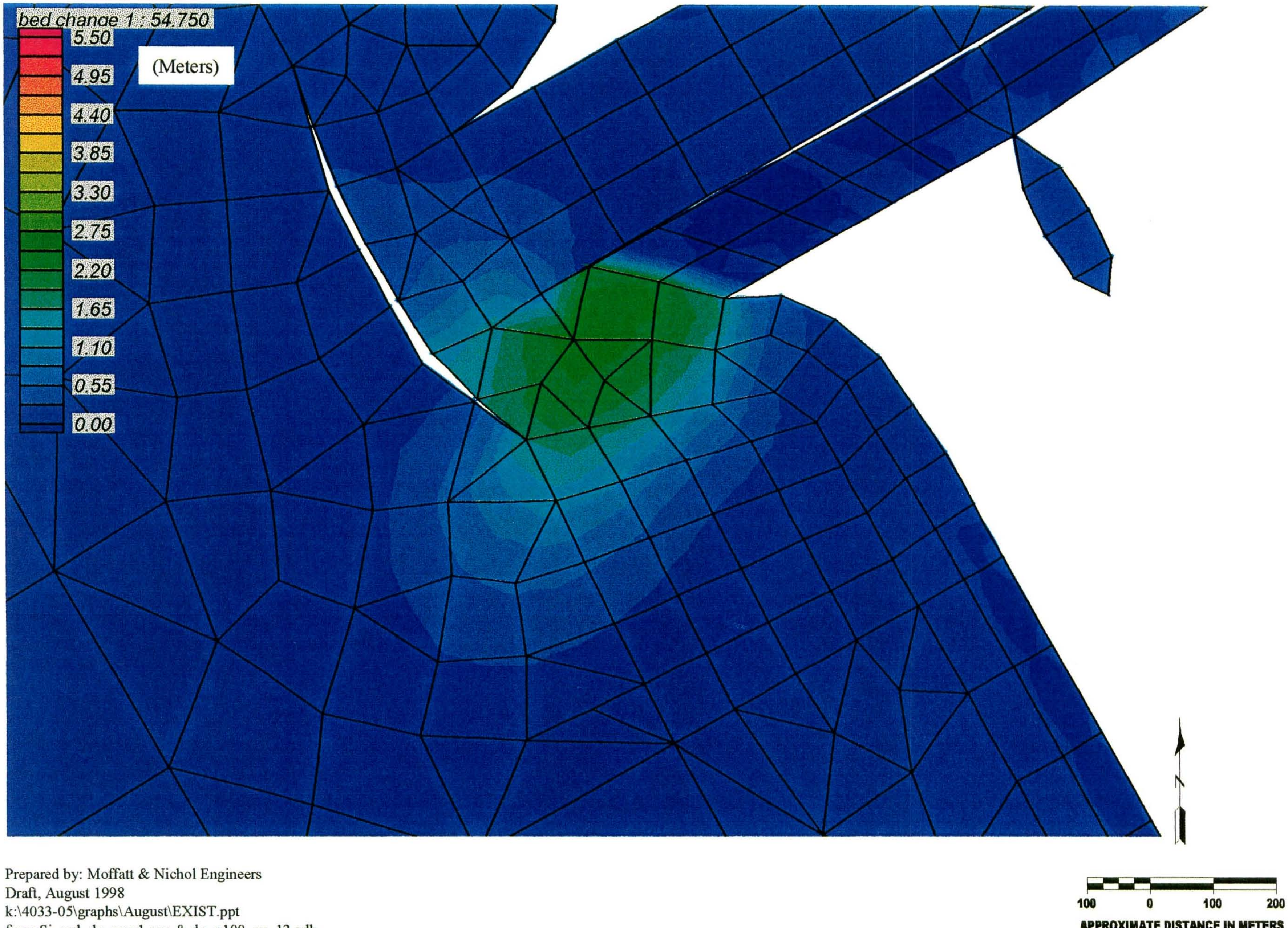


Marina del Rey and Ballona Creek Sediment Transport Studies	Bed Change, Alternative 1 Bathymetry, Neap Tide, 25-Year Flood, Average Wave	Figure 5.45



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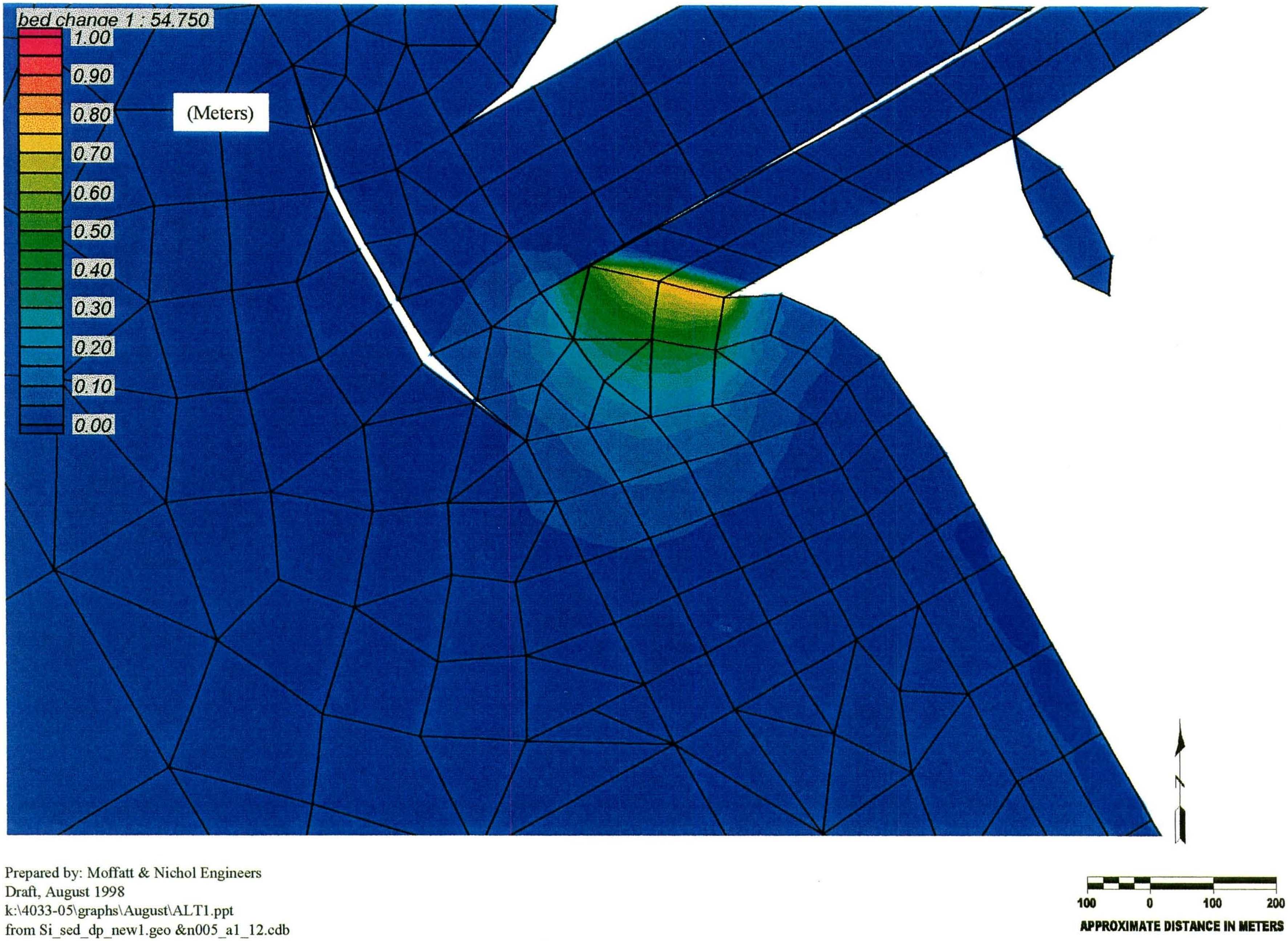
Marina del Rey and Ballona Creek Sediment Transport Studies	Bed Change, Existing Bathymetry, Neap Tide, 100-Year Flood, Average Wave	**Figure** 5.46



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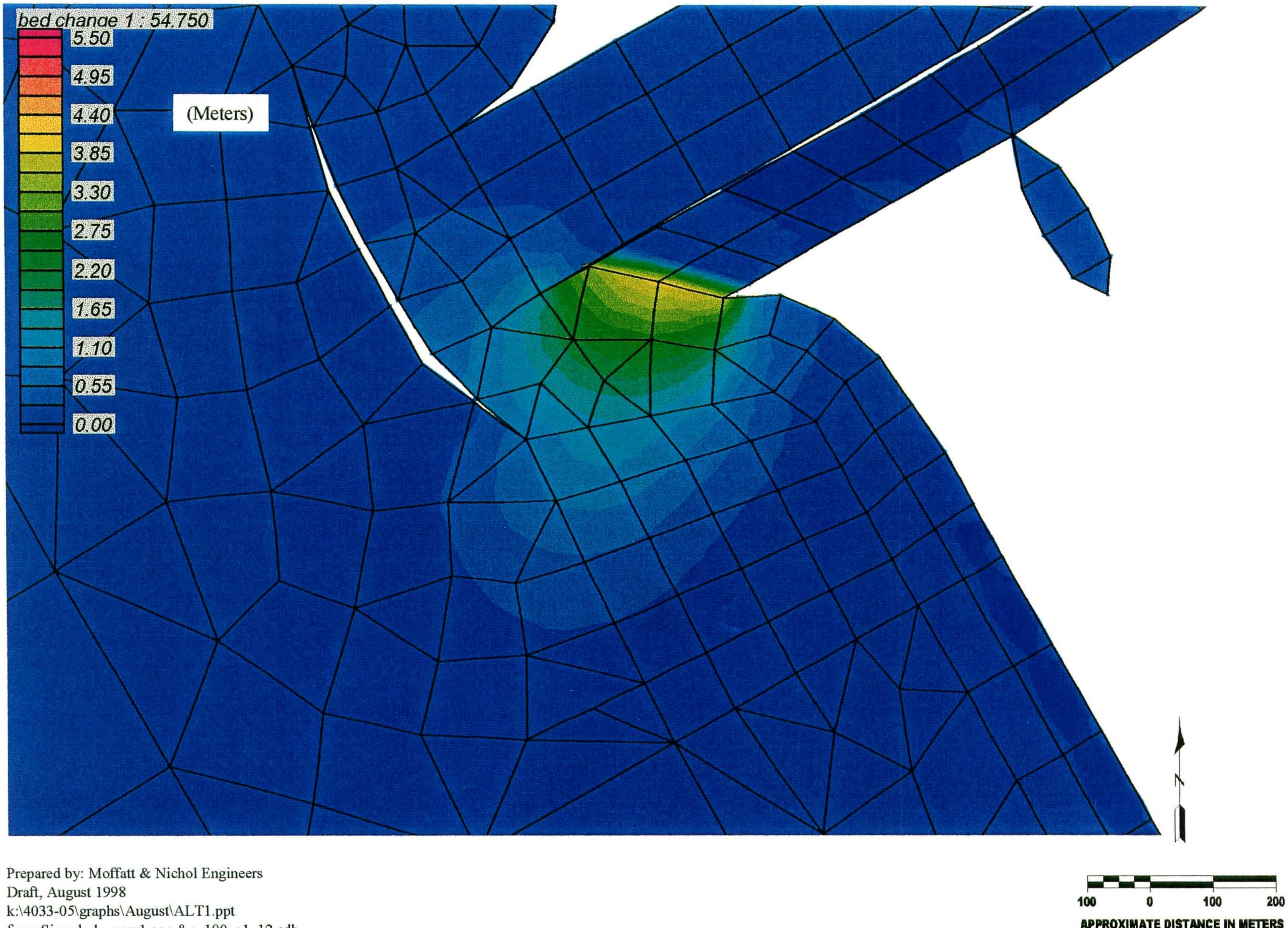
Marina del Rey and Ballona Creek Sediment Transport Studies	Bed Change, Alternative 1 Bathymetry, Neap Tide, 5-Year Flood, Average Wave	Figure 5.47



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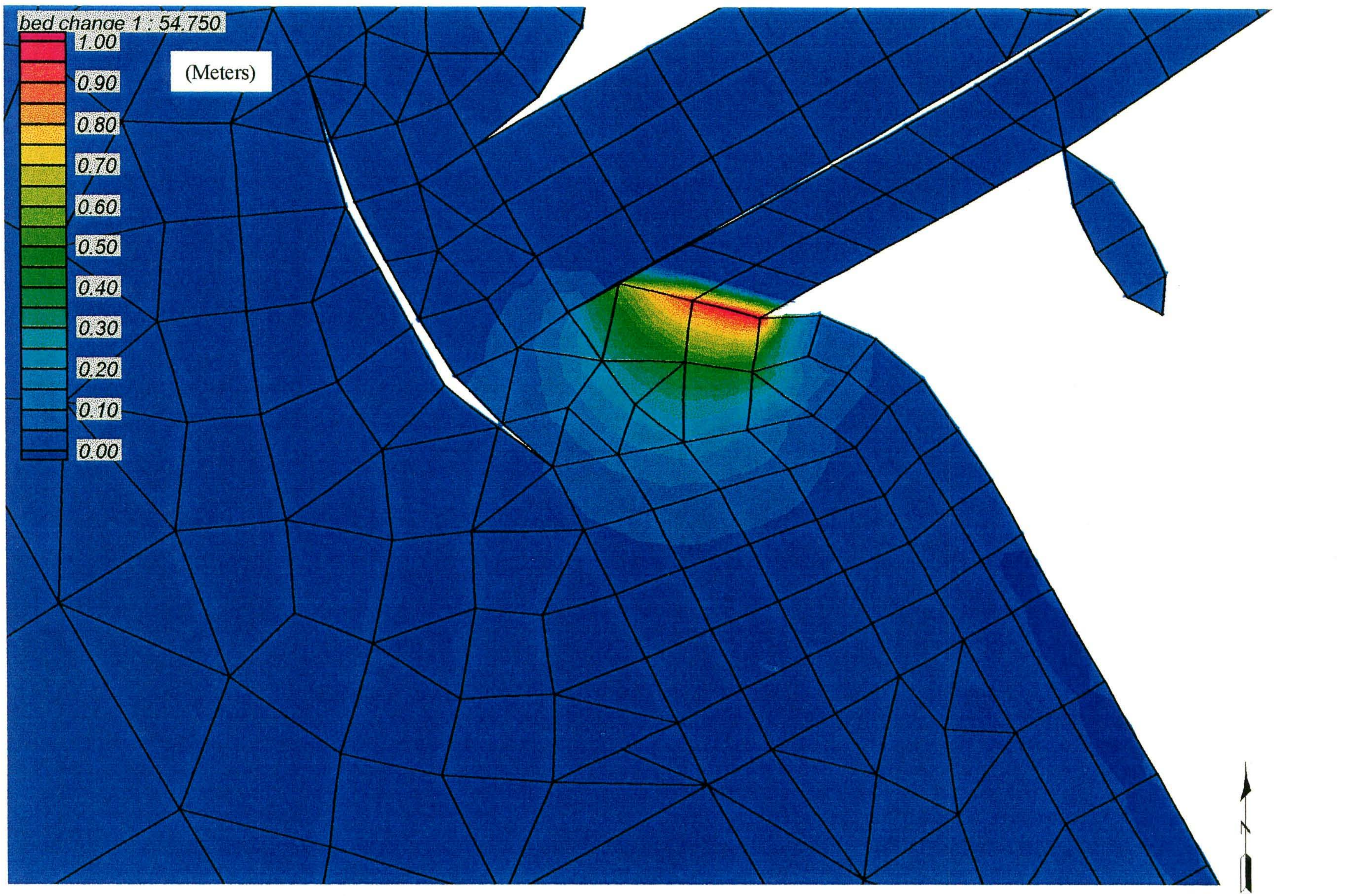
Marina del Rey and Ballona Creek Sediment Transport Studies	Bed Change, Alternative 1 Bathymetry, Neap Tide, 100-Year Flood, Average Wave	**Figure** 5.48



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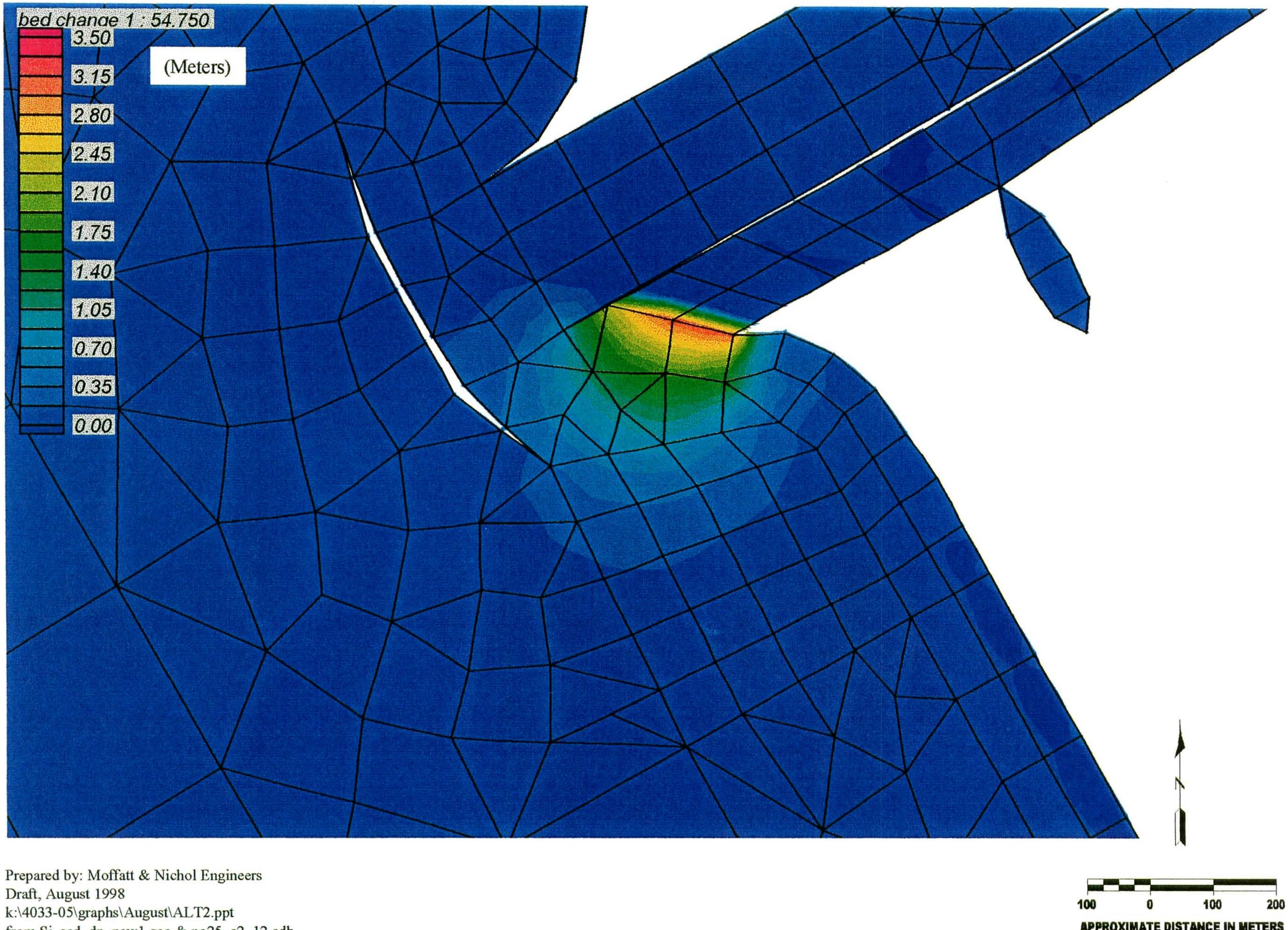
Marina del Rey and Ballona Creek Sediment Transport Studies	Bed Change, Alternative 2 Bathymetry, Neap Tide, 5-Year Flood, Average Wave	**Figure** 5.49



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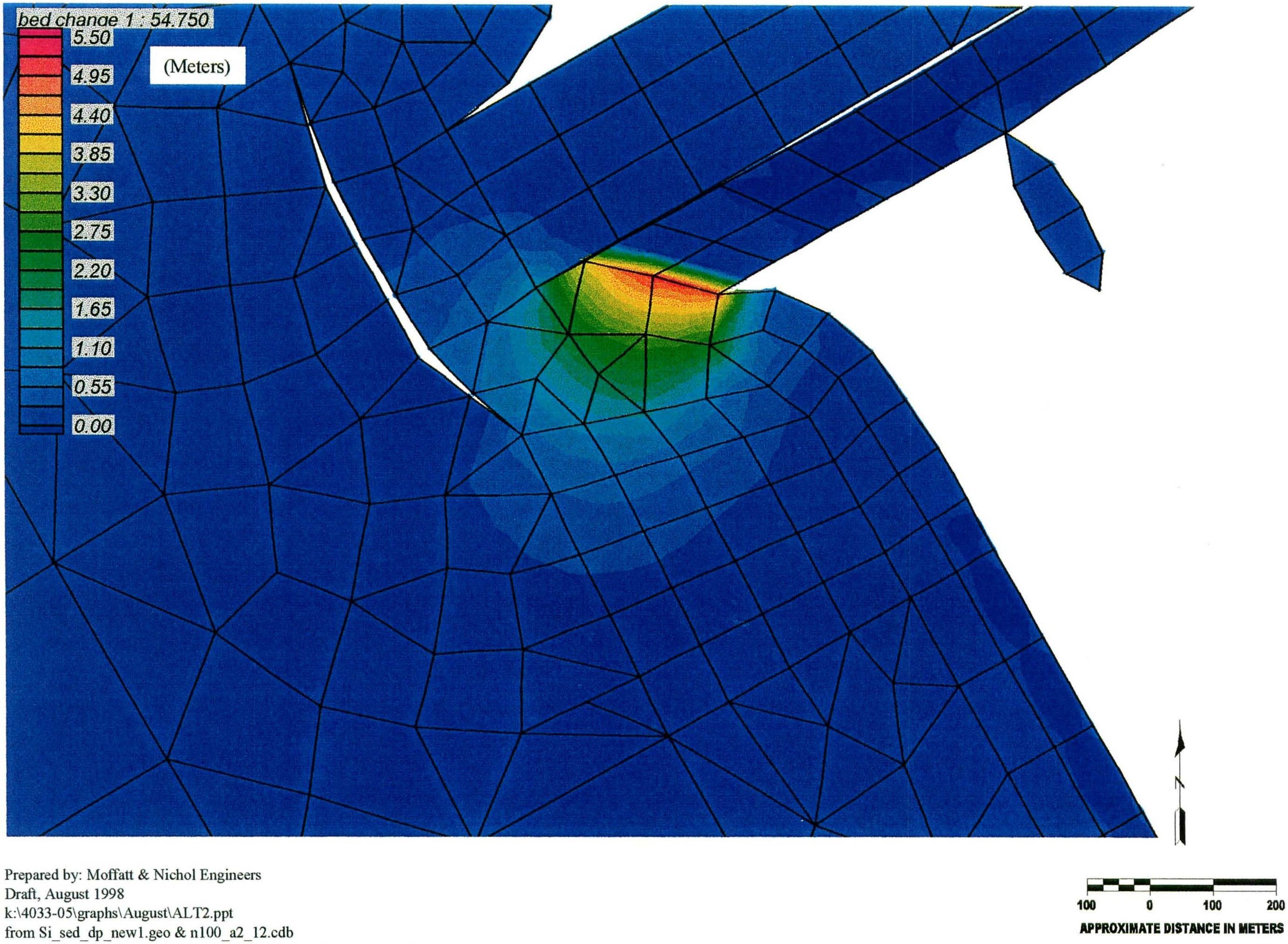
Marina del Rey and Ballona Creek Sediment Transport Studies	Bed Change, Alternative 2 Bathymetry, Neap Tide, 25-Year Flood, Average Wave	Figure 5.50



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Marina del Rey and Ballona Creek Sediment Transport Studies

Bed Change, Alternative 2 Bathymetry, Neap Tide, 100-Year Flood, Average Wave

Figure 5.51

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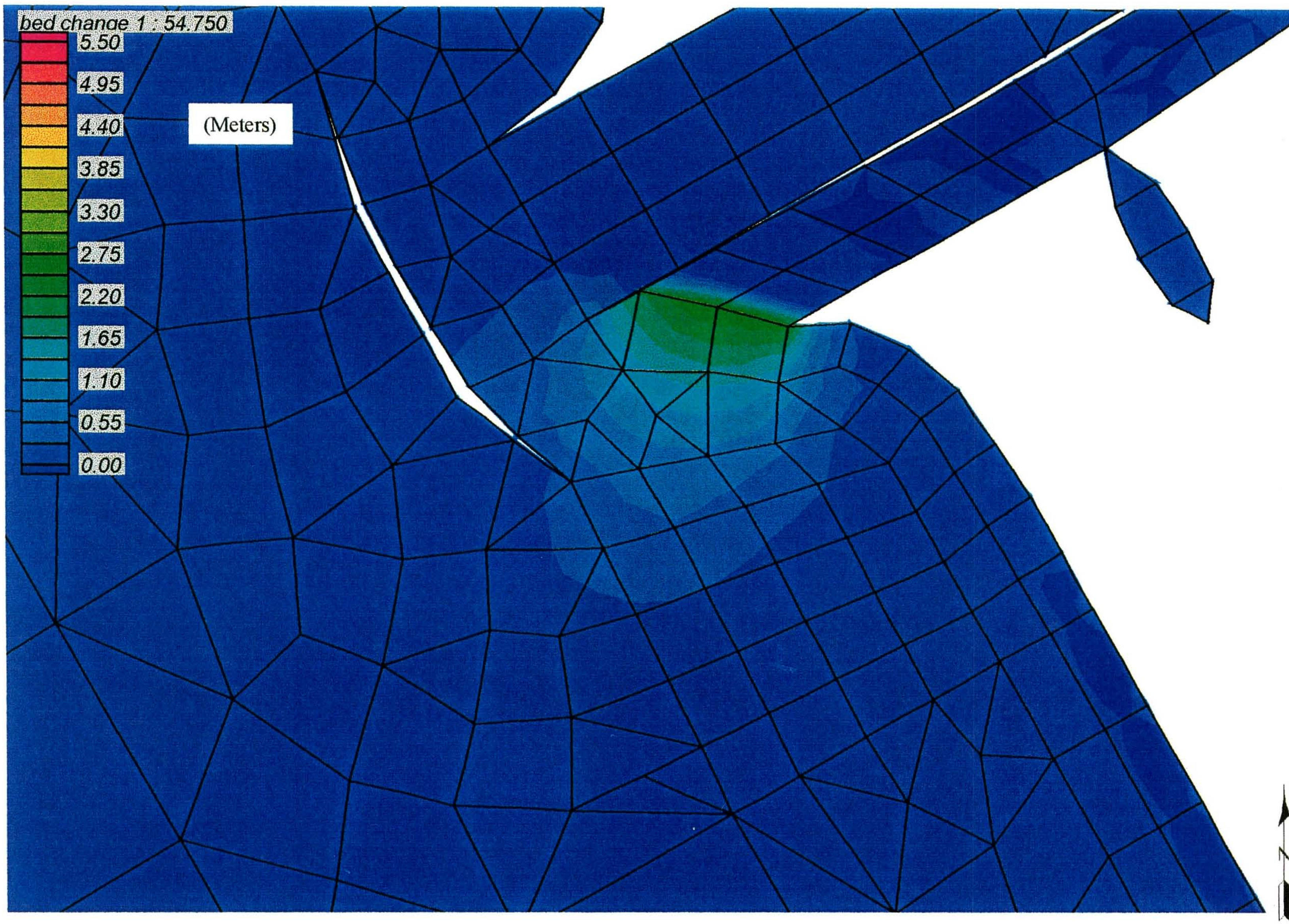
Marina del Rey and Ballona Creek Sediment Transport Studies	Bed Change, Alternative 1 Bathymetry, Spring Tide, 5-Year Flood, Average Wave	Figure 5.52



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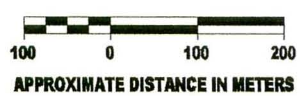
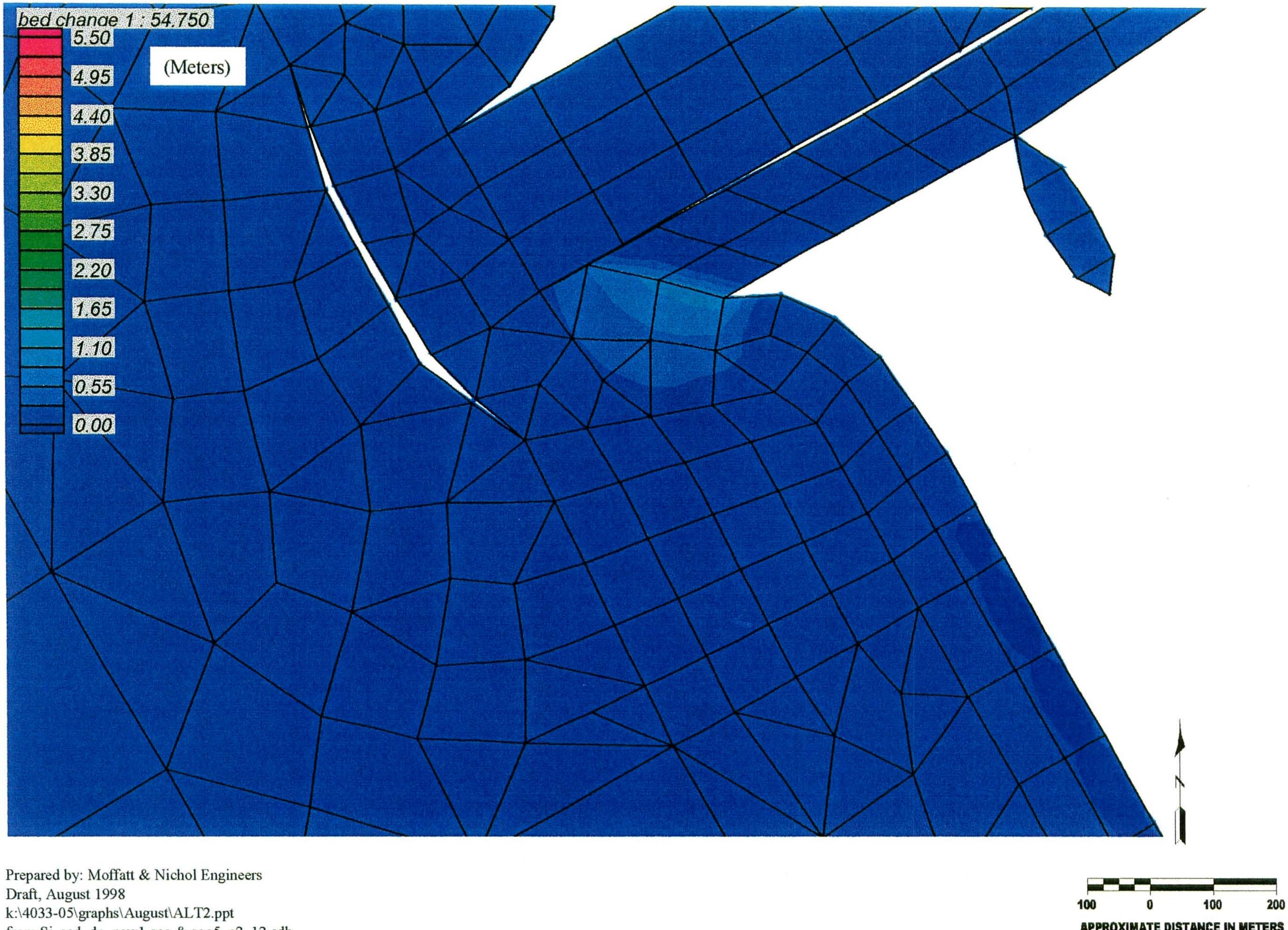


Marina del Rey and Ballona Creek Sediment Transport Studies	Bed Change, Alternative 1 Bathymetry, Spring Tide, 25-Year Flood, Average Wave	Figure 5.53



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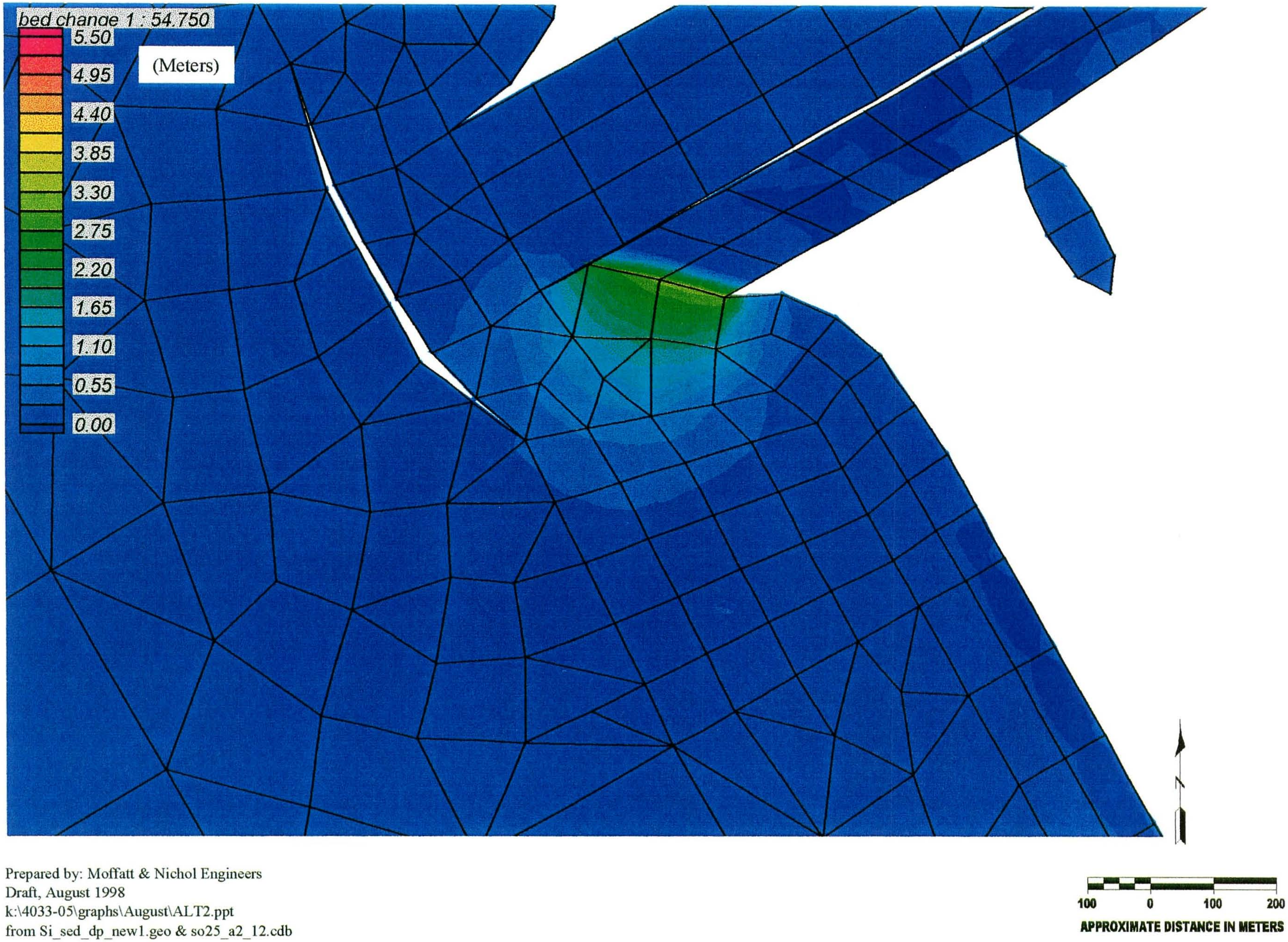
Marina del Rey and Ballona Creek Sediment Transport Studies	Bed Change, Alternative 2 Bathymetry, Spring Tide, 5-Year Flood, Average Wave	Figure 5.54



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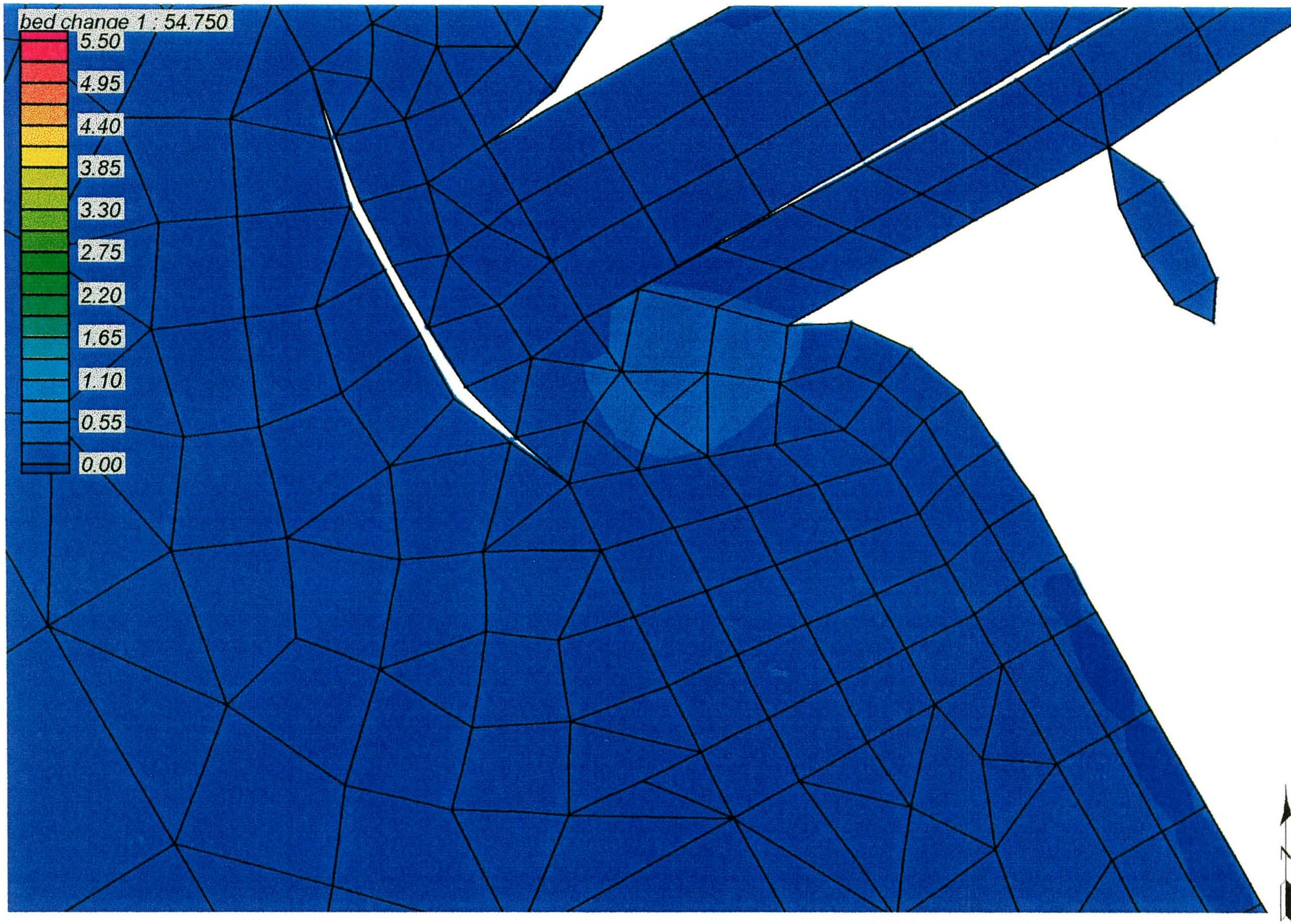
Marina del Rey and Ballona Creek Sediment Transport Studies

Bed Change, Alternative 2 Bathymetry, Spring Tide, 25-Year Flood, Average Wave

Figure 5.55



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Marina del Rey and Ballona Creek Sediment Transport Studies	Bed Change, Existing Bathymetry, Neap Tide, 5-Year Flood, Average Wave	Figure 5.56

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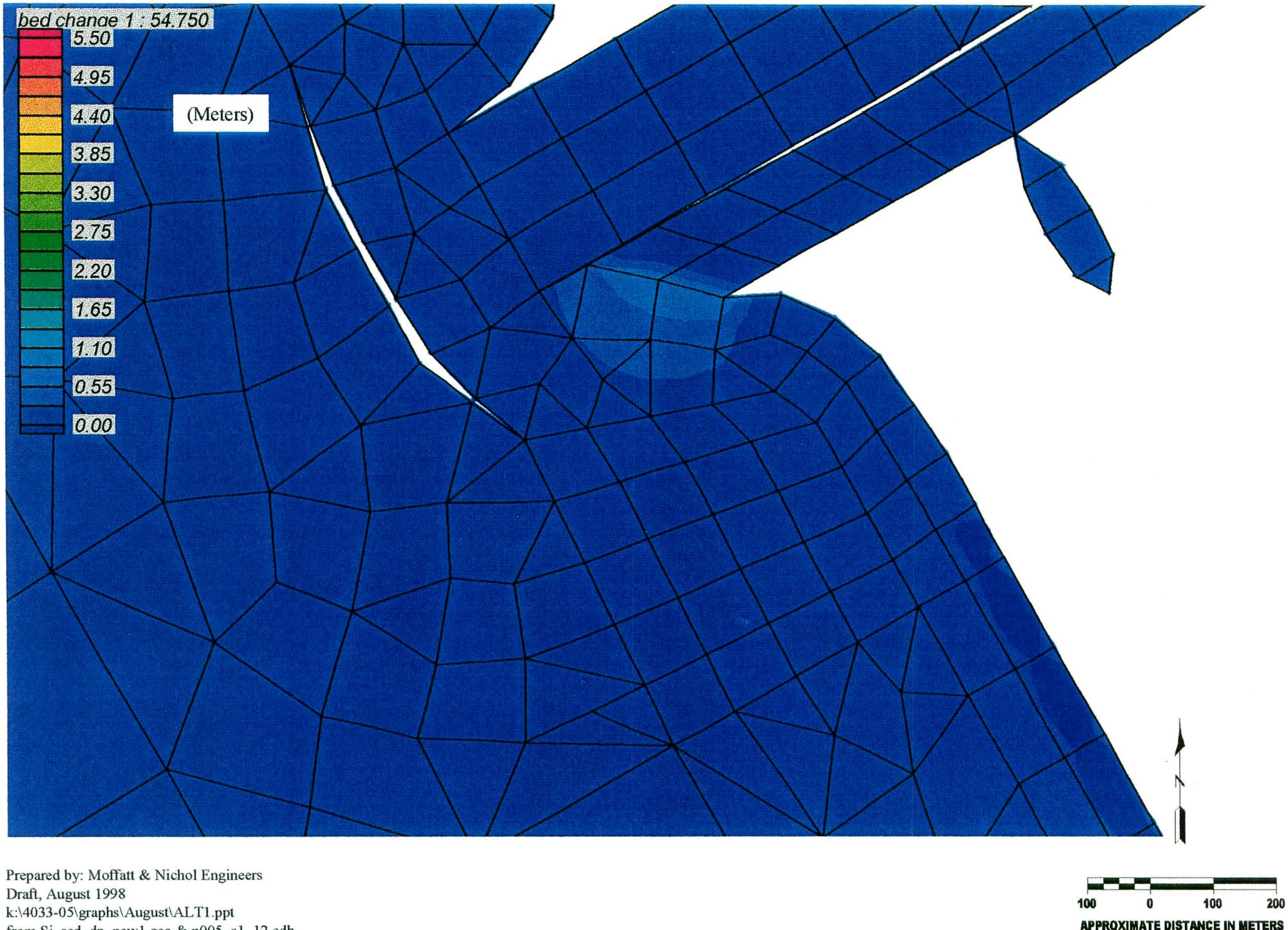
Marina del Rey and Ballona Creek Sediment Transport Studies	Bed Change, Existing Bathymetry, Neap Tide, 25-Year Flood, Average Wave	Figure 5.57



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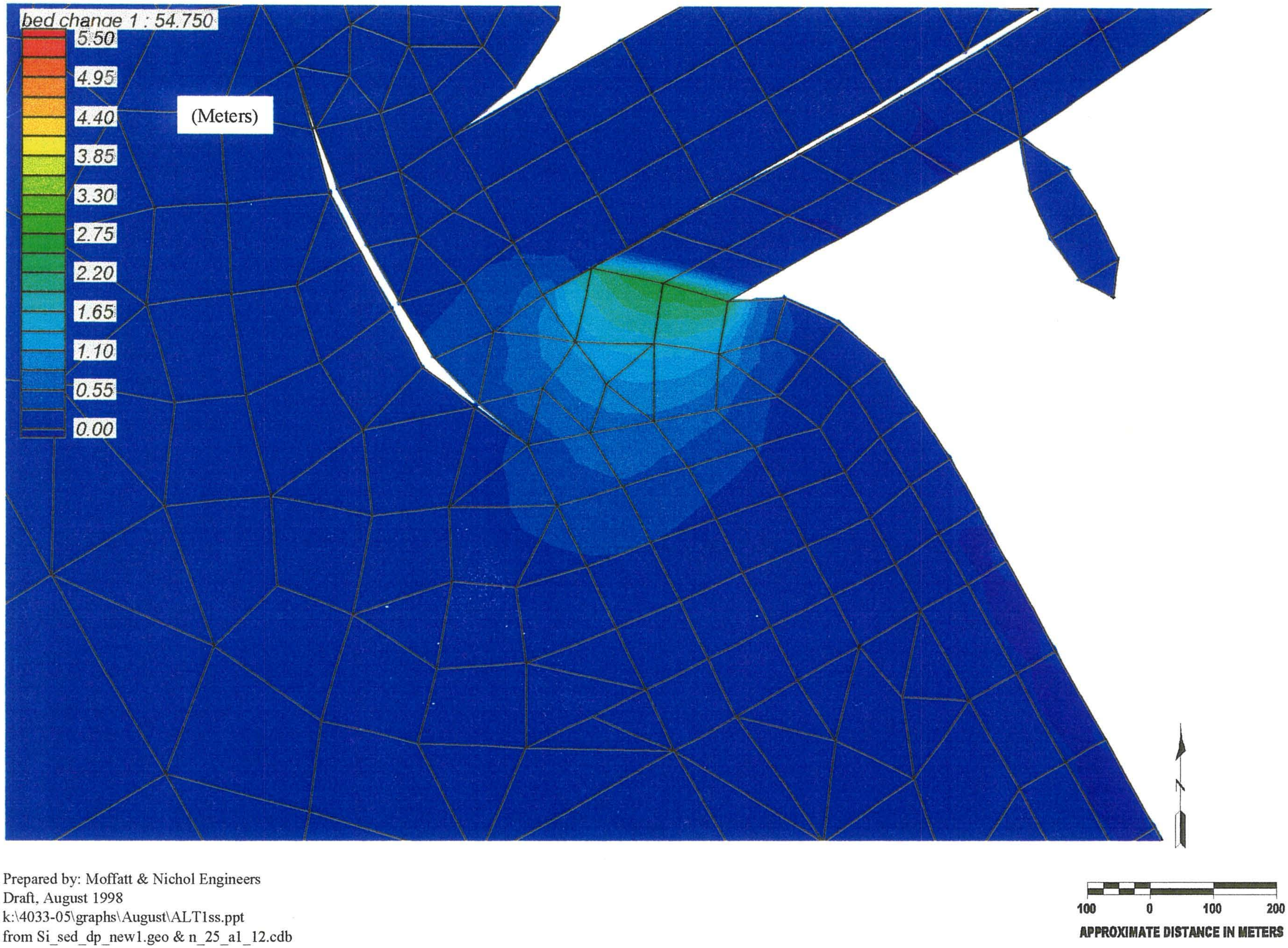
Marina del Rey and Ballona Creek Sediment Transport Studies

Bed Change, Alternative 1 Bathymetry, Neap Tide, 5-Year Flood, Average Wave

Figure 5.58



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Marina del Rey and Ballona Creek Sediment Transport Studies

Bed Change, Alternative 1 Bathymetry, Neap Tide, 25-Year Flood, Average Wave

Figure 5.59

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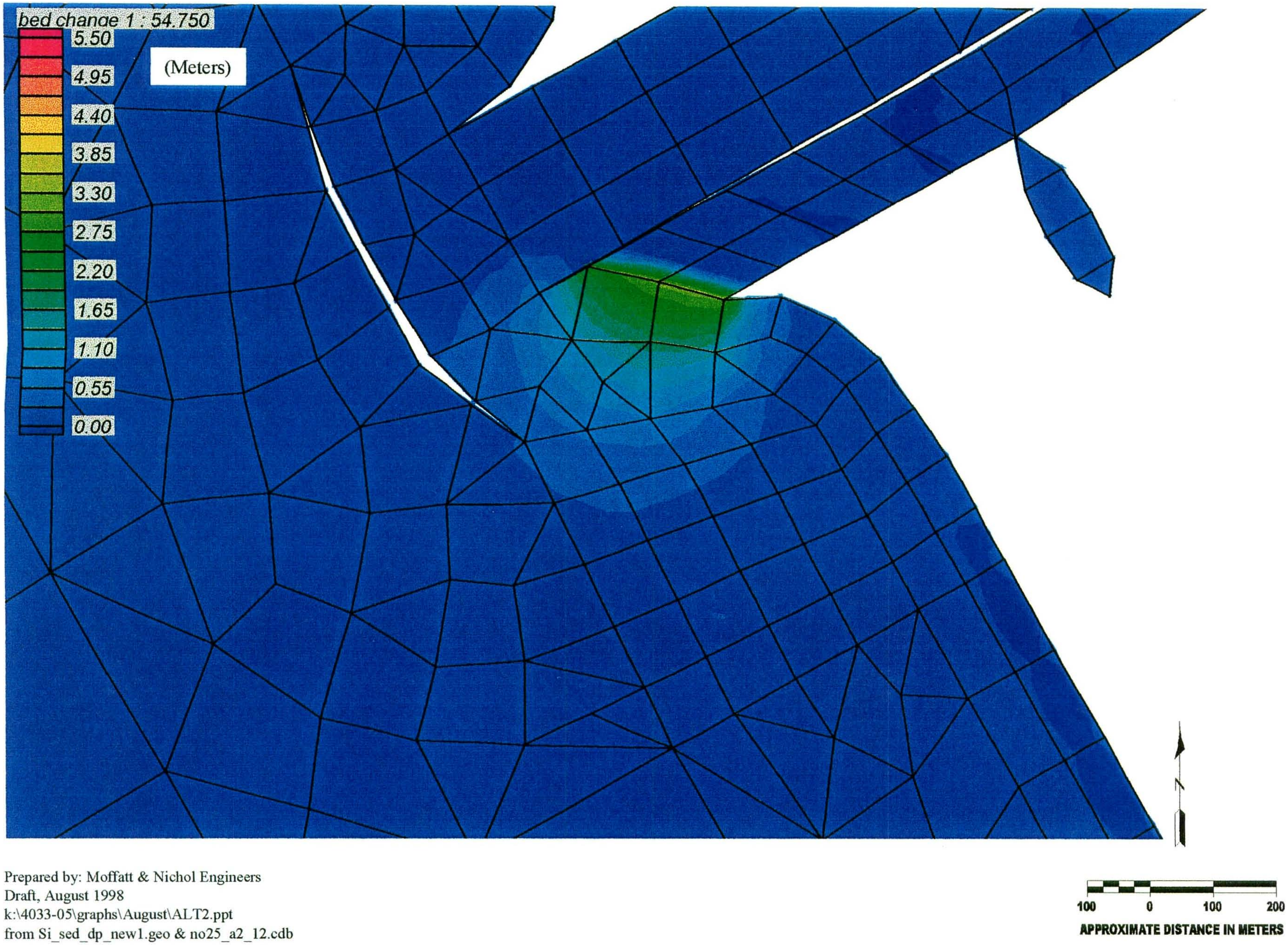
Marina del Rey and Ballona Creek Sediment Transport Studies	Bed Change, Alternative 2 Bathymetry, Neap Tide, 5-Year Flood, Average Wave	Figure 5.60



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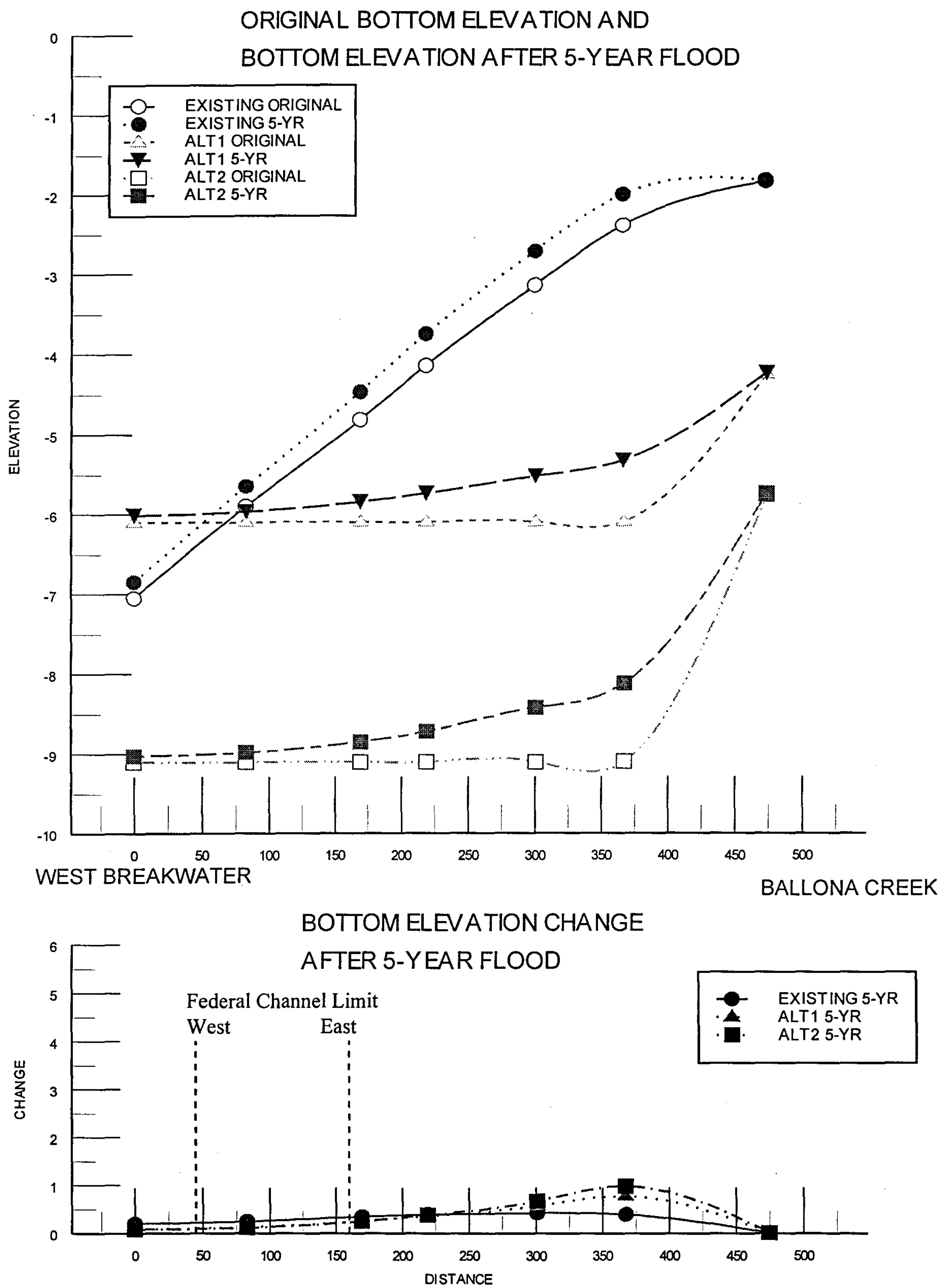
Marina del Rey and Ballona Creek Sediment Transport Studies

Bed Change, Alternative 2 Bathymetry, Neap Tide, 25-Year Flood, Average Wave

Figure 5.61



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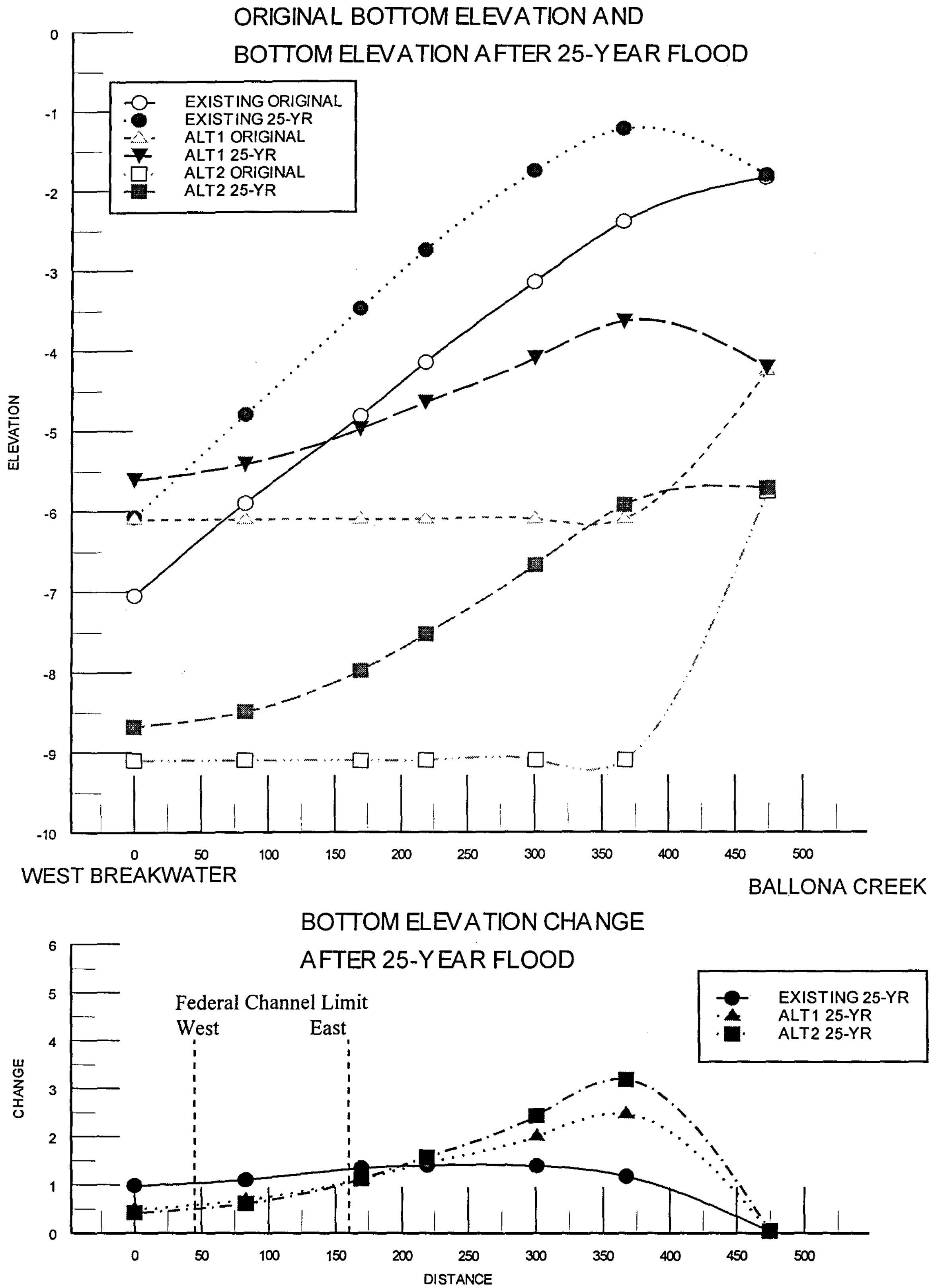
UNITS IN METERS

Marina Del Rey and Ballona Creek Sediment Transport Studies	Section A, 5-yr Flood, Bottom Elevation and Change in Bottom Elevation	Figure 5.62



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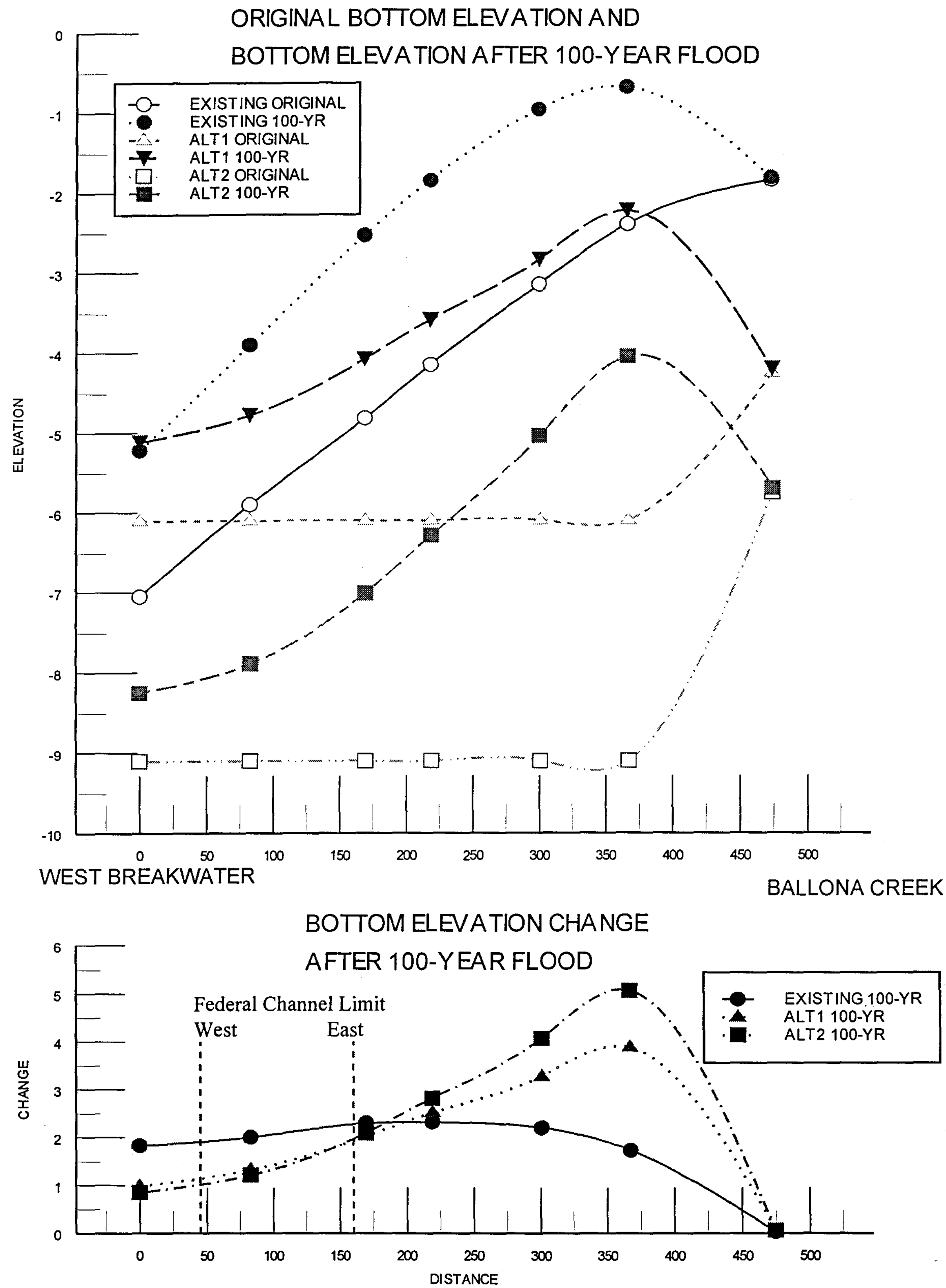
UNITS IN METERS

Marina Del Rey and Ballona Creek Sediment Transport Studies	Section A, 25-yr Flood, Bottom Elevation and Change in Bottom Elevation	Figure 5.63



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UNITS IN METERS

Marina Del Rey and Ballona Creek Sediment Transport Studies	Section A, 100-yr Flood, Bottom Elevation and Change in Bottom Elevation	Figure 5.64

6.0 DREDGED MATERIAL TREATMENT

This chapter presents a preliminary review and analysis of dredged material treatment technologies. The following three treatment technologies are evaluated:

- Physical separation.
- Cement-based stabilization/solidification.
- Physical mixing.

The technologies are analyzed in terms of purpose, process characteristics, applicability and maturity. Major field experiences of the processes are discussed, along with logistics and costs, to provide a planning basis for potential application in the management of the contaminated dredged materials from Marina del Rey.

6.1 Physical Separation

Physical separation as an alternative for treating contaminated dredged materials is reviewed and analyzed in this section. The purpose, applicability, and process characteristics of the alternative are discussed first, followed by a review of existing, tested technologies that can be considered for application in treating contaminated dredged materials. The logistics and costs of the process are examined to provide a preliminary basis for potential local application.

6.1.1 Purpose

Contaminants including metals and organic compounds are known to be primarily associated with the fines and organic particles in a contaminated dredged material. Since the acceptability of a dredged material for less restricted disposal is commonly based on contamination levels of sediment aggregates rather than those of its grain-size fractions, indiscriminate, summary disposal solutions have often been adopted, resulting in not only in ever decreasing availability of upland and aquatic areas suitable for disposal, but also significant losses of sand resources. Such problems can impact economy over time with the expected growth of future navigational maintenance dredging volumes. It is therefore important to seek solutions which reduce the total volume that is necessary for restricted disposal through maximizing sediment recovery from dredged materials for beneficial uses. Physical separation is one of the simpler processes that can be used to recover usable sediments from contaminated sediment aggregates such as dredged materials.

6.1.2 Applicability

Long an established process in mineral processing industry, the physical separation technology separates the fines from a sediment aggregate, creating a product of coarser material. The success of this technology as a means for contaminated sediment volume reduction, therefore, relies on the premise that the contaminants in an aggregate are predominantly associated with the fines, with the rest of the material being substantially clean. In general, most of the dredged materials from navigation maintenance projects satisfy this treatability criterion.

6.1.3 Process

Physical separation processes usually involve separating sediment grains by their grain size and density differences. For application in processing contaminated sediments, there are many types of processes which differ primarily in configuration, production capacity, and the level of treatment required, as well as capital investment. For all the differences, four process phases are commonly involved, and are the core of most of the processes:

- screening,
- slurrying,
- scrubbing, and
- cycloning.

A conceptual process diagram is shown in Figure 6.1. The screening phase removes coarse rocks, debris and other oversize blocks from the contaminated sediments producing a relatively uniform sandy/silty material. In the slurrying phase, water (sometimes with additives) is introduced and mixed with the sediments to create a slurry feed to the system. The slurry feed then goes through a scrubbing unit which

- further disintegrates the aggregate,
- dislodges the fines from the coarser grains, and

- removes surficial contaminants on the coarser grains through abrasive scouring among particles.

With grain-size fractions dislodged from each other and the coarser grains appreciably cleaned from abrasion, the scrubbed slurry feeds into a hydrocyclone for fractional separation through grain-size and density differences, producing

- a contaminated concentrate of fines, and
- a substantially clean coarse material.

A Met-Pro Hydrocyclone is shown in Figure 6.2 (Met-Pro Supply, Inc.). The median grain size to be recovered and the production capacity needed determines the size of the hydrocyclone to be selected. The recovery grain size Met-Pro Hydrocyclone can achieve ranges from 0.010 mm to 100 mesh. The Met-Pro Hydrocyclone is capable of handling up to 1,000 tons per hour of sediments depending on the percent solids content of the slurry (up to 85%) and, in general, the density of the solids.

The cycloned contaminated concentrate can be further treated using a variety of technologies or be disposed of in appropriate landfills. The substantially clean coarse material can be put to various beneficial uses.

The reduction of the total contaminated sediment volume to a much smaller concentrate of contaminated fines provides the opportunity of applying many existing treatment technologies that would not have been possible economically. The contaminated volume reduction also permits a wider selection of disposal options that would have been limited by the volumetric requirements.

6.1.4 Application: Linatex Process

Physical separation as a central part of contaminated sediment treatment technologies has been used mostly in a variety of contaminated sediment washing and recycling processes. Although the elements of these technologies are generally established, their systematic application to treating contaminated sediments is relatively new. Most of the more successful processes were developed and demonstrated under the coordination of EPA (1994a, 1994b), and are considered commercially ready by EPA.

A number of physical separation technologies that have been applied to contaminated sediment treatment were examined in the present study, including those of Linatex (Bergmann USA), Biotrol and AEA. The Linatex process was found to be the most applicable technology among those reviewed in terms of project capacity and application history for contaminated dredged materials. This process and its pilot demonstration project for treating contaminated dredged material in Saginaw Bay, Michigan under EPA and COE are reviewed and discussed in the following sections.

Process

The Linatex process is a mineral processing technology that has been applied in treatment or pretreatment of contaminated sediments including dredged materials from navigational channel maintenance projects. A diagram of the process as configured for the EPA/COE Saginaw Bay demonstration project is shown in Figure 6.3.

The sediments are first adequately dewatered in a storage area before being loaded onto a conveyor with a feed hopper, where sediments are screened through a grate to remove oversize blocks. The relatively uniform sediment aggregate is then fed by the conveyor into a rotary trommel where it is mixed with water to become a slurry and subsequently deagglomerated through tumbling. The tumble-washed slurry is screened at 6mm grain size, where the fraction larger than 6mm discharged to a collector as oversize material (trommel overs) and with the rest (<6mm) to be washed by a series of hydrocyclones, a dense media separator and an attrition scrubber. A schematic of a Linatex Hydrocyclone is shown in Figure 6.4. The overflow stream from the primary hydrocyclone (1) contains substantial contaminant-enriched fines and organic particles (including humic materials, e.g. leaves, twigs, roots, wood chips), which are separated by a rotary screen and discharged as separate waste streams to storage collectors. Organic particles are also separated from sediment slurry by a dense media separator, which then feeds the output sediment slurry to an attrition scrubber for washing by scrubbing to further dislodge the fines from coarse grains. The process is complete after a 2-stage hydrocycloning, which produces washed sand and feeds overflows (water and small quantity of fines) back to the beginning of the process to be recycled as process water for slurrying feed sediments.

The process creates four output products:

- clean coarse material,
- contaminated fines,
- contaminated organic particles, and
- process water.

The clean coarse material can be recycled as construction material (e.g. concrete, masonry, and asphalt). The concentrated contaminated fines and the humic materials can be further treated (e.g. stabilization and immobilization) or disposed of at a regulated landfill. The process water can be treated to remove solids (flocculation/sedimentation), oil (separation), and soluble heavy metals (dissolved air flotation), and returned to the process for reuse.

Applicable Constituents

The constituents the Linatex process has treated include

- heavy metals, and
- organic compounds.

These include cadmium, chromium, copper, lead, nickel, zinc, creosote, cyanides, fuel residuals, mercury, heavy petroleum and PCBs. The process is in principle applicable to a wide range of constituents that have the tendency to bond with sediment fines.

Throughput Capacity

50 tons per hour for separation at grain size around 0.075 mm, which translates to around 500 cubic meters per day assuming an in-situ density of 1.6 and a 2-shift, 16-hour day. Systems up to 350 tons per hour are available.

Maturity

The technology was demonstrated at Port of Toronto, Canada, in April 1991, which was profiled by EPA (1994a) with detailed results presented in a series of EPA reports (EPA/520-MR-92/015, EPA/540-AR-93/517, EPA/540/SR-93/517), and at Saginaw Bay Confined Disposal Facility in Saginaw, Michigan, in May 1992. Both demonstrations involved treatment of contaminated sediments from harbor area and navigation channels. Twenty-eight commercial systems had been applied by 1994 (EPA, 1994a). The technology is considered commercially ready by EPA.

Applicability Evaluation

The Linatex process is among the most applicable separation technologies for treating contaminated dredged materials based on a technology review conducted as part of this study. The process production capacity is much higher than other processes available, making it desirable for treating the normally large volumes of sediments from a dredging project. The contaminant constituents the process has treated generally match the contaminant profiles of a typical contaminated dredged material. The fact that the process has been demonstrated for harbor sediments adds to the potential for full-scale dredged material application. The demonstration project conducted in Saginaw Bay, Michigan, is discussed in the following section.

6.1.5 Epa/Coe Saginaw Bay Demonstration

The EPA/COE Saginaw Bay pilot-scale contaminated sediment remediation demonstration project was initiated in 1990 and completed in 1994. The project was conducted as part of the Assessment and Remediation of Contaminated Sediments (ARCS) Program under the EPA Great Lakes National Program Office. The COE Detroit District prepared the project report (EPA, 1994c) for the EPA under an interagency agreement between the COE and the EPA. The general objective of the project was to demonstrate the effectiveness of sediment washing as a treatment technology for contaminated sediments. Specific objectives include the determination of

- the efficiency of separating silt/clay and organic particles from a sandy dredged material,
- the system component performances in achieving desired separation,

- material handling and pre-processing requirements,
- process products characteristics and disposability, and
- cost basis for full-scale application.

Sediments

The sediments used in the project are dredged materials from federal navigation channels in Saginaw Bay in Lake Huron and Saginaw River, which discharges into Saginaw Bay. The combined current annual channel maintenance volume was estimated at approximately 260,000 cubic meters. A site map is shown in Figure 6.5.

The dredged sediments from the channels are typically contaminated with PCBs, PAHs, DDT and metals. For purposes of treatment performance tracking, PCB (polychlorinated biphenyl), TOC (total organic carbon), and a group of metals were used as indicator constituents. The project sediments are composed of 76.1% sand (>0.075 mm) and 23.5% fines (silt/clay; <0.075mm), with a composite median grain size of approximately 0.10-0.15mm.

Operations

The demonstration project consists of the following operational phases:

- Site preparation.
- Plant assembly.
- Dredging, transport and storage.
- Feed preparation.
- Sediment washing.
- Washed sediments management.
- Residuals management.

The COE Saginaw Bay Confined Disposal Facility, a diked island CDF northeast of Saginaw River mouth as shown in Figure 6.5, was selected to host the project setup. Figure 6.6 shows the layout of site plan and plant assembly.

The site was prepared near the dike by creating a bermed dredged material (feed) storage area (lined with geotextile fabric) for stockpiling untreated sediments, three corrugated steel settling enclosures (also lined with geotextile fabric) for settling and filtering fines discharged from the process, and three storage areas for storing washed sand, organic particles, and oversize materials separated out from the rotary trommel.

The Linatex (then Bergmann USA) sediment separation plant was shipped in modules to a coal dock at the mouth of the river, and was assembled on a COE barge over a period of 15 days. The plant barge was then towed to the site, moored against the dike and connected to the storage areas through conveyors as shown in Figure 6.6.

A total of approximately 600 cubic meters of sediments were dredged by clamshell, barged to the site, and offloaded to the feed storage area. The stockpiled dredged material was allowed to dewater for 15-25 days. The dewatered sediments were then loaded into the feed hopper of the feed material conveyor using a front-loader, which initiated the sediment separation process.

The Linatex sediment separation process used in the project is discussed in Section 6.1.2. The plant had a processing throughput of 5 tons/hour. The 600 cubic meters of sediments were processed over a combined operating period of 11 days.

The washed sand was transported to the prepared storage area through a conveyor, as were organic particles and oversize materials. The washed sand was later either used as the cover material for the organic particles and oversize materials, which were permanently placed in the CDF, or as fill for CDF maintenance. The fines, which were discharged with high water content, were collected in the settling enclosures to allow settling and filtration before permanent disposal in the CDF.

Results

The effectiveness of the Linatex sediment washing process was monitored through sampling the output streams. Table 6.1 presents the results of fines removal and contaminant reduction in the separated sand after the process compared with the untreated material.

The results show a high degree of contaminant removal from the sand. Note that the capacity of the hydrocyclones used in the pilot project permitted separation at approximately 0.045 mm, which resulted in a less than desired removal of fines from the sand. This can be remedied by increasing the hydrocyclone capacities so that the separation occurs at around 0.075 mm. With more fines being removed, the contaminant levels in the sand can be expected to decrease further.

Table 6.1 Percent Reduction of Fines and Contaminants

Material	Percent Removal (%)
Fines (<0.075 mm)	77
Total Organic Carbon (TOC)	79
Cadmium	88
Chromium	55
Copper	65
Mercury	82
Nickel	71
Lead	61
Zinc	82
PCB	82

Figure 6.7 shows the reduction of contaminant concentrations in sand and the increase of contamination levels in fines and organic particles as compared with those in dredged sediments before treatment. The enrichment factor is the ratio of the concentration in a specific grain fraction to that in the untreated composite sediments. It should be noted that the apparent enrichment of contaminants in the output fines and organic particles only suggests that the more contaminated particles were concentrated together through the process; it does not imply increased contamination levels in the fines or organic particles.

The results indicate that the Linatex sediment washing process is an effective technology in extracting the less contaminated sand fraction from a contaminated dredged material, whereby reducing the volume of sediments required for regulated disposal, and increasing the level of sand resource recovery for beneficial use.

It is noted, however, that the goals of contaminated dredged material treatment using physical separation can be economically achieved only if the dredged material is predominantly sandy. As the content of fines in a dredged material increases, the obtainable volume of clean sand decreases, and the volume of the separated contaminated fines that requires regulated disposal increases. The physical separation technology is, therefore, not suitable for treating a predominantly silty dredged material.

6.1.6 Cost

The cost of a contaminated dredged material treatment project using physical separation technology as represented by the Linatex process was estimated based on the data from the Saginaw Bay pilot project as shown in Table 6.2.

Table 6.2 Cost Breakdown of Physical Separation

Item	Unit Cost ($/cubic meter)
Site Preparation	
Work Platform Construction	0.30
Storage/Office Area Construction	5.00
Total Site Preparation	5.30
Treatment	
Plant Mobilization/Demobilization	2.00
Process Equipment Rental	7.00
Generator Rental/Operation	1.00
Supplies & Maintenance	6.00
Material Handling	4.00
Operator Labor	3.00
Total Treatment	23.00
Process Monitoring (Sampling)	2.00
PROCESS TOTAL:	**30.30**

These estimates are expected to reflect the general cost levels associated with the various components of the physical separation technology when applied to the treatment of contaminated dredged materials. Cost items that are entirely specific to the Saginaw Bay pilot project were excluded from the estimates to allow for site and project variability. These include the costs of

- dredging and barging,
- residual (fines/organic particles) treatment/disposal, and
- real estate fees.

6.1.7 Marina Del Rey Application

Potential application of the physical separation technology in the management of the contaminated dredged material from Marina del Rey is discussed in this section in terms of application plan, logistics and costs. For purposes of this study, potential treatment site, transportation routes and disposal means/destinations were tentatively selected to provide a basis for planning and costing. Further examinations are needed for developing a final plan.

Plan

The application plan examined under this study consists of the following components:

- Construct a treatment site near the Los Angeles County maintenance facility area along the southern portion of Dockweiler Beach approximately 1 mile south of Marina del Rey.
- Install a land-based separation plant on the treatment site.
- Dredge (clamshell/hydraulic) the Marina del Rey sediments, transport (barge/pump) the material to the treatment site, and stockpile the material in feed storage area.
- Treat the stockpiled dredged material by separation.
- Stockpile clean sand on beach for beneficial use by the Los Angeles County.
- Transfer contaminated fines and organics to regulated landfills for disposal.
- Treat process water and discharge it back to ocean.

The treatment site consists of storage areas for untreated dredged material, clean sand, and contaminated fines and organics, plus the treatment plant. A conceptual site plan is shown in

Figure 6.8. The southern portion of Dockweiler Beach area to the general west of the Los Angeles International Airport was selected for siting consideration based on its relatively low recreational use and distance from major residential areas. Locating the treatment site in this generally less developed area provides the opportunity for minimizing the exposure of the contaminated process residuals (fines/organics/ oversize materials) during their overland transit to the landfills.

The aerial extent required for feed storage of 230,000 cubic meters of dredged material was estimated at approximately 12 acres (46,000 square meters) assuming a storage depth of 5 meters and a dike freeboard of 1 meter. Depending on the specific location of the site, this would correspond to a track of 460 meters by 100 meters parallel to the beach. The freeboard allows for excess standing water and bulking of the material in the final stages of filling.

A contiguous track of approximately half the size of the feed storage area will be adequate for the treatment plant and the post-treatment contaminated residual storage assuming a predominantly sandy dredged material and off-site beach stockpiling of the treated clean sand.

Since the separation process generally takes months to complete for the volume of dredged material from Marina del Rey, the storage areas for the untreated material essentially function as temporary CDFs. Containment dikes/berms and overflow weirs are generally required to contain and drain (especially in the case of hydraulic placement) the stored material. Geotextile lining is also needed generally to provide protection against adverse impact on the surrounding soil and groundwater. A water treatment unit may be required at the overflow/discharge outlet to treat the effluent before discharge into the ocean. The need for a water treatment unit, however, depends on testing results on the quality of the effluent from the untreated material storage area, as well as those of the process water from the treatment plant, as discussed later.

Either clamshell dredging/barging or hydraulic dredging/pipeline pumping method can be used. The advantage of the clamshell/barging method is that the dredged material generally becomes dry enough for mechanical handling using, e.g., bulldozers relatively soon after placement. A docking, unloading, and conveying facility, however, will be required to transfer the dredged material from the moored barge to the on-shore storage area. The hydraulic pipeline method, on the other hand, eliminates the need for material handling and transfer facility and operation, whereby reducing the potential for spillage of contaminated sediments in transit. A generally longer dewatering process is expected. The required level of dewatering, however, depends on the type of loading equipment to be used for feed into the treatment plant. An excavator/loader suitable for handling relatively wet material can reduce the lead time to the start of the treatment process. Conventional earth-moving equipment can be used in the later part of the treatment process as the remaining stored material becomes sufficiently dry to redistribute the material to the conveyor hopper area and to perform loading. Dewatering techniques such as trenching in the stored material can be employed to accelerate the drying process.

The treated clean sand can be transferred to the beach through a conveyor for stockpiling for future beneficial use. The contaminated residuals can be trucked to class I/II landfills in California or neighboring states. Candidate landfills in California include

- Kettleman Hills Landfill (Waste Management Co.), Kettleman City,
- Buttonwillow Landfill (Laidlaw Environmental), Buttonwillow, and
- Westmoreland Landfill (Laidlaw Environmental), Westmoreland.

Since the contaminated residuals generally requires adequate dewatering before becoming suitable for trucking, overland hauling and landfill disposal typically will not concur with the treatment process. The start of trucking/landfill disposal depends on the dewatering capacities of the residual storage basin. Assuming that the residuals consist of 30% of the 230,000 cubic meter dredged material (i.e. 69,000 cubic meters of contaminated residuals), and a truck hauling capacity of 13 cubic meters/load, approximately 5,300 sorties will be needed to transport the residuals to a selected landfill. To complete the trucking/disposal within 9 weeks after the residuals are adequately dewatered, an approximate frequency of 6 sorties/hour is needed based on a 16-hour day, 6-day week work schedule. More frequent sorties may be scheduled to shorten the project length. Potential transportation impacts should be assessed to determine the sortie frequency that balances the project needs and the environmental effects.

The process water in the treatment plant can be reused in the system until the project is completed. The need for treating the process water before discharge into the ocean can be determined based on testing results for the water samples. A water treatment unit can be installed as a part of the feed storage facility or as a mobile unit to for process water treatment when required. The same unit can be used for treating the effluent from the feed storage basin during earlier stages of the project if such treatment is required based on the testing results for the effluent water samples.

Cost

The costs for the application plan were estimated based on a total dredging volume of 230,000 cubic meters and a hydraulic dredging/pipeline placement scheme, as shown in Table 6.3.

Table 6.3 Physical Separation Cost Estimate (Based on 230,000 m^3 Dredged Volume)

Item	Unit Cost ($/$m^3$ in-situ)	Extended Cost ($ million)
Site Construction		
Feed Storage with Water Treatment[1]	19.00	
Residual Storage & Work Platform[2]	6.00	
Total Site Construction	25.00	5.75
Plant Purchase[3]	1.30	0.30
Hydraulic Dredging & Pipeline Placement[4]	4.60	1.06
Treatment[5]		
Generator Rental/Operation	1.00	
Supplies & Maintenance	6.00	
Material Handling	4.00	
Operator Labor	3.00	
Sampling/Monitoring	2.00	
Total Treatment	16.00	3.68
Residual Disposal		
Loading[6]	0.50	
Truck & Liner[7]	15.00	
Landfill Disposal[8]	21.00	
Total Residual Disposal	36.50	8.40
PLAN TOTAL	**83.43**	**19.19**
ANNUALIZED PLAN TOTAL[9]		**5.00**

1. Assumed a storage area of approximately 460m long, 100m wide and 3m deep with a 4m high sand dike. Dredged material depth = 5m. Free board 2m. Excavation cost $5/$m^3$. Water treatment unit includes coagulation/settling, filtration/chlorination, and carbon adsorption (Walski & Schaefer, 1988).

2. Saginaw Bay estimates for full-scale project assumed applicable based on similar facility requirements (EPA,1994c).

3. Approximately \$300,000/plant including auxiliary equipment for a 200 tons/hr system (125 m^3/hr at 1.6 ton/m^3) (Ross Spears, Linatex, person. comm.).

4. Based on a 16"-26" pipeline and a 16-hr work day (Manson and Great Lakes, person. comm.).

5. Saginaw Bay full-scale project estimate assumed applicable (EPA, 1994c).

6. Assumed 30% dredged material being fines/organics/oversize materials (69,000 m^3), trucked out in 9 weeks (6-day week and 16-hr day). Truck capacity 13 m^3/load. Operating cost \$125/hr (CH2MHILL, 1997).

7. Assumed truck/liner cost of \$640/Sortie (CH2MHILL, 1997).

8. Assumed a disposal cost of \$70/m^3 (\$44/ton and a density of 1.6 ton/m^3) including taxes (CH2MHILL, 1997).

9. Based on a rate of 9% over a dredging interval of 5 years.

6.2 Stabilization/Solidification

Stabilization/solidification (S/S) with Portland Cement as an alternative for treating contaminated dredged materials is reviewed and analyzed in this section. The purpose, applicability, and process characteristics of the alternative are discussed first, followed by a review of the ECDC/ITEX DSRR process for chemical stabilization/ solidification of contaminated dredged materials. The logistics and costs of the process are examined to provide a preliminary basis for potential local application.

6.2.1 Purpose

Most dredged materials from navigation channels are of alluvial and/or coastal littoral origins. They are part of the sediment resources in the natural environment that are available for beneficial uses when regulatory criteria are met. Most often, the levels of contamination in dredged materials from maritime harbors and channels are not severe enough for the materials to be considered hazardous, which provides the opportunity for their beneficial use after appropriate treatment at a manageable cost. Stabilization/ solidification of contaminated dredged materials using Portland cement as stabilization additive is one application of a more established stabilization/solidification technology that has been applied extensively in treating industrial waste solids for construction end uses (Goumans et al., 1991; Goumans el al., 1994; EPA, 1994a; EPA, 1994b).

6.2.2 Applicability

As a relatively mature process in industrial waste recycling for construction uses, the stabilization/solidification technology immobilizes the contaminants in a dredged material in the cement matrix and creates at the same time an engineered material with adequate strength for use in construction projects. The applicability of this technology as a means for recycling the dredged materials is, therefore, determined by the achievable levels of,

- contaminant stabilization, in terms of constituent solubility/leachability in the end product with a given pH value, and
- physical performance, in terms of unconfined compressive strength (UCS), compaction characteristics, and permeability, among other parameters.

The available results of treatability tests have suggested that stabilization/solidification with Portland cement additive is an effective technology in treating the contaminated dredged materials from navigation maintenance projects for construction uses (ITEX, 1997).

6.2.3 General Process

The general process of stabilization/solidification of a dredged material with Portland cement additive consists of four core phases:

- dewatering by pumping to remove standing water in the bulk dredged material;
- raking the material to remove oversize debris;
- blending the material with a pre-formulated, Portland-cement-based additive slurry mix;
- curing to develop desired compaction characteristics, permeability, and bearing strength.

The standing water removed from the dredged material bulk container (i.e. a scow) can be reused in the plant as pumping media for creating cement additive slurry. The fines recovered (most likely contaminated) from the pumped-out standing water can be placed back in the bulk material container. After screening the material for oversize debris, the bulk material is mixed with a Portland cement additive. The percentage of the Portland cement additive is approximately 5%-

15% of the end aggregate to achieve desired stabilization and strength, which typically develops after 7 days of curing according to available studies.

The physico-chemical processes after the introduction of the Portland cement additive enable

- encapsulation/fixation of the constituents of concern in the cement matrix, whereby immobilizing the contaminants and producing a physically and chemically stable material, and

- hydration of di- and tri-calsium silicates (Ca_2S; Ca_3S) yielding calcium hydroxide ($Ca[OH]_2$), whereby enhancing the strength of the treated material.

The calcium hydroxide neutralizes the acidity in the material, and the hydration process typically yields a pH value in the range of 11-12. The metal ions of concern are precipitated on the clay particles (on the order of 2-10 microns) in the form of sulfides or hydroxides that are generally insoluble under favorable pH values. The leachability of sensitive constituents such as chlorides from the cured material in monolithic form is generally low based on data available.

6.2.4 Application: ECDC / ITEX Process

The ECDC/ITEX Dredge Sediment Recovery and Recycle (DSRR) process centers around the core phases of cement-based stabilization/solidification process discussed above. The process consists of the following stages (ECDC, 1997):

- decant standing water in the barge carrying the dredged material from the dredging site by pumping to a holding tank on shore;

- rake the dredged material in barge to remove oversize debris to roll-off boxes to be transferred for regulated disposal;

- transfer pre-formulated cement-based additive to a mixer on shore and reuse the decanted water (with fines) in the holding tank as pumping media to create an additive slurry;

- introduce the additive slurry mix into the dredged material in barge through a mixing head mounted on the end of the stick of an Caterpillar excavator and thoroughly blend with the dredged material;

- allow the treated dredged material to cure while barging it to off-loading location near the end-use site, unloading to haul trucks, and transferring to a receiving pad;

- monitor further curing of the treated material at the receiving pad for planned placement at the end-use site; and
- place the treated material at the end-use site for optimal compaction and permeability.

A process diagram is shown in Figure 6.9. Preliminary data from process testing applications indicate that the stabilized material possesses

- enhanced bearing strength,
- low moisture content,
- low permeability, and
- reduced contamination potential with constituents in least soluble, mobile or toxic forms.

The enhanced bearing strength and low permeability enable applications of the stabilized dredged material as, respectively, construction fill material and upland capping/landfill cover material, among other potential applications.

Applicable Constituents

The ECDC/ITEX process has demonstrated its effectiveness with metals in the dredged material. The stabilization additive compositions can be tailored to the need of treating specific metal constituents of concern in the dredged material. Through additive design and treatability testing, the optimal end-product pH values can be achieved so that the target metal ions are precipitated in their least soluble, mobile or toxic forms.

The process has also been shown to be effective in reducing the leachability of chlorides (ITEX, 1997). Since leaching of chlorides has been one of the major concerns with resource agencies regarding upland beneficial uses of the dredged material, the ECDC/ITEX process improves the prospect of beneficial uses of stabilized dredged materials in regions sensitive to chlorides.

It has been noted, however, the effectiveness of the ECDC/ITEX process in treating hydrocarbons remains uncertain (ECDC, pers. comm.). Since hydrocarbons are common constituents of concern in contaminated dredged materials from harbors and navigation channels, more data are necessary to ascertain limitations on applicability of this process to dredged materials with high levels of hydrocarbon contamination.

Throughput Capacity

Up to approximately 9,000 cubic meters per day.

Maturity

The ECDC/ITEX process has been applied to treating dredged materials from Ports of New York/New Jersey harbor areas at ECDC/ITEX Dredge Sediments Recovery & Recycling Facility (DSRR) at Port Newark Channel, Newark, New Jersey. The process was also demonstrated at Port of Richmond for stabilizing dredged materials. A patent is currently pending for the process.

Applicability Evaluation

The ECDC/ITEX process has been applied to dredging projects in Ports of New York/New Jersey and has demonstrated its effectiveness in handling large quantities of contaminated dredged materials.

For application in Ports of Los Angeles/Long Beach, a treatability study on sediments from Port of Los Angeles was conducted (ITEX, 1997). The study showed general effectiveness of achieving desired levels of unconfined compressive strength (UCS), pH values, and sodium-chloride fixation (low leachability). The fixation of various contaminants of concern other than chlorides, however, was not covered by the study.

In view of the general sensitivity of the stabilization process to the specific composition of the contaminant constituents in the dredged materials to be treated, it is necessary that a detailed treatability study be performed for the Marina del Rey sediments to provide a basis for determining the applicability of the process. Special attention should be given to the effectiveness of the process in stabilizing hydrocarbons and any other contaminants in the Marina del Rey sediments for which treatability data have not been established through the past experience of the process.

In addition to treatability issues, land/dock availability for cement additive stockpiling and process plant stationing as well as permitting and logistic issues have to be examined to determine the overall applicability of the process for the project.

Cost

The cost of a contaminated dredged material treatment project using cement-based stabilization/solidification technology as represented by the ECDC/ITEX process was estimated based on ECDC project experiences (ECDC, person. comm.) as shown in Table 6.4.

Table 6.4 Cost Breakdown of Cement-Based Stabilization/Solidification

Item	Unit Cost ($/cubic meter)
Sediment Treatability Testing	0.70
In-Barge Debris Removal/Dewatering/Dockside Stabilization	46.00
Transfer to Host Placement Site within 5 Miles and Placement as Fill in Compacted Lifts	12.00
Mobilization/Demobilization (150,000-300,000 cu.m. Project)	2.00
PROCESS TOTAL:	**60.70**

Costs that are not included in the above that are highly project-dependent include (ECDC, pers. comm.):

- dockage/wharfage fees, additional longshore labor charges,
- debris disposal,
- host fees for material placement by landowner, and
- wharfside material-handling permits from state, federal and local regulatory agencies.

Note that, if the treated material is designated for use as capping material for an aquatic capping project instead of upland construction fill, the cost of treated material transfer and placement ($12.00/cubic meter) should be replaced by a barging/aquatic placement of approximately $7.00/cubic meter. The corresponding total unit cost is then approximately $56.00/cubic meter.

The overall cost of the ECDC/ITEX process for its NY/NJ project experience is approximately $59-64 per cubic meter, which includes a host fee of approximately $10 per cubic meter which was charged for accepting the treated dredged material as fill at the beneficial use site.

It is noted that the host-fee fraction of the cost varies with projects depending on the region of application, the end use of the treated material, and, in general, the availability of opportunities such as concurring construction projects where large quantities of fill materials are needed. For public civil projects under the authorities of parties that have stakes in harbor and navigation channel dredging and dredged material management, beneficial use of the treated material without charge can be facilitated through cooperation. Noting that the prevailing price of fill is around $5 per cubic meter, opportunity exists for the treated material to be purchased by interested parties so that the process cost for treating the dredged material can be partially recovered.

It should also be noted that the process cost based on ECDC NY/NJ project experience can not be directly translated into the expected cost for the application of the process to a local project because of the expected differences in areas such as end uses and the associated logistics. Since many cost components such as mobilization/demobilization and material transfer costs are region-, and application-sensitive, the potential overall process cost must be estimated project-specifically.

6.2.5 Marina del Rey Application

Potential application of the cement-based stabilization/solidification technology in the management of the contaminated dredged material from Marina del Rey is discussed in this section in terms of application plan, logistics an costs. For purposes of this study, potential treatment site, transportation routes and disposal means/destinations were tentatively selected to provide a basis for planning and costing. Further examinations are needed for developing a final plan.

Treatment Site

Treatment sites potentially available in the Ports of Los Angeles and Long Beach area include the former Kaiser terminal (berths 48-53) in Port of Los Angeles, and Pier F (berths 205-207) and Pier G (berths 212-214), both in Port of Long Beach (CH2M Hill, 1999). Since the Kaiser terminal was determined to be unsuitable for sediment rehandling (COELAD, pers. comm.), Piers F and G at Long Beach were considered as potentially viable sites in this study for plan development.

Plan

The application plan examined under this study consists of the following components:

- Prepare a treatment site near the dockside of Pier F (berths 205 to 207) or Pier G (berths 212 to 214) (See Figure 6.10), including additive mix storage/handling area, debris holding/ handling area, and areas for process water holding vessel and material conveying/handling equipment.

- Dredge (clamshell) the Marina del Rey sediments, and transport (barge) the material to the treatment site.

- Treat the dredged material in barge with cement additive and through in-barge curing.

- Barge the treated material to Port of Long Beach development site and place the material as land fill.

The Pier F and Pier G sites were selected for their access to land transport (rail) facilities, which provides flexibility in cases where the treated material is to be placed at a relatively distant upland site. The proximity of these sites to the Pier S landfill site can provide the added benefits of cost savings if the material is used in the landfill project.

The Marina del Rey material will be dredged using a clamshell dredge. The relatively dry dredged material dewaters further during transit to the treatment site in Port of Los Angeles.

With two mixers treating each barge load of dredged material, a daily treatment volume of approximately 9,000 cubic meters can be achieved. The project volume of 230,000 cubic meters of dredged material can thus be treated and placed at the landfill in approximately a month.

Cost

The costs for the application plan were estimated based on a total dredging volume of 230,000 cubic meters and a clamshell dredging/placement scheme, as shown in Table 6.5.

Table 6.5 Stabilization / Solidification Cost Estimates (Based on 230,000 m^3 Dredged Volume)

Item	Unit Cost ($/$m^3$ in-situ)	Extended Cost ($ million)
Treatability Testing[1]	0.70	0.16
Clamshell Dredging/Barging to Treatment Site in POLA (Piers F/G Area)	10.00	2.30
Treatment[1]	46.00	10.58
Barging/Placement of Treated Material at POLB Landfill Site (Pier S)[1]	12.00	2.76
Residual Disposal		
Loading[2]	0.20	
Truck & Liner[3]	1.00	
Landfill Disposal[4]	1.80	
Total Disposal	3.00	0.69
PLAN TOTAL	**71.70**	**16.49**
ANNUALIZED PLAN TOTAL[5]		**4.29**

1. ECDC, 1998, pers. comm.

2. Assumed 2% dredged material being debris/oversize materials (4,600 m^3), trucked out in 26 days (16-hr days). Truck capacity 13 m^3/load. Operating cost $125/hr (CH2M HILL, 1997).

3. Assumed truck/liner cost of $640/Sortie (CH2M HILL, 1997).

4. Assumed a disposal cost of $88/$m^3$ ($44/ton and a density of 2.0 ton/m^3) including taxes (CH2M HILL, 1997).

5. Based on a rate of 9% over a dredging interval of 5 years.

6.3 Physical Mixing

Physical mixing as an alternative for treating contaminated dredged materials is discussed section. The purpose, applicability, implementation issues and a conceptual application plan are examined.

6.3.1 Purpose

The primary purpose of physically mixing a relatively silty dredged material with a clean sandy material is to produce a suitable aggregate for use as a construction fill. Significantly enhanced engineering properties can be achieved through proper design and execution of the process.

6.3.2 Applicability

The physical mixing method can be a viable option for treating a dredged material with a relatively content of fines. For a mildly contaminated dredged material, this method also results in a lower aggregate contamination level due to the addition of a significant amount of clean sand. The operational feasibility of the method, however, is project specific. Primary factors affecting its implementability include

- the availability of large quantities of clean sandy material, and
- the achievable level of mixing of the dredged material and the clean sand.

The sand resources must be economically available and the mixing scheme economically feasible at the project site for the method to be applicable.

6.3.3 Implementation Issues

The primary issue associated with the physical mixing of contaminated dredged material is the ability to engineer a structural-grade fill material from a dredged material with a relatively high content of fines.

The engineering properties of a dredged material depend on its grain size distributions among other factors. A material with relatively high percentage (e.g.> 80%) of sand is normally required to provide adequate structural strength to the fill. In cases where the dredged material contains a relatively high content of fines, the need for creating an adequately sandy fill from the dredged material may require large quantities of sand. In general, the sand content of the clean material should be higher than required for the fill if the dredged material has an acceptably high fines content in order to create a grade of material satisfying the strength requirement of the fill.

Experience at the Port of Long Beach indicated that a 10 to 1 ratio of clean sandy material to the dredged fine material was required to render the product material structurally usable as a fill (Port of Long Beach, pers. comm.), which is highly uneconomical. The cost-effectiveness of the method is therefore sensitive to the grain-size characteristics of the in-situ dredged material and the availability of sand sources.

In view of the highly material- and project-specific nature of the physical mixing method, pilot projects will be necessary to establish engineering and operational parameters such as mixture ratios and blending means/procedures for achieving the desired engineering properties.

6.4 Summary

Contaminated sediment treatment technologies that have been, or have the potential to be, applied to treating contaminated dredged materials were reviewed and analyzed. The study focused on three categories of technologies:

- physical separation,
- cement-based stabilization/solidification, and
- physical mixing.

The physical separation technology as represented by the Linatex process has been demonstrated to be effective in the treatment of large quantities of contaminated sediments at a relatively economical cost based on field pilot project data. The process reduces the volume of contaminated sediments required for restricted disposal and recovers clean sand fraction from the contaminated material for beneficial use. The process, however, becomes ineffective economically for a dredged material with a high silt/clay content.

In general, the typical unit cost of treating a dredged material using the physical separation technology is estimated to be approximately $30/cubic meter in addition to dredging, barging, and residual disposal costs. The unit cost for Marina del Rey application is approximately $84/cubic meter including site construction, plant purchase, dredging and barging, treatment, and disposal costs, based on a dredged volume of 230,000 cubic meters and the application plan as discussed in Section 6.1.4. The annualized cost is approximately $5 million/year on the same basis over the next five years. It is noted, however, the long-term cost is lower, reflecting the capitalization of the initial investment on the facility construction and plant purchase.

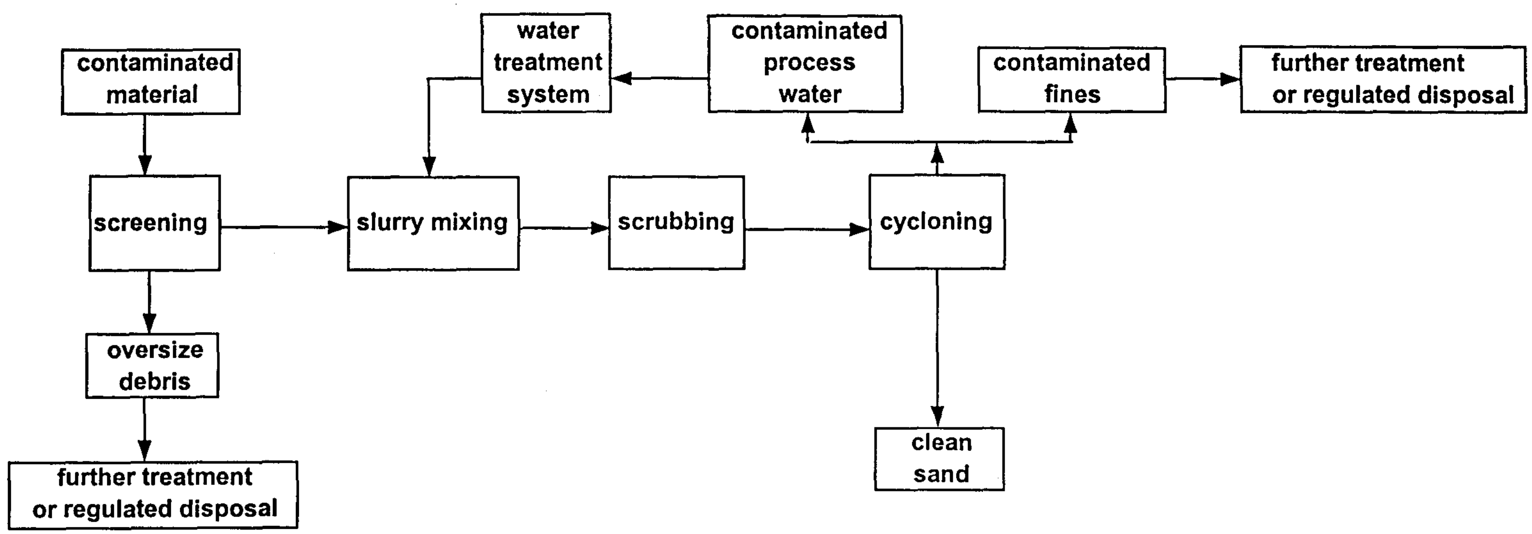
The cement-based, dredged material stabilization/solidification technology as represented by the ECDC/ITEX process has been demonstrated to be effective in treating contaminated dredged materials and producing a physically enhanced, environmentally acceptable fill material.

However, since the stabilization process is sensitive to the specific composition of the contaminant constituents in the dredged materials to be treated, it is necessary that a detailed treatability study be performed on a project-specific basis to determine the applicability of the process. Special attention should be given to the effectiveness of the process in stabilizing hydrocarbons and any other contaminants in sediments for which treatability data have not been established through the past experience of the process.

In general, the typical unit cost of treating a dredged material using the cement-based technology is estimated to be around \$60/cubic meter in addition to the dredging, barging, and residual disposal costs. The unit cost for Marina del Rey application is approximately \$72/cubic meter including dredging and barging, treatability testing, treatment, treated material barging and placement, and residual disposal costs, based on a dredged volume of 230,000 cubic meters and the application plan discussed in Section 6.2.4. The annualized cost is approximately \$4.3 million/year on the same basis over the next 5 years.

Physical dilution is a potentially viable method for rendering a clean or mildly contaminated dredged material with a relatively high content of fines usable as a construction fill. By blending the dredged material with a clean sandy material, a fill material with adequate structural strength can be engineered. The feasibility of this method, however, is highly dependent on whether a clean, highly sandy material can be obtained economically in large volumes (up to approximately 10 times the volume of the dredged material depending on the grain-size distributions of both materials). Even if the clean sand source is available, the capacity of the destination landfill to receive the resulting great volume of material presents one further constraint. In view of these issues, the feasibility of the physical mixing method for Marina del Rey application is less apparent compared with the physical separation and the cement-based stabilization/solidification methods.

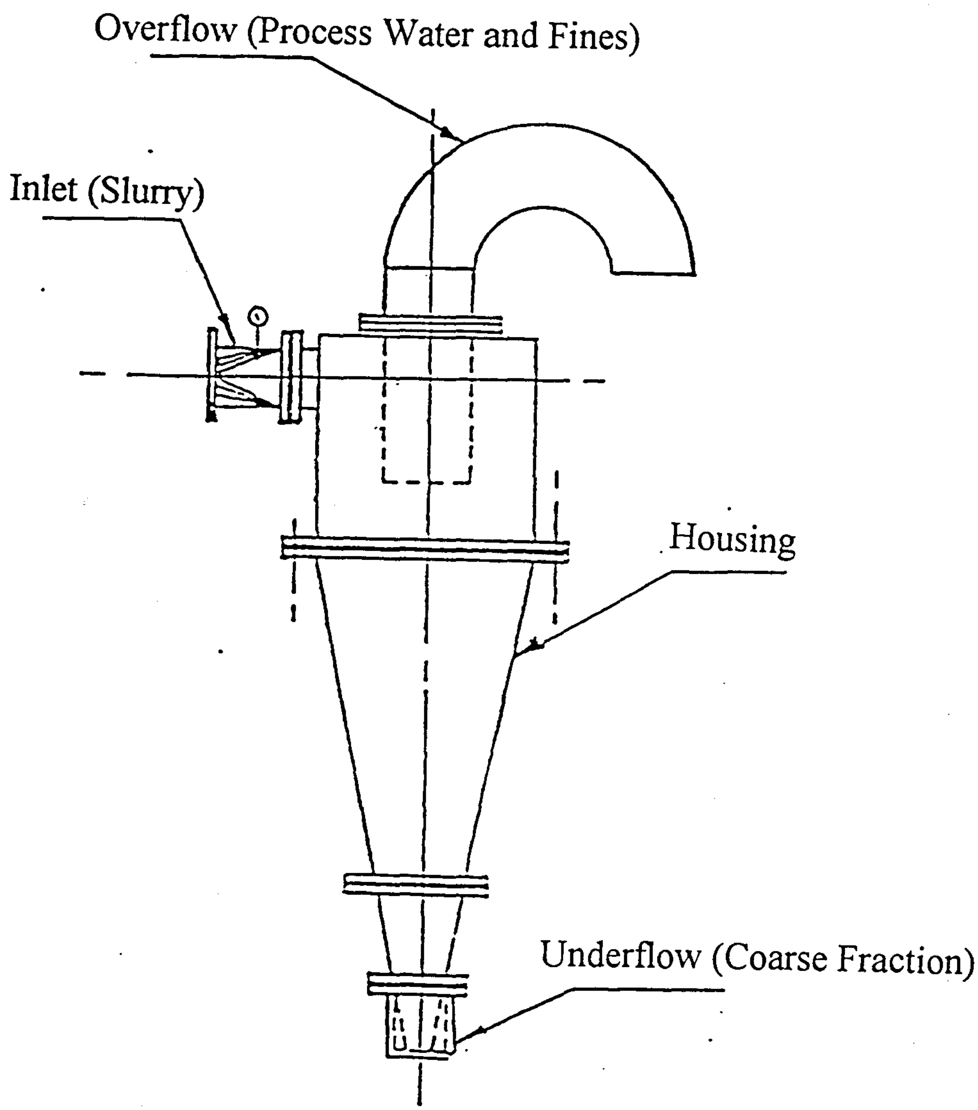
Based on the preliminary results of unit costs, the cement-based stabilization/solidification appears to be the preferred alternative of treatment for the Marina del Rey dredged material. The physical separation method, however, becomes competitive in the long term as the region starts to benefit from the initial capital investments on facility construction and plant purchase in the following decades. For a project life of 20 years with a dredging cycle of 10 years and an interest rate of 7.13%, for example, the annualized cost of physical separation was estimated to be approximately $4.5 million/year as compared to $7.3 million/year for cement-based stabilization and $39.7 million/year for physical mixing. The cost, however, is still expected to be significantly higher than that of aquatic capping at the North Energy Island Borrow Pit, which was estimated at around $1 million.



MARINA DEL REY SEDIMENT STUDY

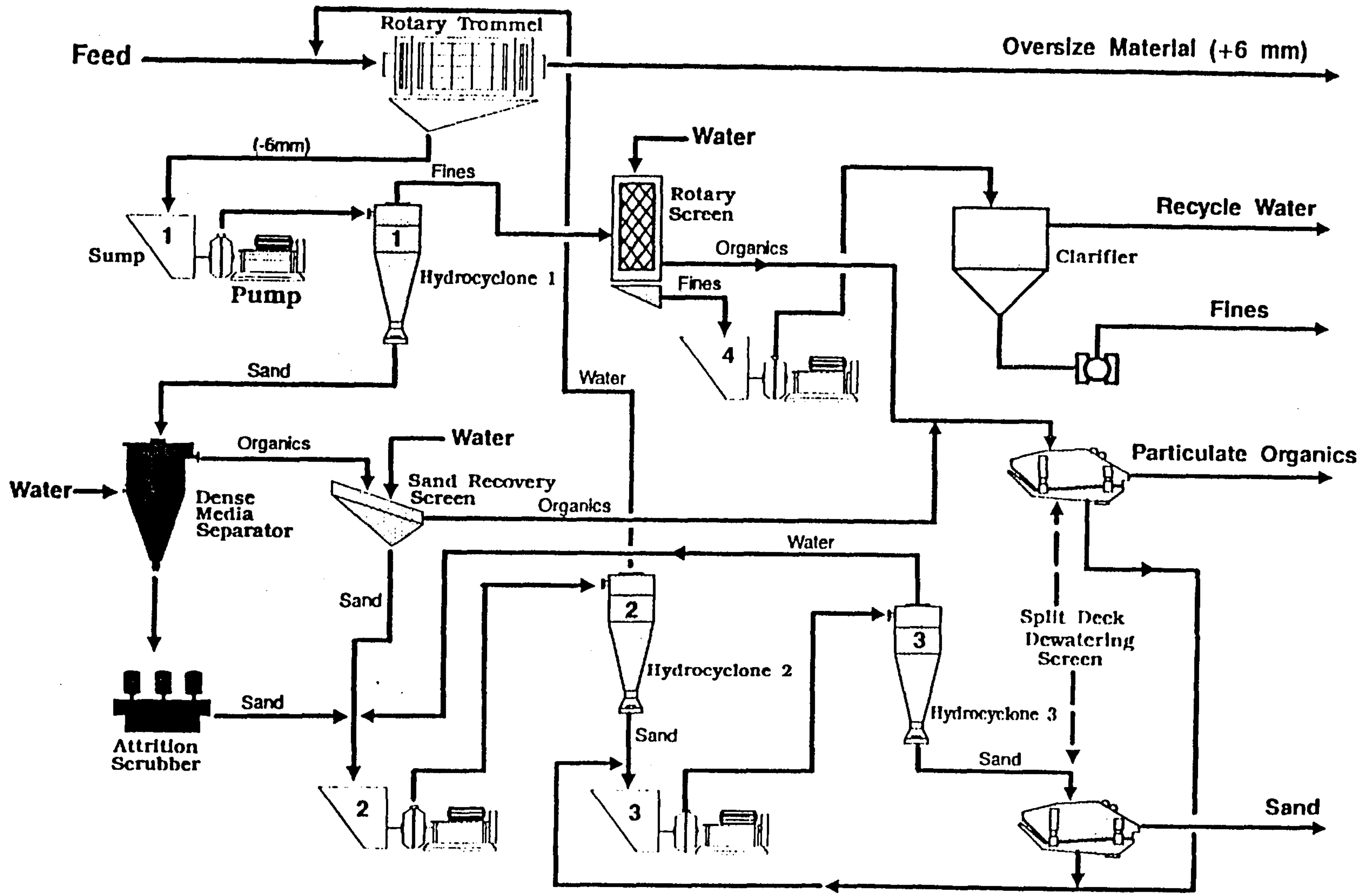
Physical Separation Process

Figure 6.1

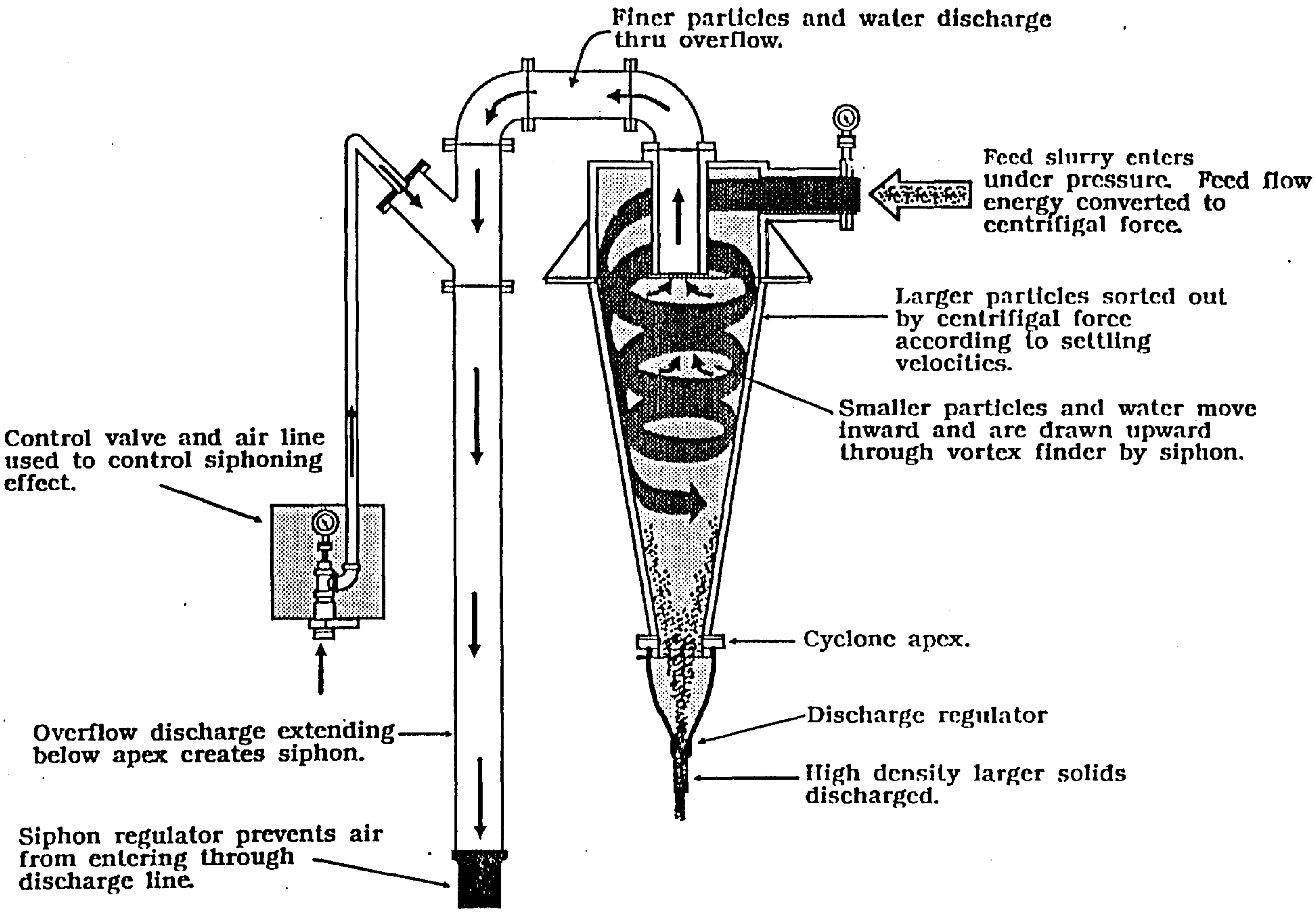


MET-PRO HYDROCYCLONE

Figure 6.2

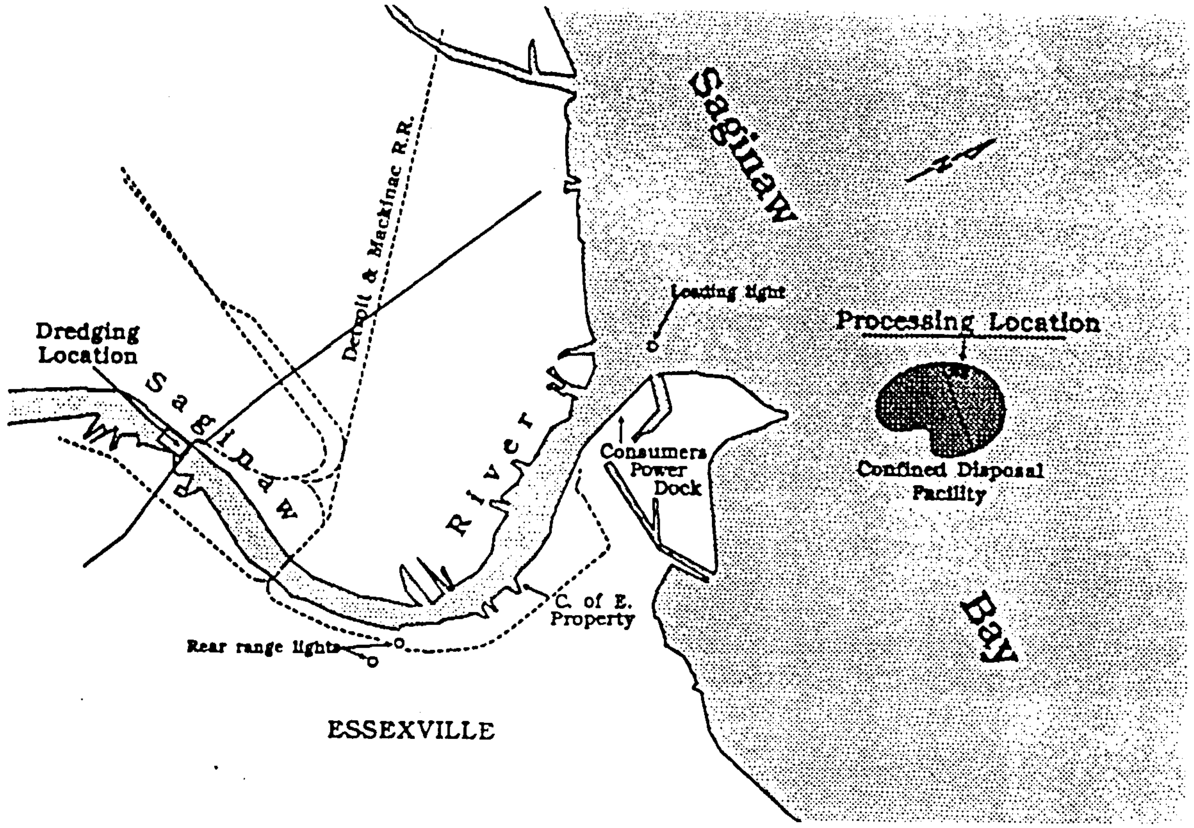


MARINA DEL REY SEDIMENT STUDY	Saginaw Bay Pilot Project: Linatex Process Configuration	Figure 6.3

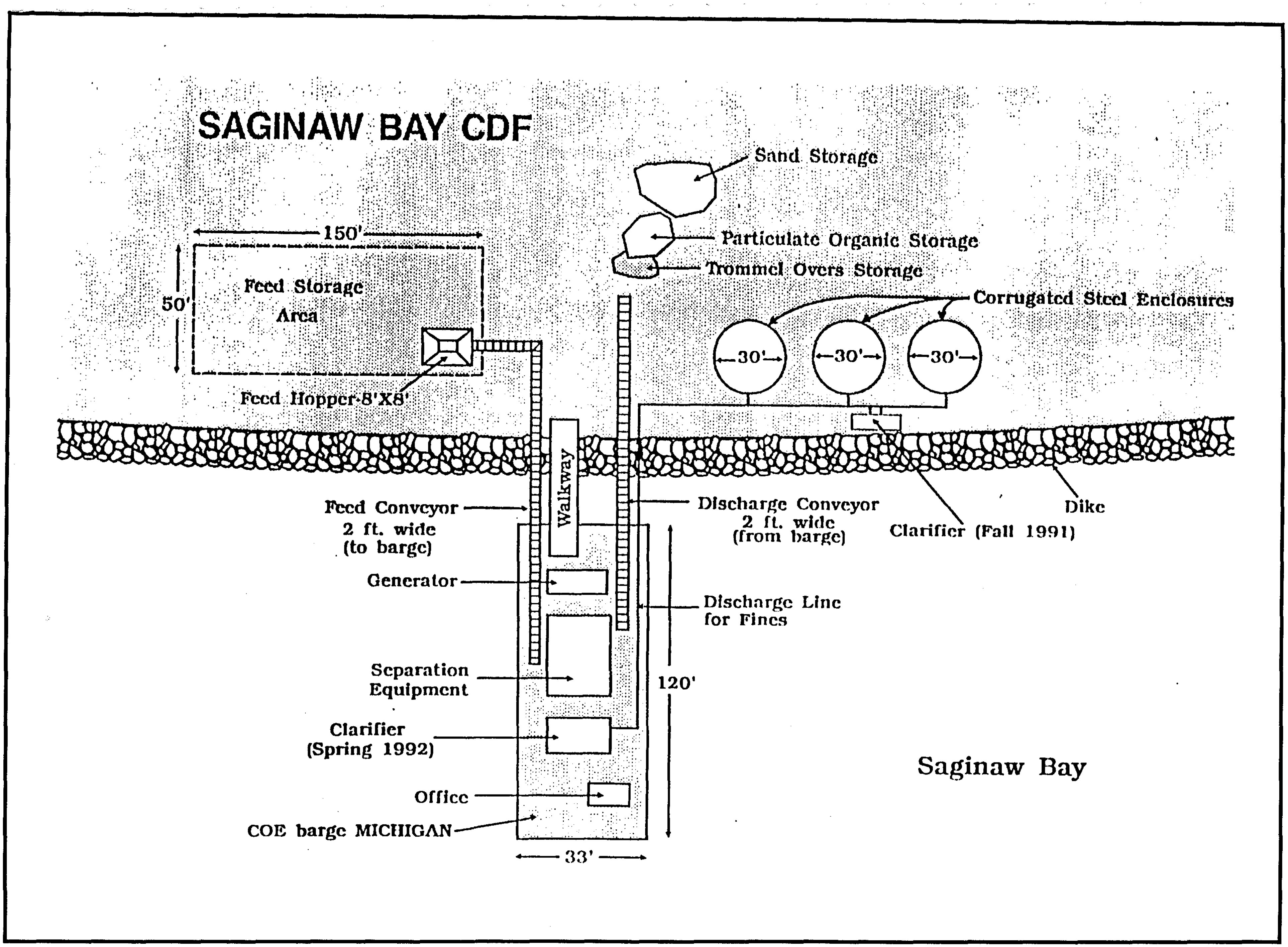


Linatex Hydrocyclone

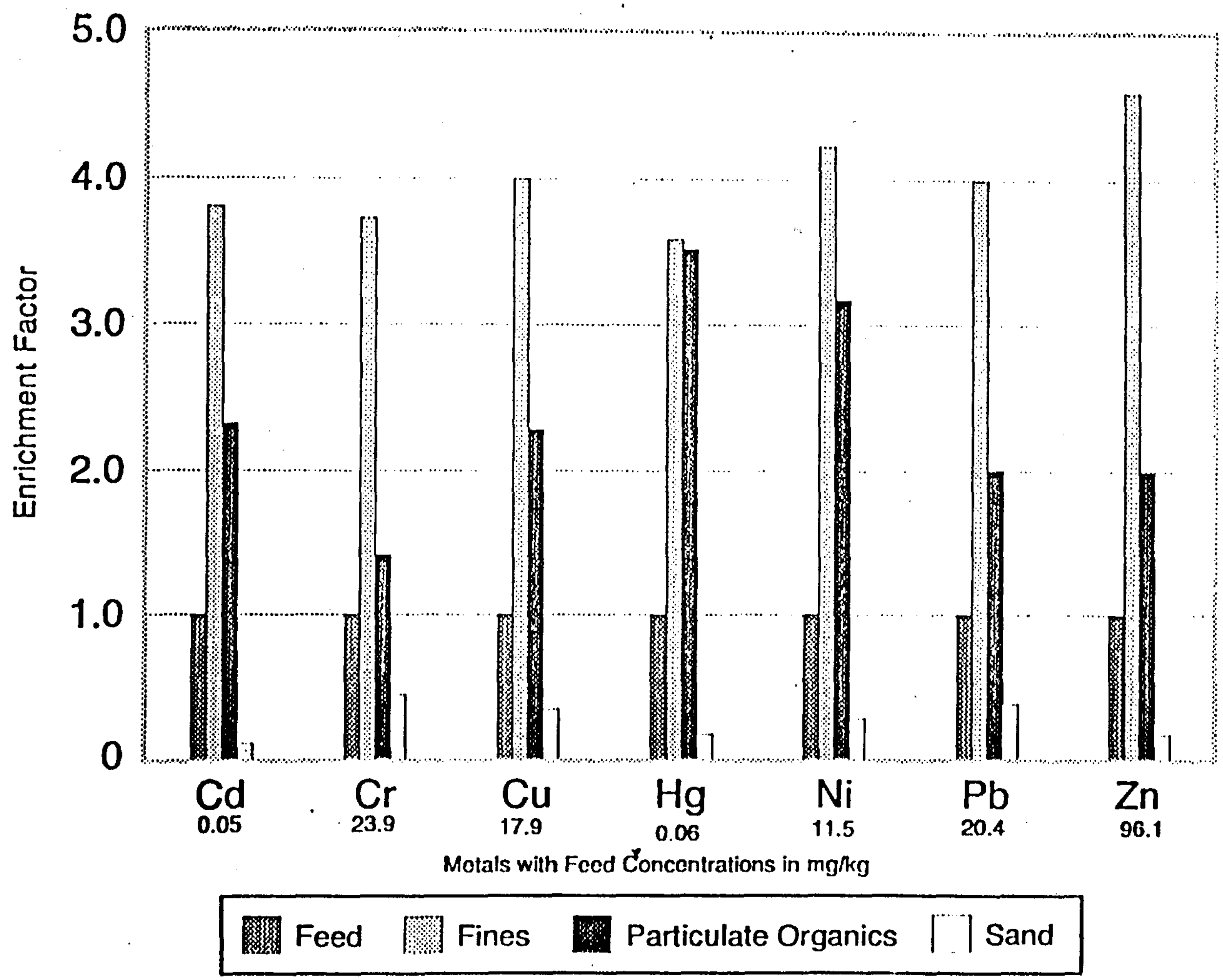
Figure 6.4



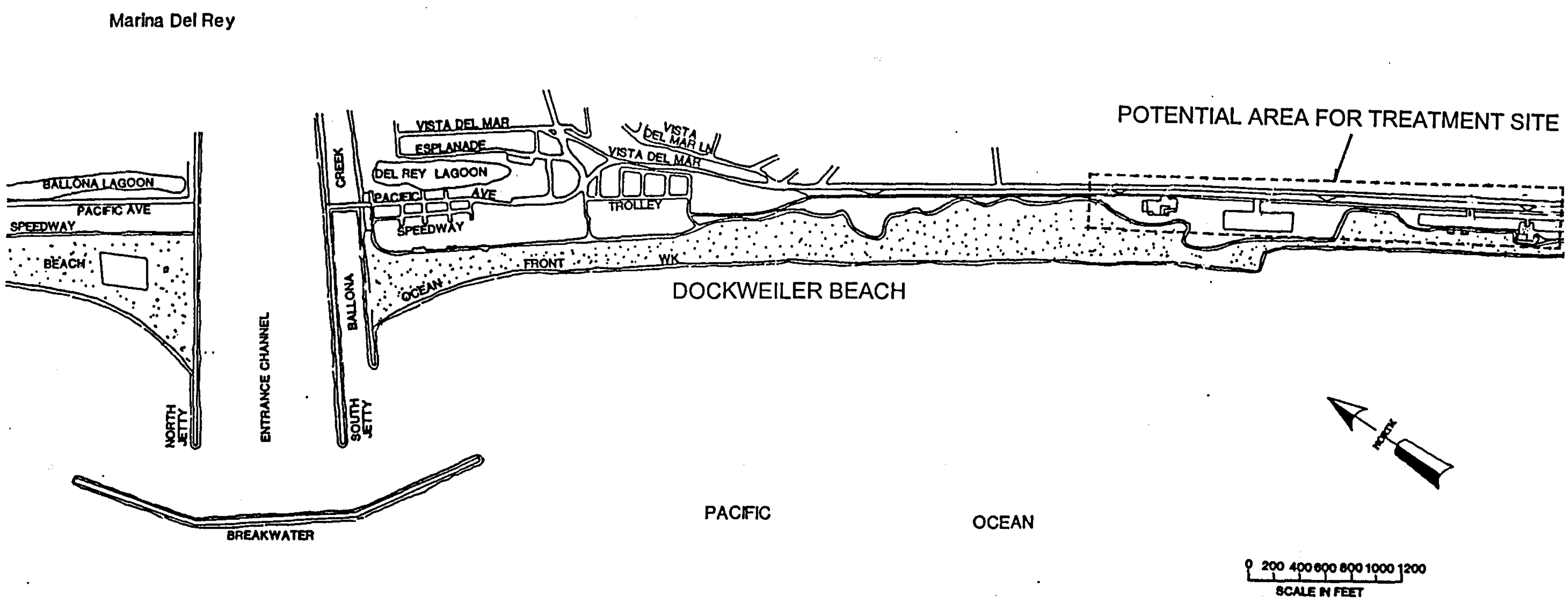
MARINA DEL REY SEDIMENT STUDY	Saginaw Bay Pilot Project: Site Map	Figure 6.5



MARINA DEL REY SEDIMENT STUDY	Saginaw Bay Pilot Project: Site Layout	Figure 6.6



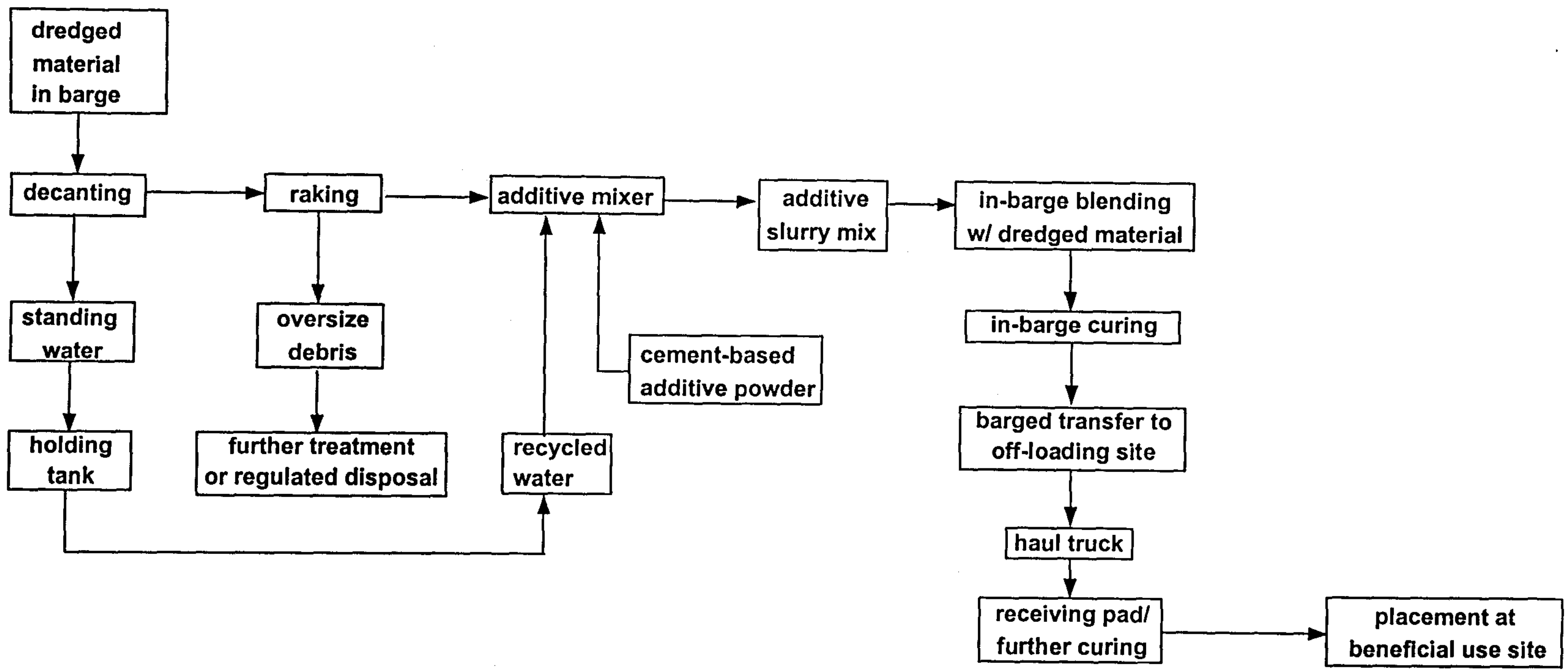
MARINA DEL REY SEDIMENT STUDY	Metal Concentrations: Treated Material Compared with Untreated Material	Figure 6.7



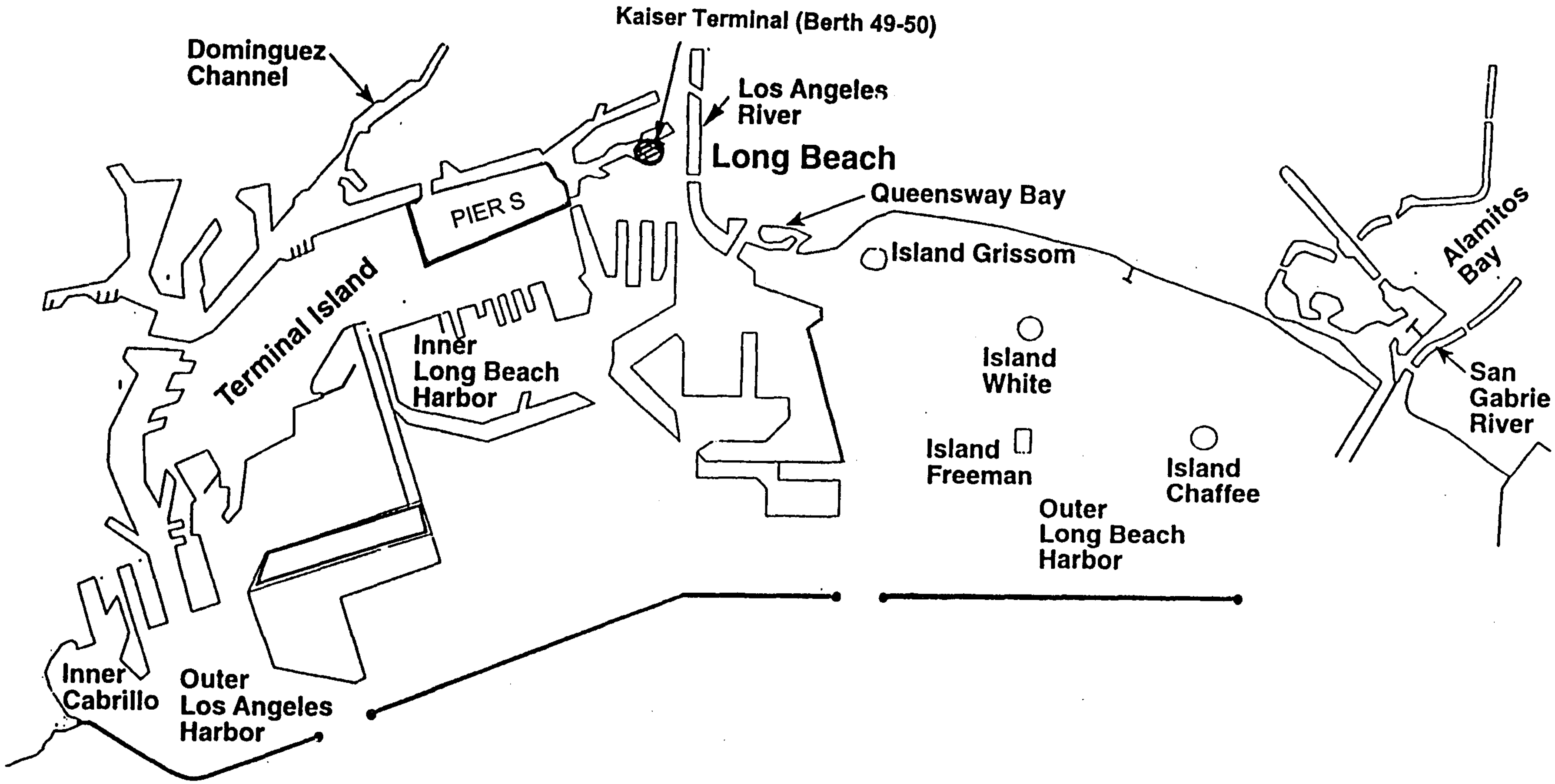
MARINA DEL REY SEDIMENT STUDY

Site Map:Physical Separation

Figure 6.8



MARINA DEL REY SEDIMENT STUDY	ECDC/ITEX DSRR Process	Figure 6.9



MARINA DEL REY SEDIMENT STUDY	Site Map: Cement-Based Treatment	Figure 6.10

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