Investigation of Stinson Beach Park Storm Damage and Evaluation of Alternative Shore Protection Measures

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G. Whelan**

July 1984

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SUMMARY

An investigation was made of storm damage during the winter of 1982-83 to the National Park Service's Stinson Beach Park. The investigation included an assessment of the storm damage, evaluation of physical processes contributing to the damage, subsequent beach recovery, and the feasibility of implementing shoreline protection measures to reduce future risk.

During the winter of 1982-83, the beach was almost completely denuded of sand, wave overwash damaged the foredune, vegetation on the foredune was destroyed, and backshore flooding occurred. Two structures and a parking lot were endangered as the shoreline receded. Subsequent recovery of the park beach was rapid. By January 1982 sand had moved back onshore and a beach berm was beginning to reform. The foredune and dune vegetation received the only permanent damage.

Based on an analysis of wave hindcast data, similar winter storms have a recurrence interval of once in 10 to 12 years. Deepwater wave heights can exceed 19 ft on the average of every 2 years and exceed 23 ft every 6 years. Wave refraction analysis indicates that breaking wave heights are from 8 to 76% lower than the corresponding deepwater waves. The highest breaking waves are from the south-southwest. The design breaking wave at Stinson Beach Park is 18.6 ft from the south-southwest with a period of 11 sec. The corresponding deepwater wave height is 21.7 ft.

Longshore sand transport at Stinson Beach Park, as determined by a wave refraction/longshore transport computer analysis, is small relative to other California coastal areas. The potential transport rate is about 310,000 cubic yards per year and is directed almost exclusively to the northwest. The longshore transport rate and direction indicate that shoreline protection measures being planned or implemented elsewhere along Stinson Beach will have little if any effect on the park beach.

Four shoreline protection alternatives were evaluated. These include no action, dune development/enhancement, construction of a rock riprap revetment, and offshore installation of artificial seaweed. The first costs (estimated

costs, excluding maintenance) range from about \$90,000 to \$475,000. The least-cost protection measure is riprap revetment, which protects the two structures and parking lot endangered during the 1982-83 winter storms. Construction of a foredune along the entire park beach is the highest cost protection measure. If no shore protection action measures are implemented, wave overwash of the foredune can be expected to occur on the average of every 2 to 3 years, and beach degradation, similar to that during the 1982-83 winter, can be expected to occur on the average of every 10 to 12 years.

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INTRODUCTION

Stinson Beach is a 2.7-mile-long sand spit that forms the barrier to Bolinas Lagoon (Fig. 1). The beach is located in Marin County, about 10 miles north of the entrance to San Francisco Bay. Seadrift, a high-density housing development, is situated on the outer 1 1/2 miles of the spit, and the Patios and Callis areas of the Stinson Beach community are located near the spit's base. Stinson Beach Park is located at the extreme southern end of the spit (base). The park is maintained by the U.S. Department of Interior's National Park Service as part of the Golden Gate National Recreation Area. The park encompasses about 0.6 miles of shoreline and has parking, picnic, and comfort station facilities.

Stinson Beach Park experienced considerable wave-induced erosion damage during the winter storms of 1982-83. Wave scour and dune overwash during these storms have threatened two parking lots, a comfort station, and a lifeguard tower. The U.S. Department of the Interior requested this report to assess the storm damage to Stinson Beach Park during the 1982-83 winter storms, evaluate the feasibility of shoreline protection measures, and evaluate the effects of structural protection measures at Seadrift and Stinson Beach Community on Stinson Beach Park.

This report includes the following: a description of Stinson Beach, a discussion of the longshore (littoral) processes, results of wave refraction/longshore transport analysis, an evaluation of the winter storm damage in 1982-83 and subsequent beach recovery, and an evaluation of alternative protection measures. The following alternative shoreline protection measures are evaluated: no action, dune development/enhancement, riprap revetment, and placement of artificial seaweed offshore to trap sand and dissipate wave energy.

PHYSICAL SETTING

Stinson Beach is a sand spit separating Bolinas Lagoon from the Pacific Ocean. The crescent-shaped spit has been formed between Duxbury Point to the northwest and Rocky Point to the south (see Fig. 1). The beach, which is fairly wide with gentle beach and nearshore slopes, is comprised of medium-to-coarse-grained sand with some gravel deposits. Beach deposits at Stinson Beach Park are from 8 to 10 ft in depth near the high-water line. The sand deposits overlie a highly cohesive clay material that is moderately resistant to the scour of waves. This underlying clay hardpan was exposed near the high-water shoreline along most of Stinson Beach Park during the winter of 1982-83.

Although Stinson Beach is situated on an open coast, it is protected to a large extent from the direct attack of deepwater waves. Prevailing waves along the north-central coast of California are from the northwest. The southwestern exposure of Stinson Beach and the presence of Duxbury Point block much of the energy from these northwesterly waves. Major storms from the southwest occur during the winter months along the north-central coast. Even with the southwesterly exposure, Stinson Beach is partially protected from southwesterly storm waves because of the orientation of the offshore depth contours and the San Francisco Bar, which tend to refract waves away from the beach. The divergence of these storm waves reduces the wave energy reaching the beach. Stinson Beach is also partially protected from westerly waves by the Farallon Islands, which block some of the wave energy from reaching the beach's shoreline. Locally, along Stinson Beach, the wave energy usually decreases toward the tip of the spit (to the northwest) because of the flatter offshore slope, shallower offshore depth contours, and higher degree of protection provided by Duxbury Point. Thus, Stinson Beach Park usually experiences higher waves than does the Seadrift development.

Underdeveloped foredune sand deposits are present along the backshore of portions of Stinson Beach, mainly at Stinson Beach Park (where they are low and narrow) and the Patios area of Stinson Beach community. Foredune sand deposits elsewhere along Stinson Beach have been extensively damaged or destroyed by development. Dune heights vary from 15 to 25 ft above mean lower low water

(MLLW) and are from 50 to 75 ft wide. The foredune is sparsely-to-moderately vegetated with beach grass and other types of dune plants. The area landward from the foredune is of low relief with a high-water table and poor runoff. Willow stands and some evergreens grow behind the foredune at the northern end of the park. Prior to development of the park, the backshore area landward from the foredune was a marshy area, receiving freshwater runoff from Easkoot Creek and hills behind the park. The marshy area was periodically flooded with salt water when wave overwash of the low foredune occurred during storms. Much of this backshore area behind the foredune has now been developed into two paved parking lots (north and central portions of park) and one unpaved overflow parking lot at the south end of the park (Fig. 2). Four structures are located on the backshore area of the park: two comfort stations (north and south ends of the park), located just landward of the foredune; and a comfort station and lifeguard tower (central portion of park), situated on top of the foredune. One small drainage empties onto the beach just south of the lifeguard tower. The foredune is absent at the south end of the north parking lot, which is used as access to the beach.

FIGURE 2. Stinson Beach Park

PHYSICAL PROCESSES

The important physical processes at Stinson Beach affecting shoreline stability are winds, waves, and tides. These processes interact in the presence of beach sediment to maintain the beach in a stable condition. Changes in the prevailing conditions, such as intense winter storms accompanied by high tides, can cause abrupt changes in beach conditions. Often these abrupt changes are in the form of beach erosion and shoreline recession, which can damage structures located in the coastal zone and affect the use of the beach. The availability of beach material is critical to maintaining a stable shoreline under prevailing conditions or for beach recovery after a storm(s).

WIND

Wind is responsible for transporting sand from Stinson Beach onto the foredune. Wind data summaries are available for Southeast Farallon Island for the period of record from 1946 to 1958. Southeast Farallon Island is located in the Gulf of Farallones, about 25 miles west of Stinson Beach. This island has the nearest wind station unaffected by land masses and should be representative of onshore wind conditions at Stinson Beach.

Figure 3 is a summary of Southeast Farallon Island wind data. Prevailing winds are from the northwest, occurring about 120 d/yr (32%). West-northwest and north-northwest winds each occur for about 15% of the year, and winds from the west occur for about 5% of the year. Winds from the other compass points each occur for less than 5% of the year. Wind speeds are generally in the 4- to 15-mph range, occurring for about 250 d/yr. Wind speeds from 16 to 25 mph can be expected to occur about 65 d/yr, and speeds in excess of 26 mph can be expected about 7 d/yr. Table 1 is a summary of the monthly wind speed occurrence. The months of December through May are the windiest months of the year, and wind storms with speeds in excess of 26 mph most frequently occur during the months of October through May. Extreme wind conditions, with speeds from 45 to 50 mph, occur on an average of once a year. Wind speeds between 50 and 55 mph occur on the average of once every 2 years, and wind speeds in excess of 55 mph occur once in 10 years.

Wind Speed Percent Occurrence

Direction	Velocity (MPH)				Total
	1-3	4-15	16-25	26	
N	0.2	0.9	0.1		1.2
NNE	0.2	0.3			0.4
NE	0.1	0.6			0.7
ENE	0.2	1.2	0.3		1.7
E	0.4	1.8	0.4		2.6
ESE	0.3	1.6	0.4		2.3
SE	0.4	2.0	0.3	0.1	2.8
SSE	0.5	2.9	1.0	0.3	4.7
S	0.5	2.2	0.3		3.0
SSW	0.5	2.1	0.3		2.9
SW	0.9	2.3	0.1		3.3
WSW	0.5	2.1	0.2		2.8
W	0.9	3.9	0.3		5.1
WNW	0.6	12.2	2.8	0.2	15.8
NW	1.5	22.7	7.3	0.7	32.2
NNW	0.4	9.6	4.5	0.6	15.1
CALM					3.4
Total	11.4	68.4	18.3	1.9	100.0

FIGURE 3. Annual Wind Rose and Wind Speed, Southeast Farallon Island (1946-1958)

TABLE 1. Wind Speed Occurrence (in hours)
Southeast Farallon Island
(1946-58)

Month	Prevailing Direction	Wind Speed, mph				
		Calm	1-3	1-15	16-25	26+
January	NW	34	49	484	119	16
February	NW	27	48	444	130	23
March	NW	20	38	484	152	31
April	NW	22	41	454	162	22
May	NW	19	49	473	161	23
June	NW	19	48	452	175	7
July	NW	12	56	524	139	2
August	NW	22	56	517	139	1
September	NW	38	83	494	69	1
October	NW	21	75	508	32	15
November	NW	21	62	504	99	13
December	NW	19	63	481	135	28
Total		274	678	5819	1502	182

The formation of backshore dune deposits requires an abundance of beach sand and predominantly onshore winds. The rudimentary nature of the foredune deposits at Stinson Beach indicates that one or both of these requirements are not met. Stinson Beach is fairly wide; at times, the unwetted portion can be as wide as 200 ft. The nature of the spit as a depositional feature indicates that an adequate supply of beach sand should exist for the formation of fairly extensive dune deposits. If so, the wind direction distribution is the principal causative factor for the underdeveloped foredune at Stinson Beach. The southwest exposure of the shoreline and the presence of Duxbury Point protects Stinson Beach from the predominantly northwesterly winds. Onshore winds conducive to the transport of beach sand onto the backshore for the formation of dunes at Stinson Beach are from the west through south-southwest. These onshore winds occur for only about 15% of the year.

TIDES

Tide data for Stinson Beach are summarized in Table 2. The summary is based on the MLLW datum. Mean sea level (MSL) is +2.95 ft, MLLW; mean higher high water (MHHW) is +5.30 ft, MLLW. The extreme high and low tides are +7.10 ft, MLLW and -2.50 ft, MLLW, respectively. During storm conditions, the still water level (SWL) can be higher than that given in tide tables published by the Department of Commerce, National Ocean Survey (U.S. Department of Commerce, National Ocean Survey 1983) because of storm surge and wave setup. Using the extreme high tide, wind setup of 1.0 ft and wave setup of 0.15 times the breaking wave height, the SWL can be as high as 9.6 ft for a breaking wave height of 10 ft.

TABLE 2. Stinson Beach Tide Data
(Based on Mean Lower
Low Water Datum)

	<u>Tide, ft</u>
Extreme High Tide	7.10
Mean Higher High Water	5.30
Mean High Water	4.60
Mean Sea Level	2.95
Mean Low Water	1.10
Mean Lower Low Water	0.00
Extreme Low Tide	-2.50

Tidal currents at Stinson Beach Park are small and have very little effect on the stability of the shoreline. The entrance to Bolinas Lagoon is the only place along Stinson Beach spit where the tidal currents may be large enough to transport beach material.

WAVES

Deepwater Wave Conditions

Statistical deepwater wave conditions off the north-central coast of California have been computed by Meteorology International Incorporated

(Meteorology International Incorporated 1977) for the period of record from 1951 to 1974. Table 3 is the annual deepwater wave summary for Station 3, located due west of San Francisco. The wave heights in the table are daily significant wave heights (H_s), defined as the average of the one-third highest waves. Prevailing deepwater waves are from the north through west-northwest, occurring on an average of 215 d/yr. Waves from the west through southeast occur for about 62 d/yr. Calm conditions or offshore waves occur for about 88 d/yr. The most prevalent wave heights are from 3.0 to 4.5 ft. Wave heights in excess of 10 ft occur on the average of about 16 d/yr.

TABLE 3. Annual Summary of Deepwater Statistical Wave Data Due West of San Francisco (1951-74). Percent Occurrence by Direction and Height (Meteorology International Incorporated 1977)

Direction/ Height, ft	N	NNW	NW	WNW	W	WSW	SW	SSW	S	SSE	SE	Calm/ Offshore	Total
0.0 - 1.5	0.21	0.49	0.95	0.74	0.27	0.07	0.08	0.06	0.04				2.91
1.5 - 3.0	2.44	4.54	5.22	4.05	1.27	0.49	0.40	0.26	0.37	0.21	0.06		19.31
3.0 - 4.5	3.43	6.45	5.27	4.49	1.82	0.60	0.50	0.61	0.53	0.28	0.08		24.06
4.5 - 6.0	1.98	4.32	2.98	1.60	1.13	0.48	0.45	0.44	0.52	0.24	0.07		14.21
6.0 - 8.0	1.08	2.04	1.22	0.70	0.51	0.29	0.24	0.32	0.37	0.13	0.05		6.95
8.0 - 10.0	0.53	0.98	0.58	0.28	0.31	0.29	0.15	0.18	0.41	0.16	0.05		3.92
10.0 - 13.0	0.53	0.86	0.28	0.15	0.24	0.08	0.10	0.30	0.47	0.08	0.01		3.10
13.0 - 16.0	0.06	0.09	0.06	0.03	0.09	0.05	0.11	0.14	0.22	0.08			0.93
16.0 - 19.0	0.04	0.03	0.02		0.01	0.03	0.01	0.06	0.07	0.01			0.28
19.0 - 23.0						0.01	0.01	0.06	0.01	0.01			0.10
23+					0.01		0.01	0.01					0.03
Calm/Offshore												24.20	24.20
TOTAL	10.30	19.80	16.58	12.04	5.66	2.39	2.06	2.44	3.01	1.20	0.32	24.20	100.00

Daily significant storm waves in excess of 5 m (16.4 ft) have been estimated by Meteorology International Incorporated (Meteorology International Incorporated, 1977) from 1951 through 1974. These storm waves are summarized in Table 4. Storm waves most often occur during the months of December and January from most of the onshore compass points. Deepwater wave heights have exceeded 16.4 ft on 43 occasions. The most severe deepwater storm waves, with wave heights exceeding 20 ft, are from the west through south-southeast. The highest daily significant storm wave for the 24 years of record was 25.9 ft

TABLE 4. Severe Storm Waves at Station 3 Due West of San Francisco (1951-74)

Date	Height, ft	Period, sec	Direction	Date	Height, ft	Period, sec	Direction
03/06/51	16.4	13	NW	05/24/67	16.7	10	NNW
12/05/51	18.4	9	NW	01/13/68	20.0	11	SSE
12/07/52	18.4	10	WSW	12/10/68	18.7	10	SSW
02/12/54	18.0	10	SSW	12/23/68	18.4	10	S
12/18/55	16.7	10	SSW	12/11/69	16.7	10	S
12/19/55	18.7	10	SSW	01/15/71	16.7	10	S
12/20/55	16.4	9	SSW	01/22/72	19.7	10	WSW
12/23/55	16.7	10	WSW	01/26/72	18.0	13	N
02/21/56	25.9	12	SW	04/17/72	16.7	10	N
02/24/57	23.3	12	SSW	04/28/72	16.4	10	NNW
02/01/58	17.1	10	SSE	12/16/72	16.4	10	SSE
04/02/58	19.7	9	SW	01/16/73	17.4	10	SSE
01/08/59	18.7	10	S	02/24/73	19.0	11	S
02/15/59	17.4	10	S	03/13/73	19.4	11	NNW
02/09/60	23.6	11	W	03/28/73	18.0	10	NNW
03/07/64	18.4	10	NNW	01/14/74	17.7	10	S
12/22/64	18.4	10	SW	01/15/74	16.7	10	SSE
12/23/64	17.4	10	WSW	01/16/74	17.1	10	SSW
12/24/64	18.7	10	W	01/18/74	16.4	10	SSW
11/13/65	16.7	10	SSE	02/28/74	18.4	10	SW
12/28/65	21.7	11	SSW	03/01/74	23.6	12	SW
				11/21/74	21.0	11	S

from the southwest on February 21, 1956. Southwesterly storm waves most often occur during the months of December through February, northwesterly storm waves from March through May.

The frequency of occurrence of deepwater storm waves in excess of 10 ft has been computed for the 24 years of record from 1951-74, based on Meteorology International Incorporated (1977) statistical wave analysis (Table 5). Deep-water waves in excess of 10 ft can be expected to occur on the average of 16 d/yr. Waves in excess of 13 ft can be expected to occur for about 5 d/yr

TABLE 5. Number of Days Deepwater Wave Heights Exceeded Indicated Height (1951-74)

Year	Significant Wave Height, ft				
	>10	>13	>16	>19	>23
1951	10	3	2	0	0
1952	12	2	1	0	0
1953	17	2	0	0	0
1954	13	4	1	0	0
1955	17	6	4	0	0
1956	11	4	1	1	1
1957	7	3	1	1	1
1958	17	6	2	1	0
1959	11	4	2	0	0
1960	18	7	1	1	1
1961	8	0	0	0	0
1962	15	3	0	0	0
1963	11	2	0	0	0
1964	16	7	4	0	0
1965	22	5	2	1	0
1966	12	4	0	0	0
1967	15	4	1	0	0
1968	25	11	3	1	0
1969	21	6	2	0	0
1970	16	3	0	0	0
1971	15	1	1	0	0
1972	31	12	5	1	0
1973	37	13	4	0	0
1974	11	11	6	2	1
Total	394	125	43	9	4
Average Annual	16	5	2	0.38	0.17

and waves in excess of 16 ft can be expected to occur 2 d/yr. Waves in excess of 19 ft can be expected to occur on the average of every 2 years and waves in excess of 23 ft can be expected to occur every 6 years.

Storm Waves of 1982-83

Eleven major storms with daily significant wave heights exceeding 10 ft were recorded at the Farallon Island wave gage (State of California, Department of Boating and Waterways 1983) from November 1982 through April 1983. Figure 4 shows the daily significant deepwater wave heights at the Farrallon Island gage from January 1982 through April 1983. Table 6 summarizes the maximum and daily significant wave heights and wave periods for these storm waves. The first

FIGURE 4. Significant Daily Wave Heights, Farallon Islands, 1982-83

TABLE 6. 1982-83 Storm Waves at the Farallon Island Wave Gage

Date	Maximum Significant Height, ft	Daily Significant Height, ft	Daily Significant Period, sec
Nov 30	21.6	20.6	12.1
Dec 1	16.7	14.2	10.9
Dec 2	13.7	10.8	11.0
Dec 15	14.9	12.7	12.6
Dec 16	17.1	15.8	13.1
Dec 17	14.8	14.5	13.0
Dec 18	16.7	13.5	13.0
Dec 19	10.7	10.0	12.3
Dec 22	18.2	14.6	11.6
Dec 23	14.7	13.7	10.6
Dec 24	11.4	10.1	11.2
Jan 18	13.3	11.4	10.3
Jan 19	14.6	13.4	12.3
Jan 20	14.4	11.7	12.0
Jan 22	11.0	10.1	10.5
Jan 23	14.0	13.2	13.8
Jan 24	17.7	15.7	12.4
Jan 25	15.9	13.9	12.6
Jan 26	25.0	19.8	12.2
Record Missing from Jan 27 - Feb 10			
Feb 11	12.8	12.8 ^(a)	9.9
Feb 12	16.2	15.0	11.8
Feb 13	16.4	12.0	13.0
Feb 14	13.3	10.6	12.0
Feb 15	13.6	10.7	14.1
Feb 16	13.6	13.2	14.0
Feb 18	14.6	14.3	10.3
Feb 19	12.8	12.8	12.5
Feb 20	14.4	12.7	13.8
Feb 21	11.6	10.6	12.9
Feb 27	17.0	13.8	11.4
Feb 28	20.5	20.5 ^(a)	14.1
Mar 1	19.8	19.8 ^(a)	14.1
Mar 2	20.1	19.0	13.5
Mar 3	10.4	10.1	10.8
Mar 4	15.9	11.8	11.3
Mar 17	14.4	13.5	13.1
Mar 18	13.3	11.5	12.0
Mar 24	15.1	15.1 ^(a)	10.5
Mar 25	13.4	11.8	10.6
Apr 2	12.1	10.7	11.9
Apr 3	13.6	12.6	9.8

(a) Only 6 h of record available for that day.

storm began on November 30, 1982, with a daily significant wave height of 20.6 ft and lasted 3 days. A second 5-day storm began on December 15, 1982. The highest daily significant wave height during this storm was 15.8 ft on December 16. A third storm occurred between December 22, and December 24, 1982, with the highest daily significant wave height of 14.6 ft. From January 18, through March 4, 1983, five consecutive storms occurred. The largest of these storms occurred between February 27 and March 4, 1983, with the highest daily significant wave height of 20.5 ft on February 28. The maximum 6-h significant wave height of 25 ft was recorded on January 26, 1983. The wave gage was damaged as a result of this storm and remained inoperable for a 15-day period through February 11, 1983. Three smaller storms of 2-day durations occurred in March and April 1983.

Table 7 is a comparison of the number of days in 1981, 1982, and 1983 when waves of a given height were exceeded at the Farallon Island gage. Daily significant wave heights in 1983 exceeded 10 ft on 32 days, exceeded 13 ft on 14 days, exceeded 16 ft on 4 days, and exceeded 19 ft on 4 days. The major difference in the winter storms of 1982-83 in comparison to 24 years of statistical wave data in Table 5 is in the frequency rather than the magnitude of the storm waves. Table 5 indicates that daily significant wave heights exceed 10 ft on the average of 16 d/yr; yet in 1983, daily significant wave heights exceeded 10 ft on 32 occasions. Only in 1973 did the daily significant wave height exceed 10 ft on more occasions (37 days) in the 24 years of record (see Table 5). The daily significant wave height in 1983 also exceeded 13 ft on more occasions than in any given year in the 24 years of record. Whereas, wave heights exceeded 19 ft on 1 or 2 days at most for the 24 years of record in Table 5 (return period of once in 2 years), daily significant wave heights exceeded 19 ft on four occasions during 1983. Daily significant wave heights did not exceed 23 ft in 1983. The return period for waves of 23 ft or higher is 6 years, and for the 24 years of record, daily significant waves exceeded 23 ft on 1 day each in 1956, 1957, 1960, and 1974.

TABLE 7. Number of Days Deepwater Wave Heights Exceeded Indicated Height, 1981-83

Year	Significant Wave Height, ft				
	>10	>13	>16	>19	>23
1981	21	8	2	0	0
1982	25	9	1	1	0
1983	32	14	4	4	0

Shallow Water Waves

When waves approach the coast at an angle to the depth contours, the part of the wave advancing in shallower water moves more slowly than that part still advancing in deeper water, causing the wave crest to bend toward an alignment with the bottom contours. This process by which the direction of travel of a wave moving into shallow water is altered because of differential change in velocity along the crest of the wave is termed wave refraction. Wave refraction coupled with shoaling determines the wave height at a given location for a given set of incident deepwater wave conditions. Wave refraction has a significant influence on wave heights at Stinson Beach because of the fairly shallow offshore depths in the Gulf of Farallones, and the presence of the Farallon Islands to the west, San Francisco Bar to the south, and Duxbury Point to the northwest. Stinson Beach is exposed to waves from the northwest through south-southeast, but because of wave refraction, the breaking wave heights at the shoreline will generally be lower than the corresponding deepwater wave heights.

A wave refraction computer program (Perry and Street 1969) was used to evaluate the effects of wave refraction on waves breaking on the Stinson Beach shoreline. From the deepwater waves (see Table 3), the program simulates the process by which each appropriate deepwater wave is propagated toward shore. Starting from a known point on a depth grid, the computer program constructs wave rays (a line perpendicular to the wave crest) step-by-step across the grid. Refraction or the change in wave direction is treated analytically in the refraction program using the principle of Snell's law:

$$\sin \alpha_2 = \frac{C_2}{C_1} \sin \alpha_1, \quad (\text{U.S. Army Corps of Engineers 1977})$$

where:

α_1 = the angle a wave crest makes with the bottom contour over which the wave is passing

α_2 = similar angle as the wave crest passes over the next bottom contour

C_1 = the wave velocity for the depth of the first contour

C_2 = the wave velocity for the depth of the second contour.

At each time step the refraction coefficient (K_R), shoaling coefficient (K_S), direction of wave advance (α), velocity of the wave (C), and wavelength (L) are computed. The wave height at the shoreline (H) is computed by multiplying the deepwater wave height (H_0) by the refraction coefficient (K_R) and shoaling coefficient (K_S):

$$H = H_0 K_R K_S$$

Two grid systems shown in Figures 5 and 6 were used in the wave refraction analysis. The outer grid with a scale of 1 grid unit = 4167 ft was used to propagate deepwater waves into intermediate depth water and the inner grid with a scale of 1 grid unit = 833 ft was used to evaluate the nearshore wave characteristics. Figures 7 and 8 are examples of the outer and inner grid refraction diagrams. Wave convergence and divergence can be clearly seen in these figures. Inner grid wave refraction diagrams for representative deepwater wave directions and periods are shown in Appendix A.

Table 8 is a summary of wave refraction coefficients, shoaling coefficients, and breaking wave ray angles (relative to the shoreline) for all representative deepwater wave directions and periods at Stinson Beach Park. Stinson Beach is exposed to deepwater waves from the northwest through south-southeast. Deepwater waves from the north, north-northwest and southeast will not strike Stinson Beach because of the orientation of the shoreline and protection

FIGURE 5. Stinson Beach Outer Grid

FIGURE 6. Stinson Beach Inner Grid

FIGURE 7. Wave Refraction Diagram, Outer Grid

FIGURE 8. Wave Refraction Diagram, Inner Grid

TABLE 8. Stinson Beach Park Wave Refraction

Deepwater Direction	Wave Period, sec						
	5	7	9	11	13	15	17
Refraction Coefficient (K_R)							
NW				0.35	0.32	0.12	0.14
WNW	0.21	0.34	0.31	0.50	0.55	0.35	0.31
W	0.35	0.46	0.59	0.39	0.32	0.30	0.20
WSW	0.65	0.51	0.40	0.31	0.25	0.23	0.21
SW	0.87	0.79	0.54	0.46	0.39	0.33	0.32
SSW	0.99	0.96	0.88	0.75	0.56		
S		0.81	0.72	0.72	0.90		
SSE		0.39	0.61	0.41			
Shoaling Coefficient (K_S)							
NW				1.28	1.40	1.99	2.04
WNW	1.23	1.14	1.26	1.21	1.27	1.46	1.57
W	0.99	1.09	1.12	1.28	1.39	1.49	1.68
WSW	0.93	1.04	1.20	1.33	1.46	1.58	1.66
SW	0.92	0.97	1.14	1.22	1.37	1.46	1.55
SSW	0.92	0.97	1.04	1.14	1.27		
S		0.99	1.07	1.07	1.18		
SSE		1.09	1.08	1.27			
Wave Angle (α)							
NW				-85.96	-84.26	-84.92	-85.87
WNW	-93.49	-91.99	-85.80	-83.87	-81.75	-89.32	-88.92
W	-102.60	-90.16	-83.47	-84.43	-80.30	-72.11	-86.00
WSW	-97.83	-85.86	-80.79	-85.20	-84.18	-82.16	-82.06
SW	-86.17	-79.83	-78.92	-77.00	-76.65	-78.57	-76.27
SSW	-69.01	-69.69	-71.51	-75.39	-76.50		
S		-59.54	-63.81	-65.29	-68.78		
SSE		-67.74	-64.38	-71.11			

provided by Duxbury Point. Wave divergence will normally occur as waves are propagated towards the Stinson Beach shoreline, resulting in refraction coefficients less than unity. The greatest refraction (lowest refraction coefficient) corresponds to higher period waves from the northwest through southwest. Shoaling coefficients are generally greater than unity, offsetting to some degree, the effects of wave refraction on shallow-water wave heights. Shallow-water wave heights at Stinson Beach, nevertheless, will generally be less than the corresponding deepwater wave heights because the product of the refraction and shoaling coefficients are less than unity. Table 9 is a summary

TABLE 9. Combined Refraction and Shoaling Coefficients for Stinson Beach Park, Seadrift, and Rocky Point

Direction	Wave Period, sec						
	5	7	9	11	13	15	17
Stinson Beach Park							
NW				0.45	0.45	0.24	0.29
WNN	0.26	0.39	0.39	0.61	0.70	0.51	0.49
W	0.35	0.50	0.66	0.50	0.44	0.45	0.34
WSW	0.60	0.53	0.48	0.41	0.37	0.36	0.35
SW	0.80	0.77	0.62	0.56	0.53	0.48	0.50
SSW	0.91	0.93	0.92	0.86	0.71		
S		0.80	0.77	0.77	1.06		
SSE		0.43	0.66	0.52			
Seadrift Beach							
NW					0.30	0.26	0.23
WNN	0.27	0.27	0.26	0.30	0.37	0.32	0.36
W	0.28	0.31	0.38	0.36	0.35	0.32	0.39
WSW	0.37	0.42	0.39	0.33	0.39	0.61	0.57
SW	0.39	0.64	0.70	0.57	0.56	0.58	0.54
SSW	0.76	0.70	0.81	0.70	0.66		
S		0.78	0.63	0.61	0.53		
SSE	0.47	0.59	0.55	0.68			
Rocky Point							
NW			0.40	0.22	0.30	0.41	0.73
WNN	0.25	0.59	0.44	0.80	0.87	0.81	0.73
W	0.71	0.58	0.74	0.58	0.47	0.56	0.51
WSW	0.89	0.86	0.78	0.51	0.32	0.46	0.37
SW	0.91	0.90	0.90	0.79	0.73	0.80	0.79
SSW	0.88	0.89	1.01	1.06	0.92		
S		0.64	0.61	0.73	1.06		
SSE		0.37	0.54	0.79			

of the combined refraction/shoaling coefficients for Stinson Beach Park, Seadrift Beach (located about 1.4 miles northwest of the park), and Rocky Point (located about 1.3 miles southeast of the park). The shallow-water wave height is equal to the deepwater wave height times the product of the refraction and shoaling coefficients ($H = H_0 K_R K_S$). Breaking wave heights at Stinson Beach Park are from 8 to 76% lower than the corresponding deepwater waves, depending on the deepwater wave direction and period. The greatest wave height reduction

occurs with deepwater waves from the northwest through west-southwest. The least amount of wave height reduction occurs with deepwater waves from the southwest through south because of the more direct exposure to waves from these directions. Wave heights at Seadrift Beach are generally lower than those at Stinson Beach Park because of the higher degree of protection provided by Duxbury Point. Longer period waves from the west-southwest and southwest can be as high or higher at Seadrift Beach than at Stinson Beach Park. Wave heights at Rocky Point are generally higher than at Stinson Beach Park because the degree of protection provided by Duxbury Point for deepwater waves from the northwest through west is not as great. Waves from the west-southwest through south-southwest are also higher at Rocky Point than Stinson Beach Park because the offshore bathymetry focuses waves from these directions at Rocky Point.

Storm Waves at Stinson Beach Park

Storm waves at Stinson Beach Park have been computed using the severe deepwater storm waves in Table 4 and the combined refraction and shoaling coefficients in Table 9. Table 10 is a comparison of deepwater storm waves with corresponding breaking wave heights at Stinson Beach Park, Seadrift Beach, and Rocky Point. The largest storm waves at Stinson Beach Park are from the south-southwest, where breaking wave heights at times can exceed 16 ft. The highest breaking wave for the period of record from 1951 to 1974, based on the wave refraction analysis, is 18.6 ft, corresponding to the storm of December 28, 1965. The deepwater significant wave height during this storm was 21.7 ft from the south-southwest with a period of 11 sec. The computed breaking wave heights at Seadrift Beach and Rocky Point during this storm were 12.3 ft and 24.7 ft, respectively.

Wave Runup at Stinson Beach Park

Wave runup on the foredune at Stinson Beach Park has been calculated to estimate the frequency of wave overwash and flooding of the backshore. Figure 9 illustrates the height of runup above MLLW for breaking wave heights from 6 to 20 ft. The curves represent runup for the estimated highest tide (7.1 ft, MLLW), MHHW (5.3 ft, MLLW), MSL (2.95 ft, MLLW), and MLLW (0.0 ft). A wind setup of 1 ft and wave setup of 0.15 times the breaking wave height were added

TABLE 10. Comparison of Deepwater and Breaking Wave Heights

Date	Deepwater Wave			Breaking Wave Height, ft		
	Height, ft	Period, sec	Direction	Rocky Point	Stinson Park	Seadrift
03/06/51	16.4	13	NW	4.9	7.4	4.9
12/05/51	18.4	9	NW	7.4	7.1	2.8
12/07/52	18.4	10	WSW	9.4	7.6	6.1
02/12/54	18.0	10	SSW	19.1	15.4	12.6
12/18/55	16.7	10	SSW	17.7	14.3	11.7
12/19/55	18.7	10	SSW	19.8	16.0	13.1
12/20/55	16.4	9	SSW	16.6	15.0	13.2
12/23/55	16.7	10	WSW	8.6	6.9	5.5
02/21/56	25.9	12	SW	25.1	13.8	14.5
02/24/57	23.3	12	SSW	21.5	16.6	15.3
02/01/58	17.1	10	SSE	13.5	8.9	11.6
04/02/58	19.7	9	SW	17.8	12.1	16.3
01/08/59	18.7	10	S	13.7	14.4	11.4
02/15/59	17.4	10	S	12.7	13.4	10.7
02/09/60	23.6	11	W	13.6	11.8	8.5
03/07/64	18.4	10	NNW	---	---	---
12/22/64	18.4	10	SW	14.5	10.3	10.4
12/23/64	17.4	10	WSW	8.9	7.2	5.8
12/24/64	18.7	10	W	10.8	9.3	6.7
11/13/65	16.7	10	SSE	13.1	8.7	11.3
12/28/65	21.7	11	SSW	24.7	18.6	12.3
05/24/67	16.7	10	NNW	---	---	---
01/13/68	20.0	11	SSE	15.7	10.4	13.6
12/10/68	18.7	10	SSW	19.8	16.0	13.1
12/23/68	18.4	10	S	13.5	14.2	11.3
12/11/69	16.7	10	S	12.2	12.9	10.2
01/15/71	16.7	10	S	12.2	12.9	10.2
01/22/72	19.7	10	WSW	10.1	8.1	6.5
01/26/72	18.0	13	N	---	---	---
04/17/72	16.7	10	N	---	---	---
04/28/72	16.4	10	NNW	---	---	---
12/16/72	16.4	10	SSE	12.9	8.5	11.1
01/16/73	17.4	10	SSW	18.4	14.9	12.2
02/24/73	19.0	11	S	13.9	14.6	11.6
03/13/73	19.4	11	NNW	---	---	---
03/28/73	18.0	10	NNW	---	---	---
01/14/74	17.7	10	S	13.0	13.6	10.8
01/15/74	16.7	10	SSW	17.7	14.3	11.7
01/16/74	17.1	10	SSW	18.1	14.6	12.0
01/18/74	16.4	10	SSW	17.4	14.0	11.5
02/28/74	18.4	10	SW	14.5	10.3	10.4
03/01/74	23.6	12	SW	22.8	12.6	13.2
11/21/74	21.0	11	S	15.4	16.2	12.9

to the tidal height. The minimum crest elevation of the foredune is about +15 ft, MLLW, occurring along the south overflow parking lot, between the north and central comfort stations and near the north park boundary. Wave overwash of the foredune at the highest estimated tide will occur when the breaking wave height exceeds 14 ft. This height corresponds to a deepwater wave height of about 19 ft, which has a recurrence interval of once in 2 to 3 years. Wave runup estimates on a 2H (horizontal) on 1V (vertical) riprap slope are also shown in Figure 9. The maximum runup at the estimated highest tide would be about 18 ft, corresponding to a breaking wave height of 20 ft.

LONGSHORE SEDIMENT TRANSPORT

Longshore (littoral) sediment transport is the process by which sediment in the nearshore zone is moved by waves and currents. Longshore transport results from the stirring up of sediment by breaking waves and movement of sediment in an alongshore direction by longshore currents generated by breaking waves. Beach sediment can also be transported offshore by this same process, where it may be temporarily deposited until a change in wave conditions causes the sediment to be transported back onshore.

Prevailing northwesterly waves along the north-central coast of California cause a general southward longshore sediment transport. Localized areas along the coast, however, can experience net northerly longshore sediment transport. These localized areas are commonly located on the south side of headlands, where the headlands provide some degree of protection from the prevailing northwesterly waves. The longshore sediment transport rates in these areas are usually small, but southwesterly storm waves can cause short periods of intense transport to the north. Stinson Beach appears to be one of these localized areas that is protected from the prevailing northwesterly waves, resulting in a net northerly longshore sediment transport of low magnitude from southerly waves. The presence of Duxbury Point to the northwest and configuration of Stinson Beach spit with the entrance to Bolinas Lagoon at the north end of the spit indicate that the net longshore sediment transport is to the north.

Note: HET = Highest Estimated Tide
 MHHW = Mean Higher High Water
 MLLW = Mean Lower Low Water
 MSL = Mean Sea Level

FIGURE 9. Wave Runup, Stinson Beach Park

J. W. Johnson, in a paper on nearshore sediment movement along the central coast of California (Johnson 1965), briefly discussed longshore sediment transport at Bolinas Bay and Stinson Beach. Johnson indicated that no data are available on the character of longshore sediment movement in this area, but based on heavy mineral studies by Moore (1965), very little nearshore sediment transport appears to occur; and the transport that does occur appears to be directed southward. Because of the discrepancy between Johnson's description of longshore transport and the apparent transport from observation of the area, a computerized longshore sediment transport analysis of Stinson Beach was performed.

Longshore Sediment Transport Analysis

The rate of longshore sediment transport is dependent on both the angle of wave approach to the shoreline and available wave energy. Large storm waves will generally move more beach material per unit time than the low waves, and reversals in transport direction can occur as a result of changes in wave approach. Where reversals occur the longshore transport rate can be evaluated by considering the 'gross' transport rate, defined as the sum of the volumes of transport in both directions past a point, or by considering the 'net' transport rate, defined as the difference in the volumes of transport from each direction past a point.

Longshore sediment transport rates are usually given in units of volume per unit time (cubic yards per year). Typical longshore transport rates along exposed coasts of the United States vary from 8,000 to 90,000 cubic yards per year (cy/yr) in the Great Lakes, to 30,000 to 1,000,000 cy/yr along the Atlantic and Pacific coasts. The two nearest areas where estimates of longshore sediment transport rates have been made are San Francisco's Ocean Beach (Perry and Street 1969) and Fort Point Beach (Ecker 1983), just inside the entrance to San Francisco Bay. The estimated longshore sediment transport rate at Ocean Beach is between 1 and 2 million cy/yr and at Fort Point Beach it is about 75,000 cy/yr.

The best method for predicting longshore sediment transport rates is to adopt the best known rate from a nearby site with modifications based on local conditions. If rates from nearby sites are unavailable or are known to vary

considerably, the next-best way to predict transport rates is to compute them from data showing historical changes in nearshore depths and shoreline position. If neither of these two methods are practical, then an acceptable practice is to use either measured or calculated wave conditions to compute the longshore component of wave energy (wave energy flux), then relate this energy through empirical relationships to the longshore sediment transport rate (U.S. Army Corps of Engineers 1977).

Since there is a lack of data on longshore sediment transport rates in the vicinity of Stinson Beach, an analysis of longshore sediment transport has been performed based on the energy flux method. The Stanford wave refraction and littoral transport computer program (Perry and Street 1969) was used to compute longshore transport rates from deepwater wave conditions (see Table 3) for Stinson Beach Park. The longshore transport program computes the monthly and annual deepwater wave energy for each wave direction, height, and period in Table 3; then, using the wave refraction data, calculates the longshore component of wave energy flux. Using an empirical relation for longshore wave energy flux and longshore transport rate developed by the Coastal Engineering Research Center (U.S. Army Corps of Engineers 1977), the monthly and annual longshore sediment transport rates are computed. Using this method, these rates are estimates of the 'potential' transport, or that volume of beach sediment that would be transported by the prevailing wave conditions if an infinite supply of beach sediment were available for transport.

Table 11 is a summary of the average monthly and annual deepwater wave energy by direction at Meteorology International's Station 3 due west of San Francisco (Meteorology International Incorporated 1977). Deepwater energy for waves from the north have not been included in the table because these waves will not reach the Stinson Beach shoreline. The months of November through April have the highest deepwater wave energy, accounting for about 75% of the annual deepwater wave energy. The majority of the deepwater wave energy is from the north-northwest through west-northwest; however, storms from the south-southwest and south provide much of the deepwater wave energy for the months of December through February.

TABLE 11. Deepwater Wave Energy Due West of San Francisco

Direction	Deepwater Wave Energy, millions of ft-lb												
	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
NNW	393.5	344.9	835.7	1150.0	1467.0	1195.0	1112.0	788.6	276.6	248.5	234.0	345.1	8390.9
NW	669.4	489.1	789.5	676.1	591.2	529.1	344.6	314.6	148.9	431.9	693.2	911.9	6589.5
WNW	749.8	561.4	797.2	462.7	214.0	115.2	42.7	84.7	157.1	306.1	754.7	717.1	4962.7
W	584.6	733.1	656.7	211.6	66.1	43.4	9.2	20.0	24.7	85.3	267.1	644.5	3346.3
WSW	548.6	114.9	321.1	150.6	25.9	8.4	0.0	0.6	5.6	92.7	84.2	413.3	1765.9
SW	359.2	481.9	153.5	143.7	23.9	1.8	7.9	1.7	0.7	69.6	201.6	186.0	1631.5
SSW	1065.0	695.9	229.4	88.3	1.7	0.0	0.9	3.5	27.8	74.6	166.8	624.7	2978.6
S	964.3	772.4	164.3	85.2	26.9	0.0	0.1	1.7	0.7	89.8	584.5	961.5	3651.4
SSE	364.3	337.7	42.8	0.7	0.9	0.0	0.0	0.0	0.0	20.0	117.3	178.7	1062.4
Total	5698.7	4531.3	3990.2	2968.9	2417.6	1892.9	1517.4	1215.4	642.1	1418.5	3103.4	4982.8	34379.2

TABLE 12. Longshore Energy Flux at Stinson Beach Park

Direction	Longshore Energy Flux, millions of ft-lb												
	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
NNW	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
NW	1.6	0.6	0.5	0.6	0.2	0.3	0.0	0.0	0.1	0.8	1.6	1.8	8.1
WNW	2.3	-1.1	-15.6	-2.2	-5.9	-1.3	-0.3	0.1	0.8	0.5	1.0	-2.6	-24.3
W	-0.7	46.9	-4.5	3.8	0.2	-0.7	-0.1	-0.2	0.3	-1.9	2.7	18.5	64.3
WSW	3.2	1.0	0.6	-0.4	0.0	-0.3	0.0	0.0	0.0	-0.1	-0.7	2.1	5.4
SW	17.5	106.0	8.4	6.7	1.9	0.1	0.7	0.1	0.0	3.7	12.3	9.2	166.6
SSW	231.5	136.8	66.1	26.0	0.5	0.0	0.3	1.1	9.7	20.7	53.1	127.5	673.3
S	380.7	338.3	47.5	32.8	10.7	0.0	0.0	0.5	0.2	29.9	253.4	336.2	1430.2
SSE	59.4	51.9	7.1	0.0	0.0	0.0	0.0	0.0	0.0	0.9	13.5	20.6	153.4
Total	695.5	680.4	110.1	67.3	7.6	-1.9	0.6	1.6	11.1	54.5	336.9	513.3	2477.0

Note: Negative values indicate energy flux is directed south, and positive values indicate energy flux is directed north.

Table 12 summarizes the monthly and annual longshore energy flux at Stinson Beach Park by deepwater wave direction. Whereas deepwater waves from the north-northwest through west-northwest makeup about 50% of the annual deepwater wave energy, they account for only about 2% of the annual longshore energy flux at Stinson Beach Park because of the protection provided by Duxbury Point. South and south-southwest waves account for only about 20% of the annual deepwater wave energy, but account for 85% of the annual longshore energy flux at the park because of the fairly direct exposure of Stinson Beach to deepwater waves from these directions.

Table 13 is a summary of the monthly and annual longshore sediment transport rates at Stinson Beach Park by deepwater wave direction. The net and gross annual transport rates are provided. Negative transport rates denote transport southeast, and positive transport rates denote transport northwest. The average annual net longshore sediment transport rate at Stinson Beach Park is about 310,000 cy and is directed to the northwest. The highest transport rates occur during the months of November through March, resulting almost exclusively from south and south-southwest storm waves. Waves from the south and south-southwest account for about 90% of the net annual longshore sediment transport. The monthly transport rates during the months of May through August are very low, and the net monthly transport is directed to the southeast only during the month of June. The southerly component of the annual longshore sediment transport is very small at Stinson Beach Park as is indicated by comparison of the annual net and gross transport rates.

TABLE 13. Longshore Sediment Transport at Stinson Beach Park

Direction	Longshore Sediment Transport, cy												Annual	
	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Net	Gross
NNW	0	0	0	0	0	0	0	0	0	0	0	0	0	0
NW	200	80	70	80	30	0	0	0	10	110	200	230	1,010	1,010
WNW	290	-150	-1,990	-280	-750	-170	-40	10	90	70	130	-330	-3,120	4,300
W	-90	6,000	-580	490	30	-90	-10	-20	40	-240	340	2,370	8,240	10,300
WSW	410	120	80	-50	10	-40	0	-10	0	-10	-80	270	700	1,080
SW	2,240	2,670	1,070	860	240	10	90	10	0	480	1,610	1,180	10,460	10,460
SSW	29,620	17,510	8,450	3,400	70	0	30	140	1,250	2,670	6,800	16,330	86,270	86,270
S	48,740	43,310	6,080	4,200	1,360	0	0	70	30	3,830	32,430	46,480	186,530	186,530
SSE	7,610	6,650	910	0	0	0	0	0	0	110	1,730	2,640	19,650	19,650
Total	89,020	76,190	14,090	8,700	990	-290	70	200	1,420	7,020	43,160	69,170	309,470	319,600

Note: Negative values indicate transport south and positive values indicate transport north.

1982-83 STORM DAMAGE TO STINSON BEACH

Stinson Beach Park, as well as the rest of Stinson Beach, experienced considerable wave-induced storm damage during the winter of 1982-83. Storm waves from the south through south-southwest denuded the beaches of sand, threatened buildings and parking areas, and overwashed the foredune, flooding the back-shore area. Eleven major storms occurred from November 1982 through April 1983, with deepwater wave heights exceeding 10 ft (see section on Waves). The duration of these storms was from 2 to 6 days each. During January and February the storms were almost continuous, with less than 4 days respite between them. The magnitude of the storm waves were not overly uncommon. The highest daily significant deepwater wave height for the 11 storms was 20.5 ft on February 28. The recurrence interval for such storm waves is from 2 to 6 years. The 1982-83 winter storms were uncommon in that the number of days when the significant wave heights exceeded 10 ft was twice the average annual occurrence of 16 times per year. The number of storms accompanied by high tides was the principal cause of the storm damage.

Damage to Stinson Beach Park facilities and beach was in the form of loss of beach sand, damage to the foredune from wave runup, loss of foredune vegetation, flooding of the backshore area from wave overwash, and threatening of structures. Wave-induced erosion at the Park completely denuded the beach of sand, leaving a 6- to 8-ft scarp at the seaward edge of the foredune. A thin veneer of coarse gravel remained on the beach face, and a fairly resistant clay hardpan was exposed near the high-water line. This hardpan appears to have restricted the vertical degradation of the beach and may have also limited the landward recession of the wave-cut scarp by partially dissipating the wave energy once the hardpan was exposed.

Based on beach profile surveys and photographs of the beach by National Park Service personnel, an estimated 11.9 cy of sand per linear foot of beach

was removed from the upper beach face (above +8 ft, MLLW) during January 1983.^(a) This corresponds to a total sand loss of about 38,000 cy above the high-tide line. If the remainder of the beach face is taken into consideration (e.g., down to the MLLW line), an estimated 100,000 to 150,000 cy of sand could have been removed from the entire park beach during the 1982-83 winter. A site inspection of Stinson Beach Park, made in January 1984, evaluated the recovery of the beach. The beach has almost totally recovered to its prestorm condition within 1 year. At the time of the inspection, the beach was fairly wide, and sand had covered the beach scarp and clay hardpan. The only remnants of the 1982-83 storm damage observed during the site inspection was the overwash damage to the foredune fronting the south overflow parking lot. The relatively quick recovery of the beach indicates that a large portion of the eroded beach sand was stored offshore during the winter storms and was subsequently transported back onshore during the summer and fall of 1983. The recovery of the foredune (especially at the south end of the park), however, will take much longer or may not occur at all without help from man.

During the storms, a real threat existed to the lifeguard tower and the central comfort station. The foundations of both structures were in danger of being undermined by wave action if the beach continued to retreat. Wave overwash of the foredune also caused flooding of the backshore in the south overflow parking lot and to a lesser extent, the north parking lot.

(a) Internal memorandum on beach erosion at Stinson Beach to General Superintendent, Golden Gate National Recreation Area, California, by J. H. Howell, February 15, 1983.

SHORELINE PROTECTION ALTERNATIVES

A number of shoreline protection measures could be implemented at Stinson Beach Park to prevent or decrease storm wave runup damage to the foredune, flooding of the backshore, and potential damage to existing structures. These include both structural and nonstructural measures, which will provide varying degrees of protection to the backshore. Evaluation of foreshore (beach) protection or enhancement alternatives will not be assessed, other than the off-shore placement of artificial seaweed, because of the rapid recovery of the beach following the 1982-83 storms. Construction of an artificial beach or placement of sand trapping structures (e.g., groins, seals) could provide protection of the backshore from storm waves; however, the construction and maintenance costs for such measures are generally very high, making them difficult to justify economically. In addition structural protection measures placed on the beach could disrupt existing equilibrium conditions of Stinson Beach spit, interrupting the longshore transport of beach material. Alterations in beach configuration could then occur for fairly long distances along the beach as it readjusts. Since the predominant direction of longshore sand transport is to the northwest toward the end of the spit, placement of structures on the beach of the park could affect the beaches at Stinson Beach community and Seadrift. The following shoreline protection alternatives are considered in this section:

- No Action
- Foredune Development/Enhancement
- Riprap Revetment
- Artificial Seaweed.

NO ACTION

The No Action alternative entails continuation of the present maintenance practices at Stinson Beach Park without implementation of any concerted protection or enhancement measures. Present maintenance practices include removal of overwash and wind-blown sand from the parking areas, repair of picnic facilities from flood damage, and infrequent enhancement of the foredune at the south end of the park. Continuation of the present actions without implementing protection or enhancement measures could result in periodic and temporary

degradation of the beach, wave overwash and backshore flooding at low points in the foredune during extreme storm events and high tides, and potential damage or destruction of some facilities and parking areas.

Stinson Beach Park is subject to seasonal erosion/accretion cycles where the beach is degraded during the high-wave energy winter months and aggraded during the low-wave energy summer months. Normal variations in the seasonal beach cycle do occur resulting from annual differences in wave intensity, causing the beach to degrade more during some years than others. Sand losses from the park beach during the 1982-83 winter were abnormally large in that the upper beach face was completely denuded of beach sand. Although storm waves during that winter were large with significant daily deepwater wave heights at times exceeding 20 ft, they were not uncommon. Storm waves in excess of 19 ft have a recurrence interval of about once every 2 1/2 years. The 1982-83 winter was uncommon in the number and duration of storms, which in some cases were accompanied by high tides. At least 11 major storms occurred during the 1982-83 winter, with deepwater significant storm waves exceeding 10 ft. The duration of some of these storms was 5 or more days, resulting in a total of 32 days when the deepwater significant wave heights exceeded 10 ft. This is twice the average annual occurrence of deepwater waves in excess of 10 ft. Based on hindcasted and recorded deepwater wave data, the recurrence interval for significant deepwater waves exceeding 10 ft on 30 or more occasions during the year is once in 10 to 12 years. Assuming that the high tides experienced in the 1982-83 winter could occur during other winters, the recurrence interval for similar degradation of the park beach would also be once in 10 to 12 years. The recovery of the park beach was rapid and complete, indicating that the majority of the eroded beach sand was temporarily stored offshore during the winter and was rapidly transported back onshore during the summer and fall of 1983. No evidence exists that would imply that the beach would not rapidly recover in the future, following a similar winter. Few data exist on the long-term stability of Stinson Beach spit as a whole and the park beach in particular, but the fairly small longshore sand transport rates, the configuration of the shoreline, the fairly wide beach, flat beach slopes, and rapid recovery of the beach imply that the spit is fairly stable, with little or no long-term erosion trend.

A potential for damage to or destruction of the lifeguard tower, central comfort station, and north parking lot does exist for the No Action alternative. Damage or destruction could occur by wave action undermining the foundations or pavement as the wave-cut scarp recedes during extreme storm events. The wave-cut scarp, with a maximum height of about 8 ft, reached within 40 ft of the lifeguard tower and 20 ft of the central comfort station and north parking lot in 1982-83. The height of the scarp at Stinson Beach Park during this time was greatest where the foredune slopes were steepest (steeper than 6H on 1V), indicating that the breaking wave energy was dissipated at the toe of the steeper slopes causing excess scour. Where the slopes were flatter, the breaking wave energy was dissipated over a wider area as the wave ran up the slope, decreasing the amount of scour at the toe of the slope. The potentially highest wave-scour areas of the foredune under the No Action alternative are shown in Figure 10. The highest potential for scour of the foredune exists between the lifeguard tower and central comfort station where the foredune slopes are the steepest. Some potential for wave-induced scour also exists in front of the north parking lot and south overflow parking lot.

Wave overwash and flooding of the backshore with the No Action alternative can be expected in areas where the foredune elevations are lowest, (i.e., along the south overflow parking lot, between the north and central comfort stations and near the north park boundary). Areas subject to runup and backshore flooding are shown in Figure 11. The recurrence interval of wave overwash and flooding in these areas is once in 2 to 3 years (southwest wave, $H_b > 14$ ft, $T = 13$ sec). Wave overwash of the foredune could further erode the dunes and destroy vegetation on the foredune. Unlike the recovery of the beach following storms, natural foredune recovery and reestablishment of dune vegetation following wave overwash can be a long process, taking many years.

FOREDUNE DEVELOPMENT/ENHANCEMENT

Development and enhancement of the foredune along Stinson Beach Park will reduce the risk of wave overwash and flooding of the backshore at the northern and southern ends of the park. Two development/enhancement schemes have been

FIGURE 10. Wave Scour Areas, Stinson Beach Park

FIGURE 11. Wave Runup and Backshore Flooding, Stinson Beach Park

analyzed. The first scheme entails the construction of a foredune with a crest elevation of +20 ft, MLLW, seaward toe at +10 ft, MLLW, seaward side slopes of 6H on 1V, landward side slopes of 4H on 1V, and top width of 25 ft. The constructed foredune would be about 2700 ft in length, extending from the northern park boundary to the south end of the south overflow parking lot as shown in Figure 12. Typical cross-sections are shown in Figure 13. The area in front of the central comfort station and lifeguard tower would only require grading of the seaward slope because the foredune elevation is presently at +20 ft, MLLW. This scheme would require the placement of approximately 38,000 cy of sand in the configuration shown in Figure 12.

The second scheme, shown in Figures 14 and 15, entails construction of a foredune in those areas where the greatest potential risk of wave overwash and flooding presently exist, (i.e., at the northern and southern ends of the park). The crest and toe elevations, and the seaward and landward slopes would be the same as for the first scheme, but the top width would be only 10 ft. The constructed foredune would be 1900 ft in length, requiring about 28,000 cy of sand.

The difference in the two dune development/enhancement schemes is in the top width of the foredune. Scheme 1 with a top width of 25 ft would require about 36% more sand than scheme 2. With scheme 1 a small portion of the north parking area and about 40 ft of the the south overflow parking area would be covered with sand. With scheme 2 only about 25 ft of the south overflow parking area would be covered with sand. Some vegetation on the existing foredune would be covered by sand with both schemes.

An elevation of +10 ft, MLLW has been used as the seaward toe of the constructed foredune because it corresponds to the existing foredune toe elevation along most of the park beach. Placement of the toe any farther seaward would subject the toe and seaward slope of the foredune to increased wave-induced scour. Seaward side slopes of the constructed foredune have been set at 6H on 1V, corresponding to the side slopes of the existing foredune. Steepening the side slopes would increase the wave-induced scour of the foredune during high tide and storm wave activity; whereas, flattening the side slopes would increase the infringement of the foredune onto the backshore recreation

FIGURE 12. Foredune Development/Enhancement, Plan View, Scheme 1

Note: HET = Highest Estimated Tide
 MHHW = Mean Higher High Water
 MLLW = Mean Lower Low Water

FIGURE 13. Foredune Development/Enhancement, Typical Cross-Sections, Scheme 1

FIGURE 14. Foredune Development/Enhancement, Plan View, Scheme 2

FIGURE 15. Foredune Development/Enhancement, Typical Cross-Sections, Scheme 2

area. The landward side slopes of the constructed foredune have been set at 4H on 1V to roughly correspond to the landward side slopes of the existing foredune. The crest elevation of the constructed foredune has been set at +20 ft, MLLW. This elevation corresponds to the stable foredune elevations existing along the central portion of the Park. The 20-ft crest elevation allows for about 3 ft of freeboard for the design wave runup elevation of +16.6 ft, MLLW. The freeboard and top width of the constructed foredune provide additional protection against wave runup and scour during extreme storm waves and high tides by increasing the reserve sand volume in the foredune that would have to be eroded before wave overwash could occur. Scheme 1 (25-ft top width) would increase the reserve sand volume by about 36%.

The principal advantage of the foredune development/enhancement alternative is that storm wave overwash and flooding of the backshore would be eliminated. Very little if any additional protection to the lifeguard tower and central comfort station would be provided against wave-induced damage or destruction. These two structures are located directly on the foredune, requiring that the toe of the constructed foredune be extended seaward. The protrusion of the foredune onto the beach in front of these two structures would increase the wave-induced scour, rapidly eroding the seaward slope of the foredune until the toe has receded to its original position. The existing slopes fronting these structures (about 4H:1V) are oversteepened, causing excess scour and formation of a high wave-cut scarp during extreme storm waves. Regrading the slopes to 6H on 1V (as in scheme 1) would allow the wave energy to be dissipated over a larger area of the seaward slope of the foredune, possibly decreasing to some degree the amount of wave-induced scour.

Since dune development/enhancement is not a 'hard' protection against wave action, it is anticipated that the seaward slope of the constructed foredune will be periodically damaged by wave action, even though the foredune has been designed to minimize wave-induced erosion. Where the existing foredune slopes are about 6H on 1V, the wave-cut scarp during the winter of 1982-83 was limited to a depth of 1 to 2 ft. Similar damage of the constructed foredune can be expected at a frequency of 6 to 12 years, requiring that additional sand be placed on the seaward slope periodically to maintain it. During severe winter

storm conditions, such as those in 1982-83, an estimated 25 to 30% of the sand comprising the foredune could be lost. Using scheme 1 the estimated sand volume to maintain the foredune is about 11,500 cy every 6 years (1,900 cy/yr).

Two other considerations concerning the dune development/enhancement alternative are establishing vegetation and controlling pedestrian traffic. Any dune enhancement program should include establishment of vegetation. Although vegetation is not considered in this study, grass-type vegetation appears to be best suited for the foredune. At the northern end of the park, the existing foredune vegetation is sparsely-to-moderately dense. The vegetation is primarily dune grass extending down the seaward slope to the elevation of wave runup under normal conditions (about +12 ft, MLLW). Should this alternative be selected, establishing grass-type vegetation on the constructed foredune is advised. Pedestrian traffic and access to the beach would also have to be controlled by means of trails and/or boardwalks over the foredune.

RIPRAP REVETMENT

Construction of a riprap revetment along the shoreline of Stinson Beach Park would reduce the risk of wave overwash and flooding of the backshore, and prevent damage or destruction of existing structures. Two revetment schemes have been analyzed. In the first scheme the revetment would extend from the north park boundary to the south end of the south overflow parking lot, a distance of about 2700 ft as shown in Figure 16. The second scheme, shown in Figure 17, entails construction of 3 riprap revetments to protect only those structures in the most danger of being damaged or destroyed by wave action: the north parking lot, central comfort station, and lifeguard tower. About 750 linear feet of riprap revetment would be required for the scheme.

Typical cross-sections of the riprap revetment are shown in Figure 18. The top elevation of the revetment is +18 ft, MLLW giving about 0.4 ft of free-board for the design wave runup (17.6 ft). The seaward side slopes have been set at 2H on 1V. The foredune elevations along much of the park shoreline are not high enough to support a riprap revetment with a crest elevation of +18 ft, MLLW. In such cases, the revetment has been extended landward 10 ft to provide

FIGURE 16. Riprap Revetment, Extent of Protection, Scheme 1

FIGURE 17. Riprap Revetment, Extent of Protection, Scheme 2

Note: MLLW = Mean Lower Low Water
SWL = Still Water Level

0 5 10
Horizontal and Vertical Scale, ft

FIGURE 18. Riprap Revetment, Typical Cross-Sections

a base as shown in Section B-B' of Figure 14. The toe of the revetment should be situated to prevent wave-induced scour and undermining of the revetment toe. The presence of the clay hardpan along the Stinson Beach Park shoreline will provide a control, preventing wave-induced scour below this clay hardpan. Based on photographs of the park beach taken by park personnel during the 1982-83 winter storms, the clay hardpan appears to be at an elevation of +8 to +10 ft, MLLW. An elevation of +8 ft, MLLW will be used for estimating the toe elevation of the revetment.

Using methods described in the Shore Protection Manual (U.S. Army Corps of Engineers 1977) for determining the size of riprap, the required median rock diameter is 1.5 ft and the maximum rock diameter is 3.0 ft. Size gradation curves for the riprap and bedding material are shown in Figure 19. The thickness of the riprap cover is 3 ft to accommodate the largest rock, and the thickness of the bedding material is 2 ft. Table 14 summarizes the estimated volume of riprap, bedding material, sand excavation, and backfill for the two schemes.

FIGURE 19. Size Gradation of Riprap and Bedding Material

TABLE 14. Estimated Volume of Materials,
Riprap Revetment

Item	Scheme 1 Volume, cy	Scheme 2 Volume, cy
Riprap	6,300	1,380
Bedding material	4,350	920
Sand excavation	14,900	6,680
Minimum backfill	12,100	4,700
Annual sand replacement	1,000	400

The riprap revetment has been set back on the backshore far enough in both schemes so enough excavated sand exists to cover the entire revetment after construction. Portions of the revetment, however, would likely be uncovered during winter storm activity, requiring that sand be periodically placed on the revetment to cover it. The estimated frequency of exposure of portions of the revetment is once every 3 years. The volume of replacement sand is estimated to be about 25% of the original backfill, so for scheme 1 about 3000 cy of sand would have to be placed on the revetment every 3 years (1000 cy/yr), and 1200 cy of sand would have to be placed on the revetment every 3 years (400 cy/yr) for scheme 2.

The riprap revetment in scheme 1 extends along the entire park beach so that not only would it protect existing structures, but also prevent wave overwash and flooding of the backshore. Scheme 2 would protect only the north parking lot, central comfort station, and lifeguard tower from wave-induced damage or destruction. Wave overwash and flooding of the backshore at the south overflow parking lot and between the north and central comfort stations would not be prevented with the second scheme. Wave overwash and flooding could be expected to continue in these areas at a frequency of about every 2 to 3 years. Since the revetment would be constructed beneath the foredune, the entire foredune along Stinson Beach Park (2700 ft) would be disrupted with scheme 1, requiring the removal of most the dune grass vegetation and some willow stands. With scheme 2 only 750 ft of foredune would be disrupted and very little dune grass and willow stands would be destroyed.

ARTIFICIAL SEAWEED

A low-cost shoreline protection method has recently been developed by William Garrett (Hodman 1983) where artificial seaweed referred to as 'Seascape' is installed offshore to trap sand and partially dissipate wave energy before reaching the shoreline. 'Seascape' is constructed of DuPont Typar®, a spunbonded polypropylene fabric unaffected by salt water or fresh water. 'Seascape' is made in units consisting of a 5-ft-long, 10-in.-diameter tube, which acts as an anchor when filled with sand or rock. Twenty 4-ft-long fronds are attached to the tube, which float vertically when submerged. 'Seascape' units are normally installed in rows parallel to the shoreline (up to 5 rows) at a distance of 300 to 600 ft offshore.

'Seascape' has been installed in New Jersey, Delaware, California, South Carolina, and at several locations in North Carolina. The most publicized installation was at the Cape Hatteras Lighthouse in Buxton, North Carolina. Some controversy exists as to the effectiveness of 'Seascape'. Property owners who have had the 'Seascape' installed appear to be satisfied with the results, claiming as much as 2 to 4 ft of sand deposition over the 'Seascape.' The U.S. Army Corps of Engineers, however, are reluctant to make a definitive conclusion of effectiveness of 'Seascape', principally because insufficient monitoring of installations have taken place. The Corps of Engineers is monitoring the installation of 5500 units installed in the fall of 1982 at the Cape Hatteras Lighthouse.

In principle, the use of 'Seascape' appears to be sound. Seaweed and kelp beds are known to trap sand and dissipate wave energy. Some questions, however, need to be answered in terms of possible 'Seascape' installation at Stinson Beach Park. These questions include the following: the effect of wave climate on the integrity of 'Seascape' units, effect of tidal range, the effect of nearshore slopes, and offshore installation of 'Seascape' in an area where seasonal sand deposition storage already exists.

The wave climate along the north-central coast of California is more severe than in areas where 'Seascape' has already been installed. Deepwater

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wave heights in excess of 20 ft are common. Although Stinson Beach is protected to a large degree from deepwater waves, nearshore wave heights of 18 ft can occur. The integrity of 'Seascape' units to wave forces of this magnitude has not been tested. The extreme tidal range at Stinson Beach is about 10 ft. 'Seascape' would be most effective over a small tidal range where the units would initiate breaking of waves over the entire tidal range. Where the tidal range is large, waves could begin breaking seaward of the 'Seascape' units at low tide and shoreward of the units at high tide. At Stinson Beach, with a beach slope of 25H on 1V, an 18-ft wave will begin breaking at a water depth of about 22 ft. Using a design SWL (still water level) of 10.8 ft, the wave will begin breaking at a depth of -9.4 ft, MLLW at a distance of about 450 ft offshore. Assuming that the 'Seascape' units are 100% efficient in initiating breaking waves, the installation of the 4 ft-high units at a depth of 13.4 ft, MLLW would cause the waves to begin breaking about 100 ft farther seaward. This effectively reduces the wave runup on the foredune. At low tide, however, large waves would break seaward of the 'Seascape' units. Although wave runup on the foredunes would not occur at low tide, erosion of the beach face and damage to the 'Seascape' units may still occur.

The effectiveness of 'Seascape' as a shoreline protection measure at Stinson Beach Park cannot be predicted at this time. Before installation of 'Seascape' units, an experimental program would have to be implemented to evaluate their effectiveness. Such an experimental program would include 5 rows of 'Seascape' with a minimum length of 500 ft to be tested over a 3- to 5-year period. A monitoring program of protected and unprotected shoreline would also have to be implemented to evaluate the effectiveness of the 'Seascape' units.

ESTIMATED COSTS OF SHORELINE PROTECTION ALTERNATIVES

Cost estimates of the shoreline protection alternatives at Stinson Beach Park are explained in this section. The purpose for developing cost estimates is for comparison of alternatives and must be considered preliminary. Detailed cost estimates will have to be developed along with detailed design analysis should one of the shoreline protection alternatives be chosen for implementation. The unit costs for developing estimates have been obtained from similar projects in this region (Noble Coastal and Harbor Engineering (1983); William Spangle and Associates^(a); U.S. Army Corps of Engineers (1977); Engineering News-Record (1984)). Unit prices for 'Seascape' were obtained from Hodman (1983) and Lebovitz (1983). Unit prices for sand are based on dredging at Alcatraz/Presidio Shoal and barging the sand to the site. These unit prices could vary significantly, depending on the type of dredging plant and size of transporting barge. Unit costs are summarized in Table 15.

Where necessary, the estimated costs have been updated to 1984 price levels. Engineering and contingency costs have been included. A value of 6% of the construction cost is used as engineering costs and 15% of the construction and engineering costs is used as contingency costs. Annual maintenance costs for foredune sand replenishment are based on the estimated annual sand losses. Rock structures that are subjected to wave action normally require some degree of maintenance. A value of 2% of the first cost (i.e., construction, engineering, and contingencies) is used for the annual maintenance costs for rock structures. The annual maintenance costs for 'Seascape' units are unknown. Based on the Stinson Beach wave climate and fairly large tidal range, a value of 5% of the first cost is used as the annual maintenance costs for 'Seascape' units.

Estimated costs for dune development/enhancement are summarized in Table 16. The first cost for construction of a 2700-ft foredune (scheme 1), extending from the north park boundary to the south end of the south overflow

(a) Information on alternative mitigation measures for storm and flood hazards in a report to Stinson Beach Disaster Task Force, from William Spangle and Associates, Inc., December 16, 1983.

TABLE 15. Unit Costs of Shoreline Protection Alternatives

<u>Item</u>		<u>Unit Costs</u>
<u>Riprap and Bedding Material</u>		
Mining, sorting, loading (0.5 in. to 3.0 in. diameter)		\$20.00/cy
Mining, screening, loading (2 in. to 6 in. diameter)		15.00/cy
Hauling (25 mi)		6.25/cy
Dumping, placing/spreading		1.50/cy
<u>Earthwork</u>		
Excavation		3.00/cy
Backfill, contouring		2.00/cy
<u>Sand</u>		
Dredging		3.25/cy
Hauling		2.50/cy
Transferring, placing, contouring		4.50/cy
<u>'Seascape'</u>		50.00/ft

parking (see Fig. 12), is \$474,800. The annual maintenance cost for replacement of eroded sand for scheme 1 is \$19,500/yr. The first cost for construction of a 1900-ft foredune shown in Figure 13 (scheme 2) is \$349,800. The annual maintenance cost for replacement of eroded sand for scheme 2 is \$14,400. The in-place cost of sand for construction of the foredune for either scheme is \$10.25/cy. The costs are based on dredging sand at Alcatraz/Presidio Shoal; transporting sand by barge (scow) to Stinson Beach, where the sand would be dumped as near to shore as possible; transferring sand to the beach by dragline; and construction of the foredune with heavy equipment. The estimated costs for dune enhancement/development do not include costs for establishment of vegetation or providing pedestrian access to the beach.

Riprap revetment costs are summarized in Table 17. The estimated first costs for construction of a revetment from the north park boundary to the south

TABLE 16. Estimated Costs for Dune Development/Enhancement

Scheme 1: 2700-ft Foredune

Dredging; 38,000 cy of sand at \$3.25/cy	\$123,500
Transporting; 38,000 cy of sand at \$2.50/cy	\$95,000
Transferring, construction of foredune; 38,000 cy of \$4.50/cy	<u>\$171,000</u>
Construction cost	\$389,500
Engineering; 6% of construction cost	\$23,400
Contingencies; 15% of construction and engineering	<u>\$61,900</u>
First cost	\$474,800
Annual maintenance; 1900 cy of sand at \$10.25/cy	\$19,500

Scheme 2: 1900-ft Foredune

Dredging; 28,000 cy of sand at \$3.25/cy	\$91,000
Transporting; 28,000 cy of sand at \$2.50/cy	\$70,000
Transferring; construction of foredune; 38,000 cy at \$2.50/cy	<u>\$126,000</u>
Construction cost	\$287,000
Engineering, 6% of construction cost	\$17,200
Contingencies, 15% of construction and engineering	<u>\$45,600</u>
First cost	\$349,800
Annual maintenance; 1400 cy of sand at \$10.25/cy	\$14,400

end of the south overflow parking (scheme 1) is \$414,200, with an annual maintenance cost of \$18,300. The estimated first cost for construction of three 250-ft revetments for protection of the north parking lot, central comfort station, and lifeguard tower is \$108,100, with an annual maintenance cost of \$6,300.

Estimated costs for installation of 'Seascape' are summarized in Table 18. The estimated first cost for installation of 5 parallel rows of 'Seascape'

TABLE 17. Estimated Costs for Riprap Revetment

Scheme 1: 2700-ft Riprap Revetment

Excavation; 14,900 cy at \$3.00/cy	\$44,700
Mining, sorting, loading riprap; 6,300 cy at \$20.00/cy	\$126,000
Mining, sorting, loading bedding material; 4,350 cy at \$15.00/cy	\$62,300
Hauling (25 mi); 10,650 cy at \$6.25/cy	\$66,600
Dumping, placing/spreading; 10,650 cy at \$1.50/cy	\$16,000
Backfilling, contouring; 12,100 cy at \$2.00/cy	<u>\$24,200</u>

Construction cost \$339,800

Engineering; 6% of construction cost \$20,400

Contingencies; 15% of construction and engineering \$54,000

First cost \$414,200

Annual maintenance

Riprap; 2% of first cost \$8,300

Sand replacement; 1000 cy/yr at \$10.25/cy \$10,000

\$18,300

Scheme 2: Three 250-ft Riprap Revetments

Excavation; 6,680 cy at \$3.00/cy	\$20,000
Mining, sorting, loading riprap; 1,380 cy at \$20.00/cy	\$27,600
Mining, sorting, loading bedding material; 920 cy at \$15.00/cy	\$13,800
Hauling (25 mi); 2,300 cy at \$6.25/cy	14,400
Dumping, placing/spreading; 2,300 cy at \$1.50/cy	\$3,500
Backfilling, contouring; 4,700 cy at \$2.00/cy	<u>\$9,400</u>

Construction cost \$88,700

Engineering; 6% of construction cost \$5,300

Contingencies; 15% of construction and engineering \$14,100

First cost \$108,100

Annual maintenance

Riprap; 2% of first cost \$2,200

Sand replacement; 400 cy at \$10.25/cy \$4,100

\$6,300

TABLE 18. Estimated Costs for Installation of 'Seascape'

Installation of 2700 ft of 'Seascape'

Purchase, installation; 2700 ft at \$50.00/ft	\$135,000
Construction cost	\$135,000
Engineering; 6% of construction cost	\$8,100
Contingencies; 15% of construction and engineering	<u>\$21,500</u>
First cost	\$164,600
Annual maintenance; 5% of First cost	\$8,200

'Seascape' Experimental Program

Purchase, installation; 500 ft at \$50/ft	\$25,000
Monitoring, assessment; Monthly at \$5,000/mo x 3 yr	<u>\$180,000</u>
Construction cost	\$205,000
Engineering; 6% of construction cost	\$12,300
Contingencies; 15% of construction and engineering	<u>\$32,600</u>
First cost	\$249,900

between the north park boundary and south end of the south overflow parking lot (2700 ft) is \$164,600. The estimated cost for implementation of a 'Seascape' experimental program including installation of 500 ft of 'Seascape' units and monthly monitoring over 3 years is \$249,900.

SUMMARY OF COSTS

The least-cost alternative shore protection plan is construction of three 250-ft revetments at an estimated first cost of \$108,100. These three revetments would protect the north parking lot, central comfort station, and life-guard tower from wave-induced storm damage. Extension of the riprap revetment for protection of the entire park beach (2700 ft) would increase the costs to about \$414,000. The riprap revetment would eliminate wave overwash and flooding of the foredune and protect existing structures from wave-induced storm damage. The highest-cost shore protection alternative is dune development/enhancement along the entire park beach. The estimated first cost for this

plan, excluding costs for establishing vegetation and providing pedestrian access to the beach, is \$474,800. The costs for dune development/enhancement at only the northern and southern ends of the park where wave overwash and flooding are most likely to occur is \$349,800. The costs for dune development/enhancement are high in comparison to the other shore protection alternatives because of dredging, transporting, and transferring sand from Alcatraz/Presidio Shoal to Stinson Beach. The costs could be lowered substantially if a sand borrow area closer to Stinson Beach could be located. The dune development/enhancement alternative would lessen the risk of wave overwash and flooding at the north and south ends of the park, but would not appreciably lessen the risk of wave damage to existing structures (i.e., namely the north parking lot, central comfort station, and lifeguard tower). The costs for installation of 5 rows of 'Seascape' along the entire park beach would be only slightly higher than protecting the three endangered structures with riprap revetments. The estimated first cost for installation of 2700 ft of 'Seascape' is \$164,600. The effectiveness of 'Seascape' as a shore protection measure at Stinson Beach Park cannot be evaluated without implementation of an experimental program. The estimated cost for such a program is about \$250,000.

All the shoreline protection alternatives discussed in this report would require some degree of maintenance to remain effective. The maintenance costs for dune development/enhancement would be the greatest because foredune sand would have to be periodically replaced. The maintenance costs for a riprap revetment would also be fairly high if it is desirable to keep the riprap covered with sand.

CONCLUSIONS

Even though Stinson Beach Park is protected to a large extent from the direct attack of deepwater waves, severe winter storms, such as those occurring during the winter of 1982-1983, can cause considerable damage to the park beach and facilities. Without implementation of shoreline protection measures, the major storm damage in the future will be the temporary loss of the beach; damage to the foredune from wave scour; loss of foredune vegetation caused by storm wave overwash and scour; flooding of the backshore at the northern and southern ends of the park caused by wave overwash; and potential damage to or destruction of the seaward portion of the north parking lot, central comfort station, and lifeguard tower from wave attack.

Eleven major storms occurred from November 1982 through April 1983, with deepwater waves exceeding 12 ft. The duration of these storms was from 2 to 6 days each. Although the magnitude of the storm waves was not overly large, the number of storms accompanied by high tides was uncommon. The average annual occurrence for deepwater waves exceeding 10 ft is 16 d/yr. During the 1982-83 winter, deepwater waves exceeded 10 ft on 32 occasions.

Deepwater waves are altered substantially by wave refraction and shoaling before reaching the Stinson Beach shoreline. Wave refraction analysis indicates that waves breaking on the Stinson Beach Park shoreline are of lesser heights than the corresponding deepwater wave heights. Breaking waves are from 8 to 76% lower, depending on the deepwater direction and period. Based on deepwater storm wave heights and the wave refraction analysis, the design wave (breaking) at Stinson Beach Park is 18.6 ft from the south-southwest with a period of 11 sec. This corresponds to a deepwater wave height of 21.7 ft. The largest deepwater wave, occurring on February 21, 1956, was 25.9 ft from the southwest with a period of 12 sec. Because of greater refraction of southwest waves, the corresponding breaking wave height at Stinson Beach Park was only 13.8 ft. Wave runup on the foredune from the design wave at the highest estimated tide is 16.6 ft above MLLW. Since much of the foredune at the north and south ends of the park is at an elevation of about 15 ft above MLLW, wave

overwash and backshore flooding can occur. Wave overwash and backshore flooding will occur at the estimated highest tide in these areas when the breaking wave height exceeds about 14 ft.

Longshore sediment transport analysis of the Stinson Beach Park shoreline indicates that under prevailing conditions, the potential longshore sediment transport rate is about 310,000 cy/yr. The transport is directed almost exclusively to the northwest toward the tip of Stinson Beach spit. The longshore sediment transport rate at Stinson Beach Park is small in comparison to other California coastal areas such as Ocean Beach, where the annual transport rate has been estimated between 1 and 2 million cy/yr. The longshore sediment transport at Stinson Beach is due almost exclusively to winter storms from the south-southeast through southwest. The predominance of longshore sediment transport to the northwest indicates that shore protection measures implemented or planned elsewhere along Stinson Beach spit would have very little effect on the park beach.

Four shoreline protection alternatives for the Stinson Beach Park shoreline were evaluated. These included No Action, dune development/enhancement, riprap revetment construction, and installation of artificial seaweed ('Seascape'). The estimated costs for protection of the entire 2700 ft of shoreline ranged from \$164,000 for installation of 'Seascape' units to \$475,000 for dune development/enhancement. Construction of a riprap revetment along the entire park beach at an estimated cost of \$414,000 would provide the greatest degree of protection against wave overwash, flooding of the backshore, and damage to existing structures. The least-cost alternative entails riprap revetment protection of those structures in the greatest danger of being damaged or destroyed by wave action, (i.e., the north parking lot, central comfort station, and lifeguard tower). The estimated cost for this alternative is \$108,000. Wave damage to the foredune and flooding of the park would not be eliminated with this plan. Dune development/enhancement appears to be the most natural protection alternative. The costs, however, are very high because of the difficulty in obtaining high quality sand for placement on the foredune. The degree of protection provided by the dune development/enhancement plan would not be as great as the riprap revetment plan. Although wave overwash and

flooding of the backshore could be eliminated, the risk of damage or destruction of structures would be marginally lessened. If no shore protection actions are implemented at Stinson Beach Park, the park beach can be expected to be temporarily degraded of sand about every 10 to 12 years. The beach is likely to recover quickly, but damage to or destruction of the north parking lot, central comfort station, and lifeguard tower are greatly heightened while the beach is in a degraded condition. Wave overwash of the foredune and backshore flooding can be expected to occur at a recurrence interval of 2 to 3 years. Wave overwash will further damage the foredune and destroy foredune vegetation.

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APPENDIX

WAVE REFRACTION DIAGRAMS OF STINSON BEACH

A.1

A.2

A.3

A.4

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